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FLOOD MITIGATION PROGRAM FOR GATUN LAKE

FINAL REPORT

Volume 1 - Report

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PREFACE

This report was prepared by the consulting team of Moffatt & Nichol, Golder Associates Inc. and Christensen Associates Inc. for the Canal Capacity Projects Division of the Panama Canal Authority (ACP).

The authors are indebted to the ACP staff members who coordinated, oversaw and contributed to the complex and extensive work tasks leading to the preparation and submittal of this report. In particular, the consultants would like to thank the following ACP personnel for their active and enthusiastic participation and assistance in this fascinating project.

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1 INTRODUCTION

1.1 Background

As part of the preparation of an overall Master Plan for expansion of the Panama Canal, the Panama Canal Authority (ACP) is studying a number of options to upgrade the Canal infrastructure and existing infrastructure limitations, including the current capacity of Gatun Spillway.

When the current spillway was designed in the early 1900's, very little information was available concerning the hydrology and hydraulics of the Rio Chagres. The current spillway capacity was designed based on discharges calculated from historical flood elevations along the river and a maximum operational lake level of 25.90 m (85 ft). In subsequent years, the maximum operating level was raised to 26.67 m (87.5 ft) to satisfy additional water demands and accommodate increasing vessel drafts. Now, after 90 years of operation, a significant amount of hydrologic data has been collected for both the river and climatology of the region. Studies since 1945 have found that the current spillway capacity is insufficient to safely handle maximum floods. The international standards for dams such as Gatun state that the spillway must be designed to meet the Probable Maximum Flood (PMF). Therefore, additional spillway capacity is needed in order to meet this criteria and to protect the Lake and the viability of the Canal system itself.

Increasing the spillway capacity will also allow the operation of the Lake at a higher elevation without compromising safety of structures and navigation. It would also increase the water yield from the Canal Lake system. Therefore, as part of the evaluation of a new spillway, consideration should also be made to evaluate the optimal maximum operating lake level (MOLL) for Gatun Lake with both the existing and proposed spillways.

As part of the evaluation of the optimal MOLL, numerous issues will have to considered including present and future water demands for navigation as well as for municipal and industrial uses. In addition to these impacts, potential flooding impacts on structures and facilities bordering the Lake will also require careful consideration as many ACP and third party structures border the lake. The imposition of higher lake elevations on the existing spillway also requires careful consideration in light of these concerns.

Finally, concerns have also been raised regarding the stability of the Gatun Dam and Spillway during a seismic event. ACP is currently undertaking an evaluation of the seismic stability of the existing locks and all other key structures as part of the long term Master Plan, and clearly the stability of the existing spillway must be included within this analysis.

1.2 Study Objectives

The primary objective of this study is to evaluate the stability and capacity of the existing Gatun Spillway and recommend alternatives for additional spillway capacity to mitigate the potential for flooding that might occur in the event of severe precipitation conditions.

The secondary and related objective is to examine the structural, environmental, hydraulic and cost implications of an increase in the existing MOLL as a result of the additional spillway capacity gained, in order to reduce the necessity of draft restrictions in the Canal during periods of prolonged dry weather.

1.3 Scope of this Report

This feasibility study first examines the existing Gatun Lake spillway and dam and assesses its hydraulic and structural capability to meet a Probable Maximum Flood (PMF) condition and also to respond to seismic events. Feasibility level design for the additional spillway capacity required is also presented.

The second component of the Flood Mitigation program then evaluates the impact of a series of incremental increases in the MOLL on the existing structures surrounding and impacted by the waters of Lake Gatun as a result of the additional spillway capacity gained. These include the existing spillway, the locks at Pedro Miguel and Gatun, a number of saddle dams, ACP installations at Gamboa and a number of third party facilities and structures located around the lake and downstream of the existing spillway.

Environmental and socio economic impacts are also evaluated, with recommendations for mitigation or avoidance of impacts where relevant. Cost estimates are provided for the modification or reconstruction of those structures that might be impacted by the incremental increases, based on conceptual designs or information available in previous reports.

Finally an economic benefits evaluation is provided in order to assist with the selection of the optimum MOLL.

2 ANALYSIS OF EXISTING SPILLWAY

2.1 Gatun Spillway

2.1.1 Introduction

Gatun Spillway controls the level of Gatun Lake and protects Gatun Dam and other Canal facilities on Gatun Lake from inundation or overtopping during floods. Raising the maximum lake operating level of Gatun Lake would reduce the amount of surcharge available for flood routing, increase hydraulic load on the spillway and gates and impact spillway operation.

This section of the report discusses the condition of the spillway shown in Figure 2-2 and Figure 2-3, and also presents recommended improvements to address existing dam safety deficiencies. These recommendations also address the potential increase in Gatun Lake maximum operating lake level (MOLL) from El. 26.67 m. (87.5 ft.) to El. 27.43m (90 ft.) PLD (Precise Level Datum).

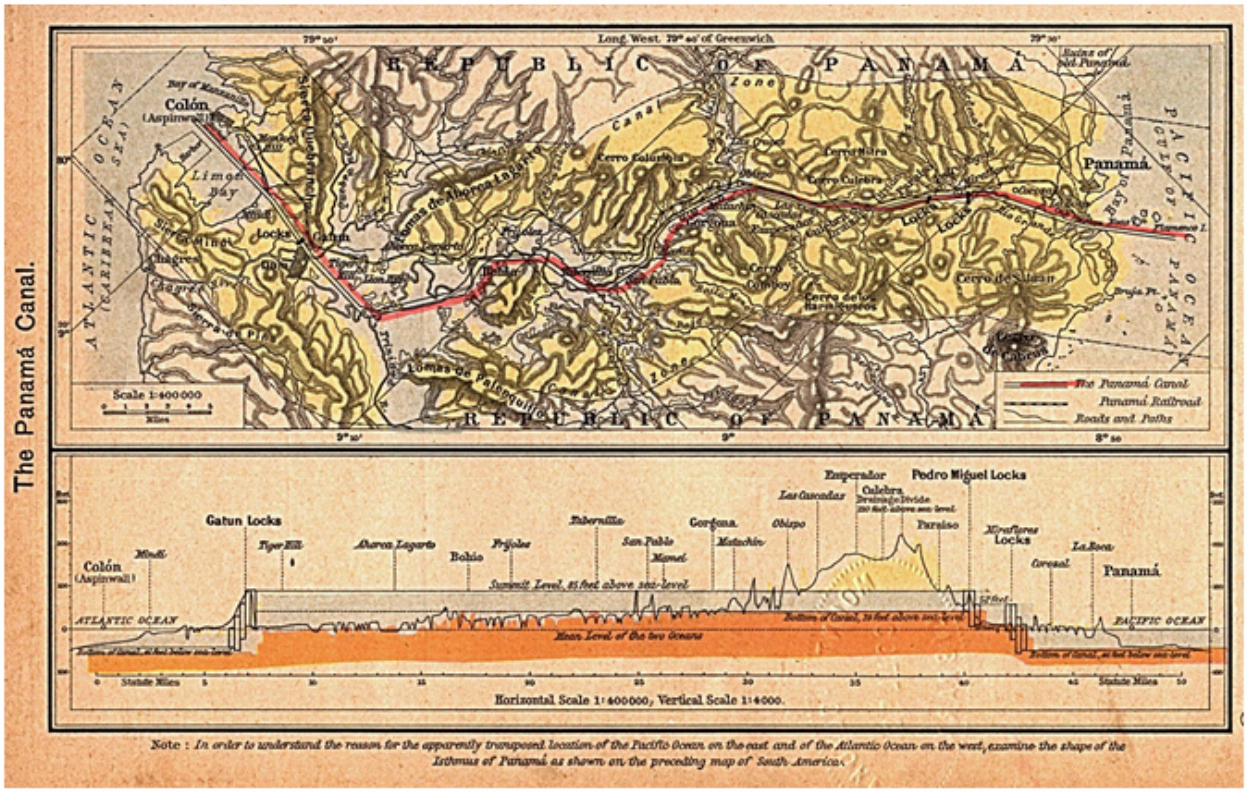
2.1.2 Description of Existing Facility

As shown on Figure 2-1, Gatun Lake provides a waterway across the central part of the Isthmus and stores water to operate locks that raise ships 25.91 m (85 ft.) from sea level to the lake surface.

At elevation 26.67 m (87.5 ft), Gatun Lake has a surface area of 436.5 km^2 (168.4 sq mi) and drains an area of 3,337 km^2 (1,289 sq mi) which includes watersheds of the Chagres, Trinidad and Gatun Rivers.

The spillway is the primary hydraulic outlet from Gatun Lake to the Rio Chagres. Its purpose is to control the lake level for water supply and navigation demands and to protect Gatun Dam, Gatun and Pedro Miguel Locks and other facilities around the lake shoreline from inundation during floods. The spillway was completed and placed in operation in October, 1913 and is now 91 years old.

Figure 2-1: Plan and Profile of Panama Canal

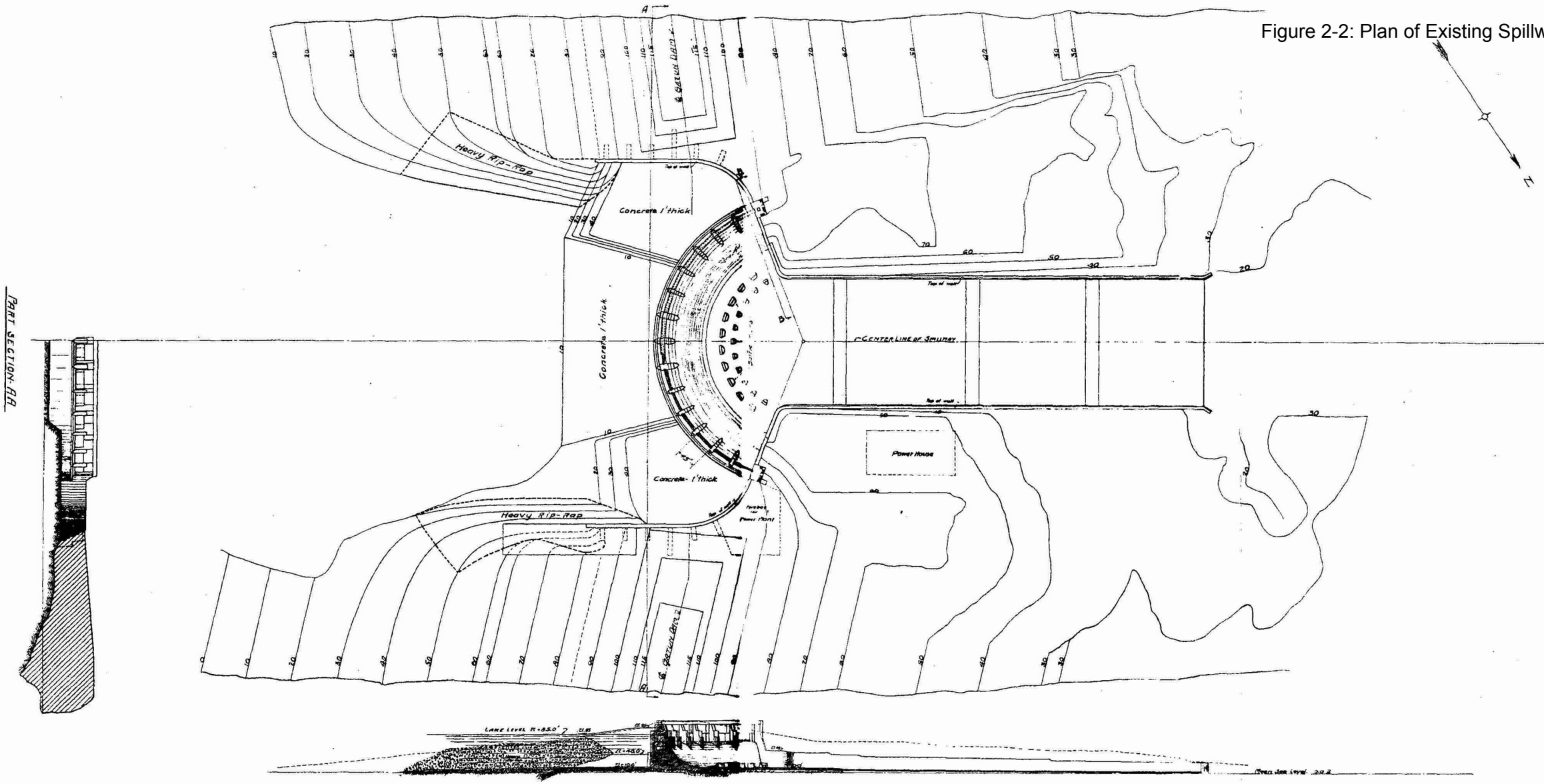


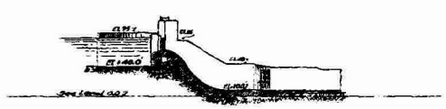
Source: University of Texas Library

Photographs of the spillway, gates and associated installations are shown in Figure 2-4 to Figure 2-12. Main features of the spillway from upstream to downstream are as follows:

- A short approach channel with a protective berm (SIP-9 project).
- A controlled spillway structure comprises a concrete gravity ogee section with fourteen vertical lift “Stoney” riveted steel spillway gates.
- A steep chute that converges flow at the toe of the ogee section where there are two rows of concrete baffle blocks on a flat concrete apron.
- A concrete lined discharge channel that discharges to the Rio Chagres.
- A roadway bridge that crosses the spillway exit channel.
- A hydroelectric plant on the right (east) side of the spillway that draws water from the approach channel and discharges into the exit channel.

Figure 2-2: Plan of Existing Spillway





Section-BB

ISTHMIAN CANAL COMMISSION
DEPARTMENT OF CONSTRUCTION AND ENGINEERING.
OFFICE OF THE ASSISTANT CHIEF ENGINEER.
CULEBRA, CANAL ZONE.

GATUN SPILLWAY
GENERAL PLAN

SCALE: 1" = 100 FEET. AUGUST 10, 1908.

MADE BY: E.K.T.
TRACED BY: N.M.
CHECKED BY: E.C.S.

SUBMITTED: Edward C. Sherman, Designing Engineer.

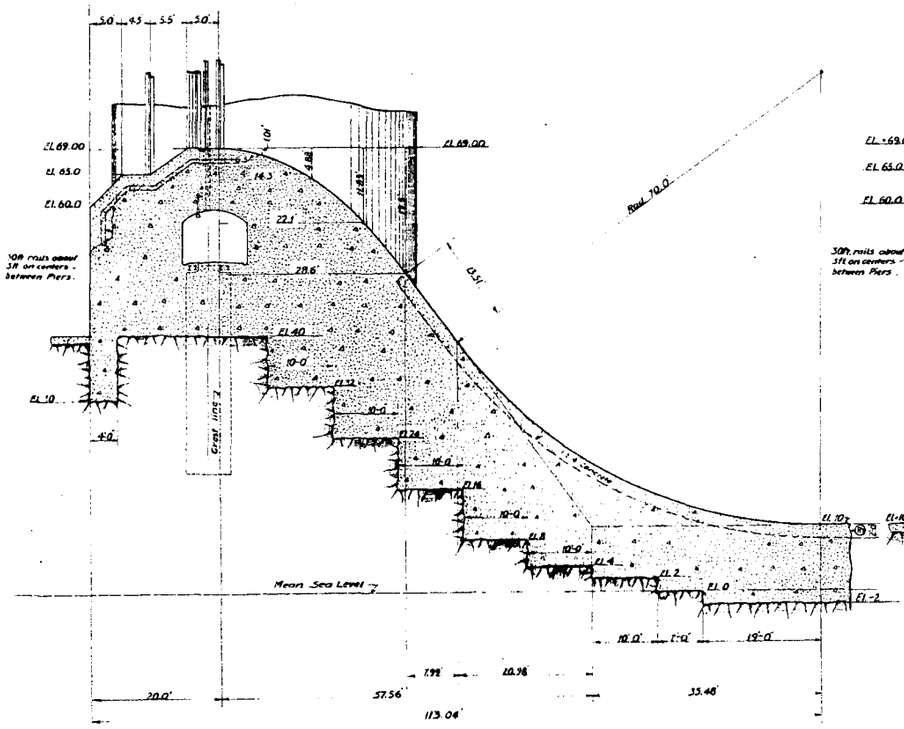
APPROVED: Geo. W. Goethals, Chairman and Chief Engineer.

RECOMMENDED: H. F. Stoddard, Assistant Chief Engineer.

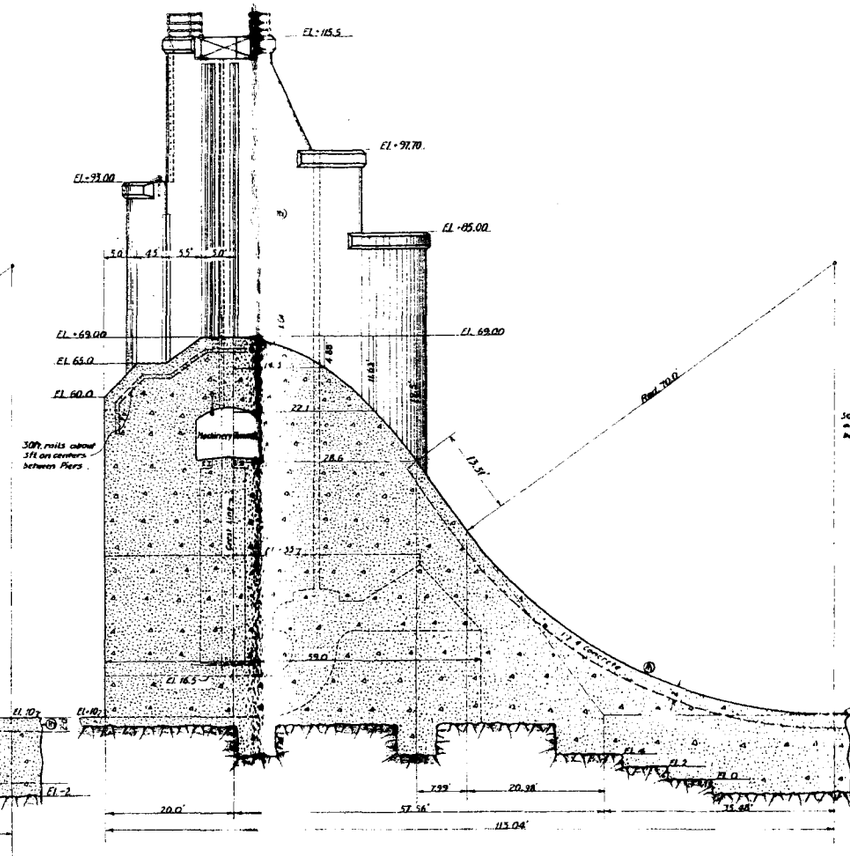
REVIEWED: H. F. Stoddard, Designing Engineer. E. E. Lee, Acting Electrical and Mechanical Engineer.

4004 G-4

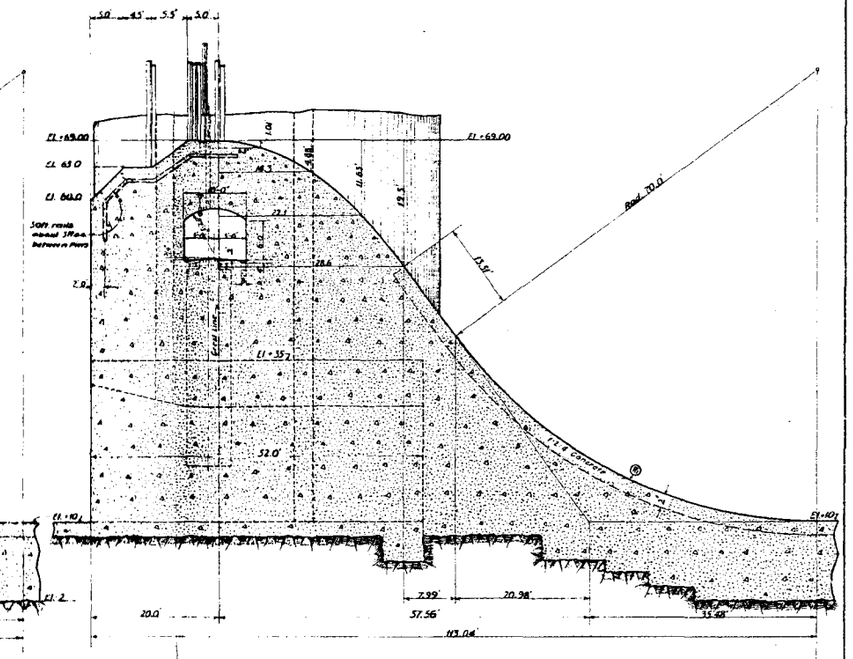
Figure 2-3: Cross Sections of Existing Spillway



SECTION C-C



SECTION B-B
Showing relation of Pier 'B' to Spillway Dam.



SECTION A-A
Showing relation of Pier 'A' to Spillway Dam.

REVISIONS

Mark	Revision	Made by	Checked by	Authority	Approved	Date
(B)	1:2:4 Concrete facing	W. J.	E.M.T.	E.C.S.		Feb 18 1910
(B)	Pier "B"	W. J.	E.M.T.	E.C.S.	J. L. L.	May 18 1910

NOTE: For location of Sections see drawing No. 4019. Equation of parabolic crest, $x^2 = 42y$. 1:2:4 Concrete facing 2ft. thick to extend to line of south face of baffles, 192.36ft. from center point.

GRAPHICAL SCALE
0 5 10 20 30 40 50 FEET

ISTHMIAN CANAL COMMISSION
DEPARTMENT OF CONSTRUCTION AND ENGINEERING.
OFFICE OF THE ASSISTANT CHIEF ENGINEER.
CULEBRA, CANAL ZONE.

GATUN SPILLWAY
SECTIONS OF SPILLWAY DAM

SCALE: 1" = 10 FEET. JANUARY 11, 1910.

MADE BY W.J.
TRACED BY
CHECKED BY E.C.S. & E.M.T.
SUBMITTED: Designing Engineer.
APPROVED: Chairman and Chief Engineer.
RECOMMENDED: Assistant Chief Engineer.

4020 G-20

REVIEWED: Edw. Schildhauer, Electrical and Mechanical Engineer; J. B. Cornish, Designing Engineer.

Figure 2-4: Gatun Spillway – General View

Figure 2-5: Approach Channel to Gatun Spillway

Figure 2-6: Upper Reach of Discharge Channel

Figure 2-7: Discharge Channel Outlet

Figure 2-8: Rio Chagres downstream of spillway

Figure 2-9: Gatun Spillway showing scour at end of discharge channel

Figure 2-10: Ogee Section, converging chute and baffle blocks

Figure 2-11: Access to top of Spillway and Stoney Gate at Bay 14

Figure 2-12: Gallery showing gate hoisting equipment

The approach channel to the spillway from the reservoir is approximately 60m (200 ft.) long. The sides of the channel are formed by concrete gravity wall sections that retain the crest and upstream embankment of Gatun Dam. The floor of the channel is a deep central section that was used for temporary river diversion during construction which is bordered by raised benches which are now submerged.

The gated control structure is 246.3 m. (808 ft.) long and 32.16 m. (105.5 ft.) high arranged along a segment of a circle in plan. The crest has a net hydraulic length of 192 m. (630 ft.) is set at El. 21.03 m (69.0 ft) and is divided by thirteen piers into fourteen 13.72m (42 ft) clear width bays.. Each bay is closed by a 13.71 m. (45 ft) wide by (5.79 m (19.0 ft) high Stoney gate which is counterweighted and raised by electrically driven power screw hoists

located in a gallery in the body of the ogee section. When closed the tops of the gates are at El. 26.79 m. (87.9 ft). When fully raised, the gate bottoms are at El. 27.88 m (91.5 ft).

Water flowing through the spillway accelerates in the approach channel reaching critical flow at the control structure and turbulent, high velocity flows continue throughout chute and exit channel until just downstream of the outlet. The concrete lined discharge channel is 293 m (960 ft.) long and 87m (285 ft.) wide and discharges to the Rio Chagres downstream of Gatun Dam. Water flowing through the spillway accelerates in the approach channel reaching critical flow at the control structure and large flows appear to maintain supercritical flow throughout chute and discharge channel until just downstream of the outlet where there is a hydraulic jump. The outlet of the discharge channel has scoured a deep stilling basin in the river bed which shows exposures of moderately erodible Gatun sandstone.

As seen in Figure 2-13 and Figure 2-14, the road bridge across the discharge channel is supported on five large concrete piers that are rectangular in cross section, which disturb flow generating large waves and amounts of spray that render the bridge unusable during large flood releases.

Figure 2-13: Discharge Channel Showing the Hydro Plant and the Bridge

Figure 2-14: Discharge channel at spillway bridge showing disturbance to flow

The spillway is protected from public access by fencing and the primary access by road is from the crest of Gatun Dam. Foot access to the crest and the gallery is at the east end of the structure via the hydroelectric powerhouse, which is also the primary point of control for the spillway gates.

2.1.3 Review of History of Spillway

Gatun Spillway was completed in October 1913 as part of the construction of the Panama Canal. Originally, various sea-level and elevated Canal concepts had been considered, leading to the decision to build an elevated Canal. A key element of the plan was to build Gatun and Miraflores lakes to form an elevated waterway for navigation across the central part of the Isthmus.

Gatun Lake was impounded by construction of an embankment dam across the Rio Chagres at Gatun. At that time, Gatun Lake was the largest artificial lake in the world. In addition to providing a water way for navigation, it also stores water needed to operate Gatun and Pedro Miguel Locks at each end of the lake.

The adoption of the elevated type of Canal, with the consequent impounding of large quantities of water by construction of dams, necessitated the construction of spillways at

Gatun and Miraflores Lakes to control water levels by regulating outflow. Under normal operations, Gatun Lake was to be filled during the rainy season to El 25.91 m (85 ft) PLD with storage above this level to be reserved for flood control.

With respect to the original design of Gatun Spillway, Sherman[1] reported in 1915.

"It was determined to provide a spillway capacity of 140,000 CFS, with the Lake at its normal level of 85 feet above sea level. The designs of the lock walls and gates were based on the assumption that the lake level would seldom be permitted to rise more than two feet above normal to elevation 87 feet."

On January 11, 1908, George Goethals sent a memorandum to the Secretary of War, recommending the operation of Gatun Lake between elevations 24.38 m (80 ft) and 26.51 m (87 ft), instead of the original plan, which was to operate the lake between elevations 25.00 m (82 ft) and 26.21 m (86 ft). Under the new operational range, the Canal capacity could be increased from 31 to 39 lockages on an average year.

Early operating experience with several floods that were close exceeding this criteria indicated that extreme peak flood flows to be handled by the spillway were probably much greater than anticipated. During these floods, spillway discharge capacity was augmented by using lock culverts at Gatun and Pedro Miguel locks. For example, during a storm that occurred from November 7 -9 1931, the maximum outflow from Gatun Lake was reported to be 5,320 cms (188,000 cfs) with a lake level at Gatun of El 26.46 m (86.8 ft.). Both sets of lock culverts were operated and some damage was reported.

In 1932, the lock culvert flow with Gatun Lake at El. 26.51 (87.0 ft.) was stated to be 765 cms (27,000 cfs) through the Gatun Lock culverts and 651 cms (23,000 cfs) through the Pedro Miguel lock culverts.

Records of inspection of the locks in the 1920s and 1930s report damage to the culverts including erosion of concrete 7.6 cm to 10 cm deep (3 to 4 inches) and failure of an entrance screen at Gatun Locks.

Addition of Madden Dam and Reservoir in 1936 increased the water storage and flood control capability of the system, however flood control plans still relied on use of the navigation locks.

"Although the Gatun Spillway is too small with reference to flood possibilities of the Gatun Lake watershed, and in comparison with the Madden Lake spillway, it can take care of 200,000 CFS approximately at a Gatun Lake elevation of 87.50 feet. Supplemented by lock culverts and Pedro Miguel emergency dam discharges, together with storage possibilities of

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[1] Sherman, E.C. The Design of Spillways of the Panama Canal, - Transactions of the International Engineering Congress, San Francisco, California September 20-25 1915

the Madden and Gatun Lakes, it appears that record breaking floods of the 1000 year recurrence interval can be successfully handled[2]*.*

Figure 2-15: General View, Madden Dam and Spillway

Ongoing advances in the science of hydrology, flood forecasting and further development of dam safety criteria led to the assessment that Gatun Flood Control Facilities should be capable of safely passing the Probable Maximum Flood (PMF). Hydrology and flood routing studies confirmed that the spillway alone was incapable of meeting this criteria. A detailed flood control plan that included use of lock culverts and emergency dams at locks was developed to bridge the gap.

As Canal traffic increased in both number and size of ships, a need for greater water supply was developed and the operating level of Lake Gatun was raised. The level of Gatun Lake is currently managed according to a rule curve that provides for a maximum operating level of Lake Gatun of 26.67 m (87.5 feet) during the dry season (November through January) reducing to El. 25.82 m (84.7 feet) during the middle of the wet season (i.e. April through August).

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[2] Matthew, G.E. Memorandum to Assistant Engineer of Maintenance, Revision – Memorandum of July 10, 1936, Probable Floods and Flood Control, Madden and Gatun Lakes, January 8, 1940.

The need for greater water supply also affected Madden Lake, which was originally planned in the 1930s to be filled to El 73.15 m (240 ft) during the wet season with the remaining storage reserved for flood control. Madden Lake is now operated at higher levels for water supply. Extensions were added to the drum gates so that in a fully raised position the crest was raised from El. 76.20 m (250 ft) to El. 76.81 m (252 ft).

In 1980, the U.S. Army Corps of Engineers (USACE)[3] stated that the safety of the Panama Canal System, and the validity of the PMF flood routing, required all available storage above El 77.42 m (254 ft) at Madden Lake and El. 26.82 m (88 ft) at Gatun Lake, and that none of this storage should be used at any time for conservation purposes.

A subsequent hydrologic study of floods in the range of 100 to 1000 year return period concluded:

"Taking into consideration that the maximum normal operating levels for Gatun and Madden should not exceed 87.50 feet for Gatun Lake and 255.00 feet for Madden Lake, I concluded that the Panama Canal Commission spillway system is not adequate to safely handle a 100 years flood. From a hydrological point of view, this is absolutely unacceptable. The spillway system should safely handle not only the 100, 500, or 1000-years flood but also the probable maximum flood (PMF)."[4]

The 1991 Panama Canal Flood Control Plan[5] established maximum permissible operating levels of Gatun and Madden Lakes at El. 26.67 m (87.5 ft) and El. 77.72 m (255 ft) respectively. These levels should only be exceeded temporarily during extreme floods. Madden Lake's flood surcharge storage lies between El. 77.72 m (255.0 ft) and the maximum flood level of El. 80.16 m (263.0 ft). Gatun Lake's flood surcharge storage lies between El. 26.67 m (87.5 ft) and El. 28.21 m (92.55 ft).

The present flood control plan also relies on a combination of early detection and response to large storms and an operating plan that includes discharging spill through existing lock facilities. It should also be noted that at elevation 28.21 m (92.55 ft), the upper levels of the Gatun and Pedro Miguel locks will be flooded with a top of lock wall elevation of 28.04 m (92.0 ft) and top of gate strut slot elevation of 26.94 m (88.4 ft).

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[3] USACE Mobile District, Development of PMF and Review of Flood Routing Procedures (Green Book) 1980 (Revised).

[4] Carlos A. Vargas On the Design of a New Spillway for Gatun Lake (Preliminary Study), July 1991.

[5] Panama Canal Commission Flood Control Plan (Red Book) 1991.

2.1.4 Flood Control Capacity

Inflow Design Flood Hydrograph

Under USACE dam safety criteria[6], Gatun Dam and Reservoir is classified as "High Hazard Potential" due to the potential for loss of life both on ships in transit and in the area inundated downstream of the dam in the event of a catastrophic dam failure.

In addition, potential high economic losses to Panama and world shipping in the event of an extended outage of the Canal also indicate that failure of Gatun Dam is also unacceptable from an economic standpoint and that the direct and indirect costs of a dam failure could greatly exceed the costs of upgrading the spillway capacity of the dam.

FEMA dam safety criteria[7] further indicate that " The PMF should be adopted as the Inflow Design Flood (IDF) in those situations where consequences attributable to dam failure for flood conditions less than the PMF are unacceptable". Given the hazard classification and the unacceptable economic consequences of dam failure the Probable Maximum Flood is selected as the inflow design flood.

The most recent Probable Maximum Flood hydrograph was developed by the US Army Corps of Engineers in 1979[8]. The flood hydrograph and reservoir routing is shown on Figure 2-17.

The computed PMF inflow hydrograph has a duration of 6 days and a peak inflow to Gatun Lake of 23,984 cms (847,000 cfs) occurring on the fourth day of the flood event. In this analysis, Madden Lake was assumed to be at El. 74.42 (254 ft.) and Gatun Lake was assumed to be at El. 26.82m (88 ft.) at the onset of the flood.

Spillway Capacity

The spillway discharge was originally calibrated during initial operation of the spillway. On December 30, 1913, and January 2-6, 1914, the discharge was measured a number of times with various gates in the fully open position. The average measured discharge per gate was determined to be 275.7 cms (9740 cfs) with the corresponding lake level at

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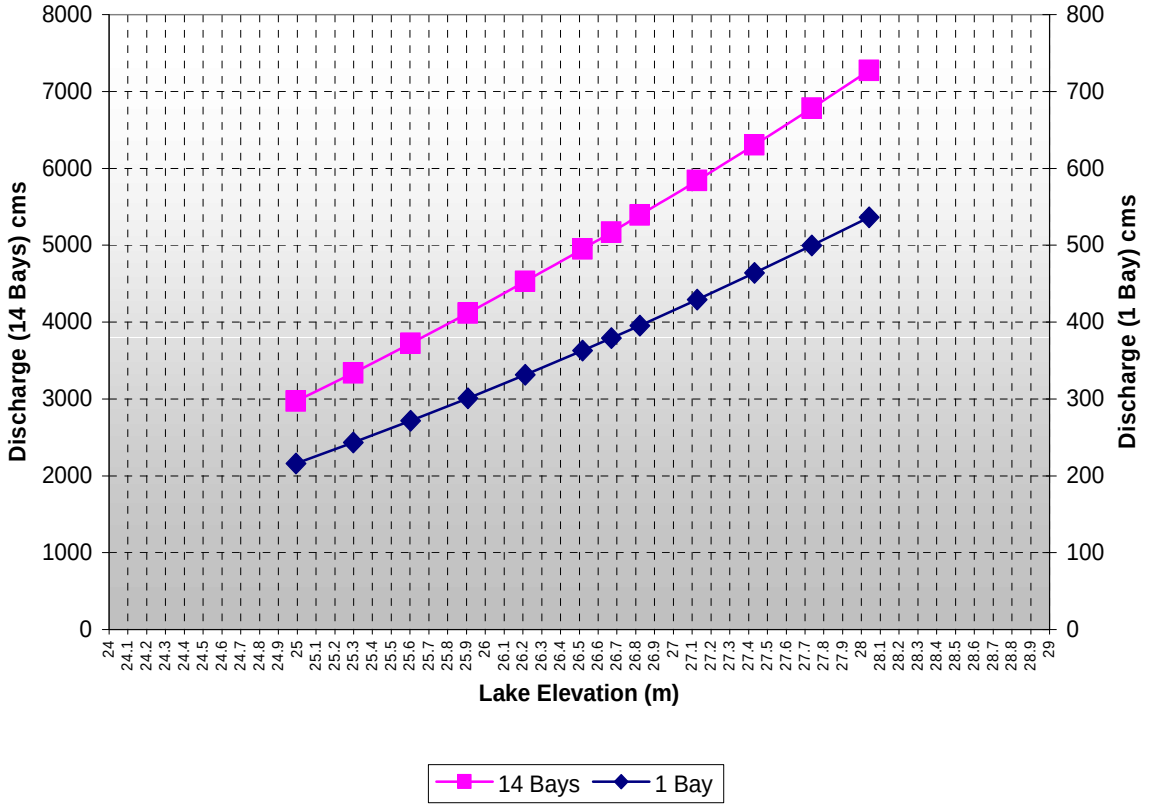
[6] USACE ER 1110-2-1155 Engineering and Design - Dam Safety Assurance Program - Appendix E. September 1997.

[7] Federal Emergency Management Agency – Federal Guidelines for Dam Safety: Selecting and Accomodating Inflow Design Floods for Dams - October 1998.

[8] USACE Mobile District, Development of PMF and Review of Flood Routing Procedures (Green Book) 1980 (Revised).

El.25.73 m (84.41 ft). A value of C=3.58 in the spillway discharge formula was derived from this discharge.

Figure 2-16: Gatun Spillway Rating Curve



The current spillway rating curve shown in Figure 2-16 which is derived from data in the 1991 Flood Control Plan, shows the estimated discharge for one bay and for all 14 bays of the spillway.

Under the current operating regime the estimated spillway capacity is 5,165.4 cms (182,416 cfs) at the current maximum operating level of El. 26.27 m (87.5 ft) PLD. The corresponding discharge at El. 27.43 (90 ft.) is 6,305 cms (222,525 cfs).

The estimated maximum flow under current PMF routing assumptions El. 28.20 m. (92.55 ft) PLD is extrapolated to be about 7500 cms (264,700 cfs).

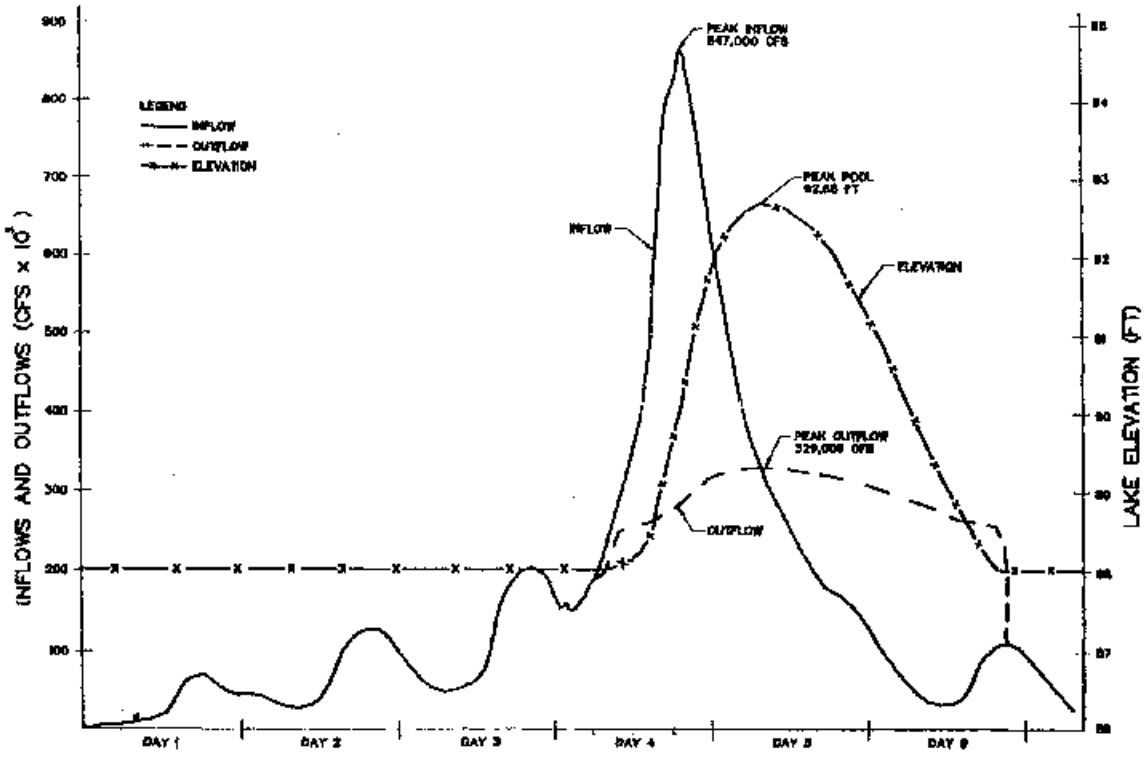
The maximum recorded discharge through the spillway is reported to be of the order of 5440 cms (192,000 cfs) which is about 73% of the maximum during the PMF routing.

The maximum total flood discharge capacity of the canal system including lock culverts, gates and Miraflores spillway is a complex issue with multiple constraints that is addressed in the ACP Flood Control Manual.

Reservoir Routing

In the USACE reservoir routing, the peak outflow from Gatun Lake through the existing spillway and lock culverts was 9316 cms (329,000 cfs) and the computed maximum pool elevation of Lake Gatun was 28.21 m. (92.55 ft.). Madden Lake had a peak inflow of 12,771 cms (451,000 cfs) and the peak outflow was 7227 cms (257,000 cfs). The corresponding computed peak pool elevation for Madden Lake was El. 80.16m (263.0 ft.)

Figure 2-17: Gatun Lake PMF Hydrograph and Reservoir Routing



2.1.5 Effect of Increase in MOLL on Flood Control Operations

Under present conditions the maximum operating level of Gatun Reservoir is El. 26.67 m (87.5 ft). The routing of the PMF through the reservoir indicates that under the present flood control plan which utilizes the existing spillway and lock culverts to control water surface elevations, the reservoir water surface will rise about 1.39 m (4.55 ft) from El. 26.82 m (88.0 ft) to El 28.21 m (92.55 ft).

An increase of MOLL will reduce the flood surcharge available for routing of the flood and either require an increase in available freeboard or increase in spillway capacity to pass the design flood. As discussed earlier in this section, existing concerns regarding the available flood discharge capacity of Gatun Spillway need to be addressed as part of ongoing dam safety responsibilities. Therefore, for planning purposes, an increase in MOLL may only

require an incremental increase in additional spillway capacity above that which is already required for dam safety under the existing flood control operating regime.

Flood control operations have been simulated under existing and proposed MOLL and the results are presented in Section 3.2.

2.1.6 Downstream Hydraulics

In addition to controlling the release of flood waters from Gatun Lake, Gatun Spillway also disippates the considerable amount of energy in the falling water. Since the dam is constructed of erodable material it is important that fast-flowing discharge from the spillway not cause erosion of the adjacent downstream embankments of the dam.

The spillway primarily relies on inducing turbulence to safely dissipate energy in the water as it flows from the reservoir to the river. There are three main areas where energy is dissipated:

- The converging section downstream of the Ogee.
- The spillway channel that conveys turbulent flow and creates some boundary friction losses. (The bridge piers also create some small losses)
- The channel outlet where a hydraulic jump forms at the abrupt increase in depth and decrease in velocity.

Given that the hydrologic loading conditions under which the spillway are now required to operate (the PMF) are considerably higher than those that were anticipated when the structure was designed it is recommended that the spillway be modeled over a full range of flow and tailwater elevations to ensure that it can safely perform as required.

Important issues that should be addressed are:

- Scour at the exit of the discharge channel.
- Potential overtopping of the walls at the lower end of the discharge channel under high tailwater conditions.
- Safety of the existing bridge across the discharge channel and the possibility of upgrading or relocating the bridge so that it is usable during floods.
- Performance of the spillway under increased tailwater elevations that are estimated to reach El. 8.1 m (26.6 ft) under present conditions and El. 14.02m (46 ft) with additional spillway capacity.
- Potential effects of asymmetric flow in the converging section due to gate failure or partial blockage of the spillway by either large debris or a stranded vessel.

2.1.7 Structural Stability

Raising the maximum lake operating level of Gatun Lake would increase hydrostatic load on the spillway structure. The static and seismic stability of the ogee section was analyzed using current dam safety criteria to determine whether the maximum operating level could be raised to El. 27.43 m. (90.0 ft.). The lateral stability of the spillway piers was also investigated.

Static and Seismic Stability Analysis of Ogee Control Section

A stability analysis of the existing spillway ogee control section was undertaken to verify that the existing spillway structure will meet stability criteria under a higher normal operating water surface elevation and the 1,000 and 5,000-year (Level I and Level II) return period earthquakes. These analyses are described in the stability analysis of the existing spillway, which are included within Appendices A and B to this report.

The major findings of the analyses are:

- The results of the current stability analysis compare well with those of a recent ACP analysis, indicating that the current stability analysis model and loading cases for the two analyses are substantially the same.
- The Gatun spillway can satisfy overall sliding and overturning stability criteria under the normal operating reservoir storage condition if the maximum operating level is increased up to El. 27.43 m (90.0 ft).
- The Gatun spillway can also meet these stability criteria for a PMF reservoir level of El. 28.05 m (92.0 ft).
- Under the Level I and Level II earthquake conditions the total horizontal driving force is substantially increased but since the minimum required shear friction factor of safety against sliding is much lower for the transient earthquake condition, the sliding stability criterion can still be met.

Lateral Stability of Spillway Piers

Thirteen piers and two abutment blocks separate the fourteen spillway bays. Three 4.67 m (15-ft.) wide piers are located at the center of the structure and are flanked by five thinner piers on each side. The thinner piers are 2.10 m (6.9 ft.) wide upstream of the gates and increase to a width of 2.59 m (8.5 ft.) downstream of the gates. Figure 2-18 shows the geometry of this pier. The thinner pier was used for the preliminary lateral stability investigation described below since it was identified as being the most critical for bending stresses at the base.

A possible deficiency in the lateral stability of the piers was previously identified by a stability evaluation performed by ACP. The ACP paper[6] describes a preliminary finite element analysis performed to investigate the lateral stability of the piers. The paper states:

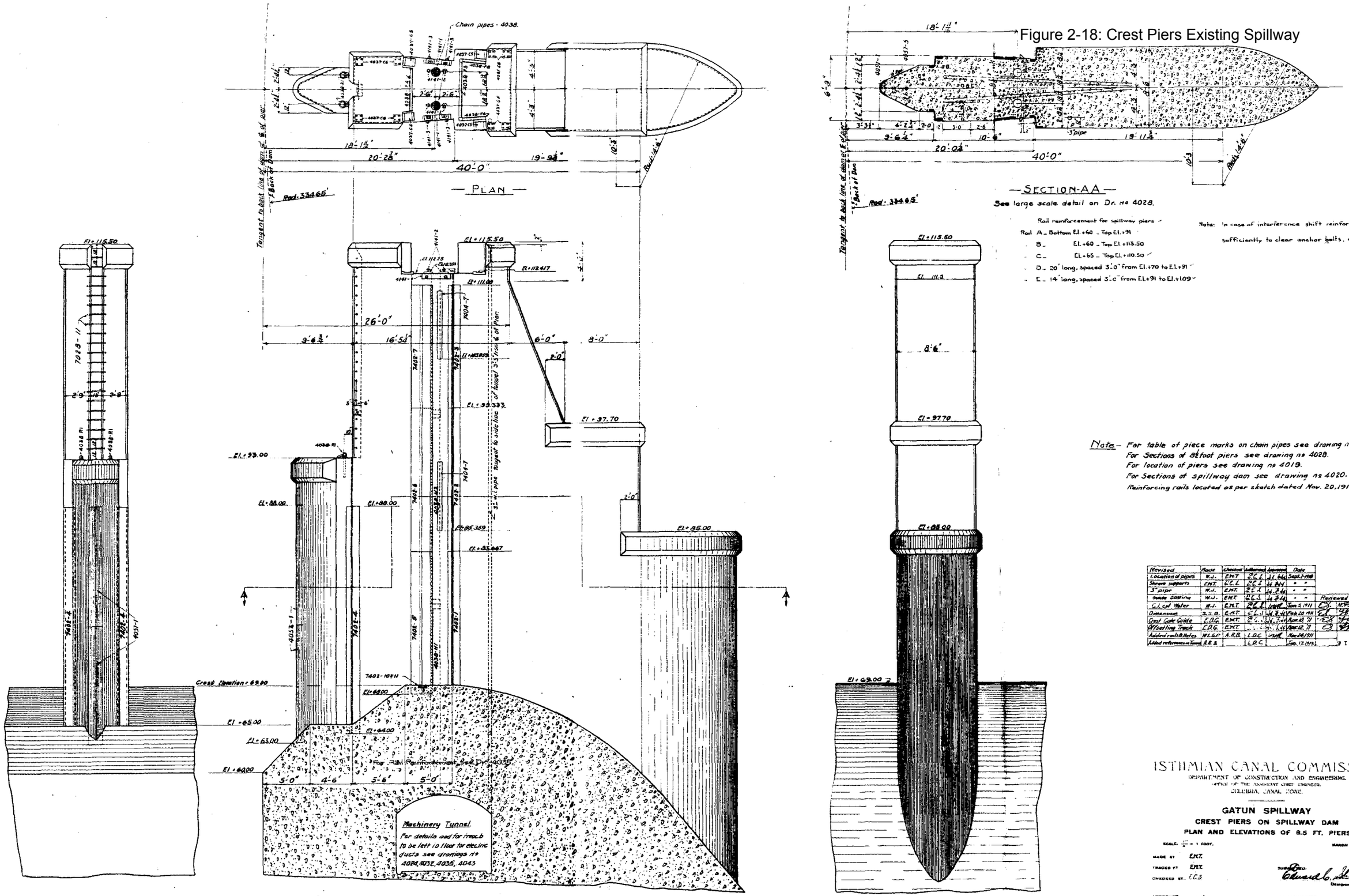
“Examining old construction drawings, it seems that no reinforcement between the spillway section and the piers was used; only massive concrete dumped from cars into chutes was used.” Based on this statement which was confirmed by ACP, the consultants have assumed that there is no steel reinforcement in the piers.

Earthquake forces in the lateral direction would induce high bending stresses in the piers with maximums at the ogee crest level. The bending stresses would probably be higher than could be resisted by the tensile strength of the concrete alone. Except in the design of very massive structures, tensile strength of concrete is usually neglected (assumed to be zero) and reinforcement must be provided. It seems unlikely that the piers would not contain some vertical reinforcement; however it is possible that such reinforcement would be deficient to resist earthquake loading. However lacking reinforcement information it should be assumed that none exists.

The project team performed a preliminary analysis of the lateral earthquake loading on the piers to determine approximate level of seismic forces and the requirements for stabilization. The analysis was a simplified pseudo-static analysis considering the horizontal inertial forces (pier mass times earthquake acceleration) acting at the center of gravity of the pier plus hydrodynamic pressure forces acting on the pier surface determined by the Westergaard approach. Potential inertial, impact or restraining forces of the gates on the piers during an earthquake were not considered in this preliminary analysis but would be considered in further more detailed structural analyses for rehabilitation design.

Shears and bending moments at the base of the pier were computed and reinforcing requirements were determined. Reinforcing requirements at the base of the piers were quite high due to high bending moments and it is likely that reinforcement (if any) embedded in the piers would be insufficient. Alternative remediation methods including post tensioned steel tendons and stiffening struts across the top of the spillway bays were considered. The recommended remediation concept is described in the next section of this report.

Figure 2-18: Crest Piers Existing Spillway



2.2 Recommended Improvements

The following paragraphs describe proposed improvements to the existing spillway to address structural deficiencies in the spillway piers and to improve the operation and safety of the spillway gates and their operating systems. It is considered that reinforcement of the spillway piers and replacement of the gates and hoists is a dam safety issue relative to Gatun Spillway at both its existing MOLL or with a raised MOLL. It is therefore recommended that the proposed improvements be implemented indepently of any decision to raise the MOLL.

Seismic Remediation of Spillway Piers

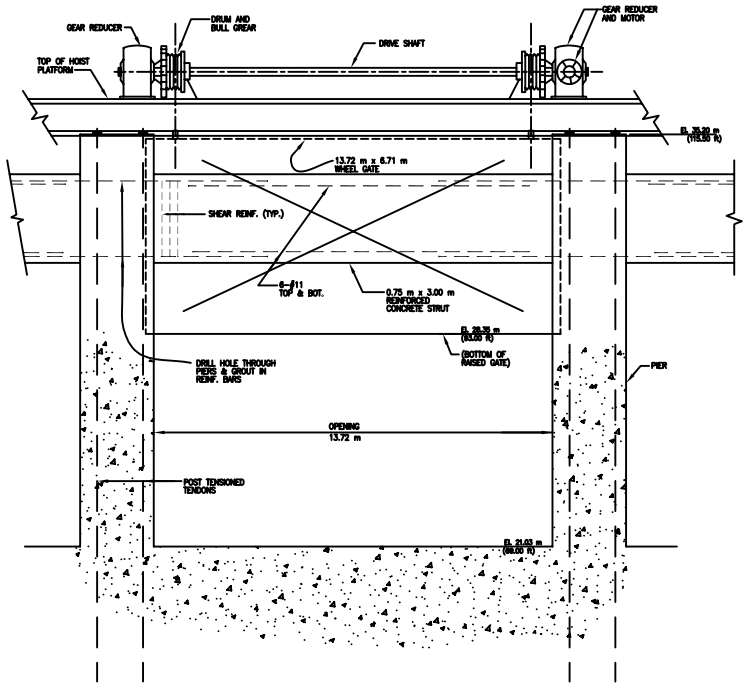
The remediation concept involves installing vertical steel reinforcement in the form post-tensioned tendons to alleviate tensile bending stresses in the concrete. The tendons would be installed in holes drilled from the top of the piers down into the massive ogee section concrete. Heavy struts or truss work would be constructed across the bays to restrain the top of the piers against lateral movement. Figure 2-19 shows the conceptual arrangement of the proposed remediation and the proposed rehabilitation of the spillway gates and operators.

The arrangement consists of the following:

- Two lines of vertical post-tensioned tendons installed in the piers to reduce tensile bending stresses due to lateral seismic loads.
- Horizontal reinforced concrete struts between the piers to convert them from free-standing cantilevers to moment frames. The struts are provided to reduce the high bending moments and reduce the post-tensioning force requirement. Steel trusses could be used to provide the same level of support as the reinforced concrete struts shown on the figure. The most favorable alternative would be determined during final design based on cost and constructability.

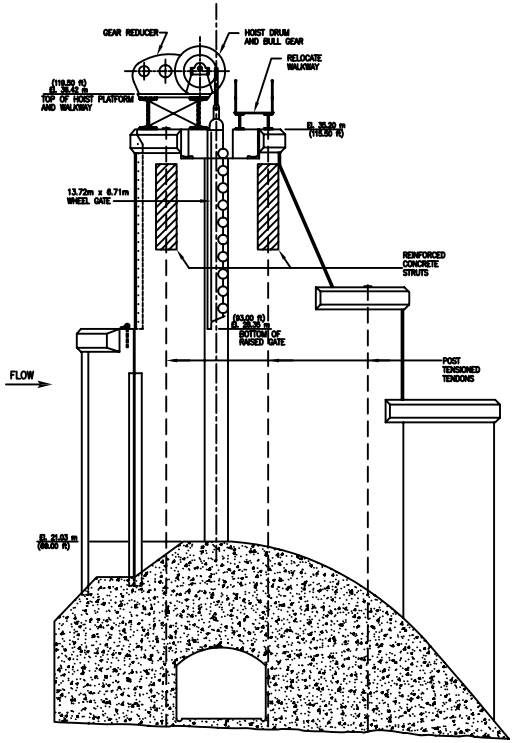
The final design of the seismic remediation of the piers should be carried out using a detailed finite element seismic analysis of the piers that is beyond the scope of work of this feasibility level study. However, before the design is undertaken, field investigations should also be conducted to determine the material properties of the piers for the analysis.

Figure 2-19: Gatun Spillway Proposed Modifications



UPSTREAM VIEW

SCALE: 1:25



SIDE ELEVATION

SCALE: 1:25

NO	DATE	REVISIONS	BY	CHK	APPD

CHRISTENSEN ASSOCIATES INC.
P.O. BOX 9087
SAN RAFAEL, CALIFORNIA 94912

DESIGNED | DRAWN | CHECKED | RECOMMENDED
SUBMITTED | APPROVED

PANAMA CANAL AUTHORITY

FEASIBILITY STUDY OF RAISING GATUN LAKE LEVEL
EXISTING GATUN SPILLWAY
PROPOSED MODIFICATIONS

DWG. NO.
SHEET OF
REV.

Rehabilitation of Gates and Operating System

Gatun Spillway is over 90 years old. Designed in the early 1900's before a century of advances in hydrology, hydraulic and structural engineering, the spillway has served its purpose well and with adequate maintenance it will provide many more years of service. However, the vertical lift Stoney Gates shown in Figure 2-20 have reached the end of their expected design life after more than 90 years of service. This particular design of spillway gates was superseded some time ago by more modern technology.

The chief concern regarding the existing gates is their future reliability. Failure to open during a large flood would further reduce the capacity of the spillway below that needed to control lake elevations and protect Canal facilities. Another concern is that failure of one or more of the gates could cause the uncontrolled release of an undesirable amount of water, possibly reducing the water surface level in Gatun Lake with unacceptable impacts on Canal operation.

The gates are vulnerable to minor damage to the spillway structure during an earthquake that could render several gates inoperable due to jamming of hoists or counterweights or flooding of the gallery that houses the electric hoists. The hoist system of the Stoney gates is also vulnerable to mechanical failure with age. This risk is exacerbated by the difficulty in inspecting and maintaining the hoist mechanism inside the piers.

It is therefore recommended that the existing gates and operating system should be replaced with vertical lift fixed-wheel gates and operators. The gates would be 13.72 m (45 ft) wide by 6.71 m (22 ft) high based on a crest elevation of 21.03 m (69.0 ft), MOLL of 27.43 m (90.0 ft) and 0.3 m (1 ft) freeboard. The elevation of the bottom of the gate when raised would be set 0.3 m (1 ft) above PMF water surface level.

The arrangement of the proposed new gates and operators is shown in Figure 2-19. The new fixed-wheel gate would be installed in the same slot as the old gates with some modifications in the slot to accommodate the new gate seals and the wheel track. The new gate operators would consist of a motor driven gear operated hoist attached to a steel frame located at the top of the piers. The existing walkway would be relocated to provide space for the hoisting system and its supporting structure.

The above gate and hoist configuration appears to be technically feasible and was therefore used for developing preliminary estimates of cost for rehabilitation of the gates. In the engineering design phase of the rehabilitation process a set of design criteria would be developed with ACP to define specific criteria and requirements for upgrading the gates. ACP has indicated that provision of a permanent gantry crane and consideration of hydraulic gate hoists may be required. These requirements and any others would be fully addressed in the design process.

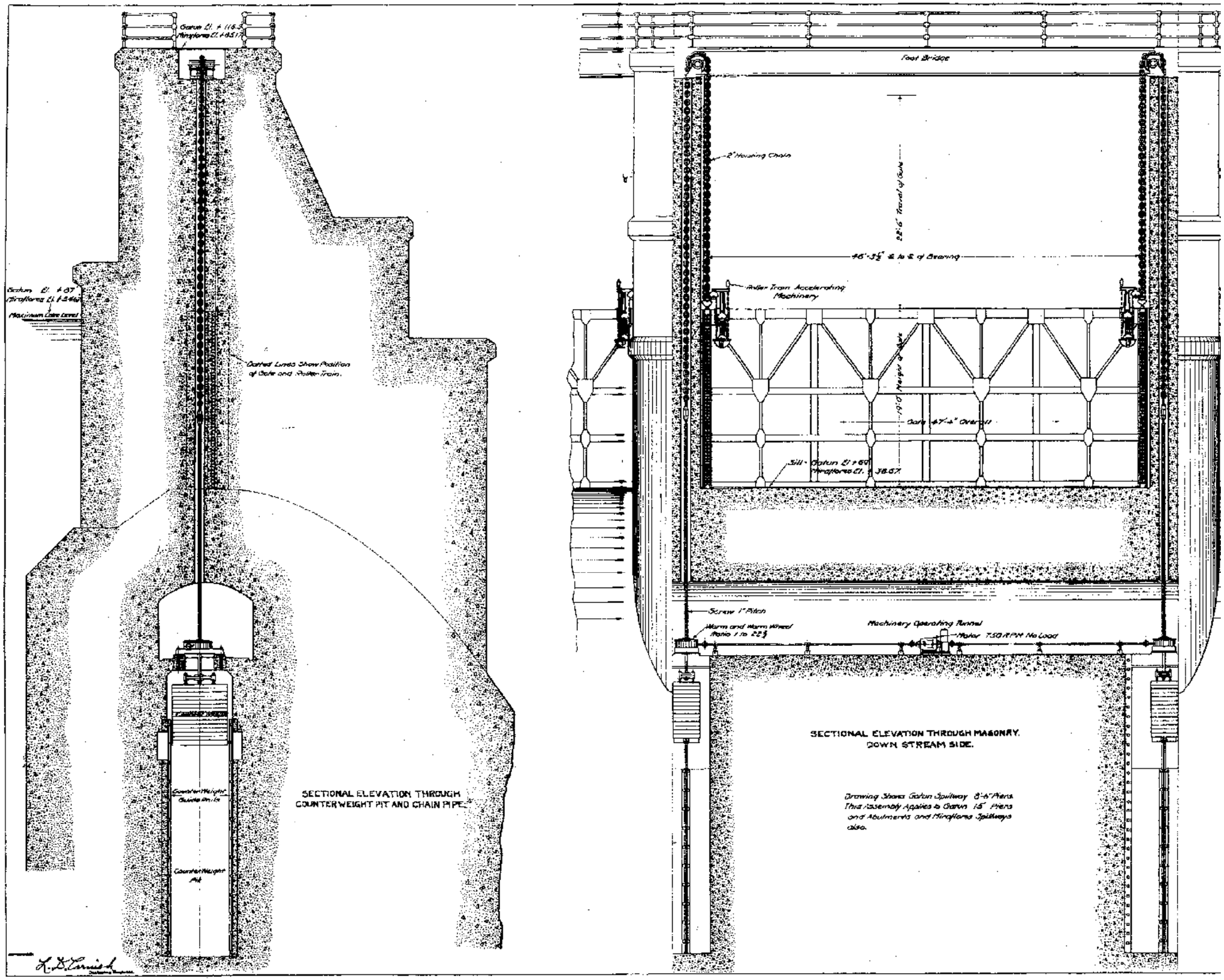


Figure 2-20
Existing Stoney Gates & Piers

Cost Estimates

Planning level estimates are presented for the recommended rehabilitation project for the existing spillway as shown in Figure 2-19. The base costs used in the estimates are applicable to the fourth quarter of 2004 and do not include any allowance for escalation or inflation to the estimated date of construction.

Mobilization shown in the estimate is an allowance for costs incurred in transporting construction equipment to and from the site and establishing and decommissioning construction equipment, temporary services and facilities at the project site.

Quantities used in the estimates have been calculated using dimensions taken from the preliminary drawings included in the study. Unit prices include contractor overhead and profit and are based on historical records for similar work. They include allowances for performing all the activities normally required to execute the work shown on the drawings including preparatory work, supply of miscellaneous ancillary materials and cleanup of the site upon completion.

Estimated costs for supply of gates and hoists are based on preliminary informal vendor quotations and include transportation to the project site. Installation cost estimates are also based on informal quotations from vendors with an allowance added for local labor and expense.

Cost estimates for this type of rehabilitation work on operating water resources facilities are subject to considerable variance depending on such factors as perception of risk, permitting constraints, owner requirements contractual terms, and the overall balance between supply and demand for construction services at the time the bids are solicited from contractors. Construction costs have also been subject to rapid inflation due to high increases in the prices of certain materials including fuel, steel and concrete. Recent fluctuations in foreign currency exchange will also impact the cost of imported equipment such as hydraulic gates.

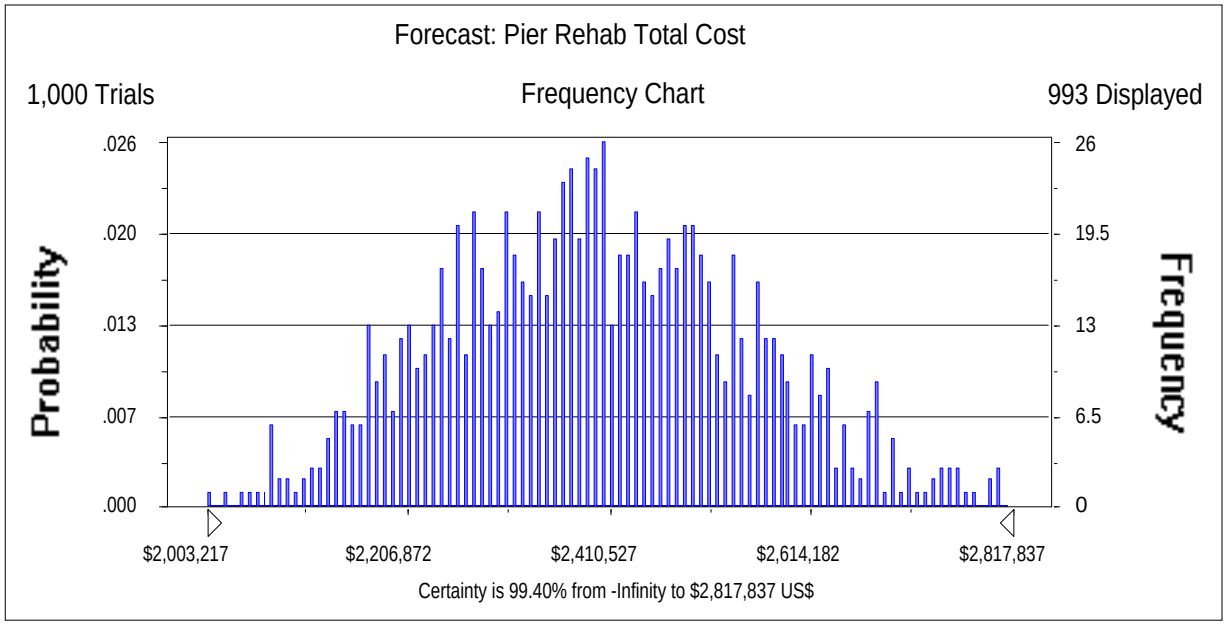
Cost Simulation Methodology

A Monte Carlo simulation of the cost estimate was run in order to examine the sensitivity of the final cost to the large number of variables in quantities and unit rates inherent in this feasibility type of assessment. The model runs 1,000 statistical trials of the estimated total cost of the project or project component, using variables to the quantities or unit rates input by the user. In this way a range of outcomes is forecasted, giving a probability that the project cost will exceed or be less than a specific amount. The model also ranks the variable items in order of their level of impact on the final project total, which then gives the user an indication of those cost items that can be prioritized during later phases of development of the project.

At this feasibility level of assessment, allowances for physical contingencies and engineering and construction management have been included. The allowances do not include other potential project related costs such as permitting, environmental baseline studies[9], legal, mitigation, force majeure, cost escalation, lost power generation, taxes, import duties, currency exchange rates, financing costs or other indirect costs or expenses. Supplementary estimates of these additional costs should be included in any budgeting or financial plans that require overall costs.

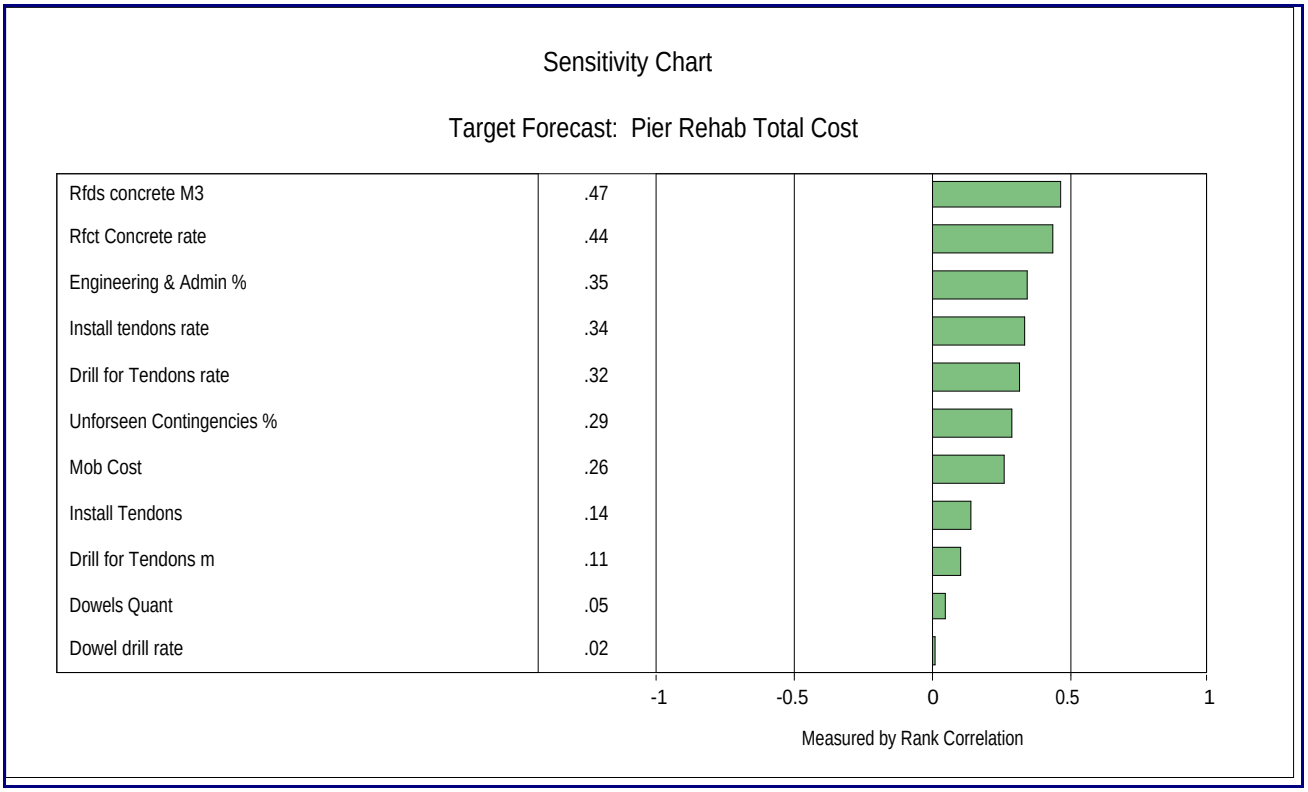
Using the cost simulation approach, the expected costs for the rehabilitation of the piers to the existing spillway is seen in Figure 2-21 to vary from $2.00 to 2.82 million with an 80% probability that the cost will not exceed $2.534 millon. Cost sensitivity rankings for the main variables are shown in Figure 2-22.

Figure 2-21: Cost Simulation - Pier Rehabilitation at Existing Spillway



[9] Environmental Impacts and recommended mitigation items are presented later in this report.

Figure 2-22: Cost Sensitivity Rankings for Pier Rehabilitation Project



The estimated cost of spillway pier rehabilitation shown on Table 2-1 reflects the 80% exceedence limit obtained from the cost simulation. This is considered to be an acceptable confidence level for budgeting purposes. The shaded cells indicated those items or rates where variables were applied and these uncertainties will narrow as more information is obtained on ground conditions and other cost parameters. This work requires a specialty contractor who has the expertise and equipment to accurately drill long holes in concrete and install post-tensioned steel tendons.

Table 2-1: Cost Estimate - Pier Rehabilitation at Existing Spillway (80% Confidence Values)

Description	Unit	Quantity	Unit Price	Amount
Pier Rehabilitation				
Mobilization	LS	1	$348,864.68	$348,865
Drill for dowels	M	486	$158.57	$77,067
Reinforced Concrete	CM	850	$697.73	$593,070
Drill for Tendons	M	1640	$279.09	$457,710
Install Tendons	M	1640	$266.41	$436,905
Subtotal				**$1,913,618**
Contingency			16%	$306,179
Subtotal				**$2,219,797**
Engineering, CM, Administration			14%	$314,471
Estimated Project Cost (80 Percentile value)				**$2,534,268**

Note: Quantities and unit rates represent the 80 Percentile values obtained from the Simulation model.

The 80 Percentile confidence level estimated cost of gate and hoist replacement is shown on Table 2-2 to be US$27.331 million. The range of costs and item sensitivity rankings are then presented in Figure 2-23 and Figure 2-24. This work requires design and fabrication of large gates and hoists and mobilization of a large barge and crane for gate installation. This work could also be subject to delays due to floods and operational constraints. Should ACP decide to proceed with rehabilitation of the spillway to extend its service life then additional investigations should be carried out to confirm the absence of steel reinforcement prior to preparation of detailed design drawings and contract documents.

Table 2-2: Cost Estimate for Spillway Gate Rehabilitation (80% Confidence Values)

Description	Unit	Quantity	Unit Price	Amount (US$)
Mobilization	LS	1	939,681	$939,681
Remove Gates & Hoists	EA	14	22,780	$318,922
Chip Guide Slots	CM	210	481	$100,909
Stage 2 Concrete	CM	210	587	$123,333
Supply Gates & Hoists	EA	14	803,356	$11,246,980
Install Gate Guides	SET	14	33,636	$470,908
Install Gates & Hoists	EA	14	280,302	$3,924,234
Hoist Platform	TN	200	3,203	$640,691
Walkway	Bay	14	10,678	$149,495
Electrical	LS	1	704,760	$704,760
Misc. Mechanical	LS	1	560,605	$560,605
Subtotal				**$19,180,517**
Contingency			20%	$3,884,055
Subtotal				**$23,064,572**
Engineering, CM, Administration			19%	$4,266,946
Estimated Project Cost (80 Percentile Value)				**$27,331,518**

Figure 2-23: Cost Simulation - Gates Replacement at Existing Spillway

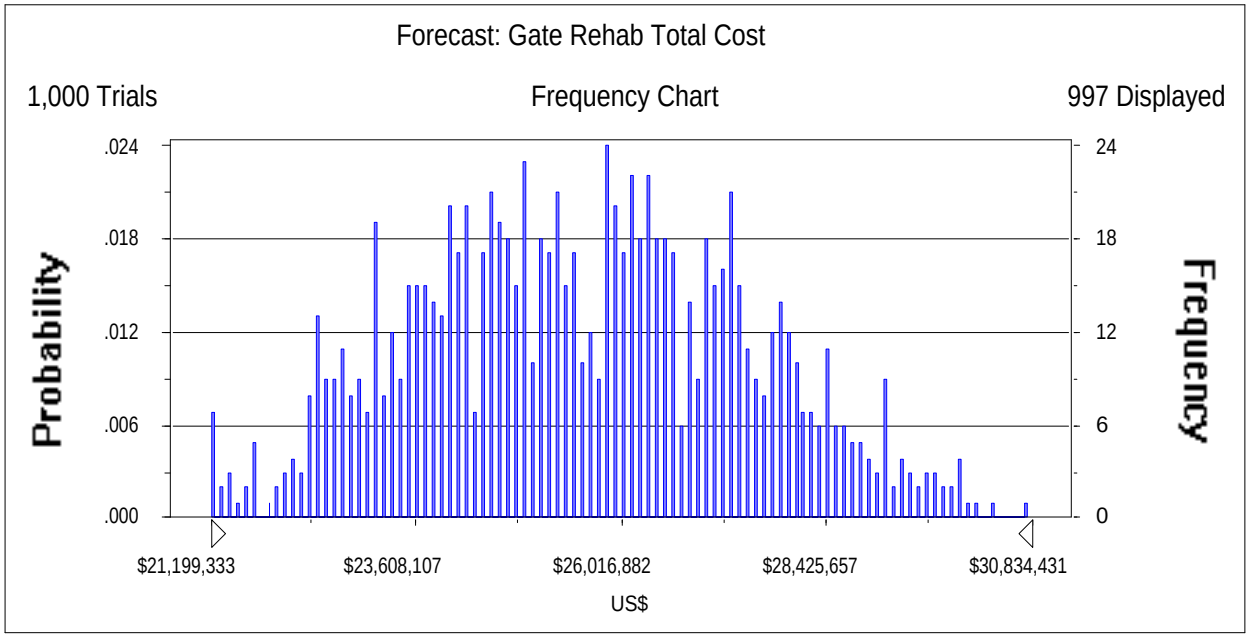
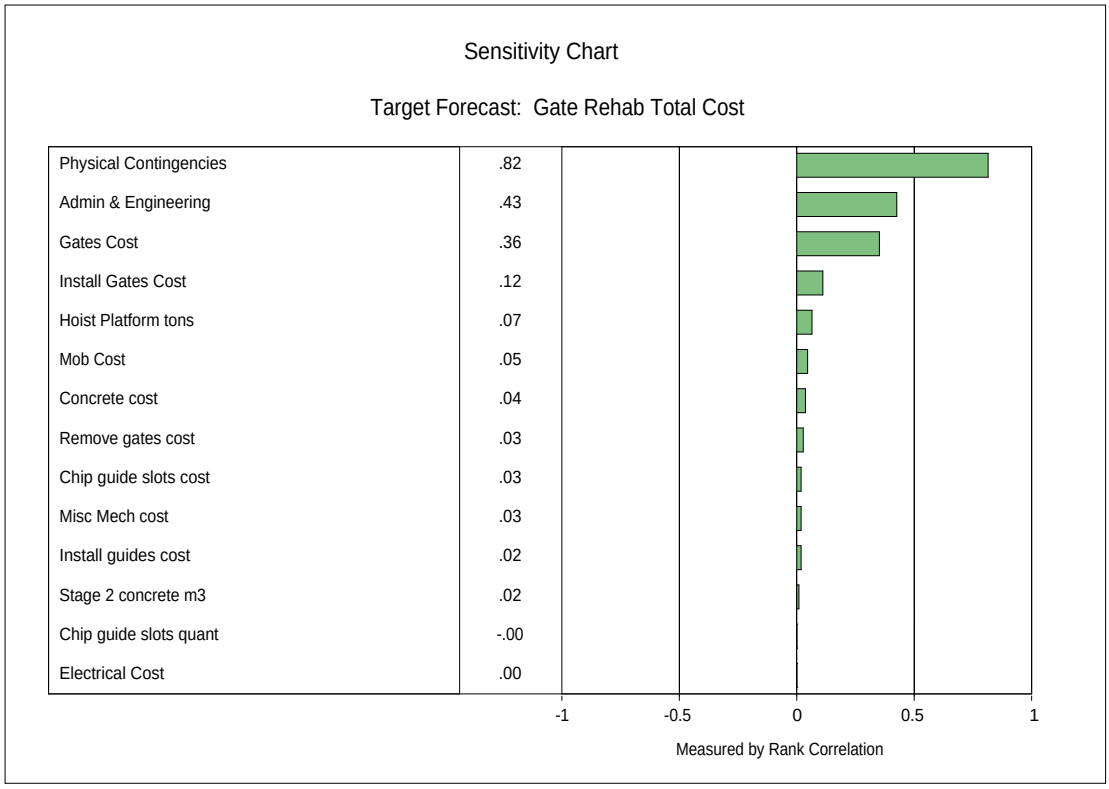


Figure 2-24: Cost Sensitivity Rankings for Gate Rehabilitation Project



Implementation Schedule

Sequencing of Construction

Gatun spillway is critically important to the safe operation of the Panama Canal. If remediation work is carried out prior to addition of a new spillway then careful consideration must be given to the challenge of maintaining the existing spillway in operation during remediation work.

Strengthening of the existing piers could be carried out without restricting spillway capacity as long as the locations of the tendons are clear of existing embedded parts.

Replacing the spillway gates would reduce spillway capacity while bulkhead gates are installed to facilitate replacement of the gates and embedded parts. If the gates are replaced before construction of a new spillway then the work will need be carefully staged to maintain adequate spillway capacity. A detailed construction plan with emergency response plans would need to be developed to address this issue and be incorporated in ACP's overall flood control plan.

Preliminary Construction Plan

Work will commence with mobilization of plant and equipment. In addition to the crews performing the work it is expected that tendon installation will require one barge mounted

crane and gate replacement will require at least two barge mounted cranes with a supporting fleet of transportation barges and boats.

Access from downstream would be limited by operation of the spillway and the hydroelectric powerhouse which discharges to the discharge channel.

A total of eight tendons will be installed in each pier. The work may proceed independently of gate replacement and could be carried out without reducing the available capacity of the spillway.

Construction of reinforced concrete struts between the piers will require erection of a temporary truss to support formwork and concrete and provide a work platform for workers.

The sequence for rehabilitation of each gate will be:

- Install and seal caisson
- Remove and dispose of existing gate
- Remove and dispose of existing operating mechanism
- Remove existing gate guides
- Chip recess for new guides and second stage concrete (ea. 1m x 0.5m x 14m)
- Install and embed new guides (14 m approx)
- Install new gate.
- Install hoist platform and walkway
- Install gate hoist
- Complete mechanical /electrical installations
- Test gate
- Remove caisson

The length of time required for gate rehabilitation depends on how many bays can be taken out of service at one time. A key concern is that once the bulkhead gate is installed and the gate removed there would be no safe way to open and close that bay of the spillway. The consequence is a reduction of spillway capacity. Another concern is loss of stored water in the event that there is a failure of the bulkhead gate while the permanent gate is removed.

Assuming that this work will start before construction of the new spillway, work would proceed on one gate bay at a time. Based on an estimated duration of three months work for each gate, including installation of new embedded parts and seals the project would take 42 months to complete unless the work can be accelerated by devising a scheme to work on two or more gates simultaneously. Since it expected that this work might be undertaken by the ACP Industrial Division, time for completion of the project may not have severe financial

implications. In the event that the work was undertaken by a private contractor, working on one gate at one time may be cost prohibitive due to the need to maintain high cost plant at the site for an extended period of time.

2.3 Summary & Recommendations

Primary findings and recommendations arising from the analyses of the existing spillway are:

- Although Gatun Spillway has successfully operated for 91 years, it now requires upgrading to meet modern dam safety requirements.[10,11,12,13] Modifications needed to accomodate a higher MOLL can be incorporated into these upgrades.
- Gatun Dam has been determined to be a "High Hazard" structure. Due to this designation and the potential economic consequences of a protracted outage of the canal due to dam failure, the recommended design flood is the Probable Maximum Flood.
- The hydraulic capacity of the spillway alone is insufficient to meet flood control requirements for Gatun Dam and other canal structures in the event of the Probable Maximum Flood. The present flood control plan envisages supplemental discharge of water through lock structures. Previous releases of spill through lock culverts have caused damage to the lock structures. Additional spillway capacity is required and could be best acquired by building an additional spillway.
- The inflow design flood and the ranges of headwater and tailwater elevations under which the spillway is now required to operate are considerably different than those that were anticipated when the structure was designed. It is recommended that the spillway be modeled over the full range of hydraulic conditions to ensure that it can safely perform as required.
- The 1,000 and 5,000-year (Level I and Level II) return period earthquakes were selected for structural stability analyses. Stability analyses of the existing spillway ogee section under hydraulic and seismic loading indicate that it can satisfy overall

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[10] Federal Emergency Management Agency – Federal Guidelines for Dam Safety: Selecting and Accomodating Inflow Design Floods for Dams - October 1998

[11] ICOLD – Selection of Design Flood - Bulletin 82, 1992

[12] ICOLD - Seismic Design and Evaluation of Structured Appurtenant to Dams - Bulletin 123, 2002

[13] USSD (Formerly USCOLD) – Guidelines for Earthquake Design and Evaluation of Structures Appurtenant to Dams, - May 1995

sliding and overturning stability criteria under a maximum operating lake level of El. 27.44 m (90.0 ft) and a PMF reservoir level of El. 28.05 m (92.0 ft.).

- Previous structural analyses of the spillway piers by ACP have identified a lateral stability deficiency due to a lack of vertical steel reinforcement. Preliminary investigations indicate that this deficiency could be remedied by retrofitting vertical tendons in the piers and horizontal struts between the tops of the piers.
- The spillway gates have exceeded their expected service life and are due for replacement. It is recommended that the existing vertical lift Stoney gates be replaced with vertical lift fixed wheel gates with overhead electric hoists.
- For planning purposes, the estimated construction cost of rehabilitation of the spillway piers is $2,534,000. The estimated construction cost of replacement of the spillway gates and hoists is $27,331,000.
- Gatun spillway is critical to the continued and safe operation of the Panama Canal. If remediation work is carried out prior to addition of a new spillway then consideration must be given to maintaining adequate spillway capacity at the existing spillway during remediation work.
- Strengthening of the existing piers could be carried out without restricting spillway capacity.

3 NEW SPILLWAY

3.1 Introduction

Gatun Spillway controls the level of Gatun Lake to protect Gatun Dam, lock structures and other facilities on Gatun Lake from inundation or overtopping during floods. Under present conditions, additional spillway capacity is required to safely discharge the Inflow Design Flood (PMF) without overtopping canal structures

The present spillway was designed at the turn of the century based on a design flood discharge of 3,964 cms (140,000 cfs) estimated from observations (high water marks) of historic floods in the Rio Chagres near Bohio[14]. Under the current inflow design flood and flood routing carried out tby the US Army Corps of Engineers and shown in ACP's current flood control plan the estimated peak PMF inflow to Gatun Lake (accounting for routing through Madden Lake) is 23,994 cms (847,000 cfs). The peak outflow is reduced to 9264 cms (327,000 cfs) by routing through Gatun Lake with an estimated peak discharge of 7510 cms (265,030 cfs) through Gatun Spillway with the balance achieved by outflow through lock culverts and spilling over lock gates. Since use of the locks in the past during smaller floods has caused damage to the structures and hydraulic equipment, there are also benefits in either eliminating this practice or reducing its frequency.

The proposal to raise the Maximum Operating Lake Level (MOLL) of Gatun Lake would reduce the amount of surcharge available for flood routing and therefore require additional spillway capacity. It would also increase hydrostatic loads on the spillway and spillway gates under normal operating conditions and impact spillway operation

This section of the report explores these issues and discusses the feasibility and cost of adding additional spillway capacity to improve flood control and also to enable ACP to increase the MOLL.

[14]To account for differences in drainage area between Gatun damsite and Bohio, the observed flow data at Bohio was multiplied by 1.62.

3.2 Functional Requirements for New Spillway

3.2.1 Hazard Classification and Selection of Inflow Design Flood

Gatun Dam is classified as a “high hazard” dam as previously discussed in Section 2.1.4. Considering the potential catastrophic consequences of failure of Gatun Dam to human life and Panama’s economy, the Probable Maximum Flood (PMF) has been selected as the inflow design flood for Gatun Spillway.

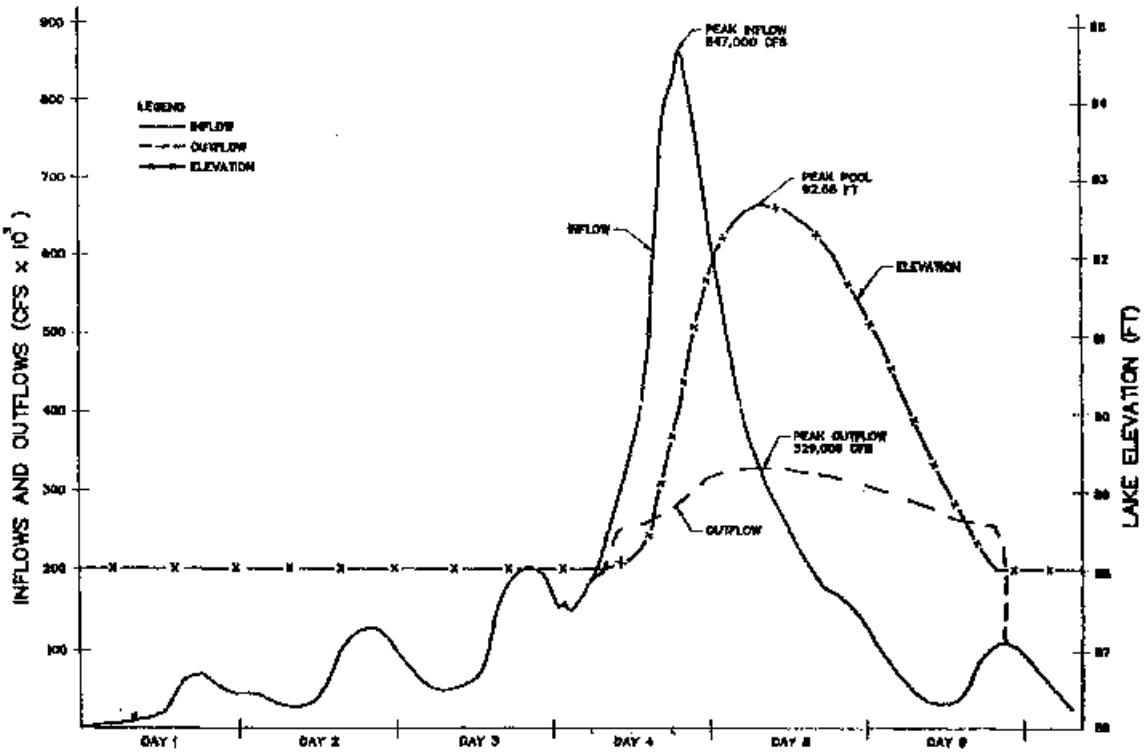
3.2.2 PMF & Reservoir Routing

The most recent PMF inflow hydrograph was developed by the US Army Corps of Engineers in 1979. The flood hydrograph and reservoir routing is shown on Figure 3-1.

The computed PMF inflow hydrograph has a duration of 6 days and a peak inflow to Gatun Lake of 23,984 cms (847,000 cfs) occurring on the fourth day of the flood event. In this analysis, Madden Lake was assumed to be at El. 74.68 m (245 ft) and Gatun Lake was assumed to be at El. 26.82 m (88 ft) at the onset of the flood. The routed peak outflow from Gatun Lake through the existing spillway and lock culverts was 9,316 cms (329,000 cfs) and the computed maximum pool elevation of Lake Gatun was 28.21 m (92.55 ft). Madden Lake had a peak inflow of 12,771 cms (451,000 cfs) and the peak outflow was 7,227 cms (257,000 cfs). The corresponding computed peak pool elevation for Madden Lake was El. 80.16 m (263.0 ft).

Under present conditions the maximum operating level of Gatun Reservoir is El 26.67 m (87.5 ft). The routing of the PMF through the reservoir indicates that under the present flood control plan which utilizes the existing spillway and lock culverts to control water surface elevations, the reservoir water surface will rise about 1.39 m (4.55 ft) from El. 26.82 m (88.0 ft) to El 28.21 m (92.55 ft). It should be noted that the historical maximum measured water level in the lake occurred on November 23rd, 2002 and was recorded as 26.89 m (88.22 ft).

Figure 3-1: Gatun Lake PMF Hydrograph and Reservoir Routing



In order to determine the hydraulic capacity needed for the new spillway, it was necessary to develop a tool capable of replicating the routing results of the existing spillway, lock culverts, and locks for the design event (PMF flood) (The USACE HEC1 model runs were not available). This tool could then be also modified to include the new spillway design characteristics and develop preliminary hydraulic designs for the proposed new spillway.

To develop this tool, a spreadsheet routing model was created. First, the USACE inflow and outflow hydrographs were digitized from the routing results in the USACE PMF Study. The rating curves developed for the existing spillway, lock culverts, and locks and reported in the Green Book were also recorded and input within the spreadsheet model. Highly detailed stage-storage data provided by ACP was also input within the spreadsheet model, and a mathematical function was developed to describe the stage-storage relationship for Gatun Lake. Various functions were tested (based on using various lake elevations, e.g. El. 77 ft – 100 ft, El. 85 ft – 93 ft, etc.) until the best agreement between the modeled and USACE outflow hydrographs was reached. The

final elevation range used to describe the stage-storage function was 23.47 m – 28.35 m (El. 77 ft – El 93 ft). The spreadsheet model was configured so that active elevations could be set for each outflow device (existing spillway, lock culverts, locks) as well as a toggle to turn each one on/off. See Figure 3-2 for an example of the input sheet for the spreadsheet tool.

Figure 3-2: Example of Input Sheet for Developed Spreadsheet Model

SINGLE RESERVOIR ROUTING

ROUTING TIME STEP

deltat = 60 min

INFLOW HYDROGRAPH

See Routing Page - Taken from Green Book

STAGE-STORAGE

Ks = 4128319000

b = 1.02624997

Zo = 77 ft

INITIAL WATER LEVEL

Zi = 87.98 ft

WATER LEVEL FOR SPILLWAY CAPACITY TO CONTROL

Zc = 88 ft

EXISTING LOCK CULVERTS

Use (Y or N) ? Y

Lock Culvert Outflow = 54,000 cfs

Elev When Engaged = 88.0 ft

EXISTING LOCKS

Use (Y or N) ? N

Lock Outflow = 160,000 cfs

Elev When Engaged = 88.0 ft

EXISTING SPILLWAY

Cw = See Routing Page - Taken From Green Book

L = See Routing Page - Taken From Green Book

Zcr = 68.89 ft

NEW SPILLWAY - CHRISTENSEN

# of gates = 0

Cw = 3.61

L = 40.57 ft

Zcr = 53 ft

COMPUTED RESULTS

Norm Surf Area =	***161.84***	***acres***
Max Surf Area =	***163.33***	***acres***
Peak Stage =	***92.57***	***ft***
Peak Outflow =	***329,631***	***cfs***

In addition to the accepted USACE PMF hydrograph, the spreadsheet model was verified against other routings presented in the USACE Green Book.

Figure 3-3 to Figure 3-4 show the model results versus the USACE results. The figures show that he spreadsheet model does an excellent job replicating the USACE routing results, and that this model can be used for preliminary designs of the proposed new spillway.

Figure 3-3: Spreadsheet Model Results – PMF Storm – 847,000 cfs

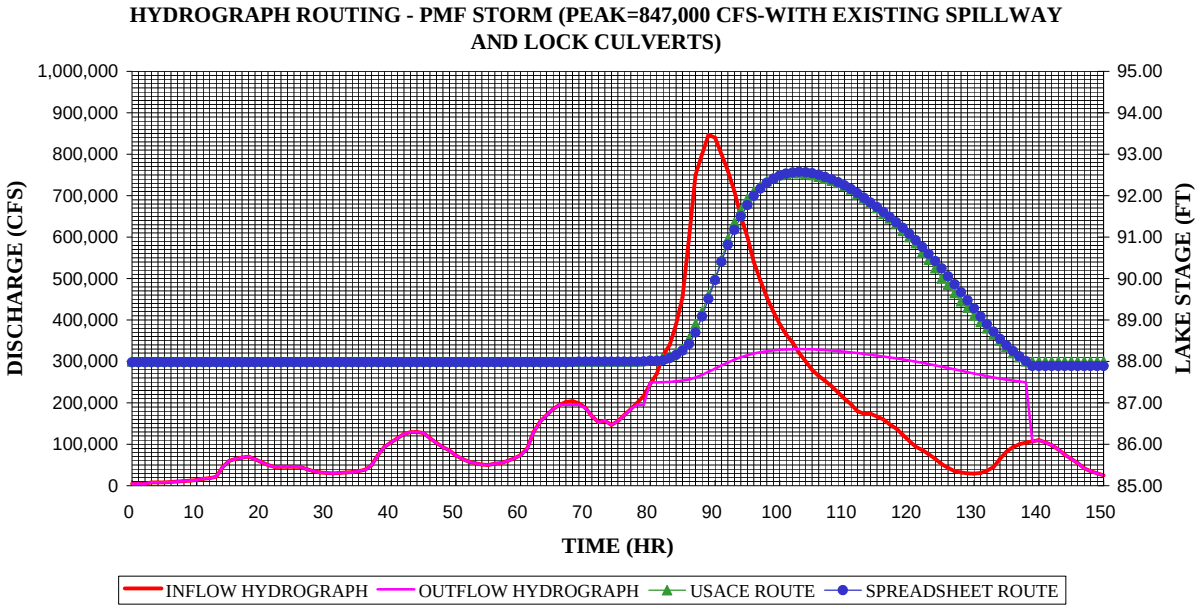
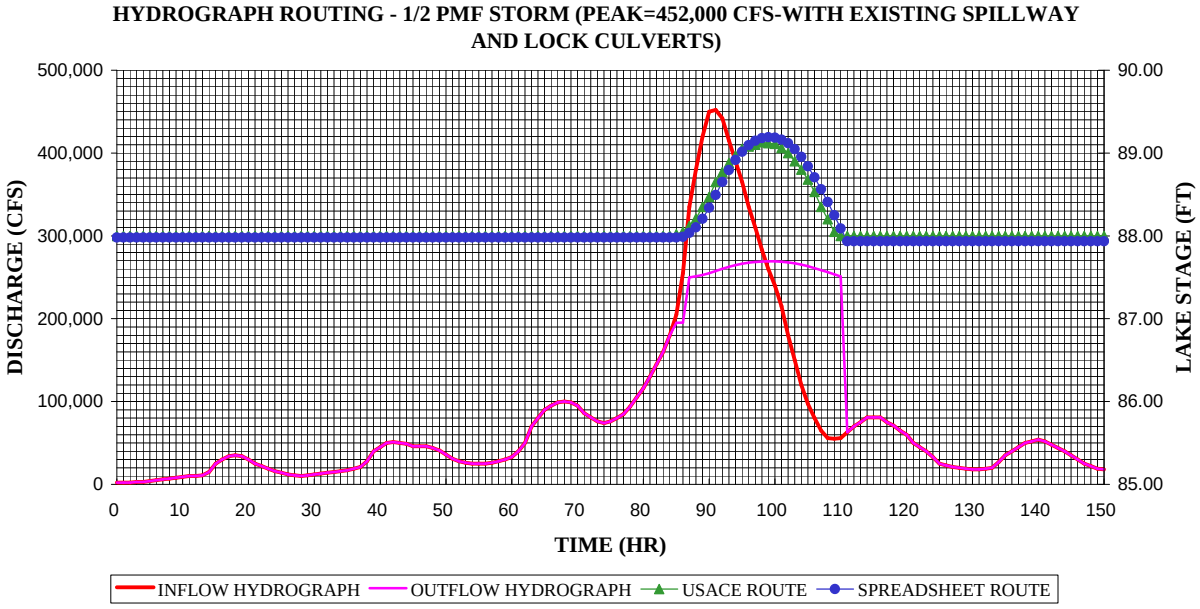


Figure 3-4: Spreadsheet Model Results – ½ PMF Storm – 452,000 cfs



3.2.3 Preliminary Hydraulic Design of New Spillway

Previous studies of increasing the spillway capacity at Gatun Lake have explored the possibility of augmenting spillway capacity by discharging water through the lock structures. In 1932, the lock culvert flow with Gatun Lake at El. 26.51 m (87 ft) was stated to be 765 cms (27,000 cfs) through the Gatun Lock culverts and 650 cms (23,000 cfs) through the Pedro Miguel lock culverts. In fact, the current ACP Flood Control Plan assumes that the lock culverts would be used during the PMF flood event.

Records of inspection of the locks in the 1920s and 1930s report damage to the culverts including erosion of concrete 7.6 to 10 cm (3 to 4 inches) deep and failure of an entrance screen at Gatun Lock. The conclusion of previous studies and direct guidance from ACP is that if a new spillway is constructed it should have suitable capacity to safely regulate the level of Gatun Lake without resorting to use of the lock culverts for flood control. So for the present study, it is assumed that the outlet capacity must be provided by the existing and additional spillway without discharge through the locks. This conservative assumption provides a small operational safety margin in that the lock culverts could be used in an emergency.

Another factor that ACP asked the project team to consider was the possibility of increasing the existing spillway capacity. As can be seen in Figure 2-6, there is a considerable amount of constriction associated with the existing spillway when the gates are fully open. The effective hydraulic length of the spillway crest is considerably shorter than the actual length due to the hydraulically inefficient shape of the “piers” which hold the spillway gates in place (note amount and “length” of turbulence in Figure 2-6).

Consideration was given to increasing the hydraulic length of the spillway crest by removing several piers and installing flap gates but was rejected due to difficulties associated with the curvature of the spillway crest. Another possibility considered was lowering the spillway crest – but this approach was determined to be impractical as it would reduce the hydraulic discharge coefficent of the crest, reduce the stability of the ogee, require replacement of the piers, and be difficult to construct while keeping the spillway operational. Therefore, the spreadsheet model was used without altering the existing spillway characteristics or including the lock culvert flows to determine the preliminary hydraulic design requirements for the new spillway.

The hydraulic design criteria used for the sizing of the new spillway are listed with the other design criteria in Section 3.4. However, the design flood used for the new spillway design was the PMF, and the maximum lake elevation allowed for the PMF event was held to 28.04 m (92.0 ft). See Section 3.4 for a full explanation of the reasoning behind these criteria.

The hydraulic rating curve for the new spillway was developed based on the approach and criteria developed by the USBR[15] and US Army Corps of Engineers[16].For modeling purposes, the average weir coefficient was determined to be 3.61, the effective length of each spillway gate was estimated to be 12.37 m (40.57 ft), and the spillway crest was designed to be 16.15 m (53.0 ft). The combination of spillway width and crest elevation represents a balance to achieve the required discharge capacity within the constraints of the topography of the dam abutment, estimated depths to the Gatun series rocks and feasible gate dimensions. This balance will be refined when the inflow design flood and flood routing is updated and design-level site investigation data is available.

Using all of the above information, the hydraulic sizing of the new spillway was completed for the range of lake levels studied – El 26.67 m (87.5 ft) – 27.43 m (90.0 ft) at 0.152 m (0.5 ft) intervals. (see Section 4.1 for explanation) The spreadsheet model developed for the existing PMF flood was taken and the number of gates for the new spillway was increased until the maximum PMF elevation was at or below 28.04 m (92.0 ft). Table 3-1 through Table 3-6 show the finalized spreadsheet inputs and the corresponding routing graph for each MOLL.

[15] USBR, Design of Small Dams, 1987

[16] US Army Corps of Engineers WES Vicksburg, MS, Hydraulic Design Criteria 1988

Table 3-1: New Spillway Hydraulic Design Results for MOLL of 26.67 m (87.5 ft)

SINGLE RESERVOIR ROUTING

ROUTING TIME STEP

deltat = 60 min

INFLOW HYDROGRAPH

See Routing Page - Taken from Green Book

STAGE-STORAGE

Ks = 4128319000
b = 1.02624997
Zo = 77 ft

INITIAL WATER LEVEL

Zi = 87.5 ft

WATER LEVEL FOR SPILLWAY CAPACITY TO CONTROL

Zc = 87.7 ft

EXISTING LOCK CULVERTS

Use (Y or N) ? N
Lock Culvert Outflow = 54,000 cfs
Elev When Engaged = 87.8 ft

EXISTING LOCKS

Use (Y or N) ? N
Lock Outflow = 160,000 cfs
Elev When Engaged = 88.0 ft

EXISTING SPILLWAY

Cw = See Routing Page - Taken From Green Book
L = See Routing Page - Taken From Green Book
Zcr = 68.89 ft

NEW SPILLWAY - CHRISTENSEN

# of gates = 2
Cw = 3.61
L = 40.57 ft
Zcr = 53 ft

COMPUTED RESULTS

Norm Surf Area =	***161.65***	***sq. mi.***
Max Surf Area =	***163.17***	***sq. mi.***
Peak Stage =	***92.0***	***ft***
Peak Outflow =	***336,755***	***cfs***

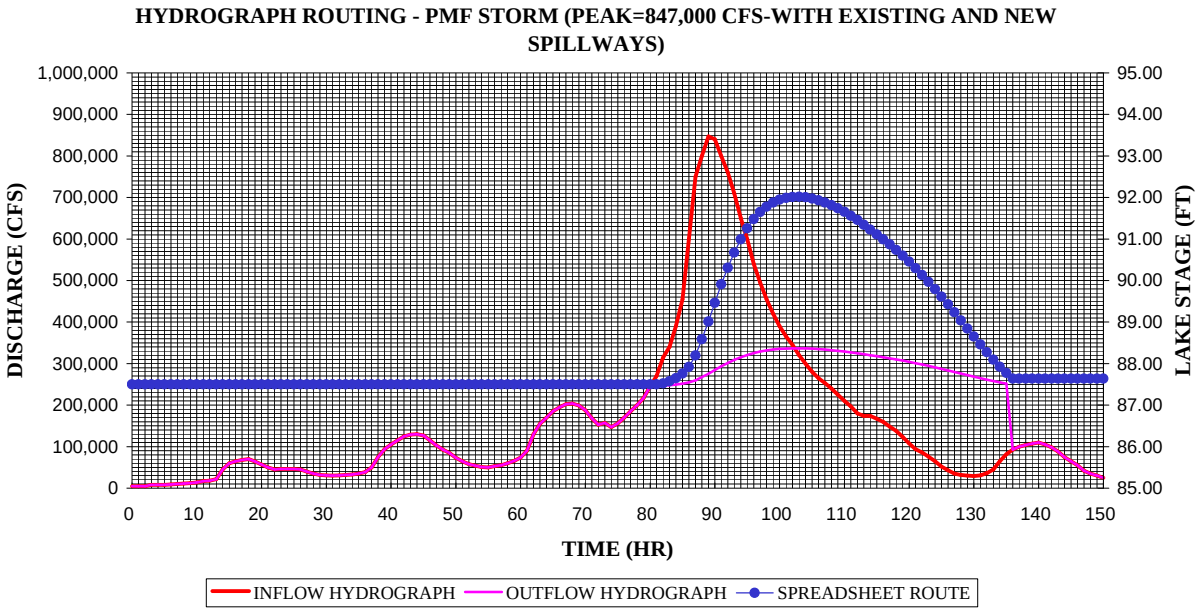


Table 3-2: New Spillway Hydraulic Design Results for MOLL of 26.82 m (88.0 ft)

SINGLE RESERVOIR ROUTING

ROUTING TIME STEP

deltat = 60 min

INFLOW HYDROGRAPH

See Routing Page - Taken from Green Book

STAGE-STORAGE

Ks = 4128319000
b = 1.02624997
Zo = 77 ft

INITIAL WATER LEVEL

Zi = 88 ft

WATER LEVEL FOR SPILLWAY CAPACITY TO CONTROL

Zc = 88.2 ft

EXISTING LOCK CULVERTS

Use (Y or N) ? N
Lock Culvert Outflow = 54,000 cfs
Elev When Engaged = 87.8 ft

EXISTING LOCKS

Use (Y or N) ? N
Lock Outflow = 160,000 cfs
Elev When Engaged = 88.0 ft

EXISTING SPILLWAY

Cw = See Routing Page - Taken From Green Book
L = See Routing Page - Taken From Green Book
Zcr = 68.89 ft

NEW SPILLWAY - CHRISTENSEN

# of gates = 3
Cw = 3.61
L = 40.57 ft
Zcr = 53 ft

COMPUTED RESULTS

Norm Surf Area =	***161.84***	***sq. mi.***
Max Surf Area =	***163.14***	***sq. mi.***
Peak Stage =	***91.9***	***ft***
Peak Outflow =	***370,003***	***cfs***

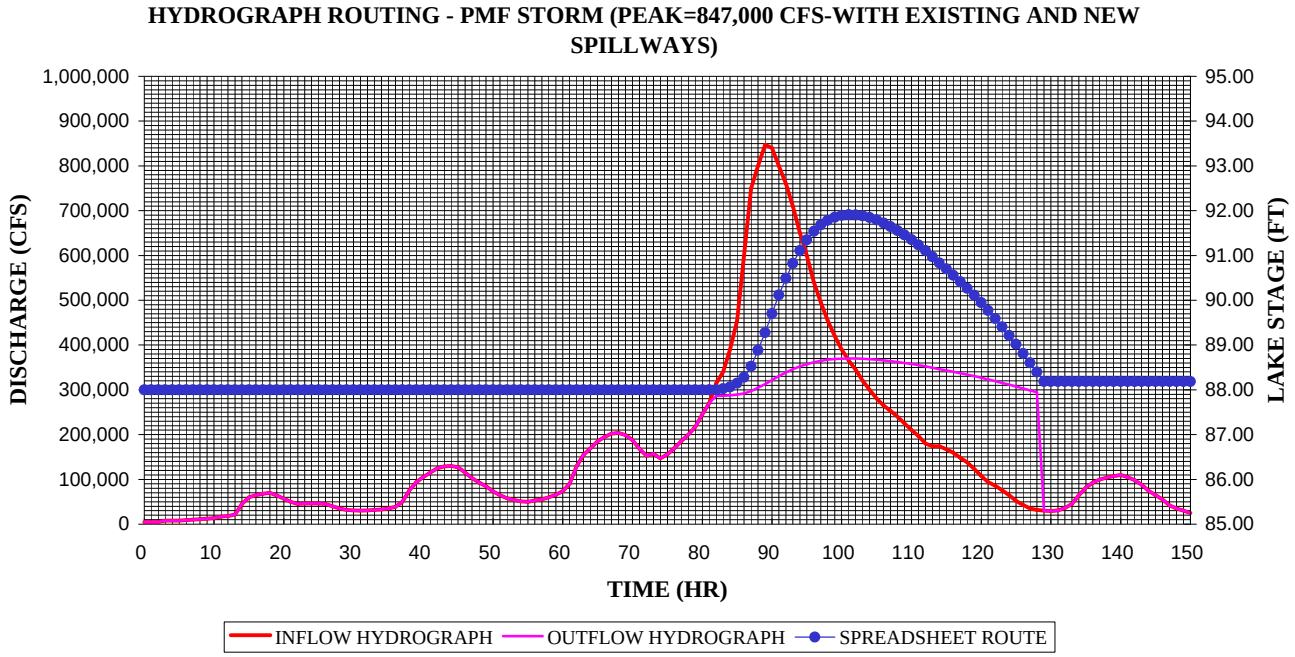


Table 3-3: New Spillway Hydraulic Design Results for MOLL of 26.97 m (88.5 ft)

SINGLE RESERVOIR ROUTING

ROUTING TIME STEP

deltat = 60 min

INFLOW HYDROGRAPH

See Routing Page - Taken from Green Book

STAGE-STORAGE

Ks = 4128319000
b = 1.02624997
Zo = 77 ft

INITIAL WATER LEVEL

Zi = 88.5 ft

WATER LEVEL FOR SPILLWAY CAPACITY TO CONTROL

Zc = 88.7 ft

EXISTING LOCK CULVERTS

Use (Y or N) ? N
Lock Culvert Outflow = 54,000 cfs
Elev When Engaged = 87.8 ft

EXISTING LOCKS

Use (Y or N) ? N
Lock Outflow = 160,000 cfs
Elev When Engaged = 88.0 ft

EXISTING SPILLWAY

Cw = See Routing Page - Taken From Green Book
L = See Routing Page - Taken From Green Book
Zcr = 68.89 ft

NEW SPILLWAY - CHRISTENSEN

# of gates = 4
Cw = 3.61
L = 40.57 ft
Zcr = 53 ft

COMPUTED RESULTS

Norm Surf Area =	***162.03***	***sq. mi.***
Max Surf Area =	***163.13***	***sq. mi.***
Peak Stage =	***91.9***	***ft***
Peak Outflow =	***404,254***	***cfs***

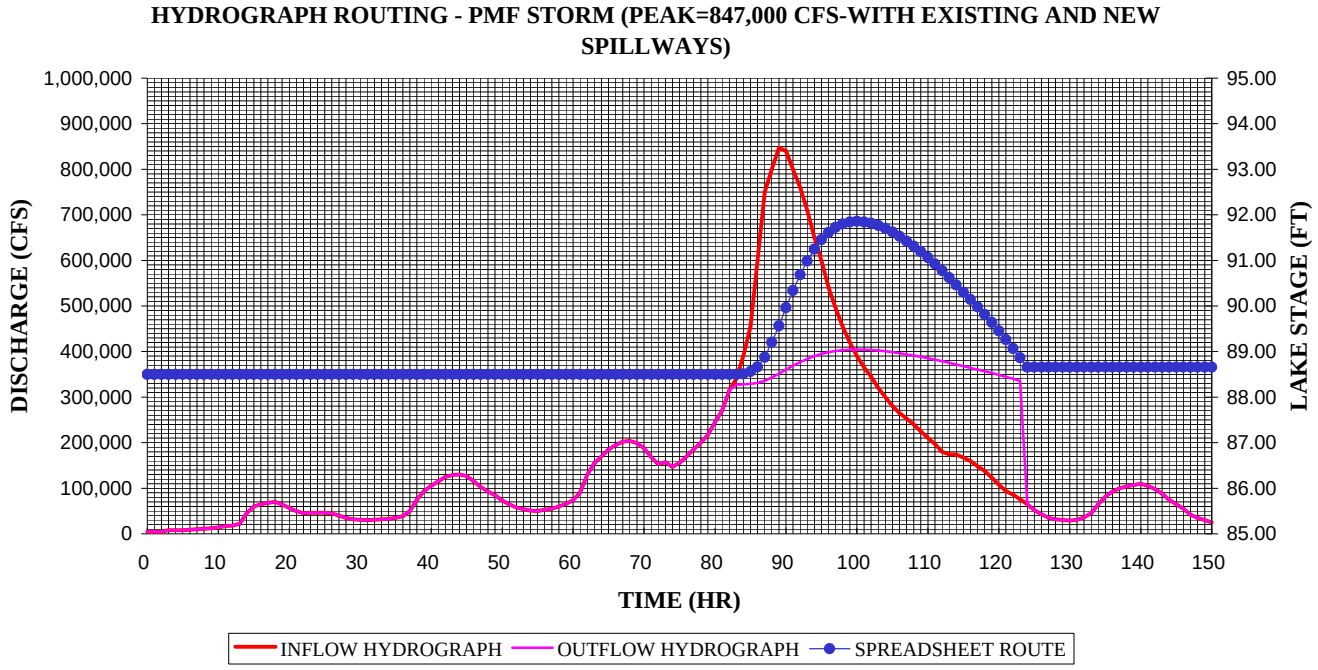


Table 3-4: New Spillway Hydraulic Design Results for MOLL of 27.13 m (89.0 ft)

SINGLE RESERVOIR ROUTING

ROUTING TIME STEP

deltat = 60 min

INFLOW HYDROGRAPH

See Routing Page - Taken from Green Book

STAGE-STORAGE

Ks = 4128319000
b = 1.02624997
Zo = 77 ft

INITIAL WATER LEVEL

Zi = 89 ft

WATER LEVEL FOR SPILLWAY CAPACITY TO CONTROL

Zc = 89.2 ft

EXISTING LOCK CULVERTS

Use (Y or N) ? N
Lock Culvert Outflow = 54,000 cfs
Elev When Engaged = 87.8 ft

EXISTING LOCKS

Use (Y or N) ? N
Lock Outflow = 160,000 cfs
Elev When Engaged = 88.0 ft

EXISTING SPILLWAY

Cw = See Routing Page - Taken From Green Book
L = See Routing Page - Taken From Green Book
Zcr = 68.89 ft

NEW SPILLWAY - CHRISTENSEN

# of gates = 5
Cw = 3.61
L = 40.57 ft
Zcr = 53 ft

COMPUTED RESULTS

Norm Surf Area =	***162.21***	***sq. mi.***
Max Surf Area =	***163.13***	***sq. mi.***
Peak Stage =	***91.9***	***ft***
Peak Outflow =	***439,841***	***cfs***

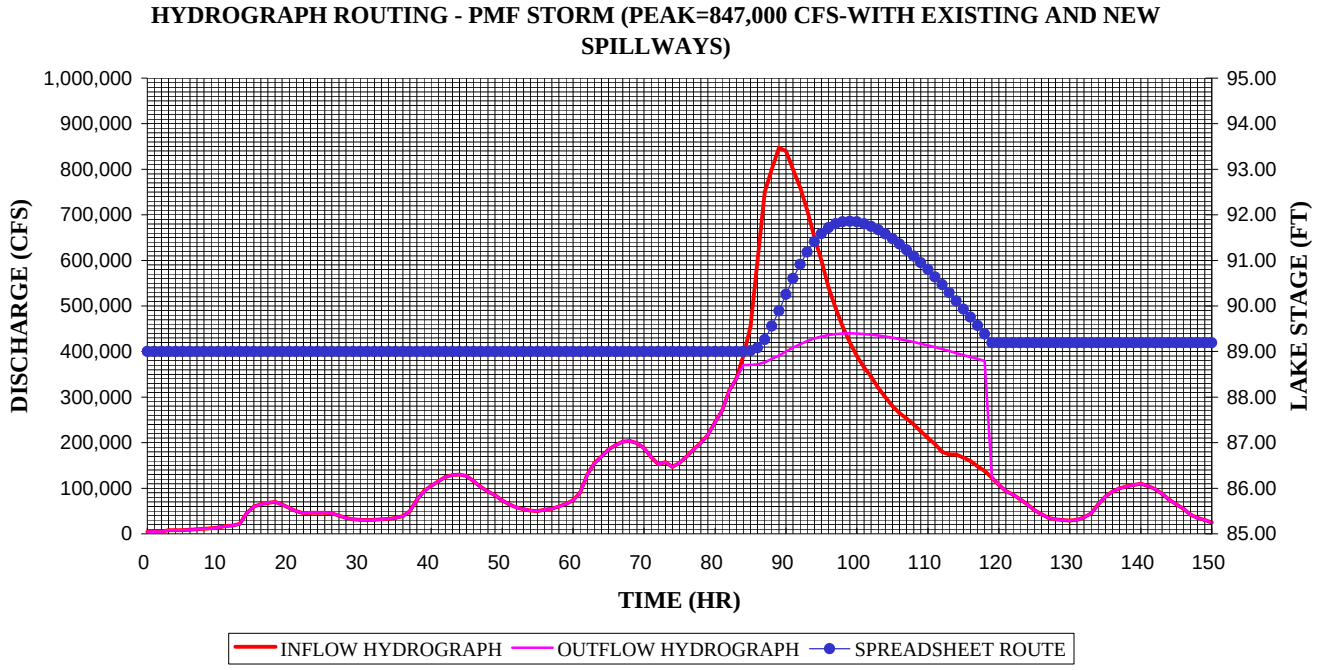


Table 3-5: New Spillway Hydraulic Design Results for MOLL of 27.28 m (89.5 ft)

SINGLE RESERVOIR ROUTING

ROUTING TIME STEP

deltat = 60 min

INFLOW HYDROGRAPH

See Routing Page - Taken from Green Book

STAGE-STORAGE

Ks = 4128319000
b = 1.02624997
Zo = 77 ft

INITIAL WATER LEVEL

Zi = 89.5 ft

WATER LEVEL FOR SPILLWAY CAPACITY TO CONTROL

Zc = 89.7 ft

EXISTING LOCK CULVERTS

Use (Y or N) ? N
Lock Culvert Outflow = 54,000 cfs
Elev When Engaged = 87.8 ft

EXISTING LOCKS

Use (Y or N) ? N
Lock Outflow = 160,000 cfs
Elev When Engaged = 88.0 ft

EXISTING SPILLWAY

Cw = See Routing Page - Taken From Green Book
L = See Routing Page - Taken From Green Book
Zcr = 68.89 ft

NEW SPILLWAY - CHRISTENSEN

# of gates = 6
Cw = 3.61
L = 40.57 ft
Zcr = 53 ft

COMPUTED RESULTS

Norm Surf Area =	***162.39***	***sq. mi.***
Max Surf Area =	***163.14***	***sq. mi.***
Peak Stage =	***91.9***	***ft***
Peak Outflow =	***476,620***	***cfs***

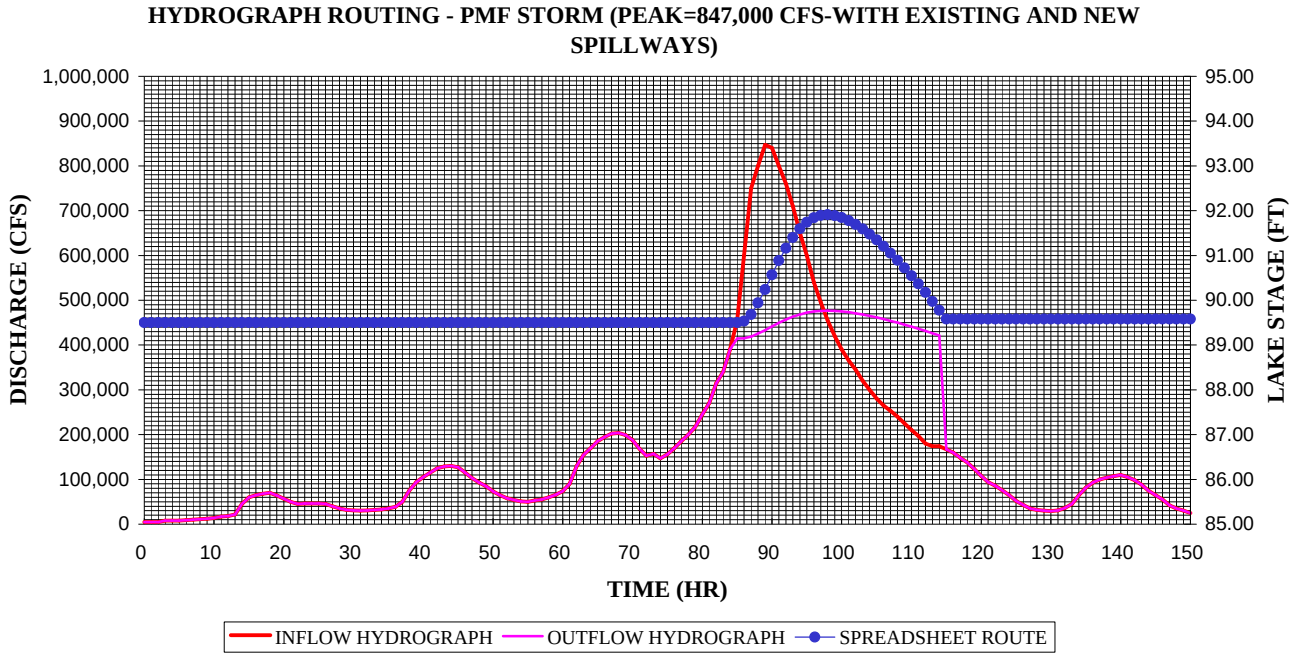


Table 3-6: New Spillway Hydraulic Design Results for MOLL of 27.43 m (90.0 ft)

SINGLE RESERVOIR ROUTING

ROUTING TIME STEP

deltat = 60 min

INFLOW HYDROGRAPH

See Routing Page - Taken from Green Book

STAGE-STORAGE

Ks = 4128319000
b = 1.02624997
Zo = 77 ft

INITIAL WATER LEVEL

Zi = 90 ft

WATER LEVEL FOR SPILLWAY CAPACITY TO CONTROL

Zc = 90.2 ft

EXISTING LOCK CULVERTS

Use (Y or N) ? N
Lock Culvert Outflow = 54,000 cfs
Elev When Engaged = 87.8 ft

EXISTING LOCKS

Use (Y or N) ? N
Lock Outflow = 160,000 cfs
Elev When Engaged = 88.0 ft

EXISTING SPILLWAY

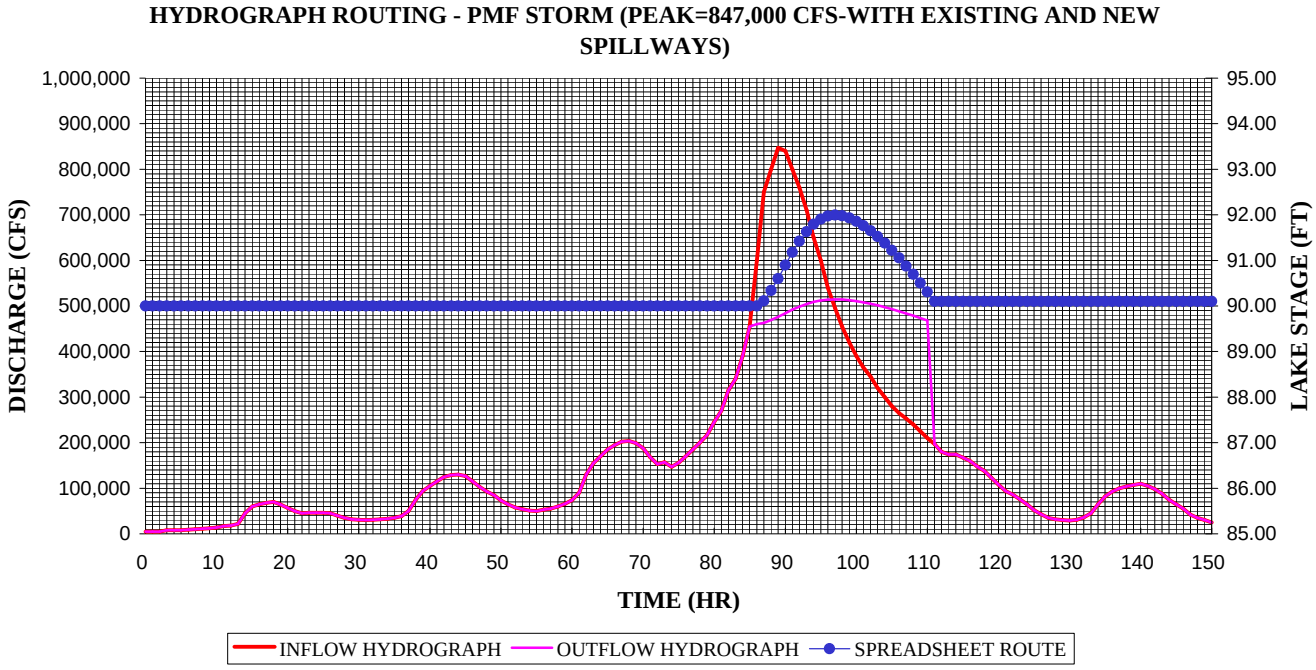
Cw = See Routing Page - Taken From Green Book
L = See Routing Page - Taken From Green Book
Zcr = 68.89 ft

NEW SPILLWAY - CHRISTENSEN

# of gates = 7
Cw = 3.61
L = 40.57 ft
Zcr = 53 ft

COMPUTED RESULTS

Norm Surf Area =	***162.55***	***sq. mi.***
Max Surf Area =	***163.16***	***sq. mi.***
Peak Stage =	***92.0***	***ft***
Peak Outflow =	***514,536***	***cfs***



As can be seen in the above figures, as the MOLL is raised the available surcharge depth and therefore volume is reduced necessitating additional spillway capacity (more gates) and higher design discharges to control the pool elevation. For these calculations, it was assumed that the outflow would equal inflow until the lake level rose 0.06 m (0.2 ft) above the MOLL after which the sum of the existing and new spillway capacities control (just as the current USACE PMF routings do). Table 3-7 shows the resulting number of gates and corresponding peak discharge required for each MOLL. Please note that the number gates required to pass the PMF flood ranges from two to seven, depending on the MOLL selected and that the total discharge for both spillways ranges from 9,536 cms (336,755 cfs) to 14,870 cms (514,536 cfs).

Table 3-7: Summary of New Spillway Hydraulic Design Results

MOLL		# Gates Req.	Peak Flood Elev.		Peak Total Q		Peak Ex. Spill. Q		Peak New Spill. Q	
(m)	(ft)	for New Spillway	(m)	(ft)	(cms)	(cfs)	(cms)	(cfs)	(cms)	(cfs)
26.67	87.5	2	28.04	92.0	9,536	336,755	7,267	256,624	2,269	80,131
26.82	88.0	3	28.01	91.9	10,477	370,003	7,218	254,897	3,259	115,106
26.97	88.5	4	28.01	91.9	11,447	404,254	7,218	254,897	4,229	149,357
27.13	89.0	5	28.01	91.9	12,455	439,841	7,218	254,897	5,237	184,944
27.28	89.5	6	28.01	91.9	13,496	476,620	7,218	254,897	6,279	221,723
27.43	90.0	7	28.04	92.0	14,570	514,536	7,267	256,624	7,303	257,912

3.3 Location Options

3.3.1 Available Information

Previous Reports

The following previous reports regarding increasing the spillway capacity of Gatun Lake were reviewed:

- Panama Canal Zone, Department of Operation and Maintenance - Report on Preliminary Office Study on Additional Spillway Capacity From Gatun Lake – E.S. Randolph, March 10. 1945
- USACE 1999 Study

Topography

The study of alternative locations for a new spillway(s) was based on the following mapping:

- 1:50,000 scale topographic maps of the Lake Gatun area which have contour intervals at 10 and 20 meter intervals.

- Photogrammetric mapping of the east portion of Gatun Dam and Gatun Spillway showing 1 meter contours.
- Topographic maps of the west side of Gatun Lake derived from IFSAR radar data. and portions of the west side of Gatun Lake
- Orthophotos of the Lower Rio Chagres downstream of Gatun Dam.

The vertical datum used is PLD Datum. Elevations cited from reference documents or drawings are in the datum used in the reference document.

Hydrology

The study of alternative spillway sites also considered the hydraulic capacity of the receiving streams based on previous work completed by M&N for ACP. These hydrologic analyses should be considered approximate, but the analysis is useful for order-of-magnitude estimates and to compare potential sites.

The analysis is based on the measured data from the Los Cañones gage during the November 2002 flood. This gage data was the only discharge measurements available for a large, natural watershed without significant control or development. Our previous study showed that this flood was a significant local flood with rainfall depths over the Los Cañones watershed approaching those for a 100-yr, 24-hr event. From topographic mapping the bottom width of the existing channel was estimated to be 61 m (200 ft). The drainage area for the watershed was measured to be roughly 18,780 ha (72.5 sq. mi.), and the flood discharge during this event was measured to be 365 cms (12,980 cfs). Therefore, based on the existing natural channel at Los Cañones, a preliminary natural channel bottom width and flood discharge capacity could be estimated. Although this analysis is preliminary and approximate, these estimates should be reasonable given that all of the watersheds on the western arm of the lake are largely natural. Table 3-8 outlines the ratios that were used to estimate natural channel bottom width and receiving capacity.

Table 3-8: Hydrologic Values Used to Estimate Receiving Stream Width and Capacity

Drainage Area		Natural Channel Width/Drainage Area		Natural Channel Discharge Capacity/Drainage Area	
ha	Sq. mi.	m/ha	ft/sq.mi.	cms/ha	cfs/sq.mi.
18,780	72.5	0.003	2.76	0.019	179.0

Environmental

The study also considered the environmental aspects of each potential spillway site. Since the hydrology was analyzed separately, the environmental analysis mainly considers effects on population, flora and fauna.

Geology

The study also considers the geotechnical aspects of each potential spillway site.

Field Reconnaissance

Reconnaissance of potential sites was carried out during a field trip to Panama from March 10-12, 2004. In addition to a project kick-off meeting and review of existing information provided by ACP, the team carried out an aerial and ground assessment of the project area.

Aerial Reconnaissance

An aerial reconnaissance by helicopter was carried out on March 10, 2004 to orient the project team to the Canal facilities and to identify those facilities that would be affected by an increase in maximum operating level of Gatun Lake.

The flight traversed the Canal starting in Panama City and heading northwest across Miraflores Lake then northeast to Madden Dam. It then followed the Rio Chagres from Madden Dam to Gatun Lake. The route then headed northwest to Gatun Locks, Dam and Spillway and returned to Panama City via the west side of Gatun Lake.

Ground Reconnaissance

A ground reconnaissance was carried out on March 11, 2004. The trip started from Panama City and headed northwest up the Gaillard Highway passing Miraflores Locks and then northeast up to Madden Dam.

After viewing Madden Dam and Spillway from the lookout area near the highway, the team headed northwest to the Gatun Area along the TransIsthmian Highway passing through the Canoa area which was proposed as the preferred spillway site by the USACE in their 1999 study.

Reconnaissance in the Gatun area included inspection of the existing spillway and hydroelectric plant, Gatun Dam and the potential spillway site at the west abutment.

A brief road tour was made to the south along the west side of Gatun Lake to La Treinticinco passing several potential spillway sites that the team had identified on topographic maps. A side trip was taken to the northwest through Providencia and

Achiote to see the general terrain downstream of potential spillways sites west of Gatun Dam.

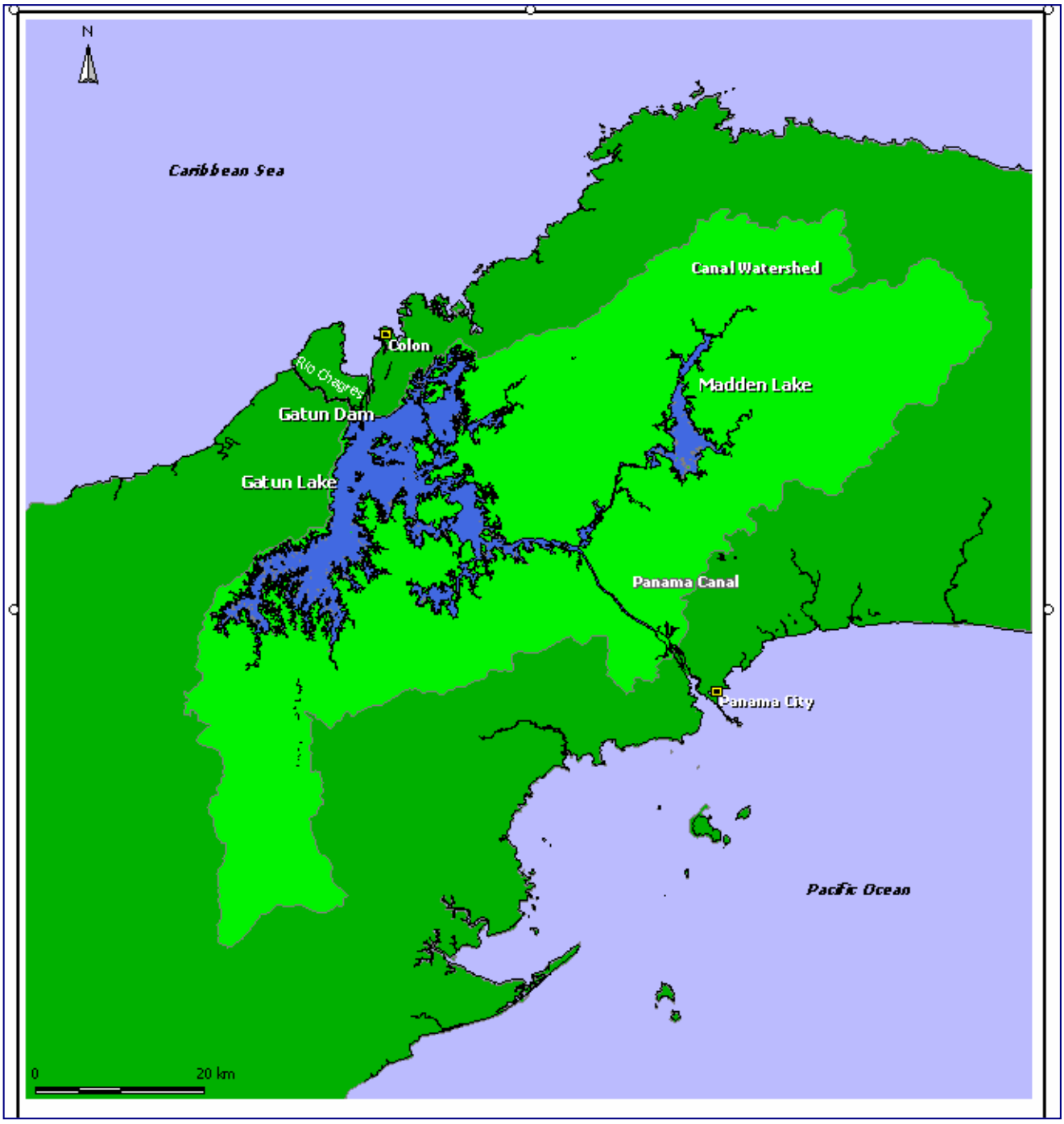
3.3.2 Siting Options

General

As seen in Figure 3-5, the Panama Canal watershed drains primarily north to the Caribbean Sea. Historically, the main outlet has been the Rio Chagres which flows into the Caribbean Sea near Colon.

Since construction of the Canal, water from Gatun Lake flows through both lock systems into the Caribbean Sea and the Pacific Ocean. Flood flows are primarily discharged to the Caribbean Sea via Gatun Spillway and the Rio Chagres. So conceptually, although additional spillway capacity could be located at either end of the Canal, discharge of flood flows to the Caribbean Sea would be more consistent with existing hydrology and the natural environment.

Figure 3-5: Panama Canal and Watershed



Pacific Coast

The concept of discharging spill from the Gatun Lake to the Pacific Ocean via Miraflores Lake and the Canal has potential navigational and environmental impacts. The currents caused by operation of the spillway would affect navigation, and the diversion of more streamflow from the Caribbean coast to the Pacific may raise environmental issues since the historical flow pattern of the watershed is toward the Atlantic (Carribean) side.

Atlantic (Caribbean) Coast

The three areas shown on Figure 3-6 have been evaluated as potential locations for a supplemental spillway, including:

- Gatun Dam (3 sub-sites)
- Canoa - which is east of Gatun Dam near Sabanita and Nuevo Colon.
- West of Gatun Dam (3 sub-sites)

The following sections describe the possibilities within each area.

At Gatun Dam

Three potential spillway sites have been identified at Gatun Dam as shown in Figure 3-7:

- West Abutment
- Spillway Hill
- East Abutment

The following subsections describe these options.

<u>West Abutment</u>

The most promising site for a new spillway at Gatun Dam is located at the west abutment of the dam. The western section of Gatun Dam was constructed upon a natural ridge which rises above El. 30.48 m (100 ft). Old geological sections of the damsite indicate that the ridge appears to be comprised of sedimentary rock overlain by about 6.1 – 9.1 m (20 to 30 ft) of clay.

The shallow depth of the embankment dam at this location appears favorable for constructing a spillway with short approach and discharge channels. The location near the abutment is also favorable for cofferdams required to facilitate construction. The spillway design would have to incorporate a bridge across the top of the structure (preferable) or along the discharge channel so that access to southern villages is not cut off.

Additional topographic and geotechnical data is required to fully evaluate the feasibility of developing a new spillway at this site. Field program recommendations for this site were made to ACP, and subsequent investigations were carried out.

Figure 3-6: Locations of Alternative Spillway Sites

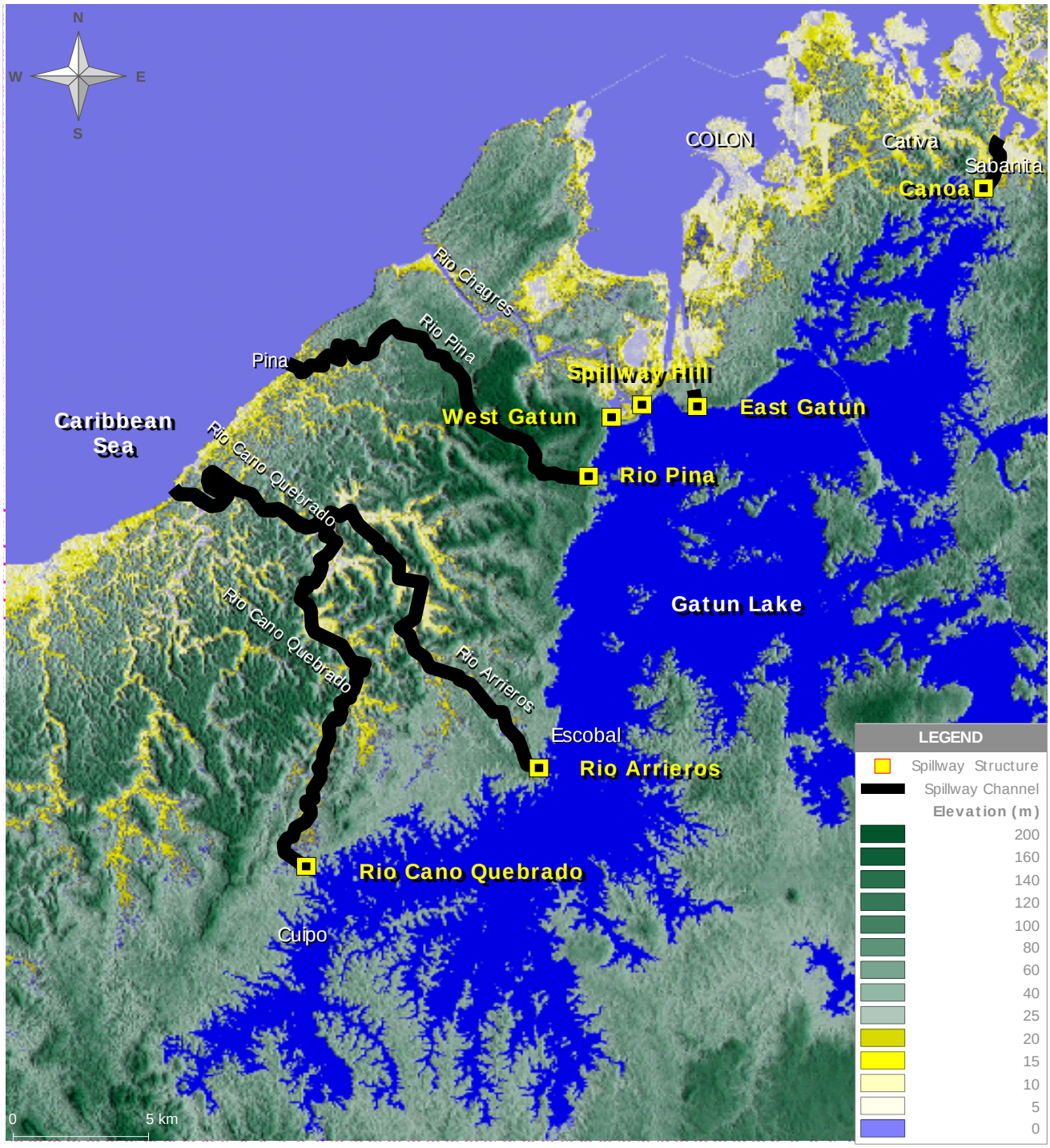
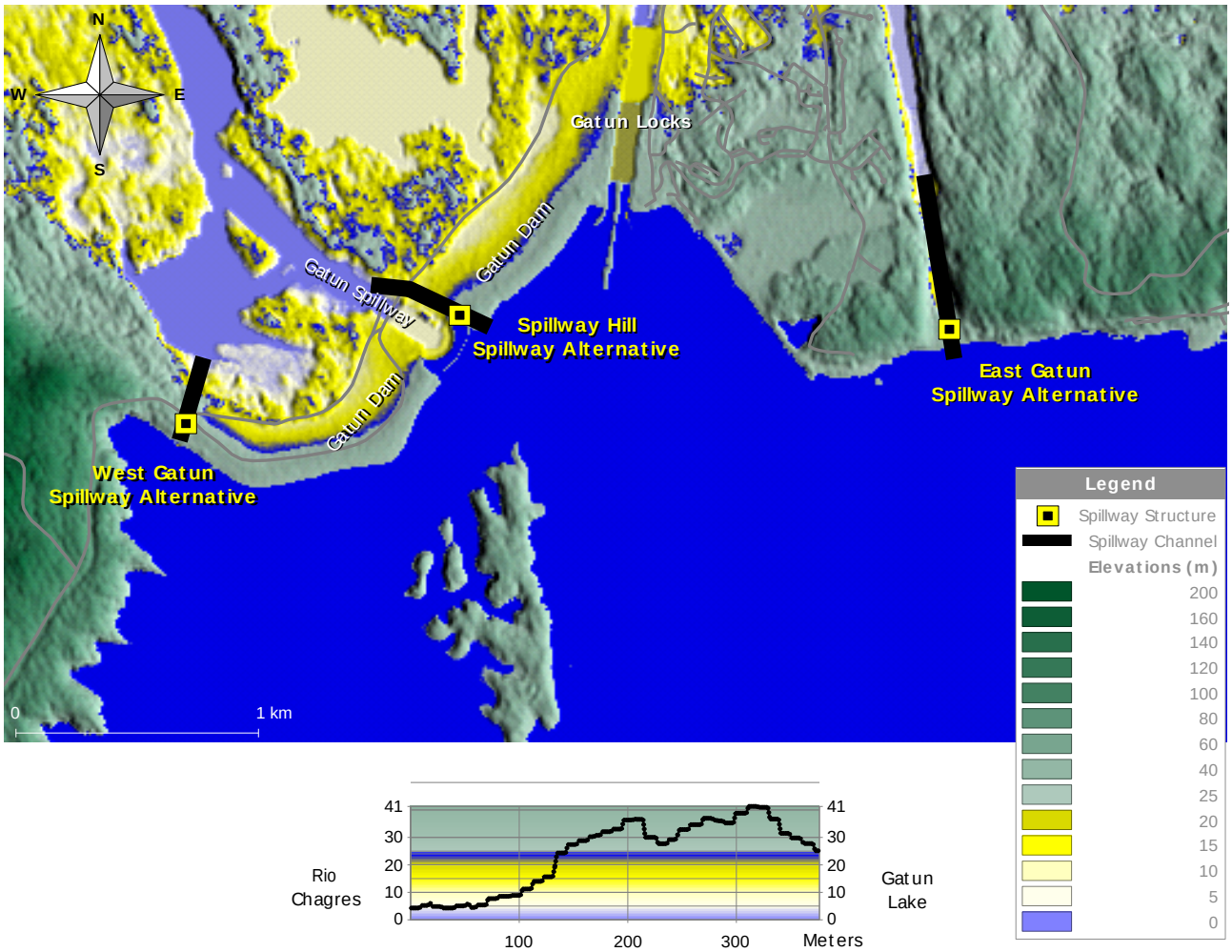
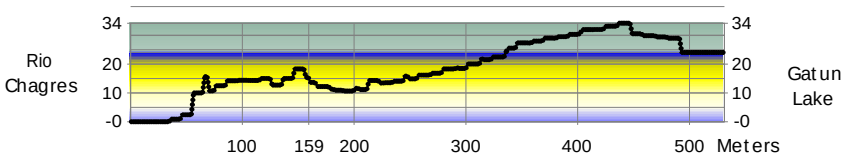


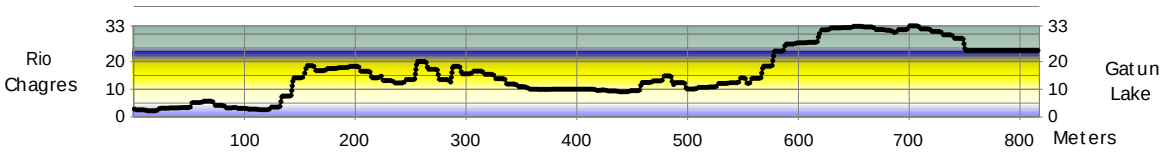
Figure 3-7: Gatun Spillway Sites



Profile - West Gatun Spillway Alternative



Profile - Spillway Hill Spillway Alternative



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Profile - East Gatun Spillway Alternative

Spillway Hill

The existing spillway at Gatun Lake was constructed upon Spillway Hill which is an outcrop of sedimentary rock which divides Gatun Dam into two sections. The spillway channel is founded on this outcrop and the downstream toe of the embankments that form Gatun Dam are also feathered into Spillway Hill.

There is a deep buried river channel immediately to the west of the present spillway which precludes expansion of the existing spillway in this direction.

To the east of the existing spillway there is a higher foundation but the flat slope of the existing embankment would require large approach walls and large cofferdams. Construction would also be constrained by the need to maintain operation of the existing spillway. For this option, it is also likely that the existing roadway would have to be modified and a bridge constructed over the proposed discharge channel.

From a navigation standpoint, it is reported that currents toward the existing spillway are a concern during high discharge due to the close proximity and alignment of the entrance of Gatun Locks relative to the existing Spillway. Therefore considering these factors, this site is ranked to be generally inferior to the site at the west abutment.

East Abutment

In the past, a deep cut has been excavated in the left abutment of Gatun Dam in preparation for future installation of new locks. It is understood that the feasibility of constructing additional locks is presently being evaluated by ACP and that this area is considered to one of the possible sites for the new locks. In addition, a spillway so close to the Gatun locks would be disadvantageous due to possible influences on currents near the navigation channels and likely conflicts with the navigation channels while the spillway is being constructed as well as during future maintenance activities. The construction and rerouting of a new railroad bridge over or around the proposed spillway also makes this location problematic. The likely changes in grades and horiziontal curvature in the tracks required to accommodate the spillway structure would be very difficult to design for and would be very costly as compared to a normal roadway.

Canoa (Sabanita)

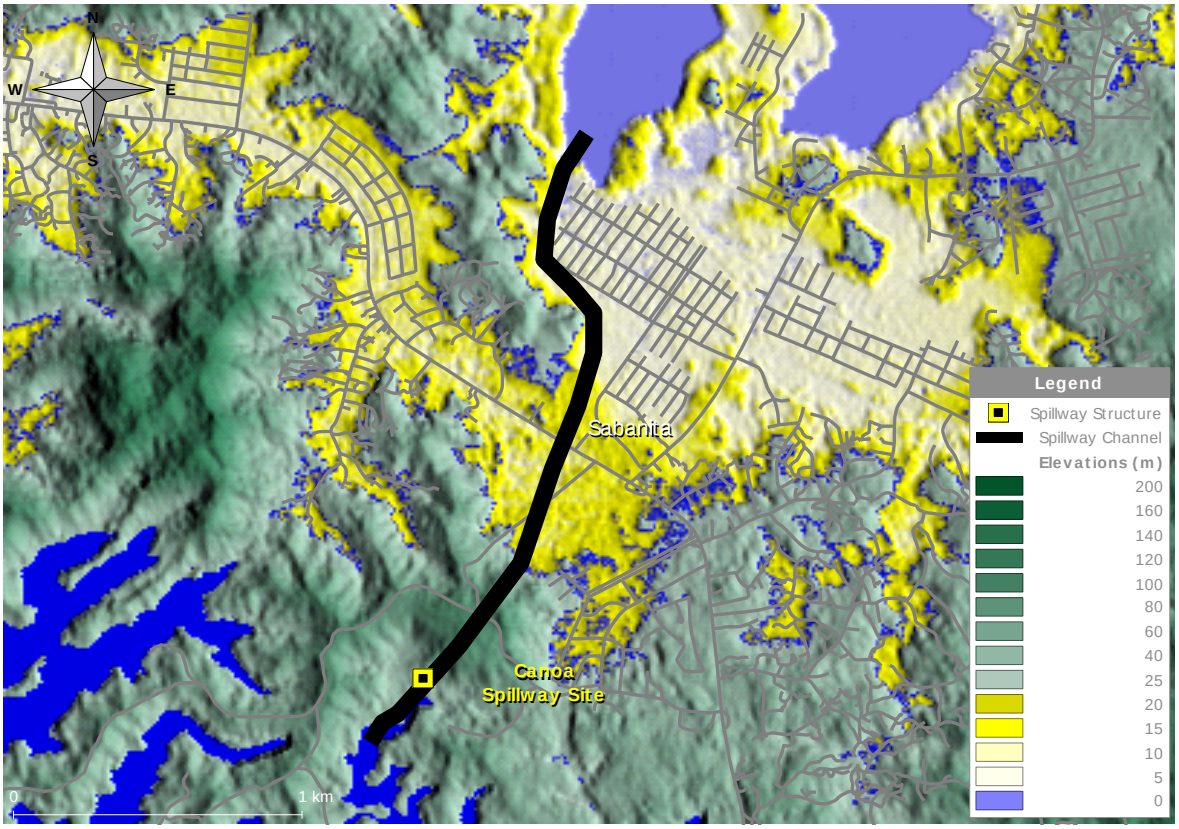
The possibility of constructing an auxiliary spillway at Canoa saddle was explored by Randolph in 1945.

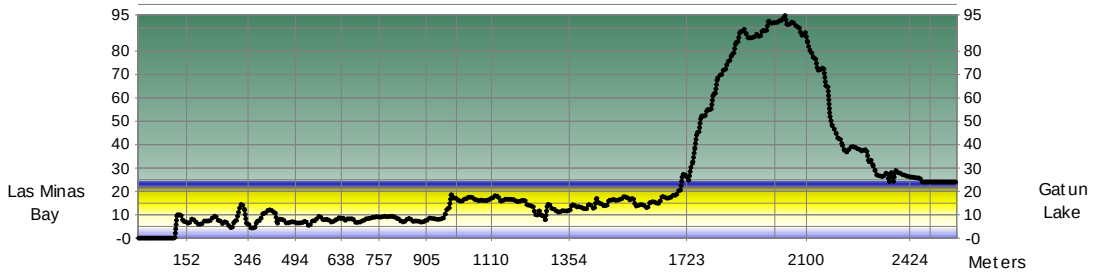
The proposed configuration of the spillway structure was similar to the existing Gatun Spillway. However, it required lengthy approach and discharge channels (see Figure 3-15). According to Randolph, the approach channel from Gatun Lake would be approximately 1524 m (5000 ft) long and require dredging of an estimated 2,220,000 cu

m (2,900,000 cubic yards) of earth and rock from the Gatun Lake shoreline and lake bottom. Downstream of the spillway structure, a 1250 m (4090 ft) long concrete lined discharge chute would be required to convey the water to Las Minas Bay where outflow would be discharged into an 2500 m (8200 ft) long discharge channel requiring an estimated 1,730,000 cu m (2,260,000 cu yards) of marine dredging in the bay.

Since 1945, this area has become heavily developed which has created very difficult environmental and land ownership constraints to the siting and development of a new spillway at this location. From the standpoint of costs and hydraulic performance, the considerable length of approach channel, discharge chute and discharge channel also make this site unattractive relative to other more compact sites.

Figure 3-8: Canoa Spillway Site





Profile - Canoa Spillway Alternative

5/19/2004
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Disruption of the heavily trafficked Trans-Isthmian Highway would also be required with the construction of a bridge over the discharge channel. Temporary re-routing of this highway is further complicated by the population center and number of structures that surround the highway allowing for no preferred detour route during construction.

West of Gatun Dam

The possibility of locating a new spillway at possible sites along the perimeter of Lake Gatun to the west of Gatun Dam was briefly considered in earlier studies (Randolph 1945). At that time, this concept was dismissed because maps and ground surface profiles indicated that the topography is not favorable relative to other sites because of the depth of excavation required for the spillway structure and the length of the required discharge channel to reach the Caribbean Sea.

For the present study the following three options shown on Figure 3-9 were identified:

Rio Piña

The Rio Piña option would require excavation of a deep cut (up to 100 m (330 ft) high) through the western rim of Gatun Lake into the Rio Piña Watershed. The total length of the waterway from the reservoir to the mouth of the Rio Piña at the Caribbean Sea would be 17.2 km (10.7 mi). Due to the topography, about two thirds of the channel would require considerable excavation to grade. As with all of these options, the spillway design would have to incorporate a bridge across the top of the structure or along the discharge channel. Due to the substantial depth of excavation required at this site, construction of a bridge at required grades would be problematic and expensive due to the surrounding topography.

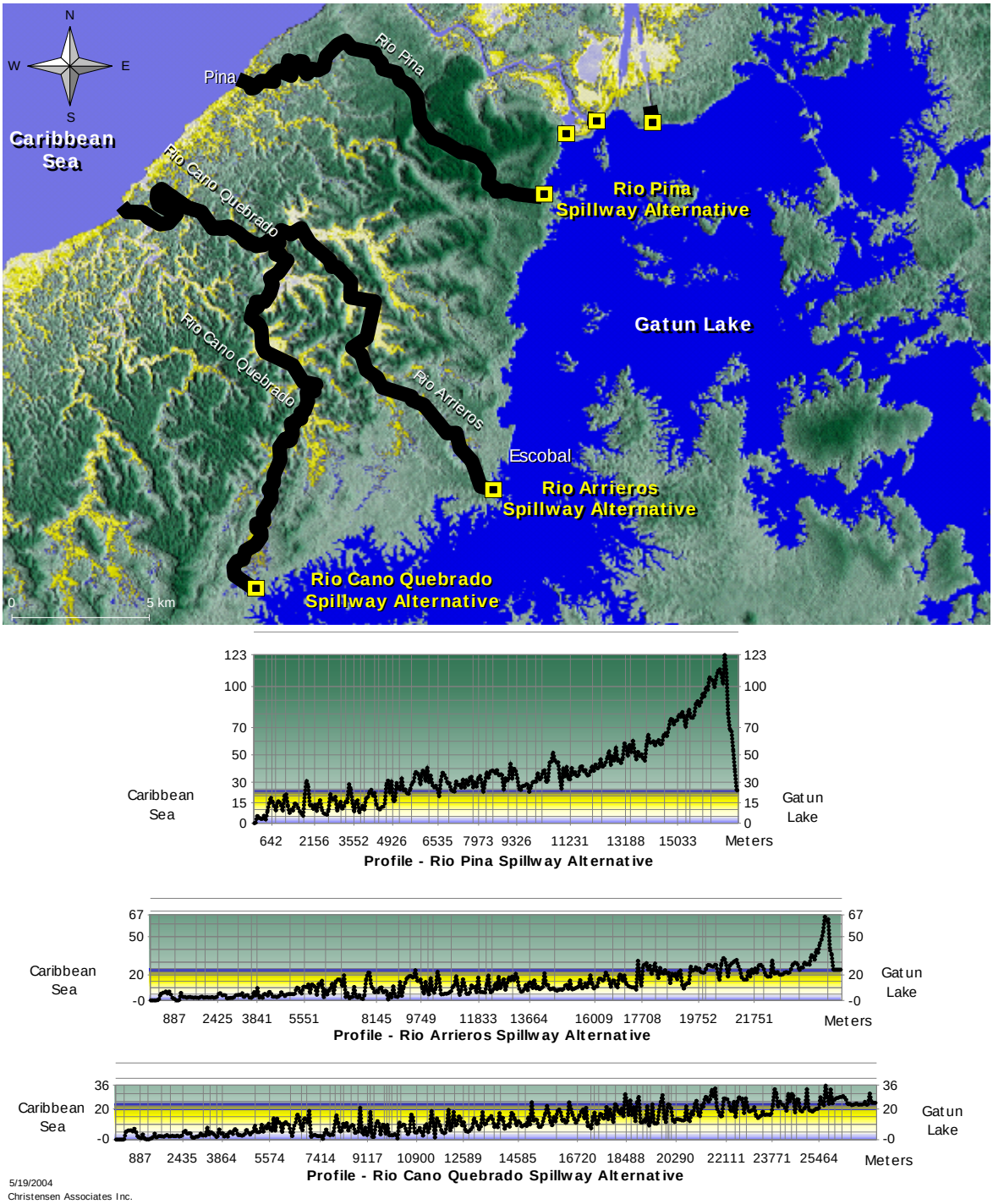
Rio Arrieros

The Rio Arrieros option would require excavation of a deep cut (up to 50 m (164 ft) high) through the western rim of Gatun Lake at a point near La Valerosa (about 1.5 km (1 mi) south of Escobal) into the Rio Arrieros watershed. Rio Arrieros is a tributary to Rio Caño Quebrado which flows into the Caribbean Sea. The total length of waterway required is 24.9 km (15.5 mi) of which about 10% would require considerable excavation to grade. Again, the spillway design would have to incorporate a bridge across the top of the structure or along the discharge channel. Due to the substantial depth of excavation required at this site, construction of a bridge would be problematic and expensive.

Rio Caño Quebrado

The Rio Caño Quebrado option would require excavation of a cut (up to 10 meters high through the western rim of Gatun Lake at Caño Canizales (about 2.5 km (1.5 mi) north of Cuipo) into the Rio Caño Quebrado watershed. The total length of the waterway from Gatun Lake to the Caribbean Sea would be about 27.5 km (17 mi). The spillway design would have to incorporate a bridge across the top of the structure or along the discharge channel. This site is preferable to the other sites along the western arm of the lake since the depth of excavation required at this site is less. However, the length of excavation required for the discharge channel would be cost prohibitive.

Figure 3-9: Western Spillway Sites



Comparison of Options

The main criteria use to compare options are as follows:

- Topography
- Hydrologic Impacts and Issues
- Environmental Impacts and Issues
- Geologic and Geotechnical Issues

Topography

Key topographic data for the alternative spillway sites are compared in Table 3-9. The length data show that the Gatun sites have an advantage of shorter length that implies less cost for excavation and lining of spillway discharge channel. The data for the length and maximum depth of excavation needed to exit Gatun Lake also show that the sites at Gatun Dam are more compact, and therefore potentially more cost-effective.

None of the three options along the western arm of the lake are attractive primarily due to unfavorable topography. The amount of excavation required to divert water into these small watersheds and the long length of discharge channel needed to reach the Caribbean Sea would make these options costly relative to other options. The costs of increasing the hydraulic capacity of existing stream channels and controlling erosion could also be considerable. In addition, the concept of discharging peak outflow of a 333,850 Ha (1289 sq. mi.) Canal watershed down tiny stream channels that presently drain small watersheds immediately raises technical feasibility and environmental impact issues.

The proposed configuration of the proposed spillway structure at Canoa was similar to the existing Gatun Spillway, however, it would also require lengthy approach and discharge channels due to the topography. The long approach channel to the weir would also impose unfavorable hydraulic flow conditions at the spillway crest, and the high velocity flows would require extensive channel linings in both the approach and exit channels.

Accordingly, all of these sites (Western sites, Canoa) were ranked below those at Gatun Dam.

Table 3-9: Comparison of Key Topography Data

REGION	SITE	LENGTH (KM)	APPROX LENGTH OF EXCAVATION (KM)	APPROX MAXIMUM EXCAVATION DEPTH (M)
Gatun				
	West Abutment	0.4	0.2	20
	Spillway Hill	0.5	0.3	15
	East Abutment	1.8	0.3	15
Sabanita (Nuevo Colon)	Canoa	2.6	0.7	75
West Gatun				
	Rio Piña	17.2	12.3	100
	Rio Arrieros	24.9	2.9	50
	Rio Caño Quebrado	27.5	2.0	10

Hydrologic Impacts and Issues

As stated previously, one of the main concerns for any potential spillway site is the hydraulic capacity of the receiving stream. Many of the potential sites are located along the hydrologic ridge of the Gatun Lake watershed so that the receiving streams would be located in the extreme headwaters of their respective watersheds. The drainage areas for these areas would be very small compared with the Gatun Lake watershed, and the potential for significant erosion and additional flooding would be a near certainty without significant excavation and reworking of the existing natural channels. In fact, most of these channels would likely have to be lined with concrete or rock.

Based on the requirements for total discharge spillway capacity discussed earlier, the range of possible discharges for the additional spillway would likely range between 2,270 – 7,300 cms (80,000 – 258,000 cfs) (Table 3-1).

Based on previous work completed by M&N for ACP, measurements for a localized 100-yr, 24-hr event at Los Cañones showed that for this natural channel, the existing hydraulic capacity was approximately 0.019 cms/ha (179 cfs/sq. mi) of drainage area. The corresponding natural channel bottom width was also determined to be 0.003 m/ha (2.76 ft/sq. mi.) of drainage area.

Using these same ratios one could then determine a preliminary estimate of the receiving capacity and channel size for the receiving streams for the potential sites. The drainage areas for each of the potential sites are shown in Figure 3-10 to Figure 3-13. Table 3-10 summarizes the resulting calculations.

Figure 3-10: Rio Chagres Drainage Area below Gatun Dam

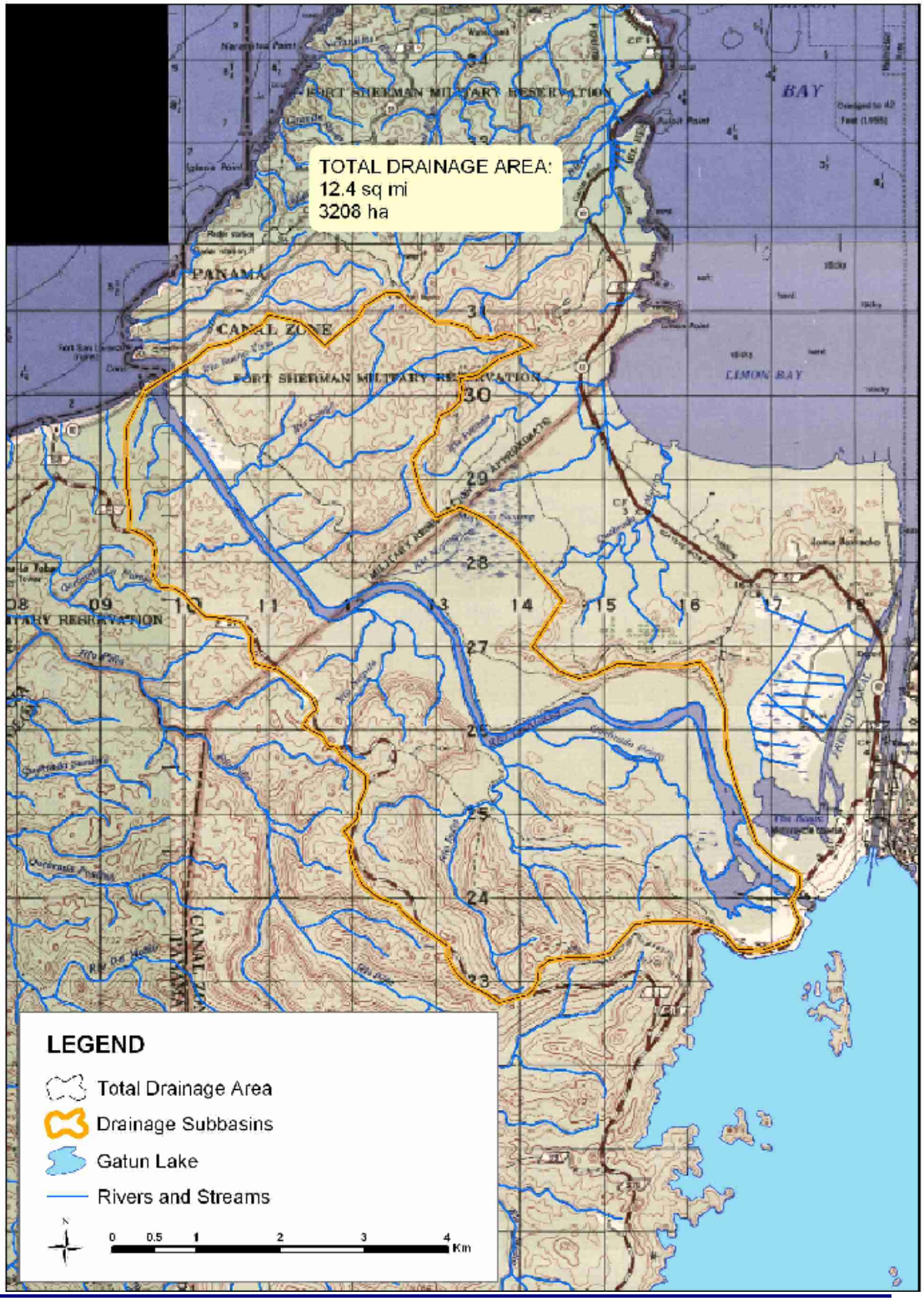


Figure 3-11: Drainage Area for Canoa Spillway Site

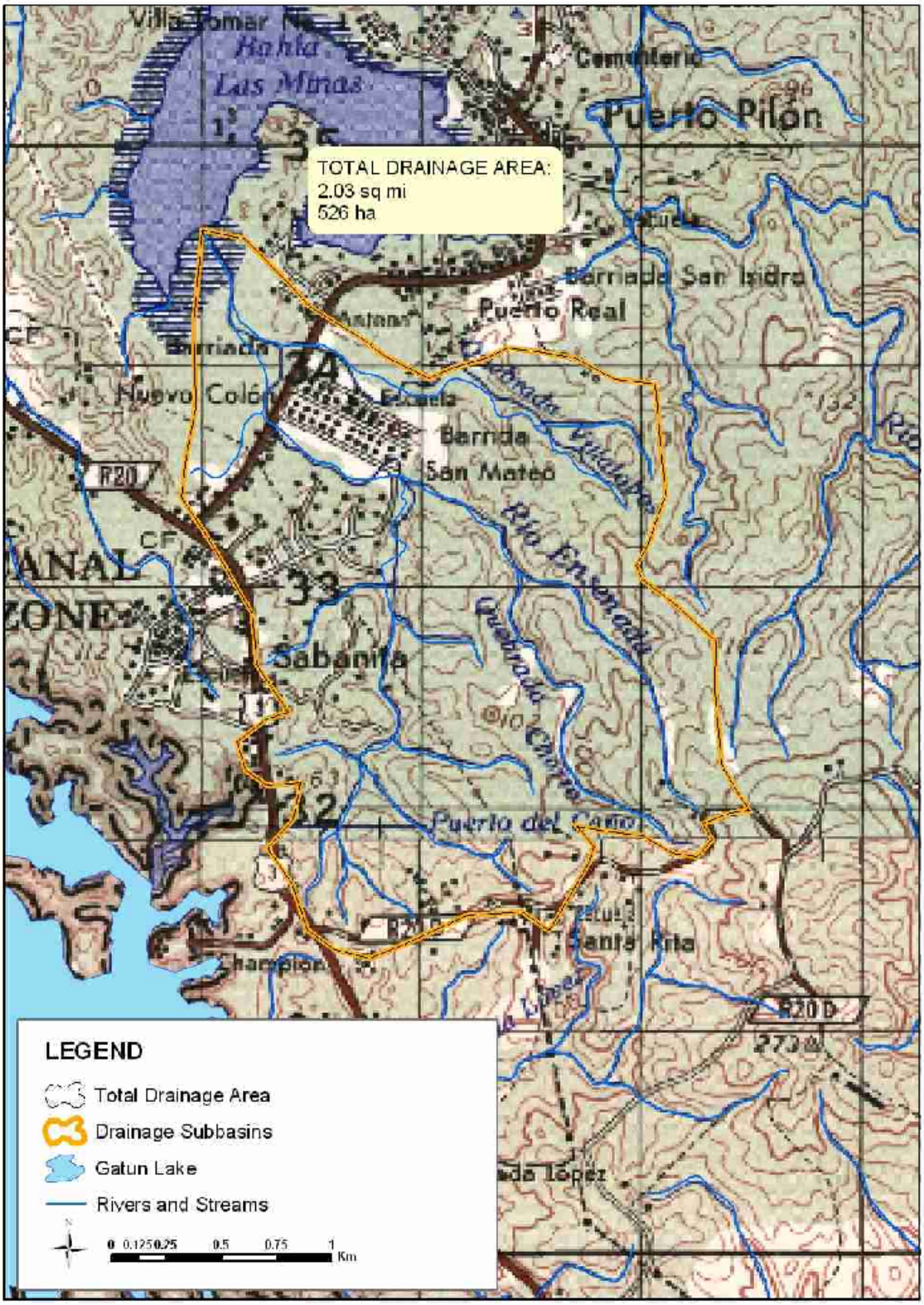


Figure 3-12: Area for Rio Piña Spillway Site

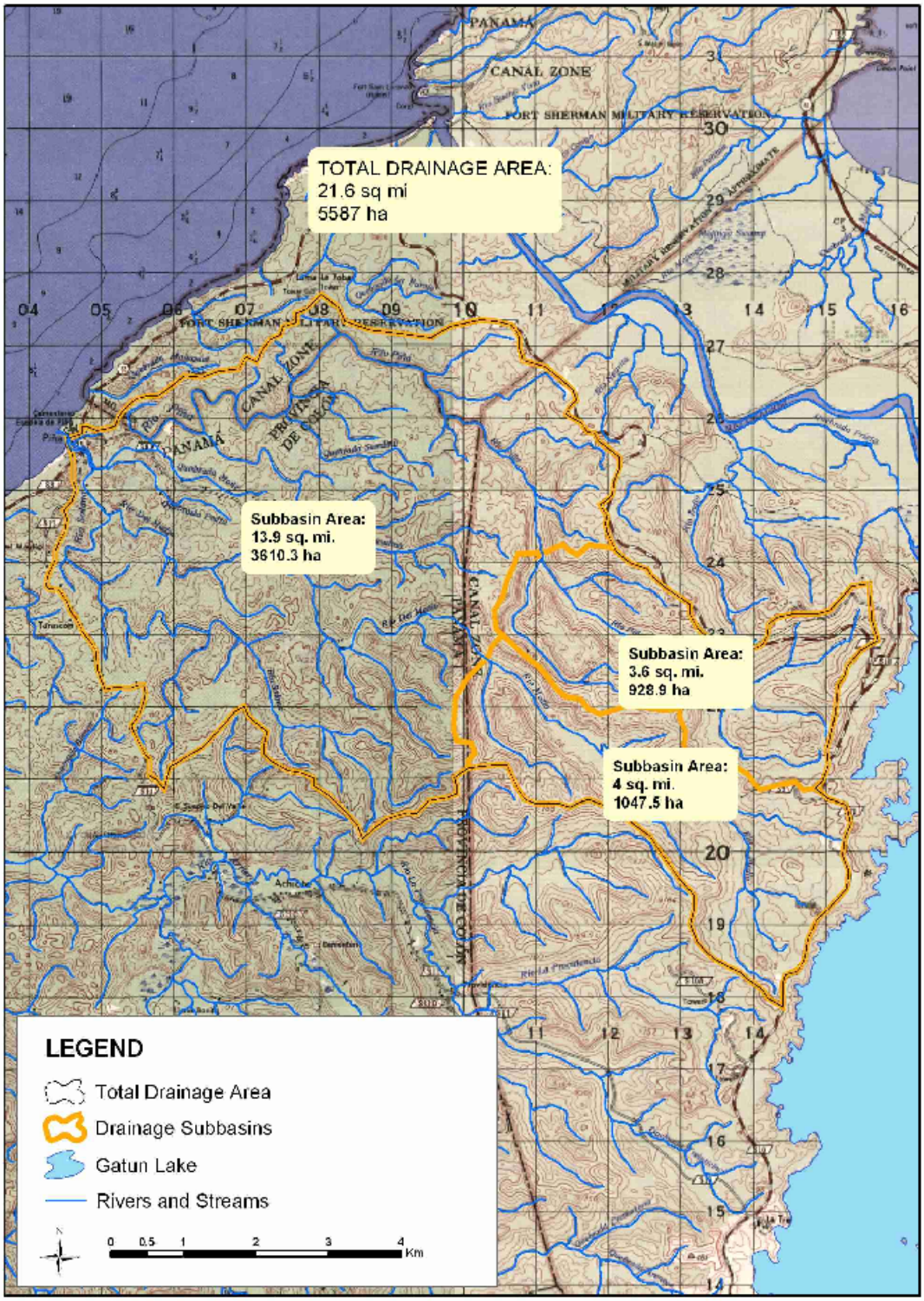


Figure 3-13: Drainage Area for Rio Arrieros/Rio Providencia/Rio Caño Quebrado Spillway Sites

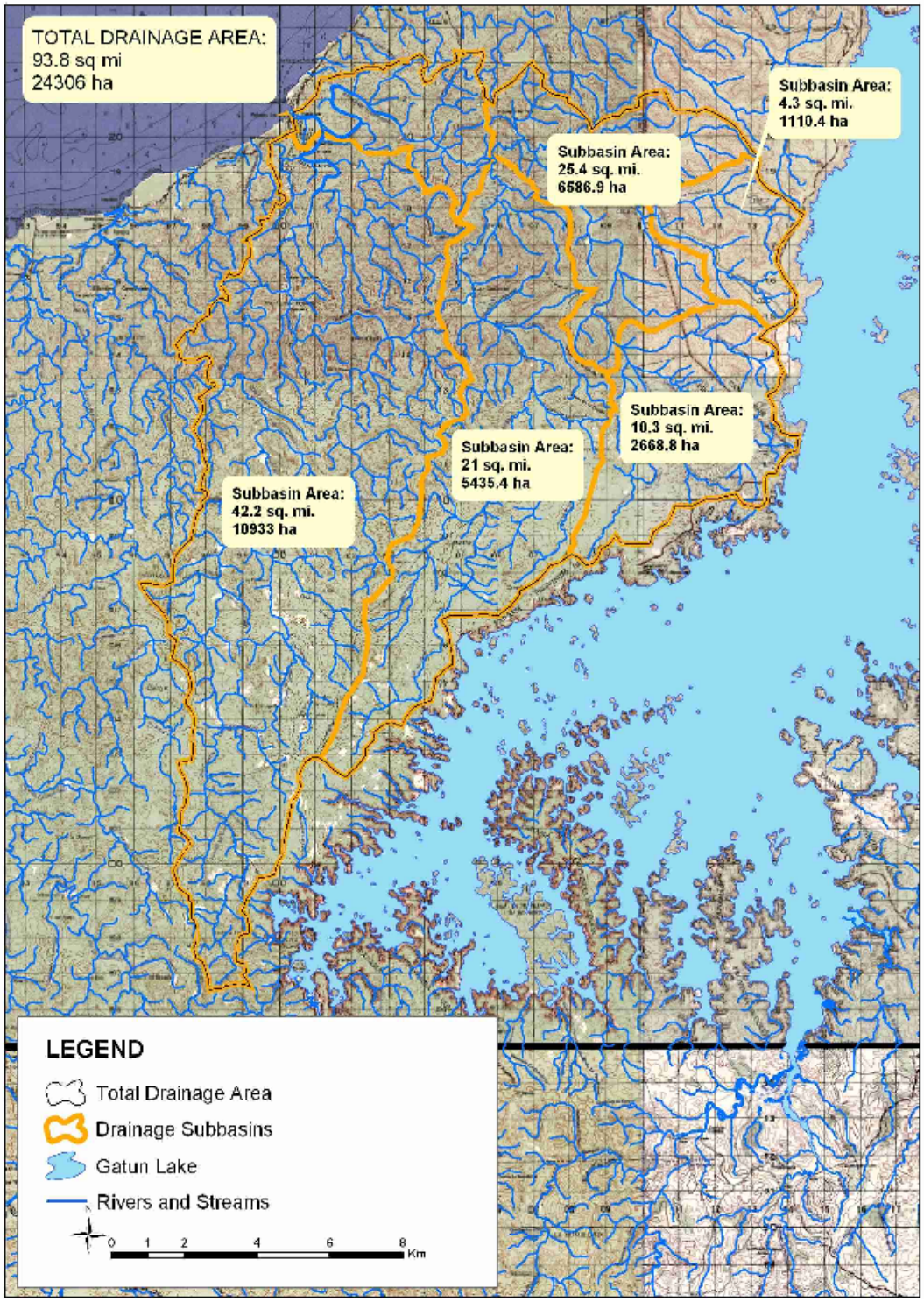


Table 3-10: Comparison of Preliminary Estimates of Hydraulic Receiving Capacity of Potential Spillway Sites

Metric Units

REGION	SITE	DRAINAGE AREA (HA)	APPROX. NATURAL CHANNEL CAPACITY (CMS)	APPROX. NATURAL CHANNEL BOTTOM WIDTH (M)
Gatun				
	West Abutment	337,000	6600	1100
	Spillway Hill	337,000	6600	1100
	East Abutment	337,000	6600	1000
Sabanita	Canoa	520	10.1	1.7
West Gatun				
	Rio Piña	5600	110	19
	Rio Arrieros/Rio Providencia	5440/10,400	107/203	18/34
	Rio Caño Quebrado	24,300	476	80

English Units

REGION	SITE	DRAINAGE AREA (SQ. MI.)	APPROX. NATURAL CHANNEL CAPACITY (CFS)	APPROX. NATURAL CHANNEL BOTTOM WIDTH (FT)
Gatun				
	West Abutment	1301 (1289+12.4)	233,000	3600
	Spillway Hill	1301	233,000	3600
	East Abutment	1301	233,000	3600
Sabanita	Canoa	2	358	5.5
West Gatun				
	Rio Piña	21.6	3,870	60
	Rio Arrieros/Rio Providencia	21.0/40.0	3,760/7,160	58/110
	Rio Caño Quebrado	93.8	16,800	260

As can be seen from the above table, any of the sites other than the ones at the current Gatun Dam will be substantially overwhelmed by the design flows needed for the additional spillway. It is almost certain that any of these other sites will require significant excavation and lining of the receiving channels over many miles to carry the required discharge to the ocean. From a hydrologic perspective, any of the sites that utilize the Rio Chagres as the receiving stream would be preferred. These sites would likely require very little modification for use. In fact, the major concern with these sites will be to verify that the discharge flows stay within the Rio Chagres and do not reach elevations which would allow flood flows to enter either the French Excavation or Limon Bay (which would likely cause strong cross currents within the navigation approach channels at either location). The construction of protective levees will be required at Mindi Dike and should be evaluated for the Mojinga Swamp for any alternatives that discharge to the lower Rio Chagres.

Environmental Impacts and Issues

In addition to the topographic and hydrologic concerns with the construction of a proposed spillway, there are also environmental concerns including socio-economic concerns as well as impacts on flora and fauna.

Socio-Economic Considerations

The socio-economic considerations for a proposed spillway mainly consist of the population centers that would be impacted by the construction. Also, those communities that live along the receiving channel would also be impacted and in some cases may be forced to move. The 2000 Census Data was used to qualitatively estimate the number of people that would be impacted by the construction of a spillway at each of the proposed sites. Figures 3-20 – 3-23 show the structures (mainly homes) identified during the 2000 census. It is difficult to see the homes at this scale, but the sites at Gatun are again preferable as there are very few (if any) structures identified along the Rio Chagres that would be impacted. The Canoa site (recommended in the 1999 USACE report) is by far the worst, with many people having to be relocated if this site were to be utilized. For the Rio Piña site, it is probable that people at the La Union village as well as some people at Piña would have to be relocated. For potential sites along Rio Arrieros and Rio Provdencia, a substantial number of people at the villages of Escobal, Providencia and Achiote would likely have to be relocated as well as possibly some people at the villages along the mouth of the Rio Caño Quebrado. For potential sites along Rio Caño Quebrado, people at the villages of Cuipo, Caño Quebrado, and the villages at the mouth of the river would likely be affected. In conclusion, from a socio-economic standpoint, the sites along Gatun Dam are preferred to the other sites at Canoa and the western arm of the lake.

Impacts on Existing Flora and Fauna

Alternative spillway locations were also evaluated based on the potential impacts to flora and fauna. As previously discussed, the Canoa and the alternative site east of Gatun Dam are less likely have significant impacts to flora and fauna due to the distrurbance that has occurred in this area by development. The four alternative sites located west of Gatun (i.e., Canoa, Rio Piña, Rio Arrieros/Rio Providencia, and Rio Caño Quebrado are expected to have more extensive impacts than the alternatives spillway sites located along Gatun Dam due to the considerable lengths of vegetation clearance required and potential loss of habitat are likely to occur as a result of the excavation required to build channels to discharge the water to the Caribbean. Based on the screening conducted the impacts associated with the preferred sites are discussed as follows:

Proposed Spillway Location 1 - West of the Existing Gatun Spillway

The proposed spillway west of the Gatun Spillway has secondary rainforest, forested wetlands, and marsh. Constructions of the spillway in this area will likely result in destruction of marsh and forested wetland and temporary and permanent adverse impacts to the ecological environment. The amount of wetland destruction will depend on the final footprint of the project but will be considerably less than the above-referenced alternative spillway sites due to the infrastructure present associated with the Gatun Dam (i.e., existing road and staging areas). Wetland areas provide habitat for a diverse array of animals, in particular, the marshes provide foraging habitat for a number of bird species. Additionally, species that utilize riparian areas including the green iguana (*Iguana verde*), the capybara (*Hydrochaeris hydrochaeris*), and the agouti (*Agouti paca)* may be impacted by construction activities on or near the banks of the Chagres River. These species may be able to relocate to undisturbed riparian areas further downstream. If a new spillway is constructed in this location, a berm may need to be constructed to keep waters discharged into the Chagres River from entering the French Canal and ultimately entering Limon Bay. Construction of this berm will likely destroy or adversely modify the hydrologic regime of secondary rainforest or other forest types located on the Chagres River floodplain, as well as impact previously drained wetland areas associated with former military training grounds adjacent to the French Canal.

Proposed Spillway Location 2 - East of the Existing Gatun Spillway

Lands to the east of the existing Gatun spillway have experienced a greater amount of human disturbance than the area west of the spillway. This area consists of wetlands that have been drained, cleared, and filled to allow access to Gatun dam and locks and construction of associated infrastructure. The ecological value of this area is low due to the amount of historical disturbance and the continuing human activity in this area.

Construction of a new spillway in either location is likely to have temporary and permanent adverse impacts on aquatic species. Construction of a new spillway and dredging and/or reconfiguration of the Chagres River channel to handle the discharge are likely to cause increased turbidity throughout this portion of the Chagres River during construction. Marked increases in turbidity can adversely impact aquatic species by reducing their ability to forage or evade predators. Also, increased sediment deposition can result in loss of habitat for certain species, such as freshwater crustaceans. Morphological alteration of the Chagres River channel by mechanical means or through increased flows during flood discharges may result in loss of habitat for certain aquatic species. More mobile species may be able to relocate to undisturbed areas downstream. Controlling turbidity will require the strict implementation of environmental management systems to ensure that turbidity is controlled within acceptable limits.

Figure 3-14: Population Impacts for Gatun Spillway Sites

Figure 3-15: Population Impacts for Canoa Spillway Site

Figure 3-16: Population Impacts for Rio Piña Spillway Site

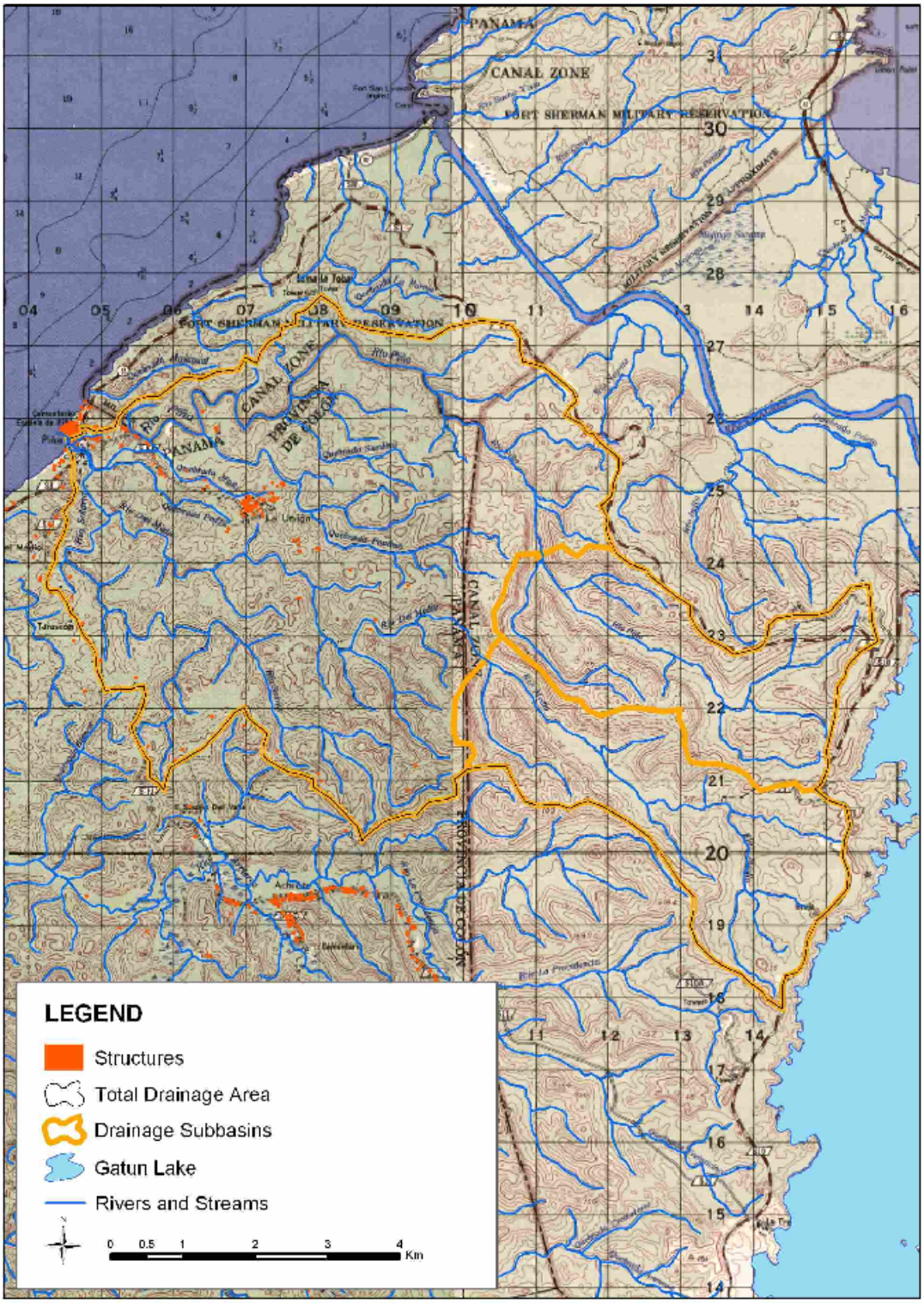
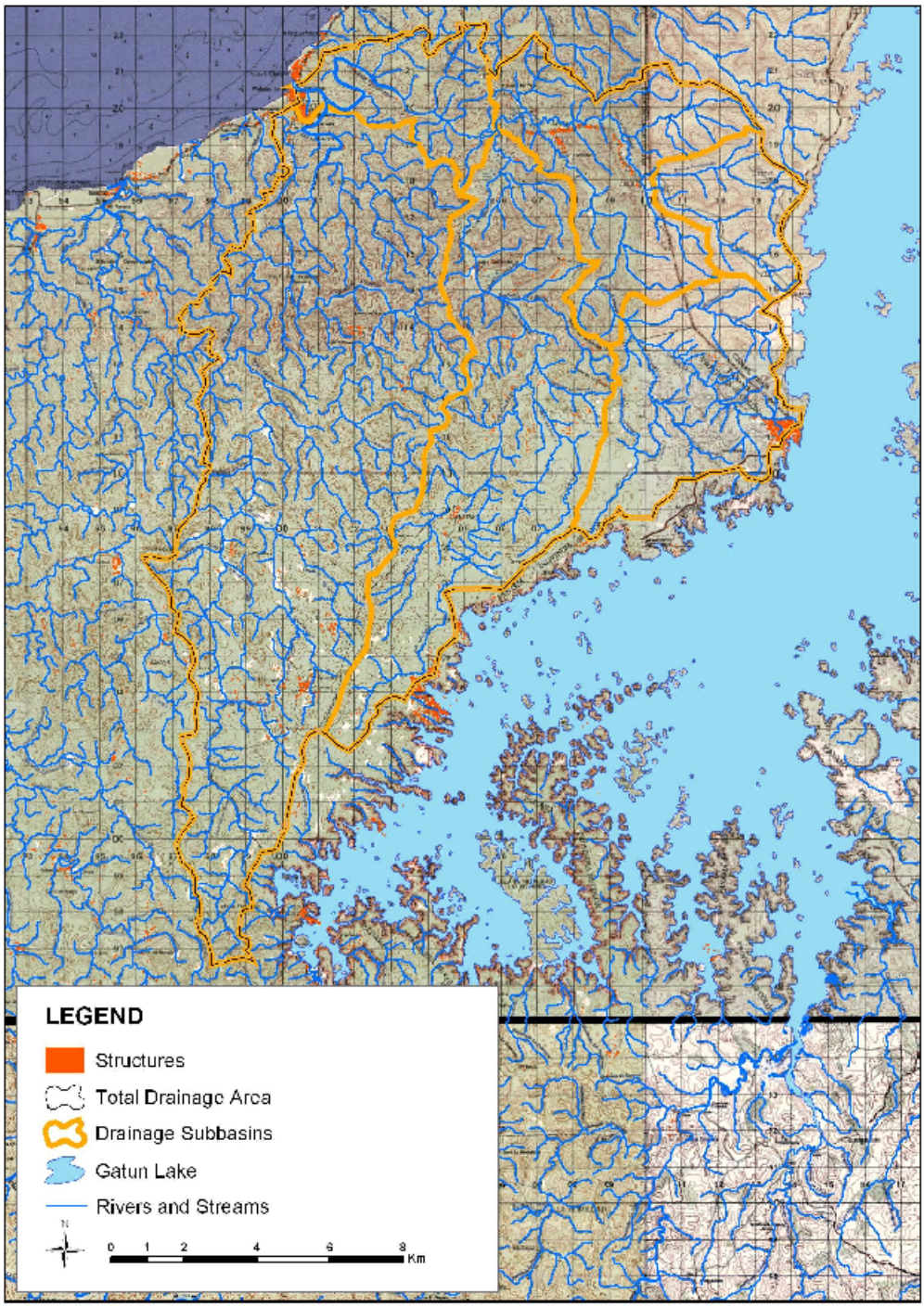


Figure 3-17: Population Impacts for for Rio Arrieros/Rio Providencia/Rio Caño Quebrado Spillway Sites



Geologic and Geotechnical Issues

Existing information relevant to the potential geologic and geotechnical conditions for the alternative spillway sites were compared. The existing information used for this evaluation includes S.M. Jones, 1947; R.H. and J.L. Stewart, 1980; and Franceschi, 1992. All three options appear to be underlain by rocks of the Gatun Formation. The potential is greater for the presence of Atlantic Muck at or near the Gatun Dam and Canoa alternative sites due to the relative proximity to the coast compared to the West Gatun Dam alternatives. A higher concentration of northwest-trending faults is shown near the West Gatun Dam alternatives. If the ground beneath the West Gatun Dam alternatives is more highly fractured related to more faulting, foundation costs would potentially be greater for the West Gatun Dam alternatives. However, the potential presence of Atlantic Muck within the foundation of the Gatun Dam or Canoa alternatives would also result in additional foundation costs. Therefore, given the information available, each of the alternatives rank equally from a geologic and geotechnical perspective.

Conclusion

The reconnaissance of alternative sites for additional spillway capacity has identified seven potential sites in three areas. The sites adjacent to Gatun Dam appear to be the most favorable from a topographic, hydrologic, environmental, and geologic perspective.

Three sites were identified at Gatun Dam. The west abutment site appears to have the most favorable topography and favorable location relative to existing and proposed additional navigational lock facilities. The spillway design will have to incorporate a bridge across the top of the structure (preferable) or across the discharge channel. Of all the sites, the west abutment site also appears be the least problematic and most cost effective to incorporate this type of structure. Subsurface conditions will also have a significant affect on feasibility and cost and therefore should be investigated before finalizing site selection. In particular, the stability of the existing hydraulic fill dam materials will require particular attention during the design and construction process.

Based on these preliminary recommendations, ACP then undertook a series of field studies that were used as the basis for the final location evaluation and the conceptual designs presented later in this report.

3.3.3 Siting Recommendations for the New Spillway

The principal recommendations from the spillway location study are:

- The site at the west abutment of Gatun Dam appears to be the most attractive location from the standpoint of technical feasibility, potential construction cost and minimizing potential environmental impacts.
- In addition to the new potential spillway, various options should be investigated to increase the existing spillway's capacity. These options include modifications to the "upstream" face of the piers to help streamline flows and increase the effective length of the spillway weir.
- In the event that development of a spillway at the proposed site proves to be infeasible, one of the alternate sites available could be given further consideration. However, since the site at the west abutment appears to be far superior to other sites, it should be given first priority.
- The spillway design will have to incorporate a bridge across the top of the structure or at a point that crosses the discharge channel in order to provide access to the downstream communities and ACP properties in the area.

3.4 Field Studies

3.4.1 Introduction

Details of the field exploration program to investigate the subsurface conditions for the preferred alternate spillway location are presented in Appendix C to this report. Based on the subsurface data collected during the field investigation in conjunction with data collected during previous investigations near the proposed spillway structure, the consultants conducted geotechnical analyses and developed geotechnical recommendations for feasibility-level design of the spillway structure. The results of these analyses and recommendations are presented in Section 3.5.3 of this report. A summary of the subsurface conditions as indicated from data collected during the field investigation is presented below.

Information used in interpreting subsurface conditions and preparing geotechnical recommendations is as follows:

- Cross Sections developed by ACP;
- Electronic topographic map of site and surveyed boring locations provided by ACP

- Preliminary Spillway layouts;
- The results of ACP's field investigation and laboratory testing program;

Numerous studies by ACP and others have addressed the geologic conditions beneath the Gatun Dam, Spillway and Locks and within the vicinity of one of the proposed spillway locations. These studies are summarized and reported by the Panamá Canal Authority Geotechnical Engineering Branch of the Engineering Division (Franceschi and de Puy, 2001; and Franceschi, 1992).

3.4.2 Field Exploration Program

The field studies for the new Spillway project were conducted under the direction of ACP personnel between June 1, 2004 and August 28, 2004. Final laboratory test results were delivered on September 29, 2004.

The primary objectives of the field exploration were to:

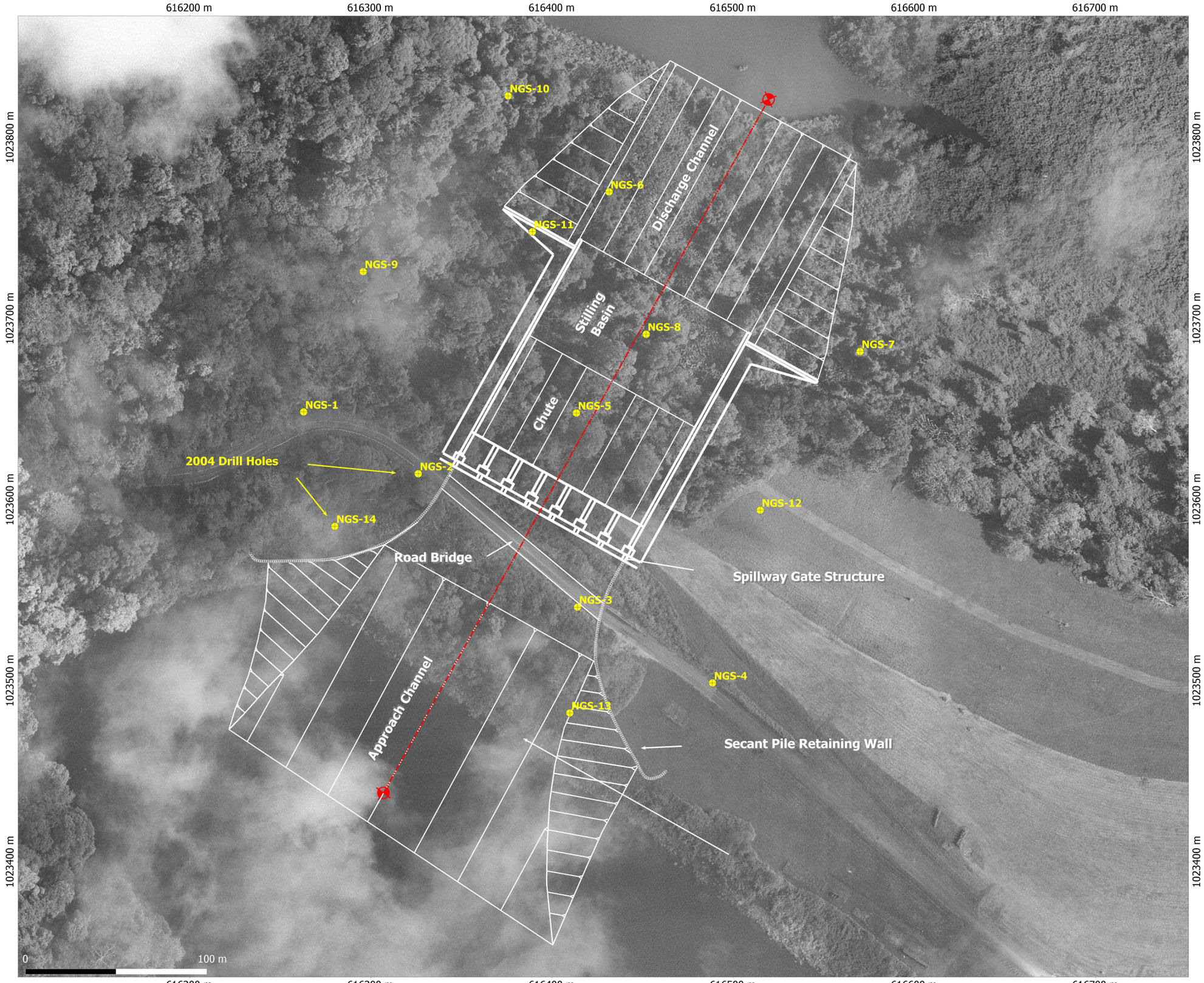
- Evaluate depth to competent rock, groundwater conditions, and characteristics of soil and rock within the footprint of the spillway structure needed for foundation design and stability analysis and design of temporary and permanent cut slopes;
- Evaluate foundation permeability and consider any special foundation treatment, such as grouting; and
- Identify special or unusual subsurface conditions that may affect the construction costs or long-term performance of the spillway.

To achieve these objectives, ACP conducted an exploration program consisting of:

- Geologic mapping within the vicinity and west of the proposed spillway
- Soil drilling and sampling, rock coring, and field and laboratory testing.

ACP conducted geologic mapping of existing soil and rock exposures within the vicinity of the proposed spillway. Information collected consisted of rock lithology, relative distribution of rock types near the spillway, major contacts between rock types and major structures, and the nature and orientation of bedding and structural discontinuities.

Fourteen soil and rock core borings were advanced within the vicinity of the proposed spillway to evaluate subsurface conditions of the area proposed for the preferred alternate spillway. The location of these boreholes is shown on Figure 3-18.



Christensen Associates Inc. 10/27/2004

Figure 3 - 18

Flood Mitigation Program
For Gatun Lake

New Gatun Spillway

Locations of Drillholes
Drilled during 2004 Field Studies

The borings were advanced to a total depth equivalent to or below the preliminary design elevations provided for the spillway to allow sufficient data collection for evaluation of strength and permeability within the foundation of the spillway. The borings were generally spaced about 150 meters or less. This spacing was sufficient for a feasibility level study. However, additional borings will likely be required before the final design of the spillway is completed.

The borings were advanced to refusal with a Failing 1500 drilling rig using 3 7/8-inch inside diameter tricone drilling methods and soil samples were collected with a split spoon sampler using ASTM D 1586 procedures on 5-foot center sampling intervals. Additional relatively undisturbed soil samples were collected using thin-walled Shelby tube samplers. The split-spoon and Shelby tube samples were collected to evaluate the shear strength characteristics of the in-situ soils for slope stability and excavation support design.

Rock core was collected using a double-tube NQ barrel. An ACP geologist logged and photographed the rock core and placed it in appropriately labeled wooden boxes. Point load tests were not conducted due to the very soft nature of the rock core recovered. Boring logs prepared by ACP for the 14 borings as well as cross sections graphically depicting the borings are included in Appendix C.

Perforated PVC tubes were installed in each of the borings following drilling for long-term measurement of groundwater levels at the spillway site. Water level readings measured by ACP are included in Appendix C.

Laboratory testing was conducted on soil and rock core samples as detailed in Table 3-11, below and included in Appendix C. The laboratory tests were performed to characterize physical properties; in-situ shear strength characteristics; and evaluate the compressive strength of the rock core. The laboratory test program included the following tests performed to the indicated standard:

Table 3-11: Laboratory Tests on Soil & Rock Core Samples

LABORATORY TEST	CURRENT VERSION OF STANDARD	NO. OF TESTS PERFORMED
PHYSICAL PROPERTIES		-
Natural Moisture Content	ASTM D 2216	77
Grain Size Analyses	ASTM D 422	71
Atterberg Limits (Liquid and Plastic Limit)	ASTM D 4318	75
Moisture-Density Relationships (Standard Proctor)	ASTM D 698	0

Specific Gravity of Solids	ASTM D 854	32
SHEAR AND COMPRESSIVE STRENGTH		-
Unconsolidated, Undrained Triaxial Tests (UU) a. Shelby Tube Samples	ASTM D 2850	7
Consolidated Undrained Triaxial Tests (CU) with pore pressure measurements Shelby Tube Samples	ASTM D 4767	1
Consolidation Tests	ASTM D 2435	0
Unconfined Compressive Strength	ASTM C39	48

3.4.3 Geologic and Geotechnical Conditions

Field Observations

During the field investigation, four typical soils were observed in split-spoon samples: fill material; Atlantic Muck; residual soil; and saprolite/weathered rock. These soils were categorized based on composition, color and/or consistency. A description of the typical soils identified based on visual classification is provided in the following paragraphs and general distribution of these soils is shown on Figure 3-19**.**

Fill material was observed to occur at the ground surface in borings NSG-1, NSG-2, NSG-4, NSG-6 through NSG-9, and NSG-11 through NSG-14. This fill material consists of a heterogeneous mix of soil types generally consisting of brown, gray yellow and locally red, dry to locally wet, nonplastic to high plasticity:

- Silt, Silty Sand and Sand, with 12-30% clay;
- Silt and Clay, with 12-30% sand; and/or
- Gravel, with 12-30% silt and sand.

Gravel in the fill material was noted to typically range in grain size from less than 3 mm to 4 cm, with a maximum grain size of 15 cm recorded in NSG-12. The gravel generally consists of rock fragments of sandstone, basalt and andesite. This unit ranged from 0 meters thick in NSG -3, NSG-5, and NSG-10 to 10.5 meters thick in NSG-4 and NSG-12. The fill material was typically loose to soft in consistency with N-values ranging from 2 blows per foot (bpf) to locally 20 bpf, with an average value of 5 bpf. Although hydraulic

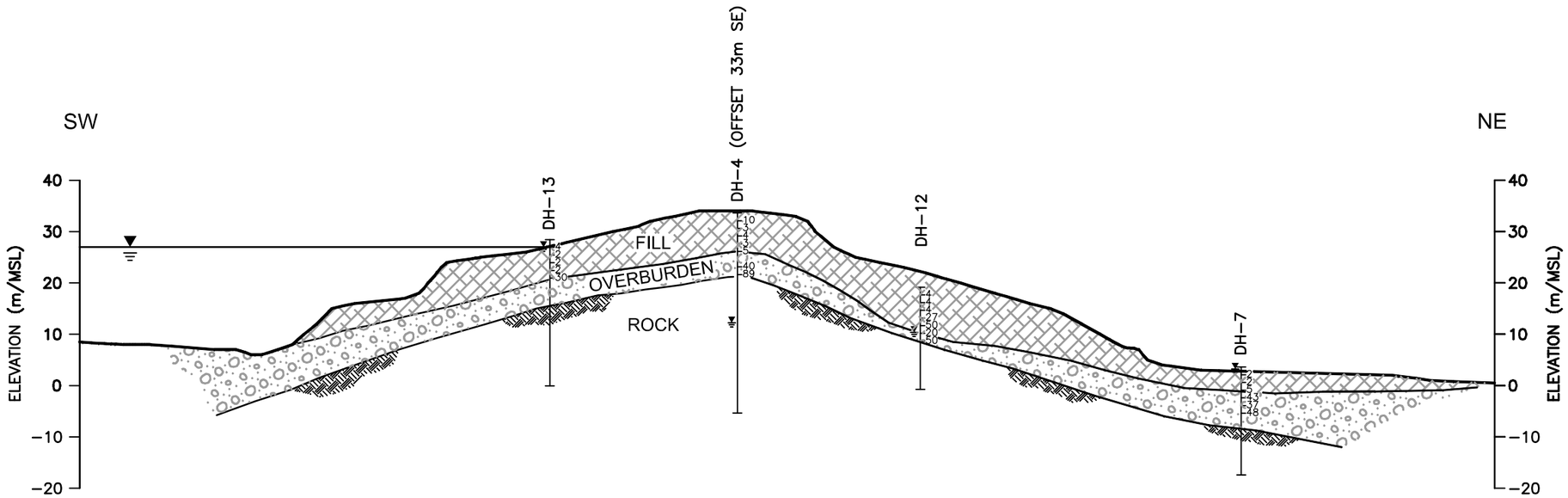
fill was used to construct portions of the Gatun Dam, hydraulic fill was not interpreted by ACP to occur in borings advanced for this field investigation.

Atlantic Muck was observed in two samples collected from NSG-7 between 1.5 to 4.5 meters below ground surface. This material is described as dark brown-black, wet, silty clay with 12% to 30% sand and “swampy sediments”. The Muck is very soft as indicated by N-values of 2 bpf. This material was recorded as Fill on the boring logs provided by ACP.

Residual soil, typically logged as overburden, was observed in 12 of the 14 borings advanced at the site. The residual soil generally consists of a mottled yellow, orange, red brown, and gray, dry to moist, silt and clay with variable amounts of sand and trace amounts of gravel. Fine-grained to medium-grained sand was also noted locally. Residual soils ranged from 0 meters thick in NSG-12 and NSG-13 to 16.5 meters thick in NSG- 6. These soils were typically firm to stiff in consistency with N-values ranging from 4 bpf (where in contact with fill material) to 44 bpf, with an average value of 21 bpf. This material is considered to be the weathering product of underlying Gatun Formation siltstone, as described in the Bedrock Section listed below.

Completely weathered to highly weathered rock was locally sampled in split-spoon samplers. This material consists of brown and gray, fine-grained, saprolitic, siltstone, that when sampled becomes silt and very fine sand with variably 5% to 30% clay. Based on drilling, the completely to highly weathered rock had a variable thickness ranging from less than 0.3 meters in NSG-14 to 8.9 meters in NSG-11. The weathered rock typically has N values ranging from 20 bpf to 89 bpf.

Figure 3-19: Geologic Profile at New Spillway



LEGEND

FILL

OVERBURDEN = RESIDUAL SOILS AND COMPLETELY TO MODERATELY WEATHERED ROCK

50 0 50
1:2000 METERS
VERTICAL SCALE EXAGGERATED 2X

REV	DATE	DES	REVISION DESCRIPTION	CADD	CHK	RVW
△	–	–	–	–		

PROJECT

MN / LAKE GATUN / PANAMA

TITLE

GEOLOGIC PROFILE A - A

Golder Associates
Atlanta, Georgia

PROJECT No.		–	FILE No.	–
DESIGN	–	–	SCALE AS SHOWN	REV. –
CADD	–	–		
CHECK			**FIGURE 2**	
REVIEW				

Drawing file: 0433718-02.dwg Oct 08, 2004 – 3:46pm

Laboratory Data

Laboratory tests were conducted to verify field classification of soils and to better characterize the soils based on their relevant engineering properties to the project. Plasticity, grain size, and natural moisture content determined in the laboratory indicate that fill material and residual soils generally behave in a similar manner.

The laboratory data support field observations in terms of the heterogeneous soil types comprising the fill material. The fill is characterized as clayey and silty sand, sandy elastic silt, and elastic silt of low to high plasticity (PI=8-39). These soils classify variably as SC, SM, ML and MH in the USCS classification system with 14% to greater than 95% of the particles passing the # 200 sieve. The Atlantic Muck encountered in NSG-7 classifies as MH (94% passing #200 sieve) with a PI of 18 and 39.

The residual soils and completely to highly weathered rock are generally characterized as elastic silt of low to high plasticity (PI=6-34). These soils classify as ML and MH in the USCS classification system with 58% to 97% of the particles passing the #200 sieve.

Unconsolidated, undrained (UU) triaxial tests were performed on five relatively undisturbed samples of fill material and two samples of residual soils. The results of these tests indicate in-situ undrained shear strength for the fill typically in the range of 20 to 30 kPa, but higher and lower values were also measured. In the residual soils, the measured undrained shear strength ranged from 4 to 74 kPa. The undrained strengths for the residual soils at the low end of the range are substantially less than expected based on the N-values and suggest that sample disturbance or localized defects in the samples may have affected the results.

Consolidated, undrained (CU) triaxial tests with pore water pressure measurements were performed on one relatively undisturbed sample of residual soils. The CU test results indicate in-situ effective stress strength parameters, c and phi, equal to 12 Kpa and 30 degrees.

3.4.4 Bedrock

Based on geologic mapping and borehole data, bedrock observed across the site is characterized by lithologies of the Gatun Formation. The Gatun Formation within the vicinity of the proposed spillway was observed in outcrop and in rock core and is generally characterized as a light gray to olive gray, fresh to slightly weathered (variably moderately to completely weathered in outcrop), weak (RH1-RH2), soft, indurated to poorly indurated, weakly cemented, massive (bedding not well developed with spacing of >1.5 meters), homogeneous, poorly jointed, fossiliferous, glauconitic, calcareous, clayey siltstone that is generally equigranular with local medium to very coarse grained fossils. Tuffaceous interlayers were observed in borings NSG-6 and NSG-7. These tuffaceous

beds are characterized as light gray to white, fresh to slightly weathered, soft to medium soft, weak, very fine grained to fine grained, and massive. Jointing is well developed in this unit.

Bedding in the siltstone is oriented generally to the northeast, dipping less than 10° to the northwest. Two through-going joint sets were also observed: E-W, 84°N (strike joint); and N20°E, 82°NW.

Top of rock within the vicinity of the proposed spillway ranges from small exposures at ground surface to about 20 meters bgs. Rock core recovery ranged from approximately 40% to 100% and rock quality designation (RQD) ranged from 20% to 100%.

Bedrock observed in the vicinity of the proposed spillway is different from that observed beneath and downstream of the existing spillway. The Gatun Formation was observed in outcrop downstream of the existing spillway and described in Borings GDS-1 through GDS-6, which were advanced through the spillway into bedrock by ACP in 1979. Based on geologic mapping of the downstream outcrop and descriptions presented in the borings logs, the rock beneath and downstream of the existing spillway is characterized as medium light gray to medium gray, fresh to slightly weathered, medium strong (RH2 to RH3), fine-grained to medium-grained, moderately-well cemented sandstone. Bedding is thin to medium bedded and oriented generally N75°E, 6°NW. Two through-going joint sets were also observed: E-W, 81°N and N20°E, 82°NW.

Numerous faults have been interpreted to occur beneath the Gatun Dam and within the vicinity of the preferred alternate spillway location. The precise location, date of last movement, and geologic and geotechnical nature of an inferred fault within the vicinity of the proposed spillway in not known. The extremely fractured rock core and significantly greater depths of weathering observed in NSG-6, NSG-9, NSG-10, and NSG-11 support the inferred occurrence of a fault in the northwest portion of the site. Sufficient detail was not collected during this field investigation, however, to estimate the exact location and orientation of such a feature in this area.

Laboratory Data

Unconfined compressive strength (UCS) tests were conducted on 48 rock core samples collected from each of the borings except NSG-10. The results of these tests indicate UCS values ranging from 272 psi in NSG-3 to 1283 in NSG-11 psi, with an average of about 650 psi for all of the samples. The rock type recorded for each of these samples is clayey siltstone, as described above.

3.4.5 Groundwater

Perforated PVC tubing was installed in each of the 14 boreholes following completion of drilling. A total of five piezometers were installed during the field investigation. Groundwater was measured in all of the piezometers on October 1, 2004 by ACP, as listed in Appendix C. Water levels across the site were highly variable ranging from 21.4 meters below ground surface in NSG-4 along the centerline of the embankment, to 0.41 meters below ground surface in NSG-7 at the downstream toe of the dam.

3.4.6 Feasibility Level Geotechnical Recommendations for New Spillway

The principal recommendations and conclusions with respect to the foundation conditions for the new spillway are summarized in Table 3-12, below.

Table 3-12: Geotechnical Recommendations for New Spillway

Issue	Recommendations	Comments
Max. allowable bearing pressure for spillway	1 MPa (20 ksf) for Sound Rock	Weathered Rock and Overburden are not suitable bearing material for spillway.
Max. allowable bearing pressure for downstream chute	1 MPa (20 ksf) for Sound Rock 0.5 Mpa (10 ksf) for Weathered Rock 150 kPa (3 ksf) for dense Overburden (dense residual soil derived from weathering rock)	Special provisions to accommodate settlement may be needed if chute is founded on Weathered Rock or Overburden
Ultimate base sliding resistance between concrete and Sound Rock	Friction angle = 38 degrees	
Ultimate sliding resistance through Sound Rock	Friction angle = 38 degrees Cohesion = 200 kPa	
Max. permanent slopes	• 8H:1V in existing fill • 2H:1V in Overburden & Weathered Rock, above lake level • 0.5H:1V in Sound Rock, above lake level • 2H:1V in Overburden, Weathered Rock & Sound Rock below lake level	Steeper slopes may be feasible when site specific data becomes available
Permanent seepage control	• Grout curtain extending min.	

measures	of 20 m below approach channel elevation • Drainage curtain extending min. of 20 meters below approach channel elevation	
Protection against piping at interface between structure and earth	• Min. 1 m thick filter composed of concrete sand placed downstream of the drainage curtain, both under the spillway and behind the walls. Filter should tie into and be drained by other spillway drainage systems.	Purpose is to protect against preferential seepage paths that may develop along the interfaces between concrete and fill and lead to erosion of fines and piping.
Issue	Recommendations	Comments
Seismic loading	• Firm ground PGA = 0.41g for ultimate limit state • Firm ground PGA = 0.22g for serviceability/operational state	• Based on 5,000 year return period • Based on 1,000 year return period
Protection against preferential seepage path developing between structure and embankment fill due to liquefaction and sliding of existing fill during earthquake	Min. width of 3 m of compacted backfill between concrete walls and existing fill placed against spillway walls with a slope of 1H:1V or flatter	
Temporary support	• For spillway excavation, sheet pile walls in existing fill and, if needed in Overburden, with tie-backs anchored in Sound Rock	See Figure 1
Ground support on right side of spillway approach channel	• Secant pile wall or • Ground improvement like jet grouting or deep soil mixing in existing fill and maximum excavation slope of 3H:1V	See Figure 3 See Figure 4
Max. temporary slopes	• 8H:1V in existing fill • 2H:1V in Overburden • 1H:1V in Weathered Rock, above lake level • 0.5H:1V in Sound Rock, above lake level	• Steeper slopes may be feasible when site specific data becomes available

	• 2H:1V in Overburden, Weathered Rock & Sound Rock below lake level	

Notes:

Sound Rock, Weathered Rock and Overburden are all part of the Gatun Formation.

3.4.7 Dam Stability Considerations

Gatun Dam is composed of fill placed by hydraulic and dry methods. Borehole information indicates the fill at the new spillway site was placed by dry methods and consists generally of soft, high plasticity silt but is locally heterogeneous. Borehole data indicates the embankment at the new spillway site is up to about 10 m high and is founded on residual soils (overburden) derived from weathering of the Gatun Formation. Embankment Slopes are about 8H:1V or flatter. ACP 's analysis of the stability of the dam considered critical sections with embankment heights up to about 30 meters and slopes similar to those at the new spillway site, which indicated factors of safety of 1.5 or more for static conditions.

At the spillway, where the embankment height is only about 10 meters, factors of safety will be much larger than 1.5.

Under seismic conditions, ACP's analyses suggest some displacement may occur in the embankment but they conclude the displacement will not result in a release of the reservoir. Further testing and refinement of the seismic analyses will be necessary in the next phase of design of the Spillway.

Because of the limitations of the seismic analyses carried out to date, it is recommended that support should be provided where excavations for the spillway result in the temporary or permanent slopes steeper than the existing embankment slopes (8H:1V).

For excavations in fill which will be done in the dry, sheet pile walls with tiebacks anchored into rock are recommended as shown in Figure 3-20. Please note that Section A-A on Figure 3-20 is shown on Figure 3-19.

In order to meet concerns over the stability of the hydraulic fill in the existing dam, the following construction sequence is envisioned:

- Construct temporary support of the existing embankment upstream of the Ogee section and on the right side of the new spillway;
- Excavate behind the temporary support and construct the spillway control structure, chute and retaining wall;

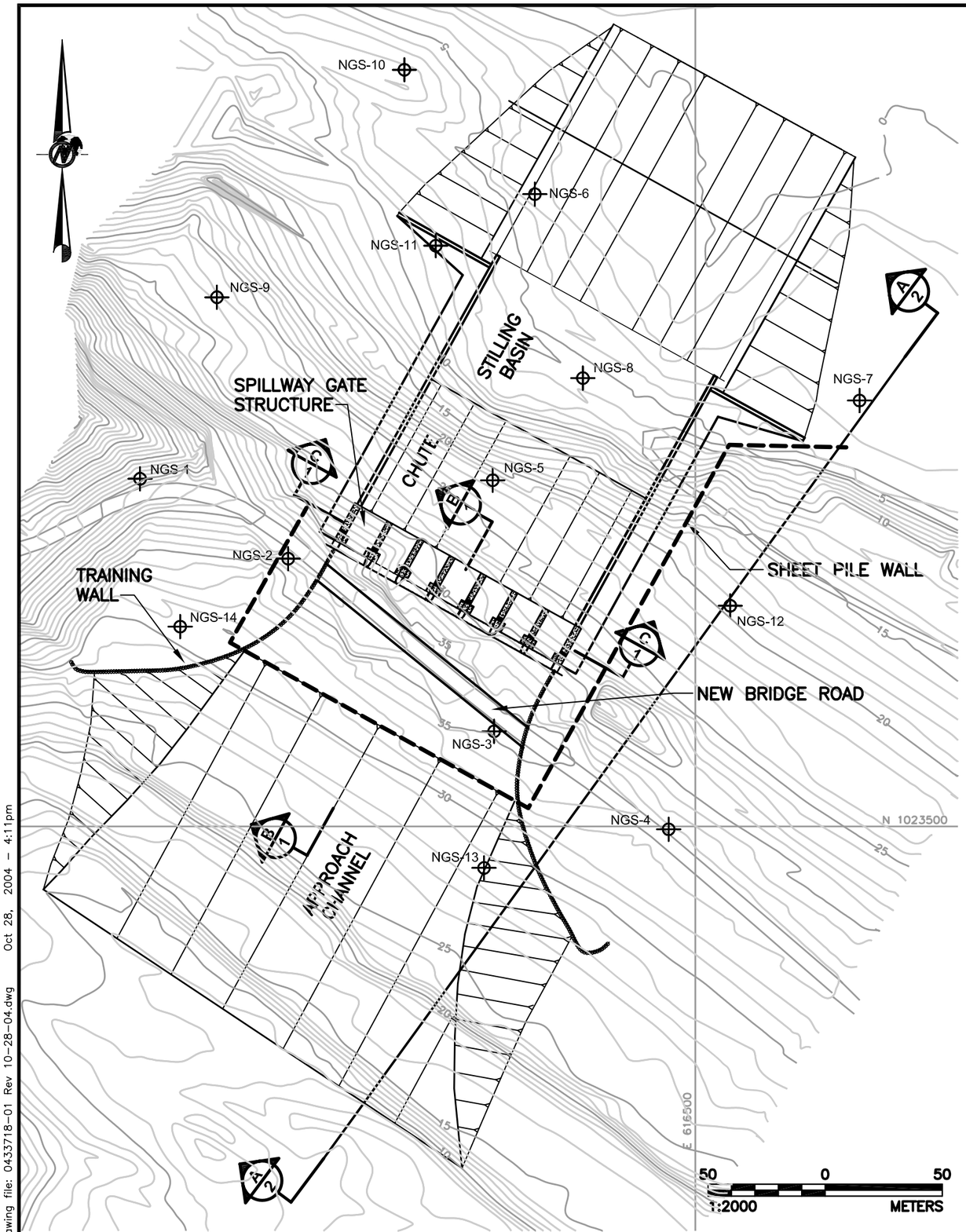
- Construct permanent support / retaining structures partially underwater, for the right side of the spillway approach channel;
- Dredge the excavation of the approach channel, backfill around the new spillway and remove the temporary support.

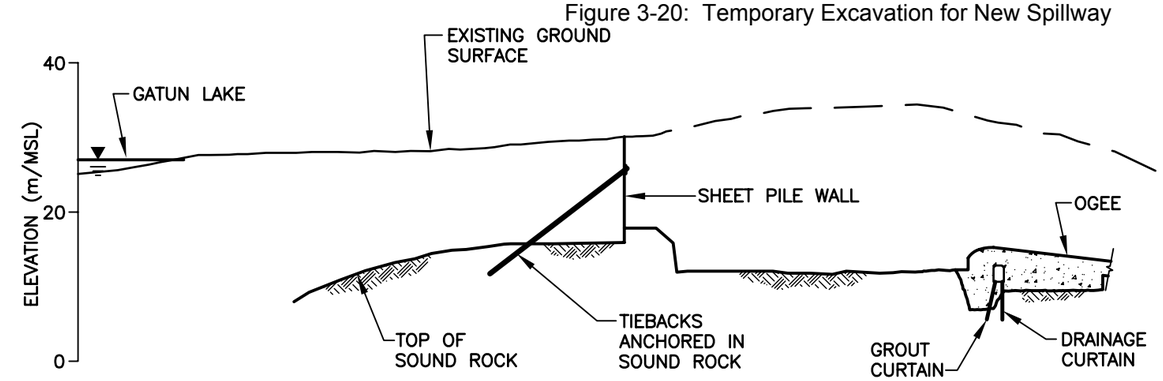
Options for supporting the embankment slope (in step 3 above) may include:

- Gravity wall tremied into place between two lines of sheet piles. This would require underwater struts, or alternatively, bentonite slurry and fewer, or perhaps no struts.
- Gravity wall tremied inside cofferdam cells.
- Secant pile wall socketed into the rock.
- Ground improvement (e.g. deep soil mixing or jet grouting) to improve the shear strength of the fill, allow a 3H:1V slope to be dredged, and thereby eliminate the need for a wall.

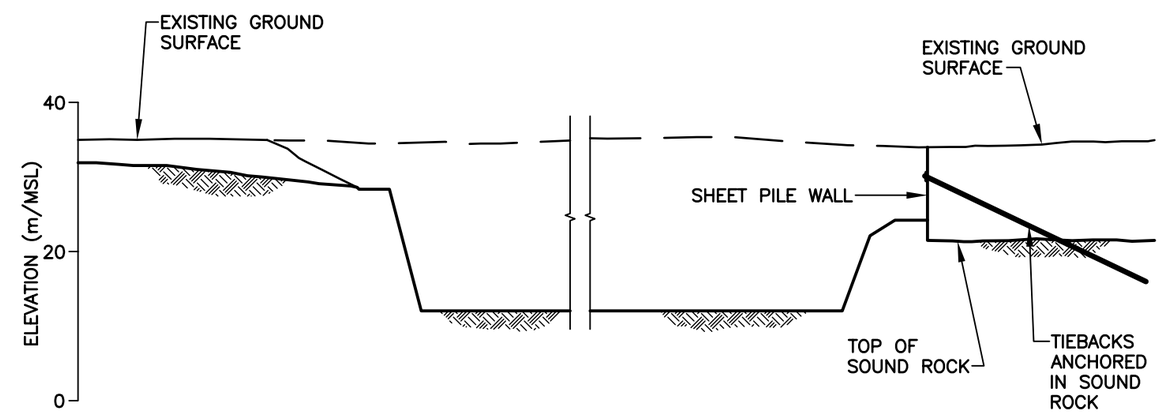
Of these, the most promising are judged to be a secant pile wall (Figure 3-21) and improvement of the fill using just jet grouting or deep soil mixing (Figure 3-22). For the feasibility study, a secant pile wall is recommended, though the further evaluation of the most cost effective, technically acceptable method during the next phase of the design is required.

Figure 3-20: Temporary Excavation for New Spillway





SECTION B - B (LOOKING WEST)



SECTION C - C (LOOKING NORTH)

20 0 20
1:800 METERS

REV	DATE	DES	REVISION DESCRIPTION	CADD	CHK	RVW
△	–	–	–	–		

PROJECT

MN / LAKE GATUN / PANAMA

TITLE

TEMPORARY EXCAVATION FOR SPILLWAY

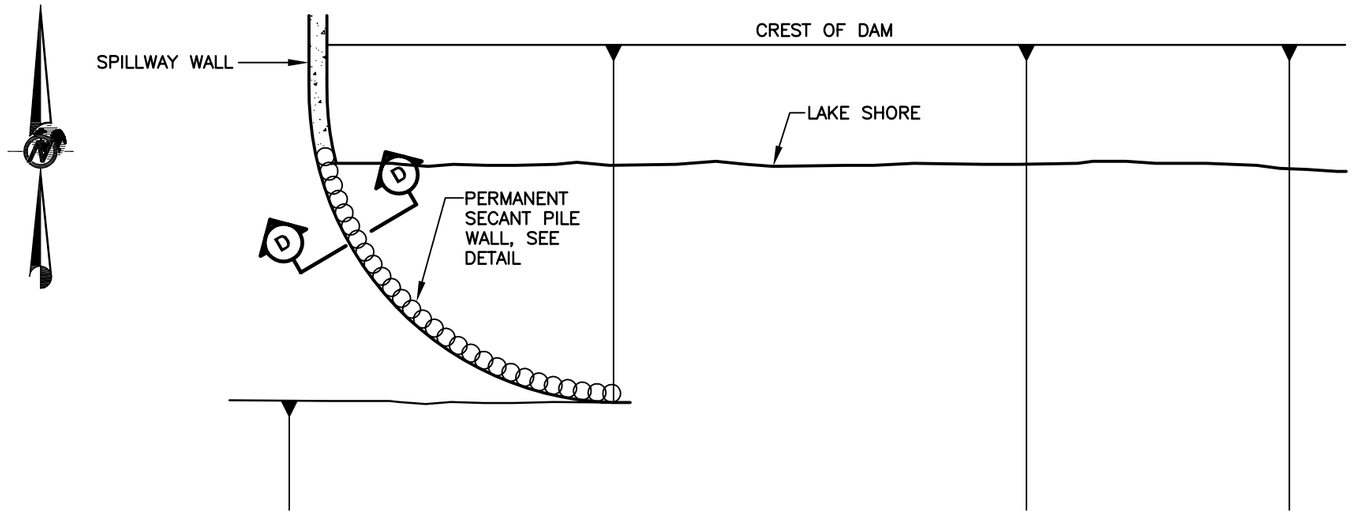
Golder Associates
Atlanta, Georgia

PROJECT No.		043-3718	FILE No.	0433718-001
DESIGN	–	–	SCALE AS SHOWN	REV. –
CADD	RJC	10/04		
CHECK				
REVIEW				

FIGURE 1

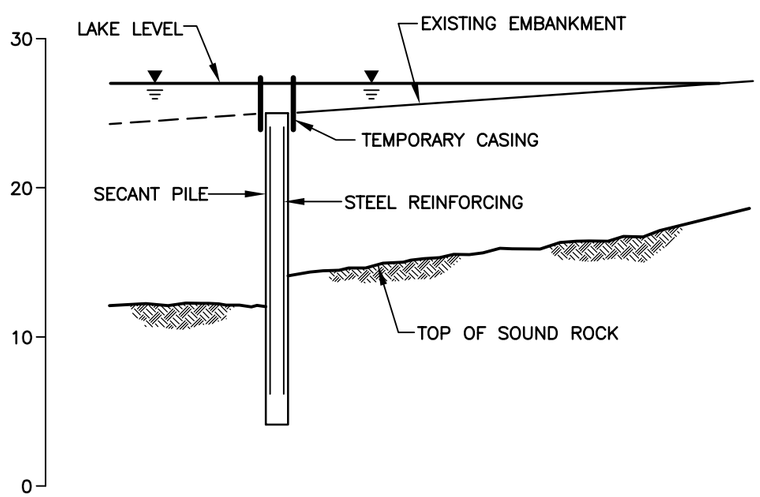
Drawing file: 0433718-01 Rev 10-28-04.dwg Oct 28, 2004 – 4:11pm

Figure 3-21: Secant Pile Wall Option



PLAN OF SECANT PILE WALL

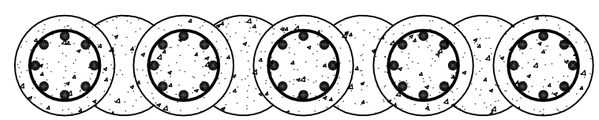
N.T.S.



SECTION D - D

10 0 10

1:400 METERS



SECANT PILE WALL DETAIL

N.T.S.

REV	DATE	DES	REVISION DESCRIPTION	CADD	CHK	RVW
△	–	–	–	–		

PROJECT

MN / LAKE GATUN / PANAMA

TITLE

SECANT PILE WALL OPTION FOR SPILLWAY APPROACH

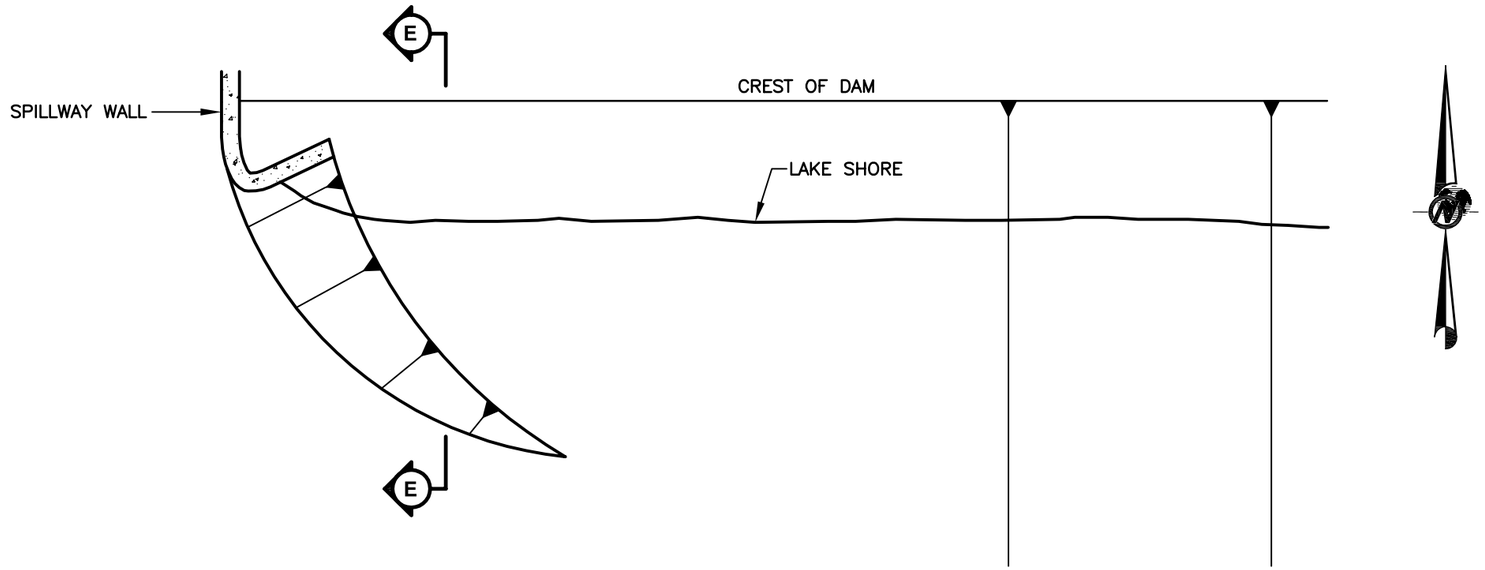
Golder Associates
Atlanta, Georgia

PROJECT No.		043–3718	FILE No.	0433718–004
DESIGN	–	–	SCALE AS SHOWN	REV. –
CADD	RJC	10/04		
CHECK				
REVIEW				

FIGURE 3

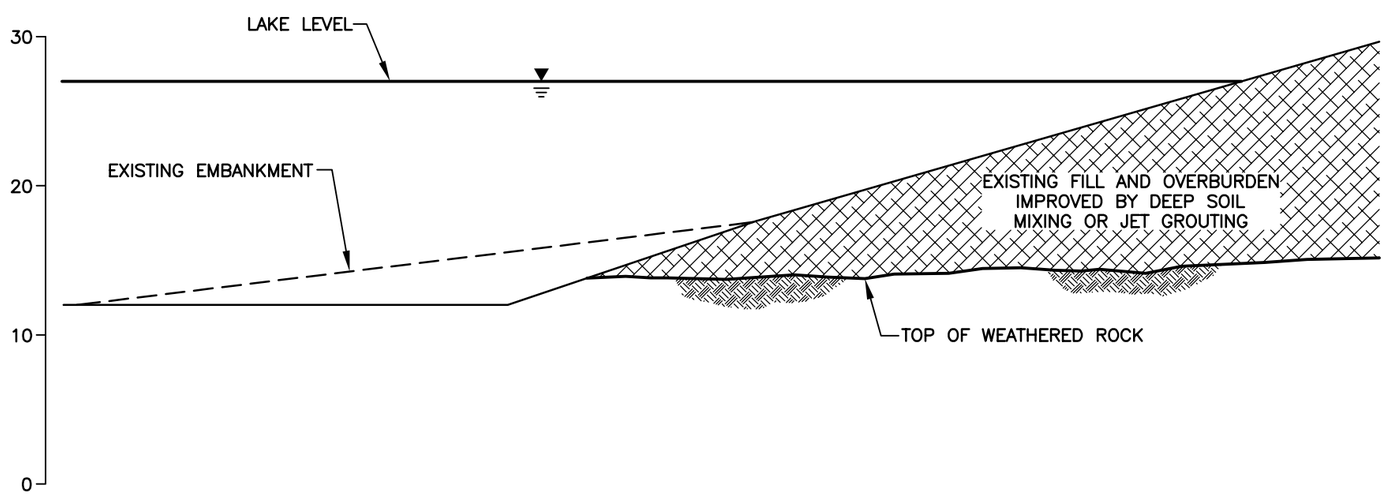
Drawing file: 0433718–03.dwg Oct 08, 2004 – 3:48pm

Figure 3-22: Ground Improvement Option



PLAN OF STEEPENED EMBANKMENT AFTER GROUND IMPROVEMENT

N.T.S.



SECTION E - E

10 0 10

1:400 METERS

REV	DATE	DES	REVISION DESCRIPTION	CADD	CHK	RVW
△	–	–	–	–		

PROJECT

MN / LAKE GATUN / PANAMA

TITLE

GROUND IMPROVEMENT OPTION FOR SPILLWAY APPROACH

Golder Associates
Atlanta, Georgia

PROJECT No.		043–3718	FILE No.	0433718–003
DESIGN	–	–	SCALE AS SHOWN	REV. –
CADD	RJC	10/04		
CHECK				
REVIEW				

FIGURE 4

Drawing file: 0433718–04.dwg Oct 08, 2004 – 3:49pm

3.4.8 Topography

Existing topographic maps and record maps obtained from ACP were used for initial site reconnaissance.

Preliminary studies of spillway locations were carried out using a 10 meter IFSAR DEM coverage of Panama. This airborne radar coverage is of limited accuracy due to dense tree cover, but provides a useful topographic surface dataset for preliminary reconnaissance.

Detailed topographic mapping of the proposed spillway site at Gatun Dam was carried out by ACP as part of the field investigation program. The scope of the survey included the west abutment of Gatun Dam, the area immediately upstream of the dam, and the channel of the Rio Chagres from the downstream toe of the dam to the Caribbean Sea.

Ground elevations were determined by field surveys on land and by bathymetry in the reservoir and in the river. A profile and series of cross sections was also surveyed on the lower Rio Chagres for backwater studies to develop water surface profiles and a tailwater rating curve downstream of the dam.

Topographic maps with a vertical resolution of 1 meter were developed from digital elevation data using a triangulation based surface mapping algorithm.

The map projections and datums used are as follows:

- Horizontal: UTM Zone 17, NAD 27 Metric Coordinates
- Vertical: Precise Level Datum (PLD) in meters.

A copy of the survey maps and digital data is provided in Appendix D.

ACP also provided copies of record drawings and maps showing pre-project and post-construction topography and a set of orthophotos covering the Gatun Dam area.

3.5 Design Criteria

In order to complete the feasibility level design of the new spillway, design criteria were developed for hydraulic and geotechnical issues. These criteria were submitted to and approved by ACP. After discussions were held with ACP, it was determined that different flood criteria should be used for sizing different features for the project. For example, it was agreed that the PMF flood should be used for the design of the new spillway, but it was also agreed that using the PMF to design the protective dikes required downstream would not be cost-effective or a prudent investment. Therefore, a maximum Canal operation flood was developed and it was agreed that it should approximate a 20-50 year flood. The new and existing spillways should be able to

handle this flood event without shutdown of the Canal locks. ACP staff stated that the December 2000 storm was approximately a 15 year flood.

To develop the Canal operational flood (later determined to be the 1/3 PMF – see next two pages), it was decided to use the December 2000 storm as a basis. Daily discharge data was available from ACP for the storm and showed that it was a six day event (just as the PMF) and that the peak discharge occurred on Day 4 (just as the PMF) at approximately 3,400 cms (120,000 cfs). To determine how the peak discharge should be modified to approximate a 20-50 year flood, the 1991 report by Carlos Vargas was reviewed and a table was found which had estimates of 24-hr peak discharges for various return period storms (see Table 3-13). As can be seen from the table, the measured peak discharge during the Dec 2000 flood would fall in-between a 10-20 year flood for the Log-Pearson III estimate using the method of moments.

Table 3-13: Flood Frequency Analysis (taken from Vargas, 1991)

-- FLOOD FREQUENCY ANALYSIS --
GATUN WATERSHED (INCLUDING MADDEN)

	GUMBEL I		LOGNORMAL		THREE PARAMETER LOGNORMAL		LOG PEARSON III MAX. LIKELIHOOD		LOG PEARSON III MOMENTS	
RETURN PERIOD	FLOOD EST.	ST.ERROR PERCENT	FLOOD EST.	ST.ERROR PERCENT	FLOOD EST.	ST.ERROR PERCENT	FLOOD EST.	ST.ERROR PERCENT	FLOOD EST.	ST.ERROR PERCENT
1.005	6130.0		13100.0		17100.0		17200.0		17100.0	
1.050	19500.0		21700.0		23400.0		23900.0		23700.0	
1.250	34800.0		33600.0		33100.0		33500.0		33300.0	
2.000	55100.0		53200.0		51200.0		50700.0		50600.0	
5.000	82400.0	5.68	84100.0	6.84	83600.0	7.61	82200.0	7.72	82400.0	7.83
10.000	100000.0	5.97	107000.0	7.93	110000.0	9.37	109000.0	9.85	109000.0	10.00
20.000	118000.0	6.23	130000.0	9.02	138000.0	11.30	139000.0	12.60	140000.0	13.20
50.000	140000.0	6.51	163000.0	10.40	180000.0	14.00	187000.0	16.70	189000.0	18.20
100.0	157000.0	6.68	189000.0	11.30	215000.0	16.00	230000.0	20.10	234000.0	22.50
200.0	174000.0	6.83	217000.0	12.20	253000.0	17.90	280000.0	23.70	285000.0	27.00
500.0	196000.0	6.98	256000.0	13.30	310000.0	20.30	360000.0	28.60	368000.0	33.30
1000.0	213000.0	7.09	288000.0	14.20	359000.0	22.10	434000.0	32.50	444000.0	38.40
2000.0	229000.0	7.17	321000.0	14.90	410000.0	23.80	517000.0	36.50	531000.0	43.40
5000.0	251000.0	7.27	366000.0	15.80	483000.0	25.90	644000.0	41.60	664000.0	50.00
10000.0	268000.0	7.34	404000.0	16.50	545000.0	27.50	761000.0	45.70	786000.0	55.30

Given that this estimate matched the reported return period, it was felt that the peak discharges from the Vargas report could be used to estimate the Canal operational flood event (20-50 year flood). Given that the duration of the measured event was similar to the PMF hydrograph, the project team investigated the applicability of scaling of the existing PMF hydrograph to develop the Canal operational flood. In addition to matching the peak flood discharges, checks were also made against the cumulative flood volumes of each event as well. The Vargas statistics and the measured event were 24-hr average peak discharges. Therefore, the instantaneous peak of the synthetic

hydrograph could be higher. In any case, the Dec 2000 storm had a peak discharge of 645 million cu. m. (22.8 billion cu. ft.) and a cumulative six-day flood volume of 645,650 cu. m (22.8 million cu. ft).

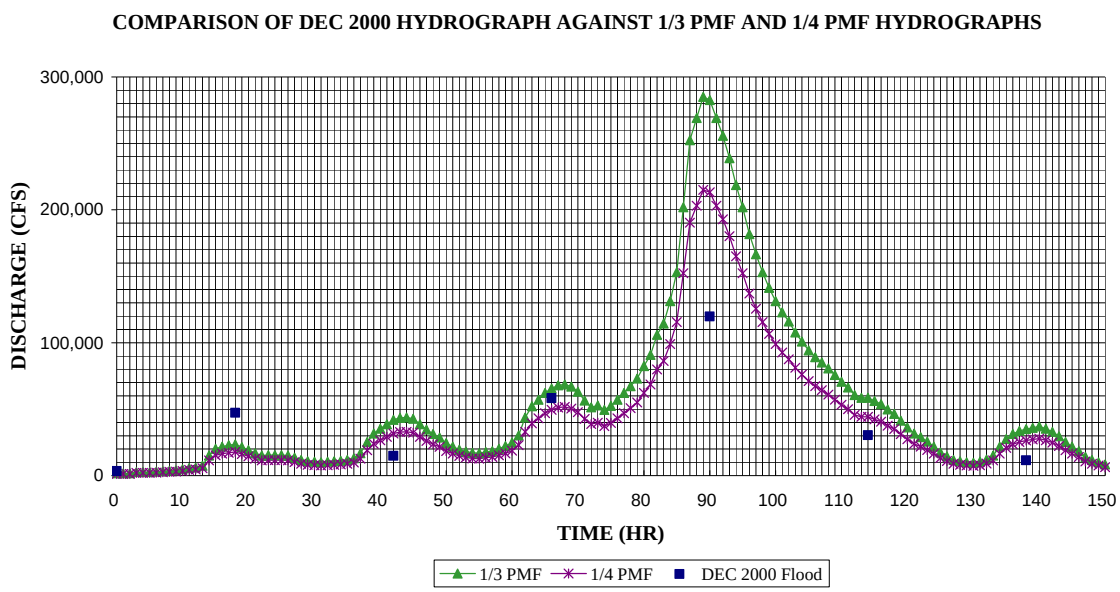
Synthetic flood hydrographs were then developed using the shape of the existing PMF hydrograph. A ratio of the hourly discharge to the peak discharge was first computed for the existing PMF hydrograph. This hourly ratio was then multiplied by the new peak discharge (1/3 PMF, ¼ PMF, etc.) The peak discharges and the cumulative flood volumes for each of the hydrographs were then compared to the Dec 2000 storm and the Vargas statistics. After evaluating several hydrographs, the 1/3 PMF and ¼ PMF hydrographs were selected to use. Figure 3-23 shows the comparison of the 1/3 and ¼ PMF hydrographs to the Dec 2000 event. Table 3-14 shows the comparison of the cumulative flood volumes. As can be seen from the results, the peak discharges for the 1/3 and ¼ PMF flood hydrographs are higher than those listed in Table 3-13 but the discharges are 24-hr average peaks. Therefore, the instantaneous hydrograph discharges should be somewhat higher. However, the general shapes of the synthetic hydrographs match the Dec 2000 flood quite well. For the cumulative flood volumes, the ¼ PMF storm is roughly equivalent to the Dec 2000 event while the 1/3 PMF flood volume is approximately 30% greater. The ½ PMF flood volume is nearly double. Based on these results, it was considered that the 1/3 PMF storm should be used for the Canal operational flood.

There was one additional finding when making these comparisons. If the current PMF instantaneous discharge of 23,894 cms (847,000 cfs) is compared to the values presented in Table 3-14, it appears that the current PMF flood is only four times greater than the assumed 15-year event. Given that the current PMF was developed in the early 1970's and over 30 years of additional hydrologic data is now available, it would be prudent to revisit the PMF at the next design stage to verify that it is still applicable.

Table 3-14: Comparison of Peak Flood Discharges and Cumulative Flood Volumes

	Peak Flood Discharge		Cumulative Flood Volume	
Event	(cms)	(cfs)	(cu. m)	(cu. ft)
Dec 2000	3,398	120,000	645,850,637	22,808,000,000
1/4 PMF	6,088	215,000	632,287,771	22,329,031,877
1/3 PMF	8,070	285,000	838,148,905	29,598,949,233
1/2 PMF	12,799	452,000	1,225,020,765	43,261,200,000
PMF	23,984	847,000	2,490,919,729	87,966,000,000

Figure 3-23: Comparison of Dec 2000 Hydrograph against 1/3 and 1/4 PMF Hydrographs



3.5.1 Design Flood

Criteria recommended for the Design Flood are:

Spillway Design Flood

1) High hazard - use PMF

2) No overtopping or failure of Gatun Dam during PMF

3) Coincident wind and waves (assume COE conditions) during flood peak assumed – PMF maximum elevation being lowered from El 92.55 to 92.0 to keep water level at top of lock coping – assume that sandbags can be used to combat waves (at the locks) at this time since no Canal operations during PMF – based on COE guidance of worst case waves at 0.91 m (3.0 ft) at Gatun the analysis shown in Section 4.1 denotes that at least 1.4 m (4.5 ft) of sandbags should be on top of the lock wall at Gatun to minimize damage from waves during the PMF. For the worst case waves of 0.34 m (1.1 ft) at Pedro Miguel, at least 0.5 m (1.5 ft) of sandbags should be placed on top of the Pedro Miguel lock wall to minimize damage from waves during the PMF. Gatun Dam should be fine given a general controlling elevation of 32.0 m (105 ft) and some isolated low points of 30.0 m (98.4 ft).

4) Overtopping allowed but no structural failure of principal Canal facilities during PMF

5) No Canal operation during PMF

6) No use of lock culverts but spill over lock gates allowed during PMF

7) Mindi Dike overtopping allowed during PMF

8) Mojinga Swamp inundation allowed during PMF – Overtopping of Fort Sherman Road also allowed and failure of roadway likely.

Canal Operation Basis Design Flood

1) Return Period = 1/3 PMF (approximately a 20-50 year flood) - 1/4 PMF found to be very similar to Dec 2000 event)

2) Canal facilities to remain in operation

3) Some wave overtopping OK if does not prevent safe operation of Canal facilities

4) No supplemental use of lock culverts and spill over lock gates.

5) Mindi Dike not overtopped due to navigation impacts downstream of Gatun.

6) Mojinga Swamp inundated but Fort Sherman Road protected from failure

3.5.2 Design Elevations

1) Existing Gatun Dam general crest elevation = 32.0 m (105 ft).

2) Existing Gatun spillway bottom of gates elevation = 91.5 ft. - to be raised to El. 93.0 ft

   The elevation of 93 feet is based on the PMF elevation of 92 feet plus some reasonable allowance for waves and passage of floating debris such as trees under the gates without losing spillway capacity. Since the new gates required for an increase in MOLL are higher and need extensions to the piers - providing some additional height is not seen as a big design problem and would be good insurance in case the PMF ever goes up.

   It appears that whether or not the MOLL gets raised, there are concerns about the lateral strength of the piers, the age and reliability of the existing Stoney Gates, and the reliability of the gate hoists after a seismic event. We propose strengthening the piers, replacing the gates and hoists and modifying the tops of the piers and the existing bridge to suit. Strengthening of the piers will require a combination of vertical tendons and horizontal struts across the top of the piers. With an increase in MOLL then the increase in elevation would be incorporated into this design.

3) Gatun and Pedro Miguel Locks - Top of gates to be raised to El. 91.5 ft with welded plates.

4) Gatun Locks - Top of Lock Coping at El. 92.

5) Gatun Locks - Structure subject to internal flooding during PMF but not during operational basis flood.

3.5.3 Overall Summary of Criteria

Table 3-15 presents a summary of the design criteria for the new Spillway including the geotechnical criteria.

Table 3-15: Summary of Design Criteria

		UNITS	
		Imperial	Metric
Key Assumptions			
	Spillway Design Flood	PMF	
	Operating basis design flood (return period)	1/4 to 1/3 PMF	
Design Elevations			
	Gatun Lake Maximum Normal Operating Level to Be Tested (ft)	90.00	27.43
	Peak PMF Water Surface Elevation Allowed (ft) (Based on El 92.0 stated for dam failure and also top of lock coping) Changed from USACE criteria of El 91.5 since existing spillway will be modified so that bottom of gate elevation when raised is higher.	92.00	28.04
Wave Height (Gatun Spillway)			
	PMF Conditions at Dam and Locks		
	Design wind speed (mph)	COE Guidance	COE Guidance
	Effective fetch (miles)	COE Guidance	COE Guidance
	Significant wave height (ft)	COE Guidance	COE Guidance
	Wave Runup (ft)	COE Guidance	COE Guidance
	Freeboard Required (Significant at Dam/Temporary Measures Will Be Provided at Locks	COE Guidance	COE Guidance
	Normal Operation at Locks		
	Design wind speed (mph)	M&N Memo	M&N Memo
	Effective fetch (miles)	M&N Memo	M&N Memo
	Significant wave height (ft)	1.3 ft	0.39 m
	Wave Runup (ft)	2.0 ft	0.62 m
	Freeboard Required	2.0 ft	0.62 m
Existing Gatun Spillway (**After Alterations**)			
	Spillway Crest Elevation (ft/M.)	68.89	21.00
	Gate Sill elevation (ft/M.)	68.89	21.00
	Top of Gate Elevation – closed (ft/M.)	91.00	27.74
	Height of Gate (ft/M)	22.00	6.71
	Elevation of gate sill when fully open (ft./M)	93.00	28.34
	Elevation of top of gate when fully open	115.00	35.05
	Elevation of top of walkway (ft/M)	119.00	36.27
Stability Criteria			
	Uplift Assumptions: Uplift pressure varies linearly from full reservoir head at the upstream heel of the section to tailwater head, or zero at the downstream toe of the section.		
	Shear-Friction Factor of Safety Against Sliding	(COEGuidance)	
	Normal Reservoir Loading Condition	3.0	
	PMF Reservoir Loading Condition	2.0	
	Earthquake Loading Conditions	1.1	

	Overturning Stability Criteria: Under normal reservoir loading conditions the resultant force, including uplift must remain within the kern of the base (No tensile stresses allowed). Under PMF and earthquake conditions, the resultant force must remain within the base area. If the resultant falls outside of the kern, a base separation is assumed in the tensile stress area and compression is assumed to have a triangular distribution over the unseparated area. Full reservoir uplift pressure is assumed in the base separation. In this case the shear friction factor is computed using only the unseparated area of the base.	(COEGuidance)	
	Relative displacement in earthquake between spillway walls & embankment (to protect against potential piping failure along interface)	< 3.3 ft.	< 1m
Gatun Dam and Saddle Dams	Static stability criteria	<10% reduction in factors of safety for 0.7m increase in water level	See Ref. 5
	Seismic stability criteria	Vertical deformations up to 2m. Horizontal deformations up to 10m Local failure acceptable, but no sudden release of reservoir contents	

3.5.4 Dam Stability Considerations

To satisfy dam safety requirements, the spillway control structure would be designed to meet or exceed stability criteria for sliding and overturning. At the conceptual level designs are proposed which have been shown by experience to meet stability criteria and a stability check has been made. In subsequent detailed design the configuration of these structures would be optimized and these analyses would be refined.

Sliding

The factor of safety against sliding defined as the ratio of the resultant resisting forces to the resultant driving forces should meet the minimum values shown in Table 3-16.

Table 3-16: Factors of Safety for Sliding Stability

Loading Condition	Shear Friction Factor
Normal Operating Level	3.00
PMF Lake Level	2.00
Earthquake	1.10

Overturning

The analysis should be based on the criteria that no tension can be transmitted between the concrete section and the foundation rock. If the resultant vertical force falls outside the kern of the base, a base separation is assumed in the tensile area and the vertical compression is assumed to have a triangular distribution over the unseparated area with the maximum compressive stress occurring at the downstream edge. In this case the shear friction factor is computed using only the unseparated area of the base. Principal parameters for spillway stability analysis are presented on Table 3-17.

Table 3-17: Stability Analysis Parameters

PARAMETER	VALUE		SOURCE
Unit weight of concrete	2430 kg/m^3 (150 pcf)		Assumption
Level I Earthquake PGA (horiz)	0.22g		Golder Assoc. Ref.
Level II Earthquake PGA (horiz)	0.41g		Golder Assoc. Ref.
Vertical Earthquake PGA	2/3 of Horizontal		Assumption
Foundation Properties	Foundation Interface	Rock Mass	
Friction Angle (Rock)	38 degrees	38 degrees	Golder Assoc. Ref.
Cohesion (Rock)	0 KPa (0 psi)	200 KPa (29 psi)	Golder Assoc. Ref.
Friction Angle (Concrete)	45 degrees	N/A	
Cohesion (Concrete)	1390 KPa (200psi)	N/A	

Note: Golder Associate Reference listed above refers to Alan Hull memo located in Appendix B.

Uplift Considerations

Foundation drainage should be provided in the design of the control structure to minimize uplift pressures. If no drainage is provided, uplift pressure should be assumed to vary linearly from full reservoir pressure at the upstream toe of the base to tailwater pressure (if applicable) or zero at the downstream boundary. If drainage is provided the uplift pressure should be assumed to vary linearly from full reservoir pressure at the upstream toe to one third of the difference between headwater pressure and tailwater pressure (or zero) at the line of drains, thence linearly to tailwater pressure (or zero) at the downstream boundary.

3.5.5 Canal Operational Flood Check

As stated previously, the proposed new spillway was designed to pass the current PMF flood with a maximum allowable flood elevation of 28.04 m (92.0 ft). However, the designs should also be tested with the Canal Operational Flood (1/3 PMF) to verify that the current locks could operate uninterrupted during this event based on ACP criteria. Therefore, the spreadsheet model was re-run for each proposed MOLL to determine the maximum lake elevation and peak downstream discharge for the Canal Operational Flood. The results of the model runs can be see in Table 3-18. As can be seen from the table, the maximum lake elevation for the Canal Operational Flood is very little higher than the starting MOLL. The reason for this is because of the significant spillway capacity that is now available to release the inflows as quick as they come into the lake. Given that for all cases the maximum routed lake level is at or below 27.43 m (90.0 ft), the freeboard available should be sufficient to allow Canal operations to continue during the Canal Operational Flood.

Table 3-18: Spreadsheet Model Results for Canal Operational Flood Runs

MOLL		# Gates Req.	Peak Flood Elev.		Peak Total Q		Peak Ex. Spill. Q		Peak New Spill. Q	
(m)	(ft)	for New Spillway	(m)	(ft)	(cms)	(cfs)	(cms)	(cfs)	(cms)	(cfs)
26.67	87.5	2	26.70	87.6	7,036	248,467	5,209	183,965	1,826	64,502
26.82	88.0	3	26.82	88.0	8,070	285,000	5,386	190,208	2,684	94,792
26.97	88.5	4	26.97	88.5	8,070	285,000	5,610	198,116	2,460	86,884
27.13	89.0	5	27.13	89.0	8,070	285,000	5,837	206,139	2,233	78,861
27.28	89.5	6	27.28	89.5	8,070	285,000	6,068	214,276	2,003	70,724
27.43	90.0	7	27.43	90.0	8,070	285,000	6,301	222,525	1,769	62,475

3.5.6 Tailwater Elevations

Tailwater elevations were also developed for the Rio Chagres for a range of discharges to:

- Further refine the hydraulic design of the new spillway, and

- To determine the flood elevations during the Canal Operational Flood to estimate the required heights for Mindi Dike and possibly a new dike for Mojinga Swamp.

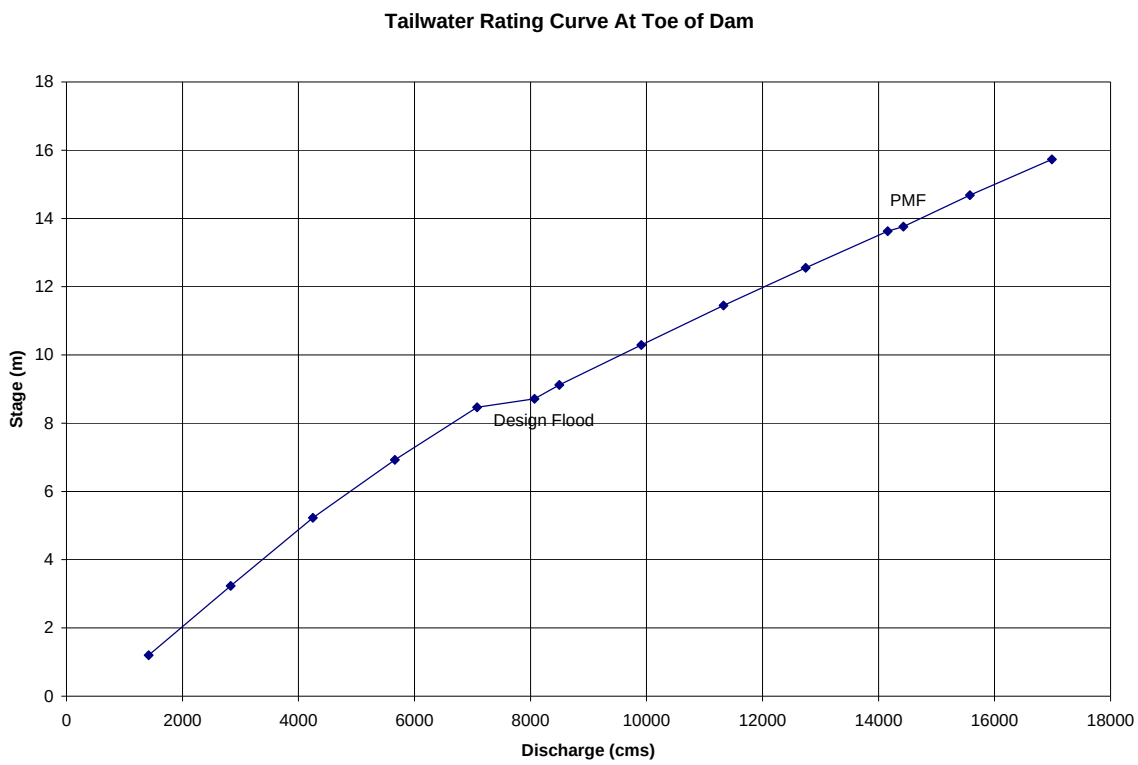
In order to determine the tailwater elevations for the expected ranges of discharges, a backwater model of the Rio Chagres was needed from the mouth of the river up to the existing and proposed spillways.

See Appendix I for a complete discussion of the backwater modelling completed to compute the tailwater elevations expected for Rio Chagres under a wide range of flows.

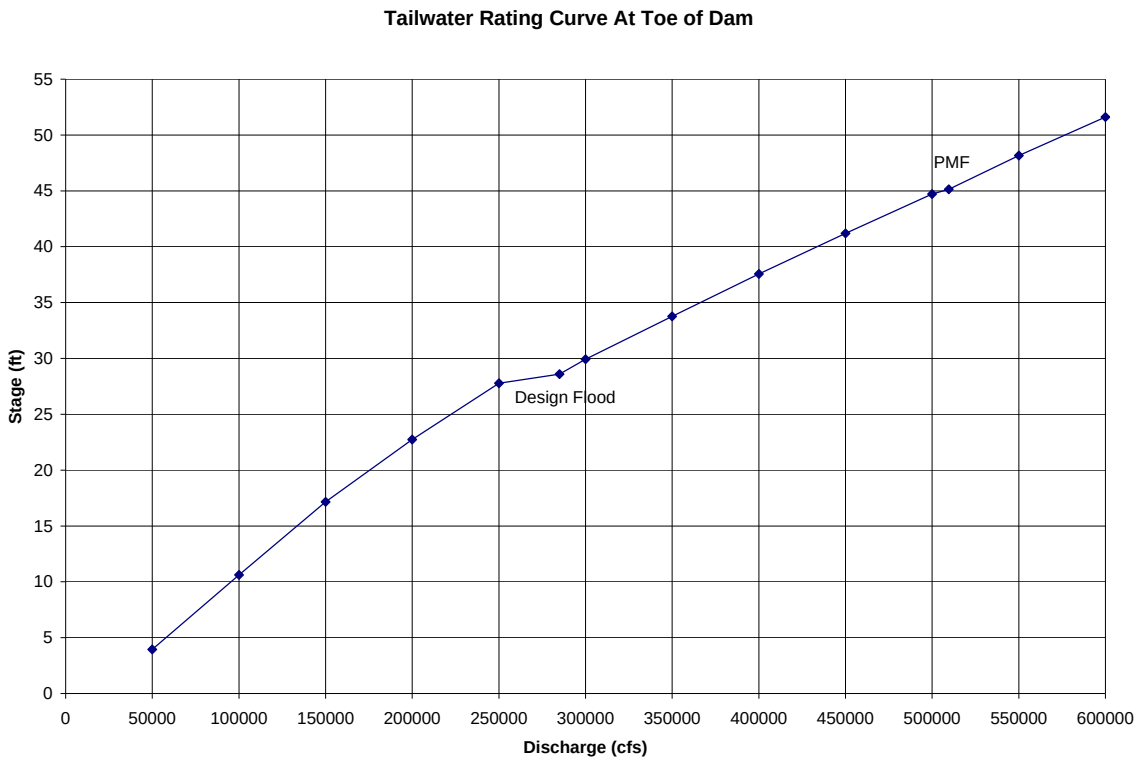
For the hydraulic design of the new spillway, a tailwater rating curve was developed for discharges ranging from 1,416 cms (50,000 cfs) to 16,990 cms (600,000 cfs) and can be seen in Figure 3-24.

Figure 3-24: Tailwater Rating Curve for Hydraulic Design of the New Spillway

Metric Units



English Units



Using the calibrated HEC-RAS model, tailwater conditions were also estimated for the Canal Operational Flood to determine water surface elevations at Mindi Dike and the Mojinga Swamp (please note that high tide and normal depth water elevations were checked for the downstream boundary condition applied – the higher elevation was used) . Table 3-19 shows the results.

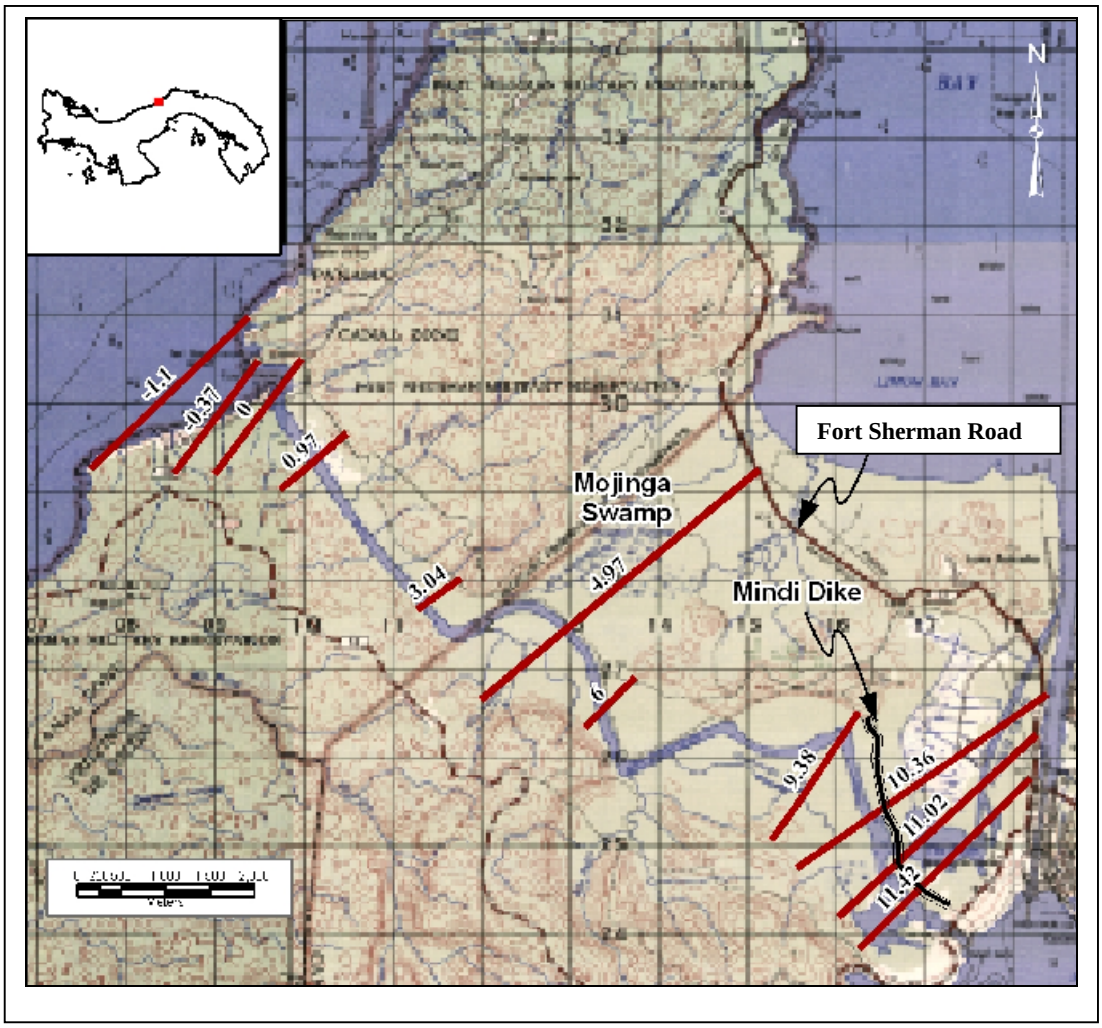
Table 3-19: Spreadsheet Model Results for Canal Operational Flood Runs

Lake Level (m – ft)	Maximum Release During PMF (cms – cfs)	Maximum Release During Canal Operational Flood (cms – cfs)	Maximum WSEL During Canal Operational Flood @ Mindi (m – ft)	Maximum WSEL During Canal Operational Flood @ Mojinga (m – ft)
26.67 - 87.5	9,535 - 336,755	7,036 - 248,467	8.53 - 28	7.32 - 24
26.82 - 88.0	10,477 - 370,003	8,070 - 285,000	8.84 - 29	7.32 - 24
26.97 - 88.5	11,447 - 404,254	8,070 - 285,000	8.84 - 29	7.32 - 24
27.13 - 89.0	12,455 - 439,841	8,070 - 285,000	8.84 - 29	7.32 - 24
27.28 - 89.5	13,496 - 476,620	8,070 - 285,000	8.84 - 29	7.32 - 24
27.43 - 90.0	14,570 - 514,536	8,070 - 285,000	8.84 - 29	7.32 - 24

The estimated peak discharges for the existing and new spillways during the Canal Operational Flood at increased lake levels range from 7,036 cms (248,467 cfs) to 8,070 cms (285,000 cfs). At these discharges, the maximum water surface elevations approach 8.8 m (29 ft) at Mindi Dike and 7.3 m (24 ft) at Mojinga Swamp. As can be seen in the figure below, if Mindi Dike is overtopped, flood waters would flow along the French excavation and would empty into the narrow approach channel at Gatun causing dangerous cross-currents which could impact navigation. For Mojinga Swamp, the road is currently only a meter above sea level and hence the Fort Sherman road would probably be washed out, preventing or limiting access to those communities until the road was reconstructed.

The most effective solution to the flooding risk is to increase elevation of the existing Mindi dike to an elevation of 10.4 m (34 ft) and possibly the construction of a dike at the Mojinga Swamp to elevation 8.8 m (29 ft). However, this last may have limited effect on controlling the flood and should be examined on a cost benefit and hazard risk basis.

Figure 3-25: HEC-RAS Model X-sections, Location of Mindi Dike & Fort Sherman Road



3.6 Conceptual Designs

3.6.1 Layout Concepts

The following considerations were taken into account in the development of the proposed conceptual design and layout of the new spillway.

- The use of an ungated spillway was rejected due to the narrow range of operation which would require a very long crest to provide the needed capacity.

- The use of an emergency fuse-plug or fuse-gate spillway was rejected due to the narrow operating range of Lake Gatun and the adverse economic affects of a protracted reduction of lake level in the event of uncontrolled spill.
- The use of a side-channel spillway was considered and rejected. The advantage of this type of spillway is the compact width for siting on a narrow or steeply sloping spillway abutment. The challenge is the dewatering requirements required to build the intake channel in an operating reservoir. Given the flat slope of the abutment this approach offered no significant advantages.
- The use of a tunnel spillway was rejected due to the low head, large flow and poor nature of the Gatun series sedimentary rocks for large tunnels. A full concrete lining of the tunnel would substantially increase tunneling costs.
- Tainter gates were selected as they are cost-effective, reliable and easy to maintain.
- The existing hydraulic embankment dam is a major constraint to location of the new spillway and has a large impact on siting. The alternative of totally avoiding the existing dam by constructing a deep cut through the left abutment was considered but was found to be impractical due to the steep topography that would require a deep excavation for the approach channel with side slopes reaching 50 to 100 meters high. Preliminary estimates of the volume of excavation required indicated amounts in range of 6 to 10 million cubic meters depending on alignment
- Geological maps indicate the presence of a lineament that projects towards the left abutment of the dam. The full nature and significance of this feature in is still not fully defined from the information currently available. Fortunately the subsurface rock topography of the west abutment offers several alignments which appear to avoid this potential constraint. This issue can be addressed during further field investigations prior to the final design phase of the project.
- Contours of the underlying rock surface were developed from the results of the field investigation program and records of previous borings. This information indicates that there is a humpback ridge of rock under the west arm of the dam with the larger and taller hump reaching an elevation of over 30 meters. The proposed spillway axis is located on the highest axis of this rock formation so that most of the spillway will be constructed in rock and minimize any interface with the existing dam.

- Rock contours downstream of the abutment indicate that the rock surface falls away less steeply to the east which could be more favorable for construction of the stilling basin.
- Locating the spillway axis further east would reduce the amount of rock excavation required but would increase the length and depth of the interface with the existing dam.
- Initially it was assumed that rock in the vicinity of the new spillway would be similar to that exposed at the outlet of the existing spillway which is a hard sandstone that appears to have fairly good resistance to erosion. However testing of drilling samples indicated that the rock in the vicinity of the new spillway is a softer siltstone/mudstone, which is expected to be more easily eroded. This has implications for design of the stilling basin below the spillway and led to the addition of a concrete lined stilling basin.

3.6.2 Proposed Concept

General Description

The proposed new spillway is sited on a narrow ridge that abuts and underlies the western flank of Gatun Dam. Preliminary drawings showing the proposed new spillway layout, profile, sections and details are shown on Figure 3-26 to Figure 3-31.

Main features of the spillway from upstream to downstream are as follows:

- An approach channel.
- A road bridge that crosses the approach channel.
- A spillway control structure comprised of a concrete gravity ogee section containing seven radial spillway gates.
- A chute extending from the downstream end of the control structure down to the stilling basin.
- A concrete lined stilling basin
- A channel extending from the stilling basin to the Rio Chagres.
- Potential floodway improvements to the lower Rio Chagres.

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Figure 3-26

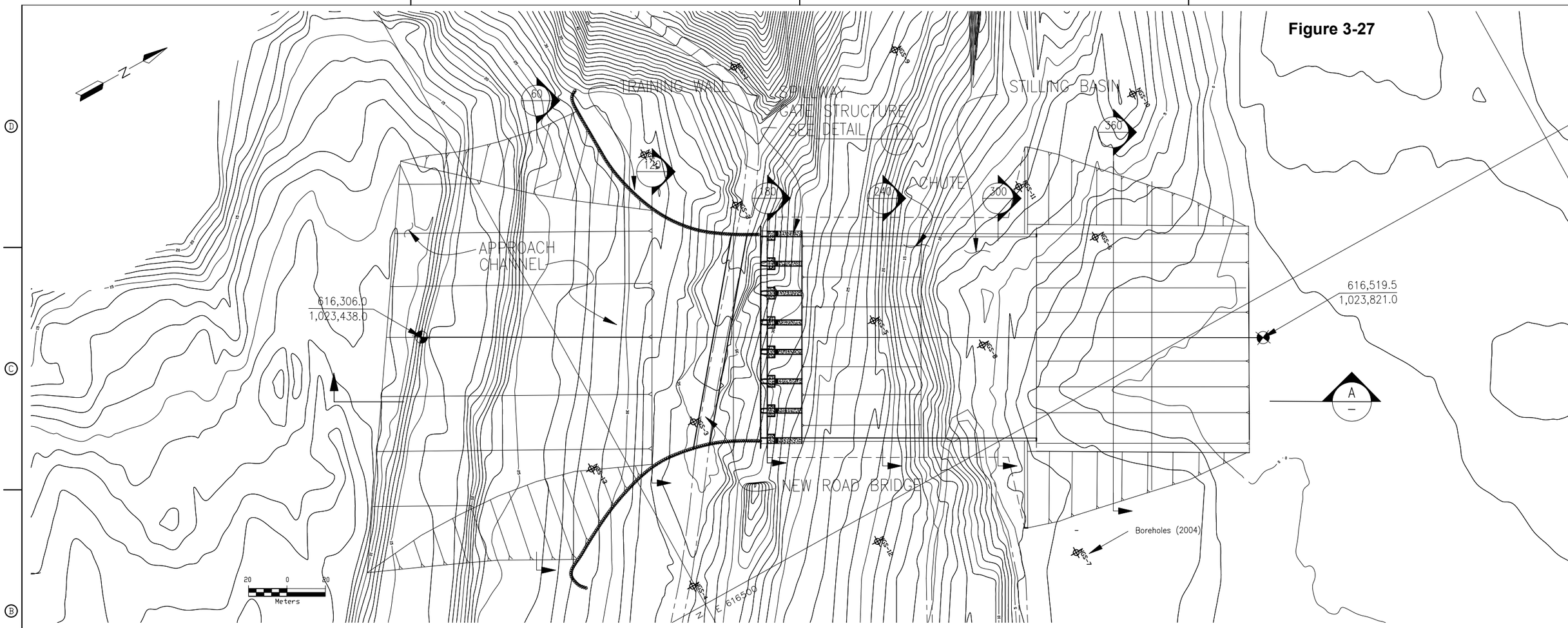
Flood Mitigation Program

For Gatun Lake

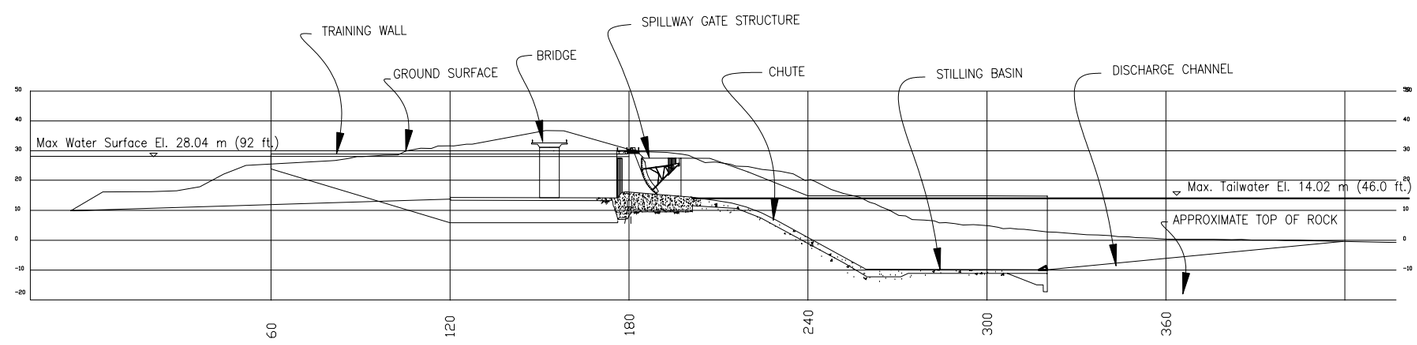
New Gatun Spillway

Location Photo Map

Figure 3-27



PLAN



PROFILE – A

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NEW GATUN SPILLWAY
PLAN AND PROFILE

Drawing No. ACP–NGS–04–001
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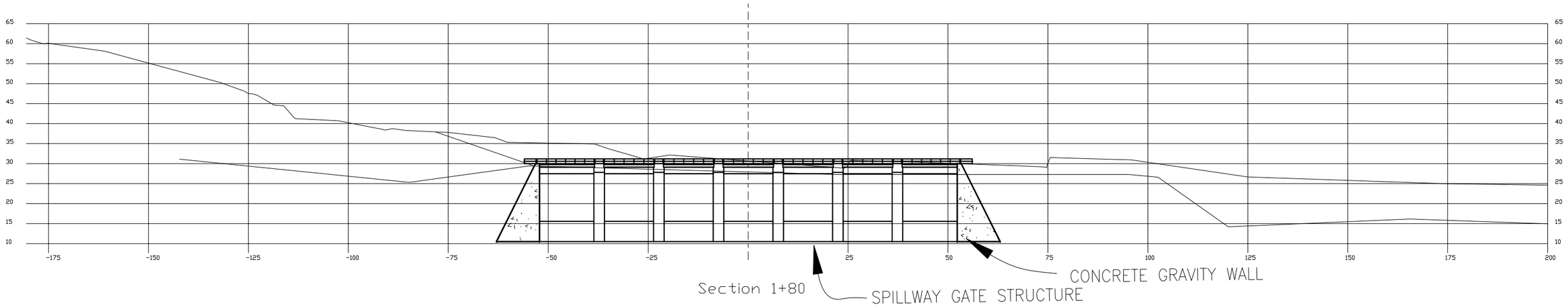
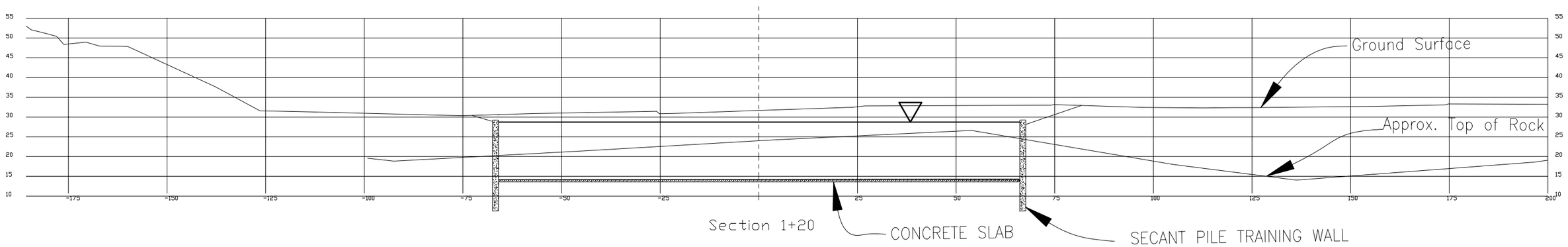
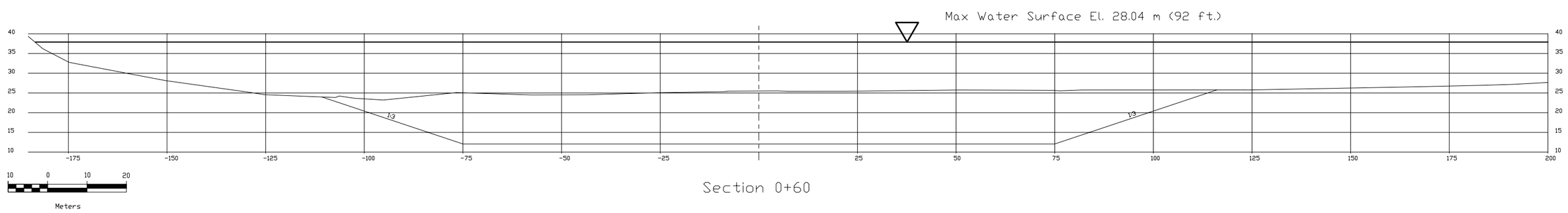
Figure 3-28







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NEW GATUN SPILLWAY
CROSS SECTIONS
UPSTREAM OF GATE STRUCTURE

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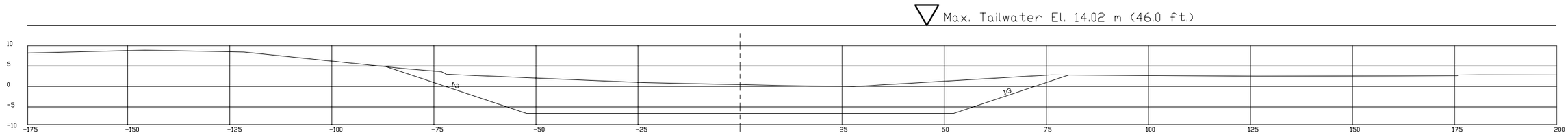
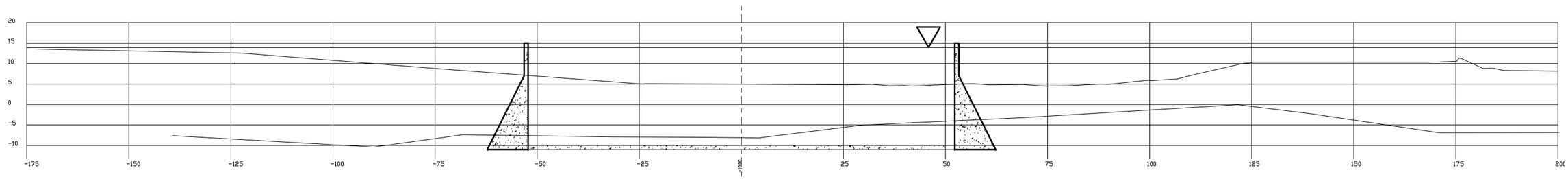
Figure 3-29



Section 3+60



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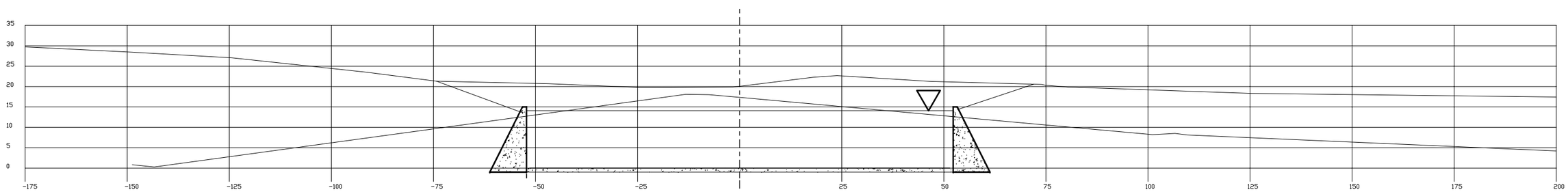

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Meters

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NEW GATUN SPILLWAY
CROSS SECTIONS
DOWNSTREAM OF GATE STRUCTURE

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Approach Channel

The approach channel to the spillway from the reservoir is about 200 m (656 ft) long. It tapers from a width of 215 m (705 ft.) at the entrance to 104.6 m (343 ft) at the face of the spillway. The invert of the channel starts at El 10 m (32.8 ft) and gently rises to El 14.15 (46.4 ft). The estimated average velocity at the channel entrance is 1.6 m/sec, increasing to 4.3 m/s at the entrance to the spillway control structure

The sides of the channel are formed by concrete retaining walls. The left (west) wall retains the toe of the slope cut into the bank at the left end of the dam. The right (east) wall retains a veneer of embankment fill overlying rock at the west abutment of Gatun Dam. Both walls will be reinforced concrete secant-pile retaining wall structures. Construction of the walls will be carried out from the surface and not require a cofferdam or dewatering.

Secant piles are drilled shafts that interlock to form a continuous wall. The common practice is to alternate the placement of the piles along the wall alignment, leaving a space of a little less than the diameter of the intermediate pile. The exact spacing is based on construction tolerances. The intermediate piles are usually formed using a heavy temporary casing that cuts through the concrete piles on either side. Every second shaft is reinforced usually with a wide flanged steel section or reinforcing steel cage.

For planning purposes it is anticipated that the walls will be of cantilever construction comprised of a single row of 1.52 m. (5 ft) diameter secant-piles socketed at least 8 m. into rock.

The floor of the entrance to the approach channel will be excavated by a dredge. Concrete erosion protection will be tremie poured on the rock surface in the area upstream of the spillway.

A road bridge is provided across the approach channel to maintain public road access while maintaining security of the spillway facility. The road bridge would be supported on concrete piers in the channel, which will be shaped to minimize undesirable hydraulic conditions.

3.6.3 Gated Control Structure

The proposed spillway gate control structure is a concrete gravity structure 110.6 m. (362.9 ft.) long and about 22 m. (72.2 ft.) high. The crest is divided into seven 12.80 m (42.0 ft) clear width bays and has a net hydraulic length of 89.6 m. (294.0 ft.) and an ogee crest elevation at El. 16.15 m (53.0 ft).

Reinforced concrete piers 2.5 m (8.2 ft.) wide piers separate the bays. The top of the piers would be at El 30.0 m (98.4 ft.). A preliminary layout and cross sections of the control structure is shown on Figure 3-30.

A preliminary stability analysis was performed on the structure using criteria presented in Section 3.4.4 above. The results of the analysis confirmed that the proposed arrangement would satisfy overturning and sliding stability criteria.

3.6.4 Gates and Hoists

The spillway bays are closed by identical 12.80 m (42.0 ft) wide by (11.60 m (38.05 ft) high tainter (radial) type gates. The gates rest on the sill a short distance downstream of the crown of the ogee crest, El. 16.15 m (53.0 ft).

When closed the tops of the gates are at El. 27.75 m (91.0 ft), 0.3 m (1.0 ft.) above the maximum MOLL level being considered for this study. The trunnion anchorage of each gate is embedded in the side of the piers.

Either cable or hydraulic gate hoists appear to be feasible. In our conceptual design the gates are operated by conventional overhead electric motor driven cable hoists with double drums utilizing multiple parts of wire rope. The hoists are located on platforms at the top of the spillway piers and are accessible via an access bridge that services all seven bays. The hoists would be housed in small permanent enclosures (not shown on drawings) to protect them from the weather. In the engineering design phase the selection and design of hoists would be given further consideration relative to design and operating criteria for the new spillway to be developed with ACP.

ACP has also suggested that provision of a permanent gantry crane be considered. This would also be addressed in the design phase as it would also affect the design of the spillway structure.

A set of stoplogs would be provided to allow regular maintenance or replacement of the spillway gates. In the event that ACP wishes to reduce the risk that gate failure could result in an unacceptable loss of water from Gatun Lake, the contingency plan of providing wheeled stoplogs that can be closed against flow could be considered in the next phase of design. This change may also require an increase in the number of piers and gates to reduce the width of the spillway bays and the size of the opening to be closed off in the event of failure of a gate.

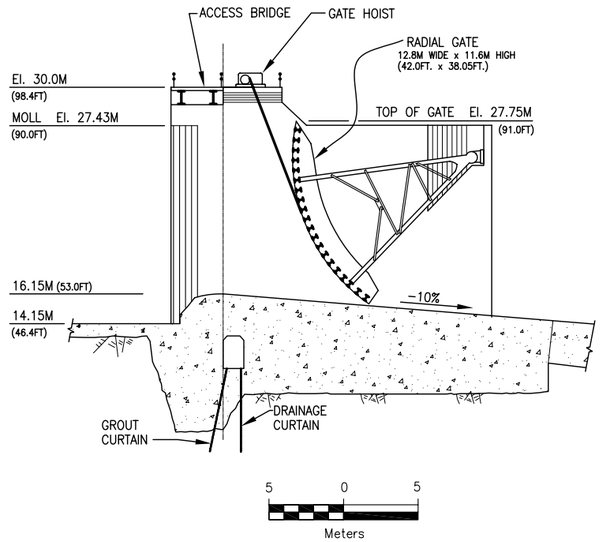
3.6.5 Spillway Chute

The concrete lined spillway chute is 80 m (262 ft.) long and 104.6 m (343.2 ft.) wide and discharges to the stilling basin. The floor of the chute slopes downward at 10% from the

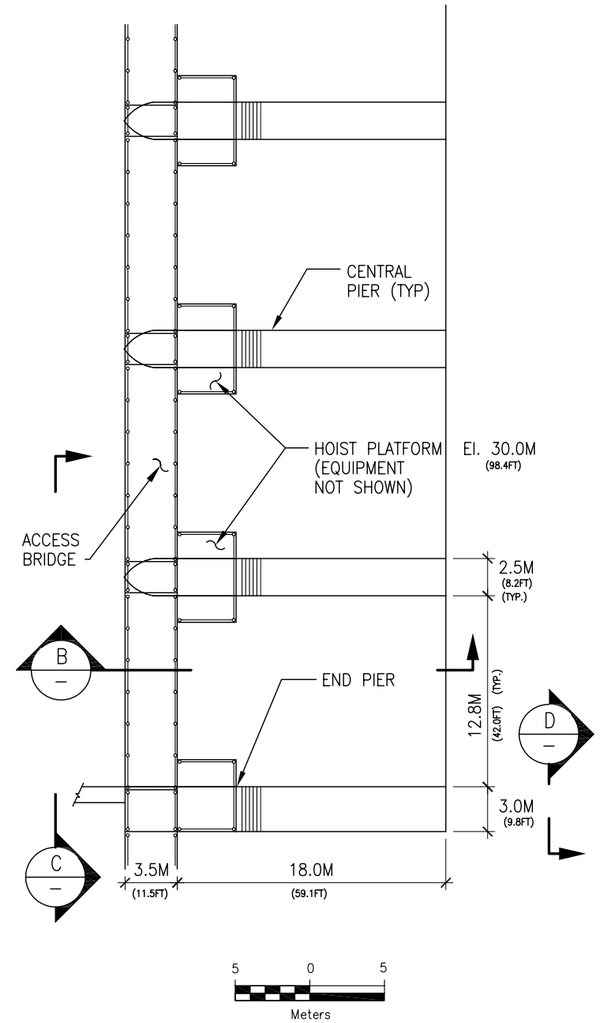
end of the control structure for a distance of 20 m and then steepens to a slope of 40% for an additional distance of 60 m. The toe of the chute is at El. -10m (-32.8 ft.)

Water flowing through the spillway accelerates in the approach channel reaching critical flow at the control structure and high flows will maintain supercritical flow throughout chute. At full capacity of 6234 cms (220,000 cfs) the jet entering the stilling basin would be traveling at an estimated average velocity of 24 m/s (79 ft/sec) with a nominal depth of 2.5 m (8 ft).

Figure 3-30

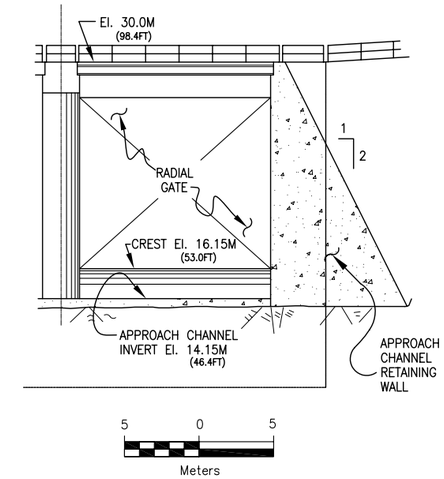


SECTION B

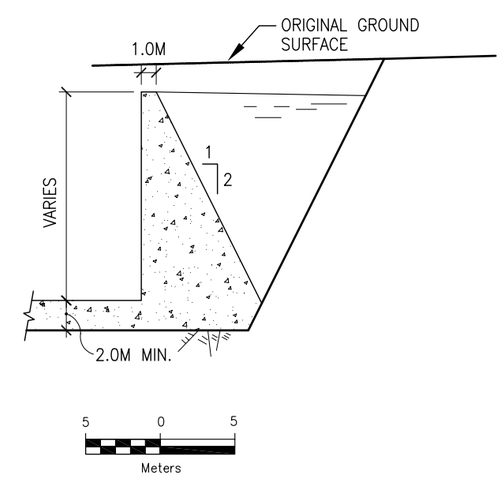


DETAIL 1

SPILLWAY GATE STRUCTURE
PLAN (GATES & OPERATORS NOT SHOWN)



SECTION C



SECTION D

DOWNSTREAM CHUTE
& TRAINING/RETAINING WALL

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GATE STRUCTURE
DETAIL AND SECTIONS

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3.6.6 Stilling Basin

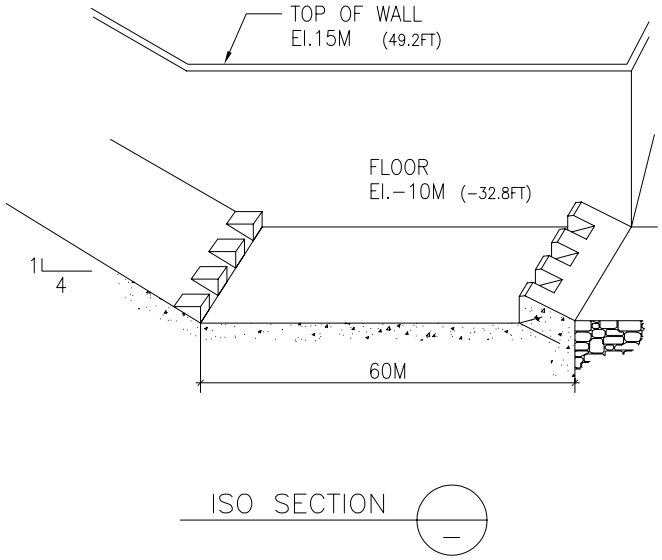
The concrete lined stilling basin at the toe of the chute is a hydraulic jump stilling basin that relies on turbulence in the transition between supercritical to subcritical flow to dissipate energy. and reduce water velocity. The stilling basin configuration is based on a USBR Type II configuration and is designed so that the hydraulic jump occurs within the basin under normal discharge (low tailwater) and PMF (high tailwater) conditions.

Figure 3-31 shows a schematic view of the stilling basin. The basin is the same width as the chute, 104.6 m (343.2 ft).) and the floor is at El. -10m (-32.8 ft.). The length of the basin is about 60 m (197 ft.) long. There would be dentated blocks at the entrance of the basin and a dentated sill at the outlet.

3.6.7 Outlet Channel

An outlet channel approximately 100 m (328 ft) long will be dredged from the end of the stilling basin out to channel that connects to the Rio Chagres downstream of the existing spillway. The issue of tailwater levels on the lower Rio Chagres during spillway operation and their potential affect on operation of the existing spillway, hydroelectric plant, shipping channel from Limon Bay to Gatun Locks needs further investigation. Potential hydraulic improvements to the channel and banks of the lower Rio Chagres and development of a flood reservation downsteam of the dam to maintain the flood plain may be desirable.

Figure 3-31: Schematic of Stilling Basin



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STILLING BASIN

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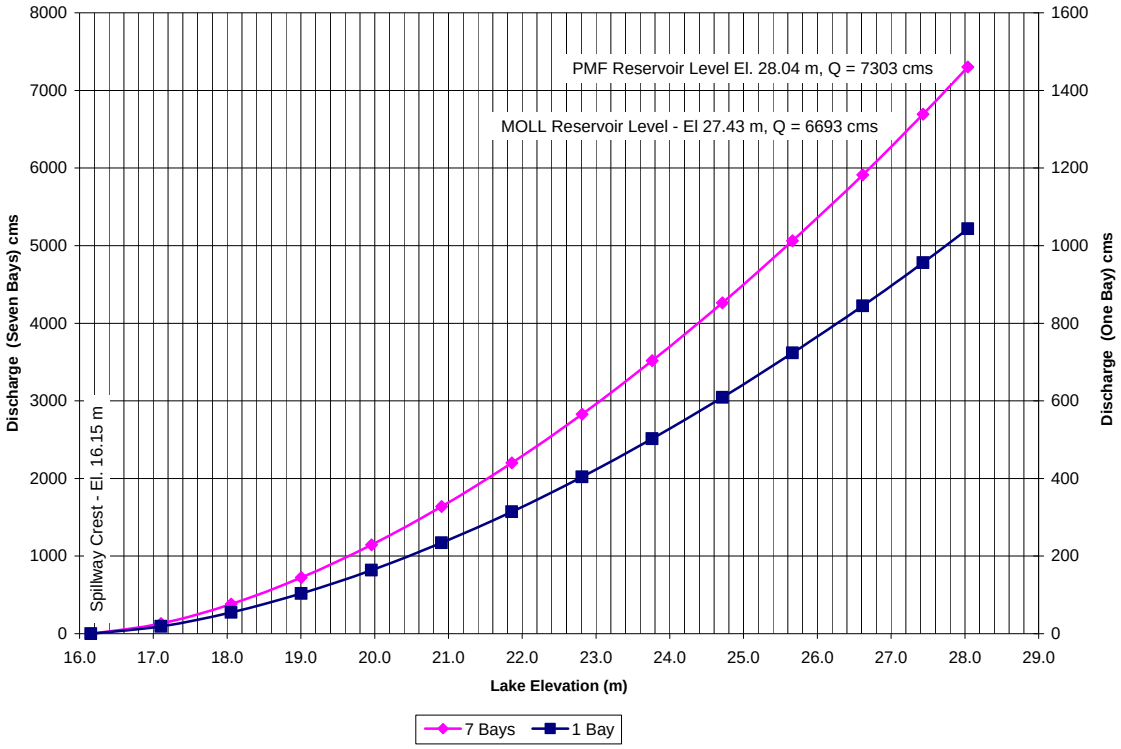
3.6.8 Discharge Rating

The spillway rating curves show the estimated discharge for one bay and all seven bays of the spillway. Under the current operating regime the estimated maximum spillway capacity is 5912 cms (208,780 cfs) at the current maximum operating level of El. 26.67 m (87.5 ft) PLD and the estimated maximum discharge at El. 27.43 (90 ft.) is 6693 cms (236,360 cfs). The estimated maximum flow under PMF routing assumptions El. 28.04 m. (92.0 ft) PLD is 7303 cms (257,910 cfs).

Figure 3-32: New Gatun Spillway Rating Curve



3.7 Environmental Impacts

3.7.1 Introduction

The two flood mitigation projects covered in this report include the construction of a new spillway and the potential raising of the MOLL of Lake Gatun. The technical, hydraulic and environmental impacts of each of the project are inter-related and consequently much of the baseline environmental and socio-economic evaluations are common to both projects.

In order to avoid unnecessary repetition of the results of the analyses, the evaluation process and impact assessment approaches are presented in this section. The discussion on environmental impacts presented in this section is limited to those that are specific to the new spillway project. The impacts associated with the MOLL project are then presented in Chapter 4.

In general the objectives of the environmental assessment of both projects are:

- To identify the potential beneficial and adverse environmental and socio-economic impacts that could result from the construction and operation of the Projects.
- To assess potential temporary, residual, and cumulative impacts produced by the construction and operation of the Projects. To assess the magnitude of the impacts identified in terms of duration and spatial context. The impacts described in this report are potential in nature with varying degrees of occurrence and are based in many cases on worst-case scenarios assuming static environmental conditions, which in reality vary naturally and may actually heighten or lessen impacts.
- To rank the potential impacts identified according to their importance with regard to the environmental setting in the Project areas.

3.7.2 Methodology

The environmental assessment for both projects was performed through a review of existing environmental reports and data, site visits, and an interactive process between ACP, and the engineering and environmental consulting team. The interaction between ACP and consultants allowed the evaluation of design options and the planning of the Projects during this feasibility phase in order to minimize the adverse impacts and to maximize the beneficial impacts during the construction and operation phases.

Site visits were conducted on March 10-12 and May 4-7, 2004. The March site visit consisted of a reconnaissance by helicopter of the Canal facilities to identify those

facilities that would be affected by an increase in maximum operating level of Gatun Lake. The aerial reconnaissance consisted of a flight from Panama City across Miraflores Lake then northeast to the Madden Dam and to Gatun Lake. The flight route continued northwest to the Gatun Locks, Dam and Spillway. A ground reconnaissance conducted on March 11 consisted of traveling from Panama City northwest up the Transisthmithian Highway passing Miraflores Locks and then northeast up to Madden Dam. After viewing the Madden Dam and Spillway from a lookout area near the highway, the reconnaissance continued northwest to the Gatun Area passing through the Canoa area which was proposed as the preferred spillway site by the USACE in their 1999 study.

A road tour was also conducted to the south along the west side of Gatun Lake to La Treinticinco passing several potential spillway sites that had been identified on topographic maps as potential alternatives and described in the Interim Report of Potential Sites For Additional Spillway Capacity.

The May site reconnaissance was conducted to evaluate ACP and third party structures that would be affected by an increase in the maximum operating level of Gatun Lake. The reconnaissance included a visit to Pedro Miguel Meteorological and Hydrological Station, Paraiso Tug Landing, Las Cruces Landing, Trans-Isthmian Highway, the Miraflores Water Treatment Pump Station, Gamboa Bridge, Gamboa Rain Forest Resort, Barro Colorado Island, Gatun Locks, and populated areas of Santa Rosa, Embera (site of indigenous community near Gamboa), Limon, Nueva Providencia, La Represa water intake, Espinar, Mount Hope water intake, Escobal, Cuipo, La Laguna, La Arenosa, Lagartera, and the Laguna Alta water intake.

An analysis of the potential impacts associated with the construction and operation of the Projects was performed during the desk-top environmental assessment, by overlaying the proposed activities on the existing environmental conditions as established during the site visits and based on a review of available literature.

The terms Direct Impact Areas (DIA) and Indirect Impact Areas (IIA) were used to classify areas that would be affected by the Projects. The Direct Impact Areas for the MOLL were defined as the Gatun Lake coastline including the Chagres River downstream of Alajeula Dam. The DIA for the proposed new spillway was defined as the footprint of the proposed structure and downstream of the proposed new spillway. The IIA for the MOLL project was defined as a 1-km distance from the shoreline while the IIA for the proposed spillway was defined as San Lorenzo Park, listed as a protected area.

The economic benefits of these projects will extend beyond the borders of the Canal watershed into the rest of the country, as such the economic impacts associated with both Projects will have a significant impact to the the country of Panama.

During the social and environmental evaluation of impacts, applicable environmental measures were evaluated to determine if the proposed environmental management systems to be implemented during the construction and operation activities are adequate.

A qualitative and quantitative identification and assessment of environmental impacts using modified Leopold cause-effect matrices was conducted by the environmental specialists based on information provided by review of available literature, field studies, identification of sensitive areas, and their interaction with design and implementation activities.

During the identification of impacts the following were considered:

- the cause or the promoting agent of change in those activities being implemented by the Projects
- the social and environmental impacts or changes that will be observed as a result of the implementation of the Project activities.

The prediction and evaluation of social and environmental impacts described in this chapter provide a balance of the interests of the ACP, the general and affected public, and the Government of Panama. This balance has been achieved by incorporating project specific information from a variety of sources that have been involved in the development of these projects such as the environmental and engineering departments of ACP, the various technical studies reviewed during the preparation of the existing conditions report for these projects and the site reconnaissance conducted. Since this phase is a feasibility study, no input from the general and affected public was obtained during this study.

3.7.3 Identification of impacts

The potential environmental impacts were identified based on the activities related to the construction and operation of the Projects. The actions or agents that can potentially lead to a change of an environmental indicator, when the activity is implemented, were identified for each of the Project activities.

Project Activities

The following activities are anticipated during construction:

- ***Transportation and Mobilization*** includes all the activities related to the transportation of equipment, machinery, supplies and personnel to and from the proposed spillway site and other sites requiring retrofitting or demolition. Transportation activities are anticipated to take place over land and water.

- ***Site Preparation*** includes all the activities related to stripping of vegetation and earthwork (cut and fill) that is required on the landward and seaward portions of the site for the construction of the proposed spillway, laydown areas, storage, and construction support-building areas. Also includes retrofit and demolition of facilities affected by the MOLL.
- ***Construction of Infrastructure and Utilities*** includes the construction of internal access roadways, installation of the administrative buildings, power, firewater system, effluent treatment system and other services to support the construction of the new spillway and associated infrastructure.
- ***Construction of Spillway*** includes the construction of foundations and superstructure for the new spillway including the placement of cofferdams, and select structural fill.

The following activities are anticipated during the operation and abandonment of the Project:

- ***Operating a New Spillway*** includes all of the operations associated with the proposed spillway including the discharge of water to the original Chagres River channel. This also includes discharge of above normal flows, which may impact the Ft. Sherman Road that leads to the San Lorenzo Protected Area.
- ***Maintenance of the Spillway and Related Structures*** includes all the activities required to ensure the proper civil, mechanical, electrical and automatic operational systems of the new spillway and associated infrastructure.
- ***Closure of Operations and Abandonment*** includes all the activities related to the closure of the plant and its installations at the end of the estimated useful life and the activities related to the proper abandonment of the installations and the rehabilitation of the site.

Environmental Components and Change Indicators

Based on the information collected during the preparation of an existing conditions report for the various physical, biological, socio-economic and cultural components represented in the Direct Impact Area (DIA) of the Projects, change indicators (events that show the occurrence of an impact) have been identified based on the susceptibility of the component to exogenous agents. A code has been assigned for each of the indicators identified in each environmental component analyzed. These codes are then used to facilitate the information management in the process of impact identification and its easy representation in the matrix of environmental impact assessment.

Table 3-20: Change Indicators

Environmental Component	Code	Change Indicators	Effects
Air	A-1	Alteration in the air quality	Refers to the environmental effects such as: dust and particulate generation (PM10), atmospheric emissions of mobile sources (NO2, SO2 and CO).
Noise	R-1	Increase of the noise levels	Considers the increase of environmental noise above baseline levels due to the temporary/permanent introduction of noise levels.
Water	H-1	Increase in turbidity	Refers to the direct increase in suspended particulate material in Gatun Lake and Chagres River downstream of the new Spillway and the alteration of the aquatic habitat.
	H-2	Alteration of physical-chemical quantity and quality of Gatun Lake and the Chagres River downstream of the new spillway	Water quantity refers to the water demand required for the Projects and the supply of the resource and competence use. The physical quality refers to changes in the water flow and volumes and their relation to the typical sediment transport and downstream discharges. This indicator also refers to changes in the physical or chemical quality of the water that may be produced by accidental spills of hydrocarbons (such as combustibles, greases and oils) or other substances related to discharges that deteriorate the quality of the resource.
Soils and Landforms	SU-1	Alteration of the soil structure	Refers to effects such as soil mixing and the compacting of surface soils at the new spillway site and alteration of the soil that would be inundated under the proposed Gatun Lake Maximum Operating Lake Level.
	SU-2	Alteration of the physical-chemical quality	The physical quality refers to the surface erosion caused by proposed Maximum Operating Gatun Lake Level or water-related transport of sediments, or the combination of both. The chemical quality refers to characteristics of the soil that may be affected by accidental hydrocarbon spills (such as combustibles, greases and oils) or other substances related to discharges that deteriorate the quality of the resource.

	SU-3	Alteration of landforms	Refers to the alteration of landscape caused by cut and fill activities during the construction phase at the proposed spillway site and the loss of landscape caused by the proposed Gatun Lake Maximum Operating Level. Also refers to a loss of scenic value.
	SU-4	Alteration of Gatun Lake/Chagres River Morphology	Refers to the changes in the deposition of sediments patterns caused by the interaction with new fixed structures on Gatun Lake. Also refers to the morphological changes to the Chagres River channel to accommodate discharge from the new spillway.
Terrestrial and Aquatic Flora and Fauna	FF-1	Loss of terrestrial vegetation cover	Refers to the removal of vegetation at the proposed location of the new spillway and the loss of flora associated with the proposed Gatun Lake Maximum Operating Level and discharge associated with the new spillway.
	FF-2	Alteration of the structure and composition of the aquatic communities	Refers to the direct disturbance of the lake substratum and the effects caused to the plankton and benthic communities and habitats in the area of the proposed new spillway site and along the Gatun Lake shoreline subject to the proposed Maximum Operating Level.
	FF-3	Changes in the fish catch	Effects to the artisan fishing and the decrease or increase in fish catch caused by fluctuations in the benthic and plankton communities and turbidity.
	FF-4	Reduction in the threatened or endangered terrestrial and aquatic fauna	Refers to the decrease in the number of sightings in the area or the direct eradication of species with conservation ranking that are directly related to accidental contacts with Project elements or indirectly related due to the effects on their habitats.
Social	S-1	Alteration in the demographic composition	Refers to induced migration of the population seeking employment and the creation of substandard settlements.

	S-2	Alteration of local customs	Refers to the alteration of the normal social relationships established in the population in the direct and indirect influence areas by interacting with the non-local workers outside the direct impact area.
	S-3	Nuisance to the population	Refers to the impacts on the population living in the areas of indirect impact near terrestrial or aquatic corridors used for transporting equipment, machinery, supplies and personnel required for the projects and the increase in vehicular traffic and the potential for accidents.
	S-4	Interruption of infrastructure	Refers to the impacts on the transit conditions and on structures such as roads, bridges and waterways that impede the normal traffic flow and the planning and permitting that will take place to prevent damage to infrastructure.
	S-5	Restrictive access to fishing and recreational facilities	Refers to the restrictive access to artisan fishing and recreational use of the Gatun Lake due to the construction of the proposed spillway and proposed increase in the Gatun Lake Maximum Operating Level.
	S-6	ACP and Third Party Infrastructure Impacts	Refers to the impacts on ACP and third party structures subject to the proposed Maximum Operating Level.
Economic	E-1	Increase in the reliability of the Canal and the associated revenue streams from tolls.	Refers to the increase in the reliability of operation of the Canal during El Nino events without draft restrictions which would result in stabilizing and increasing the tonnage traffic ultimately resulting in minimizing the risk to traffic through the Canal and increasing the revenues that will be generated to ACP and the Republic of Panama.
	E-2	Increase in the demand of goods and services	Refers to the increase in the purchase of supplies, goods and services directly related to the construction and operation of the proposed spillway, which will result in the increase of available quality and the supply of some goods and services at local, regional and national levels.
	E-3	Job Creation	Refers to the demand of workers (skilled and non-skilled) that will be required throughout the construction of the proposed spillway and the

			operation phase. The development of this Spillway will produce two types of employment: direct employment, during the construction of the Spillway which will help alleviate the levels of unemployment in the area; and indirect employment created by the increase in the demand for local goods and services.
	E-4	Alteration of the income derived from Gatun Lake	Refers to the potential reduction in economic activities due to increase in lake elevation.
Cultural	AR-1	Alteration or destruction of cultural resources and potential impacts to San Lorenzo Protected Area	Refers to unknown cultural and paleontological (including observation trenches) resources uncovered during the construction activities and the effects caused to them. In addition, includes potential unknown impacts associated with the construction and operation of the new Spillway depending on the ultimate spillway and downstream channel design.

Interaction of Project Activities and Environmental Components

After identifying the project activities, the environmental components and their change indicators in the study area, an interaction matrix presented in Table 3-2 identifies the potential impacts from the project development activities during the construction and operation phases of the Projects.

The definition of Environmental Impact used in this study refers to the product of the interaction of an activity causing a change on a given resource. The changes observed in this resource are the environmental effects, with importance determined through an assessment scheme to establish its significance for the development of the Projects.

For identifying the environmental impacts that potentially will be observed in the study area as a consequence of the development of the construction and operation activities of the Projects, the matrix presented in Table 3-21 has been prepared. The list of codes assigned in Table 3-20 presents the interpretation of this table.

Table 3-21: Matrix of Interaction of Environmental Components and Project Activities

ENVIRONMENT	COMPONENT	PROJECT PHASES										
		Construction							Operation			
		Transportation and Mobilization	*Site preparation*	*Construction of Infrastructure and Utilities*	*Construction of Spillway*	*Retrofit of Saddle Dams*	*Retrofit of Locks*	*Reconstruction*	Operating Gatun at Maximum Level	*Operating new Spillway*	*Maintenance of the Spillway and structures*	*Closure of Operations and Abandonment*
PHYSICAL	Air	A-1	A-1	A-1	A-1	A-1	A-1	A-1				A-1
	Noise	R-1	R-1	R-1	R-1	R-1	R-1	R-1		R-1	R-1	R-1
	Water	H-1	H-1	H-1	H-1	H-I		H-1	H-1	H-1		H-1
			H-2		H-2	H-2		H-2	H-2	H-2		H-2
	Soil		SU-1	SU-1	SU-1							SU-1
			SU-2	SU-2	SU-2	SU-2			SU-2		SU-2	SU-2
			SU-3	SU-3	SU-3							SU-3
					SU-4	SU-4			SU-4			
BIOLOGICAL	Terrestrial and Aquatic Flora and Fauna		FF-1	FF-1	FF-1				FF-1	FF-1		FF-1
			FF-2		FF-2	FF-2			FF-2	FF-2	FF-2	FF-2
			FF-3		FF-3	FF-3			FF-3			FF-3
			FF-4	FF-4	FF-4	FF-4			FF-4	FF-4		FF-4
SOCIAL	Social		S-1	S-1	S-1							
		S-2	S-2	S-2	S-2	S-2		S-2				
		S-3	S-3	S-3	S-3	S-3		SS-3				
		S-4	S-4	S-4	S-4	SS-4	SS-4	SS-4				
			S-5	S-5	S-5				S-5			
			S-6						S-6			
	Economic	E-1	E-1	E-1	E-1	E-1	E-1	E-1	E-1	E-1	E-1	
		E-2	E-2	E-2	E-2	E-2	E-2	E-2	E-2	E-2	E-2	E-2
		E-3	E-3	E-3	E-3	E-3	E-3	E-3	E-3	E-3	E-3	E-3
			E-4	E-4	E-4							
	Cultural		AR-1	AR-1	AR-1							

3.7.4 Evaluation of Impacts

The evaluation of impacts consisted of defining the attributes to be assessed for each of the impacts analyzed and assigning a relative value for each of these attributes. The process developed a ranking of each of the impacts generated by the Project activities during the construction and operation phases of the Projects as described as follows.

Evaluation Criteria

In the process of assessing the environmental impact of the Projects, the attributes and the values were defined for the impact analysis. The attributes established for the environmental impacts were based on the characteristics and the spatial-time behavior produced by the interaction of the project activities and the environmental component affected.

The attributes defined for the evaluation of the potential impacts included:

- Type
- Geographic extent
- Length
- Magnitude
- Probability of occurrence
- Frequency
- Reversibility

The ranking of these attributes is shown in Table 3-23. The definition of these attributes is based on the behavior of known typical impacts, derived from the construction and operation of similar projects.

Table 3-22: Ranking Criteria

Impact Attribute	Classification	Definition
Type	Beneficial	Net benefit for the resource
	Neutral	No net benefit or detriment for the resource
	Adverse	Net detriment for the resource
Geographic Extent	Direct	Confined to the area directly affected by the Projects.
	Local	Impacts extend beyond the areas directly affected but it is located within the boundaries of the assessment study area that will be specified for each discipline or indicator.
	Regional	Impacts extend beyond the local or administrative boundaries specified for each discipline or indicator. It is considered as an indirect impact.
Duration	Short-term	Less than a year
	Medium-term	Between 1 and 5 years
	Long-term	More than 5 years
Magnitude	None	No change is foreseen
	Low	Predicted disturbance will be slightly greater than the existing typical conditions.
	Medium	Predicted effects are much higher than the typical existing conditions but without exceeding the criteria established in the permissible limits or without causing changes in the economic, social, and biological parameters within the ranks of natural variability or social tolerance.
	High	Predictable effects exceed the criteria established or the limits permitted related to potential adverse effect, or they cause a detectable change in social, economic and biological parameters beyond the natural variability or social tolerance.
Frequency	Permanent	Occurs continually
	Temporary	Limited to a specific period (For example: during construction)
	Periodic/ Occasional	Occurs intermittently but repeatedly either at equal time intervals or at irregular time intervals (occasional) (For example, during maintenance activities)
Occurrence Probability	Low	Low probability
	Medium	Possible or probable

Impact Attribute	Classification	Definition
	High	High occurrence
Reversibility	Short-Term	Effect could be reverted in less than a year
	Medium-Term	Effect could be reverted in greater than a year but in less than ten years
	Irreversible	Permanent effect

Evaluation of Impacts

The evaluation of impacts performed by a multi-disciplinary team was carried out using a Leopold modified matrix, in which the environmental and social factors potentially impacted and the Project activities that can lead to a potential impact were represented.

The evaluation method used for the matrix consisted of assigning values, in a relative scale, to all the attributes of the impact analyzed for each of the interrelations of project activity to environmental effect.

Table 3-23 presents the relative scale of values established by the multi-disciplinary team for each of the attributes.

Table 3-23: Scale of Values for Ranking of Impacts

Type (C)		Duration (Du)	
Negative	-1	Long-Term	3
Positive	1	Medium-Term	2
Neutral	0	Short-Term	1
Magnitude (M)		Frequency (F)	
High	3	Permanent	3
Medium	2	Periodic	2
Low	1	Temporary	1
Probability of Occurrence(Po)		Reversibility (R)	
High	1	Irreversible	3
Medium	0.9-0.5	Reversible at medium-term	2
Low	0.4-0.1	Reversible at short-term	1
Geographic Extent (E)			
Regional			3
Local			2
Direct			1

Using the above ranking, values were assigned to each interaction analyzed to determine an index using the following equation that represents the quantitative and qualitative characteristics of the impact:

$$Ca = C \times Po \times (M + E + Du + F + R)$$

The information obtained from the literature review, field studies, observations, suggestions and recommendations from the engineering and environmental representatives of the consulting team and ACP were reviewed prior to assigning values to each of the impacts according to their attributes. Appendix F (Spillway impact table) contains all of the assigned values and final scores given to all potential changes that may occur during the environmental and social component interactions with the Project activities.

With the aim to visualize these quantitative and qualitative characteristics of the impact analyzed in the matrix of interactions, a rank of values was established assigning a color code to each one as shown in Table 3-24.

Table 3-24: Ranks of Values and Color Code

Ranks of Value				
			Predicted Effect	Color Code
15	To	+.1	Positive	
0		0	NEUTRAL	
-5	To	-.1	Slightly negative	
-10	To	-5.1	Moderately negative	
-15	To	-10.1	Highly negative	

The results from this evaluation process are shown in the matrix of evaluation of impacts for the construction and operation phases of the new spillway are presented in Table 3-25 and Table 3-26, respectively.

Table 3-25: Evaluation Impacts – Construction Phase – Spillway

ENVIRONMENT	COMPONENT	INDICATOR	PROJECT PHASE			
			Construction			
			Transportation and Mobilization	*Site Preparation*	*Construction of Infrastructure and Utilities*	*Construction of Spillway*
PHYSICAL	Air	A-1				
	Noise	R-1				
	Water	H-1				
		H-2				
	Soil	SU-1				
		SU-2				
		SU-3				
		SU-4				
BIOLOGICAL	Terrestrial and aquatic flora and fauna	FF-1				
		FF-2				
		FF-3				
		FF-4				
SOCIAL	Social	S-1				
		S-2				
		S-3				
		S-4				
		S-5				
		S-6				
	Economic	E-1				
		E-2				
		E-3				
		E-4				
	Cultural	AR-1				

Table 3-26: Evaluation Impacts – Operation Phase – Spillway

ENVIRONMENT	COMPONENT	INDICATOR	PROJECT PHASE		
			Operation		
			Operating New Spillway	*Maintenance of the Spillway and Structures*	*Closure of Operations and Abandonment*
PHYSICAL	Air	A-1			
	Noise	R-1			
	Water	H-1			
		H-2			
	Soil	SU-1			
		SU-2			
		SU-3			
		SU-4			
BIOLOGICAL	Terrestrial and aquatic flora and fauna	FF-1			
		FF-2			
		FF-3			
		FF-4			
SOCIAL	Social	S-1			
		S-2			
		S-3			
		S-4			
		S-5			
		S-6			
	Economic	E-1			
		E-2			
		E-3			
		E-4			
	Cultural	AR-1			

3.7.5 Analysis of Impacts-New Spillway

This section presents an analysis of the environmental impacts that potentially may occur during the Spillway construction and operation and abandonment phases, determined according to the process described above.

Physical Environment

Air Quality

Construction Phase

The Spillway construction activities most likely to affect air quality in the region will be associated with the transportation of equipment to the site, site preparation, cement dust from concrete batching operations, and construction of the proposed spillway. The maximum air quality impacts during construction are anticipated to be temporary and intermittent from construction sources. These emissions will occur during the construction period when heavy equipment is engaged in the earthwork operations during the clearing, grading, and compaction of the site, as well as the installation of cofferdams, demolition, foundation installation, buildings and associated structures.

The air pollutant emissions due to these sources are primarily associated with potential dust or particulate matter (PM) associated with earthwork operations and potential demolition. Since the preferred spillway location adjacent to the Gatun dam has a rolling terrain and is heavily vegetated, the earthwork operations including site clearing are expected to be of moderate duration compared to the length of the operational life of the proposed spillway. Air quality impacts during construction are considered to be a local impact, since the dispersion of dust will be less than 1 km from the proposed alternative spillway site.

Construction equipment used to prepare the site will produce emissions to the atmosphere from the combustion of fuels such as diesel and gasoline. These gases include NOx, hydrocarbons, CO, PM, and SO2. These emissions are expected to be temporary and intermittent during the construction phase producing low air quality impacts.

The preferred spillway site is located in a rural area within the Canal Operating Area and near the San Lorenzo National Park where the air quality is considered to be relatively good and representative of rural tropical areas where wind transported particulate matter are major sources of air pollution other than water crafts and motor vehicles operating within the vicinity of Gatun Lake. Ambient air quality impacts associated with the construction phase are considered insignificant and the impacts are designated as slightly negative to moderately negative, with moderately negative impacts associated with site preparation, construction of infrastructure and utilities, and spillway construction. Ambient air quality impacts to San Lorenzo Park and the general areas will be minimized by implementation of Best Management Practices to control vechicle and construction related equipment emissions duing this pahse of the project.

Operation Phase

The air quality impacts during the operation of the spillway are expected to be the same as the existing air quality conditions prior to the implementation of this Spillway and therefore these impacts are classified as neutral, except for impacts associated with closure of operations and spillway abandonment, which are designated as slightly negative. The predicted air quality as established from a literature review as presented in the Existing Conditions Report indicates that the ambient air quality of the Direct and Indirect Impact Area are predicted to be significantly below the World Bank Ambient Air Quality Guidelines (World Bank, 1999).

Noise

Construction Phase

Noise impacts from the transportation of construction equipment, materials, and personnel on-site for construction of the new spillway will be temporary and intermittent. All construction activities associated with the new spillway will be carried out within ACP controlled lands that are more than 8 kilometers from the nearest residences such as Nueva Provencia or other undocumented residences near the Spillway site. In the early stages of the construction of the new spillway work performed onsite will consist entirely of grading, surveying, soil boring, and test drilling required for determining specific geophysical characteristics. Active construction will begin with site preparation work and will proceed in accordance with the Spillway schedule. Construction activity will be continuous until completion of the construction.

Typical spillway and building construction equipment will be used, including light and heavy earth-moving equipment, large cranes, lighter mobile cranes and hoisting equipment, concrete placement equipment, miscellaneous pumps and compressors, pile driving and sheet pile driving equipment, welding and metalworking equipment, and heavy trucks and other vehicles.

There will be some construction activities, such as grading, compaction, and pile driving that will increase instantaneous and short-term ambient sound levels significantly. Site preparation activities are relatively short in duration and could dramatically increase near ambient sound levels for short periods, relative to the duration of the construction phase of the Spillway.

The noise levels generated during the construction phase are expected to be localized, intermittent, and of short-term duration. The noise level is not expected to constitute a significant adverse impact. Noise levels produced during construction may cause the limited wildlife in the area to migrate temporarily away from the Spillway site to areas were they will not be disturbed. For these reasons, the noise impacts associated with site preparation, construction of infrastructure and utilities, and spillway construction have been classified as moderately negative while other construction related activities have been classified as slightly negative.

Operation Phase

The noise impacts associated with the operations phase of the spillway are predicted to be at the levels of noise that currently exist from the normal operation of the Canal and Gatun Spillway. Due to the ability to release greater quantities of water during flood events and in accordance to the scheduled maximum operating levels of Gatun, temporary increases in the noise level are expected during releases and the impacts have been designated as slightly negative.

Soils and Landforms

Construction Phase

The most significant impact to soils and landforms will occur during the construction phase of the Spillway during the construction associated with the new spillway (west abutment of Gatun Lake) and the associated site preparation, and the potential channel widening of the existing Chagres River to accommodate the anticipated discharge. To a lesser extent other impacts to soil and landscape are anticipated to occur with the construction of infrastructure such as internal access roadways, temporary construction field offices, and temporary laydown areas required to support the construction phase. The new spillway location is situated in a green-field site and provides some potential laydown areas in the vicinity of the Gatun Dam. The western section of the Gatun Dam was constructed on a natural ridge that rises above El. 30.48 m (100 ft). Geological information at the dam site indicates that the ridge is comprised of sedimentary rock overlain by 6.1 to 9.1 meters of clay. The location of the spillway at the west Gatun Dam abutment provides not only favorable conditions for spillway construction and for the cofferdam required to facilitate construction, but is also provides a short approach to the discharge channel.

The proposed site at the west abutment of Gatun Dam can accommodate the design flows for the additional spillway, while minimizing the required excavation of the Chagres River receiving water as well limiting the amount of site clearing that would be required compared to other alterative spillway sites.

The modification of soils and landforms at the location of the new spillway site will require blasting, excavation, and compaction of soils that will result in a permanent impact to landscape of the immediate footprint of the spillway and receiving channel. The earthwork activity is estimated that will be carried out within the first one and one-half years of the construction period. Soils in the area of the new spill way and construction related areas that will be required to be cleared will result in vegetation removal, soil removal including mixing, compaction by heavy equipment which will result in a destruction of their matrix system consisting of both soil nutrients and root matrix that will expose the soil to extreme temperatures, direct and intense rainfall resulting in erosion, landslides, and decomposition of the humus zones.

The alteration of the physical and chemical quality of the soil by accidental hydrocarbon spills during the construction (machinery, equipment and motor vehicles) is estimated as a slightly significant impact since appropriate control and handling measures will be applied for counteracting these types of incidents.

Impacts to soil and landforms in the direct vicinity of the new spillway are considered significant and permanent.

Soil excavation in the area of the proposed spillway will also require the appropriate screening and management of unexploded ordinances prior to any activities associated with the spillway, especially in the areas of spillway site where various signs of unexploded ordinances were posted.

Borrow material, rip-rap, and aggregate materials will likely be needed for construction of the Spillway. These materials should be obtained from appropriate sites and suppliers. Likewise, excavated material may need to be disposed of off-site. All excavated materials should be disposed of at approved disposal locations. The impacts of obtaining borrow material, rip-rap, and aggregate, and disposal of excavated materials are not included in this report and may need to be addressed in a separate environmental assessment.

Impacts to soil and landforms from the construction phase of the spillway range from neutral to moderately negative.

Operation Phase

Earthquake Hazard

The spillway site is located in a seismically active region of Panama will probably be subjected to moderate to severe earthquakes during the life of the Spillway. The

spillway site lies in a area that recent studies by Cowan (2001)[17] and U.S. Geological Survey (Petersen et. al., 2003)[18] have identified as the principal source of large earthquakes of significance to the Panamá Canal system, including Lake Gatun. The studies provide comprehensive summaries of historical earthquakes, major active faults onshore and offshore of the Panamá Canal area. Quaternary-age faults and their general past activity were summarized by Cowan et al. (1998)[19] and presented in Figure 1 of Volume 2 of the Flood Mitigation Program for Gatun Lake. Figure 1 shows the location of known and inferred faults in the Panama Canal Basin. Schweig et al (1999) provides further details of the location and activity of faults important to the Panama Canal Zone. These results of the reconnaissance-level field investigations reported several faults including Rio Gatun fault that is inferred to be located about 13 km from Gatun Dam. Francsechi (1992)[20] also conducted a comprehensive assessment of the geology of the Gatun Dam.

The available fault studies indicate that while some faults are known to within the Panama Canal Basin, there are few data to constrain accurately their locations. The lack of quantitative data on past activity on faults has been a major limitation to the assessment of earthquake ground motions in the Panama Canal Zone as described by Petersen et al. (2003).

Quantification of these sources and application of earthquake ground motion attenuation relations have been used to develop a probabilistic seismic hazard assessment for six sites along the Panamá Canal, including Gatun Dam. Earthquake shaking of about 0.15g is estimated to have a return period at Gatun Dam of about 500 years. This level of earthquake shaking indicates that the Lake Gatun area has a moderate level of earthquake hazard.

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[17]Cowan, H. (2001): Design earthquakes of the southeast area of the Canal basin, Panama. Unpublished report HC-ACP-01 to ACP.

[18]Petersen, M.; Schweig, E., Mueller, C.; Harmsen, S.; Frankel, A. (2003). Probabilistic seismic hazard analysis for sites in the Panama Canal Zone. Unpublished U.S. Geological Survey Report to ACP.

[19]Cowan, H.; Dart, R.; M. (1998) Map of Quaternary faults and folds of Panama and its offshore regions; a project of International Lithosphere Program Task Group II-2, major active faults of the world. *U.S. Geological Survey Open-file Report 98-779*.

[20]Franceschi, P. (1992): Geology of the Gatun Dam site. Unpublished report of Panama Canal Commission Engineering and Construction Engineering Division, Geotechnical Branch.

Hurricanes/Flood Hazards

The Spillway has several positive impacts, which include an increased spillway capacity that would allow the Maximum Operating Level (MOLL) of Gatun Lake to be raised. This increase in spillway capacity benefits both navigation and water supply for the Canal lock system. An additional positive impact is that a new spillway would have the sufficient capacity to pass the 100-year flood when the lake is operating at its current MOLL of 26.67 m (87.5 ft) PLD.

Water Resources

Construction Phase

During the construction phase, water will be used for dust control, soil compacting and concrete mix. Water required for these activities can be extracted directly from Gatun Lake, while potable water for construction workers will be delivered to the Spillway site while the new proposed spillway is being constructed. Estimated water volume that will be extracted daily from the Gatun Lake during the construction phase cannot be established at this time, but the quantities should not be significant and the impact on the water quantity is considered neutral because of the insignificant amount of water required compared to the capacity of the Gatun Lake reservoir.

The construction of a new spillway will produce a temporary increase in the turbidity of Gatun Lake and the Chagres River. The degree of re-suspension of the material in the water (turbidity plume) is related to several factors such as sediment characteristics (type of material and size), wind and currents prevailing during the execution of these activities in Gatun Lake and the Chagres River and the type of construction and methods used. The turbidity plume can adversely affect the aquatic species and habitats causing a reduction of light and obstructing the respiratory mechanisms of the aquatic species. The effects of the turbidity with proper mitigations can vary from a negative to slightly negative. Deposition of suspended materials may render previously suitable areas unsuitable habitat for certain aquatic species. The impact takes into consideration that the species and habitats in the construction areas have some level of tolerance to the increase of suspension materials and to the light reduction and their ability to migrate to avoid temporary increases in turbidity. The dredging that will be required to provide for a safe discharge of water into the Chagres River channel is considered to be one of the activities generating the type of impact defined above.

The alteration of the physical and chemical quality of the water by accidental hydrocarbon spills (fuel, grease and oils) or other substances from work barges and ships, equipment and machinery used during the water construction activities are considered as a slightly negative impact. All possible precautions will be taken during the construction phase to avoid hydrocarbon spills. The potential for a spill shall be minimized by using trained fuel handling personnel, establishing spill prevention

procedures, maintaining spill cleanup equipment, and preparing a contingency plan. Sanitary waste collected on-site will be either treated on-site of trucked off to an offsite disposal at an approved recycle/disposal facility.

Impacts to water resources due to construction of the Spillway range from neutral to moderately negative. The impacts to water resources from site preparation and construction of the spillway were designated as moderately negative due potential increases in turbidity around the construction zones.

Operation Phase

The construction of a new spillway will allow for an increased spillway capacity that would allow the Maximum Operating Lake Level (MOLL) of Gatun Lake to be increased which would benefit both navigation and water supply for the Canal lock system. The ability to store additional water capacity in Gatun Lake results in a permanent, positive impact for the operation of the Spillway, while also improving the reliability of the operation of the Canal without competing with other current users of the resource in the area.

In addition, the operation of a new spillway would provide for adequate flood surcharge capacity to pass the 100-year flood when the lake is being operated at its current MOLL of 26.67 m (87.5 ft). This additional spillway capacity is needed even at the current MOLL for the reservoir and dam to meet international safety standards and would allow for an improved management of the lake against flooding.

All of the water required for the operation of the proposed new spillway and for providing for the needs of staff and other personnel will be processed from raw water from Gatun Lake or provided by other sources.

The impacts related to the management of surface drainage and discharge of effluents from the spillway during the operation phase are considered as slightly negative; however, the magnitude of the effect will be reduced in a short-term when all the systems designed for its management and treatment are completed and the monitoring of the effluent discharge parameters for ensuring the compliance with both Panamanian and international discharge standards are implemented in order to establish adequate controls.

The environmental management and treatment systems that will be utilized during the operation of the spillway include the proper management of industrial and domestic wastewater discharges from the spillway and ACP facilities and equipment.

A potential impact to the water environment is the discharge of contaminated water to the lake. However, this impact can be managed by including the following treatment systems for residual waters prior to discharge:

- An American Petroleum Institute (API) oil/water separator for surface water runoff contaminated with oil and grease,
- Sanitary sewage treatment units will be used to treat sanitary waste from the ACP facilities, permanent employee camp, and to treat sewage from spillway facilities.
- Waste streams are discharged to lake after treatment through the appropriate treatment system.
- Water collected from the process areas and utility areas will be routed to the API oil/water separator. Collected oil is stored in appropriate tanks and is then trucked to an offsite disposal at an approved recycle/disposal facility.
- Plant effluent will be monitored to ensure compliance with Panamanian and/or applicable World Bank Guidelines for Liquid Effluents for Process Wastewater, Domestic Sewage and Contaminated Stormwater Discharge to Surface Waters effluent limits for those parameters applicable.

Potential impacts to water resources from the operation phase of the spillway range from neutral to moderately negative, with most impacts classified as slightly negative.

Biological Environment

Terrestrial and Aquatic Flora and Fauna

Construction Phase

The study area is located in the tropical humid life zone, which provides habitat for a variety of flora and fauna, many of which are considered rare, threatened, or endangered. The impacts to flora and fauna associated with the transportation and mobilization phase of construction are considered to be neutral to slightly negative. Most transportation activities would take place by boat or by land over existing roadways. Increased road traffic may cause additional mortality of animals crossing roadways for dispersal, foraging, or breeding. Impacts associated with transportation and mobilization are expected to be short-term and minor.

The impacts to flora and fauna associated with site preparation for the construction of the proposed new spillway are slightly to highly negative. Earthwork associated with site preparation would cause permanent irreversible loss of wetlands and forest lands at the spillway location and alteration of the morphology of the Chagres River, which would result in habitat loss for species utilizing these areas. The estimated footprint of the proposed spillway is approximately 4.12 hectares (10.19 acres) and it is located in an area of wetlands and forest. A wetland delineation was not conducted as part of the feasibility study, but wetlands were observed in the vicinity of the spillway location during site visits. The location and extent of changes to the morphology of the Chagres River

downstream of the spillway will be determined by several factors, including spillway location, discharge, and streambed and bank substrate. Motile species may be able to move to unimpacted habitats within the watershed; however, it is unclear whether these habitats would be able to support the influx of new individuals. Less motile or non-motile species would be directly and adversely impacted by earthwork associated with site preparation. Short-term, temporary increases in turbidity and sedimentation are likely to cause temporary alteration in the structure and composition of plankton and benthic communities, which may have impacts on fish abundance and diversity. Cofferdams will be used on the both the upstream and downstream sides of the spillway during construction and will control turbidity to some extent. Aquatic species previously present in areas impacted by turbidity and sedimentation during site preparation are expected to re-colonize these areas once disturbance ceases.

Construction of the new spillway would have a slightly to moderately negative impact on flora and fauna. The clearing and cutting of this area would result in permanent loss of vegetative cover and morphological alteration of the Chagres River channel, which would mean the loss of habitat for some species. Aquatic communities are likely to be impacted by turbidity generated during construction; however, aquatic species are expected to recolonize these areas once construction activities cease.

Table 3-27 presents a list of benthic macroinvertebrate organisms encountered during surveys of Gatun Lake in 1992 and 1993.

Table 3-27: Benthic Macroinvertebrates Observed in Gatun Lake, 1992-1993[21]

Benthic Macroinvertebrates Observed in Gatun Lake, 1992-1993	
Bryozoans	
Nematodes	
Annelids	Oligochaetes (*Branchiura sowerbyi* dominant)
	Hirudinea
Ostracods	
Mollusks	*Corbicula sp*
	Melanoides tuberculata
	Sphaerium sp.
	Pyrogophorus coronatus

[21] Study of Alternatives to the Panama Canal, Biological Inventory Report, University of Panama and ANCON, 1994.

Decapods	
Aquatic insects	Diptera
	Chaoboridae
	Odonata

Table 3-28 presents a list of commonly captured freshwater fish species in the Panama Canal Watershed. Data for the table is from capture data from the 1992 and 1993 surveys conducted for the biological inventory portion of the Panama Canal Alternatives Study.

Table 3-28: Commonly Captured Freshwater Fish Species in the Panama Canal Watershed, 1992-1993

Common Name	Scientific Name
Peacock bass	*Cichla ocellaris*
Tigerfish	*Hoplias microlepsis*
Black belt cichlid	*Vieja maculicauda*
Sábalo pipon	*Brycon chagrensis*
Bocachico	*Cyphocharax magdalenae*
Guatemalan chulín	*Rhamdia guatemalensis*

Several thousand species of flora and fauna are found within the Panama Canal Watershed. Many of these species are included in conservation rankings established in international conservation agreements and some are protected by Panamanian laws. A complete EIA with species inventory should be conducted to determine whether the animal species identified in the conservation lists forage, breed, or nest in the study area, but their presence in directly impacted areas is considered unlikely due to the historic or current human activity in most of these locations. However, the Spillway site and its surroundings may provide at a minimum a temporary migratory route for many of these species. The impacts on these species due to construction of the Spillway are considered neutral to moderately negative.

Table 3-29 presents the conservation ranking of fauna found in the Panama Canal Watershed as established by international agreements: Convention of International Trade of Endangered Species of Wild Fauna and Flora (CITES) and the International Union for the Conservation of Nature (IUCN). It is unknown whether these species inhabit the Direct Impact Area, but all assumed to be found within the Indirect Impact Area.

Table 3-29: Fauna Found in the Panama Canal Watershed with International Conservation Rankings

Class	Scientific Name	English Common Name	Spanish Common Name	CITES Status	IUCN Status	Endemic to Panamá
Mammalia	*Tamandua mexicana*	Northern Tamandua	Oso Hormiguero Común	3		
Mammalia	*Dasypus novemcinctus*	Nine-banded armadillo	Armadillo Narizón Común		LR	
Mammalia	*Dasyrprocta punctata*	Central American agouti	Ñeque	3		
Mammalia	*Speothos venaticus*	Bush dog	Perro de Monte	1	VU	
Mammalia	*Urocyon cinereoargenteus*	Grey fox	Zorro Gris		LR	
Mammalia	*Lontra longicaudis*	Neotropical River Otter	Lobo de Agua	1	DD	
Mammalia	*Nasua narica*	White-nosed coati	Cozumbo	3		
Mammalia	*Panthera onca*	Jaguar	Tigre, Jaguar	1	NT	
Mammalia	*Leopardus pardalis*	Ocelot	Ocelot, manigordo	1	LR	
Mammalia	*L. wiedii*	Margay	Tigrillo de Cola Larga	1	LR	
Mammalia	*Herpailurus yaguarondi*	Jaguarundi	Yaguarundi	1	LR	
Mammalia	*Puma concolor*	Puma	Puma	2	LR	
Mammalia	*Trichechus manatus*	West Indian Manatee	Manatí	1	VU	
Mammalia	*Tapirus bairdii*	Baird's Tapir	Danta	1	VU	
Mammalia	*Tayassu pecari*	White-lipped peccary	Chancho de Monte	2		
Mammalia	*Tayassu. tajacu*	Collared peccary	Javelina	2	LR	
Mammalia	*Mazama americana*	Red brocket deer	Corzuela Roja	DD		
Mammalia	*Alouatta palliata*	Golden-mantled howling monkey	Mono Congo	1		

Mammalia	*Ateles geoffroyi*	Geoffroy's spider monkey	Mono arana colorado	2		
Mammalia	*Saguinus geoffroyi*	Geoffroy's tamarin	Tití de Chocó	1		
Mammalia	*Aotus lemurinus*	Lemurine night monkey	Mono Nocturno Subtropical	2	VU	
Mammalia	*Cebus capucinus*	White-faced capuchin	Mono Capuchino	2		
Aves	*Dendrocygna viduata*	White-faced whistling duck	Pato de Cara Blanca	3		
Aves	*Cairina moschata*	Muscovy duck	Pato real	3		
Aves	*Elanus leucurus*	White-tailed kite	Milano Cola Blanca	2		
Aves	*Elanoides forficatus*	Swallow-tailed kite	Milano Tijereta	2		
Aves	*Leptodon cayanensis*	Gray-headed kite	Gavilán Cabeza Gris	2		
Aves	*Chondrohierax uncinatus*	Hook-billed kite	Gavilán Pico Gancho	2		
Aves	*Harpagus bidentatus*	Double-toothed kite	Gavilán Bidentado	2		
Aves	*Ictinia plumbea*	Plumbeous kite	Milano Plomizo	2		
Aves	*Accipiter superciliosus*	Tiny hawk	Gavilán Chiquito	2		
Aves	*Buteo magnirostris*	Roadside hawk	Aguililla Caminera	2		
Aves	*B. jamaicensis*	Red-tailed hawk	Aguililla Cola Roja	2		
Aves	*B. albonotatus*	Zone-tailed hawk	Gavilán Negro	2		
Aves	*Buteogallus anthracinus*	Common black-hawk	Gavilán Cangrejero	2		
Aves	*Harpia harpyja*	Harpy eagle	Harpía	1	**NT**	
Aves	*Spizaetus tyrannus*	Black hawk eagle	Águila Crestada Negra	2		
Aves	*Herpetotheres cachinnans*	Laughing falcon	Halcón Guaco	2		
Aves	*Micrastur ruficollis*	Barred-forest falcon	Halcón-Selvático Barrado	2		

Aves	*M. semitorquatus*	Collared-forest falcon	Halcón Collarejo	2		
Aves	*M. mirandollei*	Slaty-backed forest falcon	Halcón Dorsigris	2		
Aves	*Milvago chimachima*	Yellow-headed Caracara	Chimachima	2		
Aves	*Falco peregrinus*	Peregrine Falcon	Halcón Peregrino	1		
Aves	*Brotogeris jugularis*	Orange-chinned parakeet	Perico Ala Amarilla	2		
Aves	*Pionus menstruus*	Blue-headed parrot	Loro Cabeciazul	2		
Aves	*Amazona autumnalis*	Red-lored parrot	Loro Cachete Amarillo	2		
Aves	*A. farinosa*	Mealy parrot	Loro Corona Azul	2		
Aves	*A. ochrocephala*	Yellow-crowned parrot	Loro Coroniamarillo	2		
Aves	*Ara macao*	Scarlet macaw	Guacamaya Roja	1		
Aves	*A. chloropterus*	Red and green macaw	Guacamayo Rojo Verde	2		
Aves	*A. severa*	Chestnut-fronted macaw	Guacamaya Frenticastaña	2		
Aves	*Lophostrix cristata*	Crested owl	Buho Cuerno Blanco	2		
Aves	*Pulsatrix perspicillata*	Spectacled owl	Buho de Anteojos	2		
Aves	*Strix virgata*	Mottled owl	Buho Café	2		
Aves	*S. nigrolineata*	Black and white owl	Buho Blanquinegro	2		
Aves	*Tyto alba*	Barn owl	Lechuza de Campanario	2		
Aves	*Dendrocygna autumnalis*	Black-bellied whistling duck	Sirirí Vientre Negro	3		
Aves	*Penelope purpurascens*	Crested guan	Pava Cojolita	3		
Aves	*Crax rubra*	Great Curassow	Hocofaisán	3	NT	
Amphibia	*Atelopus zeteki*	Golden frog	Rana dorado	1	EN	YES

Amphibia	*Atelopus limosus*		Sapo Limisa		NT	YES
Amphibia	*Dendrobates auratus*	Poison dart frog		2	LC	
Amphibia	*Phyllobates lugubris*	Lovely poison frog		2		
Reptilia	*Oscaecilia ochrocephala*				LC	YES
Reptilia	*Crocodylus acutus*	American crocodile	Crocodile aguja	1	VU	
Reptilia	*Caiman crocodilus* ssp. *fuscus*	Brown caiman	Babilla	2		
Reptilia	*Iguana iguana*	Green iguana	Iguana verde	2		
Reptilia	*Boa constrictor*	Boa constrictor	Boa constrictor	2		

Source: Appendix F of Volume 1 of the Panamá Canal Gaillard Cut Widening Feasibility Study 1987, PMCC Report 1989, Biological Inventory Report 1993, Condit et al. 2001, and InfoNatura, 2004)

CITES Status

Appendix 1 - Species are rare or endangered, and trade will not be permitted for primarily commercial purposes.

Appendix 2 - Species are not rare or endangered at present, but could become so if trade is not regulated.

Appendix 3 - Species are not endangered, but are managed by the listing nation.

IUCN Status

CR - critically endangered; a calculated probability of extinction during the next 10 years of >50%

DD - data deficient; there in inadequate information to make a direct, or indirect, assessment of its risk of extinction.

EN - endangered; calculated probability of extinction during the next 20 years of >20%

LC - does not qualify as critically endangered, endangered, vulnerable, or near threatened (2001)

LR – does not satisfy the categories of critically endangered, endangered, or vulnerable (1996).

NT - is close to qualifying for critically endangered, endangered, or vulnerable status

VU - vulnerable; calculated probability of extinction during the next 100 years is >10%

Operation Phase

Operation of the new spillway and discharge of water to the original Chagres River channel would have varying impacts on the flora and fauna of this area. In general, the flora and fauna in the riparian corridor are adapted to periodic or seasonal changes in discharge (water quantity and velocity); however, discharges from the new spillway during storm events may be of a shorter duration and a higher flow than discharges from the original spillway. These shorter duration, higher flow discharges may cause channel scouring, including scouring of vegetation and morphologic changes in the channel. Scoured vegetation would likely re-establish during non-release periods. Morphologic

alteration of the channel may destroy certain habitat types and create new habitats, as the channel evolves to accommodate the discharge regime. More motile species would be able to move to suitable areas, while less motile species may perish in habitats that area no longer suitable for their needs. Impacts to rare or threatened species are expected to be minor as such species are not likely to be present in this area due to historic and current human activities. The overall impact on flora and fauna of operating the new spillway is considered slightly negative.

Maintenance of the spillway and related structures is not likely to result in loss of terrestrial vegetative cover. These activities may result in temporary localized changes in the structure and composition of aquatic communities, but these communities are expected to return to their original condition once maintenance operations cease. Maintenance operations are not expected to impact rare or endangered fauna as all structures would be located within areas that have been previously or are currently disturbed by human activities. The overall impact on flora and fauna of maintaining the spillway and related structures is expected to be neutral.

Closure of operations and abandonment may result in some loss of terrestrial vegetative cover as structures are demolished or decommissioned. However, native vegetative may be planted or allowed to colonize these areas once human activities cease. Likewise, alteration of the structure and composition of aquatic communities may occur when submerged or shoreline structures are removed or decommissioned. However, these impacts are expected to be positive as the areas would move towards a more natural condition. Impacts to rare or endangered species are expected to be neutral to slightly positive since these species are not expected to be found in areas of human activity. Over the long-term, rare or endangered species may benefit from closure or abandonment as these areas may evolve through succession into needed habitats. Due to the magnitude of this undertaking, it is unlikely that any of the major operations would cease. Likewise, closure and abandonment of the spillway or other major structures is not anticipated; therefore, closure of operations and abandonment are considered to have only a slightly negative impact on biological resources.

Social, Economic and Cultural Environment

Social

Construction Phase

Alteration of Local Customs. The arrival of people with different customs from other regions of the country or other countries will interact with the local communities near the spillway site and may alter the pre-established local custom and social relation system. This construction phase impact is considered neutral to slightly negative during the initial construction phase. The inter-relation impacts will be dependent on where the labor necessary to construct this Spillway originates from and where they are housed if they are non-local. These impacts will decrease as soon as the non-local labor has completed the Spillway.

Nuisances to the Population. The construction activities related to this Spillway will include the transportation and mobilization of equipment, machinery, supplies and personnel to the Spillway site. Personnel and equipment will be transported overland or water during the mobilization phase of the Spillway and will create temporary nuisances to the population living close to the transportation routes and to Gatun Lake operations. The route that will be mainly used during the construction phase will be the Transisthmian Highway from Panama City or Colon to the proposed spillway. This route, which already has a high vehicle traffic volume, connects Panama City and Colon and crosses several communities along the highway. The resulting potential impacts from the use of this highway from the additional transportation of equipment, materials and personnel and the potential increase of accidents along this route is considered moderately negative during the initial mobilization and as a slight impact during the other construction phases. Control measures will be implemented to minimize the impacts associated with these activities. Impacts to the populations living around the transportation during the construction phase range are considered moderately negative (transportation and mobilization).

Interruption and Deterioration of the Infrastructure. The interruption of the normal traffic patterns during the construction of the spillway will be caused by transporting oversize and large heavy equipment and machinery exceeding the current weight limits of the road bridges. To accommodate these loads, the bridges will be reinforced or modified if it is necessary as a result of the study to be submitted to and approved by Ministry of Public Works. This impact is considered as slightly negative and temporary. Transportation of equipment over water will be conducted using work barges, cranes and others similar construction equipment that is anticipated during the construction of the spillway.

Access Restriction to Fishing Areas and Recreational Activities. The purpose of restricting access to certain portions of Gatun Lake during the construction of the proposed spillway is to protect the fishermen and individuals passing through the area from hazards related to the construction and operation of a Spillway. The access near the spillway installation and other facilities during the construction and operation phase will be restricted because of risk management concerns during the construction of the spillway and its installations and safety of any personnel in the area.

Although, the area near the existing spillway is restricted to recreational activities, there is the potential that construction related equipment and materials may be transported to the site via barges from locations other than the restricted spillway area. For this reason, restrictions to these recreational areas in the spillway area were evaluated.

Restricted access to the spillway and downstream of the Chagres River during the construction phase is considered a slightly negative impact.

Operation Phase

Operation of the proposed new spillway, maintenance of the spillway and infrastructure, and closure of operations and abandonment are not expected to greatly alter the demographic composition of the area, alter local customs or traffic patterns, or interrupt infrastructure. Therefore, the impact of the operation phase on these change indicators is neutral.

Access Restriction to Fishing Areas and Recreational Areas. The access near the proposed spillway and downstream of the Chagres River will be restricted during the operation of the Spillway because of risk management concerns. Non-authorized personnel will be restricted in special areas or during water discharges from the spillway, which are considered as high-risk activities for passers-by. Standard procedure currently used at the existing Gatun Spillway will be observed which include at a minimum several placards posted in the area of the proposed spillway and downstream of the Chagres River warning of the dangers associated with the release of spillway discharge water and an audible alarm at a specified period of time prior to the release of water.

Economic

Construction Phase

The proposed Spillway will result in a capital investment of approximately US$90 million. The resulting Spillway would permit an increase in water storage during both the Wet and Dry Seasons. This increase in storage would improve the reliability of the Panama Canal to serve its customers without draft restrictions during the dry periods. Historically, investment in Canal related projects has had a multiplicative positive impact on the Panamanian economy such as an increase in tax revenues and fees, an increase

in demand for goods and services, and job creation. However, there may be a temporary reduction in income generated from non-shipping sources, such as fishing and tourism, due to construction related restrictions, etc. For all these reasons, the impacts associated with the construction phase of the Spillway are considered highly beneficial.

Increase in Tax Revenue and Fees from Increase in Reliability of the Canal. At the regional and national level, importing and or the local purchase of equipment and materials for the construction of the spillway will represent an increase of tax revenues from both customs and from purchases of goods and services in-country. The impact from an increase in income tax revenues and royalties benefits will be derived from the development of the spillway will be highly positive and will impact the regional and national levels through the multiplicative direct and indirect impacts of the investment in the form of taxes generated from salaries, tax revenues to the government, taxes paid by contractors, the revenues from the sale of fuel and other products to the ships, and the induced impacts from tourism, the duty free market, and ports.

Increase in Demand of Goods and Services. At a national level, benefits will be derived from the spillway. The spillway will cause a general increase in business activity which will affect local, regional and national economies in a positive manner. The construction activities will generate an increase in the demand for goods and services directly and indirectly associated with the spillway. This increase in demand will enhance the local economy by the production of goods and supplies of construction materials, local support services (hotels, restaurants, transportation, stores and warehouses) and an increase in trade among the districts. The spillway will require a high standard of goods and therefore this demand may result in an increase in the quality and the supply of some goods and services offered locally. For all of the above reasons mentioned above, the effect on goods and services during the construction phase is considered as highly positive.

Job Creation. Considering the level of unemployment and the limited available low wage jobs at the local level, the construction of the spillway provides a new source of employment, that could increase per capita income of some qualified local residents in the spillway's direct area of influence, and could result in an indirect benefit to the construction sector and to the local fishing and agriculture sectors by the increased demand of these goods.

The creation of new employment opportunities will allow an influx of capital to areas that have been traditionally lacking of funds to meet basic necessities. Such increases in employment may likely lead to the improvement of social conditions such as housing, health, education, etc. from the added tax revenues associated with the spillway.

The construction of the spillway will raise the need for skills training in the population, and thus, this will bring the specialization of labor related to construction activities and

the development of the potential abilities of the population of the region. Training provided to employees hired during the construction of the spillway will provide useful skills that can be applied to other industries and/or similar projects.

The creation of employment opportunities provides a significant positive impact considering that in 2000 census indicated that 16% of the population greater than 18 years of age was unemployed (CIA, 2002). Construction of the spillway will provide the opportunity for the local labor force to be employed during the construction phase and to obtain skills and work specialization through technology transfers and training.

Construction of the spillway will provide direct employment to a variety of workers throughout the construction phase that may last for several years. The labor demand although not estimated at this time can be significant over the duration of the construction.

In addition to the direct employment, it is estimated that construction activity generates at least three to five indirect jobs for each direct job position. Therefore, induced jobs will be derived from the construction of the spillway.

The temporary increase in employment in the area both directly and indirectly during the construction phase will result in a highly positive impact.

Operation Phase. The economic performance of the Panama Canal is very much aligned with the global economy in particular the economy of the United States, the main user of the Canal, Japan and Europe. As reported in the 2002 annual report, the most recent annual report available on the internet, the uncertainty in the world economy resulted in a 2.8% downturn in cargo tonnage as compared with the previous year. The report also indicated a drop in the transits of 2.3% and an average vessel increase of 3.9%. The actual transit by larger vessels resulted in a positive impact on toll revenue to the level of B/.588.8 million. The outlook for economic growth as reported in the annual report is projected to increase by 2.9% trend as the world economies recover.

The construction of the new spillway will alleviate concerns related to the capacity of the existing spillway under extreme storm conditions. In the event that the existing spillway is unable to accommodate the maximum flood condition, overtopping of the Gatun and Pedro Miguel Lock structures would be a real possibility. Unconfirmed sources within ACP estimate that this eventuality could lead to the closure of the Canal for up to three months, with consequently severe negative economic impacts for ACP, Panama and world economies.

The proposed new locks would also drive the water requirements even higher which would reduce the overall reliability of the Canal if no additional water resources or rise in MOLL is implemented.

Cultural

Alteration or Destruction of the Archaeological Heritage. Archaeological and cultural sites are know to exist in the area near the proposed spillway. These sites include observation trenches from the 1750s that were constructed to guard the trade route across the isthmuths as well as paleontological remains. The review of existing information shows that the possibilities of finding new archaeological/cultural resources in the areas where earthwork will be performed exists. It is recommended that a Phase 1 Archaeological/Cultural Resources Survey be conducted in all new construction areas prior to the start of earthmoving activities to identify any previously archaeological/cultural sites. The protocol for handling previously unidentified resources that are unearthed during earthmoving activities is discussed in the Mitigation and Monitoring Report.

3.7.6 Conclusions

The environmental assessment to identify the potential impacts has been performed through a review of existing environmental reports and data, site visits, and an interactive process between ACP staff and the engineering and environmental consulting team members. This interaction allowed the evaluation of design options and the planning of the Spillway during this feasibility phase in order to minimize the adverse impacts and to maximize the beneficial impacts during the construction and operation phases.

Based on the information collected during the preparation of an existing conditions report for the various physical, biological, socio-economic and cultural components represented in the Direct Impact Area (DIA) of the Spillway, change indicators (events that show the occurrence of an impact) have been identified based on the susceptibility of the component to exogenous agents. This analysis is summarized in Table 3-1. Using the environmental change indicators and the anticipated Spillway activities, the evaluation of the impacts was performed using a Leopold modified matrix. Twenty-three change indicators were evaluated for both the construction and operational phases of the Spillway for various physical, biological, and social components that would be potentially impacted by Spillway activities. Impacts to these components were ranked as either positive, neutral, slightly negative, moderately, negative, or highly negative.

Most impacts associated with the Construction phase were classified as either slightly or moderately negative. Impacts relating to the loss of terrestrial vegetation cover and the alteration of the structure and composition of aquatic communities due to construction activities were classified as highly negative. The estimated cost of mitigating the impacts to wetlands affected by construction of the new spillway is B/$40,760. The

economic impact of the construction phase on tax revenue generation, demand for goods and services, and job creation was classified as highly beneficial.

The Spillway will result is some negative impacts to the physical and biological environment. Efforts will be made to avoid, minimize, or mitigate these impacts by implementing Best Management Practices (BMPs) for construction and operation activities and adhering to accepted international standards. Anticipated negative impacts to the social environment, such as restricted access to certain areas and increased traffic flow will be addressed by following the procedures outlined in the Social and Environmental Management Plan. All proposed social and environmental mitigation measures are presented in Appendix F - Environmental Management, Monitoring, and Mitigation Report.

3.8 Cost Estimates

3.8.1 Capital costs

Costs used in the estimates are applicable to the fourth quarter of 2004 and do not include any allowance for escalation or inflation to the expected date of construction. Allowances for interim financing of costs during construction (AFUDC) are also excluded.

Quantities used in the estimates have been calculated using dimensions taken from the preliminary drawings included in the study. Unit prices include allowances for contractors overhead and profit and are based on experience cost data including bid prices, previous estimates and informal budget prices from contractors or suppliers. Lump sums are used for small items or those which do not lend themselves to unit price costing. Estimated costs for gates and hoists are based on experience data and include transportation to the project site.

Mobilization shown in the estimate is a provision for costs incurred in transporting construction equipment to and from the site and establishing and decommissioning construction equipment, temporary services and facilities at the project site.

Allowances for unforeseen items (contingencies) and engineering and construction management have been included in the cost estimates. At this point, the estimates are raw engineering design and construction costs which do not include other costs such as permitting, environmental, legal, mitigation, force majeure, cost escalation, lost power generation, taxes, import duties, currency exchange rates, financing costs or other indirect costs or expenses. Estimates of these additional costs should be included in any budgeting or financial plans that require overall capital or investment costs.

Estimates and actual bids for this type of heavy civil construction work are subject to considerable variance depending on such factors as perception of risk, contractual terms, permitting constraints and the supply and demand for construction services at the time that bids are solicited.

It should be noted that construction costs have recently been subject to increases in the prices of certain materials including fuel, steel and concrete. Fluctuations in foreign currency can also impact the cost of imported equipment such as spillway gates. The following cost simulation attempts to incorporate these potential variables using the Monte Carlo simulation techniques described and used in Section 2 of the report.

The 80 Percentile confidence level estimate for the construction of the new spillway, is projected by the cost simulation model to be $91.038 million, as indicated in Table 3-30. As seen in Figure 3-33, the range of costs varies from $78.2 to $98.9 million with the relative sensitivity of the cost elements presented in Figure 3-34.

Table 3-30: Cost Estimate for New Spillway (80% Confidence Values)

Item	Quantity	Unit	Unit Cost	Amount
Site work				
Mobilization	1	Lot	3,411,042.07	$ 3,411,042
Site Clearing	7	ha	17,055.21	$ 110,859
Road Relocation	1	Lot	111,978.65	$ 111,979
Dewatering and care of the river	1	Lot	3,411,042.07	$ 3,411,042
Temporary Cofferdams	3,801	sq m	551.28	$ 2,095,413
Dredge Excavation - Spoil	331,700	cu.m	3.72	$ 1,234,301
Dredge Excavation - Rock	32,533	cu.m	13.20	$ 429,598
Excavation - Overburden	264,600	cu.m	4.51	$ 1,194,299
Excavation - Rock	365,800	cu.m	11.37	$ 4,159,197
Backfill	50,525	cu.m	10.34	$ 522,251
Riprap	107,500	cu.m	21.36	$ 2,296,424
Subtotal				**$ 18,976,406**
Structures				
Approach Channel Walls	6,696	sq.m	1,447.11	$ 9,689,840
Foundation Preparation	20,000	sq.m	5.17	$ 103,365
Grouting and Drainage	1	Lot	227,402.80	$ 227,403
Concrete - Ogee	14,120	cu.m	155.05	$ 2,189,269
Concrete - Floor Slabs	23,800	cu.m	186.06	$ 4,428,153
Concrete - Walls	33,500	cu.m	225.00	$ 7,537,500
Concrete - Piers	6,468	cu.m	425.00	$ 2,748,900
Reinforcement	4,526	tonne	1,602.16	$ 7,251,359
Trunnion Anchorage	67	tonne	15,520.24	$ 1,039,856
Embedded Steel	95	tonne	9,550.92	$ 907,337
Embedded Stainless Steel	26	tonne	19,535.97	$ 507,935
Stoplog Guides	32	tonne	$5,500.00	$ 176,000
Install Stoplog Guides	7	ea	$10,000.00	$ 70,000
Spillway Access Bridge	340	sq.m	747.67	$ 254,209
Highway Bridge	1,343	sq.m	1,154.34	$ 1,550,666
Subtotal				**$ 38,681,792**
Mechanical/Electrical				
Radial Gates - Supply	557	tonne	15,150.71	$ 8,438,947
Hoists (80 Tonne Cap)	7	ea	157,631.49	$ 1,103,420
Shipping	7	ea	84,759.23	$ 593,315
Installation	7	ea	264,872.58	$ 1,854,108
Stoplog guides - supply	37	tonne	$8,800.00	$ 325,600
Stoplogs - transport	1	lot	$40,000.00	$ 40,000
Miscellaneous Metal	1	All.	55,300.23	$ 55,300
Misc. Mechanical/Electrical	1	All.	516,824.56	$ 516,825
Subtotal				**$ 12,927,514**
Subtotal - Construction Cost				**$ 70,585,712**
Contingency		17%		$ 12,176,035
Subtotal				**$ 82,761,747**
Engineering, CM, Admin		10%		$ 8,276,175
Estimated Cost of Project (80 percentile Value)				**$ 91,037,922**

Note: Totals, quantities and unit rates shown are 80% confidence values output from 1000 trials.

Figure 3-33: Range of Probable Project Costs for new Spillway

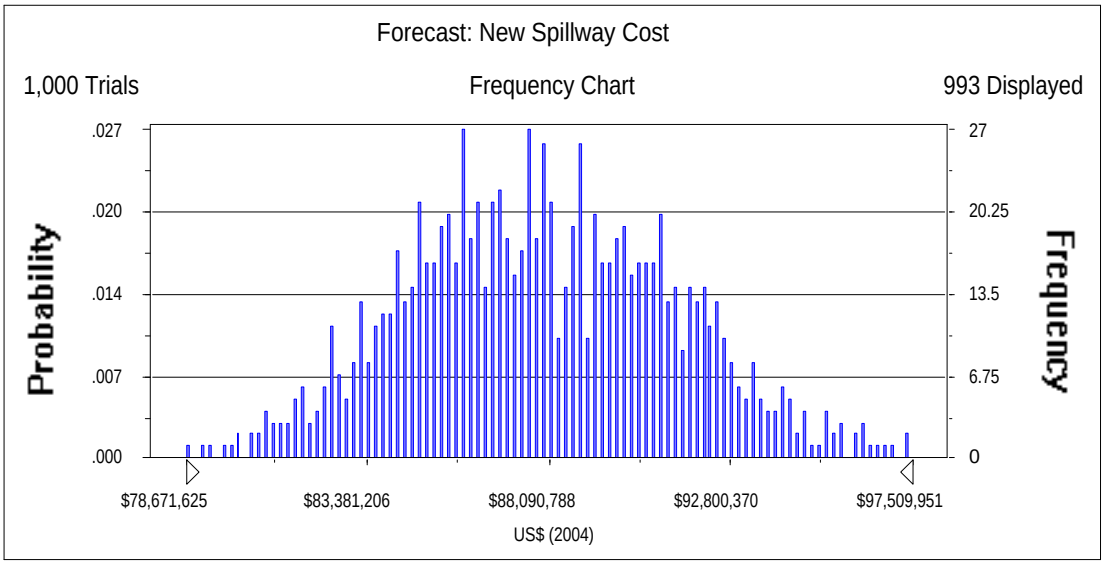
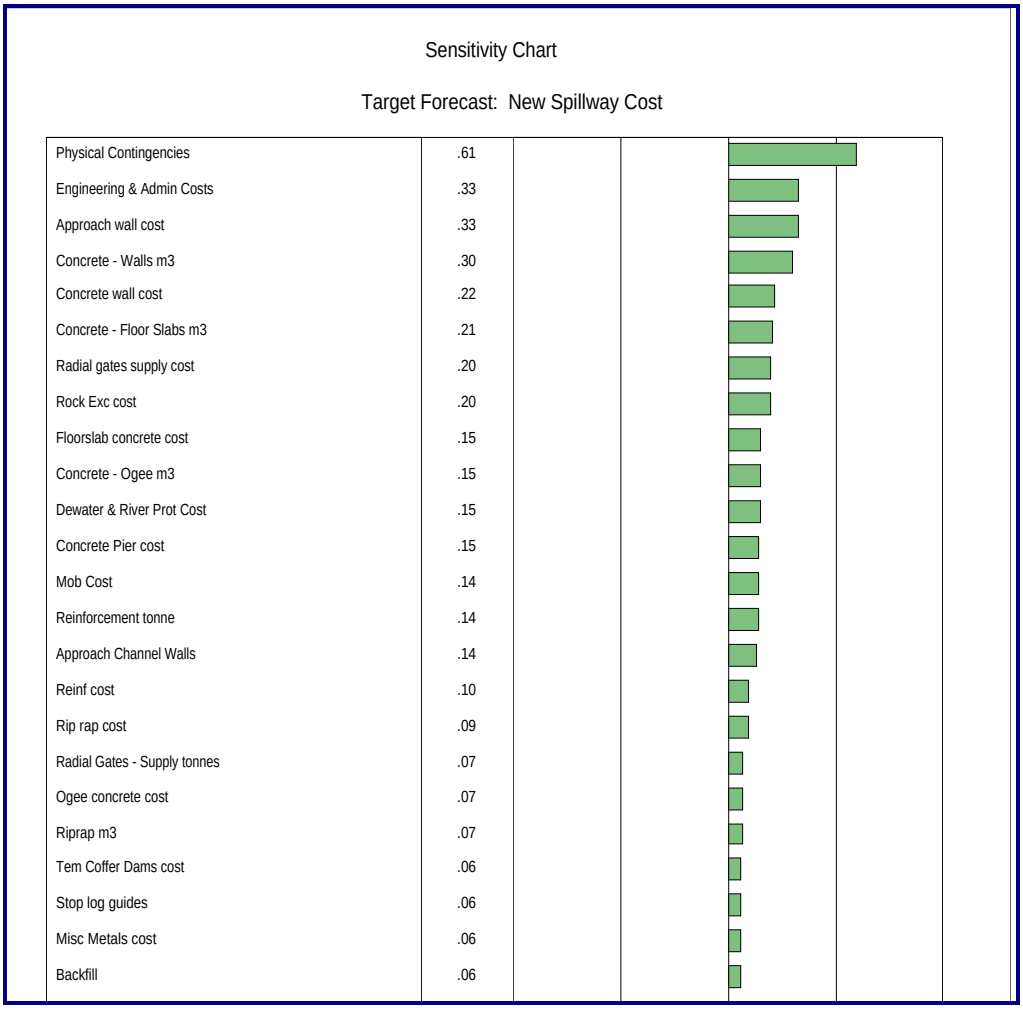


Figure 3-34: Cost Estimate Sensitivity Ranking for New Spillway



In addition to the new spillway costs, preliminary estimates were also developed for raising Mindi Dike and construction of a new dike across the Mojinga Swamp to prevent overtopping during the Canal Operational Flood. The results from the tailwater modeling were taken and a 1.5 m (5 ft) freeboard was added (as was done for the original Mindi Dike) to determine the required heights for the dikes. A dike top width of 6 m (20 ft) and sideslopes of 4/1 were assumed (also based on the existing Mindi Dike). A construction cost of $US 7.5/cu. m was assumed based on use of excavated material from the spillway construction and past projects in Panama. Table 3-31 outlines the estimated costs.

Table 3-31: Cost Estimate for Potential Raising of Mindi and Mojinga Dikes

DOWNSTREAM DIKE CONSTRUCTION COSTS
Use 1/3 PMF Storm to Design Protection for Dikes and Spillway Design for El 92.0

Mindi Dike Elev. (ft) =	25		Mojinga Dike Elev. (ft) =	3
Mindi Dike Length (ft) =	7000		Mojinga Dike Length (ft) =	5800
Existing Top of Ground (ft) =	3		Existing Top of Ground (ft) =	3
Design Freeboard (ft) =	5	(Based on Existing Mindi Dike)		
Design Top Width (ft) =	20	(Based on Existing Mindi Dike)		
Design Sideslope =	4	:1		
Estimated $/yd Const. Cost =	$ 5.75			

MOLL	Peak Q (cfs)	Tailwater Elev. @ Mindi Dike (ft)	Design Top of Mindi Dike (ft)	**Cost to Raise Mindi Dike**	Tailwater Elev. @ Mojinga Dike (ft)	Design Top of Mojinga Dike (ft)	**Cost to Raise Mojinga Dike**
87.5	247,850	28	33	**$ 2,720,000**	24	29	**$ 3,980,000**
88	285,000	29	34	**$ 3,110,000**	24	29	**$ 3,980,000**
88.5	285,000	29	34	**$ 3,110,000**	24	29	**$ 3,980,000**
89	285,000	29	34	**$ 3,110,000**	24	29	**$ 3,980,000**
89.5	285,000	29	34	**$ 3,110,000**	24	29	**$ 3,980,000**
90	285,000	29	34	**$ 3,110,000**	24	29	**$ 3,980,000**

The cross-currents that could be generated if Mindi Dike were to overtop would be significant. Hence consideration should be given to raising the Dike so that navigation approaches to Gatun Locks are not impacted by the project. The construction of a protective dike at Mojinga is a more related to the acceptance of the risk of overwashing Fort Sherman Road near Limon Bay. Recent flooding events with existing spillway discharges up to 5,100 cms (180,000 cfs) have not caused any reported problems along Fort Sherman Road, so the dense vegetation may be helping contain flows within the floodplain. If a decision is made to construct a dike at Mojinga, it should be constructed as near to Gatun Road as possible to limit wetland impacts in the large floodplain.

Since the raising of Mindi Dike is recommended as part of the new spillway project, its cost should be included in the overall estimate for the new spillway. Therefore, the total cost for the new spillway is $US 94.1 million.

3.9 Project Implementation

3.9.1 Project Schedule

As stated previously, it appears that the construction of this project could be accomplished in two years. As for implementation of the entire project, it is estimated that it would take 52 months or 4.33 years to complete the entire project, as indicated in Figure 3-35. Rehabilitation of the existing spillway would follow commissioning of the new spillway to ensure that adequate spillway capacity is available during the retrofit.

3.9.2 Construction Sequencing

The preconstruction phase of the new spillway project will take about two years to complete and the sequencing of major activities is shown on Figure 3-35. Design, fabrication and delivery of the spillway gates is a time-critical item that would take at

least one year. Depending on the manufacturer, the gates would probably be shipped in sections and erected at the site. A construction laydown close to the site would be needed.

Protection of Gatun Dam during construction is major concern, so early in the program steps would be taken to isolate the construction site from the dam. The concrete secant pile retaining/seepage cutoff walls would be constructed on the upstream side of the dam and any local ground improvement would be carried out. A temporary sheetpile cofferdam would be constructed across the approach channel between the walls to isolate the spillway excavation from the reservoir. Because the rock elevation is high upstream of the spillway excavation most of the cofferdam would be formed by the Gatun series of sedimentary rocks. The east perimeter of the spillway site which abuts the dam would be stabilized by local ground improvement with temporary support as needed. Further geotechnical investigations are needed to determine the complete scope of this work.

The site for the spillway ogee and discharge channel would be excavated using bulldozers (with rippers for the rock), excavators and scrapers. Spoil would be hauled off-site to disposal sites (namely Mindi Dike). The spillway control structure foundation would be prepared by trimming and slush concreteing and a grouting program would be carried out along the spillway axis. Drains would later be drilled downstream of the grout curtain.

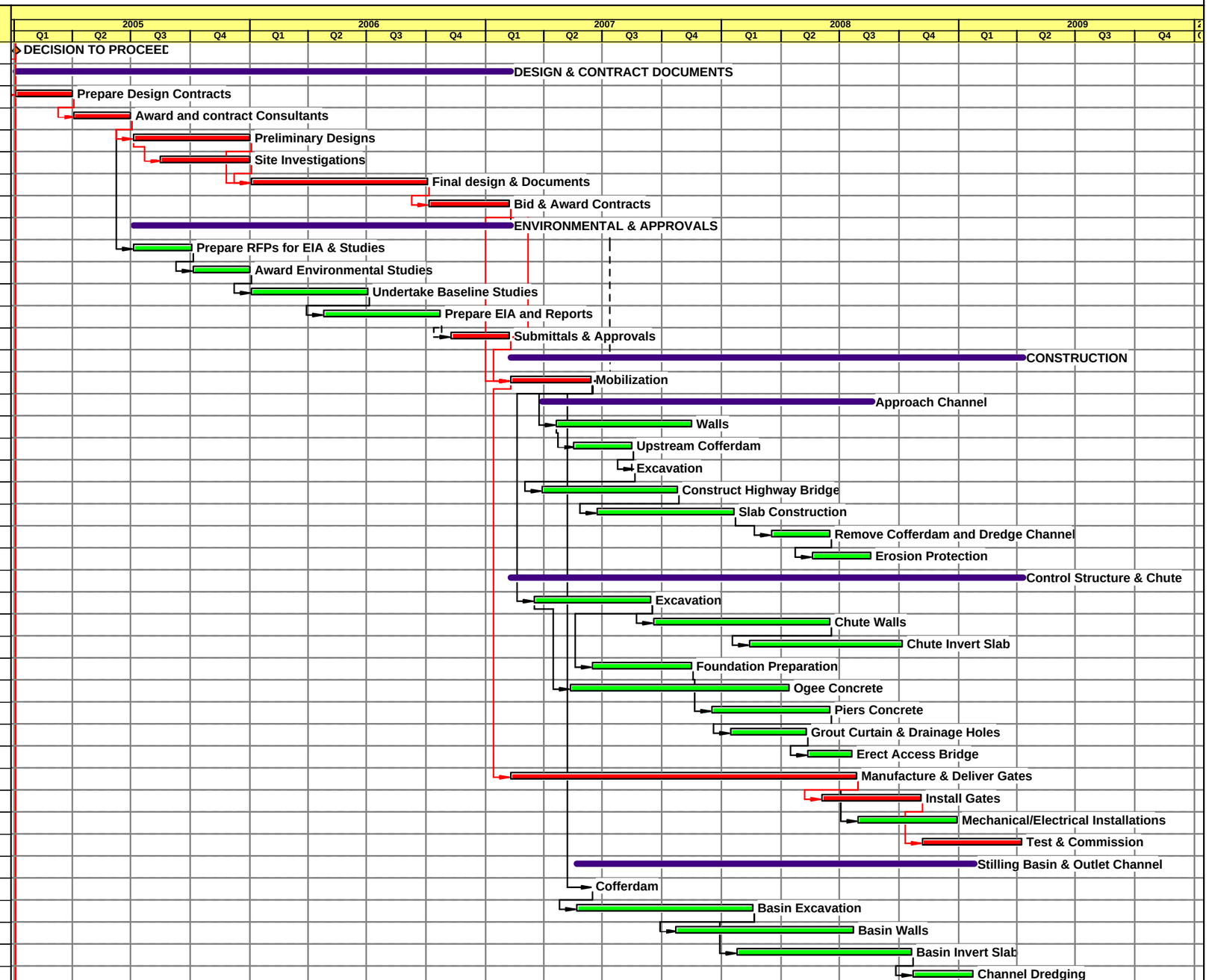
The concrete gravity walls at the sides of the spillway discharge channel would be constructed and the reinforced floor slab would be anchored to rock.

The stilling basin area would require a partial cofferdam at the downstream end that would be constructed using temporary sheetpiling. The stilling basin would be excavated to rock and the sidewalls and concrete floor slab would be constructed in the dry. The cofferdam would be removed and the exit of the stilling basin would be dredged to the required to hydraulic profile and protected with riprap as necessary.

When the spillway structure has been completed and the gates installed the approach channel upstream of the spillway will be flooded and the temporary cofferdam across the approach channel will be removed. The floor of the approach channel between the retaining/cutoff walls would be dredged out to complete the channel.

FLOOD MITIGATION PROGRAM FOR GATUN LAKE

Activity ID	Description	Orig Dur	Early Start	Early Finish
1000	DECISION TO PROCEED	0	03JAN05	
1010	DESIGN & CONTRACT DOCUMENTS	109w *	03JAN05	08FEB07
1030	Prepare Design Contracts	13w	03JAN05	01APR05
1040	Award and contract Consultants	13w	04APR05	01JUL05
1050	Preliminary Designs	26w	05JUL05	02JAN06
1060	Site Investigations	20w	16AUG05	02JAN06
1070	Final design & Documents	39w	03JAN06	03OCT06
1080	Bid & Award Contracts	18w	04OCT06	08FEB07
1140	ENVIRONMENTAL & APPROVALS	83w *	05JUL05	08FEB07
1150	Prepare RFPs for EIA & Studies	13w	05JUL05	03OCT05
1160	Award Environmental Studies	13w	04OCT05	02JAN06
1170	Undertake Baseline Studies	26w	03JAN06	03JUL06
1180	Prepare EIA and Reports	26w	25APR06	24OCT06
1190	Submittals & Approvals	13w	08NOV06	08FEB07
1020	CONSTRUCTION	112w *	09FEB07	10APR09
1090	Mobilization	18w	09FEB07	14JUN07
1100	Approach Channel	72w *	30MAR07	20AUG08
1110	Walls	30w	20APR07	16NOV07
1120	Upstream Cofferdam	13w	18MAY07	17AUG07
1130	Excavation	0	20AUG07	17AUG07
1200	Construct Highway Bridge	30w	30MAR07	26OCT07
1210	Slab Construction	30w	22JUN07	22JAN08
1220	Remove Cofferdam and Dredge Channel	13w	19MAR08	17JUN08
1230	Erosion Protection	13w	21MAY08	20AUG08
1240	Control Structure & Chute	112w *	09FEB07	10APR09
1250	Excavation	26w	16MAR07	14SEP07
1260	Chute Walls	39w	17SEP07	17JUN08
1270	Chute Invert Slab	34w	13FEB08	08OCT08
1280	Foundation Preparation	22w	15JUN07	16NOV07
1290	Ogee Concrete	48w	11MAY07	15APR08
1300	Piers Concrete	26w	17DEC07	17JUN08
1310	Grout Curtain & Drainage Holes	17w	16JAN08	13MAY08
1320	Erect Access Bridge	10w	14MAY08	23JUL08
1330	Manufacture & Deliver Gates	76w	09FEB07	30JUL08
1340	Install Gates	22w	04JUN08	05NOV08
1350	Mechanical/Electrical Installations	22w	31JUL08	02JAN09
1360	Test & Commission	22w	06NOV08	10APR09
1370	Stilling Basin & Outlet Channel	87w *	21MAY07	26JAN09
1380	Cofferdam	1d	15JUN07	15JUN07
1390	Basin Excavation	39w	21MAY07	20FEB08
1400	Basin Walls	39w	23OCT07	24JUL08
1410	Basin Invert Slab	39w	24JAN08	23OCT08
1420	Channel Dredging	13w	24OCT08	26JAN09



Start date	03JAN05
Finish date	10APR09
Data date	03JAN05
Run date	28OCT04
Page number	1A

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New Spillway Project
Figure 3-35: Implementation Schedule

Early bar
Progress bar
Critical bar
Summary bar
Start milestone point
Finish milestone point

During construction the public road that crosses the centerline of Gatun Dam would be routed around the construction site and temporary fencing would be installed for safety and security. The existing road would be run across the top of the cofferdam upstream of the excavation for the new spillway. The new bridge would then be constructed between the walls of the approach channel and the road would be relocated onto the bridge before the cofferdam is removed.

The power supply to the spillway will be via a new distribution line to be constructed from existing lines near the existing spillway to the new spillway.

Upon completion, the spillway would be commissioned and tested to ensure that all electrical and mechanical systems are operational and to observe the hydraulics of flow through the structure.

3.10 Physical Model Recommendations

For the new spillway, the three main issues that may benefit from hydraulic modeling are:

- The approach flow
- Stilling basin
- Outflow channel

3.10.1 Approach Channel. Weir and Spillway Chute

The location of the spillway in a small inlet at the left abutment of the dam necessitates a curved and somewhat shallow approach with high velocities upstream from the control weir. Therefore, flow distribution across the weir, discharge capacity, turbulence, air entrainment and wave generation will all be important considerations that must be explored. Physical hydraulic modeling can show the flow paths to the spillway and the velocity distribution. As a result of model testing the size or shape of the approach channel may be modified to improve flow conditions, address potential erosion or reduce excavation and construction costs.

3.10.2 Stilling Basin

Because there would be two spillways of approximately equal capacity the range of tailwater levels that each must handle at full capacity is larger than that for a single spillway. At full capacity, tailwater elevation can vary from El. 7.0 m (23 ft.) to El. 14.0 m (46 ft.)

Due to the relatively low head and high flow, the estimated Froude Number of the incoming jet to the stilling basin is 4.9 which is near the lower limit for jump stability with a USBR Type II Stilling basin. Model testing of the basin is advisable and would probably result in some specific improvements to the design that could improve performance and/or reduce costs.

At the detailed design phase of the project, consideration should be given to using a sloping apron (or Type V) basin to better address the wide range in tailwater elevation while reducing the amount of rock excavation required by a horizontal apron at this specific site. Model testing would help to identify the design parameters for the depth, slop and length of the apron.

3.10.3 Outflow Channel

The impact of the new spillway on tailwater level downstream of Gatun Dam and flow in the lower Rio Chagres to the Caribbean Sea should also be explored. Improvements to the hydraulics of the river channel and floodplain could be explored to reduce tailwater levels and inundation levels downstream of Gatun Dam.

3.11 Additional Study Needs

Additional study needs currently envisioned for the next phases of the spillway project include:

- Detailed hydrologic study to confirm the magnitude of inflow design hydrograph
- Routing of the inflow design flood accounting for routing through Madden and Gatun Lakes and both the new and existing spillways.
- Environmental scoping and field studies
- Detailed Geotechnical investigations
- Additional topographic coverage of the west perimeter of the site.
- Permeability testing of the sub soils and existing dam
- Materials investigations to identify sources of concrete aggregate, and riprap.
- Identification of spoil disposal sites.
- Physical hydraulic modeling described above
- Optimization of spillway configuration and alignment.
- Investigations of the lower Rio Chagres and floodplain areas.
- Investigations of the Mindi Dyke and Mojinga Swamp areas.

3.12 Summary & Recommendations

3.12.1 Summary

Basis of Need

Additional spillway capacity is required at Gatun Lake to meet modern dam safety requirements for Gatun Dam. Additional spillway capacity would also enable ACP to increase the maximum operating level of the lake and reduce the flood surcharge that must be maintained to pass large floods.

The capacity of the new spillway under the current operating regime is 5912 cms (208,780 cfs) at the current maximum operating level of El. 26.67 m (87.5 ft) PLD and the estimated maximum discharge at El. 27.43 (90 ft.) is 6693 cms (236,360 cfs). The estimated maximum flow under PMF routing assumptions El. 28.04 m. (92.0 ft) PLD is 7303 cms (257,910 cfs).

Establishment of PMF

The computations used for this report indicate that the current PMF flood appears to be only four times greater than a reported 15-year event. Given that the current PMF was developed in the late 1970's and over 30 years of additional hydrologic data is now available, it is recommended that the PMF be re-evaluated prior to the design stage, in order to verify its applicability.

Alternatives

Study of alternative locations for new spillway have determined that the most promising site is at the left (west) side of the dam where the embankment dam abuts a natural ridge that is comprised of Gatun series sedimentary rocks covered with a mantle of natural soil and embankment fill.

The proposed spillway arrangement is for a gated overflow spillway with seven crest gates that are twice as high as the gates on the existing spillway. The gates would be of the radial gate design instead of the present vertical lift gates on the existing spillway.

Recommended Configuration

Development of additional spillway capacity at the west abutment of Gatun dam appears to be feasible.The recommended project configuration is for a gated overflow surface spillway with a short approach channel, discharge channel and concrete lined stilling basin.

Site Investigation

The site investigation undertaken by ACP for this feasibility study was used to determine the optimum location and configuration of the new spillway structures. However, there

are still some unresolved issues related to the competence and surface profile of the rock layers in the area of the proposed spillway and the stilling basin. More information is also required to map the location of the apparent fault line within the project site.

Hydraulic Modelling

Design of the approach channel, stilling basin and outflow channel would benefit from hydraulic modelling. In addition the potential effects of higher tailwater on the safety and operation of the existing spillway should be explored in conjunction with recommended hydraulic modeling of operation under existing conditions.

Potential Design Refinements

Design refinements that should be explored at the detailed design phase, when additional site investigation data is available include:

- Rotating the spillway counterclockwise to improve the approach conditions.
- Reduce the length of training wall on the left (west side of the spillway channel).
- Use of ground improvement techniques to improve the stability of embankment fill.
- Move the control structure upstream to reduce the cost of the training walls
- Trim down the size of gravity walls on the discharge chute immediately downstream of the control structure.
- Explore the use of secant pile walls on the discharge channel and stilling basin to reduce the need for temporary cofferdams and ground support.
- Optimize the hydraulic and structural design of the spillway gate structure.
- Optimize the size and number of spillway gates.
- Optimize the stilling basin design. If feasible, a sloping apron Type V Basin may prove to be superior to Type II design.

Construction Schedule & Costs

The spillway would take at least two years to construct and is estimated to cost approximately $US 91.0 million. The full project implementation schedule is currently estimated to be 52 months or 4.33 years including environmental studies, additional field investigations and final design. Raising of Mindi Dike is also recommended to limit impacts to navigation during the Canal Operational Flood. This construction is estimated to cost on the order of $US 3.1 million. Therefore, the total cost for the new spillway project is estimated to be $US 94.1 million. If a dike is required to protect Gatun Road across Mojinga Swamp, this would incur an additional cost of $US 4.0 million.

4 LAKE GATUN MAXIMUM OPERATING LEVEL

4.1 Introduction

Increasing the spillway capacity will also allow the operation of the Lake at a higher elevation without compromising safety of structures and navigation. It would also increase the water yield from the Canal Lake system. Therefore, as part of the evaluation of a new spillway, consideration was also made to evaluate the optimal maximum operating lake level (MOLL) for Gatun Lake utilizing both the existing and proposed spillways.

As shipping traffic continues to increase and ships become larger, the current Canal system will not be able to meet projected demands. As evidenced during the El Nino phenomenon of 1997-1998, the Canal is already vulnerable to water shortages during extreme droughts. ACP has completed numerous past studies to investigate the costs of developing additional water supplies from adjacent watersheds most notably at Rio Indio. However, an additional way to provide additional water supply for Gatun Lake is to raise the maximum and average operating lake level so that additional water volume is captured for municipal, industrial and lockage water demands. This section considers the impacts and benefits of raising the maximum operating level of Gatun Lake once the new spillway has been constructed. The first step to be completed for this study was to determine the range of maximum operating lake levels that are feasible and that would be practicable to study.

4.2 Range of Maximum Operating Lake Levels Considered

The first step to be completed for this study was to determine the range of maximum operating lake levels that are feasible and that would be practicable to study.

ACP has eminent domain within the Canal watershed and over elevations below 30.48 m (100 ft) around the lake shoreline. However, after a field investigation was completed to develop an inventory of structures around the lake which would be impacted by a rise in lake water levels, it became apparent that the existing locks would be the limiting factor in raising the lake level. With a coping elevation of El 28.04 m (92.0 ft) and an elevation of 26.88 m (88.2 ft) for the top of the guard gate strut slot opening for the miter gates, almost any rise in maximum water level within the lake would require modifications to the existing locks.

4.2.1 Wind/Wave Study

Clearly one of the principal concerns related to an increased MOLL will be the effect of wind driven waves or ship/vessel wash overtopping the lock gates, adjacent structures or flooding machinery chambers through drainage or ventilation shafts now above the

existing maximum water level. A detailed wave runup and overtopping analysis was therefore completed using the top of the coping and lock gates as critical elevations at the Gatun and Pedro Miguel Locks. A summary of the analysis is provided in the following sections, and a detailed description of the analysis and findings can be found in Appendix G.

According to studies provided by ACP (PMF Study, Flood Control Manual), the design waves heights at Gatun are estimated to be 0.91 m (3.0 feet), while at Pedro Miguel they are estimated to be 0.34 m (1.1 ft). Preliminary calculations using the values at Gatun (0.91 m – 3.0 ft wave) showed that even raising the MOLL by 0.23 m (0.75 ft) would be problematic for workers on top of the lock wall. After reviewing the wind summaries at the Gatun station (which show winds predominantly from the north) and the importance of this design criterion to the overall study, it was decided that a revisiting of the design wave height to be used at Gatun was warranted. A similar review of the wind data for Pedro Miguel showed that the proposed design wave height of 0.34 m (1.1 ft) was reasonable.

In order to estimate the appropriate design wave height and overtopping design criteria at Gatun, it was necessary to hindcast wave data for the location of interest which in turn required an analysis of the winds experienced at Gatun.

4.2.2 Wind climate

Historical Observations

The “Green Book” study by the USACE (USACE, 1979) utilized wind data from gages at Balboa Heights, Cristobal, and Madden Dam. Design conditions for Gatun Locks were based on records for Cristobal presented in Hydrologic Studies published in 1947 and 1968. Based on the records from 1909 to 1947, the highest average wind velocities are experienced in the four month dry season (mid December to mid April), when northeast trade winds are predominant. The prevailing wind direction at Cristobal was from the north to northwest.

The USACE defined wind design conditions for wave generation at Gatun Lock by a wind velocity of 41 miles per hour (mph) and duration of 41 minutes (Green Book, 1979). However, during the period from 1907 to 1946, the mean annual wind velocity was 10 miles per hour (mph) with a prevailing wind direction from the north. During the same time period, the maximum 30-minute and 1-hour winds were 33 and 31 mph respectively, both prevailing from the north.

Based on the data identified above, it became apparent that the design conditions and criteria presented by the USACE did not account for the variability in direction and frequency of winds. Thus, the magnitude, duration and direction of winds were evaluated to determine appropriate design criteria.

Records from Gatun (1985 to 2003)

Wind data presented in 1947 and 1968 reports were not available for further analysis; therefore sample data from the ACP station at Gatun for the period from 1985 to 2003 was used for this investigation.

The monthly average instantaneous wind speed (from measurements taken at 20 second intervals) was 3.8 mph (1.7 m/s) with the prevailing wind direction consistently from the north to northwest for the majority of months. Average monthly winds prevailed from the south to southeast (waves generated in this direction would directly impact Gatun Lock) repeatedly during the wet season months of September and October from 1995 to 1999. The highest average monthly wind speed, with a corresponding average direction from the south southeast (SSE; 160 degrees), was 4.3 knots (1.9 m/s); this was recorded during July 1997.

Frequency distributions for average hourly and peak instantaneous (peak hourly measured at 20-second intervals) were developed by ACP for the Gatun station (Tables 4-1 and 4-2). Wind roses for the average and peak hourly data illustrate the frequency, magnitude, and direction of winds at Gatun (Figures 4-1 & 4-2).

Table 4-1: Frequency Distribution for Average Hourly Winds @ 20s interval

Direction	Calm	1-3 knots	4-6 knots	7-10 knots	11-16 knots	Total
N – ENE		16.69%	16.39%	5.26%	0.02%	38.35%
WNW - NNW		3.32%	2.00%	0.54%	0.01%	5.86%
E		0.97%	0.09%	0.01%		1.06%
ESE		1.54%	0.15%	0.03%		1.72%
SE		2.67%	0.66%	0.07%		3.40%
SSE		2.97%	1.15%	0.18%	0.00%	4.30%
S		3.50%	1.39%	0.31%	0.01%	5.22%
SSW		2.75%	0.51%	0.05%	0.00%	3.31%
SW		1.64%	0.39%	0.03%		2.07%
WSW		1.23%	0.39%	0.04%		1.66%
W		1.95%	1.10%	0.32%	0.00%	3.37%
Subtotal E - W		***19.21%***	***5.83%***	***1.05%***	***0.02%***	***26.10%***
Total	**7.8%**	**50.85%**	**31.85%**	**9.84%**	**0.05%**	**100.00%**
Cumulative P < Uobs	**7.8%**	**58.3%**	**90.1%**	**99.9%**	**100.0%**	

Table 4-2: Frequency Distribution for Peak Hourly Winds @ 20s interval (Knots)

Direction	Calm	1-3 knots	4-6 knots	7-10 knots	11-16 knots	17 – 21 knots	➢ 21 knots	Total
N –ENE		5.71%	6.37%	11.82%	14.69%	1.96%	0.11%	40.66%
NNW		0.94%	1.71%	2.23%	1.44%	0.07%	0.00%	6.39%
E		0.65%	0.44%	0.20%	0.07%	0.00%	0.00%	1.37%
ESE		0.68%	0.77%	0.38%	0.12%	0.01%	0.00%	1.96%
SE		0.65%	1.33%	1.05%	0.50%	0.02%	0.00%	3.56%
SSE		0.55%	1.67%	1.59%	0.63%	0.04%	0.00%	4.48%
S		0.60%	2.23%	1.73%	0.83%	0.07%	0.01%	5.47%
SSW		0.48%	1.77%	1.06%	0.27%	0.02%	0.00%	3.60%
SW		0.45%	1.04%	0.64%	0.22%	0.01%	0.00%	2.37%
WSW		0.42%	0.70%	0.61%	0.27%	0.02%	0.00%	2.03%
W		0.58%	1.08%	1.22%	0.81%	0.05%	0.01%	3.73%
Subtotal E – W		***5.08%***	***11.02%***	***8.49%***	***3.72%***	***0.24%***	***0.03%***	***28.58%***
Total	**0.4%**	**15.6%**	**25.1%**	**29.6%**	**26.3%**	**2.9%**	**0.2%**	**100.00%**
Cum. P < Uobs	**0.4%**	**16.0%**	**41.1%**	**70.7%**	**96.9%**	**99.8%**	**100.0%**	

During the period from 1985 to 2003, the average hourly winds were directed from between the east-north-east (ENE; 67.5 degrees) and west-northwest (WNW; 292.5 degrees) approximately 66.5% of the time. Average hourly winds that may result in wave generation from the south occurred for only 26.1% of the duration. Winds were calm 7.4% of the time.

Average hourly recorded wind speeds were less than 3 knots (1.54 m/s) for greater than 58% of the period. The maximum wind speed, with a minimum of one hour duration, was between 11 to 16 knots (5.7 to 8.2m/s). Average hourly winds observed at this speed (5.7 to 8.2m/s) were measured from the east-southeast to the west-southwest for a total of 26 hours for the 19 year period, less than a 0.02% occurrence.

Maximum hourly recorded wind speeds were less than 10 knots (11.5 m/s) for greater than 70% of the period. The maximum instantaneous wind speed from the south was 33 knots (17 m/s). The peak hourly wind speed (20 second interval) exceeded 21 knots (10.8 m/s) less than 0.2% of the time. Peak hourly winds observed at a speed greater

than 10.8 m/s were measured from the east-southeast to the west-southwest for a total of 13 hours during the 19 year period, less than a 0.03% occurrence.

Figure 4-1: Wind Rose Average Hourly Wind Speed at Gatun (1985 to 2003)

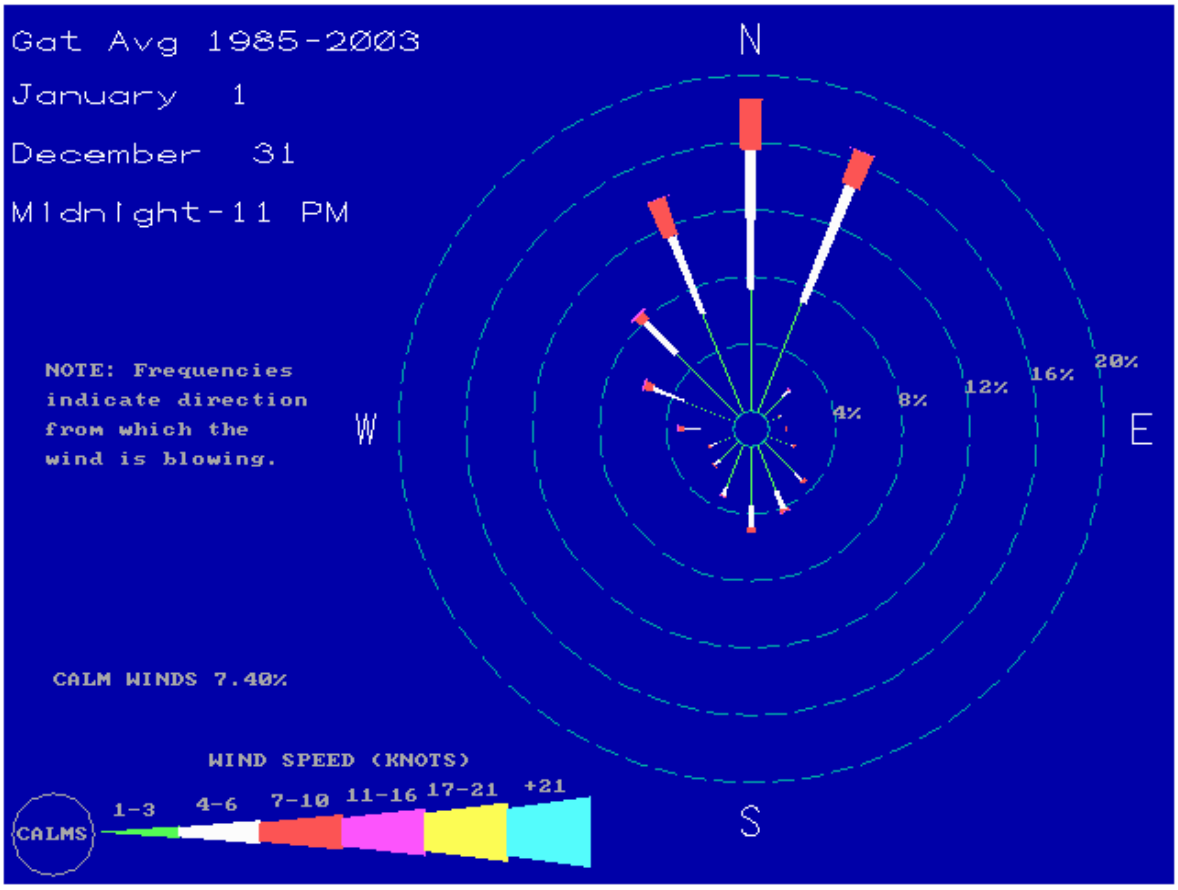
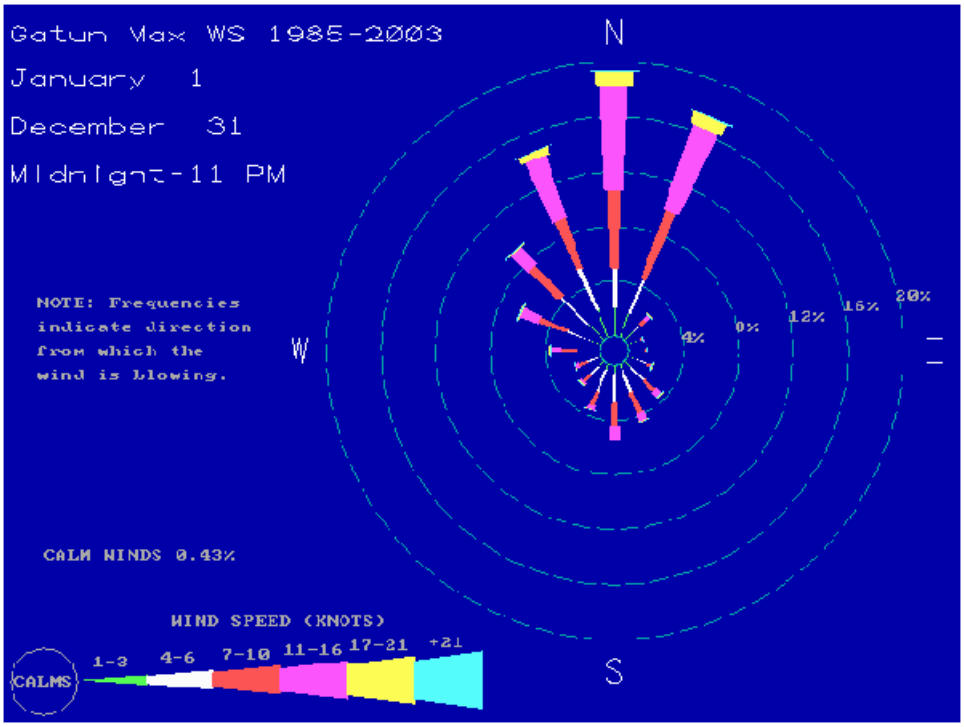


Figure 4-2: Wind Rose Peak Hourly Wind Speed at Gatun (1985 to 2003)



4.2.3 Wave Hindcast

The above wind records from the Gatun station for the period from 1985 to 2003 were then used to hindcast wave data at the Gatun Locks. Methods presented in the Automated Coastal Engineering Software (ACES) application and the Coastal Engineering Manual (CEM) were employed to estimate the wave height for restricted fetches. For both of these methods, wave height is estimated as a function of the observed wind velocity, fetch length, and duration using empirical methods. Site specific wave data was not available at Gatun Lake for calibration.

Several assumptions applicable to the methods applied for estimating wave growth as a function of wind speed include:

- Energy from the presence of other wave trains is neglected
- Relatively constant wind speed and direction prevail and will result in wave heights that are constant through time but will vary along fetch
- Fixed value of drag
- Duration for fetch limited growth to occur was derived assuming a JONSWAP spectrum

- Observed wind speeds are corrected for height of observation to 10 m (from 25 m) and also for limited fetch (less than 16 km) following the guidelines in the ACES Manual

It is important to note that wind transformation from land to water is highly site specific and it is recommended that observations be made at the locks to calibrate or check the accuracy of the wave height approximations.

Figure 4-3 is a fetch diagram for Gatun Lake illustrating the irregular nature of the fetch in the vicinity of the Gatun locks. The fetch is restricted by islands located both to the southeast and southwest. The maximum fetch is 10.2 km, extending due south. Wind-induced waves will propagate in the direction which the fetch parallel component of the wind velocity vector (Ua) will be a maximum.

A spreadsheet tool was developed to perform the computations using formula identified in the ACES Users Manual (USACE, 1992). Deep water restricted fetch conditions were deemed appropriate given the relative water depth at the location of interest. The restricted fetch deep water formulas are applied to estimate wave height (H) and period (T) under both duration and fetch limited conditions and are shown in Appendix G.

Figure 4-3: Fetch Diagram for Estimating Wave Heights at Gatun Lake

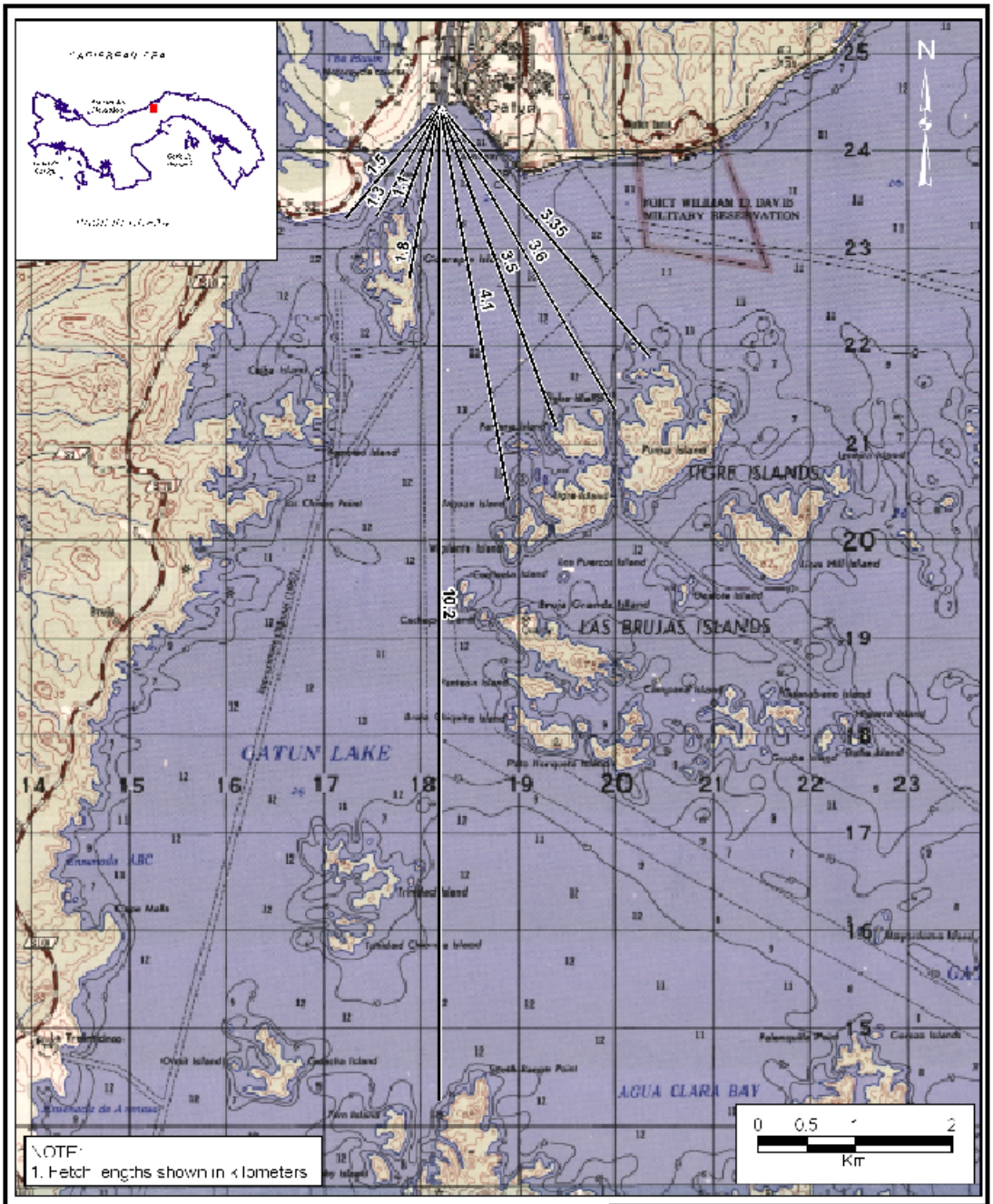


Figure 3. Fetch Diagram at Gatun Lake

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Wave Heights Based on Records from Gatun 1985 to 2003

Following procedures in the ACES manual, both fetch and duration limited conditions were investigated to determine the appropriate design wave height to use at Gatun locks. For a detailed explanation of this portion the study, see Appendix G.

Table 4-3 provides a summary of design scenarios evaluated using the ACES program and associated probability of occurrence based on the 19 years of recorded wind velocity. Scenarios 1,2, and 3 represent wave heights generated based on the average hourly data assuming fetch limited conditions. Scenario 4 is based on the peak hourly 20-s measurement data. It is conservatively assumed that fetch limiting conditions apply.

Table 4-3: Summary of Estimated Wave Heights and Frequency

Scenario	Wave Height (m)	Wave Height (ft)	Probability Greater Than or Equal To H[22]
1. Fetch limited	0.05	0.16	14.06%
2 Fetch limited	0.10	0.33	0.57%
3. Fetch limited	0.37	1.21	< 0.01%
4. Unrestricted Fetch, U_{obs}= 33 knots, Duration (0.5 hr), F=10.2km	0.26	0.85	<0.01%

Therefore, based on sample data from 1985 to 2003 it appears that for day to day operations, design wave heights from 0.01 to 0.05 m are appropriate for waves coming from the south. Wave heights on the order of 0.1 m to 0.37 m should be considered for extreme events where waves are coming from the south. These conclusions were verified during interviews with ACP personnel who stated that maximum wind wave heights observed during the last 20 years at Gatun were approximately 0.31 m (1.0 ft).

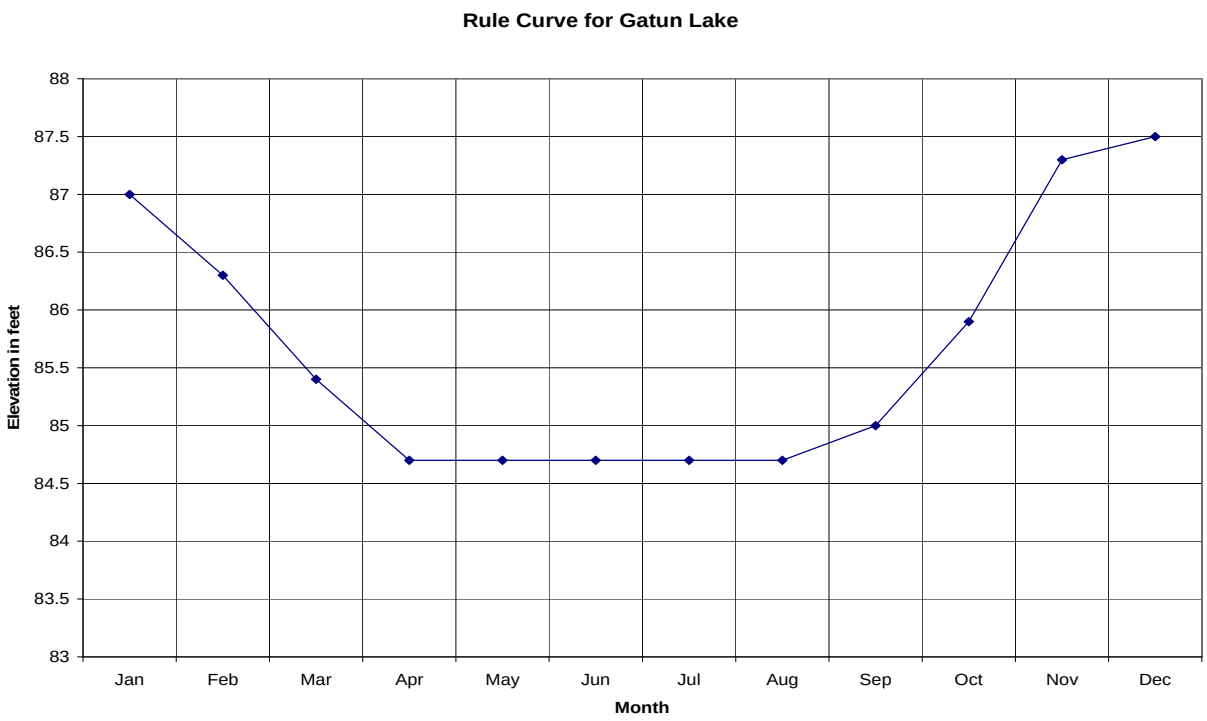
Both sample error and the uncertainty introduced in application of empirical methods should be considered in the selection of representative design wave heights. The estimates for wave height have a high level of uncertainty. It is recommended that observations be made at the locks to calibrate or check the accuracy of the wave height approximations during later phases of the design process.

[22]Based on sample data from 1985 to 2003. Does not account for joint probability of water surface elevation being at MOLL.

Joint Probability

The estimated frequency identified in Table 4-3 is independent of the water surface elevation. In evaluating wave overtopping and runup it is necessary to also consider the variability in water surface elevation. Gatun Lake will be at the MOLL for only a short period immediately after the rainy season. For example, during the 19 year period, the estimated maximum wind-induced waves would have occurred during the months of July and September (1995 to 1999). The target operating elevation for the water surface is 25.82 m (84.7 ft) Precise Level Datum (PLD) and 25.91 m (85 ft) PLD during the rainy months of July and September, respectively (Figure 4-4). Therefore, the joint probability of occurrence for high wave climate and high water levels would be significantly lower than that reported in Table 4-3, and the assumption of complete independence would lead to a very conservative design.

Figure 4-4: Existing Rule Curve for Gatun Lake



4.2.4 Wave runup and overtopping

Wave overtopping is a key design consideration in assessing the feasibility of raising the Maximum Operating Lake Level (MOLL) of Gatun Lake. Excessive wave overtopping may preclude workers from safely operating on the walkway outside of the lock or may result in damage to the structure.

Design Criteria

A literature review was conducted to identify acceptable criteria for average overtopping and peak discharge conditions. Average mean overtopping discharge and peak individual wave discharge were considered. Table 4-4 summarizes the range of criteria cited in the references. There is a large variance between the criteria cited in the Coastal Engineering Manual (CEM) (USACE, Draft 2002)[23] and the Design and Assessment Manual for Wave Overtopping (Wallingford Ltd)[24] and the design criteria recommended by Smith et. al(1994)[25]. For this preliminary assessment, the more stringent criteria were assumed to apply.

Table 4-4: Design Criteria for Wave Overtopping

Reference	Average Overtopping Discharge ($m^3/s/m$)		Peak Overtopping (m^3/m)
	Pedestrians	*Structures*	*Pedestrians*
CEM (USACE, Draft 2002) from Simm (1991)	1.0×10^{-4} to 9×10^{-4}; dangerous on vertical breakwaters	2×10^{-3}; start of damage to embankment seawalls if crest not protected 3×10^{-5}; start of damage to buildings	-
Smith et. al. (1994)		1×10^{-2}	1.6
HR Wallingford (1999)	Same as CEM	Same as CEM	0.04

Methods

Overtopping of a structure will occur as a result of waves running up on the face of the lock wall. Two empirical approaches were applied to estimate average wave overtopping. A general method presented in ACES (1991)[26] was followed that is applicable to both sloping and vertical structures. The second method was developed by

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[23]US Army Corps of Engineers. Draft 2002. “Coastal Engineering Manual."

[24]HR Wallingford Ltd, February 1999. Wave Overtopping of Seawalls, Design and Assessment Manual, R&D Technical Report W178.

[25]Smith, G.M., Seiffert J.W.W and van der Meer J.W. 1994. “Erosion and Overtopping of a grass Dike, Large Scale model Tests.” International Conference on Coastal Engineering, Kobe, Japan.

[26]United States Army Corps of Engineers, 1991. Automated Coastal Engineering System (ACES) User’s Guide.

Allsop et al (1995)[27] and is applicable specifically to vertical walls in reflecting wave environments.

ACES Methods for Estimating Wave Runup and Average Overtopping

The empirical methods presented for wave runup and overtopping in ACES were derived for smooth impermeable structures. Run-up and overtopping are estimated independently for monochromatic waves or jointly for irregular waves. Irregular waves are represented by a significant wave height and assumed to conform to a Rayleigh distribution; overtopping is estimated as a summation of the contribution for individual runups.

Wallingford Ltd Design Manual Methods for Estimating Average Overtopping

The Wallingford design manual recommends an empirical method derived by Allsop et al (1995). The Allsop et al (1995) method differentiates between deep water and shallow water waves. In deepwater conditions, the waves are observed to reflect back from the seawall, whereas in shallow water the waves are prone to break over the structure. Allsop et al (1995) defined a parameter used to determine whether waves will be in reflecting mode:

For a more detailed explanation of these methodologies, see Appendix G.

Estimates of wave runup and overtopping

Wave runup and overtopping were evaluated for three design scenarios, applying the methods discussed above. Scenarios, defined in Table 4-5, are labeled consistently with Table 4-3. As discussed previously, all three scenarios may be characterized as extreme events based on the record from 1985 to 2003.

[27] Allsop N W H, Besley, P and Madurni, L. May 1995. “Overtopping performance of vertical and composite breakwaters, seawalls and low reflection alternatives.” Paper to the final MCS Project Workshop, Aldernerney.

Table 4-5: Description of Scenarios with Estimated Wave Runup

Scenario	Wave Height (m)	Wave Height (ft)	Assumed Period (s)	Wave Runup (m)	Wave Runup (ft)	Probability Greater Than or Equal To H[28]
2. Restricted Fetch, U_{obs}= 16 knots, Fetch limited	0.10	0.33	1.5	0.11	0.36	0.57%
3. Restricted Fetch, U_{obs}= 16 knots, Fetch limited	0.37	1.21	2.13	0.42	1.38	< 0.01%

Wave runup on the lock wall was estimated based on the empirical method in ACES program (Table 4-5). The runup is calculated independent of the water surface elevation. The runup for Scenario 2 is less than 0.11 m (0.37 ft) and Scenario 3 is estimated to be 0.42 m (1.38 ft).

Average Wave Overtopping

Tables 4-6 & 4-7 summarize the estimated average wave overtopping for a series of events based on the ACES and Wallingford Ltd methods respectively. Overtopping was estimated for a range of freeboard conditions from 4 feet to 0.50 feet for each of the scenarios.

Applying the ACES monochromatic method for estimating wave overtopping results in no overtopping if the freeboard is greater than 0.11 m, for Scenario 2 (Table 8). Estimates of wave overtopping, using the Wallingford Method, all lie within the range of acceptable tolerance for pedestrians and structural integrity for Scenario 4 (H=0.10 m). The conclusion of no structural damage is based on a simple rule of thumb for typical structures found at the waters edge.

Under Scenario 3 (H=0.37 m), with the application of the ACES method, theoretically there would be no wave overtopping for a freeboard greater than or equal 0.42 m (1.38 ft). Following the Wallingford Method, average wave overtopping is estimated to exceed the tolerance level for both pedestrian safety at a freeboard less than or equal to 0.31m (1.0 ft) and structural damage for a freeboard less than or equal to 0.46 m (1.50 ft).

[28]Based on sample data from 1985 to 2003. Does not account for joint probability of water surface elevation being at MOLL.

Table 4-6: Estimated Average Wave Overtopping - ACES method

Freeboard		Scenario 2 H=0.10 m	Scenario 3 H=0.37 m
(m)	(ft)	(m3/m/s)	(m3/m/s)
1.22	4.00	-	-
0.91	3.00	-	-
0.61	2.00	-	-
0.46	1.50	-	-
0.30	1.00	-	4.00E-03
0.15	0.50	-	1.80E-02

CEM Acceptable Limits:

Structural damage starts at 2E-03 to 2E-02 $m^3/s/m$

Pedestrian risk starts between 1.0E-04 to 9E-04 $m^3/s/m$

Under existing conditions, when the surface water level of Gatun Lake is at the MOLL of 26.67 m PLD (87.5 ft PLD), there is 1.2 m (4 ft) of freeboard relative to crest of the walkway at the lock (27.89 m PLD or 91.5 ft PLD) and 1.37 m (4.5 ft) of freeboard relative to the crest of the lock structure. The average wave overtopping, resulting from H=0.37 to H=0.60 m, would be within the acceptable range for both pedestrians and structures.

Table 4-7: Estimated Average Wave Overtopping - Wallingford Method

Freeboard		Scenario 2 H=0.10 m	Scenario 3 H=0.37 m
(m)	(ft)	(m3/m/s)	(m3/m/s)
1.22	4.00	9.4E-18	3.7E-06
0.91	3.00	4.5E-14	3.7E-05
0.61	2.00	2.2E-10	3.6E-04
0.46	1.50	1.5E-08	1.1E-03
0.30	1.00	1.0E-06	3.6E-03
0.15	0.50	7.1E-05	1.1E-02

CEM Acceptable Limits:

Structural damage starts between 2E-03 to 2E-02 $m^3/s/m$

Pedestrian risk starts between 1.0E-04 to 9E-04 $m^3/s/m$

If the MOLL were raised from 26.67 m (87.5 ft) PLD to 27.43 m PLD (90.0 ft PLD) there would be 0.46 m (1.5 ft) of freeboard between the crest of the walkway at the lock and the water surface at the MOLL; 0.61 m (2 ft) of freeboard would exist between the MOLL and the crest of the lock structure. Under wave conditions of Hs=0.37 to Hs=0.46 m (and water surface at MOLL) overtopping would be within the accepted range as prescribed by the Coastal Engineering Manual for structures.

Based on the above investigations, raising the MOLL of Gatun from 26.67 m (87.5 ft) up to 27.43 m (90 ft) PLD (allowing 0.61 m (2.0 ft) of freeboard to the top of the lock wall), should not result in a high risk of structural damage to the lock during low frequency events. Wave overtopping may put pedestrians at risk but only during extreme events.

Check of boat wake heights

In addition to the above wind waves, an analysis was also completed for boat wakes to determine if boat wakes would exceed wind waves. Guidance given in PIANC was used for the analysis. For the calculation of boat wakes, it is important to remember that there are two separate types of waves generated by ships. The first are primary waves which are generated at the bow and stern of the boat. The second are secondary waves which are generated along side of the ship. The equation for calculating primary wave heights includes the following variables:

- Vessel Speed
- Water Depth
- Channel Cross-sectional Area
- Cross-sectional Area of Ship Underwater

The equation for estimating secondary wave heights includes the following variables:

- Vessel Speed
- Water Depth
- Distance from Ship to Point of Interest
- Ship Shape Coefficient

A model was created to calculate estimated boat wake heights for both primary and secondary waves. For the primary waves, two different conditions were calculated. The first condition consisted of a Panamax ship approaching the knuckle of the lock. The channel width was increased to 45.7 m (150 ft) to account for the angle of the knuckle. This calculation resulted in an estimated primary boat wake wave height of 0.32 m (1.06 ft). Another condition was also modeled to approximate when a Panamax ship is located within the first lock being handled by locomotives. The channel depth is increased to account for the first lock floor elevation and the resulting wave height is calculated to be 0.26 m (0.85 ft). The vessel speed of the ship for both of these cases was assumed to be 1.8 knots based on vessel speeds of ~0.92 m/s (3 ft/s) observed at the Pedro Miguel locks. The wave model was also checked against two ships observed at Pedro Miguel, and the model results matched observed wave heights fairly well. Another case was investigated with tugs approaching the knuckle, but this case resulted in lower wave heights. Table 4-8 shows the results of the model calculations of the controlling conditions.

For secondary wave heights, multiple cases were also run to determine which cases would control. Cases were run for the tugs working alongside the locks with speeds of 5 knots with resulting wave heights of 0.06 – 0.08 m (0.20 – 0.26 ft), as well as out in the

lake where the tug speeds approach 10 knots (based on travel times given in the lockage handbook and ACP personnel experience). For these cases out in the lake, varying distances to shore were assumed from 15.2 – 61 m (50 – 200 ft). The resulting wave heights ranged from 0.37 – 0.59 m (1.22 – 1.93 ft). See Table 4-8 for the model calculations.

Based on ACP concerns, propeller wash waves were also investigated. An exhaustive literature search yielded design procedures for only the estimation of maximum velocities behind propellers, not wave heights. However, ACP personnel at the locks stated that waves of 30 – 40 cm (1 – 1.3 ft) have been observed when ships are headed toward Gatun Lake and the propeller is next to the lock gates (worst case).

Based on the above model calculations and observations, it appears that for day to day operations, design boat wake wave heights up to 0.39 m (1.3 ft – propeller wash – worst case) may be experienced at the locks. Therefore, the design wave height to be used at the locks should be 0.39 m (1.3 ft). However, for other structures within the lake, the wave heights experienced likely approach 0.59 m (1.93 ft).

Table 4-8: Summary of Boat Wake Height Calculations

Estimation of Boat Wake Wave Heights

Based on PIANC,1987

From ACP Lockage Handbook:

Distance in Nautical Miles from Gatun to Pedro Miguel =	27.67	nautical miles
Scheduled Running Time from Gatun to Pedro Miguel =	173	min
Average Speed from Gatun to Pedro Miguel =	**9.6**	knots

Primary Waves (Front and Back of Ship)

Approaching First Lock @ Knuckle

h (Channel Depth) =	48	ft
Channel Width =	150	ft
Channel Area (Ac) =	7200	ft^2
Ship Width =	106	ft
Ship Draft =	43	ft
Ship Blocked Area (As)=	4558	ft^2
As/Ac =	0.63	
Vs (Vessel Speed) =	1.8	knots
Vs^2/2g =	0.0060	ft
(2*z/h)/((1-As/Ac-z/h)^-2)-1) =	0.0060	ft
z =	1.06	ft

Within First Lock

h (Channel Depth) =	68	ft
Channel Width =	110	ft
Channel Area (Ac) =	7480	ft^2
Ship Width =	106	ft
Ship Draft =	43	ft
Ship Blocked Area (As)=	4558	ft^2
As/Ac =	0.61	
Vs (Vessel Speed) =	1.8	knots
Vs^2/2g =	0.0042	ft
(2*z/h)/((1-As/Ac-z/h)^-2)-1) =	0.0042	ft
z =	0.85	ft

Secondary Waves (Side of Ship)

Approaching First Lock @ Knuckle - Tugs Working

h (Channel Depth) =	48	ft
S (Distance From Ship to Point of Interest) =	5	ft
Vs (Vessel Speed) =	5	knots
alpha (Ship Coefficient) =	1.2	
Hi =	0.26	ft

Within First Lock - Tugs Working

h (Channel Depth) =	68	ft
S (Distance From Ship to Point of Interest) =	5	ft
Vs (Vessel Speed) =	5	knots
alpha (Ship Coefficient) =	1.2	
Hi =	0.20	ft

Out in Lake - 200 ft Away - Tugs Travelling

h (Channel Depth) =	48	ft
S (Distance From Ship to Point of Interest) =	200	ft
Vs (Vessel Speed) =	10	knots
alpha (Ship Coefficient) =	1.2	
Hi =	1.22	ft

Out in Lake - 100 ft Away - Tugs Travelling

h (Channel Depth) =	48	ft
S (Distance From Ship to Point of Interest) =	100	ft
Vs (Vessel Speed) =	10	knots
alpha (Ship Coefficient) =	1.2	
Hi =	1.54	ft

Out in Lake - 50 ft Away - Tugs Travelling

h (Channel Depth) =	48	ft
S (Distance From Ship to Point of Interest) =	50	ft
Vs (Vessel Speed) =	10	knots
alpha (Ship Coefficient) =	1.2	
Hi =	1.93	ft

Revised estimates of wave runup and overtopping

Wave runup and overtopping were re-evaluated for multiple design scenarios, applying the methods discussed above. Scenarios were run for wave heights from 0.15 – 0.91 m (0.5 – 3.0 ft) for lake stillwater levels from 26.67 – 27.74 m (87.5 – 91.0 ft) at 0.15 m (0.5 ft) intervals (and also the 0.39 m (1.3 ft) wave at the locks) to bracket any possible solution. The results can be seen in Table 4-9.

Table 4-9: Summary of Estimated Runup and Average Wave Overtopping – Wallingford Method

Summary of Elevation of Estimated Runup

Stillwater (ft PLD)	H=0.5ft (ft PLD)	H=1.0ft (ft PLD)	H=1.3ft (ft PLD)	H=1.5ft (ft PLD)	H=2.0ft (ft PLD)	H=2.5ft (ft PLD)	H=3.0ft (ft PLD)
87.5	88.0	89.0	89.5	90.5	92.5	95.0	98.0
88	88.0	89.0	90.0	90.5	92.5	95.0	98.0
88.5	88.5	89.5	90.5	91.0	93.0	95.5	98.5
89	89.0	90.0	91.0	91.5	93.5	96.0	99.0
89.5	89.5	90.5	91.5	92.0	94.0	96.5	99.5
90	90.0	91.0	92.0	92.5	94.5	97.0	100.0
90.5	90.5	91.5	92.5	93.0	95.0	97.5	100.5
91	91.0	92.0	93.0	93.5	95.5	98.0	101.0

Summary of Estimated Average Wave Overtopping

Stillwater (ft PLD)	Freeboard (m)	Freeboard (ft)	H=0.5ft *(m3/m/s)*	H=1.0ft *(m3/m/s)*	H=1.3ft *(m3/m/s)*	H=1.5ft *(m3/m/s)*	H=2 ft *(m3/m/s)*	H=2.5ft *(m3/m/s)*	H=3ft *(m3/m/s)*
87.5	1.22	4.00	2.04E-12	3.90E-07	7.53E-06	2.92E-05	2.87E-04	1.22E-03	3.36E-03
88	1.07	3.50	3.30E-11	1.57E-06	2.19E-05	7.38E-05	5.75E-04	2.13E-03	5.34E-03
88.5	0.91	3.00	5.31E-10	6.29E-06	6.39E-05	1.86E-04	1.15E-03	3.71E-03	8.49E-03
89	0.76	2.50	8.56E-09	2.53E-05	1.86E-04	4.71E-04	2.31E-03	6.46E-03	1.35E-02
89.5	0.61	2.00	1.38E-07	1.01E-04	5.42E-04	1.19E-03	4.62E-03	1.13E-02	2.15E-02
90	0.46	1.50	2.22E-06	4.07E-04	1.58E-03	3.00E-03	9.26E-03	1.96E-02	3.41E-02
90.5	0.30	1.00	3.59E-05	1.63E-03	4.60E-03	7.59E-03	1.86E-02	3.43E-02	5.42E-02
91	0.15	0.50	5.78E-04	6.56E-03	1.34E-02	1.92E-02	3.72E-02	5.97E-02	8.61E-02

CEM Acceptable Limits:
Structural damage starts at 2E-03 to 2E-02 $m^3/s/m$
Pedestrian risk starts between 1.0E-04 to 9E-04 $m^3/s/m$

Under existing conditions, when the surface water level of Gatun Lake is at the MOLL of 26.67 m PLD (87.5 ft PLD), there is 1.2 m (4 ft) of freeboard relative to crest of the walkway at the lock 27.89 m PLD (91.5 ft PLD) and 1.37 m (4.5 ft) of freeboard relative to the crest of the lock structure. The average wave overtopping, resulting from H=0.37 to H=0.60 m (1.2 – 2 ft), would be within the acceptable range for both pedestrians and structures.

If the MOLL were raised from 26.67 m PLD (87.5 ft PLD) to 27.43 m PLD (90.0 ft PLD) there would be 0.46 m (1.5 ft) of freeboard between the crest of the walkway at the lock and the water surface at the MOLL; 0.61 m (2 ft) of freeboard would exist between the MOLL and the crest of the lock structure. Under wave conditions of Hs=0.37 to Hs=0.46 m (1.2 – 1.5 ft) (and water surface at MOLL) overtopping would be within the accepted range as prescribed by the Coastal Engineering Manual for structures. As for pedestrian risk, there would be some issues at the top of the lock gate with the lake level raised to 89.5 - 90.0, but another walkway could be used at those times of the year when the lake would be at the MOLL. Pilot operations could also possibly be adjusted as to when the ships propeller could be engaged when traveling toward the lake (so that the worst case wave would be closer to 0.34 m (1.1 ft) rather than 0.39 m (1.3 ft).

Based on the above investigations, raising the MOLL of Gatun from 26.67 m (87.5 ft) up to 27.43 m (90 ft) PLD (allowing 0.61 m (2.0 ft) of freeboard to the top of the lock wall), should not result in a high risk of structural damage to the lock during low frequency events. Wave overtopping may put pedestrians at risk but other walkways could be used when the lake is at the proposed MOLL. As for the other structures within the lake, it would appear that 1.07 m (3.5 ft) of freeboard is required for pedestrian safety while 0.61 m (2.0 ft) of freeboard is required to minimize structural damage.

4.2.5 Summary and recommendations

The design wave heights at Gatun Lake historically used by ACP are estimated to be 0.91 m (3.0 ft), while at Pedro Miguel they are estimated to be 0.34 m (1.1 ft).

Based on an initial review of the wind data at the Gatun station (which show winds predominantly from the north), it was determined further investigation of the design wave height was merited. A similar review of the wind data for Pedro Miguel showed that the proposed design wave height of 0.34 m (1.1 ft) was reasonable.

In order to estimate the appropriate design wave height and overtopping design criteria at Gatun, wave data was hindcast using the wind record (1985 to 2003) experienced at Gatun station. It was determined, that for day to day operations, design wave heights from 0.01 to 0.05 m are appropriate. Wave heights on the order of 0.1 m to 0.37 m should be considered for extreme events. These results were verified by ACP personnel who stated that the maximum waves experienced at Gatun during the last 20 years were approximately 0.31 m (1.0 ft).

Based on the calculations completed for estimating boat wakes, it appears that for day to day operations, design boat wake wave heights up to 0.39 m (1.3 ft) may be experienced at the locks. However, for other structures within the lake, the wave heights experienced likely approach 0.59 m (1.93 ft).

Therefore, based on the investigations summarized in Table 4-9, raising the MOLL of Gatun from 26.67 m (87.5 ft) up to 27.43 m (90 ft) PLD, would not result in an unacceptable risk of structural damage to the lock during low frequency events. Wave overtopping may put pedestrians at risk but other walkways could be used when the lake is at the proposed MOLL. As for other structures in the lake, it would appear that 1.07 m (3.5 ft) of freeboard is required for pedestrian safety while 0.61 m (2.0 ft) of freeboard is required to minimize structural damage. It is further recommended that observations should be made at the locks and other areas in the lake to check the accuracy of these proposed design wave heights at later design stages. However, given the verification of the above calculations by ACP personnel experience, it appears that these estimates are reasonable.

In conclusion, based on the above analyses, the range of MOLLs that were considered for this study was from 26.67 m (87.5 ft) to 27.43 m (90.0 ft). While the final selected MOLL may be less than 27.43 m (90.0 ft) based on freeboard and the outcome of additional wave studies, the project team believed that the absolute maximum the MOLL could reach with the current lock elevations is 27.43 m (90.0 ft).

4.3 Water Yield Analyses

4.3.1 Approach

1999 USACE Study Approach

In 1999, the U.S. Army Corps of Engineers (USACE) performed a reconnaissance study for identification, definition, and evaluation of water supply projects to meet future needs at the Panama Canal. The USACE examined multiple alternatives designed to accomplish three primary objectives. These objectives were:

- To satisfy long-term (through the year 2050) municipal and industrial water supply needs without adversely affecting the operation of the Panama Canal;
- To provide sufficient navigation waters to meet existing and future Panama Canal transit demands without restricting vessel operation and to maintain historic reliability levels (calculated to be 99.6 percent); and
- To take advantage of projects by supplementing hydropower production as demands for other water uses increase. If possible, increase current capacity and energy production as solutions are implemented.

The Raise Gatun Lake alternative was examined as part of the reconnaissance study. Section 23 of the 1999 USACE report details the analysis of this alternative. The revised operating lake elevation chosen for analysis was 27.13 m (89 ft) Precise Level Datum (PLD). Proportional operating rule curves and flood curves were developed for this operating elevation and the future hydrologic reliability of the Panama Canal was examined using an existing HEC-5 model. The model scenarios considered demands ranging from existing levels up to 180 percent of existing levels.

The USACE estimated the existing hydrologic reliability of the Panama Canal as approximately 99.6 percent, while the hydrologic reliability with a demand ratio of 1.8 would be 86.3 percent without additional water supplies. The hydrologic reliability under existing demands would be 99.75% if the maximum operating elevation were increased to elevation 27.13 m (89.0 ft), and considering the increased operating elevation and a demand ratio of 1.8, the hydrologic reliability would be 86.7%. Project costs and benefits were also examined and a benefit to cost ratio of 2.6 was obtained for the Raise Gatun Lake project alternative.

Current Approach

As part of the current feasibility study for raising Gatun Lake maximum operating levels, an updated water yield analysis was performed to quantify the system reliability and economic benefits.

The existing HEC-5 model, developed by the USACE and ACP, was updated and applied by ACP and Moffatt & Nichol staff. The model was employed to assess the yield and reliability of the existing system under current and anticipated future operating conditions. HEC-5 is a deterministic model developed by USACE for application to flood control for reservoir management. The model may be applied on a daily or monthly time step and the output may be used for quantifying reliability. The required input parameters for application of HEC-5 include reservoir physical geometry, operating criteria (including rule and flood curves), stage discharge relationship for maximum outflow; time series of inflows to the reservoir system, rainfall, evaporation, and water demands.

The hydrologic period of record simulated was from January 1948 to December 1999 (52 years) using inflows provided by ACP. Reservoir stage storage relationships were provided by ACP as part of the baseline HEC-5 model. Multiple operating conditions were evaluated with the Maximum Operating Lake Level (MOLL) at Gatun Lake ranging from the existing elevation of 26.67 m PLD (87.5 ft PLD) to 27.43 m PLD (90 ft PLD). Demand scenarios with and without Post-Panamax (PP) Canal construction were also simulated based on projections provided by ACP. Existing and proposed spillway stage-discharge relationships for baseline and various MOLL alternatives were also developed and input into the model. Detailed assumptions for the water yield analysis are discussed in a later section.

For each of the scenarios, the hydrologic reliability and power generation capability were estimated using the HEC-5 model. The system yield/reliability is presented as a percentage of volume that the water demands in the lake can or cannot be met using current and proposed operational rule curves and associated policies, following the reporting procedure used by the USACE (1999). Section 4.2.4 summarizes the results of the water yield analysis.

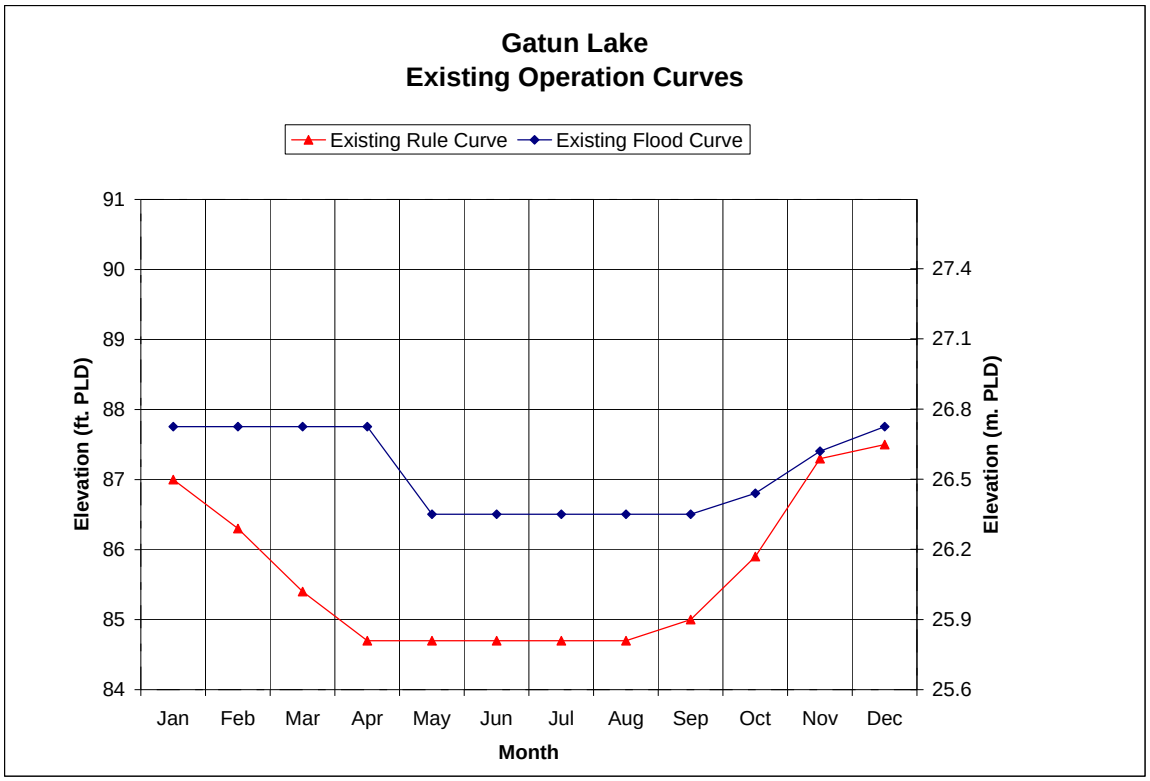
The potential economic benefits for each scenario were evaluated based on the model results and valuation water for both navigation and municipal and industrial (M & I) use. The basis of assumptions and summary of results of the economic are presented in Section 5.0. The next sections describe in detail the basis of the assumptions used in completing the HEC-5 modeling for the system yield and reliability analysis for both baseline and projected scenario conditions.

4.3.2 Baseline Conditions

Operational Curves

In accordance with the direction by ACP, water demand data (municipal, industrial and lockage water use) from the period 1993 to 1997 was used to establish the baseline conditions. The base case scenario was established by modifying the latest input data file developed by ACP (with a water supply pool now beginning at El 24.2 m (79.5 ft PLD) to reflect existing rule curve and flood curve operations for Gatun Lake shown below in Figure 4-5. For the Post-Panamax scenarios, the base of the conservation pool was assumed to be at El 24.9 m (81.5 ft PLD).

Figure 4-5: Existing Operational Curves at Gatun Lake



Municipal and Industrial Demand

For the baseline conditions, average 1993 to 1997 municipal and industrial (M&I) demands were assumed as presented in Table 4-10. Note that demands at Madden Lake affect water availability from Madden Lake to supply Gatun Lake and must be considered as part of the system analysis.

Table 4-10: 1993-1997 Municipal and Industrial Demands

Month	Gatun			Madden			Total		
	Cms	cfs	Lockages per day	Cms	cfs	Lockages per day	Cms	cfs	Lockages per day
Jan	3.5	123	1.45	5.2	185	2.17	8.7	308	3.62
Feb	3.6	126	1.48	5.3	188	2.21	8.9	314	3.69
Mar	3.5	123	1.45	5.4	190	2.23	8.9	313	3.68
Apr	3.6	127	1.49	5.4	190	2.23	9.0	317	3.73
May	3.3	117	1.37	5.4	191	2.24	8.7	308	3.62
Jun	3.6	127	1.49	5.3	188	2.21	8.9	315	3.70
Jul	3.4	121	1.42	5.3	188	2.21	8.8	309	3.63
Aug	3.5	124	1.46	5.3	187	2.20	8.8	311	3.65
Sep	3.3	115	1.35	5.3	187	2.20	8.6	302	3.55
Oct	3.5	124	1.46	5.1	180	2.12	8.6	304	3.57
Nov	3.5	123	1.45	5.2	182	2.14	8.6	305	3.58
Dec	3.4	119	1.40	5.2	183	2.15	8.6	302	3.55
Average	**3.5**	**122**	**1.43**	**5.3**	**187**	**2.20**	**8.8**	**309**	**3.63**

Lockage Requirements

For the baseline conditions, average 1993 to 1997 lockage demands were assumed as presented in Table 4-11. Based on the historical data from 1993 to 1997, the combined average annual municipal & industrial demands from Gatun and Madden Lakes (8.76 cms (cubic meters per second) 309 cfs (200 MGD)) and the lockages, 84.4 cms (2978 cfs) (1925 MGD), amount to a total of 93.2 cms (3287 cfs) (2127 MGD). This is equivalent to 38.68 lockages per day for the baseline conditions, as documented in the 1999 analysis by the USACE.

Table 4-11: 1993-1997 Panama Canal Lockages

Month	Lockage Water Use		
	(cms)	(cfs)	Lockages per day
January	90.8	3203	37.64
February	90.7	3199	37.59
March	91.0	3210	37.72
April	88.0	3106	36.50
May	84.2	2971	34.91
June	79.8	2816	33.09
July	79.9	2820	33.14
August	81.0	2859	33.60
September	78.9	2784	32.71
October	84.1	2968	34.88
November	80.5	2842	33.40
December	83.9	2962	34.81
Average	**84.4**	**2978**	**34.99**

4.3.3 Projected Future Conditions and Scenario Analyses

Operational Curves

For the baseline scenario, the model was run at the current Maximum Operating Lake Level (MOLL) of 26.7 m (87.5 ft) PLD with the rule and flood curves depicted in Figure 4-8. Additional model runs were performed, raising the MOLL in 0.152 m (0.5 ft) increments, from 26.7 m (87.5 ft) PLD up to a maximum MOLL of 27.4 m (90.0 ft) PLD. Both the rule curve and flood curve were modified for each proposed MOLL.

It is noted that the operational curves could be modified following a number of different methods. No attempt was made to optimize the operational curves at this feasibility level of study. In order to optimize the proposed rule and flood curves, detailed hydrologic analysis should be completed to weigh conservation vs. flood control needs on a month-by-month basis once the proposed spillway design has been approved. For simplicity at this preliminary stage, two methods were considered.

Following the USACE previous work completed for the 1999 study (referred to hereafter as the USACE Method), both rule curves and flood curves would be established based on the percentage of storage volume in the baseline conditions. For example, under baseline conditions for the month of April, the elevation of the rule curve was established at 68% of the Total Storage Volume between El 24.24 m (79.5 ft) and the existing MOLL elevation of 26.7 m (87.5 ft); (770.10 Million cubic meters(Mcm) (623852 acre feet) at an elevation of 25.82 m (84.7 ft) PLD). Similarly if the MOLL were raised to 26.8 m (88 ft), the elevation at which an equivalent storage volume was 68% of the new Total Storage Volume between El 24.24 m (79.5 ft) to 26.8 m (88.0 ft) (734.23 Mcm (594,800 acre feet)) would increase the rule curve elevation to 25.91 m (85.0 ft) PLD for the month of April.

A second method was also applied whereby the operational curves would be modified to maintain a constant volume as defined by existing operational curves (referred to hereafter as the Constant Volume Method). This method implicitly assumes that the volume between the rule curve elevation and the current MOLL has been optimized by ACP for the current spillway capacity to optimize between conservation and flood control needs. For example, under existing operating rules, during the month of April the rule curve is 25.82 m (84.7 ft) PLD, corresponding to storage volume of 5131.3 Mcm (4156820 acre feet) total storage. The difference in storage between the specified elevation 25.82 m (84.7 ft) PLD and the maximum MOLL of 26.7 m (87.5 ft) PLD (5497.63 Mcm (4453580 acre feet)) is 366.33 Mcm (296760 acre feet). To establish a new rule curve, with an MOLL of 26.8 m (88 ft) PLD and total storage of 5564.31 Mcm (4507600 acre feet), the volume for the month of April, assuming the constant difference in volume, would be 5197.99 Mcm (4210840 acre feet) corresponding to an elevation of 25.98 m (85.2 ft) PLD. It is important to note that the stage-storage relationship established in the HEC-5 model was used as the basis for modifications of rule and flood curves. Figure 4-6 illustrates the proposed rule curves using both the USACE and Constant Volume Methods for proposed MOLLs of 26.8 m (88 ft) and 27.4 m (90 ft) PLD.

Figure 4-6: Potential Rule Curves following USACE & Constant Volume Methods

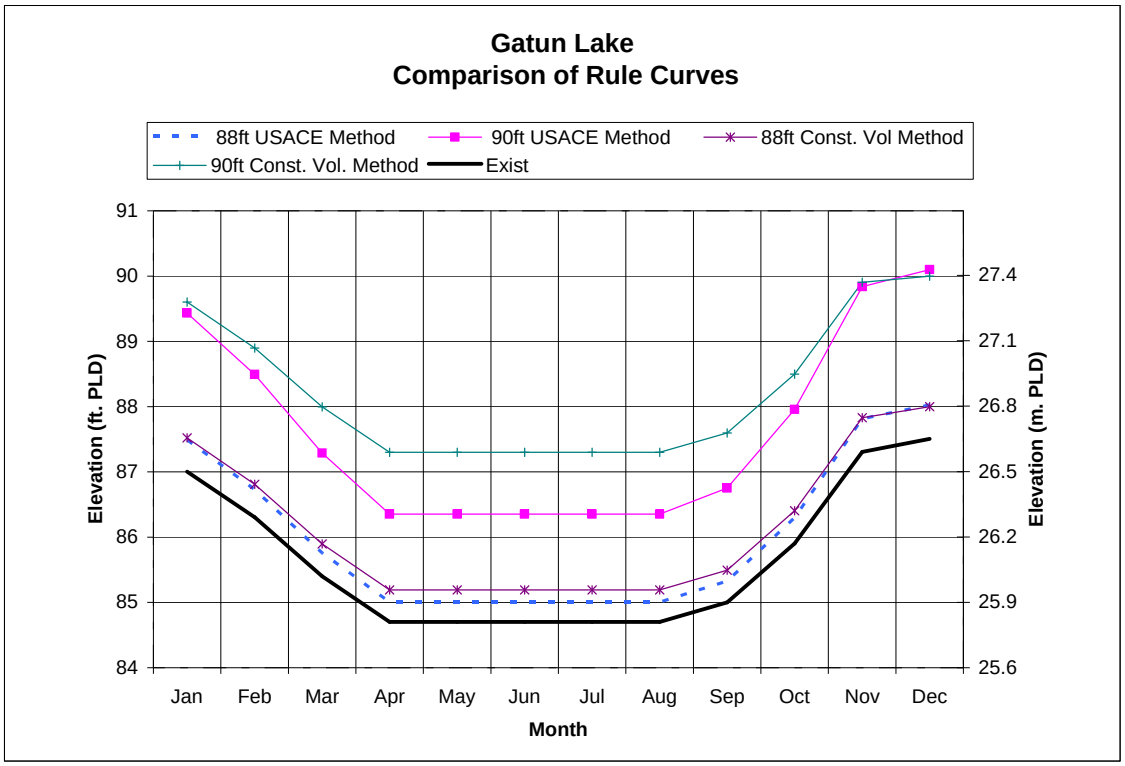
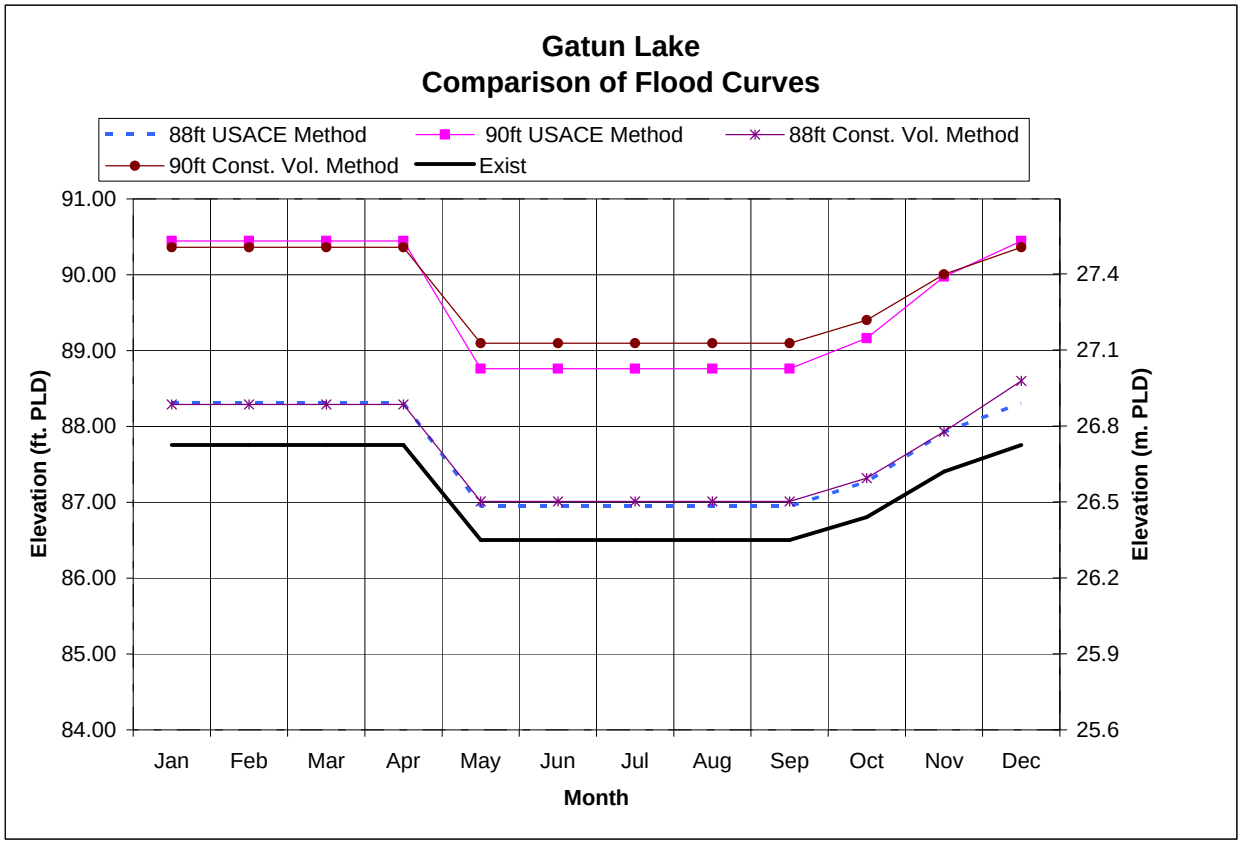


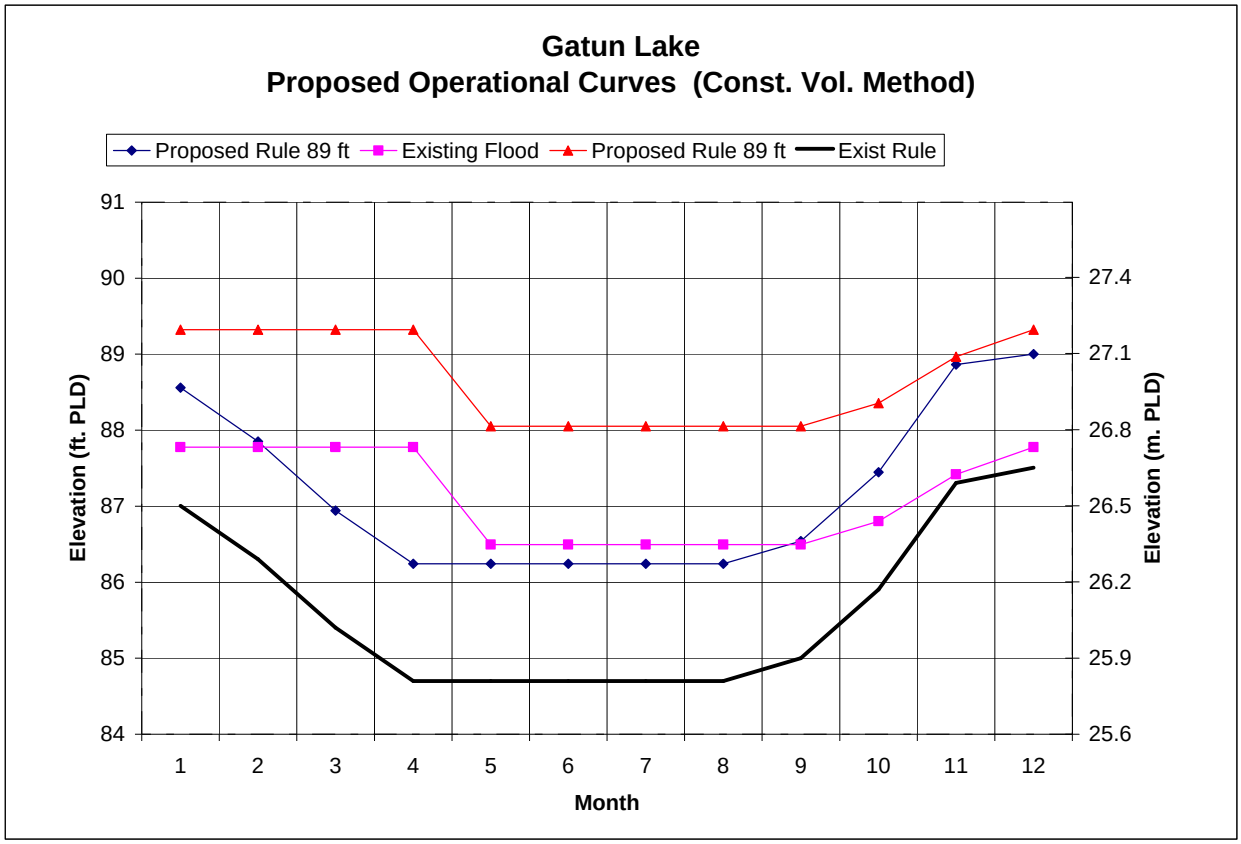
Figure 4-7 illustrates the proposed flood curves for the scenarios whereby the MOLL would be increased to 26.8 m (88 ft) PLD and 27.4 m (90 ft) PLD using both methodologies.

Figure 4-7: Potential Flood Curves following USACE and Constant Volume Methods



Although the rule and flood curves developed by the two methods would be similar at the lower MOLL of 26.8 m (88 ft) PLD, they deviate at the higher MOLL of 27.4 m (90 ft) PLD (Figures 4-6 & 4-7). The rule and flood curves generated from the constant volume method provide more volume for conservation uses and were used for the present preliminary analyses. M&N believes that the USACE method is too conservative since it would provide more relative surcharge volume that is currently provided (given that a new spillway is being constructed as part of the flood mitigation project). Figure 4-8 illustrates the proposed rule and flood curve for existing conditions as well as an example scenario raising the MOLL to 27.13 m (89 ft) PLD. The constant volume method was applied to develop rule and flood curves for various MOLL scenarios in 15.24 cm (0.5 ft) increments from 26.7 m (87.5 ft) to 27.4 m (90 ft) PLD. During later design stages, a detailed hydrologic study should be completed to further optimize the proposed rule and flood operational curves for the selected MOLL and new spillway configuration. It is posited that the rule curve could likely be raised even further during the wet season since additional spillway capacity would then be available.

Figure 4-8: Existing and Proposed Operational Curves for Scenario Raising MOLL to 27.13 m (89 ft) PLD



Projected Water Demand

Average annual water demand projections for Gatun and Madden Lakes were provided by ACP until 2025 and are presented in this section. Projected demands for lockages and municipal and industrial users were provided for two cases: 1) no new Post-Panamax Canal construction, and 2) a new Post-Panamax canal is constructed with three water saving basins. The three water saving basins alternative was used to determine the future lockage demands with new construction based on the guidance provided by the ACP Locks Team. ACP staff have stated that this is the most likely scenario for the new Post-Panamax Canal and the water usage per lockage is equivalent to the existing Canal (55 MG per lockage). ACP provided lockage demand through 2025 and M&I demand through 2060. Table 4-12 summarizes the forecast water demand for locks and M&I users under both cases through the year 2030. In the table, Post-Panamax (PP) refers to conditions anticipated after new canal construction. For the Post-Panamax scenarios, the new lock system is assumed to be completed and online in 2015. For purpose of this study, the 2030 lockage demands are assumed to be the same as those projected for 2025.

Table 4-12: Forecast Water Demand for Locks and Municipal & Industrial Use

		Baseline (1993-1997)	2010	2020	2020 with PP	2030[2]	2030[2] with PP
Municipal & Industrial Demand							
Gatun Lake	Cms	3.5	5.1	5.9	5.9	6.6	6.6
	Cfs	122	181	208	208	234	234
	MGD	79	117	134	134	151	151
	Lockages per day[1]	**1.44**	**2.12**	**2.44**	**2.44**	**2.75**	**2.75**
Madden Lake	Cms	5.3	7.8	9.0	9.0	10.1	10.1
	Cfs	187	276	318	318	357	357
	MGD	120	178	204.94	205	230	230
	Lockages per day[1]	**2.19**	**3.24**	**3.73**	**3.73**	**4.19**	**4.19**
Total M & I	Cms	8.8	13.0	14.9	14.9	16.8	16.8
	Cfs	309	457	526	526	592	592
	MGD	199	295	339	339	382	382
	Lockages per day[1]	**3.6**	**5.4**	**6.2**	**6.2**	**6.9**	**6.9**
Lockage Demand							
	Cms	84.4	80.4	88.2	84.8	88.2	88.4
	Cfs	2978	2839	3112	2992	3112	3120
	MGD	1922	1832	2008	1931	2008	2013
	Lockages per day[1]	**34.9**	**33.3**	**36.5**	**35.1**	**36.5**	**36.6**
TOTAL M & I and Lockages							
	Cms	93.2	93.4	103.1	99.7	104.9	105.2
	cfs	3287	3296	3638	3518	3703	3712
	MGD	2121	2126	2347	2270	2389	2395
	Lockages per day[1]	***38.6***	***38.7***	***42.7***	***41.3***	***43.4***	***43.5***

[1] Equivalent Lockage is based on 55MG/lockage
[2] 2030 water diversion to locks is assumed to be equivalent to 2025 ACP projections.

Based on discussions with ACP staff, the estimated average annual demands for M&I and lockage demands in the future are assumed to have the same monthly distribution throughout the year as recorded for the 1993 to 1997 period. Table 4-13 illustrates the distribution of monthly demands that were assumed for lockage consumption for the cases run assuming the Post-Panamax Locks will not be built. Table 4-14 presents the monthly lockage demands considering construction of the Post-Panamax Locks, and Table 4-15 summarizes the monthly M&I demands. For the HEC-5 model runs, the monthly demands for M&I and lockages were combined to form a single time series to be included within the model. Two different sets of runs were completed for each MOLL

scenario consisting of one demand time series which included the proposed Post-Panamax locks and another which did not. The M&I demands are identical for both of these demand time series.

Table 4-13: Forecast Monthly Lockage Demands (Without Post-Panamax Locks)

Month	Base line	2010	2020	2030	Base line	2010	2020	2030	Base line	2010	2020	2030
	(cms)				(cfs)				(Lockages per day)			
Jan	90.8	86.5	94.8	94.8	3203	3053	3346	3346	37.64	35.88	39.32	39.32
Feb	90.7	86.4	94.7	94.7	3199	3049	3342	3342	37.59	35.83	39.27	39.27
Mar	91.0	86.7	95.0	95.0	3210	3060	3354	3354	37.72	35.96	39.41	39.41
Apr	88.0	83.9	92.0	92.0	3106	2961	3245	3245	36.50	34.79	38.13	38.13
May	84.2	80.3	88.0	88.0	2971	2832	3104	3104	34.91	33.28	36.47	36.47
June	79.8	76.1	83.4	83.4	2816	2684	2942	2942	33.09	31.54	34.57	34.57
July	79.9	76.2	83.5	83.5	2820	2688	2946	2946	33.14	31.59	34.62	34.62
Aug	81.0	77.2	84.6	84.6	2859	2725	2987	2987	33.60	32.02	35.10	35.10
Sept	78.9	75.2	82.4	82.4	2784	2654	2909	2909	32.71	31.19	34.18	34.18
Oct	84.1	80.2	87.9	87.9	2968	2829	3101	3101	34.88	33.24	36.44	36.44
Nov	80.5	76.8	84.1	84.1	2842	2709	2969	2969	33.40	31.83	34.89	34.89
Dec	83.9	80.0	87.7	87.7	2962	2823	3095	3095	34.81	33.17	36.37	36.37
Avg.	**84.4**	**80.5**	**88.2**	**88.2**	**2978**	**2839**	**3112**	**3112**	**34.99**	**33.36**	**36.57**	**36.57**

Table 4-14: Forecast Monthly Lockage Demands (With Post-Panamax Locks)

Month	Base line	2010	2020	2030	Base line	2010	2020	2030	Base line	2010	2020	2030
	(cms)				(cfs)				(Lockages per day)			
Jan	90.8	86.5	91.2	95.1	3203	3053	3218	3356	37.64	35.88	37.81	39.44
Feb	90.7	86.4	91.1	95.0	3199	3049	3214	3351	37.59	35.83	37.77	39.38
Mar	91.0	86.7	91.4	95.3	3210	3060	3225	3363	37.72	35.96	37.90	39.52
Apr	88.0	83.9	88.4	92.2	3106	2961	3121	3254	36.50	34.79	36.67	38.24
May	84.2	80.3	84.6	88.2	2971	2832	2985	3112	34.91	33.28	35.08	36.57
June	79.8	76.1	80.2	83.6	2816	2684	2829	2950	33.09	31.54	33.24	34.67
July	79.9	76.2	80.3	83.7	2820	2688	2833	2954	33.14	31.59	33.29	34.71
Aug	81.0	77.2	81.4	84.9	2859	2725	2872	2995	33.60	32.02	33.75	35.19
Sept	78.9	75.2	79.3	82.7	2784	2654	2797	2917	32.71	31.19	32.87	34.28
Oct	84.1	80.2	84.5	88.1	2968	2829	2982	3109	34.88	33.24	35.04	36.53
Nov	80.5	76.8	80.9	84.4	2842	2709	2855	2977	33.40	31.83	33.55	34.98
Dec	83.9	80.0	84.3	87.9	2962	2823	2976	3103	34.81	33.17	34.97	36.46
Avg.	**84.4**	**80.5**	**84.8**	**88.4**	**2978**	**2839**	**2992**	**3120**	**34.99**	**33.36**	**35.16**	**36.66**

Table 4-15: Forecast Monthly Municipal and Industrial Demands (Both Scenarios)

Month	Base line	2010	2020	2030	Base line	2010	2020	2030	Base line	2010	2020	2030
	(cms)				(cfs)				(Lockages per day)			
Jan	8.7	12.9	14.8	16.7	308	456	524	590	3.62	5.36	6.16	6.93
Feb	8.9	13.1	15.2	17.0	314	464	535	601	3.69	5.45	6.29	7.06
Mar	8.9	13.1	15.1	17.0	313	463	533	599	3.68	5.44	6.26	7.04
Apr	9.0	13.3	15.3	17.2	317	469	540	607	3.73	5.51	6.35	7.13
May	8.7	12.9	14.8	16.7	308	456	524	590	3.62	5.36	6.16	6.93
June	8.9	13.2	15.2	17.1	315	466	536	603	3.70	5.48	6.30	7.09
July	8.8	13.0	14.9	16.8	309	457	526	592	3.63	5.37	6.18	6.96
Aug	8.8	13.0	15.0	16.9	311	460	529	595	3.65	5.41	6.22	6.99
Septr	8.6	12.7	14.6	16.4	302	447	514	578	3.55	5.25	6.04	6.79
Oct	8.6	12.8	14.7	16.5	304	450	518	582	3.57	5.29	6.09	6.84
Nov	8.6	12.8	14.7	16.5	305	451	519	584	3.58	5.30	6.10	6.86
Dec	8.6	12.7	14.6	16.4	302	447	514	578	3.55	5.25	6.04	6.79
Avg.	**8.8**	**13.0**	**14.9**	**16.8**	**309**	**457**	**526**	**592**	**3.63**	**5.37**	**6.18**	**6.96**

Stage Discharge Relationship

The existing HEC-5 model input for the stage discharge relationship for maximum outflow was revised to reflect the new spillway design as identified in Table 4-16 and Table 4-17. These curves were developed by adding the maximum outflows for both the existing and proposed spillway designs for the various MOLLs. The existing spillway curve was taken directly from the HEC-5 model. The rating curves for the proposed spillway were calculated using hydraulic relationships developed during the PMF design routing process. It should be noted that these curves incorporate the recent design criteria change in which the new spillway designs were developed by sizing the proposed spillway so that the maximum lake level during the PMF was kept below El 28 m (92.0 ft) PLD.

Table 4-16: Spillway Rating Table (Existing and Auxiliary Spillways) (Metric Units)

Elevation	Maximum Outlet Capacity for specified MOLL (cms)						
(m PLD)	EXISTING	26.7 m (87.5 ft) PLD	26.8 m (88 ft) PLD	27.0 m (88.5 ft) PLD	27.1 m (89 ft) PLD	27.3 m (89.5 ft) PLD	27.4 m (90 ft) PLD
12.20	0	0	0	0	0	0	0
15.24	0	0	0	0	0	0	0
18.29	0	229	344	458	573	687	802
21.34	54	702	1,026	1,350	1,674	1,998	2,322
23.48	1,213	2,229	2,737	3,246	3,754	4,262	4,771
23.78	1,669	2,742	3,279	3,816	4,352	4,889	5,426
24.09	1,968	3,100	3,666	4,232	4,797	5,363	5,929
24.39	2,286	3,477	4,072	4,667	5,263	5,858	6,453
24.70	2,621	3,871	4,497	5,122	5,747	6,373	6,998
25.00	2,972	4,283	4,939	5,595	6,251	6,906	7,562
25.30	3,338	4,712	5,399	6,085	6,772	7,459	8,146
25.61	3,720	5,156	5,875	6,593	7,311	8,029	8,748
25.91	4,116	5,617	6,367	7,117	7,867	8,617	9,368
26.22	4,527	6,092	6,875	7,657	8,440	9,223	10,005
26.52	4,952	6,583	7,398	8,213	9,029	9,844	10,660
26.83	5,390	7,088	7,936	8,785	9,634	10,482	11,331
27.13	5,842	7,606	8,489	9,371	10,254	11,136	12,018
27.44	6,306	8,139	9,056	9,972	10,889	11,805	12,722
28.96	8,502	10,689	11,783	12,876	13,970	15,063	16,157
30.49	9,919	12,480	13,761	15,042	16,323	17,604	18,885
32.01	11,335	14,291	15,769	17,246	18,724	20,202	21,680

Table 4-17: Spillway Rating Table (Existing and Auxiliary Spillways) (English Units)

Elevation (ft PLD)	Maximum Outlet Capacity for specified MOLL (cfs)						
	EXISTING	87.5 ft PLD	88 ft PLD	88.5 ft PLD	89 ft PLD	89.5 ft PLD	90 ft PLD
40	0	0	0	0	0	0	0
50	0	0	0	0	0	0	0
60	0	8,086	12,128	16,171	20,214	24,257	28,299
70	1,890	24,759	36,194	47,629	59,064	70,498	81,933
77	42,790	78,662	96,598	114,534	132,470	150,406	168,342
78	58,878	96,761	115,703	134,645	153,586	172,528	191,469
79	69,463	109,394	129,359	149,324	169,290	189,255	209,221
80	80,672	122,686	143,693	164,700	185,707	206,713	227,720
81	92,479	136,611	158,677	180,743	202,809	224,875	246,941
82	104,860	151,144	174,287	197,429	220,571	243,713	266,856
83	117,796	166,267	190,502	214,738	238,973	263,209	287,444
84	131,268	181,959	207,304	232,650	257,995	283,340	308,686
85	145,260	198,203	224,675	251,147	277,619	304,090	330,562
86	159,756	214,985	242,599	270,213	297,828	325,442	353,056
87	174,743	232,289	261,062	289,834	318,607	347,380	376,153
88	190,208	250,102	280,050	309,997	339,944	369,891	399,838
89	206,139	268,413	299,550	330,687	361,824	392,961	424,098
90	222,525	287,210	319,552	351,894	384,236	416,579	448,921
95	300,000	377,184	415,776	454,369	492,961	531,553	570,145
100	350,000	440,399	485,599	530,799	575,998	621,198	666,398
105	400,000	504,293	556,439	608,586	660,732	712,878	765,025

4.3.4 Water Yield Analyses Results

The HEC-5 model was applied to assess the hydrologic reliability of the system and potential hydropower generation. The model was applied for scenarios referred to hereafter by the MOLL for Gatun Lake ranging from Existing Conditions (26.7 m (87.5 ft) PLD) to 27.4 m (90 ft) PLD. The hydrologic reliability for each MOLL is estimated based on the systems volumetric yield over the 52 year period of record. The model also provides a time series of monthly lake levels for each scenario which may be used to qualitatively assess the potential need for draft restrictions. Water allocation for hydropower production can be examined to assess any loss in power production due to increases in M&I and lockage demands at Gatun Lake as well as Madden Lake.

Hydrologic Reliability

The hydrologic reliability was estimated following the same volumetric methodology employed by the USACE(1999). A volumetric estimate captures both the magnitude and duration of the system's reliability. The reliability is estimated as follows:

$$\mathrm{Re}\,liability = \left[1 - \left(\frac{\sum Volume_of_Water_Shortage}{\sum Volume_of_Water_\mathrm{Re}\,quired_for_Diversion}\right)\right] \text{x } 100\%$$

As for the USACE study, it is assumed that all municipal and industrial demands will be met as a priority over operation of the lock system. Therefore, all M&I demands at Madden will be met and the total system reliability may be estimated as a function of the shortages at Gatun for diversion to the locks.

For example, under the baseline conditions (26.68 m (87.5 ft) PLD MOLL and 1993 to 1997 average demands), the reliability is estimated as a function of the total shortage over a 52 year period was 65,719 Million Gallons (MG) compared to the required 37,969,184 MG; the reliability is estimated to be 99.8%. It should be noted that the results differ from the USACE 1999 study which yielded a hydrologic reliability under existing conditions of 99.6% primarily because of the change in assumed base level of reservoir storage from 24.85 m (81.5 ft) PLD (assumed by the USACE) to 24.24 m (79.5 ft) PLD applied for this evaluation for the Base, 2010, 2020 and 2030 demand scenarios. When comparing results of future scenarios to existing conditions, the reliability of 99.6% should be applied.

The results of the HEC-5 model simulations for existing conditions through the year 2030, with and without Post-Panamax construction, for varying MOLL scenarios are presented graphically in Figure 4-9 and are summarized in Table 4-18.

The x-axis, along the bottom of Figure 4-9 reflects the year of simulation. The top x-axis displays the demand in equivalent lockages per day (assuming 55 MG/lockage). As shown, the reliability of the Canal under existing operating and demand conditions is 99.83%. Under the baseline alternative in 2030 (without PP locks) the reliability with the MOLL remaining at 26.28 m (87.5 ft) PLD is 99.53%. Please note that the results may seem counterintuitive since there is not as much change as one would expect with the Post-Panamax scenarios. However, the reason that this occurs is because the current projections show that when the Post-Panamax locks come online, they will be running at full capacity and the existing locks will be running at half capacity. Since the Post-Panamax ships can carry much more tonnage for the same lockage volume, the dip in reliabilty is not so marked.

The HEC-5 model results indicate that the base level of reliability could be maintained through the year 2030 (without PP locks) under the scenarios whereby the MOLL would be equal to or greater than 26.83 m (88 ft) PLD. Note that the base level of reliability

can be maintained through year 2030 if the MOLL is raised to 27.4 m (90.0 ft) (with PP locks).

Figure 4-9: Panama Canal Hydrologic Reliability for Varying MOLL at Gatun Lake.

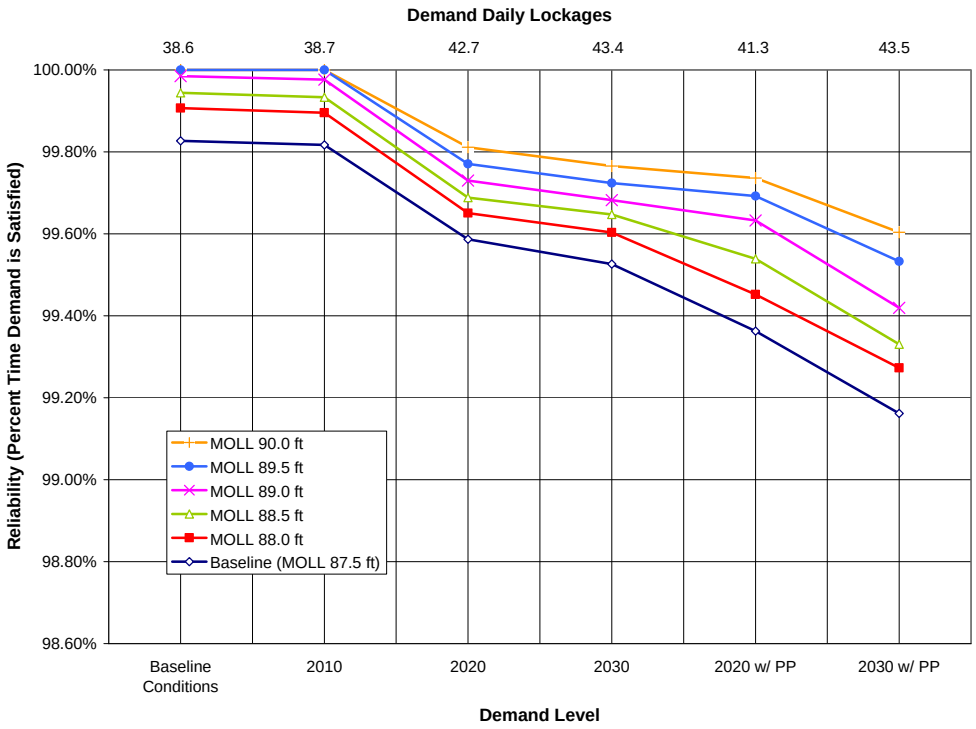


Table 4-18: Hydrologic Reliability Estimated Based on Monthly Data from 1948 – 1999. Panamax scenarios use a minimum lake level of 24.2 m (79.5 ft); Post-Panamax, 24.8 m (81.5 ft)

Scenario (Defined by MOLL)	BASE	2010	2020	2020 Post-Panamax	2030	2030 Post-Panamax
Baseline Conditions 26.67 m (87.5 ft)	99.83%	99.82%	99.59%	99.36%	99.53%	99.16%
26.82 m (88 ft)	99.91%	99.90%	99.65%	99.45%	99.60%	99.27%
26.97 m (88.5 ft)	99.94%	99.93%	99.69%	99.54%	99.65%	99.33%
27.13 m (89 ft)	99.98%	99.98%	99.73%	99.63%	99.68%	99.42%
27.28 m (89.5 ft)	100.00%	100.00%	99.77%	99.69%	99.72%	99.53%
27.43 m (90 ft)	100.00%	100.00%	99.81%	99.74%	99.77%	99.60%

Potential Draft Restrictions

In addition to the volumetric hydrologic reliability, Moffatt & Nichol recommend that potential draft restrictions that could occur under various MOLLs be evaluated. ACP is currently undertaking this task.

Hydropower Production

Because of increasing demands from M&I users as well as the Canal locks, the amount of water available for hydropower production at Gatun Lake and Madden Lake may decline over time, unless modifications are made to operating rules. The HEC-5 model results provide a comparison of future power generation capability with existing levels.

Table 4-19 presents the average annual power generation at Gatun Lake for varying MOLL scenarios under projected demands through the year 2030. Table 4-20 presents the average annual power generation at Madden Lake under the same scenarios. It is interesting to note that the interaction between the Gatun and Madden lake operational rules caused more water to be spilled from Madden under the higher MOLLs (to help match the higher rule curve for Gatun) so that hydroelectric power generation for Madden was generally greater for the higher MOLLs while the opposite was true for Gatun. However, the reductions at Gatun generally were far greater than the increases at Madden so that there was a small net reduction in power generation at both sites as a result of increasing the MOLL of Gatun Lake.

Table 4-19: Average Annual Generation at Gatun Lake for 52 year period (MWH)

MOLL Scenario	*BASE*	*2010*	*2020*	*2020 w/ PP*	*2030*	*2030 w/ PP*
26.67 m (87.5 ft)	140,289	139,957	124,478	131,352	121,403	121,403
26.82 m (88.0 ft)	139,946	139,342	124,449	131,352	121,071	121,403
26.97 m (88.5 ft)	139,761	138,927	123,841	130,841	121,071	120,716
27.13 m (89.0 ft)	139,602	138,927	123,796	130,829	120,727	120,716
27.28 m (89.5 ft)	138,927	138,594	123,452	130,486	120,384	120,670
27.43 m (90.0 ft)	138,927	138,927	123,109	130,216	120,074	120,085

Table 4-20: Average Annual Generation at Madden Lake for 52 year period (MWH)

MOLL Scenario	*BASE*	*2010*	*2020*	*2020 w/ PP*	*2030*	*2030 w/ PP*
26.67 m (87.5 ft)	198,933	191,344	187,223	186,985	183,689	183,101
26.82 m (88.0 ft)	198,990	191,397	187,256	187,117	183,745	183,347
26.97 m (88.5 ft)	199,036	191,439	187,302	187,260	183,797	183,434
27.13 m (89.0 ft)	199,083	191,473	187,372	187,341	183,864	183,579
27.28 m (89.5 ft)	199,127	191,514	187,406	187,406	183,902	183,666
27.43 m (90.0 ft)	198,933	191,344	187,223	186,985	183,689	183,101

2060 Scenario – Full Buildout

As can be seen from the reliability results above, the growth rate of the projected water demands over the next 25 years is not high enough to fully realize a large additional water yield benefit of raising the MOLL. However, given that the current locks have been in service over 90 years and any works constructed would likely be in service over the next 100 years (at least), it was felt that an additional HEC-5 model run was warranted to determine the economic benefits realized if the current locks and the proposed Post-Panamax Canal were constructed and operating at full capacity (i.e., both existing lanes and one Post-Panamax lane). For the purpose of this study, this run utilized the 2060 M&I water demand. Based on the ACP projections, the maximum daily capacity of the expanded Canal is estimated to be 36.5 lockages for the existing locks plus 17.1 lockages for the Post-Panamax Canal.

Table 4-21 and Table 4-22 summarize the projected demands and estimated reliability for each of the scenarios through 2060.

Table 4-21: Summary of Demands (in equivalent lockages)

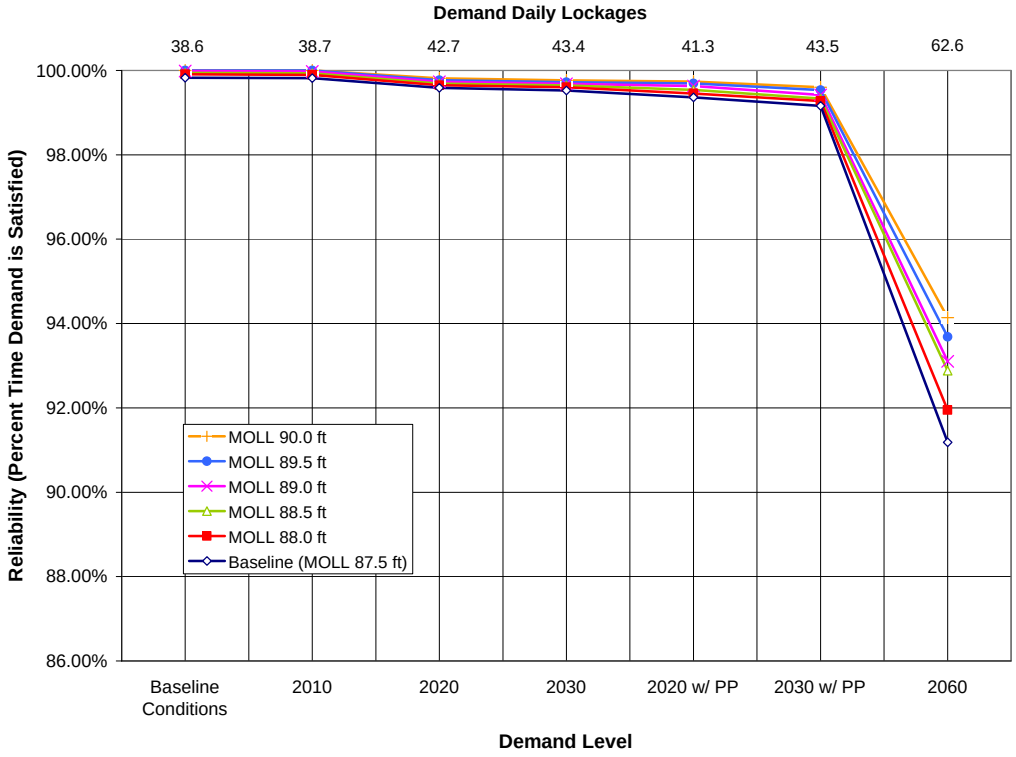
Demand	BASE	2010	2020	2020 PP	2030	2030 PP	2060 PP
M&I	3.6	5.4	6.2	6.2	6.9	6.9	9.1
Navigation	34.9	33.3	36.5	35.1	36.5	36.6	53.5
% Navigation Panamax	100%	100%	100%	52.4%	100%	55.7%	68.2%
Total Demand	38.6	38.7	42.7	41.3	43.4	43.5	62.6

Table 4-22: Hydrologic Reliability Estimated Based on Monthly Data from 1948 - 1999

MOLL Scenario	BASE	2010	2020	2020 PP	2030	2030 PP	2060 PP
26.67 m (87.5 ft)	99.83%	99.82%	99.59%	99.36%	99.53%	99.16%	91.19%
26.82 m (88.0 ft)	99.91%	99.90%	99.65%	99.45%	99.60%	99.27%	91.95%
26.97 m (88.5 ft)	99.94%	99.93%	99.69%	99.54%	99.65%	99.33%	92.88%
27.13 m (89.0 ft)	99.98%	99.98%	99.73%	99.63%	99.68%	99.42%	93.10%
27.28 m (89.5 ft)	100.00%	100.00%	99.77%	99.69%	99.72%	99.53%	93.69%
27.43 m (90.0 ft)	100.00%	100.00%	99.81%	99.74%	99.77%	99.60%	94.14%

Figure 4-10 illustrates the reduction in hydrologic reliability that would be realized from 2030 to 2060, once the system has reached full capacity. From the table and figure, it can be seen that the significant benefits from raising the MOLL occur at the full build out of the Canal. The relative differences (or gain in percentages) in reliability from the raise MOLL scenarios when compared to the baseline conditions are much larger. For example, the 27.43 m (90.0 ft) MOLL scenario will provide a full 2.95% increase in reliability compared to the baseline conditions at 26.67 m (87.5 ft) (94.14% - 91.19%) for the full buildout scenario, whereas the 27.43 m (90.0 ft) MOLL scenario will only provide a 0.44% increase in reliability compared to the baseline conditions at 26.67 m (87.5 ft) (99.60% - 99.16%), for the 2030 PP scenario.

Figure 4-10: Panama Canal Hydrologic Reliability for Varying MOLL at GatunLake



4.3.5 Conclusions

As can be seen from the above analyses, the existing water supply system developed for the existing Panama Canal is quite complex and has many competing interests for water in the future. Based on the analyses above, it appears that raising the MOLL to at least an elevation of 27.13 m (89.0 ft) is required to provide the same reliability that the current MOLL provides (under existing demands) for projected demands through 2030 without PP locks. It also appears that raising the MOLL to the current proposed maximum of 27.43 m (90.0 ft) would be enough to provide the current level of reliability through 2030 with PP locks. Once demands surpass the 2030 projections toward full buildout demands, raising the MOLL will provide a much larger relative benefit as the system becomes more and more hydrologically stressed.

For future work, the project team would recommend studying lockage water demands past the current limit of 2025 to determine if and when the projected demands would begin to approach full buildout conditions. The project team would also suggest that more detailed reliability analyses be performed utilizing water levels in the lake to account for the impact of draft restrictions at different lake levels. This would allow ACP to better evaluate the costs of raising the MOLL to provide additional draft for the Post-Panamax canal versus additional dredging in the lake.

4.4 Impact on Existing Structures

This section summarizes the evaluation of the existing structures and facilities that will likely be impacted by raising the Gatun Lake maximum operating level. The information in this section is largely based upon the site visit performed from May 4 to 7, 2004 and supplemented by information provided in the 1999 USACE Study as well as additional information provided to the project team at the kickoff meeting. The fieldwork was necessary to verify and/or obtain the critical elevations of the multiple structures and facilities that are likely to be impacted by increasing the Maximum Operating Lake Level (MOLL). This section is designed to identify the structures that will be affected by raising the MOLL, in addition to Section 4.6 which outlines the cost implication to replace or retrofit these structures.

4.4.1 Inventory of Structures

Data Sources and Field Reconnaissance Study

During the kickoff meeting, the project team accompanied by ACP personnel conducted a helicopter survey of the lake to gain an overall understanding of the important features around the lake that could possibly be impacted by raising the MOLL of Gatun Lake. ACP also provided numerous reports and other datasets that were culled through to develop a list of potential structures and features that would be impacted by the project. However, after reviewing all of the available datasets, it was found that numerous structures had conflicting critical elevations reported, and that many structures had very little information available. Therefore, it was decided that a detailed field investigation was warranted to rectify these issues.

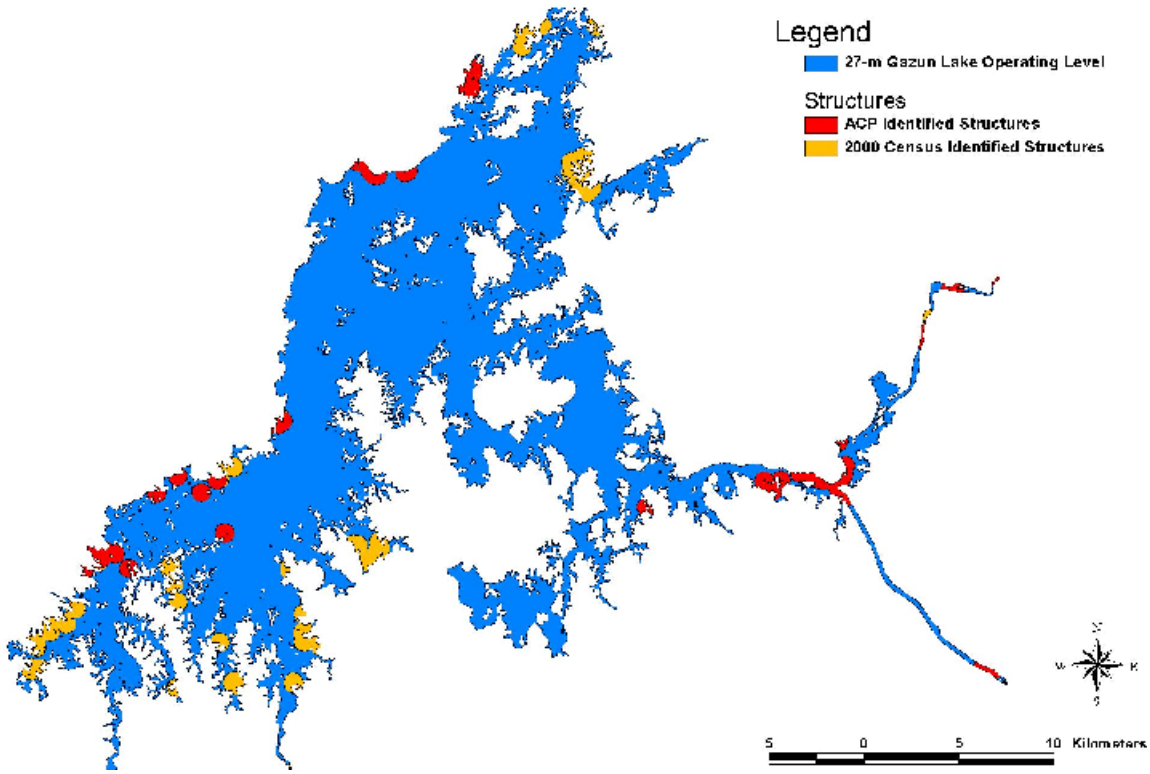
A field reconnaissance study was undertaken to document the various structures around the lake that might be impacted by raising the MOLL. This task involved utilizing ACP boats and/or vehicles to access the majority of the ACP waterfront structures and various private facilities and villages around Lake Gatun. The field observations took four days to complete and involved visiting the majority of the structures and facilities around the lake. For each structure or facility an overall site description with brief details and measurements were obtained, along with photographic documentation. This enabled the project team to obtain and/or verify the critical elevations for the various structures and to develop potential replacement or retrofit concepts with corresponding opinion of probable costs. During the investigation inventory logs were maintained for each structure to identify general information, location, and critical elevations.

May 4, 2004 was the first day of the site inventory and involved investigating the structures starting at the Pedro Miguel Meteorological and Hydrographic Facility and continuing to various ACP and third party structures and facilities located in Paraiso,

Gamboa, and up the Chargres River to Madden Dam. The second day, May 5th, involved boarding the boat in Gamboa and traveling to the Smithsonian facility on the island of Barro Colorado. From there, the team headed to Gatun to inventory the various ACP and third party structures and facilities around the Gatun Locks. In Gatun, the team boarded a van and traveled toward Fort Sherman to estimate the approximate road and bridge elevations and inventory the areas located around the proposed spillway site and around French Canal. The final stop was at the Gatun Locks to meet with lock personnel and to verify and locate the critical elevations throughout the facility. The third day, May 6th, involved boarding the boat in Gamboa and heading to the Monte Lirio Bridge and then continuing to the towns of El Limon, New Providencia, La Represa and finishing at the Melia Resort and Mount Hope water intake. In the afternoon the team traveled by boat to the western side of the lake to investigate the towns of Escobal and Cuipo. On the fourth day, May 7th, the team traveled to the La Laguna to investigate the village and water intake structure. From La Laguna, the team split up with half continuing west to the villages of La Arenosa and Lagartera, while the other half headed back toward the Pedro Miguel Locks to investigated Las Cruces landing and inspected the rubber boot protecting the hydraulic arms for the gates at the Miraflores Locks.

Figure 4-11 shows the general location of the sites that were visited during the field reconnaissance study.

Figure 4-11: General Location of Structures Impacted by Raising the MOLL of Gatun Lake



The structures that will be impacted with a revised operating lake level consist of the Gatun Locks, Pedro Miguel Locks, Gatun Spillway and Dam, numerous saddle dams, multiple ACP structures and facilities, bridges, water supply intake structures, and other third party facilities located along the perimeter of Gatun Lake and its tributaries.

The primary structures and facilities appurtenant to and potentially affected by any change in the operation of Gatun Lake can be grouped according to general types as follows: ACP Installations (Gatun and Pedro Miguel Locks, Spillways, Embankments, ACP Dredging and Operational Facilities) and Third-party Installations (Panama Canal Railroad, Bridges, Water Intake Structures, Other Structures). These features are addressed in the following paragraphs and the features requiring modification are detailed under each section separately.

ACP Installations

Gatun and Pedro Miguel Locks

The upper chambers at Gatun and Pedro Miguel Locks were built on the standard Panama Canal footprint having nominal chamber dimensions of 304.8 by 33.5 m (1,000

by 110 ft). As noted in the paper by Hodges, 1915,[29] the freeboard of the terminal locks of the summit level was assumed at 1.52 m (5 ft) above high water of Gatun Lake, placing the coping of upper Gatun and Pedro Miguel Locks at elevation 28.04 m (92 ft) PLD. As can be seen in Figure 4-12 below, freeboard at the gates is now much less than the original design.

Figure 4-12: North Gates at Pedro Miguel Locks

The sill of the opening through which the gate-operating strut protrudes is 1.08 m (3.54 ft) below the top of the lock wall. The paper by E.S. Randolph, "Report on Preliminary Office Study on Additional Spillway Capacity from Gatun Lake", and supplemented by the USACE 1979 – PMF Study (Green Book) gives the following critical elevation for the locks.

[29]ASCE, Transactions of the International Engineering Congress – The Panama Canal, 1915.

Table 4-23: Critical Elevations in Feet for Lock Structures (Taken from Randolph, 1945, USACE, 1979 – PMF Study, and ACP Field Data)

Specific Portion of Structure	Gatun	Pedro Miguel
Guard Gate Strut Slots	26.86 m (88.125 ft)	26.96 m (88.46 ft)
Walls Under Emergency Dams	27.20 m (89.25 ft)	27.20 m (89.25 ft)
Top of Floatwell Chamber	26.85 m (88.08 ft)	26.97 m (88.50 ft)
Floor, Upper Regulating Valve Rooms	27.03 m (88.67 ft)	27.13 m (89.00 ft)
Upper Chain Fender Bulkheads	27.18 m (89.17 ft)	27.27 m (89.47 ft)
Bulkhead in Guard Valve Rooms	26.77 m (87.83 ft)	26.90 m (88.25 ft)
Top of Stairway, Guard Valve Rooms	26.62 m (87.33 ft)	Not Reported
Caisson Duct	Not Reported	27.28 m (89.50 ft)
Top of Miter Gate	26.94 m (88.4 ft)	26.94 m (88.4 ft)
Top of Miter Gate Walkways	27.89 m (91.50 ft)	27.89 m (91.50 ft)
Top of Lock Walls and Backfill	28.04 m (92.00 ft)	28.04 m (92.00 ft)
Drainage Slots for Mule Rails (invert)	27.71 m (90.9 ft)	27.71 m (90.9 ft)
Bottom of Pit Under Locomotive	26.82 m (88.0 ft)	Not Reported

Proposed Post-Panamax Locks

In addition to the existing locks, modifications to the proposed Post-Panamax locks will have to be incorporated based on the MOLL finally selected. However, after numerous discussions with ACP staff concerning this issue, ACP directed the project team to not incorporate expected impacts and additional costs associated with the raising the gates and lock walls required due to raising the MOLL against this project.

Spillways

As stated in the 1999 USACE study, the existing spillway at Gatun Dam is a gated structure having a sill elevation of 21.0 m (68.89 ft) MSL. There are 14 gate bays, each 13.7 m (45 ft) wide, closed by vertical lift (Stoney) gates 5.8 m (19 ft) high, and separated by reinforced concrete piers, 10 piers 2.6 m (8.5 ft) wide and 3 piers, at the center of the spillway, 4.6 m (15 ft) wide. The stated top of gate elevation in the closed position is 26.79 m (87.89 ft) MSL. When fully raised the gates have a bottom elevation of 27.89 m (91.5 ft) MSL. A walkway bridge spans across the tops of the piers at elevation 35.2 m (115.5 ft) MSL imposing a top restraint on the height to which the gates can be raised. This height is further restricted by the gate operating mechanism, consisting of a lifting chain, pulley, and counterweight system with the chains attached to the tops of the gates.

Embankments

Gatun Dam

The dam as constructed has a general top elevation of 32.0 m (105 ft) MSL, has side slopes varying from 7.67 horizontal on 1 vertical to 11.11 horizontal on 1 vertical in critical areas, is 2.40 km (1.5 mi) long, and is approximately 701 m (2,300 ft) wide at the widest point of its base. However, recent topographic surveys show that certain portions of the crest (both on east and west sections) are actually below elevation 30 m (98.4 ft).

Pedro Miguel Dam

The dam at Pedro Miguel Lock comprises the southern enclosure for the Gatun Lake. It was constructed from the earth and rock fill removed from the lock pit and extends northward from the upper west wingwall of the lock to a hill. It is practically an extension of the west wingwall of the lock. The top elevation of the dam is at elevation 32.0 m (105 ft) MSL and its total length is approximately 427 m (1,400 ft).

Saddle Dams

The following is a listing of the saddle dams around Gatun Lake based on an inventory prepared by the ACP.

Table 4-24: Gatun Lake Saddle Dam Inventory (1999 USACE Study)

Saddle Dam Name	Type of Saddle Dam	Crest elevation (m - ft)	Top elevation of bedrock (m- ft)	Length of Saddle Dam in Its Lowest Part (m. – ft.)
Arroyo No. 1	Natural	35.6 - 116.8	27.4 - 89.9	106.98 - 351
Arroyo No. 2	Natural	34.44 - 113.0	22.62 - 74.2	295.05 - 968
Arroyo No. 3	Natural	35.6 - 116.8	26.21 - 86.0	205.13 - 673
Arroyo No. 4	Natural	34.99 - 114.8	24.4 - 80.1	95.1 - 312
Arroyo No. 5	Natural	32.92 - 108.0	19.81 - 65.0	Not mentioned
Arroyo No. 6	Natural	34.99 - 114.8	22.31 - 73.2	114.91 - 377
Barro No. 1	Natural	43.31 - 142.1	37.49 - 123.0	Not mentioned
Barro No. 2	Natural	31.7 - 104.0	Unknown	Not mentioned
Barro No. 3[1/]	Natural	31.7 - 104.0	Not mentioned	Not mentioned
Canoa No. 1	Natural	36.3 - 119.1	34.99 - 114.8	295.05 - 968
Canoa No. 2	Natural	36.3 - 119.1	Unknown	Not mentioned
Canoa No. 3	Natural	36.3 - 119.1	Not mentioned	Not mentioned
Caño No. 1	Natural	34.99 - 114.8	24.69 - 81.0	99.97 - 328
Caño No.2	Man-made	34.99 - 114.8	23.41 - 76.8	99.97 - 328
Caño No. 3	Man-made	34.75 - 114.0	23.5 - 77.1	Not mentioned
Caño No. 4[2/]	Man-made	32.0 - 105.0	23.5 - 77.1	Not mentioned
Caño No. 5	Natural	28.4 - 93.2	22.8 -74.8	Not mentioned
Caño No. 6	Natural	29.8 - 97.8	15.21 - 49.9	Not mentioned
Egronal	Natural	39.29 - 128.9	36.61 - 120.1	45.11 - 148
Escobal No. 1	Natural	34.99 - 114.8	27.4 - 89.9	70.1 - 230
Escobal No. 2[3/]	Natural	34.99 - 114.8	25.6 - 84.0	Not mentioned
Escobal No. 3	Natural	34.99 - 114..8	25.3 - 83.0	Not mentioned
Lagarto No. 1[4/]	Natural	32.0 - 105.0	27.4 - 89.9	Not mentioned
Lagarto No. 2[4/]	Natural	31.7 - 104.0	27.4 - 89.9	Not mentioned
Lagarto No. 3[5/]	Natural	34.75 - 114.0	25.91 - 85.0	15.0 - 49.2

[1/] People around this saddle have cut a trench over 4.6 m (15 ft) deep through the saddle for access to and from the lake.

[2/] This saddle dam has a filter on the downstream slope to protect it against piping. This filter was made in 1987. There are also three iron pipes for water level measurements in the downstream slope.

[3/] At the top of the saddle there are some homes.

[4/] Some houses have been built on this saddle dam and the main road from Escobal to Gatun crosses the saddle dam.

[5/] There is a house on this saddle dam.

Dredging and Operational Facilities

ACP has numerous dredging piers, fuel barge piers, tug and launch landings, and other operational facilities around Gatun Lake that will likely be impacted by the project. Typically the facilities are in satisfactory to good condition. However many will require modifications to accommodate a higher MOLL. A summary of these facilities and the controlling critical elevations can be found in Table 4-28.

Table 4-25: Critical Elevations for ACP Dredging and Other Operational Facilities With Fixed Docks

Facility	Top of Dock Elevation
Pedro Miguel Met & Hyd Station	27.37 m (89.8 ft)
Las Cruces Landing	27.10 m (88.9 ft)
Paraiso Launch Landing	27.58 m (90.5 ft)
Paraiso Tug Landing	27.50 m (90.2 ft)
Siri Landing, Gamboa	27.43 m (90.0 ft)
Gamboa Tug Landing	27.74 m (91.0 ft)
Gamboa Launch Landing	27.13 m (89.0 ft)
Gamboa Boat Slips/Landing	26.91 m (88.3 ft)
Dredging Division Main Dock	28.35 m (93.0 ft)
Atlas Landing	28.35 m (93.0 ft)
Tender/Tug Landing	28.35 m (93.0 ft)
Hydrographic Survey Pier, Gamboa	26.91 m (88.3 ft)
Gamboa Fuel Barge Landing	27.37 m (89.8 ft)
Marginal Fueling Wharf	27.68 m (90.8 ft)
Dock 45, Gatun	27.25 m (89.4 ft)
Mitre Gate Repair Pier	27.25 m (89.4 ft)
Gatun Diving Facility Pier	28.16 m (92.4 ft)
Gatun Tug Landing	27.28 m (89.5 ft)
Gatun Launch Landing	27.28 m (89.5 ft)
Gatun Water Bus Landing	26.97 m (88.5 ft)
Gatun Fuel Barge Landing	27.25 m (89.4 ft)
Gatun Small Boat Pier	27.10 m (88.9 ft)

Source: Recent Site Investigation and Supplemented by the USACE 1999 Report

Representative photos of some of the facilities identified above are provided in Photographs 8 through 15 in Appendix H.

Additionally, floating docks are located at Las Cruces Landing, Hydrographic Survey Pier in Gamboa, and next to the Fuel Barge Landing in Gatun. The guide piles or lateral support mechanisms for these docks define their critical elevations. Figure 4-13 shows

the floating docks at Las Cruces Landing, with additional photographs included in Appendix H of this report.

Figure 4-13: Floating Docks at Las Cruces Landing

Third Party Installations

Panama Canal Railroad

According the to 1999 USACE Report, the Panama Canal Railroad runs approximately 75.6 m (47 mi) through the Isthmus from Colon to Panama City. Its route closely follows the alignment of the Panama Canal. The railway runs through or immediately adjacent to Gatun Lake and Miraflores Lake for approximately half its length. During the construction of the Canal, approximately 71,277,460 M^3 (93,224,000 CY) of material was deposited into the railroad embankments. According to the 1999 USACE report, the railway elevation at bridge crossings in Gatun Lake are 29.87 m (98 ft) MSL with a low steel elevation of 28.04 m (92 ft) MSL, indicating that the railroad is approximately 1.37 m (4.5 ft) above the existing maximum operating lake level.

However, the 1945 Randolph Report states that the critical grade across Gatun valley is 28.58 m (93.75 ft). The ACP Flood Control Manual states that the railroad fill at Monte Lirio is at 28.04 m (92.00 ft). During the field investigation, the project team found that the low chord elevation was 27.8 m (91.2 ft) while the top of the railway fill near the bridge was approximately 29.57 m (97.0 ft). The Panama Canal Railroad recently completed a large project where the majority of the railway in Gatun Lake was raised by 0.31 to 0.61 m (1 to 2 ft). Based on the latest drawings, the lowest elevation along the track is now 28.87 m (94.7 ft).

It was also reported by Panama Railroad officials that one section of the rail bed was subject to waves pushing rocks up onto the track. However, it appears that the problem is more of a function of the rock being undersized for the ship wash being experienced at this site. Based on Hudson's stability criterion and the average rock size reported by Panama Canal Railroad (PCR), it would take a wave of at least 1.37 m (4.5 ft) to move this size of rock.

Bridges

Based on the recent site inspection and supplemented by the 1945 Randolph Report, the following bridges in Table 4-26 will be impacted by increasing the MOLL.

Table 4-26: Critical Elevations in Feet for Other Bridges

Bridge	Controlling Critical Elevation
Gamboa (bottom of steel)	28.65 m (94.0 ft)
Monte Lirio (bottom of steel)	27.8 m (91.2 ft)

The Monte Lirio Bridge is a bascule railroad bridge consisting of steel girders supported on concrete piers. The Gamboa Bridge supports the Panama Railroad and a single lane of vehicle traffic. The bridge consists of steel girders supported on concrete piers, and spans over the Chargres River.

Figure 4-14 and Figure 4-15 provide elevation views of the Monte Lirio Bridge and Gamboa Bridge, respectively.

Figure 4-14: Monte Lirio Bridge

Figure 4-15: Gamboa Bridge

Water Intake Structures

Gatun lake hydropower intake structure. The Gatun Lake Hydroelectric plant lies between the lock and dam and consists of six (6) Francis type turbine units. The units vary in size and each are served by a 3.05 m (10 ft) diameter penstock with the top of the intake at elevation 922.40 m (73.5 ft) MSL. Intake and control for each penstock is located in a gate structure located immediately at the north end of the Gatun Spillway. Each penstock is gated at its inlet and has a draft vent pipe immediately inside the face of the concrete intake structure. The vent pipes are 0.914 m (3 ft) in diameter and their inlets are centered at elevation 27.89 m (91.5 ft) MSL. The gate house which rests atop the intake structure has its floor at elevation 28.96 m (95 ft) MSL.

Mount Hope

The tower's concrete floor is at elevation of 27.95 m (91.7 ft) MSL and will likely not be affected too greatly. However, the intake tower has a top elevation of 27.58 m (90.5 ft) MSL and will require modification. The steel walk bridge leading to it would also require modification to maintain critical operational equipment access if this is a major concern. Figure 4-16 shows the inlet structure at the Mount Hope location.

Figure 4-16: Mount Hope Water Intake Structure

Paraiso

The Paraiso potable water intake facility is in good condition, but will require some modification to accommodate a higher MOLL. The tower's concrete floor elevation is at 27.74 m (91.0 ft) MSL with the centerline of the discharge pipe located at elevation 28.37 m (93.08 ft) MSL. There is a steel sheet pile bulkhead delineating the waterfront perimeter of the facility, with the top at elevation 27.58 m (90.5 ft) MSL, and a timber pier at elevation 27.22 m (89.3 ft) MSL. The entry level to the pump station is around elevation 27.74 m (91.0 ft) MSL. Photograph 2 in Appendix H shows the lower deck elevation of the Paraiso Water Intake.

Gamboa

The Gamboa potable water intake facilities are also in good condition, but will require modification for a higher MOLL. The tower's concrete floor elevation is 28.65 m (94.0 ft) MSL and the grated walkway around the base of the tower is at EL 26.91 m (88.3 ft) MSL. The stone bulkhead that delineates the waterfront is at EL 26.91 m (88.3 ft) MSL. The stone bulkhead and walkway around the base of the tower will most likely need to be raised. Photograph 3 in Appendix H provides an elevation view of the Gamboa Water Intake.

La Represa

The La Represa water intake is in good condition, with the main deck at elevation 31.21 m (102.4 ft) MSL with the centerline of the discharge pipe located at approximately 29.87 m (98.0 ft) MSL. This structure will not require retrofitting or raising to accommodate a

higher MOLL. Photograph 4 in Appendix H provides an elevation view of the Water Intake.

Laguna Alta

It is understood that the Laguna Alta Water Intake facility was built in 2002. The structure is in excellent condition with the top of intake floor at elevation 28.93 m (94.9 ft) with the centerline of the discharge pipe located at elevation 29.38 m (96.4 ft) MSL The floor of the pump house building is at approximately elevation 30.48 m (100 ft) MSL. This structure will not require retrofitting or raising to accommodate a higher MOLL. Photograph 5 in Appendix H provides an elevation view of the water Intake.

Other Structures

In addition to the ACP facilities located around the lake, there are numerous third party structures and boat docks that will be impacted by the project. While many of the structures are either small houses near the water or boat docks for residential use, there are larger structures around some of the local resorts, villages and at the Smithsonian research center on Barro Colorado, shown in Figure 4-17, below.

Figure 4-17: Landing Docks at the Smithsonian Center on Barro Colorado

A representative summary of the larger facilities and the controlling critical elevations can be found below in Table 4-27. The full summary of other structures is listed in Appendix H

Table 4-27: Representative Critical Elevations for Third Party Structures Which Have Fixed Docks (Obtained During Recent Site Investigation and Information Provided by the Smithsonian)

Facility	Top of Dock Elevation
Private Boat Landing at Gamboa Resort	26.91 m (88.3 ft)
Public Boat Landing in Gamboa	27.07 m (88.8 ft)
Gamboa Resort Outdoor Facilities	27.68 m (90.8 ft)
Gamboa Resort Storage Building	26.76 m (87.8 ft)
Smithsonian Landing near Gamboa	27.68 m (90.8 ft)
Smithsonian Facility Main Pier – Barro Colorado	27.07 m (88.8 ft)
Smithsonian Facility Floating Piers – Barro Colorado	27.65 m (90.7 ft)
Tourist Pier in Gatun	28.62 m (93.9 ft)
Timber Pier/Floating Docks at Melia Resort	27.10 m (88.9 ft)

One area of concern is at the Gamboa Rainforest Resort. It appears that much of the park and recreational area behind the hotel is around elevation 27.7 m (91 ft). The size of this area is approximately 40,000 square meters (10 acres) and would be flooded if the MOLL is increased more than 0.9144 m (3 ft). Possible remedies would be to raise the existing grade, which would be quite expensive. A less expensive alternative would involve installing a dike around the shoreline perimeter to prevent flooding the area.

In addition to the larger facilities, the consultant investigated multiple villages to determine the potential impact on houses, schools, and public facilities. The villages included Santa Rosa, Escobal, El Limon, Cuipo, La Providencia, La Represa, La Arenosa, Lagartera, and La Laguna.

Representative photos of some of the structures and villages identified above are provided in Photographs 17 through 26 in Appendix H.

4.4.2 Assessment of Impacts

Approach

In order to determine the structures and facilities that will require retrofitting or replacing to accommodate the increased MOLL, it is necessary to determine the controlling MOLL for each structure. The criteria were determined based on the potential wind waves and boat wake waves and the associated wave run-up for each region throughout Gatun Lake and its tributaries. In comparing potential wind waves versus boat wakes (based on the analysis completed in Section 4.1, it was found that the boat wakes would control. For the purposes of this analysis, it was assumed that a boat wake wave height of 0.61 m (2 ft) are frequent and should be the basis of design. For this wave condition, it was found that any structure that is less than 0.61 m (2 ft) higher in elevation than the MOLL will likely require major retrofitting or replacing (based on wave overtopping and damage criteria). It was also determined that structures that are between 0.61 to 1.07 meters (2 to 3.5 feet) higher than the MOLL would likely require minor retrofitting to accommodate wave overtopping for pedestrian risk.

Assessment of Physical Impacts

Given the freeboard limitations described above, an assessment of the physical impacts was undertaken. A spreadsheet was created that included a description of each structure along with the critical elevations associated with it. The freeboard criteria could then be programmed along with the opinion of probable cost for both the major and minor retrofit conditions. Columns for each MOLL were then added and programmed such that the MOLL minus the required freeboard could be checked to determine the opinion of probable cost to retrofit/replace the given structure for each MOLL. The costs for all affected structures for a given MOLL could then be summed to determine the total retrofit/replace costs. Appendix H contains the spreadsheet created for this assessment. Discussions of individual modifications and estimated costs for the given structures can be found in Section 4.5.

4.5 Environmental Impact

4.5.1 Introduction

This section presents an analysis of the environmental impacts that potentially may occur during the construction and operation phases of the increases in the Maximum Operating Lake Level. The methodology used in evaluating the impacts is described earlier in Chapter 3 under Section 3-7.

4.5.2 Impact Assessment

The results from the evaluation of impacts for the construction and operation phases of the MOLL are presented in Table 4-28 and Table 4-29, respectively, and Appendix F (MOLL Impacts)

Table 4-28: Evaluation Impacts – Construction Phase – MOLL

ENVIRONMENT	COMPONENT	INDICATOR	PROJECT PHASE					
			Construction					
			Transportation and Mobilization	*Site Preparation*	*Construction of Infrastructure and Utilities*	*Retrofit of Saddle Dams*	*Retrofit of Locks*	*Reconstruction*
PHYSICAL	Air	A-1						
	Noise	R-1						
	Water	H-1						
		H-2						
	Soil	SU-1						
		SU-2						
		SU-3						
		SU-4						
BIOLOGICAL	Terrestrial and aquatic flora and fauna	FF-1						
		FF-2						
		FF-3						
		FF-4						
SOCIAL	Social	S-1						
		S-2						
		S-3						
		S-4						
		S-5						
		S-6						
	Economic	E-1						
		E-2						
		E-3						
		E-4						
	Cultural	AR-1						

Table 4-29: Evaluation Impacts – Operation Phase – MOLL

ENVIRONMENT	COMPONENT	INDICATOR	PROJECT PHASE	
			Operation	
			Operating Gatun at Maximum Level	*Maintenance of the Spillway and Structures*
PHYSICAL	Air	A-1		
	Noise	R-1		
	Water	H-1		
		H-2		
	Soil	SU-1		
		SU-2		
		SU-3		
		SU-4		
BIOLOGICAL	Terrestrial and aquatic flora and fauna	FF-1		
		FF-2		
		FF-3		
		FF-4		
SOCIAL	Social	S-1		
		S-2		
		S-3		
		S-4		
		S-5		
		S-6		
	Economic	E-1		
		E-2		
		E-3		
		E-4		
	Cultural	AR-1		

Physical Environment

Air Quality

Construction Phase

The demolition/retrofit of existing ACP structures that cannot function at the proposed Gatun Lake Maximum Operating Level, the demolition of non-ACP structures such as homes and commercial establishments that are below 27.43 m (92 ft) in elevation, and the transportation of the equipment needed for demolition are likely to cause air quality impacts. The maximum air quality impacts during construction are anticipated to be temporary and intermittent from construction sources. These emissions will occur during the construction period when heavy equipment is engaged in the earthwork operations during the demolition, grading, and compaction of the sites.

The air pollutant emissions due to these sources are primarily associated with potential dust or particulate matter (PM) associated with earthwork operations and potential demolition. Air quality impacts during construction/demolition are considered to be a local impact, since the dispersion of dust will be less than 1 km from the any ACP structures that will require retrofitting or any third party structures scheduled for demolition.

Construction equipment used for demolition and site preparation will produce emissions to the atmosphere from the combustion of fuels such as diesel and gasoline. These gases include NOx, hydrocarbons, CO, PM, and SO2. These emissions are expected to be temporary and intermittent during the construction phase producing low air quality impacts.

Operation Phase

The air quality impacts during the operation of the MOLL are expected to be the same as the existing air quality conditions prior to the implementation of this MOLL and therefore these impacts are classified as neutral. The predicted air quality as established from a literature review as presented in the Existing Conditions Report indicates that the ambient air quality of the Direct and Indirect Impact Area are predicted to be significantly below the World Bank Ambient Air Quality Guidelines (World Bank, 1999).

Noise

Construction Phase

Noise impacts from the transportation of construction equipment, materials, and personnel on-site for affected structures requiring retrofitting will be temporary and intermittent, as will the noise impacts associated with demolition of impacted third party structures. All construction activities associated with the retrofit of ACP structures will be

carried out within ACP controlled lands that are more than 8 kilometers from the nearest residences such as Nueva Provencia or other undocumented residences near the sites. In the early stages of retrofitting affected ACP structures and infrastructure construction, work performed onsite will consist entirely of grading, surveying, soil boring, and test drilling required for determining specific geophysical characteristics.

The noise levels generated during the construction phase are expected to be localized, intermittent, and of short-term duration. The noise level is not expected to constitute a significant adverse impact. Noise levels produced during construction may cause the limited wildlife in the area to migrate temporarily away from the sites to areas were they will not be disturbed. For these reasons, the noise impacts associated with demolition, retrofitting, and construction of infrastructure and utilities have been classified as moderately negative while other construction related activities have been classified as slightly negative.

Operation Phase

The noise impacts associated with the operations phase of the MOLL are predicted to be at the levels of noise that currently exist from the normal operation of the Canal and Gatun Spillway.

Soils and Landforms

Operation Phase

The impacts anticipated from the proposed maximum operating level of Gatun Lake at 27.43 m (90 ft) are related to the potential changes on the Gatun Lake coast line morphology as a result of a higher water level. A higher maximum operating level of the Gatun Lake will result in an incremental loss of land area, a modification of the shoreline topography around the Gatun Lake, and increase the potential for erosion, and land slides. In some cases the loss of property as a result of a higher maximum operating level may result in homeowners whose dwelling lies close to the shore line relocating to higher ground. This action may result in additional clearing and/or deforestation of additional areas resulting in an increase to shoreline erosion and sediment transport to the lake as additional land is removed of protective vegetation, canopy, and soil matrix. In addition, the physical and chemical quality of the soil may be altered by accidental hydrocarbon spills or other process-related substances during the operation of the MOLL. Impacts to soil and landforms from operation of the MOLL range from neutral to slightly negative.

Water Resources

Construction Phase

Demolition and retrofitting of ACP and third party infrastructure is expected to have a slightly negative impact on water resources due to increasing in turbidity associated with these activities. Turbidity increases are expected to be localized, minor, and temporary.

Operation Phase

The operation of Gatun Lake at a higher elevation has a negative impact in increasing the potential to accelerate sedimentation by eroding shoreline soils. Sedimentation has a direct effect on turbidity that impacts aquatic habitats, and recreational and sustainable fishing by local communities. A higher water level also has the effect of allowing chemicals substances such as fertilizers, fungicides, and pesticides which may be present in near-shoreline soils that will become inundated to reach Gatun Lake. These chemicals can contribute to the depletion of oxygen content in the lake and contribute to eutrophication, reduce the biodiversity of the lake, and contribute to a reduction of water quality.

The operation of Gatun Lake at a Maximum Operating Level will result in the evaluation of several surface water intake structures and possibly the modification of such structures to accommodate a higher water level. Table 4-6 presents the intake structures that would be impacted by a new Maximum Operating Gatun Lake Level.

The impacts related to the management of surface drainage and discharge of effluents from ACP structures during the operation phase are considered as slightly negative; however, the magnitude of the effect will be reduced in a short-term when all the systems designed for management and treatment are completed and the monitoring of the effluent discharge parameters for ensuring the compliance with both Panamanian and international discharge standards are implemented in order to establish adequate controls.

The environmental management and treatment systems that will be utilized during the operation of the MOLL include the proper management of industrial and domestic wastewater discharges from ACP facilities and equipment.

A potential impact to the water environment is the discharge of contaminated water to the lake. However, this impact can be managed by including the following treatment systems for residual waters prior to discharge:

- An API oil/water separator for surface water runoff contaminated with oil and grease,
- Sanitary sewage treatment units will be used to treat sanitary waste from the ACP facilities, permanent employee camp, and to treat sewage from spillway facilities.
- Waste streams are discharged to lake after treatment through the appropriate treatment system.
- Water collected from the process areas and utility areas will be routed to the API oil/water separator. Collected oil is stored in appropriate tanks and is then trucked to an offsite disposal at an approved recycle/disposal facility.
- Plant effluent will be monitored to ensure compliance with Panamanian and/or applicable World Bank Guidelines for Liquid Effluents for Process Wastewater, Domestic Sewage and Contaminated Stormwater Discharge to Surface Waters effluent limits for those parameters applicable.

Potential impacts to water resources from the operation phase of the MOLL range from neutral to moderately negative, with most impacts classified as slightly negative.

Biological Environment

Terrestrial and Aquatic Flora and Fauna

Construction Phase

Retrofitting of the saddle dams to allow for safe operation under the proposed maximum operating level is likely to have a neutral to slightly negative impact on flora and fauna. Many of the saddle dams may need to be cleared so that additional fill can be placed to achieve the desired elevation. Motile species utilizing these areas may be able to relocate to undisturbed habitats within the watershed; however it is uncertain whether these habitats can support the influx of new individuals. Less motile or non-motile species would be directly and adversely impacted by these activities. It is assumed that vegetative cover would be re-established through the planting of tree seedlings, which would create patches of early successional habitats within the watershed. Species utilizing such habitats may benefit from these activities; however, such disturbance may provide additional entry points into the watershed for invasive and exotic species. See Tables 3-8, 3-9, and 3-10, in Chapter 3 for a list of species found in Lake Gatun and the Canal Zone.

Retrofitting the locks and reconstruction of ACP and third party structures is likely to have a neutral to only slightly negative impact on flora and fauna since most construction activities would take place in areas that have been or are currently disturbed by human activities. Most structures are expected to be reconstructed near to, but at a higher elevation, than their original location. Reconstruction of certain structures may result in permanent loss of some secondary forest habitat.

Operation Phase

During the operation phase, significant impacts to the flora and fauna would come from operation of Gatun Lake at the Maximum Operating Level (MOLL). A permanent increase in the water elevation of Gatun Lake would cause changes in the floristic community of the shoreline, which would have varying impacts on faunal populations. Upland vegetation would die off in inundated areas and may be replaced by wetland vegetation if water levels are not prohibitively deep. Emergent wetland vegetation and vegetation types found in saturation wetlands may also die off in areas where the water level has risen due to implementation of the MOLL. Areas of emergent vegetation may transition into open water areas, or areas dominated by submergent or free-floating vegetation. Motile animal species would move into or out of these changing areas depending on the suitability of the habitat for their needs. Less motile species, or species whose preferred habitat is limited, may be negatively impacted by operation at the MOLL because they cannot move out of unsuitable habitats or because preferred habitat areas are not available. Some species are likely to benefit from operation at the MOLL. Several aquatic and wetland species would likely benefit from operation at MOLL due to creation of additional littoral, lacustrine, and palustrine habitats. Impacts to rare and endangered species are expected to be minor as most inundation would occur in areas that have been or are currently disturbed by human activities. The overall impact on flora and fauna of operating Gatun Lake at the MOLL is considered moderately negative.

Social, Economic and Cultural Environment

Social

Construction Phase

Alteration in the Demographic Composition - Population growth in the Canal watershed has increased substantially as verified by the Division of Statistics and Census. Human populations in the Canal watershed have been documented as having grown from 22,000 in 1950 to 113,000 in 1990. This trend continues with the 2000 census indicating population of 141,436 (see Tables 4-3, 4-4, and 4-5). Because the actual territorial watershed of the Panama Canal is not the same as that of 1950, a direct

comparison of actual population growth between the last censuses cannot be made. The rate of population growth in the watershed during the prior census has shown an average growth rate of 4% per annum (1980-1990) in watershed. The growth of population in the watershed has been reported to be greater than the growth of the Metropolitan Panama as exhibited by communities such as La Cumbres and Chilibre, which lie within 1.3 km of the Transisthmian Highway. Although the Soberanía National Park provides some level of protection from growth along the east Gatun Lake, growth has also been noted in rural areas, including villages near and within the watershed national parks. The growth in the watershed has been attributed to its proximity to the Panama City and the Transisthmian Highway (Ibáñez et al., 2002).

ACP maintains jurisdiction on land use up to the 30.5 m (100 ft) elevation around the Gatun Lake. Interoceanic Regional Authority (ARI) has responsibility over Canal related real estate and development plans for their use. During the field reconnaissance studies, it was observed that areas within the shoreline of Gatun Lake exhibited a pattern of past population invasion (induced development). These communities are characterized by the proliferation of sub-standard to standard settlements on properties either belonging to Interoceanic Regional Authority (ARI) or operated by ACP. This population invasion is caused by the migration of individuals and families from nearby Metropolitan Panama/Colon and from interior areas of the country in pursuit of property and employment.

The characteristic of populated areas within the watershed can be described as those who live in communities along Transisthmian Highway such as Chilibre and Las Cumbres and rural communities around the Gatun Lake. Those populated centers close to the highway are employed by industrial and commercial enterprises either locally, or in Panama City or Colon. The rural settlements vary, but the majority of these rural residential areas have neither paved roads nor electricity and use the lake as the main source for subsistence fishing and transportation among local communities. Rural communities at the south end of Gatun Lake have undergone significant deforestation as the land use has changed over time from subsistence farmers to cattle ranches after the farming operations declined in productivity. Table 4-30, Table 4-31, and Table 4-32 present the demographics of the population living around the perimeter of Gatun Lake. The data indicates that a total of 1,862 homes were identified in the 2000 census with a total population of 7,940. The majority (53%) of the population greater than 18 years of age were employed while 16% were engaged in agriculture.

The population of Panama City and Colon, along with communities that lie along the Transisthmian Highway, will continue to increase and, with the growth of the populations in these cities, the pressure to expand into the Canal watershed and (specifically) ACP operated areas within Gatun Lake shoreline will increase. Although many of the settlements that exist around the Gatun Lake appear to be old established communities, evidence of new construction was observed during the site reconnaissance and was

confirmed during the 2000 census. The normal growth of populations and the commencement of the construction associated with this MOLL, as well as other Canal infrastructure projects, will likely increase the migration of people looking for employment and accelerate the invasion of ACP operated lands in the vicinity of the Gatun Lake. This type of illegal settlement will have to be addressed and discouraged because a continued expansion into the watershed will lead to further environmental and social impacts resulting from deforestation, loss of soil protection, erosion induced by intense precipitation, and overall water quality impacts to the Lake from pollution. This type of impact is considered as neutral to moderately negative related to the proposed MOLL but can be a significant impact to the entire Lake Gatun if the rainforest continues to be depleted and is not able to replenish the Canal.

The measures that will be required to protect the watershed include both administrative and enforcement initiatives. Although the government of Panama has set aside much of the remaining forest as parks or preserves, it allows the continuation of land clearing by farmers inviting land speculators to build resorts and condominiums.

Table 4-30: Population Breakdown in the Current Canal Watershed, Province of Panama-Census 2000

		Total Homes	Total Persons	Men	Women	Total Persons Greater than 18 years of age	Employed	Engaged in Agriculture	Unemployed	Economicall y Active	Illiterate
ARRAIJAN											
80101038	BARRIADA OMAR TORRIJOS	1370	5316	2707	2609	3181	1944	25	307	1674	151
80103009	LA GLORIA	91	354	191	163	223	116	23	18	151	24
80103010	LAS GUABITAS	7	24	12	12	14	6	1	1	13	1
80103012	NUEVO EMPERADOR	393	1591	816	775	1035	525	45	122	601	33
80103016	SANTA CLARA	27	112	55	57	67	40	10	4	37	11
80103019	LAS GUABITAS ARRIBA	13	48	24	24	29	16	10	3	16	0
80103022	MOSCU	2	5	3	2	3	2	2	0	2	1
SANTA CLARA											
80104002	HUILE	114	442	251	191	291	146	21	26	184	29
80104003	SANTA CLARA	274	1302	682	620	767	413	104	25	538	61
CAIMITO											
80302005	LORIDA ARRIBA	9	40	20	20	26	14	10	0	19	3
80302008	QUEBRADA GRANDE	8	46	24	22	20	9	8	0	22	4
80305003	BOCA DE LA QUEBRADA LAS LAJA	4	21	11	10	8	4	4	0	9	3
80305007	CIRICITO ARRIBA	7	29	18	11	16	10	7	1	10	6
80305008	CIRI DE LOS SOTOS	64	302	162	140	164	100	83	2	122	22
80305009	QUEBRADA LA CACHORRA	1	8	3	5	2	3	3	0	4	0
80305010	DOS AGUAS	26	153	81	72	79	43	35	4	60	17
80305012	NUEVO PARAISO	29	102	56	46	59	29	20	2	41	6
80305013	LA BONGA O EL CRUCE	12	52	25	27	30	20	12	1	15	6
80305016	LAS LAJAS	18	94	51	43	43	31	28	0	25	7
80305017	LOS CHORROS DE CIRI	38	239	144	95	110	82	73	3	72	13
80305018	LA HAMACA	4	18	12	6	7	4	3	0	6	1
80305022	QUEBRADA BATEAL	2	12	5	7	7	3	2	1	4	0
80305023	QUEBRADA LA CANDILERA	5	19	8	11	13	5	5	0	9	2
80305031	VENADO	14	86	54	32	41	30	25	2	30	13
80305034	NUEVO PROGRESO	14	65	30	35	36	18	17	4	25	13
80305035	PUERTO CIRICITO	1	2	2	0	2	2	0	0	0	0
CIRI GRANDE											
80306002	LOMA DEL NANCE	2	7	2	5	4	2	2	0	4	2
80306004	ARENAS BLANCAS	3	13	5	8	7	3	3	0	5	0
80306012	CIRICITO ARRIBA	52	250	143	107	124	69	58	12	85	39
80306013	CIRICITO ARRIBA No1	17	99	57	42	47	31	30	1	32	15
80306020	GASPARILLAL	17	101	52	49	53	29	20	4	40	4
80306026	LA BONGA ABAJO	29	152	80	72	91	53	35	8	57	15
80306027	LA BONGA ARRIBA O LOS NEGRO	40	209	109	100	106	72	59	1	65	8
80306030	LA GAITA ABAJO	13	58	35	23	36	23	23	1	22	2
80306031	LA GAITA ARRIBA	21	135	62	73	64	38	34	13	36	12
80306043	MALGANAO	14	90	47	43	41	26	19	7	28	6
80306045	MEMBRILLAR	6	32	16	16	16	9	9	0	14	3
80306051	QUEBRADA LA PITA	8	34	20	14	19	14	12	0	11	4
80306068	LA BONGA CENTRO	19	66	42	24	41	23	18	1	34	4
80306078	LA CONGA	22	105	60	45	52	42	16	0	26	11

4–32: Population Breakdown in the Current Canal Watershed, Province of Panama-Census 2000 (Continued)

		Total Homes	Total Persons	Men	Women	Total Persons Greater than 18 years of age	Employed	Engaged in Agriculture	Unemployed	Economicall y Active	Illiterate
80306079	CIRI GRANDE	6	30	9	21	18	7	6	0	18	8
80306081	LA CONGA ARRIBA	17	74	39	35	34	29	18	1	22	5
EL CACAO											
80307001	ALTO DE CIRI	11	57	30	27	25	14	12	0	29	5
80307005	CIRI GRANDE	67	342	188	154	168	122	94	4	119	36
80307006	EL CHILENO	25	132	69	63	61	39	37	7	35	17
80307007	AGUACATE ARRIBA	63	263	146	117	144	93	75	9	95	23
80307008	EL CACAO	133	609	339	270	356	186	89	7	266	39
80307009	EL COCORITO	1	2	1	1	2	1	1	0	1	1
80307010	BAJO BONITO	64	357	205	152	158	88	81	15	130	21
80307011	EL LIMON	14	68	38	30	27	19	17	1	24	6
80307012	EL JAGUA	15	88	56	32	42	20	19	4	35	6
80307013	LAS TINAJAS	9	30	21	9	18	9	8	0	17	7
80307014	EL CAUCHAL	16	72	34	38	35	17	15	3	32	7
80307015	LAS NEGRITAS	33	166	90	76	85	58	54	8	50	16
80307018	ALTAMIRA O MATA PALO	15	54	25	29	31	15	14	1	19	4
80307020	PIRAGUAL	11	50	30	20	25	9	6	5	24	4
80307022	TRINIDAD ARRIBA	36	161	90	71	85	57	57	4	56	13
80307023	TRINIDADA DE LAS MINAS	128	607	318	289	322	181	124	17	238	45
80307024	TRINIDAD DE LOS CERROS O EL C	25	126	66	60	64	30	26	5	48	7
80307028	CIRI GRANDE ARRIBA	24	148	79	69	61	38	34	0	53	16
80307031	LOS RAUDALES	15	71	41	30	35	10	10	7	29	7
80307032	CRICITO ARRIBA No1	5	32	15	17	18	12	11	0	14	5
80307034	CIRICITO ARRIBA No2	6	42	19	23	23	14	14	0	14	4
80307035	QUEBRADA EL NANCE	13	101	52	49	41	28	28	1	36	10
80307036	PEÑA BLANCA	20	130	68	62	68	47	46	0	47	14
80307037	LA TAMBORA	9	56	31	25	24	15	15	1	21	3
80307038	CACAITO	7	32	18	14	17	7	7	4	12	3
80307039	YERBA BUENA	20	112	60	52	49	34	34	5	33	6
80307040	ARENAL	6	33	17	16	17	4	1	3	13	2
80307041	EL NAZARENO	11	63	33	30	32	7	3	16	21	5
LA TRINIDAD											
80308001	BEJUCO O LAS LAJITAS	7	34	21	13	18	11	9	1	15	8
80308002	EL AGUACATE	24	88	47	41	55	30	21	1	35	3
80308003	EL CARAÑO No1	3	5	3	2	4	2	2	0	2	2
80308005	ESCOBAL O ESCOBALITO	16	80	47	33	41	29	23	1	30	10
80308006	EL GASPARILLAL	10	70	35	35	29	14	10	1	30	6
80308008	LA FLORIDA	75	318	164	154	170	109	67	1	112	22
80308010	LA ARENOSA O EL LIMITE	23	89	48	41	55	38	22	5	30	6
80308011	LOS ORTIGALES	3	13	8	5	6	6	6	1	4	0
80308012	LA HUMILDAD	41	176	94	82	92	67	52	0	49	13
80308013	NUEVA ARENOSA	53	260	137	123	128	77	56	2	101	25
80308014	LA PITA	33	198	110	88	97	51	41	1	83	26
80308015	LA TAGUA	1	15	10	5	7	6	6	0	3	0

4–32: Population Breakdown in the Current Canal Watershed, Province of Panama-Census 2000 (contd)

		Total Homes	Total Persons	Men	Women	Total Persons Greater than 18 years of age	Employed	Engaged in Agriculture	Unemployed	Economicall y Active	Illiterate
80308016	LAJA LISA O EL CARAÑO	25	103	58	45	63	40	33	1	37	9
80308017	LOS CAÑONES	55	260	150	110	121	89	78	0	89	16
80308020	PUEBLECILLO	22	101	54	47	54	40	27	3	29	13
80308022	LOS FALDARES	16	66	33	33	37	23	16	1	29	3
80308024	LA HONDA ABAJO O LA SONADOR	42	204	111	93	96	62	55	2	72	22
80308025	HONDA ARRIBA	19	80	41	39	41	27	21	0	24	1
80308026	EL MAMEY	5	44	22	22	20	10	8	0	13	2
80308027	CAÑAZAS	3	15	10	5	6	6	4	0	5	0
80308028	AROSEMENA	4	22	13	9	10	4	4	2	8	0
80308029	LA BONGA ABAJO	2	10	7	3	8	7	7	0	2	1
80308030	QUEBRADA GRANDE	2	10	2	8	5	2	2	0	5	0
LIDICE											
80310027	CERRO TRINIDAD	10	40	24	16	24	19	17	1	12	3
SANTA ROSA											
80313001	ARENAS BLANCAS	17	69	43	26	39	29	24	1	21	2
80313004	BOCA DE LA QUEBRADA LAS LAJA	1	3	2	1	3	2	2	0	1	0
80313007	EL CEDRO	4	26	15	11	14	13	12	0	4	1
80313011	LAS LAJAS	6	27	14	13	11	9	9	0	8	4
80313012	LAS PETRAS	25	109	57	52	62	36	33	0	44	12
80313015	QUEBRADA AMARILLA	9	44	24	20	23	15	12	1	14	1
80313016	QUEBRADA BEJUCO	9	46	28	18	27	8	7	8	20	5
80313018	QUEBRADA COSTILLA	1	6	2	4	2	2	1	0	2	0
80313019	QUEBRADA LA PITA	1	1	1	0	1	1	1	0	0	0
80313020	QUEBRADA LAS PETRAS	7	27	13	14	14	11	10	0	9	1
AMADOR											
80703001	CAÑO DEL GIGANTE	7	12	7	5	9	6	5	0	4	3
80703002	CAÑO GRANDE	2	12	8	4	6	4	2	3	2	2
80703003	CERRO CAMA	240	1177	631	546	702	362	126	52	476	61
80703007	GIGANTE	6	25	13	12	11	5	3	2	13	2
80703008	LAGARTERA	62	348	194	154	139	73	51	13	125	27
80703009	LAGARTERITA O AMADOR	37	138	73	65	85	44	24	13	49	11
80703010	LAS PAVAS	53	233	122	111	115	64	44	17	76	20
80703011	LOS HULES ABAJO	116	499	267	232	297	149	88	22	216	57
80703012	TINAJONES ABAJO	10	34	20	14	23	14	9	4	11	0
80703013	TINAJONES ARRIBA	31	164	85	79	87	48	24	8	67	6
80703014	CAÑO QUEBRADO ARRIBA	2	12	8	4	10	5	4	3	3	0
80703016	ISLA BAJO	1	2	1	1	2	1	1	0	1	0
80703017	ISLA MINA	3	19	11	8	9	3	3	2	5	0
AROSEMENA											
80704001	AROSEMENA	32	125	67	58	64	30	17	10	52	7
80704005	DIVISA	11	59	33	26	22	14	7	1	17	1
80704008	EL CRUCE DE AROSEMENA	16	64	33	31	47	28	13	1	22	14

4–32: Population Breakdown in the Current Canal Watershed, Province of Panama-Census 2000 (Continued)

		Total Homes	Total Persons	Men	Women	Total Persons Greater than 18 years of age	Employed	Engaged in Agriculture	Unemployed	Economicall y Active	Illiterate
80704010	QUEBRADA GRANDE ARRIBA	8	32	20	12	23	13	10	0	14	5
80704011	LA DIVISA	3	9	6	3	6	6	3	0	2	1
80704012	LOS COROZALES ADENTRO	1	1	1	0	1	1	0	0	0	1
EL ARADO											
80705004	EL LIRIO	98	423	233	190	255	141	73	13	165	26
80705005	LA CELESTINA	16	68	35	33	40	25	13	2	30	11
80705007	MOSCU	8	32	18	14	18	12	6	1	11	5
80705014	CABECERA DE BERNARDINO	2	8	4	4	5	3	3	0	4	2
80705016	CERRO VIEJO	4	10	5	5	8	5	5	0	3	2
HERRERA											
80709003	CAÑO QUEBRADO ARRIBA No1	8	35	21	14	25	15	12	0	12	2
80709004	CERRO LA SILLA	3	6	4	2	6	4	3	0	2	4
80709006	GATO DE AGUA	3	23	12	11	11	5	5	0	12	3
80709012	RIECITO	9	32	21	11	22	16	10	1	8	1
80709016	LAS ZANGUENGAS	35	134	81	53	80	55	32	3	44	9
HURTADO											
80710007	DIVISA	6	37	18	19	22	10	6	1	16	7
ITURRALDA											
80711001	ARENOSA	56	242	138	104	140	82	16	17	88	9
80711003	CAÑO QUEBRADO ABAJO	8	29	17	12	14	7	4	1	10	4
80711004	COCACOLA	6	13	8	5	11	6	5	0	5	2
80711005	CUCHIRVO	5	21	14	7	11	8	6	0	6	2
80711006	CERRO CAMA	2	8	3	5	4	2	2	0	4	1
80711007	LA COLORADA	94	415	203	212	235	131	64	14	161	30
80711008	LA LEONA	3	18	13	5	11	9	7	1	5	1
80711009	LOS HULES ARRIBA	16	72	44	28	39	21	12	7	20	4
80711013	QUEBRADA BONITA	1	2	2	0	1	1	1	0	1	0
80711017	EL ZAHINO O COROZALES ADENTR	10	36	20	16	17	11	10	0	15	0
80711018	EL AMARGO	1	9	5	4	4	2	2	0	3	0
80711019	EL IGUANO	14	39	21	18	23	16	11	2	15	3
80711020	CAÑO GRANDE	4	17	11	6	11	7	6	1	8	1
80711021	LOS COROZALES ADENTRO	2	6	4	2	6	1	0	1	4	0
LA REPRESA											
80712001	ALTOS DEL JOBO	68	277	158	119	162	84	48	5	113	23
80712002	CALABACITO	4	13	9	4	10	7	2	0	6	4
80712004	CAÑITO	3	11	6	5	4	3	3	0	7	2
80712005	EL CAÑITO	18	58	36	22	42	22	18	1	26	5
80712007	EL CUIPAL	6	25	13	12	16	5	2	1	10	5
80712008	LA LAGUNA	51	246	122	124	148	70	8	17	98	9
80712010	PUEBLO NUEVO	15	47	26	21	29	16	10	1	23	2
80712011	QUEBRADA LAGARTO	3	13	8	5	10	4	0	0	6	4

4–32: Population Breakdown in the Current Canal Watershed, Province of Panama-Census 2000 (Continued)

		Total Homes	Total Persons	Men	Women	Total Persons Greater than 18 years of age	Employed	Engaged in Agriculture	Unemployed	Economicall y Active	Illiterate
80712013	RIO PESCADO	2	6	3	3	6	2	2	0	4	0
MENDOZA											
80714001	CAÑO QUEBRADO ABAJO	23	116	62	54	61	28	18	8	52	4
OBALDIA											
80715005	BAJITO VERDE	5	28	16	12	15	9	8	2	9	6
80715010	CAÑACITA	8	29	21	8	21	13	4	2	11	8
ANCON											
80814010	CHIVA CHIVA	2	2	2	0	2	1	0	0	1	0
80814014	PEDRO MIGUEL	175	543	222	321	422	149	0	38	279	2
80814015	PARAISO	283	1087	514	573	778	360	2	102	466	10
80814052	MOCAMBO ARRIBA	6	20	12	8	13	8	2	0	9	0
80814088	NUEVO CHAGRES	109	435	245	190	227	124	10	29	154	27
80814090	PEDRO MIGUEL ADENTRO	2	8	3	5	6	1	0	2	3	0
CHILIBRE											
80815005	CAIMITILLO	253	1017	524	493	603	352	14	65	348	22
80815006	CALZADA LARGA	323	1318	697	621	803	435	37	91	500	23
80815007	CERRO BACHICHE	6	31	14	17	17	9	4	0	17	0
80815009	CHILIBRE	1521	6246	3277	2969	3966	2007	42	419	2488	122
80815011	BOQUERON ABAJO	1	1	1	0	1	1	1	0	0	0
80815012	LA BONGA O 260	26	172	93	79	65	12	4	17	72	27
80815013	GUARMAL	308	1299	652	647	738	410	13	74	484	49
80815017	EL CEDRO	20	66	38	28	41	24	2	7	20	2
80815021	NUEVO CAIMITILLO	234	1007	536	471	558	324	56	57	359	63
80815022	LA PUENTE ARRIBA	29	120	71	49	69	41	13	7	41	3
80815023	LA VENTA	2	3	3	0	2	2	1	0	1	1
80815027	QUEBRADA ANCHA	37	145	93	52	79	51	29	1	55	0
80815028	QUEBRADA BENITEZ	8	30	18	12	17	9	6	4	9	3
80815030	MAURO O SANTA LIBRADA	6	14	11	3	12	3	3	5	5	5
80815031	QUEBRADA FEA	10	27	17	10	13	7	6	2	9	6
80815032	PELUCA	3	11	8	3	8	6	6	0	5	1
80815033	QUEBRADA ÑAJU	2	5	3	2	3	2	1	0	2	1
80815034	QUEBRADA OSCURA	2	4	3	1	3	2	2	0	2	2
80815036	SAN FRANCISCO	7	29	17	12	14	4	4	4	12	2
80815038	QUEBRADA TRANQUILLA	32	143	80	63	71	37	12	2	61	9
80815040	RIO INDIO	23	40	28	12	39	28	10	0	12	6
80815041	SAN JUAN DE PEQUENI	21	95	53	42	46	10	6	9	38	4
80815042	SABANAS DE CHILIBRE	108	408	217	191	236	128	29	34	137	26
80815044	VICTORIANO LORENZO	35	145	84	61	75	42	10	4	61	12
80815046	SAN VICENTE	2488	10333	5258	5075	6117	3497	82	661	3661	243
80815049	VILLA UNIDA	3223	13095	6596	6499	7791	4147	85	884	4885	248
80815050	QUEBRADA PEÑA BLANCA	14	51	29	22	24	16	15	3	17	7
80815052	ALTOS DEL LAGO	12	28	21	7	19	15	1	1	6	1

4–32: Population Breakdown in the Current Canal Watershed, Province of Panama-Census 2000 (Continued)

		Total Homes	Total Persons	Men	Women	Total Persons Greater than 18 years of age	Employed	Engaged in Agriculture	Unemployed	Economicall y Active	Illiterate
80815056	SAN CRISTOBAL	27	80	52	28	43	34	26	2	30	20
80815057	QUEBRADA CANDIDO	2	2	2	0	2	2	2	0	0	0
80815058	QUEBRADA SAN ANTONIO	2	6	4	2	6	3	3	0	3	2
80815060	QUEBRADA MONO CONGO	21	94	56	38	41	27	22	2	30	3
80815063	QUEBRADA SAN JUAN	3	6	4	2	4	3	3	0	2	0
80815064	CABECERA DE QUEBRADA TRANC	1	7	5	2	7	5	5	0	2	1
80815065	COMUNIDAD EMBERA DRUA O DO	16	76	36	40	38	21	1	2	29	14
80815066	EL REPORTE VIEJO	1	1	0	1	1	0	0	0	1	0
80815067	QUEBRADA FEA ARRIBA	1	6	4	2	2	1	0	0	3	0
80815068	SAN ANTONIO	95	335	180	155	196	124	8	33	89	6
80815069	CAIMITILLO CENTRO	65	247	138	109	165	82	5	15	103	26
80815070	CALLE DEL IDAAN	14	62	30	32	34	20	0	4	16	0
80815073	AGUA FRIA	1	1	1	0	1	1	0	0	0	0
80815074	RIO PIEDRA	5	11	7	4	10	5	5	1	4	5
80815078	LOS ALTOS DE CERRO AZUL	45	94	60	34	80	65	2	5	19	1
80815081	BOQUERON ARRIBA	4	14	8	6	9	2	2	2	6	4
80815082	CABECERA DE QUEBRADA CULEE	1	1	1	0	1	1	1	0	0	0
80815083	CABECERA DE RIO PIEDRA	1	1	1	0	1	1	1	0	0	1
80815084	CANDELARIA ARRIBA	4	13	7	6	8	3	2	0	7	2
80815085	CIUDAD EL AMANECER	202	701	331	370	445	343	3	34	148	2
80815086	FILO DEL MAMEY	2	2	2	0	2	2	2	0	0	2
80815088	LA POLICIA	2	3	2	1	2	2	2	0	1	0
80815089	LA PUENTE	13	36	21	15	28	19	13	2	11	2
80815090	NUEVO MEXICO No. 1	195	736	372	364	396	232	3	50	217	6
80815091	NUEVO MEXICO No. 2	209	650	356	294	375	228	1	42	190	16
80815092	PARANA PURU O PUEBLO MAQUE	13	42	23	19	24	9	0	0	25	4
80815093	QUEBRADA BEJUCOSA	1	1	1	0	1	1	1	0	0	1
80815094	SAN PABLO	1	4	4	0	2	2	2	0	0	1
80815095	SANTA LIBRADA	2	8	2	6	5	0	0	1	5	0
80815096	URBANIZACION SAN LORENZO	184	661	314	347	429	323	7	27	161	1
LAS CUMBRES											
80816001	ALCALDE DIAZ	4375	18277	9090	9187	11698	6915	42	1194	6480	268
80816005	LA CABIMA	3492	14270	7131	7139	9092	5710	27	875	4594	262
80816007	LA LAGUNA O EL TECAL	15	31	24	7	23	20	10	2	5	3
80816017	MOCAMBO ARRIBA	117	539	272	267	279	142	16	40	200	10
80816047	MARIA HENRIQUEZ	46	152	82	70	99	52	7	14	46	13
PACORA											
80817079	LOS ALTOS DE CERRO AZUL	9	33	18	15	23	15	0	0	16	0
			37203								

Table 4-31: Population Breakdown in the Current Canal Watershed, Province of Colon-Census 2000

		Total Homes	Total Persons	Men	Women	Persons Greater than 18 years of	Employed	Engaged in Agriculture	Unemployed	Economically Active	Illiterate
BUENA VISTA											
30103001	ALTAMIRA	2	6	4	2	5	2	2	2	2	0
30103002	ALTO LA GLORIA	21	77	40	37	49	18	2	10	29	1
30103004	BUENA VISTA	689	2826	1411	1415	1691	1000	23	129	1009	49
30103006	EL GIRAL	301	1210	629	581	751	423	20	64	450	18
30103009	LA COCLESANA	28	122	65	57	70	31	3	10	46	1
30103010	NUEVO VERAGUAS O LA ESCANDALOSA	16	34	27	7	26	13	5	1	15	0
30103011	LAS TABLITAS	56	235	127	108	118	57	4	22	87	4
30103013	LOS PLAYONES	26	99	58	41	55	29	14	3	36	0
30103015	PEÑAS BLANCAS	77	363	196	167	184	74	20	36	151	4
30103016	PUEBLO GRANDE O ENTRADA DE SARDINILLA	156	653	335	318	393	194	3	48	261	17
30103017	QUEBRADA ANCHA O PUEBLO GRANDE (P)	85	377	194	183	217	127	14	17	124	3
30103018	QUEBRADA BONITA	81	323	162	161	195	121	2	13	110	10
30103019	QUEBRADA BONITA ADENTRO	370	1669	854	815	915	480	34	67	655	22
30103020	RIO DUQUE	249	1077	552	525	644	351	10	59	398	20
30103023	LA LOMA DE LA GLORIA	3	10	8	2	7	2	1	1	4	0
30103024	FRIJOLITO (P)	10	31	22	9	19	8	4	4	12	0
30103025	BARRIADA NUEVA ESPERANZA No.1	16	69	30	39	41	23	1	2	24	2
30103026	BARRIADA NUEVA ESPERANZA No.2	29	121	64	57	65	35	0	7	37	1
30103027	ALTOS DE DIVISA	79	379	203	176	212	108	12	22	160	2
30103028	CAMPO ALEGRE	61	252	121	131	130	70	7	13	86	6
30103030	RINCON CALIENTE	6	24	15	9	13	8	0	0	5	0
30103031	DOS RIOS	57	269	132	137	122	64	2	19	86	3
30103032	EL VALLE DEL GIRAL	42	183	98	85	101	57	2	5	75	3
30103033	RIO GATUN	4	16	10	6	8	4	4	0	9	0
30103034	SARDINILLA (P)	1	3	1	2	2	2	0	0	0	0
CIRICITO											
30105001	ARAÑAGATAL (P)	13	54	32	22	35	21	15	2	21	1
30105002	ARRECIFAL	3	6	3	3	6	2	1	0	4	0
30105003	CA-A BRAVA	14	74	36	38	47	26	7	6	20	3
30105004	CAÑO DE CIRICITO	2	10	7	3	5	1	1	0	5	0
30105005	CAÑO DE VIVIANO	5	10	7	3	9	8	5	0	2	0
30105006	CAÑA BRAVA No.1 (…CA-O QUEBRADO)	3	20	8	12	10	8	4	0	6	2
30105007	CIRICITO (P)	23	72	38	34	49	31	12	1	25	2
30105008	LOS CHORROS DE CIRI GRANDE (P)	18	92	50	42	46	22	16	9	31	1
30105009	CUIPO (P)	46	239	127	112	133	70	4	10	90	1
30105010	CHORRILLO (P)	90	449	245	204	228	113	44	36	174	7
30105011	EL CONGAL (P)	21	97	49	48	57	33	20	2	34	4
30105013	FRENTE DE CIRICITO	5	19	12	7	12	8	7	0	5	0
30105015	LA HUMILDAD (P)	11	37	19	18	23	13	9	0	14	2
30105017	LOS CEDROS DEL LAGO (LOS CEDROS)	21	80	44	36	43	23	15	3	31	1
30105019	MARAÑONCITO	58	266	142	124	154	72	9	16	116	3
30105020	NUEVA AMERICA	64	349	200	149	177	99	20	10	138	4
30105021	PABLON (P)	10	44	26	18	21	13	13	0	18	0
30105022	PALMIRA	1	11	6	5	7	7	4	0	1	0

4-33: Population Breakdown in the Current Canal Watershed, Province of Colon-Census 2000 (Continued)

		Total Homes	Total Persons	Men	Women	Persons Greater than 18 years of	Employed	Engaged in Agriculture	Unemployed	Economically Active	Illiterate
30105024	EL PEPINO (P)	9	39	21	18	22	13	7	0	20	0
30105025	NUEVO PORVENIR	29	121	64	57	64	26	16	19	39	1
30105026	TRINIDAD DE LOS LAGOS	6	28	15	13	16	9	2	1	7	1
30105030	LOS LAGUITOS	19	104	54	50	37	19	11	4	33	0
30105031	NUEVO CIRICITO (P)	8	46	22	24	25	14	3	0	21	1
CRISTOBAL											
30106003	GATUN O RIO GATUN (P)	25	96	48	48	55	38	2	3	31	1
30106005	GAMBOA	110	341	170	171	250	147	0	21	118	4
30106015	LOMA FLORES	131	578	314	264	289	121	27	44	206	8
30106016	LA TREINTAICINCO	0	7	3	4	2	1	1	0	1	0
30106025	SAN ANDRES	3	12	5	7	7	4	0	1	5	0
30106026	BARRIADA SAN JOSE (P)	9	58	25	33	36	24	0	3	23	0
30106032	ESCOBAL (P)	17	68	34	34	40	17	1	7	28	2
30106039	LA HUMILDAD (P)	2	15	3	12	3	0	0	1	6	0
30106042	CHORRILLO (P)	23	119	63	56	63	29	8	8	49	1
30106043	CUIPO (P)	1	10	7	3	7	4	4	1	4	0
30106048	PABLON (P)	6	26	14	12	16	6	3	2	12	1
30106050	ARAÑAGATAL (P)	2	8	5	3	5	1	0	0	4	1
30106054	LOS CHORROS DE CIRI GRANDE (P)	2	8	7	1	3	2	0	0	1	0
30106055	QUEBRADA LOPEZ (P)	1	12	6	6	2	1	1	0	5	0
30106058	EL LIMON (P)	35	151	80	71	94	45	1	13	61	3
30106059	GUAYABALITO (P)	2	14	7	7	6	4	1	0	8	0
30106060	SANTA ROSA (P)	12	45	26	19	32	19	2	2	14	0
30106061	CARTAGENITA (P)	2	11	6	5	7	2	0	4	3	0
30106062	ISLA VERDE (P)	2	6	3	3	4	3	0	0	1	0
30106063	EL VALLE DE SANTA CRUZ (P)	6	20	12	8	10	9	2	1	6	0
30106064	TRAPICHITO (P)	2	6	3	3	6	3	3	0	3	2
30106066	SALAMANCA (P)	10	906	892	14	889	15	3	2	841	13
30106068	NUEVO VIGIA (P)	18	78	33	45	39	24	1	2	28	0
30106069	PUEBLO VIEJO (P)	4	9	7	2	6	5	5	0	3	0
30106070	ALTO EL CHORRO (…GATUN (P))	2	4	2	2	4	2	1	0	2	0
30106071	BAJO BONITO	1	1	1	0	1	1	1	0	0	0
30106075	ISLA CRISTOBAL	1	1	0	1	1	0	0	0	1	0
30106082	NUEVO CIRICITO (P)	5	19	12	7	13	6	6	1	6	0
30106085	SALAMANQUITA	8	26	15	11	18	13	7	1	7	0
30106088	VINO TINTO	2	9	4	5	4	2	2	1	3	0
30106089	SAN ANTONIO	11	42	23	19	28	15	0	6	12	1
ESCOBAL											
30107011	LA HUMILDAD (P)	16	71	44	27	35	27	23	0	22	1
30107013	LA ULLAMA	15	56	36	20	34	22	16	1	25	1
30107014	LAS CRUCES	5	14	9	5	10	7	5	0	5	2
30107016	LOS NEGROS	2	14	5	9	6	3	2	0	6	0
30107018	VINO TINTO (P)	21	62	36	26	34	18	16	3	25	1

Table 4–33: Population Breakdown in the Current Canal Watershed, Province of Colon-Census 2000 (Continued)

		Total Homes	Total Persons	Men	Women	Persons Greater than 18 years of	Employed	Engaged in Agriculture	Unemployed	Economically Active	Illiterate
30107022	CAMPO ALEGRE (P)	10	31	17	14	19	11	10	1	13	0
30107025	LA VALEROSA	1	8	4	4	4	2	2	0	4	0
LIMON											
30108001	AGUAS CLARAS	60	244	122	122	155	78	2	21	90	1
30108002	AGUA SUCIA	4	16	14	2	12	10	8	0	4	2
30108003	ALFAGIA	18	73	37	36	44	26	14	2	25	2
30108008	VILLA LIMON (CARRETERA A LIMON)	85	380	188	192	204	98	4	12	167	3
30108009	CARTAGENITA (P)	32	111	56	55	66	33	1	8	41	1
30108010	EL LIMON (P)	140	574	304	270	362	160	4	61	229	4
30108012	QUEBRADA ANCHA	99	443	224	219	250	127	8	52	147	13
30108015	QUEBRADA PATO	1	2	2	0	2	1	1	0	1	0
30108016	QUEBRADA GRANDE	49	240	130	110	142	90	9	9	80	5
30108017	QUEBRADA LARGA	6	31	14	17	13	11	5	0	11	0
30108019	QUEBRADA NORBERTO	7	23	13	10	12	2	1	4	9	0
30108020	QUEBRADA PLATA No.1	2	4	3	1	4	1	1	1	2	0
30108021	GATUN O RIO GATUN (P)	304	1434	751	683	752	428	7	76	542	25
30108022	SALUD	6	11	8	3	9	6	6	0	4	2
30108025	NUEVA ESPERANZA (BARRIADA NUEVA ESPERAN	7	26	16	10	12	6	1	1	13	2
30108028	CHINILLA	1	4	3	1	4	2	2	0	2	0
30108029	BAJO BONITO	7	36	22	14	16	8	6	2	16	1
30108030	QUEBRADA EL JOBO	1	8	4	4	3	2	1	1	2	0
30108031	QUEBRADA ANCHA ARRIBA	22	84	49	35	50	26	7	5	28	0
30108032	CERRO AZUL	13	38	22	16	27	15	12	3	12	1
30108034	EL VALLE DE QUEBRADA ANCHA	56	254	127	127	131	76	2	23	86	2
30108035	QUEBRADA DOMINGO	1	12	3	9	4	1	1	0	6	0
30108036	QUEBRADA PLATA No.2	2	8	5	3	6	4	1	0	3	1
30108037	RIO RITA (P)	9	36	19	17	18	11	0	5	10	0
NUEVA PROVIDENCIA											
30109004	NUEVA PROVIDENCIA (P)	103	457	247	210	241	149	29	13	164	8
30109006	RIO RITA (P)	243	1123	566	557	616	360	11	64	392	13
30109007	NUEVA ITALIA	140	600	313	287	323	194	7	28	190	6
30109008	BARRIADA DE NISPERO	15	59	36	23	30	14	1	10	21	1
30109009	LOS PINOS	101	431	223	208	230	151	1	8	137	3
30109010	NUEVA PROVIDENCIA No.1	1	7	4	3	3	1	0	1	1	0
30109011	QUEBRADA LOPEZ (P)	2	13	9	4	10	6	0	1	4	0
30109012	RIO RITA ARRIBA	80	375	196	179	203	121	1	9	131	4
PUERTO PILON											
30110002	AGUAS CLARA No.2 (…AGUAS CLARAS ARRIBA)	16	82	44	38	35	20	11	9	22	1
30110005	ALTO EL CHORRO (…GATUN (P))	15	68	39	29	31	20	4	3	23	0
30110013	RIO GATUN GRANDE (P)	5	16	10	6	10	6	2	0	6	0
30110015	SANTA RITA ARRIBA (P)	9	22	11	11	18	10	1	0	9	0
30110026	SIERRA LLORONA (P)	25	153	87	66	80	61	48	7	42	1

Table 4–33: Population Breakdown in the Current Canal Watershed, Province of Colon-Census 2000 (Continued)

		Total Homes	Total Persons	Men	Women	Persons Greater than 18 years of	Employed	Engaged in Agriculture	Unemployed	Economically Active	Illiterate
SABANITAS											
30111002	CAMPEON	32	132	64	68	84	48	0	4	56	0
30111006	QUEBRADA LOPEZ (P)	89	374	206	168	218	106	1	32	146	4
30111007	RIO RITA ARRIBA (RIO RITA)	100	426	231	195	206	115	6	30	140	1
30111017	BARRIADA SAN ANDRES	90	397	205	192	219	124	0	28	132	8
30111018	BARRIADA SAN JOSE (P)	135	603	307	296	328	215	4	23	208	7
30111019	EL LAGO (P)	9	19	12	7	13	5	0	6	5	0
SALAMANCA											
30112004	EL VALLE DE SANTA CRUZ (P)	60	248	143	105	132	77	27	14	87	2
30112005	GATUNCILLO ARRIBA	10	33	15	18	25	13	10	0	15	0
30112006	NUEVO PARAISO (P)	106	414	193	221	258	133	19	14	171	9
30112014	SALAMANCA (P)	107	452	235	217	242	131	28	20	184	5
30112015	SARDINILLA (P)	95	397	212	185	236	116	30	17	167	10
30112017	BOQUERON ABAJO (P)	49	205	112	93	126	71	32	14	67	7
30112018	SALAMANQUITA	75	310	168	142	168	68	25	39	123	13
30112019	NUEVO OCU	77	342	186	156	189	100	29	9	149	6
30112020	NUEVO SAN JOSE	65	259	141	118	160	90	26	12	99	7
30112021	COROZAL	6	22	11	11	14	8	3	1	10	0
30112022	BOQUERON ARRIBA	33	130	83	47	88	49	26	11	43	5
30112025	SANTA LIBRADA	7	20	16	4	18	6	5	4	10	1
30112026	SANTO DOMINGO	15	55	31	24	32	20	16	5	18	0
30112027	CABECERA DE GATUNCILLO	1	7	5	2	2	2	1	0	3	0
30112028	EL BAJO	2	2	2	0	2	1	1	0	1	0
30112029	EL PELIGRO	1	4	3	1	4	2	2	1	1	0
30112030	ENTRADA A LA BANDERA	1	5	3	2	3	3	0	0	2	0
30112031	LA BANDERA	2	15	9	6	4	2	2	1	4	0
SAN JUAN											
30113001	BARRIADA JUAN DEMOSTENES AROSEMENA	1238	5392	2838	2554	3032	1725	77	246	1926	75
30113003	CAMPANA	14	50	31	19	30	10	8	5	22	1
30113007	EL VALLE DE LA UNION	188	820	409	411	466	245	17	24	331	11
30113008	GATUNCILLO	877	3695	1910	1785	2204	1199	61	165	1407	56
30113010	MADROÑAL	8	17	13	4	12	6	1	4	7	0
30113011	NUEVO SAN JUAN	264	1138	578	560	641	325	23	73	443	32
30113012	NUEVO VIGIA (P)	452	1998	1040	958	1153	588	23	129	784	32
30113015	ISLA VERDE (P)	14	59	33	26	33	25	9	3	19	1
30113016	ENTRADA NUEVO SAN JUAN	30	93	45	48	67	40	0	1	40	0
30113017	NUEVO PARAISO (P)	1	1	1	0	1	1	0	0	0	0
30113019	PUEBLO GRANDE O QUEBRADA ANCHA (P)	10	58	33	25	28	11	0	0	27	0
30113022	SARDINILLA (P)	1	4	2	2	2	0	0	0	3	0
SANTA ROSA											
30114001	AGUAS CLARAS	20	52	31	21	37	21	17	4	18	0

Table 4–33: Population Breakdown in the Current Canal Watershed, Province of Colon-Census 2000 (Continued)

		Total Homes	Total Persons	Men	Women	Persons Greater than 18 years of	Employed	Engaged in Agriculture	Unemployed	Economically Active	Illiterate
30114003	FRIJOLITO (P)	8	44	23	21	24	12	7	2	16	0
30114004	GUAYABALITO (P)	17	62	31	31	35	17	3	3	31	2
30114006	PALENQUE	121	455	235	220	267	165	30	16	155	12
30114007	SANTA ROSA (P)	34	117	68	49	75	39	8	9	41	1
30114009	LOMA DE MACHO	1	5	3	2	2	1	1	0	1	0
	Total		683								

Table 4-32: Populated Areas of Gatun Lake

		Total Homes	Total Persons	Men	Women	Persons Over the Age of 18	Employed	Engaged in Agriculture	Umemployed	Economically Active	Illiterate
CIRICITO		86	397	210	187	220	109	29	29	143	4
	CUIPO (P)	46	239	127	112	133	70	4	10	90	1
	LA HUMILDAD (P)	11	37	19	18	23	13	9	0	14	2
	NUEVO PORVENIR	29	121	64	57	64	26	16	19	39	1
CRISTOBAL		296	1170	611	559	685	335	34	87	424	17
	GAMBOA	110	341	170	171	250	147	0	21	118	4
	LOMA FLORES	131	578	314	264	289	121	27	44	206	8
	LA TREINTAICINCO	0	7	3	4	2	1	1	0	1	0
	ESCOBAL (P)	17	68	34	34	40	17	1	7	28	2
	LA HUMILDAD (P)	2	15	3	12	3	0	0	1	6	0
	CUIPO (P)	1	10	7	3	7	4	4	1	4	0
	EL LIMON (P)	35	151	80	71	94	45	1	13	61	3
ESCOBAL		31	127	80	47	69	49	39	1	47	2
	LA HUMILDAD (P)	16	71	44	27	35	27	23	0	22	1
	LA ULLAMA	15	56	36	20	34	22	16	1	25	1
LIMON		140	574	304	270	362	160	4	61	229	4
	EL LIMON (P)	140	574	304	270	362	160	4	61	229	4
NUEVA PROVIDENCIA		104	464	251	213	244	150	29	14	165	8
	NUEVA PROVIDENCIA (P)	103	457	247	210	241	149	29	13	164	8
	NUEVA PROVIDENCIA No.1	1	7	4	3	3	1	0	1	1	0
LA TRINIDAD		84	367	202	165	198	138	101	8	117	29
	ESCOBAL O ESCOBALITO	16	80	47	33	41	29	23	1	30	10
	LA ARENOSA O EL LIMITE	23	89	48	41	55	38	22	5	30	6
	LA HUMILDAD	41	176	94	82	92	67	52	0	49	13
	AROSEMENA	4	22	13	9	10	4	4	2	8	0
AMADOR		99	486	267	219	224	117	75	26	174	38
	LAGARTERA	62	348	194	154	139	73	51	13	125	27
	LAGARTERITA O AMADOR	37	138	73	65	85	44	24	13	49	11
AROSEMENA		32	125	67	58	64	30	17	10	52	7
	AROSEMENA	32	125	67	58	64	30	17	10	52	7
ITURRALDA		59	260	151	109	151	91	23	18	93	10
	ARENOSA	56	242	138	104	140	82	16	17	88	9
	LA LEONA	3	18	13	5	11	9	7	1	5	1
	Total	1862	7940	4286	3654	4434	2358	702	508	2888	238

Alteration of Local Customs. The arrival of people with different customs from other regions of the country or other countries will interact with the local communities near construction sites and may alter the pre-established local custom and social relation system. This construction phase impact is considered neutral to slightly negative during the initial construction phase. The inter-relation impacts will be dependent on where the labor necessary to retrofit or demolish affected strcutures originates from and where they are housed if they are non-local. These impacts will decrease as soon as the non-local labor has completed the construction activities.

Nuisances to the Population. The construction activities related to this MOLL will include the transportation and mobilization of equipment, machinery, supplies and personnel to the construction sites. Personnel and equipment will be transported overland or water during the mobilization phase of the MOLL and will create temporary nuisances to the population living close to the transportation routes and to Gatun Lake operations. The route that will be mainly used during the construction phase will be the Transisthmian Highway from Panama City or Colon to the proposed spillway site and ACP affected structures. This route, which already has a high vehicle traffic volume, connects Panama City and Colon and crosses several communities along the highway. The resulting potential impacts from the use of this highway from the additional transportation of equipment, materials and personnel and the potential increase of accidents along this route is considered moderately negative during the initial mobilization and as a slight impact during the other construction phases. Control measures will be implemented to minimize the impacts associated with these activities. Impacts to the populations living around the transportation during the construction phase range from neutral (retrofit of locks) to moderately negative (transportation and mobilization).

Interruption and Deterioration of the Infrastructure. The interruption of the normal traffic patterns during the construction phase of the MOLL will be caused by transporting oversize and large heavy equipment and machinery exceeding the current weight limits of the road bridges. To accommodate these loads, the bridges will be reinforced or modified if it is necessary as a result of the study to be submitted to and approved by Ministry of Public Works. This impact is considered as slightly negative and temporary. Transportation of equipment over water will be conducted using work barges, cranes and others similar construction equipment that is anticipated during the construction of the spillway and retrofitting of affected existing ACP and third party structures.

Traffic utilizing the Panama Canal Railway may also be disrupted during the construction phase of the MOLL. The Panama Canal Railway provides ocean-to ocean transshipment service for freight and passengers on a track that runs parallel to the Canal. The 47.6 mile railway was rehabilitated in 2000 but the track and associated power lines will likely need to be raised in certain areas and lengthening equalization culverts added to accommodate the higher MOLL.

Access Restriction to Fishing Areas and Recreational Activities. The purpose of restricting access to certain portions of Gatun Lake during the retrofit of ACP and third party facilities is to protect the fishermen and individuals passing through the area from hazards related to the construction and operation of a MOLL.

Operation Phase

Operation at the proposed MOLL is not expected to greatly alter the demographic composition of the area, alter local customs or traffic patterns, or interrupt infrastructure. Therefore, the impact of the operation phase on these change indicators is neutral.

Access Restriction to Fishing Areas and Recreational Areas.

Raising the MOLL will result in both positive and negative impacts to the local communities and recreational users of the lake. The extreme ends of Gatun Lake near communities that utilize the lake for fishing and transportation should have a positive impact from the proposed Maximum Operating Level. These areas have numerous trees that extend from the lake bottom to just below the surface of the current lake level. The increase in MOLL should provide an additional buffer from these obstacles to allow small water crafts to maneuver and improve the navigation for both the local and recreational users of the lake. These obstacles historically may have created a barrier for additional development in these remote areas. However, the increased MOLL may prevent passage of boats under the Monte Lirio and Gamboa bridges unless they are raised. The current critical controlling elevations of the Monte Lirio bridge and the Gamboa bridge are 91.2 ft (27.8 m) and 94.0 ft (28.65 m). Raising the MOLL without raising the bridge elevations may prohibit villagers, fishermen, and others from accessing fishing areas, residences, and tourist areas. However, discussions were held with ACP staff and a required clearance was set for the Gamboa bridge of 1.2 m (4 ft). Any lake levels higher than this would require the bridge to be raised. The impacts of the operation phase on this change indicator are designated as neutral to moderately negative.

ACP and Third Party Infrastructure Impacts. Table 4-33 presents a list of facilities and residences observed during the site reconnaissance field visits. The majority of ACP structures and main populated areas along the Gatun Lake shoreline were observed during the site reconnaissance from a boat. Due to time constraints and the nature of this feasibility study and the objective of not to interact with potentially affected third party facilities, exact measurements of the location of all these facilities and their corresponding elevations were estimated. The 1999 US army Corps of Engineers (USACE) reconnaissance study, maps indicating the location of ACP and third party structures using data complied by ACP and Division of Statistics and Census data were

used in evaluating potentially affected facilities. Impacts to ACP and third party infrastructure are expected to be slightly negative.

A facility was considered impacted if the estimated facility elevation was lower than the proposed Maximum Operating Level plus an additional 2 feet of freeboard [28.04m (92 ft)]. Many of the ACP structures and various docks, piers and intake structures shown in Table 4-33 are affected by the proposed higher lake elevation. The estimated cost of relocation settlement payments and land compensation payments to third party entities is $1,135,883. Land compensation payments were estimated as 20% of the estimated value of the structure. Relocation settlement payments were also estimated as 20% of the value of the impacted structure. Although the directly affected households will be acutely impacted by the MOLL, this group of households represent a very small percentage of the overall population residing within the Canal watershed for this reason this impact was evaluated to be slightly negative. These households residing on Canal control property will require a review of ACP policies concerning the control and use of watershed lands.

Table 4-33: Affected Facilities Due to New Maximum Gatun Lake Operating Level

Facility Structure	Structure Type	Gatun Lake Elevation	Description of Structure	Elevation Above Water Level	Critical Elevation of Structure	Potentially Impacted[1]
Pedro Miguel Met. & Hyd. Boat Dock Facility	Boat Facility to Check Stations	84.79	Top of dock	5	89.8	Yes
Paraiso - Launch Landing	Coffer cell with concrete deck and concrete fascia. Slab on grad	84.79	Landing	5.75	90.5	Yes
Paraiso - Launch Landing	Coffer cell with concrete deck and concrete fascia. Slab on grade. South side of dock structure.	84.79	Electric Sub Station & Circuit Board, 20' back of structure	5.75	90.5	Yes
Paraiso - Launch Landing	Coffer cell with concrete deck and concrete fascia. Slab on grade. South side of dock structure.	84.79	Staff building	15.25	100.0	
Paraiso - Tug Landing	Coffer cell with slab on grade and concrete fascia. north side of	84.79	Top of deck	5.45	90.2	Yes
Next to (north of) Paraiso Landing	10 structures/houses ~ 10' above lake level, 15 lighting	84.79		10	94.8	
Paraiso Low lift pumps	Pump station	84.79	Timber Pier around Concrete towe	4.5	89.3	Yes
Paraiso Low lift pumps	Pump station	84.79	Top of SSP	5.75	90.5	Yes
Paraiso Low lift pumps	Pump station	84.79	Deck of intake structure	6.2	91.0	Yes
Gamboa Raw Water Pump Station	Pump station	84.79	Top of stone bulkhead	3.5	88.3	Yes
Gamboa Raw Water Pump Station	Pump station	84.79	Top of access walkway grating	3.5	88.3	Yes
Gamboa Raw Water Pump Station	Pump station	84.79	Bottom EL of pump house	3.5	88.3	Yes
Gamboa Raw Water Pump Station	Pump station	84.79	Top of tower deck	9.2	94.0	
Gamboa Raw Water Pump Station	Pump station	84.79	Pump house	9.2	94.0	
RR/vehicle bridge - Gamboa Bridge	Bridge	84.79	Water pipe	8.25	93.0	
RR/vehicle bridge - Gamboa Bridge	Bridge	84.79	Bottom of girder	9.25	94.0	
RR/vehicle bridge - Gamboa Bridge	Bridge	84.79	Top of RR tie	12	96.8	
RR/vehicle bridge - Gamboa Bridge	Bridge	84.79	Road	14	98.8	
	Light house looking structure with bridge to structure	84.79	Top of deck/bridge	10.5	95.3	
Private boat landing for Gamboa Resort	Concrete Slab on Retained Fill	84.79	Top of concrete on shore	3.5	88.3	Yes
Private boat landing for Gamboa Resort	Concrete Slab on Retained Fill	84.79	Restaurant deck	12	96.8	
Public boat landing	Concrete deck supported on pipe piles. Boat ramp to right. 2 flo	84.79	Top of deck	4	88.8	Yes
Public boat landing	Concrete deck supported on pipe piles. Boat ramp to right. 2 flo	84.79	Boat house	10	94.8	
Boat houses	6 Makeshift Boat Sheds and Wooden Shack	84.79	Deck EL	4	88.8	Yes
Gamboa Resort Hotel	Tennis courts, playground, floating dock, gazeebos, picnic area,	84.79	General park area	6	90.8	Yes
Gamboa Resort Hotel	Tennis courts, playground, floating dock, gazeebos, picnic area,	84.79	Gazeebo	6	90.8	Yes
Gamboa Resort Hotel	Tennis courts, playground, floating dock, gazeebos, picnic area,	84.79	Tennis courts	8	92.8	
Gamboa Resort Hotel	Tennis courts, playground, floating dock, gazeebos, picnic area,	84.79	Outdoor pavilion	8	92.8	
Gamboa Resort Hotel	Tennis courts, playground, floating dock, gazeebos, picnic area,	84.79	Pond for storm water			Yes
Hotel Recreation Area		84.79	Warehouse and shop area, 3 build	3	87.8	Yes
Hotel Recreation Area		84.79	Large closed building, 60'x40' she	5	89.8	Yes
Houses and shelter, Embera		84.79	4 houses	10	94.8	
Houses and shelter, Embera		84.79	1 house	20	104.8	
Houses and shelter, Embera		84.79	8 houses	20	104.8	
Santa Rosa houses	Average house, 30'x30', stucco, 1-story.	84.79	5 houses	8	92.8	
Santa Rosa houses	Average house, 30'x30', stucco, 1-story.	84.79	3-4 houses	10	94.8	
Santa Rosa houses	Average house, 30'x30', stucco, 1-story.	84.79	6-8 houses	15	99.8	
Siri Landing, Gamboa	Timber wharf structure, supported on timber piles.	84.79	Small John boat landing	3	87.8	Yes
Siri Landing, Gamboa	Timber wharf structure, supported on timber piles.	84.79	Top of deck	5.25	90.0	Yes
Houses north of Santa Rosa		84.79	3 houses	10	94.8	
Houses north of Santa Rosa		84.79	1 house	15	99.8	
Water intake		84.79	Wall for pump	15	99.8	
Transisthmian Highway Bridge		84.79		60	144.8	
Tug and Launch Landing, Gamboa	Concrete deck supported on pipe piles 14" dia.	84.79	Top of Launch Landing	4.2	89.0	Yes
Tug and Launch Landing, Gamboa	Concrete deck supported on pipe piles 14" dia.	84.79	Top of Tug Landing	6.2	91.0	Yes
Tug and Launch Landing, Gamboa	Concrete deck supported on pipe piles 14" dia.	84.79	Aphalt pavement	8.2	93.0	

Table 4–35: Affected Facilities Due to New Maximum Gatun Lake Operating Level (Continued)

Facility Structure	Structure Type	Gatun Lake Elevation	Description of Structure	Elevation Above Water Level	Critical Elevation of Structure	Potentially Impacted[1]
Tug and Launch Landing, Gamboa	Concrete deck supported on pipe piles 14" dia.	84.79	Ambulance clinic (2 Story)	8.2	93.0	
Tug and Launch Landing, Gamboa	Concrete deck supported on pipe piles 14" dia.	84.79	Substation	8.2	93.0	
Tug and Launch Landing, Gamboa	Concrete deck supported on pipe piles 14" dia.	84.79	Open grate gangway			Yes
Met & Hyd Station		84.79		15	99.8	
Madden Dam		84.79	Wall for hydropower walkway	10	94.8	
Madden Dam		84.79	Rock wall	10	94.8	
Gamboa Launch Landing	Boat slips	84.79	Top of deck	3.5	88.3	Yes
Inlet area around dredging area, Gamboa		84.79	To top of wall and deck	8.2	93.0	
Inlet area around dredging area, Gamboa		84.79	Concrete Dredging Wharf	8.2	93.0	
Inlet area around dredging area, Gamboa		84.79	SSP wall	8.2	93.0	
Area inside boat slips and SSP wall, Gamboa		84.79	SSP wall	6.5	91.3	Yes
Hydrographic Survey Pier	Timber pier	84.79	Top of deck	3.5	88.3	Yes
Fuel Barge Pier, Gamboa		84.79	Front deck elev at bollards	5	89.8	Yes
Fuel Barge Pier, Gamboa		84.79	Back top of deck elev.	6	90.8	Yes
Marginal Fueling Wharf	Wharf structure	84.79	Top of deck	6	90.8	Yes
	Concrete pier on concrete piles	84.79	Top of deck	3.5	88.3	Yes
Gamboa Smithsonian Pier	Fixed concrete pier and floating dock	84.79	Top of grade at picnic area	6	90.8	Yes
Gamboa Smithsonian Pier	Fixed concrete pier and floating dock	84.79	Concrete pier	8.2	93.0	
Gamboa Smithsonian Pier	Fixed concrete pier and floating dock	84.79	Floating dock			Yes
Lighting Facilities ~ 15		84.79				Yes
Nav. Aids		84.85				Yes
Smithsonian Institute	Misc. piers and floating docks	84.85	Misc. concrete dock	3	87.9	Yes
Smithsonian Institute	Misc. piers and floating docks	84.85	Covered boat house	4	88.9	Yes
Smithsonian Institute	Misc. piers and floating docks	84.85	House	4.5	89.4	Yes
Smithsonian Institute	Misc. piers and floating docks	84.85	Lowest pile at floating dock	6.5	91.4	Yes
Smithsonian Institute	Misc. piers and floating docks	84.85	Top of main pier deck	8.5	93.4	
Dock 45, Gatun	Concrete Slab on Retained Fill	84.85	Concrete slab	4.5	89.4	Yes
Dock 45, Gatun	Anchored SSP bulkhead with floating pier and covered dock.	84.85	SSP bulkhead, no cap with concr	4.5	89.4	Yes
Dock 45, Gatun	Warehouse Structure	84.85	Domed warehouse	7.5	92.4	
Mitre Gate Repair Pier	Concrete deck with piles	84.85	Pier	4.5	89.4	Yes
Mitre Gate Repair Pier	Open Concrete Building	84.85	Large covered structure with RR s	5	89.9	Yes
Mitre Gate Repair Pier	Timber Pier	84.85	Timber deck, piles and bracing	5	89.9	Yes
Mitre Gate Repair Pier	Asphalt Pavement	84.85	Asphalt road	6	90.9	Yes
Mitre Gate Repair Pier	Warehouse Structure	84.85	4 warehouses about 15' from cove	6	90.9	Yes
Mitre Gate Repair Pier	Warehouse Structure	84.85	2 warehouses	8	92.9	
Mitre Gate Repair Pier	Warehouse Structure	84.85	2 warehouses about 15' from cove	8	92.9	
Diving Facility, Gatun	T-head pier	84.85	Top of deck	7.5	92.4	
Diving Facility, Gatun	Mooring/Breasting Posts/Piles	84.85	3 Dolphins	7.5	92.4	
Diving Facility, Gatun	Building	84.85	1 story building	12	96.9	
Diving Facility, Gatun	Building	84.85	2 story building	12.5	97.4	
Tourist pier	Floating docks, fixed walkways, tourist area	84.85	Fixed walkway	9	93.9	
Tourist pier	Floating docks, fixed walkways, tourist area	84.85	Misc. structures	10	94.9	
Tourist pier	Floating docks, fixed walkways, tourist area	84.85	Floating pier			Yes
Launch Landing	Boat slips with concrete deck and piles	84.85	Elec. Substation	5	89.9	Yes
Launch Landing	Boat slips with concrete deck and piles	84.85	Building (Shop)	5	89.9	Yes
Launch Landing	Boat slips with concrete deck and piles	84.85	Office Building	5	89.9	Yes
Launch Landing	Boat slips with concrete deck and piles	84.85	Concrete pile supported platform	5	89.9	Yes
Launch Landing	Boat slips with concrete deck and piles	84.85	Launch landing/conc. Pier - covere	5	89.9	Yes

Table 4–35: Affected Facilities Due to New Maximum Gatun Lake Operating Level (Continued)

Facility Structure	Structure Type	Gatun Lake Elevation	Description of Structure	Elevation Above Water Level	Critical Elevation of Structure	Potentially Impacted[1]
Launch Landing	Boat slips with concrete deck and piles	84.85	Fuel tank	11	95.9	
Tugs/Water bus, Gatun	Pile supported pier with concrete deck, covered facility	84.85	Concrete pier	4.5	89.4	Yes
Tugs/Water bus, Gatun	Pile supported pier with concrete deck, covered facility	84.85	Two Story Building	7	91.9	Yes
Equipment yard, Gatun		84.85				Yes
Fuel Pier	5 bitts, 2 life rings, 4 light poles	84.85	Asphalt pavement	3.5	88.4	Yes
Fuel Pier	5 bitts, 2 life rings, 4 light poles	84.85	Fuel pier	4.5	89.4	Yes
Fuel Pier	5 bitts, 2 life rings, 4 light poles	84.85	Walkway	7	91.9	Yes
Loading pier	Concrete deck on piles	84.85	Pier	4	88.9	Yes
Loading pier	Concrete deck on piles	84.85	2 Mooring posts	7	91.9	Yes
Roadway to Fort Sherman	Bridge	84.85	Roadway near Fort Sherman	2	86.9	Yes
Bridge over French Canal	Bridge	84.85	Water pipe	11.5	96.4	
Bridge over French Canal	Bridge	84.85	Top of roadway	14	98.9	
Down stream of dam	Near proposed spillway	84.85				Yes
Bridge over existing spillway	Concrete bridge	84.85	Top of Bridge to Spillway Floor	17.5	102.4	
Gatun locks		84.85	Electric motors			Yes
Gatun locks		84.85	Chain fender bulkhead			Yes
Gatun locks		84.85	Transformer room			Yes
Gatun locks		84.85	Bottom of valve chamber			Yes
Monte Lirio Bridge	Steel gider RR bridge with lift station, 3 spans ~200' long	84.87	Top of concrete	5.25	90.1	Yes
Monte Lirio Bridge	Steel gider RR bridge with lift station, 3 spans ~200' long	84.87	Bottom of girder	6.3	91.2	Yes
Monte Lirio Bridge	Steel gider RR bridge with lift station, 3 spans ~200' long	84.87	Two pipes, 9' deep girder	6.3	91.2	Yes
Monte Lirio Bridge	Steel gider RR bridge with lift station, 3 spans ~200' long	84.87	RR on levy	12.13	97.0	
Monte Lirio Bridge	Steel gider RR bridge with lift station, 3 spans ~200' long	84.87	Base of Rail	13.45	98.3	
Town of El Limon	Village area	84.87	Covered boat house	4	88.9	Yes
Town of El Limon	Village area	84.87	2 timber piers	4	88.9	Yes
Town of El Limon	Village area	84.87	Concrete pier	5	89.9	Yes
Town of El Limon	Village area	84.87	Larger house	6	90.9	Yes
Town of El Limon	Village area	84.87	1 house	7	91.9	Yes
Town of El Limon	Village area	84.87	Most houses	10	94.9	
Town of El Limon	Village area	84.87	Water intake	10	94.9	
La Provedencia		84.87	Concrete bulkhead	4	88.9	Yes
La Provedencia		84.87	Thatched roof hut	7	91.9	Yes
La Provedencia		84.87	Bottom of new house	10	94.9	
La Provedencia	Village area	84.87	Most houses	10	94.9	
La Represa Water Intake	Concrete foundation with concrete house on top	84.87	Bottom of slab, 6'-8" slab	17.5	102.4	
Water intake for Melia Resort		84.87	Top of floor with intake	3	87.9	Yes
Water intake for Melia Resort		84.87	Timber pier	3	87.9	Yes
Melia Resort	Floating docks (2) and pier	84.87	Top of timber dock	4	88.9	Yes
Melia Resort	Floating docks (2) and pier	84.87	Top of boat house	10	94.9	
Large 2 story house and out-building		84.87	Out-building/cabin	5	89.9	Yes
Large 2 story house and out-building		84.87	2 story house on stilts	10	94.9	
Mt. Hope intake		84.87	Concrete intake	4.625	89.5	Yes
Mt. Hope intake		84.87	Unknown conc. Structure	5.5	90.4	Yes
Escobal	Houses and private piers	84.87	1 concrete pier, poor cond.	3	87.9	Yes
Escobal	Houses and private piers	84.87	1 timber pier, fair cond.	3	87.9	Yes
Escobal	Houses and private piers	84.87	1 hut, poor cond.	5	89.9	Yes
Escobal	Houses and private piers	84.87	2 CMU houses, fair cond.	20	104.9	
Escobal	Village area	84.87	Most houses	20	104.9	

Table 4–35: Affected Facilities Due to New Maximum Gatun Lake Operating Level (Continued)

Facility Structure	Structure Type	Gatun Lake Elevation	Description of Structure	Elevation Above Water Level	Critical Elevation of Structure	Potentially Impacted[1]
Saddle Dam		84.87				
Lower Met & Hyd station	2 concrete structures with floating piers	84.87	Floor of concrete structures	6	90.9	Yes
Lower Met & Hyd station	2 concrete structures with floating piers	84.87	Building structure	25	109.9	
Cuipo - Misc. structures/houses		84.87	Covered concrete pier, fair cond.	4	88.9	Yes
Cuipo - Misc. structures/houses		84.87	4 huts	4	88.9	Yes
Cuipo - Misc. structures/houses		84.87	Timber pier	4	88.9	Yes
Cuipo - Misc. structures/houses		84.87	5 large houses	6	90.9	Yes
Cuipo - Misc. structures/houses		84.87	Bath house	6	90.9	Yes
Cuipo - Misc. structures/houses		84.87	Water intake structure, CMU	8	92.9	
Cuipo - Misc. structures/houses		84.87	Shed	10	94.9	
Cuipo - Misc. structures/houses		84.87	2 houses behind	10	94.9	
Cuipo - Misc. structures/houses		84.87	Timber pier	10	94.9	
Cuipo - Misc. structures/houses		84.87	Misc. buildings	20	104.9	
Cuipo - Misc. structures/houses		84.87	Main water building	25	109.9	
La Laguana Village and Park area	Village area	84.87	Covered boat house and pier	3	87.9	Yes
La Laguana Village and Park area	Village area	84.87	Boat storage	4	88.9	Yes
La Laguana Village and Park area	Village area	84.87	Private intake timber pier (4'x20') a	4	88.9	Yes
La Laguana Village and Park area	Village area	84.87	Covered boat house	5	89.9	Yes
La Laguana Village and Park area	Village area	84.87	Thatched roof hut	5	89.9	Yes
La Laguana Village and Park area	Village area	84.87	1 small house	5	89.9	Yes
La Laguana Village and Park area	Village area	84.87	Misc. huts ~4	7	91.9	Yes
La Laguana Village and Park area	Village area	84.87	Main building	8	92.9	
La Laguana Village and Park area	Village area	84.87	2 story house	15	99.9	
Laguna Alta Water Intake	Concrete pipe supported structure on concrete deck	84.87	Out-building	6	90.9	Yes
Laguna Alta Water Intake	Concrete pipe supported structure on concrete deck	84.87	Low steel	8.5	93.4	
Laguna Alta Water Intake	Concrete pipe supported structure on concrete deck	84.87	Top of intake floor	10	94.9	
Laguna Alta Water Intake	Concrete pipe supported structure on concrete deck	84.87	Centerline discharge pipe	11.5	96.4	
Laguna Alta Water Intake	Concrete pipe supported structure on concrete deck	84.87	Pump house building	15	99.9	
Las Cruces Landing	Floating piers and bulkhead	84.87	Floating piers	3.5	88.4	Yes
Las Cruces Landing	Floating piers and bulkhead	84.87	Existing grade/conc. Slab	4	88.9	Yes
Las Cruces Landing	Floating piers and bulkhead	84.87	Floating dock guide pile system	5	89.9	Yes
Las Cruces Landing	Floating piers and bulkhead	84.87	Guide pile system	6.25	91.1	Yes
Las Cruces Landing	Floating piers and bulkhead	84.87	Large Boat Shelter	7	91.9	Yes
Las Cruces Landing	Floating piers and bulkhead	84.87	SSP wall	7	91.9	Yes
Las Cruces Landing	Floating piers and bulkhead	84.87	Fuel tank slab	10	94.9	
Las Cruces Landing	Floating piers and bulkhead	84.87	Building	15	99.9	
Las Cruces Landing	Floating piers and bulkhead	84.87	Shed	15	99.9	
Mira Flores Locks	Lock structures, Bandeck-Boot					Yes
La Arenosa	Village area	84.87	CMU Building	2	86.9	Yes
La Arenosa	Village area	84.87	2 Houses	4	88.9	Yes
La Arenosa	Village area	84.87	CMU House	5	89.9	Yes
La Arenosa	Village area	84.87	2 Houses	6	90.9	Yes
La Arenosa	Village area	84.87	Small School	8	92.9	
La Arenosa	Village area	84.87	5+ Houses	12	96.9	
La Garterra Grande	Small Village on an island	84.87	Multiple Tin Sheds	10	94.9	
La Garterra	Intake Structure	84.87	Local Water Intake	15	99.9	
La Garterra	Village area	84.87	Multiple Houses	12	96.9	
Note: For the purposes of evaluating potentially impacted structures an elevation of 90 feet was used.						
This elevation represents a proposed Maxiumum Opearting Lake Level of 90 plus 2 feet of freeboard.						

Table 4-34: Residences Affected Due to the Proposed Changes in MOLL

Community	Potentially Impacted Residences	Total Number of Homes
Gamboa Resort	No residences affected	110
Santa Rosa	No residences affected	31
Embera	No residences affected	5
El Limon	2	140
Nueva Providencia	1	103
Espinar near Melia Resort	1	Not Available
Escobal	0	17
Cuipo	10	46
La Laguna	6	15
La Arenosa	6	56
Lagaterita o Amador	No residences affected	37
La Leona	No residences affected. Two private docks affected	Not available
Coca Cola	Private dock	Not available
Campo Alegre	6	Not available
Lagartera	2	Not available

Economic

Construction Phase

The proposed increase in the MOLL will result in a capital investment of US$25 million. The resulting MOLL would permit an increase in water storage during both the Wet and Dry Seasons. This increase in storage would improve the reliability of the Panama Canal to serve its customers without draft restrictions during the dry periods. Historically, investment in Canal related projects has had a multiplicative positive impact on the Panamanian economy such as an increase in tax revenues and fees, an increase in demand for goods and services, and job creation. However, there may be a temporary reduction in income generated from non-shipping sources, such as fishing and tourism, due to construction related restrictions, etc. For all these reasons, the impacts associated with the construction phase of the MOLL are considered slightly negative to highly beneficial.

Increase in Tax Revenue and Fees from Increase in Reliability of the Canal.. The impact from an increase in income tax revenues and royalties benefits will be derived from the development of the MOLL will be highly positive and will impact the regional and national levels through the multiplicative direct and indirect impacts of the investment in the form of taxes generated from salaries, tax revenues to the government, taxes paid by contractors, the revenues from the sale of fuel and other products to the ships, and the induced impacts from tourism, the duty free market, and ports.

Alteration of the Income Derived from Gatun Lake. Income derived from Gatun Lake from non-shipping sources, such as tourism and fishing, may be temporarily reduced due to construction disturbance and access restrictions. Impacts to tourism and academic generated revenue may be most noticeable at the Gamboa Rainforest Resort and the Smithsonian Tropical Research Institute (STRI). Much of the recreational area around the Gamboa Resort is around elevation 91 ft (27.7 m) and may be impacted by wave action under the new MOLL. Several docks and landings on Barro Colorado Island and Península Gigante associated with the Smithsonian Tropical Research Institute would need to be replaced or retrofitted to accommodate the new MOLL. The construction activity associated with protection, replacement, or retrofit of these structures may cause a decline in the number of people visiting the Gamboa Resort and STRI during the construction phase. These impacts are expected to be temporary and minor and thus are designated as moderately negative.

Operation Phase. The economic performance of the Panama Canal is very much aligned with the global economy in particular the economy of the United States, the main user of the Canal, Japan and Europe. As reported in the 2002 annual report, the most recent annual report available on the internet, the uncertainty in the world economy resulted in a 2.8% downturn in cargo tonnage as compared with the previous year. The report also indicated a drop in the transits of 2.3% and an average vessel increase of 3.9%. The actual transit by larger vessels resulted in a positive impact on toll revenue to the level of B/.588.8 million. The outlook for economic growth as reported in the annual report is projected to increase by 2.9% trend as the world economies recover.

Increase in Toll Revenue and Fees from Increase in Reliability of the Canal. The operation of the MOLL will provide an increase in toll revenues associated with the incremental increase in the reliability of the Canal to operate without draft restrictions. The proposed MOLL will result in an increase in tolls collected and potentially additional lockages for additional transits and tonnage across the Canal as well as an incremental increase in demand for goods and services and job creation associated with this MOLL. For all these reasons, the impact associated with the operation phase of the MOLL is considered highly beneficial.

Increase of the Demand of Goods and Services. At a national level, benefits will be derived from the MOLL during the operational phase. The MOLL will cause a general incremental increase in business activity which will affect local, regional and national economies in a positive manner. The operation of the spillway and the incremental reliability associate with Operating Gatun Lake at the Maximum Operating Level will generate and increase in the demand for goods and services directly and indirectly associated with the MOLL. This increase in demand will enhance the local economy by the production of goods and supplies of construction materials, local support services (hotels, restaurants, transportation, stores and warehouses) and an increase in trade among the districts. The MOLL will require a high standard of goods and therefore this demand may result in an increase in the quality and the supply of some goods and services offered locally. For all of the above reasons mentioned above, the effect on goods and services during the operation phase is considered as highly positive.

Cultural

Alteration or Destruction of the Archaeological Heritage. No archaeological impacts are anticipated from operation of the Gatun Lake at the Maximum Operating Level. The review of existing information shows that the possibilities of finding archaeological resources in the areas that will be flooded or where where earthwork will be performed are very low and thus. However, it is recommended that a Phase 1 Archaeological Resources Survey be conducted in all new construction areas prior to the start of earthmoving activities to identify any previously unknown archaeological sites. The protocol for handling previously unidentified resources that are unearthed during earthmoving activities is discussed in the Mitigation and Monitoring Report.

4.5.3 Conclusions

This chapter describes the potential environmental and social impacts associated with raising the Gatun Lake Maximum Operating Level (MOLL) during the construction and operation phases. Due to the nature of this study, the data available and the time/schedule constraints of a feasibility study, do not allow for a complete quantitative

analysis of the impacts but rather a partial quantitative and qualitative analysis of the impacts.

As discussed in Chapter 3, the environmental assessment to identify the potential impacts has been performed through a review of existing environmental reports and data, site visits, and an interactive process between ACP, and the engineering and environmental consulting team. The interaction between ACP and consultants allowed the evaluation of design options and the planning of the MOLL during this feasibility phase in order to minimize the adverse impacts and to maximize the beneficial impacts during the construction and operation phases.

Based on the information collected during the preparation of an existing conditions report for the various physical, biological, socio-economic and cultural components represented in the Direct Impact Area (DIA) of the MOLL, change indicators (events that show the occurrence of an impact) have been identified based on the susceptibility of the component to exogenous agents. This analysis is summarized in Table 3-1. Using the environmental change indicators and the anticipated MOLL activities, the evaluation of the impacts was performed using a Leopold modified matrix. Twenty-three change indicators were evaluated for both the construction and operational phases of the MOLL for various physical, biological, and social components that would be potentially impacted by MOLL activities. Impacts to these components were ranked as either positive, neutral, slightly negative, moderately, negative, or highly negative.

Most impacts associated with the Construction phase were classified as either slightly or moderately negative. Several ACP structures were identified that would require retrofitting or reconstruction to accommodate operation at the new maximum operating level. In addition, several third party structures (mainly residences) were identified at elevations below the proposed maximum operating level and required freeboard (28.04 m or 92 ft). The estimated cost of retrofitting or rebuilding these structures is B/$25,727,949.67. The estimated cost of relocation settlement payments to third parties is B/$1,135,894. The economic impact of the construction phase on tax revenue generation, demand for goods and services, and job creation was classified as highly beneficial.

Most of the impacts associated with the Operation phase were designated as neutral or slightly negative. Moderately negative impacts to water resources and biological resources are associated with operating Gatun at the proposed maximum operating level of 27.43 m (90 ft) due to predicted increases in turbidity, loss of terrestrial vegetation cover, and reduction in the number of threatened or endangered fauna in the area. Highly positive economic impacts are anticipated from the Operation phase from an increase in tolls and taxes generated from the Canal, a greater demand for goods and services, and job creation.

The MOLL will result is some negative impacts to the physical and biological environment. Efforts will be made to avoid, minimize, or mitigate these impacts by implementing Best Management Practices (BMPs) for construction and operation activities and adhering to accepted international standards. Anticipated negative impacts to the social environment, such as restricted access to certain areas and increased traffic flow will be addressed by following the procedures outlined in the Social and Environmental Management Plan. Social impacts relating to the condemnation or retrofit of third party infrastructure located below 28.04 m (92 ft) will be mitigated by compensation for structure and land costs, as well a relocation compensation payment for affected residents. All proposed social and environmental mitigation measures are presented in Section 4.6 of this report.

4.6 Environmental Monitoring & Mitigation

4.6.1 Introduction

An Environmental and Social management plan (Plan) is required for the raising of the Gatun Lake Maximum Operating Level and the construction of a new spillway (Project) during the construction and operation phases. This Plan should incorporate the proposed measures for mitigation, control, prevention, protection and compensation associated with the environmental and social impacts identified for the projects. The mitigation measures proposed include management, monitoring, health and safety programs, and closure protocol.

The programs that comprise the Plan have been presented as data sheets for the various project activities to facilitate communication of the proposed mitigation measures and their implementation. The format of the data sheets was designed to allow this report to be used as a guide for complying with the necessary mitigation for all social and environmental impacts identified for the Project. The data sheets are structured in a sequential manner and address the potential social and environmental impacts reported under the Interim Report of Environmental and Social Impacts that will apply to various activities undertaken during the Project. Separate plans are provided for the construction and operation phases.

The Plan described in detail in Appendix F to this report has been designed to meet the overall intent of ACP policies. The environmental and social management plan proposed for the Project provides a mechanism to verify compliance with all ACP environmental policies.

In general terms the Plan prepared for the Project incorporates mitigation measures to address the impacts that were identified during the Interim Report of Environmental and Social Impacts. As the Project progresses to a more formal Environmental Impact Assessment (EIA), including a formal public consultation and disclosure, and additional

impacts are identified, additional mitigation measures may be identified and incorporated into this Plan.

The Plan assumes that all construction work related to this Project will be conducted by an independent Contractor utilizing subcontractors as needed and using standard construction practices. The impact assessment of the Project is based on the assumption that these construction practices will be fully implemented by the Contractor. In many cases these measures represent Best Management Practices (BMPs) that are widely recognized internationally and applied to major projects.

An estimated 40.76 acres of wetland may be directly impacted by the construction of the new spillway. The estimated cost of mitigation for wetlands impacted at the spillway site is B/$40,760.00.[30] This figure was calculated using information provided by ACP regarding the type and cost of mitigation required.

4.6.2 Social and Environmental Management Plan

The proposed program specifies environmental management for each of the construction and operation activities of the Project. During construction, this program involves the responsibilities of the contractor in the management of the environment while incorporating all applicable environmental policies established by ACP. Table 4-34 shows the different programs established under the Environmental Management Plan. Table 4-36 shows the relationship of the Environmental Management Plan to the potential impacts identified in Interim Report on Environmental and Social Impacts, which required the implementation of measures for prevention, mitigation protection and control.

[30]This estimate was generated using a mitigation value of $10,000/wetland hec. For estimation purposes, we assumed the entire footprint was in wetlands.

Table 4-35: Environmental Management Programs, Construction and Operation Phase of the Project

Management Program	Data Sheets
CONSTRUCTION	
Social Management	GS-1 Information and Communication to the Community
	GS-2 Protection of the Social and Economic Infrastructure
	GS-3 Support in the Hiring of Local Labor
	GS-4 Support in the Use of Local Services and Resources
	GS-5 Access Restriction
Education and Training	EC-1 Environmental Education and Training to Contractors
	AC-1 Transportation and Mobilization
	AC-2 Sign Posting
	AC-3 Site Preparation
	AC-4 Concrete and Masonry Works
	AC-5 Construction of Spillway, ACP Facilities, and Third Part Infrastructure
	AC-6 Lake/River Construction Activities
	AC-7 Clean Up
Waste Management	MR-1 Liquid Waste Management
	MR-2 Solid Waste Management
Monitoring	SM-1 Compliance Monitoring of Social Management
	SM-2 Monitoring of the Water Quality
	SM-3 Monitoring of the Air Quality
	SM-4 Monitoring of the Noise Levels
	SM-5 Monitoring of the Aquatic Sediment
	SM-6 Monitoring of the Aquatic Ecosystem
	SM-7 Monitoring of the Solid Waste Management
Health and Safety	Health and Safety Guidelines

Contingency Plan	Contingency Plan Guidelines
OPERATION	
Social Management	GO-1 Relations with the Community
	GO-2 Restriction to Fishing and Recreational Activities
Education and Training	EO-1 Education and Training of Operating Personnel
Management of the Operation Activities	AO-1 Spillway Operation
	AO-2 Maintenance Activities
Waste Management	RO-1 Liquid Waste Management
	RO-2 Solid Waste Management
	SO-1 Monitoring of the Water Quality
	SO-2 Monitoring of the Aquatic Sediment
	SO-3 Monitoring of the Aquatic Ecosystem
	SO-4 Monitoring of Shoreline
Contingency Plan	Contingency Plan Guidelines
ABANDONMENT	
Abandonment Plan Guidelines	

Table 4-36: Environmental Management Plan for the Project

Environmental Component	Impacts	Identification of the Proposed Program
CONSTRUCTION STAGE		
Air	Alteration in the air quality	AC – SM – GS – EC
Noise	Increase in the noise levels	AC – SM – GS
Water	Increase in the lake turbidity	AC- SM
	Alteration of the physical-chemical quantity and quality of Gatun Lake and Chagres River	AC – SM – EC
Soil	Alteration of the soil structure / Landforms / aquatic morphology	AC – SM – MR
	Alteration of the physical-chemical quality	EC – SM – MR
Terrestrial and Aquatic Flora and Fauna	Loss of terrestrial vegetation coverage	AC
	Alteration of the structure and composition of aquatic communities	AC – SM
	Changes in fish catch	AC – SM
	Reduction in threatened or endangered terrestrial and aquatic fauna	SM
Social	Alteration in the demographic composition	GS – EC
	Alteration of local customs	GS – EC
	Nuisances to the population	GS – AC – EC
	Interruption of infrastructure	GS – AC – EC
	Restrictive access to fishing and recreation activities	GS
Economic	Increase in the reliability of the Canal/Revenues/Goods and Services/Job creation	GS
Cultural	Alteration or destruction of the archaeological resources	AC
OPERATION STAGE		
Air	Alteration of the air quality	AO – SO
Noise	Increase in the noise levels	SO
Water	Increase in Gatun Lake and Chagres River turbidity	AO – SO

	Alteration of the physical-chemical quality of Gatun Lake and Chagres River	SO – EO
Soil	Alteration of the physical-chemical quality/ aquatic morphology	SO – RO
Terrestrial and Aquatic Flora and Fauna	Alteration of the structure and composition of the aquatic communities	SO – EO
	Changes in the fish catch	SO
	Reduction in threatened or endangered terrestrial and aquatic fauna	SO
Social	Restrictive access to fishing and recreation activities	SO – EO
Economic	Increase in the reliability of the Canal/Revenues/Goods and Services/Job creation	GO

CONSTRUCTION MANAGEMENT PROGRAMS AND PLANS	OPERATION MANAGEMENT PROGRAMS AND PLANS
1. Social Management (GS)	Social Management (GO)
2. Education and Training (EC)	Education and Training (EO)
3. Management of the Construction Activities (AC)	Management of the Operation Activities (AO)
4. Waste Management (MR)	Waste Management (RO)
5. Monitoring (SM)	Monitoring (SO)

4.6.3 Monitoring Program

The monitoring of the Project is complimentary to the environmental impact assessment and planning process conducted during the feasibility Environmental and Social Impact study. The Project monitoring program uses instruments such as inspection, monitoring, and auditing to determine if the environmental impacts identified during the feasibility environmental assessment actually occur and if the measures designed to manage them are working effectively. Monitoring is the procedure that is utilized to establish the relationship between what is forecast and planned during the Environmental Impact Assessment and what actually occurs and is implemented during the construction and operation of the Project.

4.7 Capital Cost Analyses

4.7.1 Basis for Costs

Due to the magnitude and complexity of determining an opinion of probable cost for retrofitting or replacing each individual structure located throughout Gatun Lake, it was

necessary to develop estimate square footage costs for various classifications of structures (some structural modifications were unique and discussions of these items can be found within their individual sections within later sections). In addition, each structure was categorized into one of the following structure types: Water Intake Structure, Bridge, Bulkhead/Marginal Wharf, Timber Pier, Concrete Pier, Floating Pier, Pavement/Grade, Building, House, and Miscellaneous Structure. For each structure a unit cost was assigned for a major retrofit or replacement. The unit cost for each structure was developed using historical averages based on structures of the various types. These costs were supplemented by utilizing the RSMeans *Building Construction Cost Data Manual*, 2004. This manual provides accurate cost data compiled by thousands of contractors and suppliers. Table 4-37 below provides a range of unit costs for each type of structure.

Table 4-37: Estimated Unit costs for Major Retrofit or Replacement of Structures on Gatun Lake (2004)

Structure Type	Approximate Unit Cost
Water Intake Structure	$100 to $120 per square foot
Bridge	$80 to $120 per square foot
Bulkhead/Marginal Wharf	$1000 to $2000 per linear foot
Timber Pier	$40 to $70 per square foot
Concrete Pier	$60 to $120 per square foot
Floating Pier	Varies
Pavement/Grade	$10 to $20 per square foot
Building	$35 to $70 per square foot
House	$20 to $50 per square foot
Miscellaneous Structure	Varies

Major retrofit or replacement varies from work required to raise the deck of pier structures, increasing the height of an existing bulkhead, increasing the height of the guide pile for a floating pier, providing a dike type structure around a facility on grade, or possibly jacking up an existing building or house, all of which have differing costs. Replacement alternatives involve demolishing the existing structure and constructing a new facility in its place. The operational needs of each facility will dictate the feasibility and need for certain retrofits or replacements. For example, certain piers and wharves have mooring hardware, berthing hardware, access ladders, gangways, and guide piles

that are sensitive to changes in the water level. Therefore, an in-depth condition assessment and analysis will be required for each structure to determine if there is adequate structural capacity to warrant a major retrofit option or if the structure should be completely replaced. It is also necessary to factor in the cost for demolition, which generally equals approximately 25% of the construction cost.

With regards to the minor retrofit it is necessary to evaluate each structure on an individual basis. The intent is to provide some means of waterproofing to prevent excessive infiltration of water around the facility. This may be difficult for timber piers, since water can penetrate from four sides and from below. The operational needs of each facility will determine if a minor repair is a viable solution or whether another approach is required. With this in mind, the cost for the minor retrofit only provides a rough order of magnitude cost and cannot be broken down to an individual structure type. Many structures can be protected from periodic splashing and waves by means of a concrete curb or some type of framed structure. It was estimated that this would cost about $75 to $200 per linear foot, depending on the complexity of the structure.

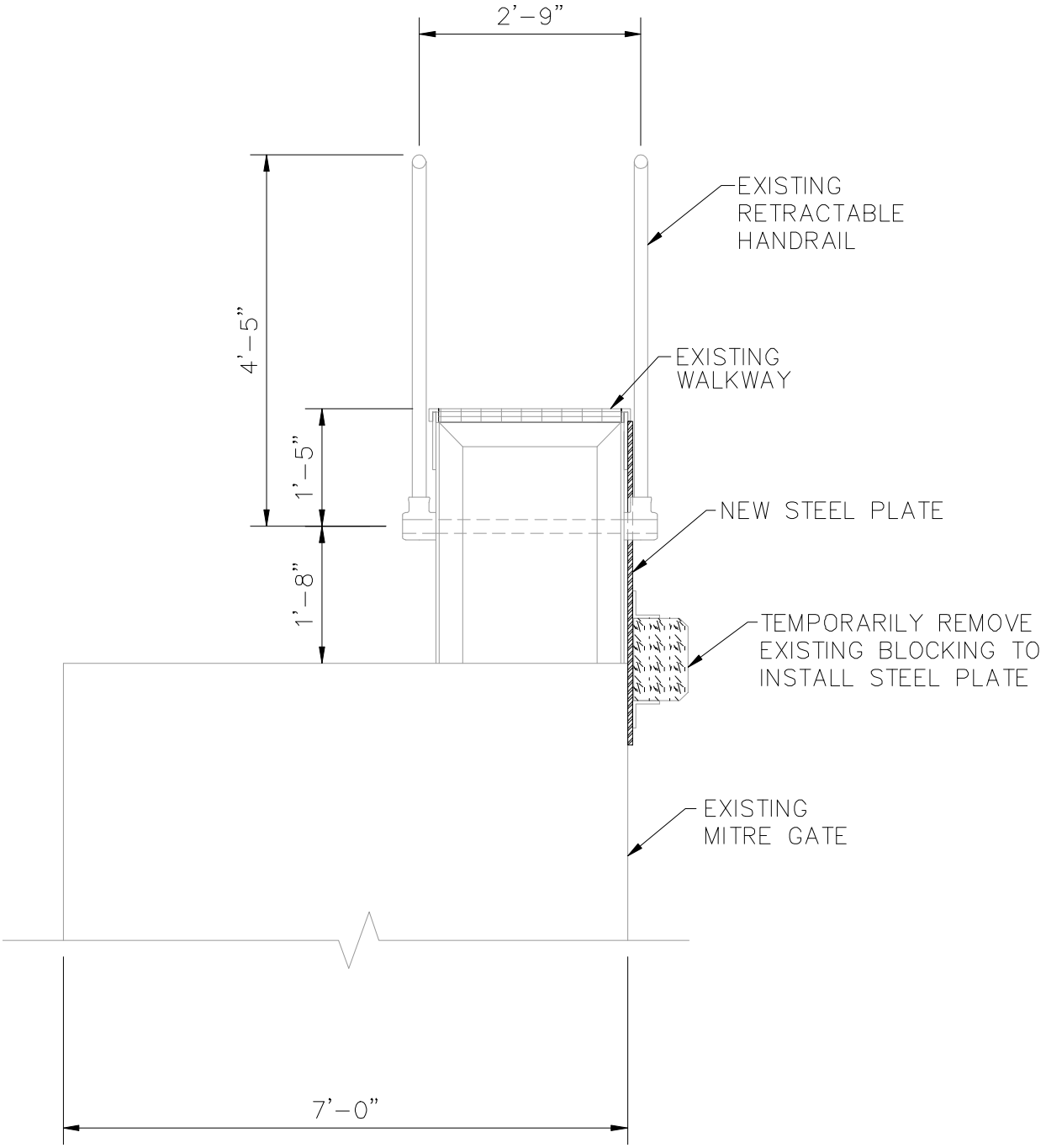
4.7.2 Locks and Gates

Any lock modifications required by the increase in MOLL must also include consideration of the impacts of vessel wakes and wind driven waves. As discussed earlier, the maximum MOLL that is feasible to consider is 27.43 m (90.0 ft) without significant modifications to the gates and locks which would render the locks unusable for an extended time. Even at the MOLLs considered for this study 26.67 m (87.5 ft) to 27.43 m (90.0 ft), substantial modifications will have to be made to the existing locks including:

- Installation of a solid vertical plate on top of the lock gate (next to the walkway - up to 27.89 m (91.5 ft)) to provide a solid structure above the top of the lock gate capable of retaining water above 26.94 m (88.4 ft). Possible installation of a parapet here as well to limit wave runup. This plate would be installed along the walkway on the outside face of the gate so that clearances within the lock wall would not be an issue when closing the gate. (see Figure 4-18) The mechanisms for the modular walkway would also have to be modified with water-tight seals and to be self-lubricating.
- Installation of a rubber piece at lake side face of the lock hinge to protect the quoin, etc. from inundation and again to allow the water level to be raised without leakage. This rubber piece would be attached to the lock wall and the gate and would be flexible to allow opening/closing of the gate. (see Figure 4-19 for a conceptual representation). A similar watertight rubber sheath will also be required for the yoke. A clear plastic watertight cover may be preferable so that visual inspections of the yoke mechanism can be made.

- Installation of rubber boots (as at Miraflores) at the openings for the hydraulic arms. The seam at the top of the boots will also have to be filled. In addition to the boots, watertight closure gates will also have to be installed in the machinery tunnels to keep water from reaching critical components if the boots were to leak. With these additional closures, additional ventilation shafts will need to be installed along the machinery tunnels.
- Retrofit of the hydraulic arms to work underwater with alternate seals, etc. Additional provision will be required to cover the body of the hydraulic cylinder and the watertight sealing of the electronic sensors for the gate position (known as CIMS). In discussions with the USACE concerning Olmsted locks, placing the wiring in metal conduit which is welded to the arm cylinder works best.
- Installation of flaps at the drainage holes for the track. These flaps would allow water to drain out of the track but would not allow water to enter the track recess from the lock. It is expected that these flaps would be recessed in the lock wall to prevent damage from transiting ships. (see Figure 4-20)

Figure 4-18: Section of Vertical Plate Installed Along Outside Edge of Gate

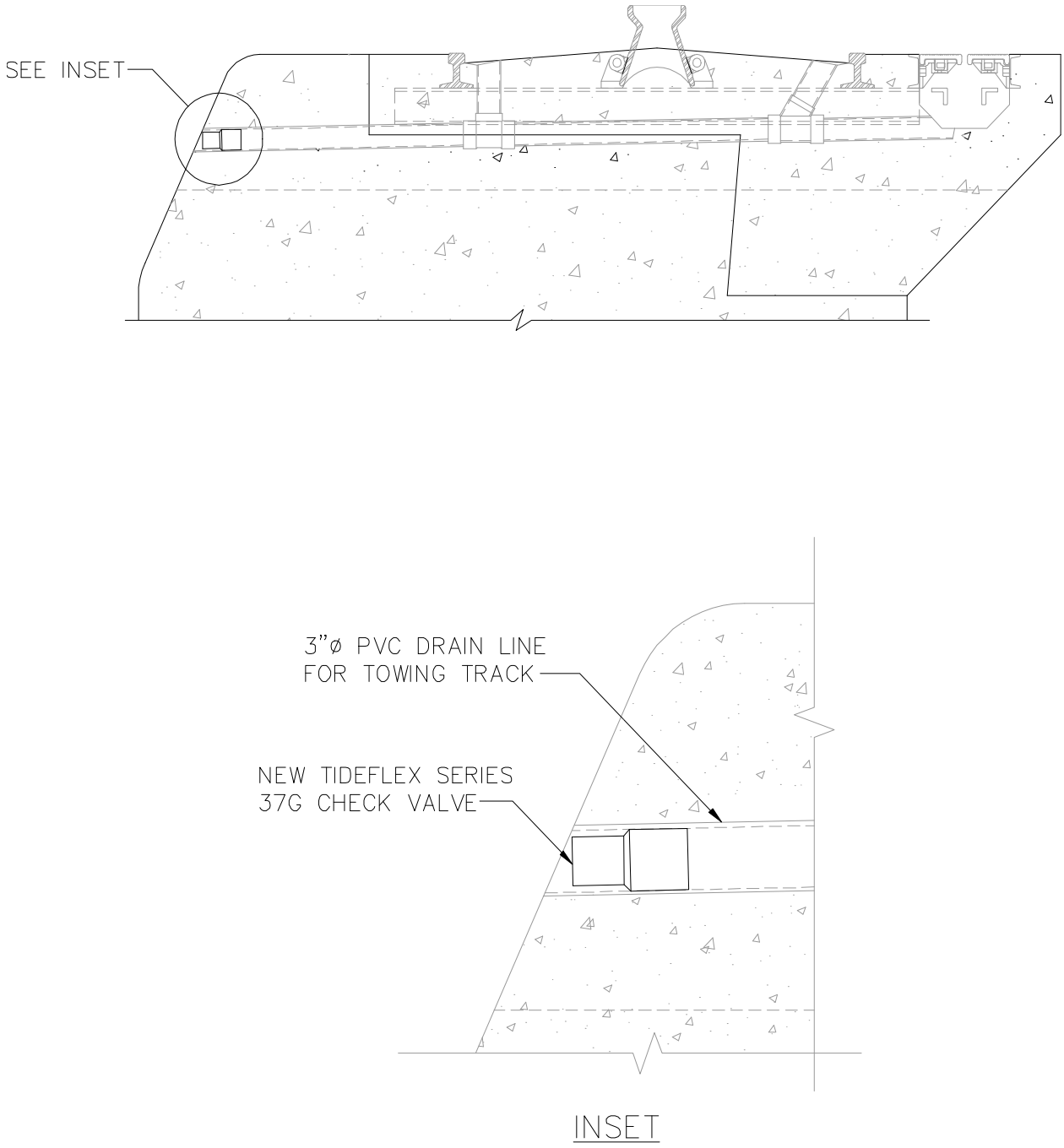


NOTE:

LATERAL BRACING MAY BE REQUIRED FOR WALKWAY

Figure 4-19: Conceptual Representation of Quoin Protection

Figure 4-20: Section of Proposed Drainage Controls in Lock Walls



For all of the proposed modifications, manufacturers were contacted and opinions of probable costs were prepared. Based on the discussions with manufacturers the following unit costs were developed:

Installation of Plate and Walkway Retrofits .....US$75k per gate

Installation of the Rubber Piece for the Protection of the Quoin .....US$35k per gate

Installation of Rubber Boots, Steel Plate Closure, Venilation.....US$100k per gate

Retrofit Existing Hydraulic Ram with Seals and Cover.....US$15k per gate

Installation of Flaps for Mule Track Drainage Holes.....US$500 each

These costs were then multiplied by the number of gates and drainage holes. Since raising the MOLL will also result in higher equalization levels in lower locks, the total cost was then multiplied by 0.5 and 0.25 to account for possible modifications required in the lower gates. These totals were then summed for each MOLL.

The resulting costs can be seen for each MOLL in the Table 4-38.

Table 4-38: Estimated Costs of Locks Modifications

MOLL	Locks & Gates Estimated Retrofit/Replacement Costs
26.67 m (87.50 ft)	$0
26.82 m (88.00 ft)	$1,400,000
26.97 m (88.50 ft)	$3,150,000
27.13 m (89.00 ft)	$3,832,500
27.28 m (89.50 ft)	$3,832,500
27.43 m (90.00 ft)	$3,832,500

4.7.3 Other ACP Facilities

For the majority of ACP facilities, the procedures and costs outlined earlier were used to determine costs.

As for the embankments, seismic stability analyses shown in in Appendix B have determined that the existing Gatun Dam and the saddle dams should be able to handle a raising of the MOLL to El 27.43 m (90.0 ft). There are some stability issues with Cano No. 4 and No. 2, but these are existing issues that warrant action whether the MOLL project is completed or not. As for top of dam elevations, Caño No.5 is the most susceptible to increased water levels in Gatun Lake. Unfortunately, time constraints did not allow visits to these sites during the field trip in May. However, given that no problems have been reported with the saddle dams under current conditions, it was assumed that 1.74 m (5.7 ft) of freeboard would be adequate for the saddle dams. Therefore, for the purposes of this study, it was assumed that the saddle dams would have to be raised to provide 1.74 m (5.7 ft) of freeboard. A cost of US$15 cu.m was assumed to be adequate for raising the saddle dams based on other studies in Panama.

It is very likely that many of the ACP fixed dock facilities will have to be raised or retrofitted in order to accommodate the project. Additionally, floating docks are located at Las Cruces Landing, Hydrographic Survey Pier in Gamboa, and next to the Fuel Barge Landing in Gatun. The guide piles or lateral support mechanisms for these docks define their critical elevations. However, since these docks are design to fluctuate with the water level, only minor retrofits will be required to enable them to accommodate the higher MOLL.

For the Gatun hydroelectric plant, the expected extensions required for the vent pipes were based upon estimates contained within the 1999 USACE report adjusted for inflation.

Table 4-39 shows the estimated costs of modifications to the ACP docks and facilities outside the Locks areas. A detailed listing of the costs can be seen in the table below and Appendix H.

Table 4-39: Estimated Cost of Modifications to ACP Facilities

MOLL	Other ACP Facilities Estimated Retrofit/Replacement Costs
26.67 m (87.50 ft)	$9,100,273
26.82 m (88.00 ft)	$11,947,010
26.97 m (88.50 ft)	$12,875,347
27.13 m (89.00 ft)	$14,398,645
27.29 m (89.50 ft)	$14,810,882
27.43 m (90.00 ft)	$16,087,340

4.7.4 Panama Canal Railroad

Given that the lowest portion of the current railroad is close to Gatun but in a location that does have an appreciable fetch from the North, a revised wave hindcast and runup and overtopping analysis was completed. It was found that the railway in the most vulnerable location requires approximately 1.68 m (5.5 ft) of freeboard to minimize damage to the ballast. This increased freeboard was also added after discussions with PCR which had real concerns about the stability of the ballast under higher water levels. It is felt that this analysis is likely conservative but should be used at this feasibility level of analysis. Costs for raising the track and power lines and lengthening equalization culverts were based upon data provided by PCR and past experience in Panama (US$98,500/km to raise track, US$750,000/km to raise power lines, US$10,000/culvert to lengthen). Table 4-40 shows the resulting costs to rehabilitate the railway for the proposed MOLLs.

Table 4-40: Estimated Costs to Retrofit PCR Railway, Towers and Affected Culverts

Trackwork for Raised Lake Levels using 5.5 feet of Freeboard After Construction of PCRR Mainline Trackwork Project													
Maximum Lake Level Elev. (FT)	Minimum Top of Rail Elev. (FT)	Track Station - Begin (FT)*	Track Station - End (FT)*	Track to be Raised (FT)	Track to be Raised (KM)	Possible No. of Towers to be Raised	Possible No. of Culverts to Extend	Lowest Existing Track Elev. (FT)	Lowest Existing Track Sta. (FT)	Cost to Raise Track	Possible Cost to Raise Towers	Possible Cost to Extend Culverts	Total Costs
87.5**								94.66	479+00				
88.00	93.50							94.66	479+00				
88.50	94.00							94.66	479+00				
89.00	94.50							94.66	479+00				
89.50	95.00	47500	49800	2300	0.70	6	0	94.66	479+00	$69,000	$526,125	$0	**$595,125**
90.00	95.50	47250	52400	5150	1.57	16	2	94.66	479+00	$154,500	$1,178,063	$20,000	**$1,352,563**

4.7.5 Other Third Party Structures

For the Monte Lirio bridge, the required freeboard was held the same as for the other structures it determining which MOLL would require the bridge to be replaced. For the Gamboa bridge, however, the minimum freeboard allowed was held to 1.23 m (4 ft) based on discussions with ACP staff. Given that the low steel elevation for the navigation span was 28.65 m (94.0 ft), an MOLL of 27.43 m (90.0 ft) would still be feasible.

Table 4-41 below shows the total estimated costs per MOLL for these other third party structures. Individual items can be seen in the spreadsheet within Appendix H.

Table 4-41: Estimated Cost of Modifications to Third Party Structures

MOLL (ft)	Other Third Party Estimated Retrofit/Replacement Costs
26.67 m (87.50 ft)	$2,934,350
26.82 m (88.00 ft)	$3,135,379
26.97 m (88.50 ft)	$3,170,279
27.13 m (89.00 ft)	$7,820,894
27.28 m (89.50 ft)	$9,747,874
27.43 m (90.00 ft)	$9,796,806

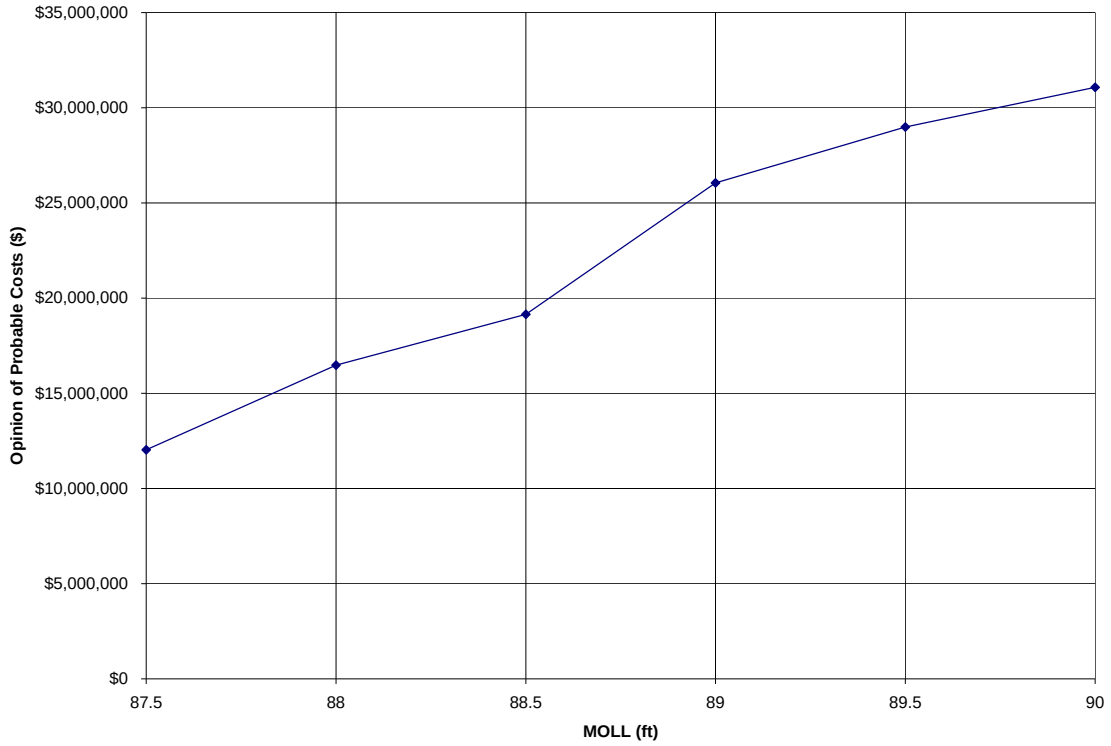
4.7.6 Summary of Costs per MOLL

For evaluating the cost implication for raising the MOLL for Gatun Lake, a table and two simple cost curves have been developed as seen in Table 4-42 and Figure 4-21 to illustrate the cost for retrofitting or replacing the existing structures at varying MOLLs. Table 4-42 reports all of the total structure retrofit costs (discussed above). The MOLL on the curve ranges from elevation 87.5 ft (26.67 m) MSL to 90 ft (27.43 m) MSL, incremented at 0.5 foot (0.15 m) intervals. It is also very important to note that these summaries do not include the additional costs that would be incurred for the proposed Post-Panamax locks to accommodate the various MOLLs.

Table 4-42: Summary of Total Retrofit Costs for MOLL Project

MOLL (ft)	Locks & Gates	Other ACP Facilities	Third Party Structures (Inc. PCR)	Total
26.67 m (87.50 ft)	$0	$9,100,273	$2,934,350	$12,034,623
26.82 m (88.00 ft)	$1,400,000	$11,947,010	$3,135,379	$16,482,389
26.97 m (88.50 ft)	$3,150,000	$12,875,347	$3,170,279	$19,145,626
27.13 m (89.00 ft)	$3,832,500	$14,398,645	$7,820,894	$26,052,039
27.28 m (89.50 ft)	$3,832,500	$14,810,882	$10,342,999	$28,986,381
27.43 m (90.00 ft)	$3,832,500	$16,087,340	$11,149,369	$31,069,209

Figure 4-21: Estimated Costs to Retrofit/Replace Existing Structures vs. MOLL



5 PROJECT BENEFITS EVALUATION

5.1 Introduction

The selection of the optimum MOLL for the Canal must consider capital costs, operational and maintenance issues, environmental impacts and the economic benefits that may be derived from the additional draft provided.

Clearly, the final decision will be based on a subjective evaluation of all of these factors and the significance of each benefit or impact will depend to a large extent on the specific area of interest of the evaluator. For example, the waterproofing of the operating arms of the lock gates is technically feasible, but also limits the ability of the maintenance crew to visually see problems with important components that will now be under water for much of the time. This is of critical importance to the maintenance crew, but of limited interest to an environmental specialist.

However, there are a number of quantifiable benefits associated with the increments in MOLL and these can be compared with the added capital and operational costs to form the basis of a cost – benefit analysis.

In particular, the economic benefits of the increased MOLL may include:

- Increased revenues from the ability to offer deeper draft during the dry season
- Avoidance of dredging
- Avoidance of shortfalls of water as demand for M&I and lockage water continues to grow
- Improved system reliability
- Increase the capacity of the Canal as transits increase with the new Locks in place

The following analysis evaluates the incremental cost for each MOLL elevation and sets out a basis for the assessment of the economic benefits analysis using the approach recommended by the US Army Corps of Engineers in its report of 1999. However, many of the parameters used by the Corps are not considered to be appropriate to the current and expected future operating conditions for the Canal. Following discussion with ACP, the consultants have been requested to defer completion of the benefits analysis until the presentation and discussions on the Draft Report have been completed.

5.2 Summary of Project Costs

Costs associated with the works necessary to accommodate incremental increases in MOLL can be summarized under the following categories:

- Modifications to ACP structures
    - Gatun and Pedro Miguel Lock Systems
    - Other Facilities
- Modifications to Third Party Structures
- Lake Gatun Shorelines
- Spillway Discharge Zones
- Environmental or Socio-economic mitigation costs

5.2.1 Modifications to ACP Structures

Modifications to the ACP structures are presented earlier in Section 4. Table 5-1 shows the incremental costs to modify critical structures at the Locks.

Table 5-1: Costs of Modifications to ACP Structures

MOLL (ft)	Construction Cost		
	Locks & Gates	Other ACP Facilities	Total
87.50	$0	$9,100,273	$9,100,273
88.00	$1,400,000	$11,947,010	$13,347,010
88.50	$3,150,000	$12,875,347	$16,025,347
89.00	$3,832,500	$14,398,645	$18,231,145
89.50	$3,832,500	$14,810,882	$18,643,382
90.00	$3,832,500	$16,087,340	$19,919,840

5.2.2 Modifications to Third Party Structures

Modifications to structures not owned by ACP include flood protection measures for the Panama Canal Railroad, communities and private facilities in and around Gatun Lake. The costs associated with each elevation increase in MOLL are indicated in Table 5-2, below.

Table 5-2: Costs of Modifications to Third Party Structures

MOLL (ft)	Construction Cost
87.50	$2,934,350
88.00	$3,135,379
88.50	$3,170,279
89.00	$7,820,894
89.50	$10,342,999
90.00	$11,149,369

5.2.3 Environmental Mitigation Measures

As outlined in Section 4.5, environmental mitigation is recommended to ameliorate the impacts of the increase in MOLL. At this time, it is recommended that mitigation costs be allocated on a percentage basis, which can then be applied to the incremental capital costs of the project.

5.2.4 Summary of Costs

Table 5-3 presents the summarized costs for each increase in MOLL. These costs can then be set against the economic benefits derived from the project in order to test the feasibility of each incremental increase in the MOLL.

Table 5-3: Summary of Costs Associated with each MOLL Increase

MOLL (ft)	Locks & Gates	Other ACP Facilities	Third Party Structures	Mitigation	Total
87.50	$0	$9,100,273	$2,934,350	$180,519	$12,215,142
88.00	$1,400,000	$11,947,010	$3,135,379	$247,236	$16,729,625
88.50	$3,150,000	$12,875,347	$3,170,279	$287,934	$19,483,560
89.00	$3,832,500	$14,398,645	$7,820,894	$390,781	$26,442,820
89.50	$3,832,500	$14,810,882	$10,342,999	$434,796	$29,421,177
90.00	$3,832,500	$16,087,340	$11,149,369	$466,038	$31,535,247

5.3 Economic Benefits Assessment

5.3.1 Background

As identified in the calculations above, modifying the operating rules at Gatun Lake will affect the ability to meet water demands and change operations of the reservoir. The increased volume of water available and change in reliability will result in an economic benefit.

In prior studies by USACE (1999), economic benefits/losses that may be incurred by raising the operating elevation of Gatun Lake were separated into four components:

- Additional Volume of Water Available for Navigation
- Reliability Water Supply for Navigation
- Reliability for Municipal and Industrial Supply (M & I)
- Hydropower

USACE (1999) assumed a 50 year planning horizon (from 2010 to 2060) and quantified the average annual benefits for raising the MOLL of Gatun Lake to 89 ft PLD. A discount rate of 12 % was assumed. It was assumed that all projected M & I demands would be met prior meeting demand for navigation.

The general methodology for estimating economic benefits, presented by the USACE (1999) is followed for this investigation for reliability of water supply for navigation and M & I, and hydropower.

5.3.2 Additional Volume of Water Available for Navigation

Based on the revisions in projected water demands, the methodology presented by the USACE (1999) for quantifying the benefits of additional volume of water available for navigation is not applicable. Theoretically, the additional storage volume of water available by raising the operating level of Gatun Lake would increase the quantity of water available for navigation and decrease the likelihood of water shortages. However, there must be a current, real demand for this additional volume of water for economic benefits to be realized. Under the USACE analysis, the additional volume provided by each MOLL (as compared to the baseline condition) that would still meet the baseline reliability was considered an economic benefit regardless of whether or not a real demand existed. In other words, the analysis realized an economic benefit for water even when the lockage demands were not high enough to use the additional water.

For example, under the baseline conditions, with the MOLL at 26.67 m PLD (87.5 ft PLD), the estimated hydrologic reliability is 99.8%. Under the scenario whereby the MOLL is raised to 26.83 m (88 ft) PLD, at a reliability of 99.8% the estimated equivalent daily lockages is 40.22 (1.62 lockages more than the baseline conditions of 38.66). The

resulting estimated annual benefit of raising the lake level to 26.83 m (88 ft) PLD, assumed to be $72,512/lockage, would be $42.9 Million. The USACE analysis realized this benefit even though navigation demands did not reach 40.22 lockages until later years. The valuation of those benefits would only be merited if the additional water can be used to meet demands for navigation, which is not the case. Also, since economic benefits were also being counted for the reliability of navigation (see next section), this was, in reality, double counting the same benefit.

5.3.3 Reliability of Navigation and M & I

The reliability of the system to provide all of the water to meet navigation and M & I demands would increase as the MOLL for Gatun Lake is raised. Table 5-4 and Table 5-5 summarize the projected demands and estimated reliability for each of the scenarios through 2030 (currently the limit of ACP projections for lockage demands).

Table 5-4: Summary of Demands (in equivalent lockages)

Demand	BASE	2010	2020	2020 PP	2030	2030 PP
M&I	3.6	5.4	6.2	6.2	6.9	6.9
Navigation	34.9	33.3	36.5	35.1	36.5	36.6
% Navigation Panamax	100%	100%	100%	52.4%	100%	55.7%
Total Demand	38.6	38.7	42.7	41.3	43.4	43.5

Source: ACP projections, October 2004

Table 5-5: Hydrologic Reliability Estimated Based on Monthly Data from 1948 - 1999

Scenario	BASE	2010	2020	2020 PP	2030	2030 PP
26.67 m (87.5 ft)	99.83%	99.82%	99.59%	99.36%	99.53%	99.16%
26.82 m (88.0 ft)	99.91%	99.90%	99.65%	99.45%	99.60%	99.27%
26.97 m (88.5 ft)	99.94%	99.93%	99.69%	99.54%	99.65%	99.33%
27.13 m (89.0 ft)	99.98%	99.98%	99.73%	99.63%	99.68%	99.42%
27.28 m (89.5 ft)	100.00%	100.00%	99.77%	99.69%	99.72%	99.53%
27.43 m (90.0 ft)	100.00%	100.00%	99.81%	99.74%	99.77%	99.60%

The economic benefit associated with the increased navigation reliability is estimated as a function of change in reliability and associated value per lockage for navigation. It is assumed that the average value of a lockage for the existing system is $72,512/lockage.[31] For purposes of this study, it is assumed that the average value for a Post-Panamax lockage is $131,139/lockage.[32] To estimate the relative percentage of Panamax to Post-Panamax lockages, the projections provided by ACP were used directly.

For example, in 2020 Post-Panamax, the navigation demand is projected to be 35.1 lockages/day; 52.4 % of the total navigation demand is Panamax ships, equating to 18.39 lockages/day Panamax with the remaining 16.71 lockages/dayPost-Panamax.

The economic benefit of modifying the operating rules to increase the MOLL from 26.68 m (87.5) to 26.83 m (88 ft) PLD is a function of the increased reliability from 99.36% to 99.45%, an increase in reliability of 0.09%. The annual benefit to navigation for 2020 Post-Panamax ships, based on the increased reliability, is estimated as 365 days x (0.00089) x (18.39 panamax lockages x ($72,512/lockage) + 16.71 Post-Panamax lockages x ($131,139/lockage)) = approximately $1,146,000/year.

Table 5-6 summarizes the estimated average annual benefit realized by an increased navigational reliability, using the approach presented by the US Army Corps of Engineers in its 1999 report. It should be noted that these estimated benefits should be adjusted in the future based on projected toll structure changes for future years.

[31] Based on ACP figures for FY 2004/2004. Equivalent to $65,920/transit X 1.1 transits/lockage.

[32] Assumes a ratio of potential income between Panamax and Post-Panamax based on expected average load of 8,500 TEUs for a Post-Panamax ship in 2015, compared to 4,700 TEUs for current Panamax vessels.

Table 5-6: Average Annual Benefits of Increased Reliability in Navigation (Relative to Baseline Conditions)

Scenario	BASE	2010	2020	2020 Post-Panamax	2030	2030 Post-Panamax
26.67 m (87.5 ft)	$0	$0	$0	$0	$0	$0
26.82 m (88.0 ft)	$739,000	$691,000	$616,000	$1,146,000	$742,000	$1,465,000
26.97 m (88.5 ft)	$1,086,000	$1,026,000	$980,000	$2,264,000	$1,168,000	$2,221,000
27.13 m (89.0 ft)	$1,461,000	$1,405,000	$1,382,000	$3,469,000	$1,506,000	$3,390,000
27.28 m (89.5 ft)	$1,601,000	$1,612,000	$1,778,000	$4,236,000	$1,909,000	$4,881,000
27.43 m (90.0 ft)	$1,601,000	$1,612,000	$2,169,000	$4,802,000	$2,309,000	$5,817,000

Following the USACE methodology, estimated average annual benefits for municipal and industrial use are assumed to have same increase in reliability as for navigation. The assumed value of water use was $45,100/equivalent lockage. This value was based on the estimate provided by the USACE in their 1999 study, adjusted for inflation. Table 5-7 identifies the estimated benefits of increased reliability on municipal and industrial water use for each of the design scenarios. However, it is our opinion that quantifying this monetary benefit is suspect as the navigation benefits are calculated by assuming that all M&I water uses are met first and all shortages will be entirely lockages. Therefore, if M&I water uses will always be met first, the only monetary benefit that the project will realize will be the reduction in navigation shortages.

Table 5-7: Summary of Average Annual Benefits of Increased Reliability of M & I (Relative to Baseline Conditions)

Scenario (Defined by MOLL)	BASE	2010	2020	2020 Post-Panamax	2030	2030 Post-Panamax
26.67 m (87.5 ft)	$0	$0	$0	$0	$0	$0
26.82 m (88.0 ft)	$48,000	$70,000	$65,000	$91,000	$88,000	$128,000
26.97 m (88.5 ft)	$71,000	$103,000	$103,000	$179,000	$139,000	$193,000
27.13 m (89.0 ft)	$95,000	$141,000	$146,000	$274,000	$179,000	$295,000
27.28 m (89.5 ft)	$104,000	$162,000	$187,000	$335,000	$226,000	$424,000
27.43 m (90.0 ft)	$104,000	$162,000	$229,000	$380,000	$274,000	$506,000

5.3.4 Hydropower

As stated previously, there will be a net slight decline in the amount of hydropower generation over time as the demands increase for navigation and water supply, given that meeting the M&I and Navigation demands takes priority. As noted earlier, raising the water surface elevation will have a minor impact on hydropower operations. The loss in power generation may be estimated in economic terms, assuming the value of power generation of $0.048/kWh (based on data from ETESA website). Table 5-8 summarizes the resulting economic loss that would be realized based on HEC-5 analysis. Additional benefits could be realized if modifications were made to the operating rules; this also may require further improvements to the generation facilities.

Table 5-8: Average Annual Benefits of Hydropower Generation (Relative to Baseline Conditions)

Scenario	*BASE*	*2010*	*2020*	2020 Post-Panamax	*2030*	2030 Post-Panamax
26.67 m (87.5 ft)	$0	$0	$0	$0	$0	$0
26.82 m (88.0 ft)	-$15,000	-$28,000	$4,000	-$36,000	-$11,000	-$24,000
26.97 m (88.5 ft)	-$21,000	-$45,000	-$25,000	-$54,000	-$9,000	-$45,000
27.13 m (89.0 ft)	-$27,000	-$43,000	-$25,000	-$48,000	-$23,000	-$41,000
27.28 m (89.5 ft)	-$57,000	-$57,000	-$38,000	-$61,000	-$36,000	-$36,000
27.43 m (90.0 ft)	-$55,000	-$39,000	-$53,000	-$70,000	-$49,000	-$60,000

5.3.5 2060 Scenario – Canal Full Build-out

As can be seen from the reliability results above, the growth rate of the projected water demands over the next 25 years is not high enough to fully realize the economic benefit of raising the MOLL. However, given that the current locks have been in service over 90 years and any works constructed would likely be in service over the next 100 years (at least), additional benefit analyses were completed for the full buildout scenario (using 2060 M&I demands) outlined in 4.3 of the report.

Table 5-9 and Table 5-10 summarize the projected demands and estimated reliability for each of the scenarios through 2060.

Table 5-9: Summary of Demands (in equivalent lockages)

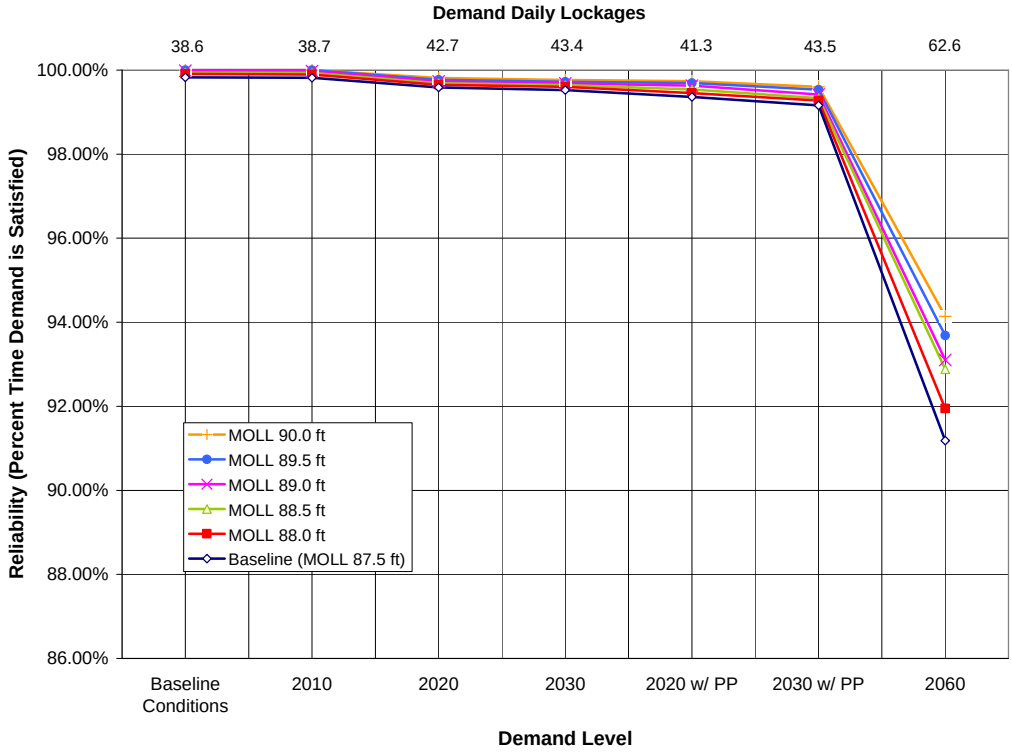
Demand	BASE	2010	2020	2020 PP	2030	2030 PP	2060
M&I	3.6	5.4	6.2	6.2	6.9	6.9	9.1
Navigation	34.9	33.3	36.5	35.1	36.5	36.6	53.5
% Navigation Panamax	100%	100%	100%	52.4%	100%	55.7%	68.2%
Total Demand	38.6	38.7	42.7	41.3	43.4	43.5	62.6

Table 5-10: Hydrologic Reliability Estimated Based on Monthly Data from 1948 - 1999

Scenario	BASE	2010	2020	2020 PP	2030	2030 PP	2060 PP
26.67 m (87.5 ft)	99.83%	99.82%	99.59%	99.36%	99.53%	99.16%	91.19%
26.82 m (88.0 ft)	99.91%	99.90%	99.65%	99.45%	99.60%	99.27%	91.95%
26.97 m (88.5 ft)	99.94%	99.93%	99.69%	99.54%	99.65%	99.33%	92.88%
27.13 m (89.0 ft)	99.98%	99.98%	99.73%	99.63%	99.68%	99.42%	93.10%
27.28 m (89.5 ft)	100.00%	100.00%	99.77%	99.69%	99.72%	99.53%	93.69%
27.43 m (90.0 ft)	100.00%	100.00%	99.81%	99.74%	99.77%	99.60%	94.14%

Figure 5-1 illustrates the reduction in hydrologic reliability that would be realized from 2030 to 2060, once the system has reached full capacity. In Table 5-11 and Figure 5-1 it is seen that raising the MOLL really begins to pay dividends when the water demands are at this level. The relative differences (or gain in percentages) in reliability from the raise MOLL scenarios when compared to the baseline conditions are much larger. For example, the 27.43 m (90.0 ft) MOLL scenario will provide a full 2.95% increase in reliability compared to the baseline conditions at 26.67 m (87.5 ft) (94.14% - 91.19%) for the full buildout scenario, whereas the 27.43 m (90.0 ft) MOLL scenario will only provide a 0.44% increase in reliability compared to the baseline conditions at 26.67 m (87.5 ft) (99.60% - 99.16%), for the 2030 PP scenario.

Figure 5-1: Panama Canal Hydrologic Reliability for Varying MOLL at Gatun Lake



The economic benefit associated with the increased navigation reliability was also estimated for the full buildout scenario and is shown in Table 5-11.

Table 5-11: Average Annual Benefits of Increased Reliability in Navigation (Relative to Baseline Conditions) US$2004

Scenario	BASE	2010	2020	2020 Post-Panamax	2030	2030 Post-Panamax	2060 Post-Panamax
Base - 26.67 m (87.5 ft)	$0	$0	$0	$0	$0	$0	$0
26.82 m (88.0 ft)	$739,000	$691,000	$616,000	$1,146,000	$742,000	$1,465,000	$13,538,000
26.97 m (88.5 ft)	$1,086,000	$1,026,000	$980,000	$2,264,000	$1,168,000	$2,221,000	$30,209,000
27.13 m (89.0 ft)	$1,461,000	$1,405,000	$1,382,000	$3,469,000	$1,506,000	$3,390,000	$34,140,000
27.28 m (89.5 ft)	$1,601,000	$1,612,000	$1,778,000	$4,236,000	$1,909,000	$4,881,000	$44,493,000
27.43 m (90.0 ft)	$1,601,000	$1,612,000	$2,169,000	$4,802,000	$2,309,000	$5,817,000	$52,586,000

Table 5-12 identifies the estimated benefits of increased reliability on municipal and industrial water use for each of the design scenarios.

Table 5-12: Summary of Average Annual Benefits of Increased Reliability of M & I (Relative to Baseline Conditions)

Scenario (Defined by MOLL)	BASE	2010	2020	2020 Post-Panamax	2030	2030 Post-Panamax	2060 Post-Panamax
Base - 26.67 m (87.5 ft)	$0	$0	$0	$0	$0	$0	$0
26.82 m (88.0 ft)	$48,000	$70,000	$65,000	$91,000	$88,000	$128,000	$1,140,000
26.97 m (88.5 ft)	$71,000	$103,000	$103,000	$179,000	$139,000	$193,000	$2,543,000
27.13 m (89.0 ft)	$95,000	$141,000	$146,000	$274,000	$179,000	$295,000	$2,874,000
27.28 m (89.5 ft)	$104,000	$162,000	$187,000	$335,000	$226,000	$424,000	$3,745,000
27.43 m (90.0 ft)	$104,000	$162,000	$229,000	$380,000	$274,000	$506,000	$4,427,000

Table 5-13 summarizes the resulting economic loss that would be realized based on HEC-5 analysis. Additional benefits could be realized if modifications were made to the operating rules; this also may require further improvements to the generation facilities.

Table 5-13: Average Annual Benefits of Hydropower Generation (Relative to Baseline Conditions)

Scenario	BASE	2010	2020	2020 Post-Panamax	2030	2030 Post-Panamax	2060 Post-Panamax
Base - 26.67 m (87.5 ft)	$0	$0	$0	$0	$0	$0	$0
26.82 m (88.0 ft)	-$15,000	-$28,000	$4,000	-$36,000	-$11,000	-$24,000	-$174,000
26.97 m (88.5 ft)	-$21,000	-$45,000	-$25,000	-$54,000	-$9,000	-$45,000	-$390,000
27.13 m (89.0 ft)	-$27,000	-$43,000	-$25,000	-$48,000	-$23,000	-$41,000	-$398,000
27.28 m (89.5 ft)	-$57,000	-$57,000	-$38,000	-$61,000	-$36,000	-$36,000	-$508,000
27.43 m (90.0 ft)	-$55,000	-$39,000	-$53,000	-$70,000	-$49,000	-$60,000	-$584,000

5.3.6 Summary of Benefits

Table 5-14 presents the summarized benefits for each increase in MOLL. These benefits can then be set against the economic costs derived from the project in order to

test the feasibility of each incremental increase in the MOLL. It should be noted that the total benefits were calculated by adding taking the benefits to navigation and M&I and subtracting the loss in hydropower production.

Table 5-14: Summary of Benefits Associated with each MOLL Increase

Scenario	BASE	2010	2020	2020 Post-Panamax	2030	2030 Post-Panamax	2060 Post-Panamax
Base - 26.67 m (87.5 ft)	$0	$0	$0	$0	$0	$0	$0
26.82 m (88.0 ft)	$772,000	$733,000	$685,000	$1,201,000	$819,000	$1,569,000	$14,504,000
26.97 m (88.5 ft)	$1,136,000	$1,084,000	$1,058,000	$2,389,000	$1,298,000	$2,369,000	$32,362,000
27.13 m (89.0 ft)	$1,529,000	$1,503,000	$1,503,000	$3,695,000	$1,662,000	$3,644,000	$36,616,000
27.28 m (89.5 ft)	$1,648,000	$1,717,000	$1,927,000	$4,510,000	$2,099,000	$5,269,000	$47,730,000
27.43 m (90.0 ft)	$1,650,000	$1,735,000	$2,345,000	$5,112,000	$2,534,000	$6,263,000	$56,429,000

It is important to note that this benefits analysis does not include the economic impact of potential draft restrictions for this project. Although the reliability analyses are based on the water supply volume, a much more meaningful and realistic factor that should be considered is the amount of time that each scenario would have to institute draft restrictions under each MOLL.

5.4 Selection of MOLL

Selection of the optimal MOLL for the Lake requires consideration of a number of factors, not all of which can be related to economic of financial terms. The final selection is dependent on the expected design life of the project future water demands (both M&I and lockages), particularly with the new locks in place. The selection of the optimal MOLL will also be very dependent on the system reliability required for future operations.

5.4.1 Incremental Costs and Economic Benefits

Table 5-15: Summary of Costs Associated with each MOLL Increase

MOLL (ft)	Locks & Gates	Other ACP Facilities	Third Party Structures	Mitigation	Total
87.50	$0	$9,100,273	$2,934,350	$180,519	$12,215,142
88.00	$1,400,000	$11,947,010	$3,135,379	$247,236	$16,729,625
88.50	$3,150,000	$12,875,347	$3,170,279	$287,934	$19,483,560
89.00	$3,832,500	$14,398,645	$7,820,894	$390,781	$26,442,820
89.50	$3,832,500	$14,810,882	$10,342,999	$434,796	$29,421,177
90.00	$3,832,500	$16,087,340	$11,149,369	$466,038	$31,535,247

Note: The evaluation criteria for flood protection or structural modifications indicated that $12.25 million should be spent on ACP and Third Party structures to avoid flooding under the exisiting MOLL. Although this work has not yet been undertaken, it will become critical at the first increment in the MOLL and is included in the incremental cost analyses.

Table 5-16: Summary of Benefits Associated with each MOLL Increase

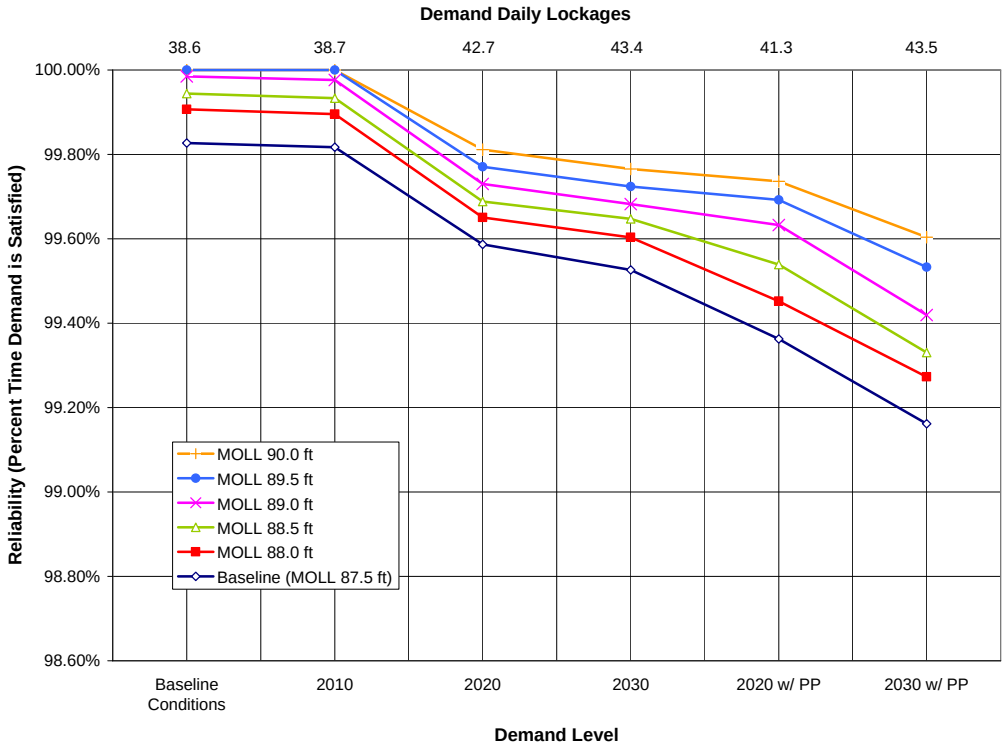
Scenario	BASE	2010	2020	2020 Post-Panamax	2030	2030 Post-Panamax	2060 Post-Panamax
Base - 26.67 m (87.5 ft)	$0	$0	$0	$0	$0	$0	$0
26.82 m (88.0 ft)	$772,000	$733,000	$685,000	$1,201,000	$819,000	$1,569,000	$14,504,000
26.97 m (88.5 ft)	$1,136,000	$1,084,000	$1,058,000	$2,389,000	$1,298,000	$2,369,000	$32,362,000
27.13 m (89.0 ft)	$1,529,000	$1,503,000	$1,503,000	$3,695,000	$1,662,000	$3,644,000	$36,616,000
27.28 m (89.5 ft)	$1,648,000	$1,717,000	$1,927,000	$4,510,000	$2,099,000	$5,269,000	$47,730,000
27.43 m (90.0 ft)	$1,650,000	$1,735,000	$2,345,000	$5,112,000	$2,534,000	$6,263,000	$56,690,000

As can be seen from the above tables, from a cost versus benefit perspective, the financial return period for the increased MOLL project will take many years if M&I and navigation water demands follow current projections through 2030. However, once water demands approach full-buildout conditions, the project becomes more attractive financially, and raising the lake level all the way to 27.43 m (90.0 ft) could be justified.

5.4.2 Reliability Considerations

Based on the analyses above, it appears that raising the MOLL to at least an elevation of 26.83 m (88.0 ft) is required to provide the same reliability that the current MOLL provides (under existing demands) for projected 2030 demands without PP locks. It also appears that raising the MOLL to the current proposed maximum of 27.43 m (90.0 ft) would be enough to provide the current level of reliability in year 2030 with PP locks.

Figure 5-2: Panama Canal Hydrologic Reliability for Varying MOLL at Gatun Lake



5.4.3 Recommended MOLL

Given the need for further examination and discussion of the selection parameters used to identify the optimum MOLL, no firm conclusions are presented in this report. However, if it is considered that the current level of reliability must be provided in the future and water demands will eventually approach full-buildout conditions, an MOLL of at least 27.13 m (89.0 ft) and up to 28.04 m (90.0 ft) should be considered. While an MOLL of 28.04 m (90.0 ft) would be preferable as water demands approach full-buildout conditions, this significantly exceeds the ACP projections.

This then indicates that a reasonable target elevation for the MOLL will lie between these two elevations, with the exact determination of the optimum MOLL to be dependent on further analysis of the costs, transit projections and prioritization of evaluation criteria.

6 SUMMARY AND RECOMMENDATIONS

6.1 Existing Spillway

Primary findings and recommendations arising from the analyses of the existing spillway are:

- Although Gatun Spillway has successfully operated for 91 years, it now requires upgrading to meet modern dam safety requirements. Modifications needed to accommodate a higher MOLL can be incorporated into these upgrades.
- Gatun Dam has been determined to be a “High Hazard” structure. Due to this designation and the potential economic consequences of a protracted outage of the canal due to dam failure, the recommended design flood is the Probable Maximum Flood.
- The hydraulic capacity of the spillway alone is insufficient to meet flood control requirements for Gatun Dam and other canal structures in the event of the Probable Maximum Flood. The present flood control plan envisages supplemental discharge of water through lock structures. Previous releases of spill through lock culverts have caused damage to the lock structures. Additional spillway capacity is required and could be best acquired by building an additional spillway.
- The inflow design flood and the ranges of headwater and tailwater elevations under which the spillway is now required to operate are considerably different than those that were anticipated when the structure was designed. It is recommended that the spillway be modeled over the full range of hydraulic conditions to ensure that it can safely perform as required
- The 1,000 and 5,000-year (Level I and Level II) return period earthquakes were selected for structural stability analyses. Stability analyses of the existing spillway ogee section under hydraulic and seismic loading indicate that it can satisfy overall sliding and overturning stability criteria under a maximum operating lake level of El. 27.44 m (90.0 ft) and a PMF reservoir level of El. 28.05 m (92.0 ft.).
- Previous structural analyses of the spillway piers by ACP have identified a lateral stability deficiency due to a lack of vertical steel reinforcement. Preliminary investigations indicate that this deficiency could be remedied by retrofitting vertical tendons in the piers and horizontal struts between the tops of the piers.

- The spillway gates have exceeded their expected service life and are due for replacement. It is recommended that the existing vertical lift Stoney gates be replaced with vertical lift fixed wheel gates with overhead electric hoists.
- For planning purposes, the estimated construction cost of rehabilitation of the spillway piers is $2,534,000. The estimated construction cost of replacement of the spillway gates and hoists is $27,331,000.
- Flood control operations are critical to the continued and safe operation of the Panama Canal so the proposed spillway improvement program should be implemented as soon as possible. Remediation work on the existing spillway could temporarily reduce spillway capacity while existing gates are removed and modifications are made to piers, gate guides and hoists. Factors that could mitigate this temporary loss of capacity include: Adding the new spillway prior to remediation work, scheduling remediation work for the dry season and scheduling the work so that only one gate is out of serivce at any time. Scheduling of the project should be refined relative to ACP's overall plan for upgrading the canal.
- Strengthening of the existing piers could be carried out without restricting spillway capacity.

6.2 New Spillway

Additional spillway capacity is required at Gatun Lake to meet modern dam safety requirements for Gatun Dam. This would also enable ACP to increase the maximum operating level of the lake and reduce the surcharge that must be maintained to pass large floods.

The capacity of the new spillway under the current operating regime is 5912 cms (208,780 cfs) at the current maximum operating level of El. 26.67 m (87.5 ft) PLD and the estimated maximum discharge at El. 27.43 (90 ft.) is 6693 cms (236,360 cfs). The estimated maximum flow under PMF routing assumptions El. 28.04 m. (92.0 ft) PLD is 7303 cms (257,910 cfs).

6.2.1 Establishment of PMF

The computations used for this report indicate that the current PMF flood appears to be only four times greater than a reported 15-year event. Given that the current PMF was developed in the early 1970's and over 30 years of additional hydrologic data is now available, it is recommended that the PMF be re-evaluated prior to the design stage, in order to verify its applicability.

6.2.2 Location Alternatives

Study of alternative locations for new spillway have determined that the most promising site is at the left (west) side of the dam where the embankment dam abuts a natural ridge that is comprised of Gatun series sedimentary rocks covered with a mantle of natural soil and embankment fill.

The proposed spillway arrangement is for a gated overflow spillway with seven crest gates that are twice as high as the gates on the existing spillway. The gates would be of the radial gate design instead of the present vertical lift gates on the existing spillway.

6.2.3 Recommended Spillway Configuration

Development of additional spillway capacity at the west abutment of Gatun dam appears to be feasible.The recommended project configuration is for a gated overflow surface spillway with a short approach channel, discharge channel and concrete lined stilling basin.

6.2.4 Site Investigation

The site investigation undertaken by ACP for this feasibility study was used to determine the optimum location and configuration of the new spillway structures. However, there are still some unresolved issues related to the competence and surface profile of the rock layers in the area of the proposed spillway and the stilling basin. More information is also required to map the location of the apparent fault line within the project site.

6.2.5 Hydraulic Modelling

Design of the approach channel, stilling basin and outflow channel would benefit from hydraulic modelling. In addition the potential effects of higher tailwater on the safety and operation of the existing spillway should be explored in conjunction with recommended hydraulic modeling of operation under existing conditions.

6.2.6 Potential Design Refinements

Design refinements that should be explored at the detailed design phase, when additional site investigation data is available include:

- Rotating the spillway counterclockwise to improve the approach conditions.
- Reduce the length of training wall on the left (west side of the spillway channel).
- Use of ground improvement techniques to improve the stability of embankment fill.
- Move the control structure upstream to reduce the cost of the training walls

- Trim down the size of gravity walls on the discharge chute immediately downstream of the control structure.
- Explore the use of secant pile walls on the discharge channel and stilling basin to reduce the need for temporary cofferdams and ground support.
- Optimize the hydraulic and structural design of the spillway gate structure.
- Optimize the size and number of spillway gates.
- Optimize the stilling basin design. If feasible, a sloping apron Type V Basin may prove to be superior to Type II design.

6.2.7 Construction Schedule & Costs

The spillway would take at least two years to construct and is estimated to cost approximately $US 91.0 million. The full project implementation schedule is currently estimated to be 52 months or 4.33 years including environmental studies, additional field investigations and final design. Raising of Mindi Dike is also recommended to limit impacts to navigation during the Canal Operational Flood. This construction is estimated to cost on the order of $US 3.1 million. Therefore, the total cost of the new spillway project is estimated to be $US 94.1 million. If a dike is required to protect Fort Sherman Road across Mojinga Swamp, this would incur an additional cost of $US 4.0 million.

6.2.8 Recommended MOLL

Given the need for further examination and discussion of the selection parameters used to identify the optimum MOLL, no firm conclusions are presented in this report. However, if it is considered that the current level of reliability must be provided in the future and water demands will eventually approach full-buildout conditions, an MOLL of at least 27.13 m (89.0 ft) and up to 28.04 m (90.0 ft) should be considered. While an MOLL of 28.04 m (90.0 ft) would be preferable as water demands approach full-buildout conditions, this significantly exceeds the ACP projections.

This then indicates that a reasonable target elevation for the MOLL will lie between these two elevations, with the exact determination of the optimum MOLL to be dependent on further analysis of the costs, transit projections and prioritization of evaluation criteria. Currently, the range of costs for the MOLL project is expected to range between $US 12.3 million and $US 31.6 million depending on the final MOLL selected.