PANAMA CANAL CONCEPT DESIGN

Atlantic Locks Structure

Third Lane Lock

Final Report

Prepared for

Canal Capacity Projects Office

By

US Army Corps of Engineers

Volume 1 – Main Report and Appendices A through C

23 July 2003

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Abbreviations

ACP	Autoridad Del Canal De Panama (Panama Canal Authority)
COE	Corps of Engineers
GIS	Geographic Information System
ITR	Independent Technical Review
MO/QA	Management Oversight/Quality Assurance
IPC	Canal Capacity Projects Office
PLD	Precise Level Datum
RCC	Roller Compacted Concrete
QCP	Quality Control Plan
TEU	Twenty-foot Equivalent Unit

Appendices

Appendix A – Project Design Criteria
Appendix B – Gate Selection Study
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1. EXECUTIVE SUMMARY

1.1. Background

This Report presents the Concept Level Design for a double-lift single-lock structure at the Atlantic Ocean side of the Canal with water saving basins. When the basins are used in operations, they would save 50% of the water required for normal lock operations. The locks and water saving system were designed to be operated with or without using the water saving basins.

The Notice To Proceed for this work was received on 25 February 2002 with an eight-month delivery date for this first configuration. Work initiated with meetings in Panama and Pittsburgh, PA, USA, that were attended by ACP staff as well as a large number of experienced Corps staff in multiple disciplines from numerous Corps District and Division Offices and research facilities.

1.2. Overview

This Report presents the concept level design for a double-lift single structure lock configuration through narratives, drawings, design calculations and cost computations. While it is complete and suits present site conditions and criteria as stated in the Scope of Work, changes in criteria can have significant effects on the presented design. The Report also presents the estimated construction cost as well as costs for continuing engineering and design and construction management. This design shows that the locks are constructible at the adjusted A-2 Gatun site, adjacent to the existing locks at an angle of 9.75 degrees, generally using conventional construction techniques. Special construction techniques would be needed to construct the entrance walls in the most efficient manner. The siting of the locks has been adjusted to provide safe and efficient traffic management. The estimated first-cost for the construction of the lock is approximately $820,000,000. Additionally, recommended items of work needed to proceed into the final design have also been identified and an estimate has been provided. A time schedule for accomplishing the design and construction is also presented. The estimated construction time for the completion of all features is five years. This can be reduced as discussed below.

A design criteria report was prepared in accordance with the requirements of the Scope of Work and was expanded as design development proceeded. The criteria is in accordance with Corps of Engineers criteria and would result in a structure that is expected to be as reliable and time durable as the existing Panama Canal Locks, while minimizing risk of operation and maintenance. In preparing the final criteria for preparation of plans and specifications for construction, the criteria should be standardized for the locks construction at both ends of the Canal.

1.3. Project Features

This is a brief overview of the evaluation, conclusions reached, and the recommendations for the major project features. The Main Report and Appendices present the detailed work along with supporting calculations and written description of methodology and procedures used. A summary of the main features is as follows:

1.3.1. Lock Gates

Using a pre-screening process to establish evaluation criteria, the possible gate types and configurations were evaluated with a minimum of design effort but using experience and engineering judgment. The initial screening was performed at the Panama Project Initial Team Meeting held in Pittsburgh Pennsylvania on April 2002. The initial screening effort identified gates to carry forward as alternatives in the gate selection study. This initial screening process showed that the rolling and miter gates were the most favorable gate types for use in the Canal double-lift structure. Because of recent design information available from a current Corps project, it was decided to also advance the evaluation of the sector type gate along with the other two gate types.

A separate detailed gate selection study has been performed and is included with the main report as Appendix B – Gate Selection Study. The significance of the lock gate selection required that these three gate types be developed to a nearly equal level of design to provide a true comparison of the types and to make a justifiable recommendation of the gate type. In order to identify costs associated with each structure type the following analysis were perform:

1. Structural design of each gate type was performed using STAADPro, a general purpose structural analysis and design software package. CMITER, a computer program developed by USACE to design and analyze miter gates was used to design the mitering lock gates. Separate finite element analysis was performed to investigate the performance of the rolling gate and the miter gates. The gate designs resulted in an estimated cost for each gate type including appurtenant features.
2. Installation plans were developed by the Corps Marine Design Center. Where appropriate, the costs for additional features (ballast, cranes, etc) required to install and remove gates was identified and included in the gate alternative cost summary.
3. The rock excavation required to construct each of the proposed gate type was computed and priced.
4. The lock masonry features were proportioned for each gate type. The quantity of concrete and reinforcing steel was estimated and priced for each option.
5. Mechanical equipment was sized and priced for each gate type.

The following table gives the summary of the gate weights and total construction costs for the respective gate type features.

Sector Gate Alternative		
Sector Gates	**Comment**	**TOTAL**
Sector Gates with Operating Equipment	Gate Weight = 4,400 MT	$156,616,482
Construction of Gate Bays and Recesses (Includes Reinforcement and Excavation)	Complex Geometry, Large, Result in Excess Water Use	$184,065,500
Associated Features	Long Span Bridge Structures, Maintenance Facility	$9,792,514
	Total	$350,474,495

Rolling Gate Alternative		
Rolling Gates	**Comment**	**TOTAL**
Rolling Gates with Operating Equipment	Gate Weight = 5,100 MT	$187,378,304
Construction of Gate Bays and Recesses (Includes Reinforcement and Excavation)	Easily Constructed but Large	$162,902,845
Associated Features	Moderate Span Bridges, Does not include separate maintenance facility	$3,852,975
	Total	$354,134,125

Miter Gate Alternative		
Miter Gates	**Comment**	**TOTAL**
Miter Gates with Operating Equipment	Gate Weight = 3,500 MT	$137,822,838
Construction of Gate Bays and Recesses (Includes Reinforcement and Excavation)	Minimal Size Recess	$64,644,085
Associated Features	12 Bridges, Maintenance Facility Infrastructure, Titan used to set gates	$6,732,514
	Total	$209,199,436

Figure 1-1 Comparative Gate Costs

The comparative cost table shows that the miter gates are significantly lower in gate weight, having a total weight of 3500 t compared to 4400 t for the sector gates and 5100 t for the rolling gates. The miter gates are also significantly lower in total feature construction cost for the gates and associated costs (gate bays, recesses and bridges, and maintenance facility infrastructure) with a total cost of $209,200,000, compared to $350,500,000 for the sector gates and $354,100,000 for the rolling gates. The lower weight of the miter gate would also provide a gate that would also be easier to handle. As expected, the cost of the gates and the recesses for the rolling gate and sector gate are significantly higher because of the recess size in comparison to the extra wall length requirements for the miter gate. The water use for the miter gate is somewhat less (0.6%) than that required for the rolling gate. The sector gate would require 12% more water than for a rolling gate. All three gate types were evaluated against the evaluation criteria stated in Appendix B - Gate Selection Study, and, as shown in the following table, the miter gate has the most favorable rating.

Evaluation Criteria and Rating								
Feature Alternative	First Cost	Ease and Frequency of Maintenance	Risk to Extented Navigation Closure Resulting from Significant Accidents	Access for Crossing Over Lock	History of Reliable Service or Precedent	Compatibility with Water Saving Basin and Filling and Emptying System	Water Conservation	WEIGHTED EVALUATION
Miter Gates	3	3	3	2	3	3	3	2.86
Rolling Gate	1	2	2	3	3	3	3	2.43
Sector Gates	2	1	2	1	1	2	1	1.43
Procedure included ranking the individual alternative by individual criteria to consider relative comparison. 3 is the highest, 1 is the lowest								

Figure 1-2 Evaluation Criteria and Rating

In summary, the miter gate has a significantly lower cost, is about equal in water usage, has a proven reliability, provides the filling and emptying system layout and conduit configuration most compatible with the project, has the lowest risk for operation and maintenance and can be the most easily maintained especially with the ACP experience from the existing Canal locks. The miter gate is the recommended gate type for use in the Atlantic Locks. Operation of the gates would be accomplished using direct-connect hydraulic cylinders similar to those being installed on the existing locks. Three methods for handling the gates for maintenance were developed and presented in the Main Report. A complete discussion on the evaluation of gate alternatives is presented in Appendix B – Gate Section Study.

An emergency closure system capable of closing open channel flow from Gatun Lake through the new Third Lane Locks is recommended as a project feature. This structure would be located upstream of the Gatun Lock Gates. The use of single gate at the Gatun entrance as opposed to double lock gates may be justified as an additional cost saving measure in consideration of the protection offered by an emergency closure system. Cost reduction could be realized in gate fabrication, masonry construction, operating equipment, and reduced length of in-the-wet entrance wall construction.

1.3.2. Locks Structure Alignment

The double-lift lock structure alignment was optimized both longitudinally and transversely through a progressive process of studying various alignments and angled possibilities within the corridor specified in the Scope of Work. It is at the best-fit location to use the geologic stratigraphy of the Gatun site. The location avoids problems with the existence of the Atlantic "muck" while placing the structures on rock foundations to minimize construction costs. The recommended alignment is at an angle of 9.75 degrees from the north navigation channel line into the existing Gatun Locks. As such, it places the locks structure at a near continuous straight line from the Atlantic entrance channel and provides only a minor deviation from the navigation channel entrance in

Gatun Lake. It has only a minor impact on the Gatun Lake mooring facility while maintaining three ship lengths as a straight line for lock entry. The offset of the alignment with the addition of water saving basins provides sufficient distance to the side of the channel entry from the Atlantic Ocean and Gatun Lake to safely manage ships entering and exiting the existing and new locks.

1.3.3. Filling and Emptying System

A number of possible filling and emptying system types were screened to select the most favorable type of system for use in these large locks. Evaluations centered around filling/emptying times with water saving basins, rates of water transfer, ship handling characteristics and hawser loads, compatibility with other project features, cost/constructibility, and integration of water saving basins. Two designs are provided as required by the Scope of Work: an interlaced bottom lateral fill/empty system similar to the existing locks system, and an in chamber longitudinal culvert system (ILCS). The interlaced bottom lateral system utilizes side wall culverts while the ILCS system has the culverts located longitudinally along the lock floor. The systems were designed to provide the required fill/empty time of about 12-15 minutes when equalizing with water saving basins and to minimize impacts for operations without water saving basins and under maintenance conditions. Filling and emptying through the sills of the gates was investigated for this lock and lift configuration but discarded due to maintenance considerations. The interlaced bottom lateral filling and emptying system is similar to the existing locks and its performance is proven. The system provides average equalization times between 10.8 and 12.9 minutes for operations without water saving basins. When using water saving basins, average equalization times are expected to range between 13.4 and 15.1 minutes. Hawser forces would be within reasonable levels throughout the equalization cycle. The ILCS design provides similar equalization times ranging between 9.2 and 11.8 minutes for operations without water saving basins. Equalization times for the bottom longitudinal system with use of water saving basins are expected to range between 13.3 and 14.9 minutes. Hawser forces would be higher with the ILCS compared to the interlaced bottom lateral but would be within acceptable limits. Both systems are expected to perform well for the double-lift configuration and are compatible with other recommended design features. The systems have been designed to operate safely and efficiently with water saving basins during normal operation. The interlaced bottom lateral system will perform well when equalizing without water saving basins and under maintenance conditions. Hydraulic performance could be slightly degraded for the ILCS when operating without water saving basins or under maintenance conditions.

1.3.4. Water Saving Basins

Water saving basins were integrated into the use of the filling/emptying system to provide 50% water savings as required. These consist of two lateral basins located to the west side of the locks for each lock level. The Moffatt & Nichol reports, previously prepared to investigate the potential use of the water saving basins systems, were used as a starting point for the design. For reasons of economy, the basins were integrated into the back of the lock walls. This minimizes lateral space requirements as well as basin conduit excavation. The basins were designed to provide a reasonable and reliable system performance time based on technology in use today. This produced a total fill/empty time with water saving basins of approximately 13-15 minutes, as requested by the ACP. Hydraulic modeling is recommended in future studies to further refine design parameters and operating conditions.

With water saving basins, the third lane locks will be more efficient than the existing locks in their use of water. A summary of projected water requirements is presented in Table 1-1. Both the double- and triple-lift configurations with water saving basins require less water than the existing locks for each container shipped.

Table 1-1 Comparison of Water Requirements

Configuration	Total Water Used Per Lockage (Liters)	Average Water Used (Liters Per Container Shipped)	Increase/Decrease Compared to Existing Gatun Locks
Existing Gatun Locks	104 Million	23,100	Baseline
Double-Lift Without WSBs	418 Million	33,500	+45%
Double-Lift With WSBs	209 Million	16,750	-27%
Triple-Lift Without WSBs	280 Million	22,400	-3%
Triple-Lift With WSBs	140 Million	11,200	-52%

1.3.5. Lock Walls

The lock walls have been designed considering the related features, with space provided for locomotives as the ship-positioning system, and provide the most economical solution for the integration of the project features. The water saving basins are located on the west side of the lock and are integrated spatially (non-structural) into the back of the walls. The valve operation monoliths are incorporated into the walls and basins. The lock walls include the lock filling/emptying culverts and are gravity monoliths founded on a rock foundation. The monoliths utilize a stub toe at the base to increase stability, to reduce bearing pressures and to eliminate the need for a concrete bearing base slab. Roller compacted concrete is used extensively for economy of construction of the lock walls; conventional concrete is used in the areas of culverts, galleries and the lock chamber face.

Table 1-2 Summary of Concrete Volumes[1]

Filling System	Roller-Compacted Concrete	Cast in Place Concrete	Total
Interlaced Bottom Lateral	1,117,000 m^3	852,000 m^3	1,969,000 m^3
In Chamber Longitudinal Culvert	1,870,000 m^3	562,000 m^3	2,432,000 m^3

1 – Includes lock chamber walls, gate monoliths, floor paving, and gate sills

Applicable loads were applied with the controlling loads being for the dewatered condition and the earthquake event. Space has been allowed at the top of walls to include a ship positioning system similar to the existing locks. The lock gate monoliths would be constructed as conventional gravity structures founded on a rock foundation. Approximately 40% of this construction would be of cast-in-place construction.

1.3.6. Entrance Walls

These walls present a special condition because of the size of the impact loads that would be transmitted by the large Post-Panamax ships. Loads developed based on discussion with the Canal pilots were considered and found to be excessively high. Criteria for ship impact was developed based on consultation with fender manufactures and PIANC design recommendations. However, a detailed investigation should be performed before final design to define the necessity and size of these loads. The wall length represents one and one-half ship length as requested by the Canal Capacity Projects Office. The Atlantic entrance walls would be constructed of roller compacted concrete and the Gatun entrance walls would be cast-in-place concrete cap walls founded on a drilled shaft foundations and would be constructed for the most part in the wet using specialized construction techniques. If tugs are used to manage the ships in lock entry, these walls could be reduced in length and size and have a significant savings in project cost.

1.3.7. Seismic Design

In selecting the design level for the earthquake event, existing reports, seismicity of the region, and studies for the existing lock system were examined and analyzed. It was determined that a mean value of 0.31g as the maximum credible earthquake (MCE) would be used for this concept level design. It has a return period of between 300 and 1,000 years. Additional studies are currently being conducted by the USGS and this value should be reevaluated upon completion of their studies, which are due at the end of 2002. More sophisticated analyses need to be conducted to verify performance under seismic events following Corps standard practices for seismically active regions.

1.4. Recommendations

In proceeding with this design process, it is recommended that Feature Design Memorandum Reports be prepared concurrently with physical modeling for the major features of work before proceeding into the design-for-construction and preparation of the construction plans and specifications. This report would provide a final evaluation of the options available for the specific site conditions, select the most appropriate features, optimize the design, and establish the design parameters. Using this document, final construction plans and specifications would be prepared. While the concept level design report presents the design for various features for the specific site, it does not optimize the design nor consider possible ongoing changes. Changes could include the method of handling the ships that would change the dimensions of the locks and reduce wall loads by eliminating the loads transmitted to the walls with the use of locomotives. Selection of emergency closure systems should be considered with development of any project specific security considerations. Also, decisions need to be made on methods of maintaining the locks and gates, and these could change certain presented designs of the features. Life cycle economic cost analysis should be performed to assess viability of purchasing a high capacity gate lifter crane to serve in emergency response scenarios and support other canal system maintenance activities. The use of a single lock gate at the Gatun Entrance should be re-evaluated in conjunction with an emergency closure system as a potential cost saving measure.

1.5. Opportunities

Other decisions that need to be made relate to funding flows and opportunities for economies of construction. If tugboats are used to handle the ships into and through the locks, the entrance walls could be reduced in length or eliminated to using short stub walls similar to those commonly used in large locks in Europe. Advance work that could be taken out of the main construction contract could include separate purchase of the lock gates (cost of $120,000,000) and/or performing the in-the-dry lock excavation (cost of $82,000,000) first as self-funded items of work. This would delay the date that borrowed money is started and reduce the period of accruing interest, thus lowering the investment cost. Another item for consideration involves the timing for providing the water saving basins. This is a costly item at $72,000,000 and may not be needed until some point in the future, after the locks are opened. Provisions could be made for including them into the lock construction but they would not be constructed until after the lock is operationally producing income and they are needed. All of these items relate to lowering the investment. Also, seismic design parameters need to be standardized and sized in accordance to the risk level acceptable to the ACP considering the associated costs for each level of design earthquake. These designs need to be prepared in order to evaluate risk and ACP needs to participate in these discussions to balance level of risk with the associated cost.

2. INTRODUCTION

This Concept Design study represents one sub-set of efforts by the Panama Canal Authority (ACP) to supplement capacity of the existing Panama Canal system.

The 82.5-km-long Panama Canal has operated since its opening to shipping in 1914, using two lanes for ships to travel between the Atlantic and Pacific Oceans. During the past 88 years, three developments have brought the Canal to its present near-capacity condition.

1. Traffic volume has likewise steadily increased, nearing canal capacity.
2. The existing water supply has become insufficient in dry years to reliably accommodate the number of lockages each year.
3. The vessel size has grown steadily and surpassed the maximum size able to pass the current locks, known as Panamax.

Routine maintenance on the nearly nine-decade old lock structures forces closures that result in traffic delays. Studies are underway to determine the most effective and efficient measures to address these concerns. The 13.7 km Gaillard Cut is being widened from 152.4 m to a maximum of 222.5 m to allow two-way passage of the larger Panamax-size ships, and to decrease the greater than 30-hour total Canal water time to provide better customer service. With the completion of the cut-widening project, the entire reach of the canal will now accommodate two-way traffic. This provides an increase in capacity, and outages, whether planned or unplanned, have a lesser impact on Canal service. As demand increases, the Canal risks losing traffic to other ocean routes or alternative means of ground-based, cross-continental, transportation. Due to these developments, the ACP created the Canal Capacity Projects Office (IPC) to oversee canal capacity studies. The IPC began studying expansion of Canal capacity with a reconnaissance study to recommend location and capacity of potential Canal water supply projects or improved management of Canal resources. Additional water supply is a key factor to Canal expansion since each passage of a ship through the canal discharges approximately 208 200 m^3 of fresh water into the sea. Current studies show that water supply will soon become the limiting factor in increasing canal capacity.

This report presents the general concept level design for a Double-Lift Lock on the Atlantic Ocean side of the Canal with water saving basins having a 50% savings of the water used in performing a lockage. The Third Lane would accommodate the next generation of ships, known as post-Panamax and include provisions for ships that have not yet been designed. This would provide room for ship growth without near- to mid-term restrictions. This double-lift lock structure is designed to a concept level for the purpose of establishing basic configuration layout and project costing. Two alternate filling and emptying systems are presented for this lock configuration. The concept design is based on the design criteria established and contained within an earlier report entitled "Project Design Criteria", dated 12 April 2002, expanded and completed and described in Appendix A - Project Design Criteria.

3. WORK PLAN AND QUALITY CONTROL PLAN

3.1. Work Plan

The Work Plan for the concept design Task Order consists of a detailed schedule prepared using Primavera Systems Inc. Software. The schedule has time lines, durations, milestones and costs for all activities. Resources and staffing are listed in a tabular form on a spreadsheet and are summarized in a Primavera Project Planner report. The spreadsheet contains coded resources (with index), hours, and subtotals for all activities. The Work Plan integrates work plans from all engineering elements. The principal technical elements included are Structural, Geotechnical, Concrete Materials, Civil, Hydraulics, Electrical and Mechanical, CADD, Geographic Information System (GIS) and Cost Engineering. A detailed staffing plan is also provided that identifies key personnel, their qualifications, and their roles in execution of the work. The schedule and resourcing and staffing spreadsheet are included in Appendix I, Work Plan.

3.2. Quality Control Plan.

3.2.1. General

The Quality Control Plan (QCP) for the concept design Task Order details the procedures for reviewing and checking the contents of the products to be provided the ACP under this proposal. In general, the QCP includes coverage in the following areas:

a. Management Philosophy related to Quality Control.

b. Quality Control Structure

c. Proposed Quality Control Approach

d Quality Management Procedures

e Staff Qualifications

f. Procedures for Quality Control Documentation

g. Quality Control Scheduling (i.e. order of reviews, checks, etc.)

h Procedures for communications, including internal communication, communication with consultants, and communication with the ACP

The Quality Control Plan is included in Appendix P, Quality Control Plan

3.2.2. Quality Control Organization

The Quality Control organization consists of an Independent Technical Review (ITR) and a Management Oversight/Quality Assurance (MO/QA) team. The ITR team consists of personnel from Corps of Engineers Districts and Divisions who have expertise in the various technical specialties involved. The members of the Independent Technical Review Team are shown in Table 2 of Appendix P, Quality Control Plan. Resumes of the Independent Technical Review Team members are included in the Appendix O, Work Plan.

The MO/QA team consists of three technical matter experts from Corps of Engineers divisions and Headquarters. The purpose of this team is to provide quality assurance review of the quality control process. This team has access to all engineering products, review comments, and responses throughout the design process to ensure that the quality control process is functioning as planned and that the requirements of the ACP are met during all phases of work.

3.2.3. Quality Control Process

The Quality Control process consists of design checks, independent technical review and quality assurance reviews. Frequent discussions between discipline lead designers and ITR members via email and telephone was used at significant decision points to resolve potential study issues in a timely manner.

The Quality Control process utilizes a seamless ITR process concurrent with the design process throughout the project duration. During the development process, engineering products are periodically posted to a secure web site (Bentley Viecon), where they are available for review by the members of the ITR and MO/QA teams. ITR and MO/QA team members access the web site at regular intervals and review and comment on the designs as they are developed. The seamless review utilizes e-mail for communication between the design team and the reviewers. At specific intervals throughout the design process, the documents will be furnished to the reviewers for a formal review, at which time the comments from reviewers and responses from designers will be entered into the Dr. Checks system. Dr. Checks is a web-based database system designed specifically for quality review documentation. The review documentation is included in Appendix Q, Quality Control Certification and Documentation

4. DESIGN CRITERIA

The concept design is being performed in accordance with Corps of Engineers criteria contained in engineering regulations, manuals, and other guidance documents, commercial standards and ACP requirements. The technical criteria is outlined in the Design Criteria Report that is included in Appendix A, Project Design Criteria. The following is a summary of the final design parameters:

Table 4-1 Summary of Final Design Parameters

	Parameter	Dimension	Comment/Source
GENERAL	Controlling Datum	PLD	Scope of Work
DESIGN VESSEL	Vessel Dimensions Panamax Post-Panamax Container Bulk Carrier	Length x Draft x Beam 294 m x 12.0 m x 32.3 m 385.7 m x 15.2 m x 54.9 m 365.8 m x 15.2 m x 56.4 m	ACP Correspondence
	Design Vessel	385.7 m x 15.2 m x 54.9 m	Scope of Work
NAVIGATION REQUIREMENTS	Entrance Channel Bottom Width	137.5 m 2.5 beam width minimum from face of entrance wall 140.5 m provided (1/2 lock chamber width plus 2-55 m beam widths m)	Confirmed at Jan. 2003 ACP Coordination Meeting
	Channel Depth	18.29 m	Scope of Work
	Gatun Lake Controlling Water Elevation	23.93 m PLD (min.) 25.91 m PLD (mean) 26.67 m PLD (max)	Scope of Work
	Atlantic Entrance Controlling Water Elevation	-0.38 m PLD (Low Tide) +0.56 m PLD (High Tide)	Scope of Work

	Parameter	Dimension	Comment/Source
	Atlantic Entrance Channel Bottom Elevation	-18.67 m PLD	
CONSTRUCTION SLOPES	Atlantic Muck Random Fill Gatun Overburden Rock	1v:8h 1v:6h > 10 m 1v:4h < 10 m 1v:2h 8v:1h	Developed by COE and Supplemented by PCC Studies
DOUBLE-LIFT LOCK	Lock Dimensions (Design Lock) Between Inner Lock Gates Maximum Usable Lock	Length x Depth over sill x Width 426.8 m x 18.3 m x 61 m 457.3 m x 18.3 m x 61 m	Scope of Work
WATER SAVING BASINS	Target	50%	Scope of Work
	Location	One Side Adjacent to Existing Locks	Scope of Work
FILLING AND EMPTYING SYSTEM	Filling and Emptying Time	12-15 min. with WSB	Coordination with ACP & Scope of Work
	Hawser Force	Existing locks and international criteria	Scope of Work

5. PROJECT FEATURES

5.1. Lock Siting And Layout

5.1.1. Alignment Optimization

A Draft Alignment Optimization Report, dated June 14, 2002, was submitted to the ACP for comments. The Draft Report was identified as an "Interim Report" and presented an array of alternatives based on project feature development at that time. The Draft Report was a "first cut" at alignment optimization. The principal alignment was identified by the Canal Capacity Projects Office, based on prior alignment evaluations. Based on ACP guidance and examination of several screening alternatives, the Draft Report made the recommendation to proceed with project development, based on construction along the 9-degree Baseline.

ACP comments were received and incorporated to guide the development of this Report. The Final Report addressing the Double-Lift Lock Alignment is included at Appendix D, Alignment Optimization Report. The Final Report finalizes the screening process and makes a recommendation, based on the criteria contained within the Scope of Work, and all available ACP-supplied data.

Alignment optimization is an iterative process, dynamic in nature, which changes as feature refinement becomes available. Navigational safety is the primary factor driving lock alignment. All prudent alternatives must elevate the safety of traversing vessels as the key driver in alignment decision-making. Ship collision risks are not tolerable.

5.1.2. Previous Studies

The Harza Report ("Evaluation of Lock Channel Alignments", August 2000) initiated the alignment optimization process. Twenty-four Third Lane alignment alternatives, some on the Atlantic side and some on the Pacific side, were evaluated, scored and ranked. The recommended alignments were the highest ranked, based on economics (excavation costs), navigational safety and lockage requirements, navigation channel characteristics and operational characteristics. The Third Lane alignments were planned assuming one-way traffic for the approach channels. The assumption was made that approach widening could be implemented at a future date, as traffic demand reached and exceeded capacity, to allow two-way traffic. Overall rankings for the Atlantic entrance (Harza Report, Section 5.0 "Conclusions", page 20) listed eight alternatives, three of which warrant further evaluation. Two of these were positioned on the east side of Gatun Locks. Alignments A1 and A2 were ranked the two top Atlantic entrance alignments, because they combine good operational qualities and ease of construction. Alignment A-1 is positioned along the 1939 excavation and Alignment A-2 is positioned to the east and immediately adjacent to the Gatun Lock east wall.

5.1.3. Optimization Process

Optimization of the lock alignment began by "building upon" Harza's A-2 alignment. Given the growing volume of traffic and delay time as vessels await lock entry permissions, maintaining Gatun lock operations is essential. Even though Alignment A-2 was one of the two recommended Harza alignments east of the Gatun Locks, it is not the optimum alignment because it interferes with Gatun lock operations. Water savings is also a crucial element in lock alignment optimization since geometric and excavation requirements of the water saving basins must be considered. Atlantic muck is prevalent along the eastern edge of the Atlantic entrance. Material properties of the muck require an extremely flat cut slope of 8:1. Positioning the water saving basins between the existing Gatun Locks and the proposed Third Lane locks, however, presents some challenges. Basin geometric requirements forces the A-2 alignment to "push" eastward, allowing for basin construction and avoiding excavation adjacent to the Gatun Lock east wall, especially along the northern end. Any unloading of this east wall could introduce east wall stability problems, thereby negating benefits of such alignment. Alignment A-2' (A-2 prime) was optimized/developed, to take advantage of utilizing the existing Middle wall PI as the rotation point. This alignment accrued several benefits:

1. Gatun Locks could safely remain fully operational for all phases of construction as the new lock approaches would be at a sufficient distance to safely manage the vessels.

2. Alignment rotation counter-clockwise from the existing locks improves ship entry/exit by reducing the turn angle to 2.2 degrees.

4. Alignment A-2' positions the Third Lane corridor primarily within the Gatun Lock land mass cut such that cofferdam cells, needed for the entrance wall construction can be positioned without encroaching on the shipping lane approach.

The Gatun site terrain was modeled with use of InRoads software, a state-of-the-art, commercially-available, MicroStation-based, civil engineering software package (SelectCAD V8.2). InRoads is designed for civil/site development applications, although it has many other capabilities. Modeling runs produce excavation volume reports given input parameters. The ACP furnished a digital terrain model (DTM) of the existing Gatun Locks site topography, extending from the Atlantic approach to Gatun Lake and extending from the existing locks eastward across the 1939 excavated areas and up the hillside to Gatun. This DTM established the three-dimensional (3-D) topography surface of the Gatun site, and was imported into InRoads. Core boring logs were also provided by the ACP, from which coordinates and elevations of the rock layer were taken to create a digital rock profile (rock DTM). The rock DTM established the 3-D "top of rock" surface along the Gatun site, and was also imported as an ASCII file into InRoads. This top of rock surface is the surface, below which rock removal techniques would be required for excavation. Once these 3-D surfaces were imported (defined), the modeling process began. Site modeling was an iterative process to establish horizontal and vertical alignments along the site topography, to establish a "best fit" between the rock line and the proposed Third Lane construction. The InRoads software stored both DTM's, and was used for all corridor-modeling runs. Horizontal alignments, vertical profiles and templates were defined and identified for each modeling run. Templates were defined for each change in cross-section. As the lock wall design advanced, establishing footing widths and thicknesses for differing monolith loadings (gate monoliths, valve monoliths, typical interior monoliths, entrance walls, etc), additional templates were defined to capture geometric changes in excavation limits (widths and elevations). Once the alignments and numerous templates were defined, the software was used to "push" each successive template along the profile, from the beginning to ending station of each geometrically defined template, along the alignment. Pushing the template from point of beginning to end, along the defined vertical alignment (profile) constitutes a modeling run for that defined corridor. For each corridor-modeling run, the software generated plan views, profiles and cross-sections for the defined range of stations, at the defined interval (100 m). Reports were created to depict excavation and embankment volumes, by station, that distinguish between material layers of rock and overburden.

Corridor-modeling was considered to be complete when the lock footprint was positioned along the alignment such that the "best fit" position of the lock had been determined. Modeling site topography to optimize lock alignment required engineering judgment in conjunction with trial-and-error processing in order to weigh all variables that contributed to this decision. Excavation requirements for the rock and overburden cuts contribute significantly to the economics of making this Third Lane cut, however, excavation volumes were only one facet of this equation. Site conditions; type, size and locations of the water saving basins and filling and emptying culverts; gate type; lock wall type and dimensions; location of competent rock integrated with the lock wall foundation loading and design; overburden variations; and other factors play an equally important role in the decision-making process for screening alternatives and arriving at the most prudent and desired alternative. Close coordination with the ACP has been an integral part of the optimization process as project development advances. Once the mathematical model was created within InRoads, excavation volumes for both rock and overburden were generated. Excavation quantity accuracy is limited only by the accuracy of the input data. One-meter topography was used to create the existing ground DTM. Rock location was defined by the subsurface core borings taken, both the pre-World War II era data along the 1939 cut, and more recent data adjacent to the existing locks.

5.1.4. Recommendation

This Alignment Optimization Report, Appendix D, is a refinement of the Draft Report. The optimization study was based on the Harza Report alternatives, available geotechnical data, surveying and mapping data, topographic data, and input from the ACP and all Production, ITR, and Oversight Committee Teams. Several alignment options were screened and a preliminary recommendation was made based on the various InRoads modeling runs made to date. The 9.75-degree baseline corridor appears to be the best overall horizontal alignment option, considering the above criteria. This recommendation is a change from the 9-degree Baseline recommended in the Draft Report.

Although several factors drove the need for this alignment shift, the primary factors were entrance wall geometry (navigation safety), cofferdam layout requirements, and determination of the water saving basins geometry and lock wall footing locations and dimensions. The recommended Double-Lift Site Plan is shown in Figure 5.1-1.

The Alignment Optimization Report, Appendix D, includes the InRoads-generated horizontal and vertical alignment and cross-sections for the 9.75-degree alignment. Although horizontal alignments may involve tangent or curved sections, curvature in the alignment is undesirable for navigation traffic. Ships must be able to traverse both the existing Gatun Locks and the proposed Third Lane, once available. The alignment of the existing Gatun Middle Wall is along an azimuth of approximately 187 degrees. The Atlantic shipping lane is aligned due north and south so that southbound traffic entering the north approach to Gatun Locks would be traveling along an azimuth of 180 degrees. Entry into Gatun Locks from the Atlantic approach requires traffic to make westward turn of approximately 7 degrees (187 – 180 degrees). Using the Point of Intersection as

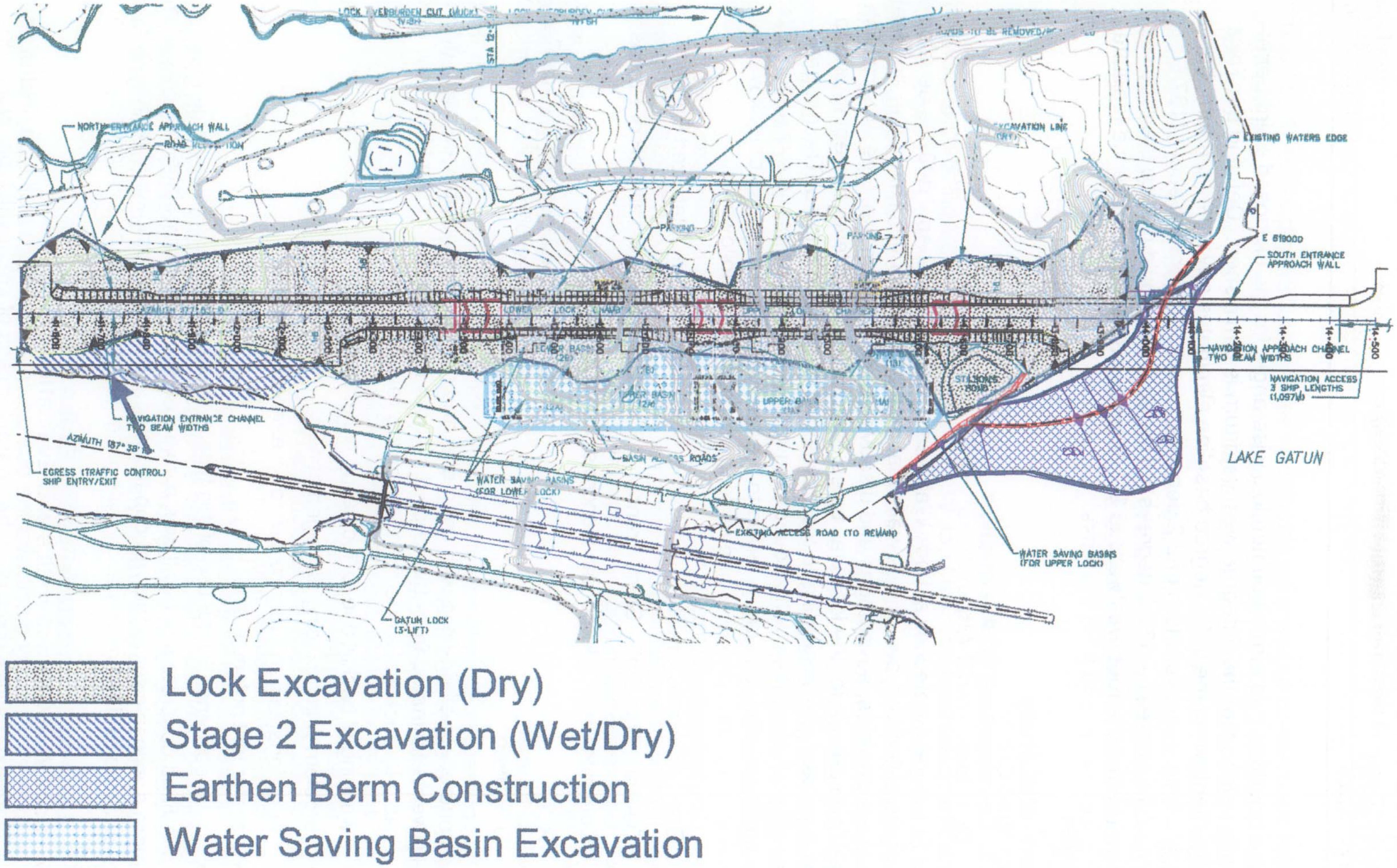

Figure 5.1-1 Proposed Double-Lift Lock Site

the point about which to turn a counterclockwise angle for the proposed Third Lane baseline is helpful that the 7 degree westward turn is diminished. Defining the Third Lane alignment along the 9.75- degree baseline eliminates any westward turn, but creates a smaller, eastward turn of 2.2 degrees, measured from the south (187.6 degrees – 9.75 degrees – 177.9 degrees). This baseline would reduce the required traffic turn (from 7 degrees westward to 2.2 degrees eastward), and hence facilitate traffic movements to and from the locks.

5.2. Lock Walls

5.2.1. Lock Wall Study

5.2.1.1. Development Of Alternatives

The Structural/Geotechnical Lock Wall Team met on 17-19 April 2002 to develop a list of the potential types of lock walls that may be used for the Third Lane, to identify the characteristics, advantages and disadvantages for each wall type, to establish evaluation criteria for the lock wall types, to conduct a first level screening and to rank the wall types according to the developed evaluation criteria. The material strength and properties of the foundation rock were not part of the evaluation criteria; this type of evaluation was deferred to the study phase. The types of lock walls developed for the screening and ranking process are as follows:

- Gravity/Perched Gravity
- Tie-Back Thin Wall – Slurry Trench
- Concrete Shell In-Filled with Concrete
- RCC
- U-Framed
- Mechanically Stabilized Earth
- Concrete Bin Wall (CIP or Precast)
- Counterfort Wall
- Grout Stone Mass w/CIP Face/Recycle Stone Excavation (Option w/ Precast Face)

The concrete shell (in-filled with concrete) and mechanically stabilized earth (MSE) walls were eliminated from the study during an initial screening. An MSE wall is not applicable for lock walls due to the following reasons:

- washing of backfill due to fluctuating water levels
- problem with replacing damaged panels
- questionable performance during an earthquake event
- settlement potential, and
- no precedence for the use of this type of wall for locks

The in-filled concrete shell wall was eliminated from the study since this type of wall is more applicable for in-the-wet construction. The roller compacted concrete (RCC) wall was reclassified as a gravity wall since it is relevant to construction type, not wall type.

5.2.1.2. Lock Wall Characteristics and Evaluation

The team extensively discussed the characteristics of each wall type, including constructibility and performance issues, as well as their advantages and disadvantages and precedence for use as a lock wall. The advantages and disadvantages for each wall type are provided in Table 5.2-1.

After the advantages and disadvantages were established for each wall type, the team developed the following criteria in order to evaluate and rank the wall types.

- Impact to Navigation due to O&M
- Reliability/Service Life
- Compatible with Soil/Geology
- Compatible with F&E
- Compatible with Ship Positioning System
- Compatible with Gates
- Compatible with WSB
- Construction Duration
- Construction Cost

The use of permanent rock anchors to stabilize the lock walls for both static and seismic design load cases was discussed. Although rock anchors are used in numerous Corps projects, several concerns were raised in their use for stabilizing the lock walls. The first was the requirement of long-term monitoring of the rock anchors during the life of the structure. Ideally, the rock anchors for this application should be fitted with re-stressable anchor heads to the anchor load to be measured by lift-off checking. The rock anchor loads may be adjusted with the use of shims. As a result, each anchor would require an access panel and a recess with proper clearances for a hydraulic ram, jacking chair and a stressing coupler assembly. The second concern was the long-term corrosion protection of the rock anchors. The use of epoxy-coated strand, corrugated sheath and grout would provide adequate protection, but particular attention and detail would be required at the anchorage to prevent corrosion. The last concern was the need to temporarily close the lock to conduct the periodic lift-off tests for anchor force verification. The time period to close the lock and conduct lift-off tests on all of the rock anchors could take a significant amount of time, considering the number of rock anchors that would be required.

Table 5.2-1 Wall Type Advantages/Disadvantages.

Gravity/Perched Gravity	
Advantages	Disadvantages
• Dependability • Longevity/Maintenance Free • Support Large Vertical Load • Simple Structure • Simple Design • Similar to Existing • QC Less Complicated	• Large Amount of Excavated Material (Except Perched) • Slow to Construct • Heat Generation/Thermal Cracking Similar to Conventional Gravity Wall

RCC	
Advantages	**Disadvantages**
• Similar to Conventional Gravity Wall • Reduced Cost • Faster Construction • Can be Easier to Construct • Facing Can be Precast • Methods of Placement	• Material Demands More Strict • Complicates Wall Culvert Placement • Frequent Rains Cause Paste Erosion/Strength Loss • Embedded Items Are Complicated • Two Batch Plants Are Required
Tie-Back Thin Wall	
Advantages	**Disadvantages**
• Cost Effective for High Rock Excavation • Minimizes Soil and Rock Excavation • Large Number of Methods of Construction • Minimizes Concrete Volumes	• Joint Leakage • Long-Term Monitoring Required of Rock Anchors to Ensure Safety • Complicated Design/Seismic • Anchor Proof Tests & QC More Extensive • Design Life Approximates Service Life • Anchor Corrosion • Adaptable to Fewer Rock Types • Specialty Contractors
U-Frame Walls	
Advantages	**Disadvantages**
• Applicable for MG Monoliths • Applicable for High-Seismic Areas • Applicable for Poor Foundations & Piles • U-Frame construction provided sufficient strength and deflection control to allow future adjacent excavation for a 4th Lane Lock	• Inefficient Design for this Application • Extensive, Deep Excavation • Expensive
Counterfort Walls	
Advantages	**Disadvantages**
• Minimize Concrete Material • Efficient Utilization of Concrete and Reinforcing for Certain Heights • Applicable for Significant Portion of Lock Walls	• Extensive Formwork • Compatibility Problems Within the Wall Culverts and Ship Positioning System • Extensive Excavation • Complicates Backfilling • Low Structural Internal Redundancy • Significant Amount of Reinforcement • Higher Design Effort

Concrete Bin Wall	
Advantages	Disadvantages
• Cheaper Fill Possible – Excavated Rock/Grout • Fast to Build • Less Concrete • Non-Linear Construction Dependencies • Rapid Prefabrication and Fast Track	• Specialized Construction • Still Require CIP for Recesses, Voids, and Reinforced Cap for Line Pull

During the meeting with ACP personnel on 19 April 2002, the use of rock anchors to stabilize the walls and to reduce the concrete and excavation quantities was discussed. After a lengthy discussion of the aforementioned concerns, the general consensus of the Design and ITR Team and ACP personnel was to eliminate rock anchors as a primary structural element in the stability of the lock monoliths.

5.2.1.3. Lock Wall Evaluation and Ranking

The team assigned a weight for each of the evaluation criterion according to degree of importance. Each wall type was then evaluated for each criterion on a scale from 1 to 10. The results of the evaluation are provided in Table 5.2-2.

Based on the results of the evaluation, three lock wall types recommended for further study were the conventional gravity wall, the perched gravity wall and the RCC gravity wall with cast-in-place concrete face. The perched gravity wall was eliminated from further study due to the following reasons:

1) The COE Geology Section and the Seismic Analysis Team raised concern that the weak tuffaceous sandstone unit would fail during a seismic event,
2) Long-term stability of the tuffaceuos sandstone in the vicinity where the cast-in-place concrete facing (rock cut subjacent to monolith base requires facing to prevent deterioration of rock) is anchored to weak rock,
3) The rock bench does not have the bearing strength and the shear strength to support the monoliths during a seismic event, and
4) The low bearing capacity and shear strength of the tuffaceous sandstone would necessitate additional rock support for an earthquake.

Table 5.2-2 Lock Wall Evaluation and Weights

Feature Alternative	Impacts to Navigation for Maint/Repair	Reliability/Service Life	Compatibility with Soil/Geology	Compatibility with thru Sill F&E	Compatibility with Wall F&E	Compatibility with Ship Positioning System	Compatibility with Gates	Compatibility with WSB	Construction Duration	Construction Cost	Accommodate 4th Lane	Weighted Evaluation
Weighting Factor	0.1	0.1	0.1	0.05	0.05	0.1	0.05	0.05	0.1	0.3	0.05	1.00
Gravity Wall:												
Conventional Gravity Wall	9	9	9	9	8	9	9	8	5	5	8	**7.00**
Concrete Bin Wall w/ grouted recycled stone	5	5	8	8	2	6	5	8	7	8	6	**6.15**
Perched Gravity Wall (3)	8	9	8	9	3	8	7	8	8	9	8	**7.75**
RCC Gravity Wall w/ CIP Face	8	7	8	9	4	9	5	6	9	9	8	**7.60**
RCC Gravity Wall w/ Precast Face	2	5	8	9	4	9	5	6	9	9	8	**6.80**
Grout Stone Mass w/ CIP Face	4	5	8	8	4	8	4	6	8	9	6	**6.55**
Grout Stone Mass w/ Precast Face	2	4	8	8	4	8	4	6	9	9	6	**6.35**
Counterfort Wall	7	7	6	9	4	6	7	5	7	5	2	**5.75**
Anchored Wall (thin wall)	2	4	6	9	4	6	7	5	5	6	2	**5.00**
U-Framed (2)	9	9	8	7	8	9	9	8	5	5	8	**6.80**

Notes:
1: The consensus of the team was that the lockwall placement method should be consistent with in-the-dry methods of construction. This is within a dewatered cofferdam or other means.
2: U-Frame compatible with Miter Gate monoliths.
3: Perched Gravity Wall may be constructed with RCC.

Gravity lock wall monoliths shall be used for the Third Lane Lock as a result of their durability, reliability and compatibility with the culverts, conduits, galleries, and recesses required for the locks. To capture economy offered by a typical perched wall, the walls would be partially stepped (perched) to reduce excavation and concrete costs while satisfying concerns mentioned above. Typical wall sections are shown on report drawing ACP-20/14 and ACP-20/15. The lock walls will be constructed using roller-compacted concrete (RCC) with cast-in-place concrete used for the chamber face and in areas of the culverts, galleries, laterals, and concrete reinforcement.

5.2.2. Gravity Wall Design

5.2.2.1. Design Considerations

The top of wall elevation varies with the lift requirement, gate alternative based on the required structural depth of bridges provided over the gates for ship positioning equipment and clearance and access to operating equipment. The width of the wall varies to accommodate access to machinery and valves, the ship positioning system and recess openings for the gates, valves, galleries, electrical equipment and machinery. The wall geometry and gate spacing are also based on consideration of operating mechanisms, gate anchorage, and minimum wall thickness adjacent to filling and emptying culverts, filling and emptying conduits, and water saving basin conduits.

The concept and preliminary design and analysis of lock wall monoliths are based on the load cases and stability requirements prescribed in the Design Criteria Report, loadings (dead, live, and seismic) and operational and structural performance requirements, and dead, live and seismic loading. Essential requirements for operational and structural performance of the lock wall monoliths include:

- Limiting global displacements and deflections to a tolerable range.
- Minimizing interference with the lock's inlet opening.
- Providing for the ease and economy of construction and maintenance.
- Economy of structural and functional monitoring and inspections.
- Durability of materials and hydraulic performance of the structure.
- Providing for an appropriate level of ultimate structural strength exceeding the factored capacity-demand.
- Ability to survive unusual and extreme events such as OBE, MDE, and impact by a vessel.

The design of the gravity lock wall monoliths is predominately based on the stability analyses that are conducted to assure that the hydraulic concrete structures meet minimum safety and performance requirements. The overall design and stability of the structures is assessed in terms of overturning, sliding and bearing. The objective of a stability analysis is to maintain horizontal, vertical and rotational equilibrium of the structure. Geotechnical information is established for the rock and engineering backfill and is provided in Table 5.2-3. Possible failure modes and planes of weakness are determined from onsite conditions, material strengths and uplift forces. Stability is ensured by providing:

1) adequate factor of safety against sliding at all possible failure planes

2) specific limitations on the magnitude of the foundation bearing pressure, and
3) constraints on the permissible location of the resultant force on any plane.

Table 5.2-3 Geotechnical Properties.

Material	Average Density		Shear Strength		Bearing Capacity Q_b kPa
	γ_{moist} kN/m^3	γ_{sat} kN/m^3	ϕ deg	c kPa	
Structural Backfill	19.0	20.0	35	N/A	N/A
Random Rockfill	18.0	19.0	33	N/A	N/A
Random Backfill	15.0	17.0	31	N/A	N/A
Tuffaceous Sandstone	18.6	18.6	31	69.0	1915

5.2.2.2. Seismic Loadings

The criticality of the new Third Lane dictates the need for very severe performance criteria under seismic loading. The current USACE regulation (ER 1110-2-1806) establishes two levels of seismic loadings and corresponding performance conditions: Maximum Design Earthquake (MDE) and Operating Basis Earthquake (OBE). The MDE represents the maximum level of ground motion for which a structure must be designed to perform without catastrophic failure, although severe damage or economic loss may be tolerated. However, the new Third Lane differs from the typical Corps experience with locks since it is categorized as a critical project from an economic point of view. Because of the exceptional circumstances around this project, loss of operation for an extended period of time is economically equivalent to catastrophic structural failure. Therefore, the MDE level of seismic demand is set

equal to the corresponding Maximum Credible Earthquake (MCE) level for those project components and subsystems that are essential for continuity of operation. For critical features, the MDE is set equal to the MCE, defined as the greatest earthquake that can be generated by a specific source. The OBE represents an earthquake that can reasonably be expected to occur within the service life of the structure, which is expected to exhibit little or no damage, and no operational interruption. Typically, critical projects are those whose failure during or immediately following an earthquake could result in loss of life.

The seismic analysis team determined an appropriate value of the peak ground acceleration (PGA) to be used in preliminary design and feasibility studies for the MDE and OBE. The values for MCE and OBE selected for preliminary design are

0.313g and 0.14g, respectively. As recommended for the preliminary stability evaluations of existing concrete structures not in contact with earth, a seismic coefficient equal to 2/3 of the PGA is used for this type of concrete structures. In addition, vertical accelerations are considered in seismic stability evaluations. However, for the simplified conservative engineering procedures that are used in preliminary analyses, vertical accelerations are not required in the preliminary calculations. When the seismic engineering evaluations proceed to the final design stage, a more sophisticated modal response- or time history-based engineering procedure will be used to assess the potential for damage to the lock features. The use of elastic design methodologies and a mean plus one standard deviation earthquake event for concept design would result in an over estimation of the required size of structural features for this study. However, investigation of higher seismic loadings associated with a mean plus one standard deviation earthquake would be performed with time-history analysis to assess damage and establish the owners true limits on tolerable damage.

5.2.2.3. Design Approaches: Yielding Versus Non-Yielding Backfill

The design of the lock walls may follow three approaches. The first is to design the walls to be so rigid that the backfill cannot yield and mobilize full active pressure. For this case, the lock walls are designed for the at-rest earth pressures for both static and earthquake conditions. The second case is when a wall deflects sufficiently for the backfill to yield and full active pressure is mobilized. For this case, the lock walls may be designed for active earth pressures for both static and seismic loadings. The third case is when the lock wall (earth retaining structure) is permitted to experience some movement during the earthquake. For this case, the lock wall is designed for at-rest or active earth pressure for the static condition (depending on the flexibility of the lock wall), and for the earthquake condition, the displacement-controlled approach is used (USACE Technical Report ITL No. =92-11). The displacement-controlled approach incorporates wall movements explicitly in the stability analysis of earth retaining structures. The displacement-controlled approach is a procedure for choosing a seismic coefficient based on upon an explicit choice of an allowable permanent displacement.

The displacement-controlled approach was considered initially as a means of reducing the size of the lock walls for the seismic loading condition. However, the use of this procedure requires the lock walls to move during an earthquake. The risk is that the lock wall monoliths may move relative to one another, especially in the area of the miter gates and valve (WSB) monoliths. This relative movement could shear hydraulic lines, bind machinery, and disrupt navigation due to misalignment of the rail for the ship positioning system. As a result, this procedure was not used to design the lock wall monoliths. The monoliths are rigid and are designed to resist at-rest earth pressures for both static and earthquake conditions.

5.2.2.4. Factors of Safety

5.2.2.4.1. Overturning Stability

The following criteria for overturning is used for the new gravity structures:

- Usual loading condition: Resultant within the kern.

- Unusual loading condition: Resultant outside kern but should fall within the middle half.
- Extreme loading condition: Resultant within the base.

5.2.2.4.2. Sliding Stability

The following factors of safety for sliding are used:

- Usual loading condition: FS = 1.7
- Unusual loading condition: FS = 1.3
- Extreme loading condition: FS = 1.1

5.2.2.4.3. Bearing

The allowable bearing capacity of the tuffaceous sandstone was established during the geotechnical investigation. However, the allowable bearing pressures are increased for the following loading conditions:

- Unusual loading condition: 15% increase
- Extreme loading condition: 50% increase

5.2.2.5. Description of Gravity Lock Wall Monoliths

Nine typical gravity-type lock wall monoliths and three miter gate monoliths are proportioned in accordance with the stability criteria and load cases prescribed in the Project Design Criteria (Paragraph 5.2.4 Masonry Features). The monoliths selected for the study are located on the west side and east side of the Third Lane Lock, between stationing STA 13+700 and STA 12+580 (Table 5.2-4). The cross sections for the typical and miter gate monoliths are provided in the Project Drawings, Double-Lift Configuration, Sections. All of the monoliths are founded on rock and are provided with toe stubs. The purposes of providing toe stubs for the monoliths are to increase the stability of the monoliths, to distribute the resultant net vertical forces of the monoliths that would reduce the resultant bearing pressures applied to the weak tuffaceous sandstone (Table 5.2-3). The monoliths for the Third Lane Lock Walls located on the east and west sides are proportioned to function as pure gravity structures and to satisfy all stability requirements for all static and seismic loading conditions. The controlling load case for the design of the lock monoliths is the seismic load (MCE) condition (Load Case 2G). The base of the monoliths and subsequently the weight must be increased for this load case to prevent a bearing failure of the reinforced concrete foundation. The internal stability of the monoliths was also checked at critical sections to determine concrete reinforcement requirements.

5.2.2.6. Shrinkage and Temperature Reinforcement

The shrinkage and temperature steel requirements for structural concrete is based on higher temperature changes in the U.S. than is experienced in Panama. ACI 318M-99 and the COE (EM 1110-2-2104) require the ratio of reinforcement area, A_s, to gross concrete area to be greater than 0.0018 and 0.0028, respectively. The COE places an upper limit on the shrinkage and temperature steel requirement of No. 29 at 300 mm. Figure 6.4 of ACI Manual of Concrete Practice 1998, Part 1, Section 207.2R-95 provides the required steel ratio for design temperature fluctuation and base length for a prescribed allowable crack width equal to 0.33 mm. For an average monolith length (expansion joint spacing) equal to approximately 14 m for the lock wall monolith concept design, the temperature fluctuations corresponding to ACI and COE steel ratio requirements are 16 C and 23 C, respectively. This is significantly higher than the temperature fluctuations expected for Panama that range from 4 C to 10 C. Considering the lower temperature fluctuations for Panama, the steel ratios required from Figure 6.4 (ACI) are 0 to 0.0001. For a monolith length equal to 14 m, the required upper limit for shrinkage and temperature steel reinforcement in each face is 420 mm^2, or a spacing of 300 mm for No. 25 rebar. This deviates from the requirement of EM 1110-2-2104 would have to be finalized during the feasibility or final design phase.

For mass concrete, the COE (EM 1110-2-2602) does not require shrinkage and temperature steel to control cracking. As a result, shrinkage and temperature steel will not be placed in the lock wall faces and surface, except in the areas of culverts, laterals and galleries. According to the requirements of EM 1110-2-2602 and EM 1110-2-2104, No. 29 at 300 mm spacing will be used for the shrinkage and temperature steel requirement in the areas of the culverts, laterals and galleries.

The recommendation is made that during the feasibility or final design phase, a thermal study (NISA) be conducted to verify the required shrinkage and temperature steel for specific concrete mix designs and laboratory tests to establish the thermal and mechanical properties for each mix design.

Table 5.2-4 Stability Criteria and Results

Location	Load Case	Stability Criteria and Results					
		Sliding		Overturning Base Compression		Bearing (MPa)	
		Actual	Required	Actual	Required	Actual	Required
Lock Wall Monoliths							
East & West STA 12+300	2G	1.16	>1.1	24.0%	>0%	2.42	<2.88
East STA 12+580 STA 13+010	2G	1.42	>1.1	30.0%	>0%	2.70	<2.88
West STA 12+582 STA 13+006	2G	1.89	>1.1	34.7%	>0%	2.79	<2.88
East STA 12+950 STA 13+006	2G	1.47	>1.1	41.2%	>0%	2.79	<2.88
West STA 13+460 STA 13+525	2G	2.14	>1.1	76.7%	>0%	1.67	<2.88
East STA 13+460 STA 13+525	2G	1.14	>1.1	38.5%	>0%	2.85	<2.88
West STA 13+100 STA 13+520	2G	1.74	>1.1	30.4%	>0%	2.89	<2.88

Location	Load Case	Stability Criteria and Results					
		Sliding		Overturning Base Compression		Bearing (MPa)	
		Actual	Required	Actual	Required	Actual	Required
East STA 13+100 STA 13+520	2G	1.17	>1.1	35.6%	>0%	2.82	<2.88
East STA 13+700	2G	1.17	>1.1	23.2%	>0%	2.62	<2.88
Miter Gate Monoliths							
East& West Atlantic MG STA 12+513	2G\[1]	1.44	>1.1	53.3%	>0%	2.04	<2.88
East & West Center MG STA 13+015	2G\[1]	1.13	>1.1	38.6%	>0%	2.35	<2.88
East & West Gatun MG STA 13+533	2G	1.17	>1.1	59.8%	>0%	1.18	<2.88
Notes: The maximum design seismic loading is the controlling load case for all lock wall monoliths analyzed for stability. The stability results reflect the size of the gravity wall required to satisfy the stability requirements for Load Case 2G.							

5.2.3. Roller Compacted Concrete Option

5.2.3.1. Introduction

This section of the report proposes the use of Roller Compacted Concrete (RCC) for certain features of the Panama Canal Atlantic Locks. In addition to identifying the location and quantity of RCC to be used it provides a brief description of the RCC process that would be envisioned for the lock structures and addresses key issues in the use of RCC.

The concrete structures are estimated to consist of 3 000 000 m^3 of concrete. There are several structures or portions of structures associated with the Atlantic lock that could be readily constructed with RCC. These are:

East & West Lock Walls	2 144 000 m^3
Lock Chamber Floor	136 000 m^3
Totals	2 280 000 m^3

These listed structures would provide large placement areas that do not include significant obstacles and embedded features. These areas would be large enough to accommodate an RCC placement operation and would proceed at a relatively high production rate. By the proper selection of materials, design features, and production and placement methods, the quality of the RCC would be at least equal to that of the conventional concrete that would otherwise be used for this application. Drawing ACP-R-20/17 shows the plan and sections of locations identified as appropriate for RCC construction.

There are several major advantages that would result from using RCC in the identified locations. The cost of in-place RCC for these structures is estimated to be at least 2/3 of the cost of in-place conventional concrete. RCC can be used without sacrificing the benefits of using conventional cast-in-place concrete. Greater cost reductions would depend on the specific features and requirements. A related advantage would be that the production rate of RCC placement is significantly higher than that of conventional concrete. It is estimated that RCC placement rates of 400-600 m^3 per hour would be a minimum for this project and up to 900 m^3 per hour achievable. This results in very rapid construction of the RCC structures. This could result in a major reduction in construction time and investment cost and an early operational date for the lock.

Because of the nature of RCC materials and mix proportions, the material properties of RCC are often more favorable for mass concrete construction when compared to most conventional mass concrete mixtures, without sacrificing the quality of the structure. The resulting mixture would exhibit less shrinkage and consequently less cracking distress. This would result in fewer joints in the structure. Strength properties of RCC are achieved with less costly mixtures than conventional concrete. This is primarily because RCC mixtures have lower cementitious material contents and higher aggregate proportions.

The successful placement of a major RCC structure requires careful coordination of many processes. Failure of any part of the process can result in work interruptions. More effort in advance of the start of an RCC operation is necessary and close controls are required during the placement operation. Conventional concrete placements, on the other hand, are a series of distinct placements that include numerous interruptions. As a result RCC can achieve high production rates so long as the process can operate at a continuous rate.

Typically RCC placement operations expose a large area of fresh RCC during placement. This is of concern in locations of extreme heat or high or intensive rainfall when exposed RCC can be damaged by those conditions. Conventional concrete construction exposes smaller areas to these damaging conditions. Recent RCC placement procedures have been developed that reduce the area of exposure while maintaining high production rates (See the discussion below on the sloping layer method).

5.2.3.2. Design Approach

The design of the lock structures is practically the same regardless of whether the mass concrete is constructed using RCC or conventional cast-in-place concrete. The selection of some of the features or orientation of some of the features may be changed to increase the benefits of using RCC.

The lock structures consist of many formed vertical faces and stepped surfaces. The construction of these features would require the use of a conventional concrete forming system. The formed surfaces of RCC structures would be constructed to look and perform like conventional concrete surfaces. The process of constructing these surfaces for an RCC structure would be done with either a conventional concrete mixture or an RCC mixture modified with the addition of cementitious grout. Except for possible minor color shade variations, the final concrete surfaces would look and perform identical to the surfaces achieved with conventional concrete.

The double-lift lock system incorporates filling and emptying culverts in the lock wall monoliths. While RCC is advantageous for monoliths that do and do not include the culverts, greater advantages can be realized if the culverts are located below the chamber floor. This allows a greater volume of the monoliths to be constructed of RCC.

5.2.3.3. RCC Features

5.2.3.3.1. RCC Mixture

Materials properties for the RCC and conventional concrete would be based on the testing of aggregate and on subsequent mixture proportioning studies. As done for conventional concrete, aggregate sources would be investigated to assure adequate quality and long-term performance. A wide range of RCC mixtures would be evaluated to assure quality and economy. The following properties are the more common properties to be determined during this testing phase. It is anticipated that a normal range of material properties will be necessary for the lock structures.

- Compressive strength.
- Cohesion.
- Angle of internal friction (∅).
- Tensile strength.
- Unit weight.
- Thermal properties.
- Creep and shrinkage.

5.2.3.3.2. RCC Production Facility

a Aggregate Handling

Because RCC is capable of high production rates, a large percentage of the aggregate would be stockpiled on site to be used for continuous placements on site and in advance of placement. It is not uncommon to require 50-100% of the aggregate to be pre-stockpiled. The actual percentage would depend on the rate and reliability of the aggregate processing and delivery system to supply aggregates as required. The land area required for stockpiles can be significant. Land areas located adjacent to the placement area are suitable for stockpile locations.

Because there would be concurrent stockpiling of RCC and conventional concrete aggregates, aggregates that are jointly used by RCC and conventional concrete would allow a reduction in the number of stockpiles. Aggregate grading adjustments that reduce the number of stockpiles would further reduce stockpile area requirements.

Barge and rail delivery of aggregates is anticipated for this project. The use of a combination of bottom dump hopper cars, side discharge cars, conveyor systems, and wheeled loaders would be necessary to handle the aggregate. Stockpiles would supply the batching and mixing plants using reclaim tunnels, supplemented with direct charging wheeled loaders. An evaluation of the delivery system may provide a basis to reduce the volume of stockpiled material if "just-in-time" delivery processes are reliable for the project.

b RCC Batching/Mixing Plant Requirements

For the rapid and successful construction of the lock walls, the batching and mixing equipment must produce consistent, high quality RCC in the desired quantities. The assumed target minimum required RCC production rate is estimated to be 400 m^3 per hour. However higher production rates may be more appropriate. The actual rate would depend on a detailed evaluation of construction sequences and resulting lift surface exposures. The use of single or multiple plants would be a factor in the final determination of production rates. Some plant configurations are capable of mixing all the RCC, conventional concrete, and other concretes rather than having separate plant operations.

Various types of mixers are suitable for the production of RCC and conventional concrete for this job. Drum mixers are acceptable for conventional concrete. Horizontal shaft mixers, known as pugmills are suitable for both RCC and conventional concrete. Horizontal shaft mixers have become the preferred type of mixing plant for high production RCC. These mixers can be configured as continuous or batch-type systems.

c Mixture Cooling

Cooling systems for concrete are a major feature of concrete plants in hot climates. The use of wet bins, wet belts, air chillers, water chillers, or ice plants would be considered for this project. Detailed evaluation of ambient conditions, material properties, and placing schedules would be necessary to determine the cooling requirements for specific monolith geometry. RCC mixtures tend to have less cooling requirements than conventional concrete because mixtures generate less heat.

5.2.3.3.3. RCC Conveyance

a Conveyance of RCC to Placement Siding

For the long and relatively narrow placement configuration and the multiple concurrent work activities, work site congestion may be a problem. The transportation of RCC from the plant to the placement siding would be done by conveyor, truck haul, or a combination of conveyor system with haul trucks. The most likely system for RCC would be one that can also support the placement of conventional concrete at other locations. The determining factor would be whether those placements will need to be done concurrently with the placement of RCC. Drawing ACP-R-20/20 depicts a system for transport of the RCC from the batch plant directly to the placement site.

b Conveyance of RCC from Placement Siding to Placement Surface

RCC would not be transported onto the RCC placement surface area using trucks or other wheeled vehicles. The impacts of tire damage, contamination, and traffic flow are extremely detrimental to RCC quality. The RCC would be transferred by a conveyor system to the lock surface placement area.

c Conveyance on the RCC Surface

On the placement surface, RCC would likely be transferred to its final placement location either by secondary conveyors, or by end dump trucks.

i. Trucks

Numerous projects have maintained relatively high production rates using trucks to distribute the RCC on the placement area. Such a system can be economical since appropriate trucks are readily available to contractors. However quality impacts from truck traffic must be considered. An evaluation of the placing sequence at the various phases of construction would be necessary to determine if truck operations are practical from space constraints and RCC quality constraints. Because of space limitations, use of trucks for conveyance of all RCC on the placement site would be difficult. Supplemental placing systems may be needed.

ii. Conveyor

Extensive use of conveyor systems for placement of RCC would free up work areas for other construction activities and allow much greater flexibility for movement of equipment and materials. Conveyors would move the concrete on the placement site in a fast, clean and efficient manner with the least amount of rehandling and segregation.

The Table 5.2-5 identifies the range of options available for each of the phases of RCC transportation. Options from columns A, B, and C can be intermixed to form a complete system. Another key factor in selection of various components would be the flexibility of the system for the various RCC and conventional concrete placements.

Table 5.2-5 RCC Transportation Methods

A Transport From Plant To Placement Siding (Staging Area)	B Transport From Placement Siding Onto RCC Placement Surface	C Transport On RCC Surface
Trucks	Stationary conveyor	Truck(s)
Stationary conveyor	Conveyor system	Conveyor system
Partial conveyor to trucks	Mobile conveyor	Mobile conveyor
Trucks to partial conveyor		

5.2.3.3.4. Spreading RCC

RCC is typically spread in single layers compacted to a 0.3 m thickness. Spreading is done using a tracked dozer. The size and number of dozers depends on the required production rate. Dozers are usually equipped with 6-way hydraulic adjustment on the blade position. Horizontal level would be maintained using a laser leveling system with the receiver and controls mounted on the dozer. The RCC would be placed continuously, and proceed in one direction full width of the lock wall surface from one end of the designated placement to the other end.

a Horizontal Layer Method

Placement of the RCC would include depositing, spreading and remixing. After the RCC has been conveyed to the placement site, the RCC would be spread into a uniform 0.3 m thick lift after compaction. Depending upon placement rate requirements, the contractor would need one or two dozers for spreading (typically a D-6 or D-7 for most applications and a smaller D-4 or D-5 for tight or confined areas or when initiating placement at the start of a lift). Where the RCC is spread onto or into the conventional concrete facing, spreading would be accomplished before the conventional concrete has begun to set. By spreading the RCC and building the 0.3 m lift thickness, tracking of the dozer on the RCC surfaces would provide most of the necessary consolidation for the RCC. Specifications would be very specific in detailing procedures to assure proper RCC spreading and consolidation.

b Sloping Layer Method

A more recent modification of constructing lifts of horizontal layers is the sloping layer method. In the sloping layer method RCC is placed in layers approximately 200 to 300 mm in thickness for a total thickness of 3 to 4 m. Instead of each layer placed in a horizontal orientation, each layer is placed on an approximate 1:10 to 1:20 slope. The length of the slope depends on the plant capacity and production rate. The goal is to minimize the exposure of fresh RCC until the next sloping layer can cover it. Typical slope lengths are 20-40 m. The sloping layer, 3 to 4 m in height and 20-40 m in length, is placed for the full width of the placement and progresses the full length of the placement. Prior to placing the next 3 to 4 m lift, bedding mortar would be placed on the mature RCC surface.

c Monolith Method

For certain placements it is advantageous to subdivide the placement area into monoliths and limit placement to a specific monolith. RCC can be placed by the horizontal layer method or the sloping layer method. Generally placements are in layers totaling more than 1 m in height and as high as 20 m. Climbing forms are required for the taller placements to form the monolith joint.

5.2.3.3.5. Lift Joint Preparation

All surfaces on which RCC and conventional concrete are to be placed would be prepared to insure adequate bonding and continuity so that the entire mass acts as a monolith without discrete joints or discontinuities. As with conventional concrete, the bond between lifts decreases as the maturity of the joint and the elapsed time from final setting of the underlying concrete lift increases. As a minimum, two generalized joint treatment processes, regular and cold joint, would be specified in order to address the range of varying field conditions that exist.

5.2.3.3.6. Compaction

After spreading, all surfaces of each RCC lift would be compacted with a self-propelled double drum vibratory roller or an equivalent single drum vibratory roller. The vibratory roller would be used to primarily to compact the top RCC surface, to seal the lift surface, to prevent undue moisture evaporation during hot weather periods, and to prevent unacceptable absorption of rain or other water prior to RCC set. It also would provide a relatively smooth surface that can be easily cleaned of any contaminants that may occur between placements. The vibratory roller would have a gross weight of not less than 9 525 kg and would produce a dynamic force of about 80 kg per lineal centimeter of drum width at a frequency of 2200 cycles per minute. Compaction and density control would be done using nuclear density meters.

5.2.3.3.7. Curing and Protection

Moist curing would be required on the exposed horizontal surfaces of the RCC. A continuous application of moisture would be required on the exposed top surfaces of the RCC during construction. While this provides adequate curing, the primary benefit is to promote lift joint bonding. Formed surfaces such as vertical and faces would require membrane curing compound seal where moist curing is not practical. The backfilled surfaces of the RCC would receive a continuous application of moisture until the time of backfilling.

The application of water could be done in several ways. One of the best methods to cure fresh RCC is the manual application of spray water from hoses attached to a water distribution system. Completed sections can be cured by uniform coverage using sprinkler systems.

5.2.3.3.8. Weather Conditions.

RCC can be placed during the entire year in Panama. However, there would be periods where placement restrictions would be necessary to protect against excessive temperatures and excessive precipitation. Placements during hot weather would require controls to limit placement temperatures during critical periods. The specification would require RCC as temperature delivered to the placement, to be controlled.

Rainfall in this basin averages 4 m per year. The heaviest accumulation occurs in the rainy season during the months of October and November. RCC would not be placed when rainfall exceeds 1 mm per hour. When RCC placement resumes following a rain stoppage, lift surfaces would require either regular or cold joint surface treatments depending on the extent of the interruption and the effects of the rainfall. Treatments range from the removal of loose contaminants and ponded water to an application of bedding mortar. The impacts of rainfall at the site should be minimized because of the consistent occurrences of rain. The work would be sequenced to minimize exposure to rainfall during threatening time periods and use tarps to cover RCC placements when rain falls on fresh RCC.

5.2.3.3.9. Vertical Facing Options

On many past RCC projects the formed faces of the RCC structure were constructed of conventional concrete (facing concrete). The placement of RCC was done concurrently with facing concrete. Facing concrete was deposited against the forms minutes in advance of the RCC placement. The RCC was compacted with the vibratory rollers and the facing concrete consolidated with internal vibrators. The facing concrete may be conveyed/hauled to the placement site in trucks, by conveyor, or be deposited from buckets from a crane located adjacent to the RCC placement.

Precast concrete wall panels could be placed to proper alignment prior to placing roller compacted. These panels would serve as the completed wall face and form for the RCC placement. Mechanical anchor (dowels) would be provided to achieve an adequate bond across the material interface. This construction process would speed construction and offer controlled quality control measures similar to plant manufacture conditions on the completed panelized lock face concrete.

A recent innovation in RCC technology is the use of grout-enriched RCC (GERCC). This is a process where a small volume of grout is applied to fresh RCC. It can be applied just in advance of the RCC or on top of the fresh RCC or a combination of both. The two materials are then vibrated together using conventional internal vibrators forming a conventional concrete. This system transforms an RCC material into a conventional concrete. It is used extensively at formed surfaces and at foundation interfaces. It allows the continuous placement of RCC in a structure with only supplemental additions of grout and vibration at perimeter areas. This is well suited for tropical areas, such as Panama, where freeze-thaw performance of exposed surfaces is not required. This material would be evaluated for this project. RCC production is maintained at a constant high level and the costly placement of conventional concrete facing is eliminated.

5.2.3.4. Summary

- RCC can be used for a significant portion of the lock structures
- Significant cost saving can be realized by using RCC
- Significant reduction in project construction time is likely
- Combining production and transportation systems with conventional concrete is advantageous
- Rainfall can be handled by schedule, placement methods, and protection measures
- Positioning culvert under chamber floor increases RCC quantity
- GERCC facing system provides excellent formed surfaces with minimal RCC placement impacts

- Sloping layer placement method is a tool to limit exposure of surfaces to adverse weather conditions

5.3. Hydraulic Features

The following paragraphs provide a brief overview and summary of hydraulic features for the third lane lock design. Additional details are provided in Appendix E, Hydraulic Analyses and Designs.

5.3.1. Design Considerations

5.3.1.1. Lock Profile

The profile of the third lane is dependent on several factors. A primary consideration in the selection of a lock profile is the potential range of water surface elevations in Gatun Lake and Limon Bay. Another important consideration is the amount of source water required per lockage. Additional factors to be considered include the required minimum sill clearance and hydraulic freeboard requirements.

5.3.1.2. Water Saving Basins

The volume of water used during a lockage can be reduced with a system of water saving basins. The basins provide a means for recovery, temporary storage and reuse of water during operation of the locks. When a lock is emptied, a portion of the water is transferred to the water saving basins. The water can then be stored in the basin until it is ready to be reused during the next lock filling operation. The net effect is a reduction in the amount of source water required during each lockage.

The water saving basin designs are based on a configuration of two water saving basins per lock located to one side with a target water savings of 50%.

5.3.1.3. Design Ships

An important consideration in the design of the third lane is the type and size of the ships expected to utilize the locks. The two design ship types specified by ACP are the bulk carrier and container ship. Ships of the bulk carrier variety are typically designed to carry dry cargo (e.g. coal, iron ore, grain). Ships designed for liquid cargo (e.g. crude oil) are usually referred to as tankers, but they can be grouped with bulk carriers for purposes of selecting the design ship. Container ships are designed to carry cargo in standard carrying boxes. A standard measure of the cargo capacity for this type of ship is a container that is 20 feet long by 8.5 feet square. The unit of measure for this container is the twenty-foot equivalent unit (TEU).

Selection of the design ship is based on the sizes and types provided in the terms of reference.

5.3.1.4. Lock Filling and Emptying Systems

The primary goal in design of the filling and emptying systems is to safely equalize within the specified target times with the use of water saving basins. For operation without water saving basins, hydraulic features are designed to minimize potential impacts to equalization times while maintaining a safe operation. The target equalization times for a double-lift configuration were determined by proportioning the target range of 8-10 minutes for a single lift of a triple-lift configuration as specified in the terms of reference. The triple-lift target range was increased by a ratio of 3/2 to provide comparable time targets for the double-lift configuration.

Based upon input from ACP, filling and emptying systems were designed to meet target times for operations without water saving basins. As requested by ACP, connections to water saving basins were designed to meet target equalization times for operations with water saving basins. End to end (longitudinal) water surface slopes in the locks were used as an indicator of hawser forces and safe performance. Computed water surface slopes for the proposed filling and emptying system were compared with existing systems and international criteria to evaluate lock performance. Additional factors considered in the design of filling and emptying systems for the third lane include maintenance conditions and the effects of salinity on lock performance.

5.3.2. Alternatives Considered

5.3.2.1. Lock Profile

The locks were evaluated between Gatun Lake levels of 26.670 and 23.927 m PLD and Atlantic Ocean levels of 0.564 and –0.381 m PLD. Lock profiles configured from a relatively high lake and tide elevation reduce water consumption by increasing wall heights that reduces spillage of water. A relatively low lake and tide configuration would have lower lock walls and increased water consumption due to spillage and would require design of a system to manage the spilled water. Once the lock configuration has been established, a high lake elevation will generally require more water than a low lake elevation due to the higher lift.

Sill elevations were evaluated for a minimum clearance of 18.288 m. Gatun Lake and Atlantic Ocean levels within the range presented in the previous paragraph were considered in the evaluation of sill clearance requirements.

Hydraulic freeboard requirements are based on an evaluation of existing lock structures, predicted ship induces surges, and overtravel during equalization operations.

5.3.2.2. Water Saving Basins

The range of possible water saving basin floor elevations and average water consumption was evaluated by performing a duration analysis across the range of Gatun Lake and Atlantic Ocean levels. The analysis showed that there was a narrow range of possible water saving basin floor elevations with resulting narrow ranges of water saving percentages and amount of water taken from the lake. Two basin floor configurations were evaluated for the design.

5.3.2.3. Design Ships

Post-Panamax container vessels with a cargo capacity of 6,600 TEU exist today with a length of 347.0 m, a beam of 42.8 m, and a draft of 14.5 m. Large bulk carriers have a length of 343.0 m, a beam of 63.0 m, and a draft of 23.0 m. Current trends in Post-Panamax container vessels seem to be leading towards a projection of 10,000-12,500 TEU capacity vessels being built within 10 years. These vessels would have a length of 350-400 m, a beam of 50-54 m, and a draft of 14-15 m. Definitive future trends in the size of bulk carriers could not be determined. To be consistent with the terms of reference, a capsize bulk carrier was assumed with a length of 290.0 m, a beam of 45.0 m, and a draft of 15.2 m.

5.3.2.4. Lock Filling and Emptying Systems

Filling and emptying systems can be grouped into three general classes. The first class is designated as end filling. In this type of system, water enters the lock only at the upstream end of the chamber. The second class includes systems that use longitudinal culverts in the lock walls or along the chamber floor to distribute flow more evenly into the lock chamber. The third class is a hybrid that incorporates elements of both the end filling and culvert systems.

In general, end-filling systems can be designed to perform efficiently and safely for lifts that are less than 3 m. For lifts greater than 3 m, these systems usually cannot safely meet performance requirements for equalization time. Connections with the water saving basins could require additional culverts to pass water to and from the locks.

Culvert system types considered include side port, in chamber longitudinal, bottom lateral, and bottom longitudinal. The side port type of system would be acceptable for the double-lift configuration of the third lane. For equalizations with water saving basins, the side port would provide acceptable performance. Hydraulic performance when equalizing without water saving basins could be degraded due to the higher lift. Under certain maintenance conditions, hydraulic performance could be degraded due to the loss of symmetry. Connection to the water saving basins could be achieved by direct connection to the lock culverts. The in chamber longitudinal system would be acceptable for the double-lift configuration of the third lane. For equalizations with water saving basins, the in chamber system would provide acceptable performance. Hydraulic performance when equalizing without water saving basins could be degraded due to the higher lift. Under certain maintenance conditions, hydraulic performance could be degraded due to the loss of symmetry. Connection to the water saving basins would require conduits from the basin to enter the lock chamber and connect to the longitudinal lock culverts. The bottom lateral system would be acceptable for the double-lift configuration of the third lane. Using an interlaced arrangement, hydraulic performance would be good for equalizations with or without water saving basins under both normal and maintenance conditions. Connection to the water saving basins could be achieved by direct connection to the lock culverts. The bottom longitudinal system would be acceptable for the double-lift configuration of the third lane given its ability to perform quickly and safely under all operating conditions. Direct connection could be made between the water saving basins and the lock wall culverts.

Hybrid filling and emptying systems were not considered due to the concerns associated with end filling systems at the design lift.

5.3.3. Recommended Plan

5.3.3.1. Lock Profile

The lock wall elevations were selected based on high lake and ocean levels and the resultant equalizing elevations in the locks. This eliminates the need to spill water when the locks would equalize at a high level.

The lake level used was 26.670 m PLD and the ocean level was 0.564 m PLD. The 1.311 m of freeboard would be used to set the minimum top of wall and gate elevations for hydraulic design concerns. This freeboard should be applied above the maximum operating elevation so that there will always be 1.311 m or more of freeboard. Figure 5.3-1 summarizes the recommended lock profile for operations without water saving basins.

A duration analysis, which considered the entire range of possible lake and ocean elevation combinations, was conducted to determine the average amount of water required for a single downbound lockage without water saving basins. The average amount of water required in terms of a water column was computed as 13.041 m. The total volume of water required can be computed by multiplying the water column by the surface area of the lock.

5.3.3.2. Water Saving Basins

The basin wall elevations were selected based on high lake and ocean levels and the resultant equalizing elevations in the water saving basins. This eliminates the need to spill water during high water periods when the locks and basins will equalize at a high level.

The 1 m of freeboard would be used to set the minimum top of wall elevations for hydraulic design concerns. This freeboard should be applied above the maximum operating elevation so that there will always be 1 m or more of freeboard.

The lake level used was 26.670 m PLD and the ocean level was 0.564 m PLD. Figure 5.3-2 summarized the recommended lock profile for operations with water saving basins.

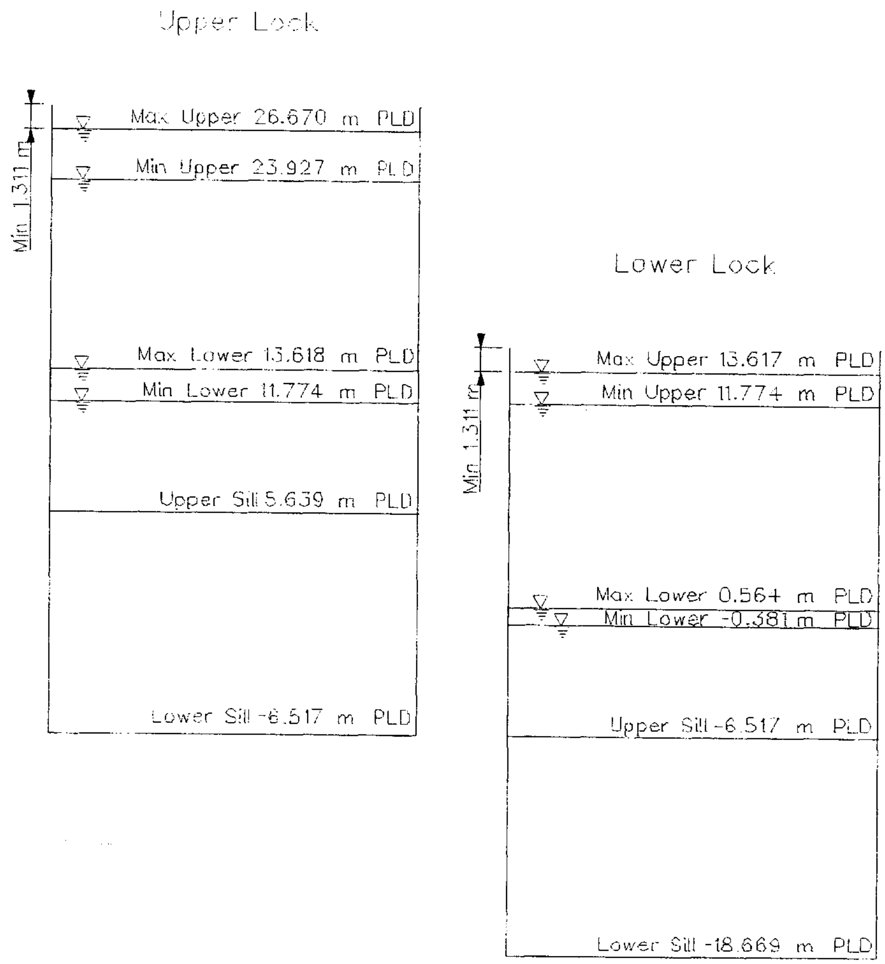

Figure 5.3-1 Lock Profile Without Water Saving Basins

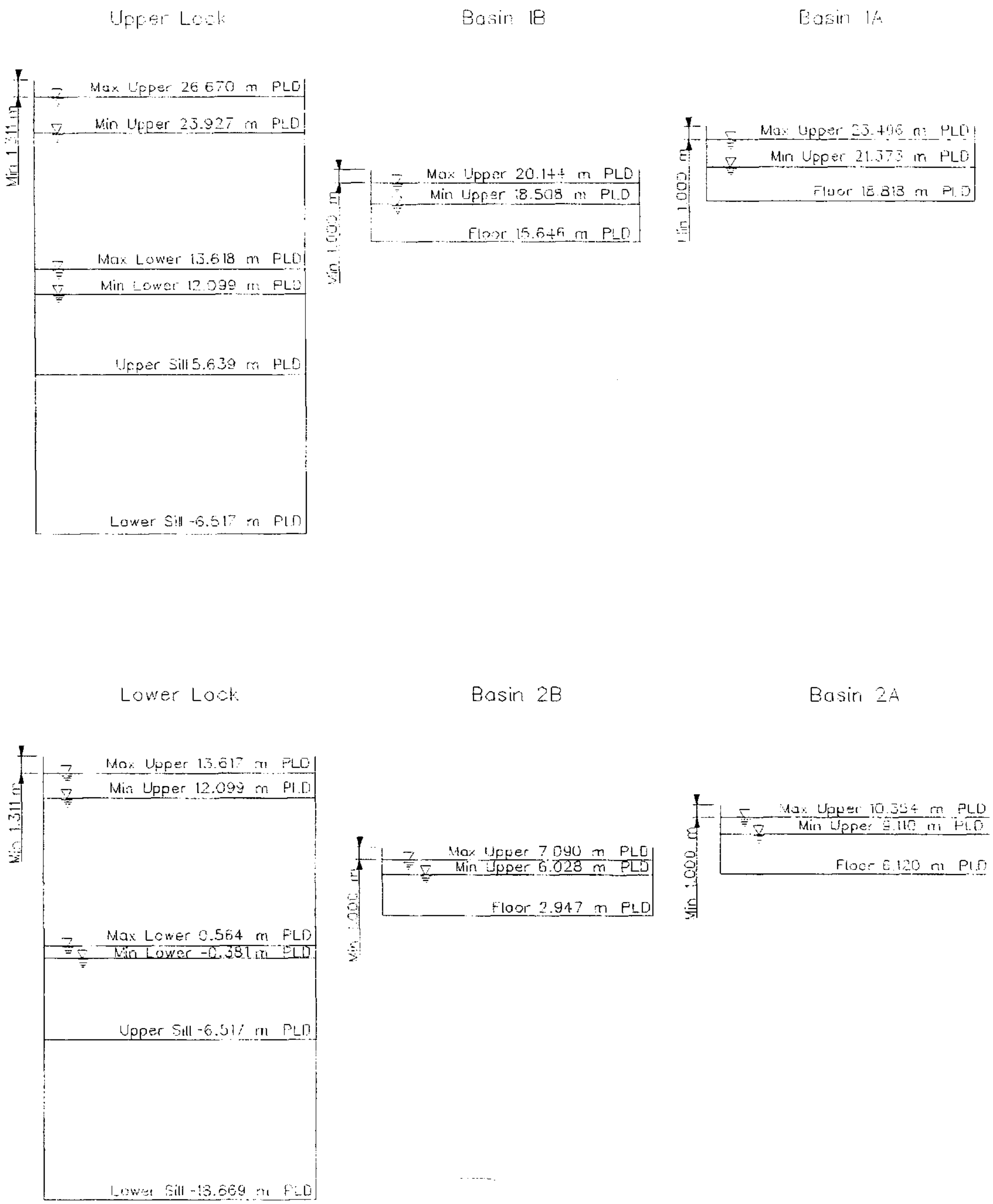

Figure 5.3-2 Lock Profile With Water Saving Basins

5.3.3.3. Design Ships

The 12,500 TEU container ship and the capsize bulk carrier are assumed for use in the concept level design. The ships were selected because they are consistent with the deadweights specified by ACP in the terms of reference. A summary of the design ships is presented in Table 5.3-1.

Table 5.3-1 Recommended Design Ships

Vessel Type	Length (m)	Beam (m)	Draft (m)	Block Coefficient	Deadweight (t)	Displacement (t)
Container	385.7	54.9	15.2	0.65	125 000	209 000
Bulk Carrier	290.0	45.0	15.2	0.85	135 000	169 000

5.3.3.4. Lock Filling and Emptying Systems

Based on hydraulic performance, the bottom longitudinal and interlaced bottom lateral systems with water saving basins on both sides of the lock would be the best alternatives for the double-lift configuration of the third lane locks. In consideration of operational advantages, water saving basins could be located on one side of the lock with minimal impact on hydraulic performance.

Based on an evaluation of the overall project, ACP selected the in chamber longitudinal culvert system (ILCS) and interlaced bottom lateral system with water saving basins on one side of the lock. The ILCS system would provide good performance when equalizing with water saving basins. Acceptable performance would be achieved for operations without water saving basins or under certain maintenance conditions; however, hydraulic performance could be degraded due to the higher head or loss of symmetry associated with these operations. The interlaced bottom lateral system would perform well for equalizations with or without water saving basins under both normal and maintenance conditions.

The ILCS system would have conventional intake and outlet manifolds in the lock walls. The main lock culverts would be 8 m wide and 7 m high with bifurcated valves that would be 4 m wide and 7 m high. Water would be transferred from the longitudinal culverts to the lock though a series of ports. There would be 64 ports along each culvert with each port being 0.6 m wide and 2 m high. Connection to the water saving basins is achieved though a set of four conduits and valves per basin that would be 6 m wide and 7 m high.

The interlaced bottom lateral system operates with a conventional intake and outlet manifold. Lock culverts would be 6 m wide by 8 m high with valves of the same size. Water would be transferred from the culverts to the lock through connections to an alternating set of laterals along the lock chamber floor. Ports in the laterals allow water to enter or exit the lock chamber. Connection to the water saving basins is achieved though a set of four conduits and valves per basin with the same dimensions as the lock culverts and valves. Two of the conduits per basin connect directly to the west wall lock culvert. The other two conduits per basin connect to crossover conduits that pass under the lock chamber.

A summary of equalization times for the selected systems for normal operation at the average lift is presented in Table 5.3-2. A summary of maximum end-to-end water surface slopes and estimated hawser forces are presented in Tables 5.3-3 and 5.3-4. The forces are less than the recommended maximum of 209 t.

Equalization times will generally be longer for operations with water saving basins due to the additional valve operations and lower driving head. Equalization without basins requires a total valve operation time of 2 minutes, which consists of a single 2-minute valve opening of the lock culvert valves. Equalization with basins requires a total valve operation time of 5 minutes, which consists of a 1-minute valve opening and closing of each basin valve (4 minutes total for the basins) and a 1-minute opening of the lock culvert valves. The additional valve operations add time to the cycle because the discharge in the culverts is not fully developed during opening and closing of the valves. In addition, the water surface differential that drives the lock to fill or empty by gravity is 50% less when equalizing with basins. This is because the lock is filled in three stages with basins versus one stage without basins. Since discharge is approximately proportional to the square root of the driving head, a 50% reduction in head will produce a 30% reduction in discharge. The reduction in discharge will translate into a longer equalization time.

Careful consideration was given in the design to minimize increases in equalization for operations with water saving basins. The flow area of the basin conduits was increased to facilitate a faster transfer of water from basin to lock. The design incorporates four conduits per basin with crossovers to increase discharge capacity. Other features such as laterals and ports were also sized to improve performance for equalizations with basins. Additional time saving was achieved by utilizing an overlapping valve schedule when equalizing with basins. Sequential operations were overlapped by approximately ½ minute.

Maximum longitudinal water surface slopes and the associated hawser forces will generally be greater for operations with water saving basins due to superposition. The initial valve operation for the first basin will set up a longitudinal water surface differential in the lock chamber. These initial slopes tend to be relatively small due to the lower initial driving head. Subsequent valve operations, however, can add to these existing longitudinal waves and produce a higher maximum hawser force. Since equalization without basins only has one valve operation, superposition does not occur and the maximum hawser forces were generally lower.

Table 5.3-2 Summary of Equalization Times

Equalization	Interlaced Bottom Lateral		ILCS	
	Without WSBs	With WSBs	Without WSBs	With WSBs
Gatun Lake to Upper Lock	12.7	14.6	11.6	14.6
Upper Lock to Lower Lock	10.8	13.5	9.2	13.7
Lower Lock to Atlantic Ocean	12.8	15.1	11.7	15.3

Table 5.3-3 Estimated Maximum Longitudinal Hawsers for Design Container Ship (Interlaced Bottom Lateral with Downstream Inner Gates Open using LOCKSIM Beta Version - Ship Effect Included)

Equalization	Operation Without WSBs		Operation With WSBs	
	Maximum Slope (1/1000)	Estimated Hawser (t)	Maximum Slope (1/1000)	Estimated Hawser (t)
Lake Gatun to Upper Lock	0.75	157	0.77	161
Upper Lock to Lower Lock	0.59	123	0.74	155
Lower Lock to Atlantic Ocean	0.21	44	0.57	119

Table 5.3-4 Estimated Maximum Longitudinal Hawsers for Design Container Ship (ILCS with Downstream Inner Gates Open using LOCKSIM Beta Version - Ship Effect Included)

Equalization	Operation Without WSBs		Operation With WSBs	
	Maximum Slope (1/1000)	Estimated Hawser (t)	Maximum Slope (1/1000)	Estimated Hawser (t)
Lake Gatun to Upper Lock	0.67	140	0.82	171
Upper Lock to Lower Lock	0.59	123	0.70	146
Lower Lock to Atlantic Ocean	0.30	63	0.73	153

5.3.4. Recommended Modeling

Filling and emptying systems for the double-lift lock configuration have been designed using existing Corps criteria to safely meet target equalization times between 12 and 15 minutes for operations with use of water saving basins. At the average lift of approximately 12.9 m, more than 400 000 000 L of water is transferred in 15 minutes or less for each equalization operation. Considering the size of the locks and the complexity of the filling and emptying system design, physical hydraulic modeling is recommended in future studies. Physical modeling prior to final design would be part of the normal design process used by the Corps for a project of this scope. Physical modeling would provide the opportunity to validate and make refinements to the design that would enhance the overall performance of the system.

A physical model of the complete filling and emptying system at an approximate scale of 1:25 is recommended to evaluate overall system performance. Typical parameters to be measured in the lock chamber with this model would include equalization times, hawser forces (longitudinal and transverse), surface turbulence, loads on lock gates, and ship effects. Pressures and discharges at various key locations (e.g. transitions, junctions, valves) within the culverts and conduits would also be measured and evaluated. The valves would be evaluated for design loads and cavitation potential for varying operating schedules. Performance of the intakes for both the lock culverts and WSB conduits should be evaluated for vortex formation. Additional parameters may be determined as needed to make improvements to the design.

Operation of the miter gates would be modeled to validate the assumptions and computations used to predict the interaction of the gate with the surrounding water during operation. This testing could be incorporated into the filling and emptying system model described in the preceding paragraph.

Culvert and conduit valve testing would require a separate model at an approximate scale of 1:10 to optimize their performance. Goals would be to minimize vibration potential and evaluate performance of the conduit configuration. Model testing would facilitate optimization of the edge and recess slot geometry to minimize vibration potential. Results would validate performance of the system to ensure reliability and long-term durability of the valves.

5.4. Lock Gates

5.4.1. Description.

The proposed 61 m wide lock double-lift lock concept design would have three gate bays referred to as the north, center, and south gate bay structures. The north gates are located at the entrance to the Atlantic Ocean, the center gates separate the upper and lower lock chambers, and the south gates are the lock gates at Gatun Lake. Double sets of gates (four individual gate leafs) would be provided at each gate bay. A vehicular bridge would be provided in proximity to the northern most gates to provide access across the lock chamber. The double gate arrangement would allow a measure of protection against loss of lake storage resulting from potential ship impacts and allows for continuing service during periodic maintenance cycles. The proposed lock gates are shown on report drawings ACP-R-21/1 through 21/20.

A Gate Selection Study was performed to determine the most advantageous lock gate system for the double-lift locks. This study is attached to the Main Report as Appendix B, Lock Gates Analyses and Designs. The Gate Selection Study shows that mitering locks gates offer a unique mesh of attributes with benefits over other alternatives in initial investment cost, reliability of continued operation, installation and removal, and ease of maintenance. The design considers hydrodynamic loads during earthquake events, expected range of Gatun Lake pool elevations, Atlantic Ocean tidal variations, torsional deflection and stresses during operational movements in water, machinery stall loads applied to a gate while motion is inhibited by an object on the sill, and installation and removal methods. Each of these issues is addressed in Appendix B.

Two miter gate designs have been considered, a single skin plate design with gate diagonals and a double skin plate design. These two types of miter gates were developed to demonstrate their costs and advantages and disadvantages. A double skin plate design is recommended for the following reasons:

- The primary advantage of single skin plate design is a built in capability to adjust the twist angle between the miter and quoin ends. However, considering the history of performance of the existing Canal gates and built-in torsional resistance of the double skin plate gate, this ability has not been required and would place additional periodic maintenance attention for the gates.
- Double skin plate design significantly reduces lifting requirements for routine installation with a ballasting installation procedure and these gates can be transported to a maintenance facility by floatation in a horizontal position like a barge.
- Both gate types have similar weights and would have similar handling requirements in an emergency response scenario.
- Single skin plate miter gates have a history of fatigue and fractural failures. Double skin plate gates are inherently more efficient in resisting torsional loads.

5.4.2. Geometry and Fabrication

The gate height selection considered variations of Gatun Lake operating pool elevations, tidal variation of the Atlantic Ocean, and operations with water saving basins. Table 5.4-

1 lists the gate and sill elevations and height of gates considered. The gate sill on the north gates was lowered approximately 0.25 m to allow the same gates to be used in both the north and center gate bays. The use of common gates is beneficial from an operational and maintenance perspective.

Table 5.4-1 Gate Heights

Gate Bay	Top of Gate Elevation	Sill Elevation	Gate Height
North Gates	14.93	-19.57	34.50
Center Gates	27.98	-6.52	34.50
South Gates	27.98	5.64	22.34

The minimum clearance over the gate sill would be 18.3 m. All gates would extend 1.3 m above maximum operating pool elevations to provide freeboard and prevent overtopping from wind and ship induced waves. The top of lock wall would be 3.62 m above the damming height of the lock gates (top of skin plate). The physical top of gate-mounted accessories extends above the top of skin plate for attachment of a walkway and connection of the hydraulic cylinder. The spacing between gate bays provides a clear usable lock dimension of 426.7 m when using double gate lockages (both gates in a gate bay). The usable chamber length would be increased to a maximum of 475.0 m by using single gate lockages (one gate in a gate bay). The adjacent pintle-to-pintle spacing within each gate bay is 45.5 m.

The gate leafs would be approximately 35.5 m long from the miter contact to the quoin contact. The angle that the gate leaf makes with the lock centerline when the gate is in the fully mitered position is 1 on 3. The gates would be straight horizontally framed, welded steel construction with the skin plate located on the upstream and downstream side to form buoyancy chambers to augment installation, transportation, and reduce operating loads. Bolted connections would be provided for removable features such as the pintle base. Riveted construction was considered and dismissed for the following reasons:

- Riveted construction is slow and labor intensive.
- Riveted construction is no longer a common fabrication process.
- Welded construction is anticipated to be the standard fabrication process used by potential gate fabricators.
- Less material is needed to form joints made by welding when compared to either bolted or riveted connections which results in lower costs and less demand operating equipment and support structures.
- Structural toughness associated with riveted structures can be achieved by welding with proper quality assurance, proper joint selection, material selection, and post fabrication stress relieving.
- Water tightness is more difficult to achieve with either bolted or riveted construction.

The girder web and flanges, diaphragms, and miscellaneous plates would be ASTM A572M GR 345 steel. The skin plate and intercostals would be ASTM A572M GR 450 steel. The gates would be supported by and swing on 0.60 m radius hemispherical pintles. The pintles would be designed to limit movement in the pintle base. The gates would be operated with direct connect hydraulic cylinders. The top of the gates would be hinged to the top of wall by means of an anchorage arrangement incorporating a turnbuckle type adjustment for leveling the gate. A gate latch system would be included within the gate recess to limit drift while the gates are in a recessed position.

Adjustable stainless steel miter and quoin contact blocks would be provided at the miter and quoin ends of the gates to seal water and transmit the thrust loads imposed on the gates through the gate framing into the concrete lock wall. Adjustable contact blocks are shown in Figure 5.4-1 and 2.

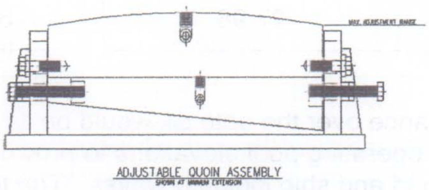

Figure 5.4-1 Adjustable Contacts (Wedge System)

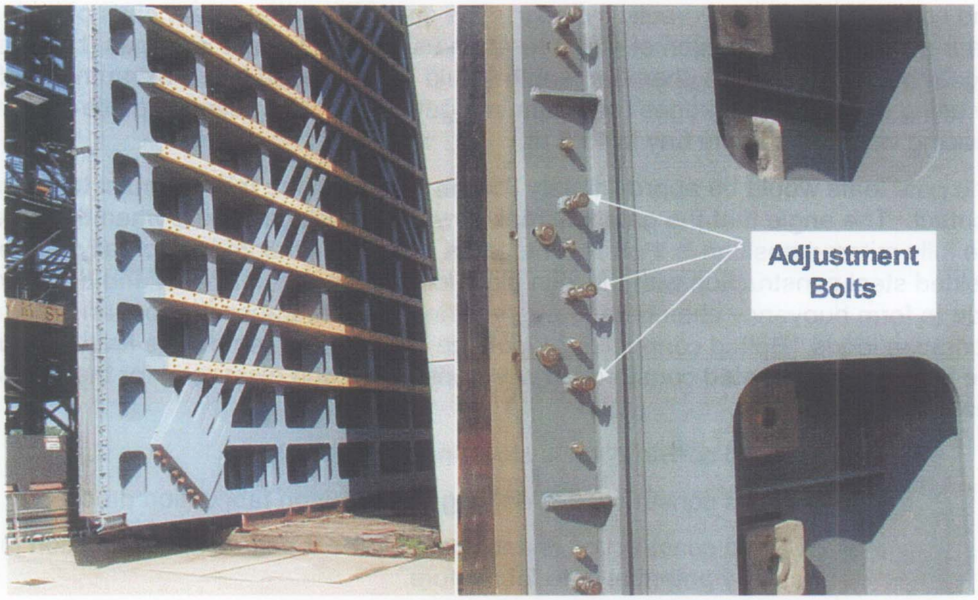

Figure 5.4-2 Adjustable Contacts

A vertical rubber to metal seal would be provided at the pintle and across the bottom of the gate between the bottom girder and the sill. The bottom seal would be located on the gate to minimize hydrostatic uplift with differential head on the gate. The gates would be equipped with mitering devices to aid in accomplishing correct mitering.

Several alternatives have been developed to install and remove the gates. The gates could be installed in a similar manner to the existing Canal gates. The existing ACP Titan crane, with a rated capacity of 350 t would be used to augment a simple ballast procedure. As a provision for future maintenance flexibility, the gates would also be designed to be lifted for maintenance or emergency conditions by a gate-lifting crane. Jacking supports would be provided on the bottom girder to allow the gates to be jacked from below in the dry. A 4 m wide bridge would cross the top of the miter gates and be designed to support needs of lock personal and maintenance activities.

5.4.3. Design Criteria

A complete description of the design criteria is listed in Appendix A, Design Criteria Report. In general, the design complies with the standards of the Corps of Engineers. Usual design conditions are described in Appendix A. In consideration of the initial expense and importance of continuing operation of the locks on the economic benefits of a new locks system, several variations to established criteria for typical navigation locks are recommended and presented below.

5.4.3.1. Seismic Design

Standard criteria of the Corps of Engineers only require that gates be designed to support an Operational Basis Earthquake (OBE). However, considering that the lock gates are a critical structure and that they would need to retain Gatun Lake level, which may take up to two years to refill if lost, this may not be appropriate as site specific criteria. The gates are supported by rock founded concrete gravity structures that are extremely stiff in the longitudinal axis of the lock. For this reason, 2/3 of the MCE earthquake was used to design the gates. The elastic analysis shows that no members have an interaction ratio above 1.0. The design presented in this report is based on a seismic coefficient of 0.21g (MCE=0.313). Evaluation for higher seismic loading would consider and permit some level of inelastic behavior that is not detrimental to structural and operational performance.

5.4.3.2. Pintle and Pintle Bushing

The pintle and pintle bushing were designed assuming that the gate self weight is reduced 60% by buoyancy. The allowable contact pressures used for the condition was limited to 10.0 MPa. The contact pressures were limited to 20.0 MPa for operation in the dry or with loss of buoyancy. Conventional bronzes bushing with typical lubrication systems are recommended until further research and performance evaluation of self-lubricating bushings are conducted in following engineering studies.

5.5. Culvert and Conduit Valves

The filling and emptying valves controlling the flow in the culverts and conduits are of the wheel gate type of welded steel construction. Reverse tainter valves were considered and dismissed due to limitations on space needed for the redundant valve and required separation transition zone. The valves would be horizontally framed with skin plating provided on both the upstream and downstream sides. The top and side seals of the valves would be molded rubber J-seals and would seal against a metal frame embedded at the culvert perimeter. The bottom lip would be a metal-to-metal seal. The culvert valve design would be based on the loading criteria and allowable stresses as provided in EM 1110-1-2105, Design of Hydraulic Steel Structures. The culvert valves proposed with Alternative 1 – Bottom Interlaced Lateral filling and emptying system would control flow through 6x8 m culvert; filling from either lock wall culvert is acceptable during maintenance conditions. In Alternative 2 – In-chamber longitudinal Culvert filling and Emptying system the 8x7 m culverts are bifurcated to 4x7 culverts at valve stations; one of the opposition 4x7 m valves in the opposite lock wall would need to shut down during maintenance periods to maintain symmetry required for smooth lock performance i.e. equal flow in each lock wall culvert is required during normal and maintenance operation.

5.6. Electrical and Mechanical Operating Systems

At a partnering meeting in Pittsburgh on 4 and 5 December 2002 with the ACP and USACE, opportunities were discussed for redesign of the Double-Lift configuration to reduce costs. It was agreed to perform a screening study of four cost-saving alternatives (included in this report as Appendix J Filling and Emptying System Screening Study).

At the meeting the group recognized that there were no apparent cost saving options in the electrical and mechanical features of the concept design that was submitted on 25 October 2002. Therefore, design of the electrical and mechanical features was not advanced with this resubmittal. Resources that would have been expended in the revision of the electrical and mechanical features was redirected to structural and hydraulic design features that offered much higher potential for cost savings.

The mechanical and electrical features are included in this report in Appendix H Electrical and Mechanical Lock Operating Systems and Appendix I Electrical and Mechanical Reference Drawings. The electrical and mechanical feature designs in these appendices still represent an appropriate cost level for this concept design. The appendices also contain relevant design features for a lock structure of this size for a Double-Lift configuration and can also be extrapolated for a Triple-Lift configuration.

5.7. Entrance Walls

5.7.1. Introduction

The Atlantic Locks Third Lane Project is designed to permit transit of vessels significantly larger than those vessels utilizing the Panama Canal at this time. The Entrance Walls are the locations where the vessels first encounter the new Locks, either by connecting with the Ship Positioning System, or by physically impacting or rubbing against the walls. The geometry, displacement and handling characteristics of these Post-Panamax ships will be the primary drivers for the design of the Entrance Walls. In Appendix F, Lock Masonry Analyses and Designs, there is an extensive discussion of these vessels and the resulting impact forces that must be mitigated. At this Concept design level, two main long and two short stub Entrance Walls are planned. The Corps Design Team optimized wall lengths after consultation with ACP Canal Pilots. The lengths of the walls are provided in Appendix F.

The two east Entrance Walls would be oriented parallel to the lock chamber walls, and flush with the lock chamber face. A significant reach of the Gatun Lake entrance wall would be built in-the wet and would be set back from the chamber wall faces, to permit space for installation of the selected fender system. A tapered sacrificial fixed end pier would be located at the ends of each wall most remote from the Locks, to protect the Entrance Walls from direct "head-on" impacts. These end sections are referred to herein as Nose Piers. The Nose Piers would be protected with fenders.

The top-of-wall elevations have been selected to permit installation of a continuous high-capacity fender system, such that maintenance of the fenders and access to anchor bolts can be accomplished entirely above the "normal" water line. The fenders are a critical component of the Gatun Lake entrance wall design and dynamic response of the walls. The energy absorbing capacity of the fenders, combined with the stiffness of the wall structures and the assumed vessel hull stiffness, would all contribute to the Gatun Lake Entrance Wall's system response in the event of a vessel impact.

5.7.2. Entrance Wall Alternatives Considered

5.7.2.1. Gravity Monoliths

Gravity Monoliths similar to those recommended for the lock chambers were evaluated in areas of in-the-dry construction. Preliminary stability analyses were performed for typical sections along the wall alignments. These wall sections would be backfilled to buttress the wall for the extreme vessel impact case. Preliminary screening found gravity entrance walls to be feasible and cost effective.

5.7.2.2. Floating Walls

Floating walls were considered but eliminated for a number of reasons. Ideal applications for floating walls occur at sites where the water is too deep for conventional foundations, or if the walls are subjected to widely varying water elevations. Neither case is applicable at the Third Lane Atlantic Locks site. The high expense of the construction of pontoons is not justified considering the minimal Gatun Lake and Atlantic Ocean tidal variations. Additionally the very large vessel impact forces would have resulted in very wide and expensive walls because the punching shear in the walls on the impact walls would have required very thick, and thus very heavy, side walls. As the side walls grow thicker, the overall pontoon width must increase. For example, the Olmsted Locks Approach Walls on the Ohio River, U.S.A. are up to 13 m wide for a maximum design vessel impact force of 4448 kN. By comparison, the maximum design vessel impact force imparted to the structure at Panama will be approximately 116 880 kN. If floating Entrance Walls were constructed, the high block coefficients of the Post-Panamax vessels would have been problematic in the case of two vessels passing in the adjacent channels without an intermediate wall to stop flow.

5.7.2.3. Walls Mounted on Sheet Pile Cells

This option was briefly considered but eliminated due to the great channel depths of as much as 30 m. The tallest sheet pile cells of which the Corps is aware were approximately 25 m in height above the mud line. Currently there is a practical limit of sheet pile length to consider as well, although more research needs to be accomplished in this regard. Although it is certainly possible to construct taller cells, the potential driving problems frequently associated with construction of sheet pile cells are exaggerated as cell height increases. Additionally the large lateral forces imposed by the extreme impact load case would have resulted in cellular structures with diameters greater than 25 m, and concrete costs would likely have been excessive.

5.7.2.4. Walls Mounted on Drilled shafts

It is recommended that the Entrance Walls to be constructed in-the-wet be mounted on 2.44 m diameter drilled shafts spaced at 30 m center-to-center, each having permanent steel casings. Drilled shafts are recommended because they are clean, relatively simple to construct, and the technology offers a high level of confidence in their integrity upon completion. They can carry extremely high axial and lateral loads. One large-diameter drilled shaft can replace large numbers of piles. Perhaps most critical to Panama, drilled shafts are seismically desirable and desirable for vessel impacts because of their high lateral load capacity.

For the Gatun Lake Entrance Wall, the installation of these foundation elements can be accomplished "in the wet" out in Gatun Lake, utilizing barge-mounted equipment. The foundation elements would support conventional cast-in-place or precast concrete Cap Beams that would span between the shafts. These rectangular cross-section beams would support the Ship Positioning Locomotives, and would accept and distribute the vessel impact reaction loads that are transmitted from the continuous fender system. A continuous fender system would be mounted on the face of all of the flexible Entrance Walls to absorb impact energy. The fenders have been designed to absorb nearly all of the impact energy from collisions that can be expected to occur within the lifetime of the structures.

5.8. Maintenance and Emergency Closures

5.8.1. Maintenance Bulkheads

Design of maintenance closures are beyond the scope of the Concept Design Contract. However, the lock has been designed to accommodate placement of a caisson or bulkheads at the Atlantic and Gatun Lake entrances. Work to the lock floors or gate sills could also be accomplished locally in a box caisson.

5.8.2. Culvert And Conduit Bulkheads

Culvert bulkheads would be provided to dewater the culvert and conduit valves. The bulkheads would be approximately the same dimensions as the valves for which they are intended to service. Two sets of culvert bulkheads would be provided to service the six culvert valve locations. One set of conduit bulkheads would be provided to service the two water saving basin hydraulic systems. The bulkheads would be of welded steel construction, horizontally framed with the skin plate on the sealing side. The bulkheads would be stored in the culvert and conduit bulkhead slots. A portable "A-frame" and power winch would be provided to aid in placement and removal of bulkheads.

5.8.3. Emergency Closure

Several alternatives for emergency closure systems have been reviewed. These include:

- Dumping rock.
- Sinking barges.
- Rising bulkhead or caisson structure
- Wicket systems
- Emergency bulkheads

In evaluating these alternatives, only the structural feature solutions seem reliable. Placement of rock would require floating plant and may wash away unless adequately sized or restrained by netting anchorage to a pile head. The required rock size would make placement difficult. Placement of bulkheads or dumped stone with floating plant is not a safe operation since this would place the crane in proximity to very turbulent waters. Access to these operations by land-based equipment may be a feasible alternative. A transfer structure on the east wall may be used to set stacking bulkheads. Viable options lead to using a rising caisson, transfer bulkhead arrangement, or wicket systems.

A rising caisson structure would be constructed and stored in a pit located below the approach floor elevation. This option would be expensive to build in consideration of the extensive rock excavation. Transfer systems for launch across the top of lock walls can be used to move bulkheads from a storage pit to a powered slot system in the lock wall. The bulkheads would stack and have end rollers to allow installation in flowing water. This arrangement, although smaller, is in service on the Bonneville Lock.

A wicket system would minimize excavation and would appear to be the most economical solution. Wicket systems of the size required for the Third Lane project do not exist. Emergency bulkhead systems and the wicket system are recommended for additional evaluation in future studies. A cost for emergency closure systems is included in the cost estimate. Scaling a system developed for the Markland Lock and Dam project on the Ohio River derived this cost.

5.9. Operating Structures

Locking operations include activities such as opening and closing of the lock gates and controlling water level within the new Third Lane Lock chambers. These operations would be closely controlled using state of the art electronic equipment such as cameras, monitor displays, and "touch screen" technology to activate lock operating systems. The new facility can also serve as the "hub" and communications control center for all daily activities occurring at the Canal's Atlantic side locking operations. This includes such activities as; directing all shipboard movement in proximity to the locks and directing all personnel activities and security within the cantonment area. This facility would function similar to an airport control tower.

Conceptual plans for a new lock operations communications center have been prepared and are included. The functional layout and vertical configuration is based on an ACP communications and model command structure shown as Figure 5.9-1. Using this model command structure, the new lock operations center span of control can be comprehensive and include all personnel movement, ground based activities, and water borne activity within the three locks' sphere of influence.

The communications center floor plan layout (sheet ACP-R-70/1) is divided into functional levels. Together, the levels contain approximately 155 m^2 of useable interior floor area. The middle level contains shared common functions. The functional activities within the spaces are not yet firmly established. Level 1, for example might be used as a security control center, or as a remote alternate site for operating existing locks 1 and 2 should the need arise.

Exterior finishes are compatible with locally available materials that have proven to require minimal maintenance cost and upkeep.

This submittal has been assembled as a tool to facilitate required follow-on design effort. It is intended the plans and the questionnaire that follows be used as tools by the ACP for the following:

a. Highlight types of questions that will provide critical information affecting the design process and final product for each function and facility.

b. Help the ACP identify those strategic personnel that can participate as technical team members of the design team and offer input during the programming and design development processes.

c. Promote dialogue and information sharing and enhance teamwork.

Technical team members established by ACP will be needed as information is collected during the design process. ACP team members would complete an "Information Index, Needs Assessment Inventory", collect and organize field data and investigate current operating processes. This information was requested from the ACP, however, the Locks Team deferred additional design efforts to following engineering studies. Other facilities will be required to support the new lock. Such functions as equipment maintenance, storage facilities, warehousing space, etc., have not been evaluated. To identify needs, condition and adequacy of all existing facilities should also be assessed for remaining useful life, spatial adequacy, and functional adaptability. Upon completing this inventory and analysis, a comprehensive list of needs and resulting building program will emerge.

Facilities associated with new lock construction would not be operational until lock construction is complete, which is well into the future. Thus, design of new lock related facilities could be delayed for several years without any operational impact.

However, there appears to be one immediate opportunity for design and construction consideration. It is important to highlight that ACP has embarked upon an innovative and comprehensive planning and design program to insure a healthy future for the Panama Canal. A high quality presentation focused toward educating visitors can effectively communicate and explain this dynamic long-range initiative. An educational exhibition that explains the need for expansion, the engineering challenges, the alternatives being developed, the timeline, and identifies individual projects would be of significant interest to canal users, the country, employees, and the international tourist industry. The presentation could be placed in a new high-quality exhibition facility with other exhibits, describing the long and rich history of the canal, showing "how locks work," and describing the canal's current limitations. A new building containing all types of presentations and interactive learning tools could also incorporate retail space for cultural and local merchandise and offer local excursions. If carefully positioned, such a facility could also act as the "magnet" for local development or historical revitalization based on tourist related business expansion.

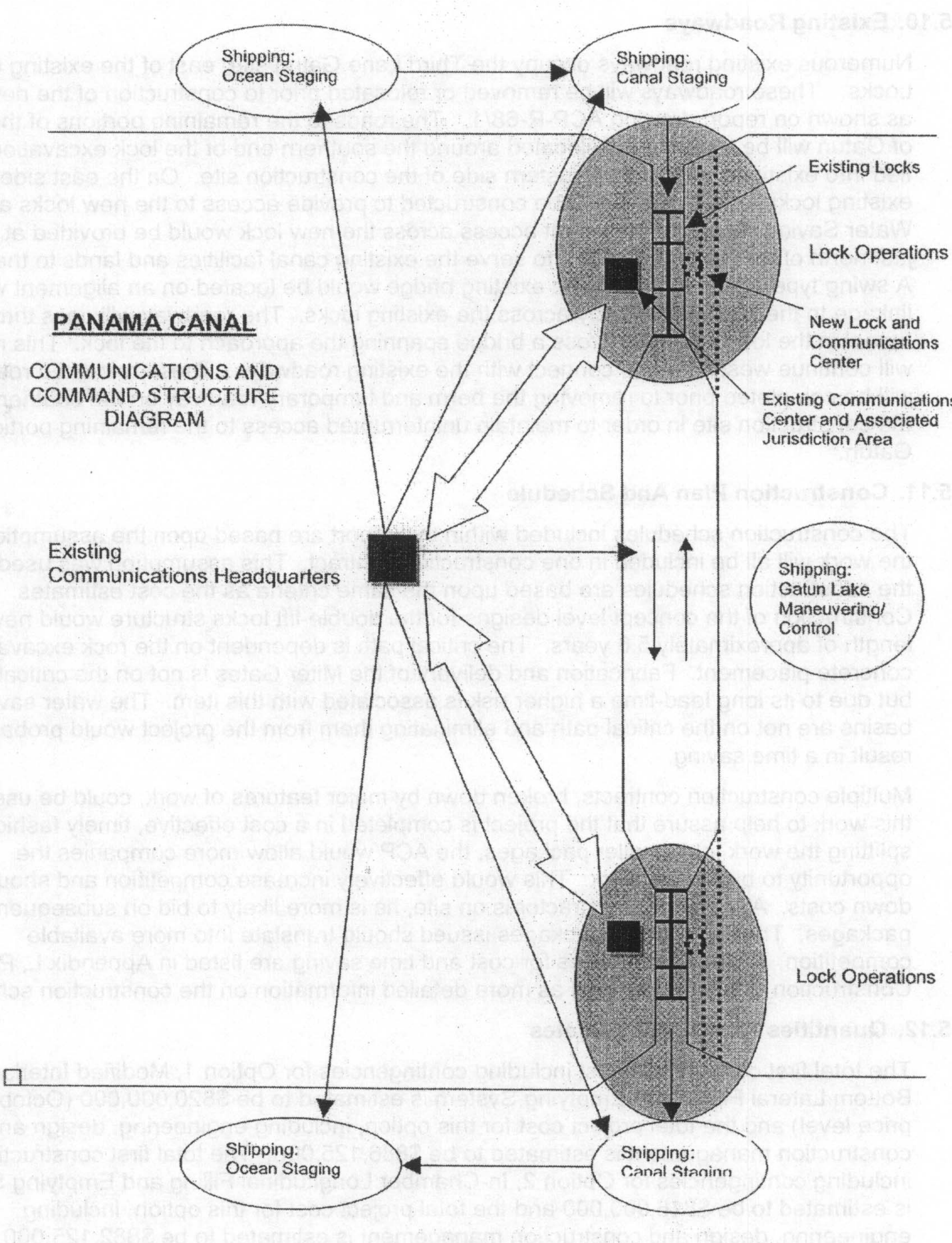

Figure 5.9-1 Functional Layout of Operating Structures

5.10. Existing Roadways

Numerous existing roadways occupy the Third Lane Gatun site, east of the existing Gatun Locks. These roadways will be removed or relocated prior to construction of the new locks as shown on report drawing ACP-R-68/1. The roads to the remaining portions of the town of Gatun will be temporarily relocated around the southern end of the lock excavation and tied into existing roads on the eastern side of the construction site. On the east side of the existing locks, new roads would be constructed to provide access to the new locks and the Water Saving Basins. Permanent access across the new lock would be provided at a point just north of the lower lock gates to serve the existing canal facilities and lands to the west. A swing type bridge similar to the existing bridge would be located on an alignment with linkage to the existing roadway across the existing locks. The roadway will pass through a tunnel in the lock wall and across a bridge spanning the approach to the lock. This roadway will continue westward and connect with the existing roadways. These relocated roadways will be completed prior to removing the berm and temporary roadway on the southern end of the construction site in order to maintain uninterrupted access to the remaining portions of Gatun.

5.11. Construction Plan And Schedule

The construction schedules included within this report are based upon the assumption that the work will all be included in one construction contract. This assumption was used so that the construction schedules are based upon the same criteria as the cost estimates. Construction of the concept level designs for the double-lift locks structure would have a length of approximately 5.0 years. The critical path is dependent on the rock excavation and concrete placement. Fabrication and delivery of the Miter Gates is not on the critical path, but due to its long lead-time a higher risk is associated with this item. The water savings basins are not on the critical path and eliminating them from the project would probably not result in a time saving.

Multiple construction contracts, broken down by major features of work, could be used for this work to help assure that the project is completed in a cost effective, timely fashion. By splitting the work into smaller packages, the ACP would allow more companies the opportunity to bid on the work. This would effectively increase competition and should drive down costs. Also, once a contractor is on site, he is more likely to bid on subsequent packages. Therefore, more packages issued should translate into more available competition. Other opportunities for cost and time saving are listed in Appendix L, Project Construction Schedules as well as more detailed information on the construction schedules.

5.12. Quantities And Cost Estimates

The total first construction cost including contingencies for Option 1, Modified Interlaced Bottom Lateral Filling and Emptying System is estimated to be $820,000,000 (October 2002 price level) and the total project cost for this option, including engineering, design and construction management is estimated to be $886,125,000. The total first construction cost including contingencies for Option 2, In-Chamber Longitudinal Filling and Emptying System is estimated to be $816,000,000 and the total project cost for this option, including engineering, design and construction management is estimated to be $882,125,000. Appendix L, Project Construction Schedule contains a detailed project schedule for this contract and some recommendations for breaking this work into separate contracts that would potentially reduce construction costs and time. These estimates contain the costs for preparing separate Feature Design Memorandum reports, hydraulic and physical modeling, subsurface investigations, preparation of plans and specifications and construction management.

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PANAMA CANAL CONCEPT DESIGN

Atlantic Locks Structure

Third Lane Lock

Appendix A

Project Design Criteria

Prepared for

Canal Capacity Projects Office

By

US Army Corps of Engineers®

Final Report
23 July 2003

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List of Abbreviations

ACI	American Concrete Institute
ACP	Autoridad Del Canal De Panama (Panama Canal Authority)
AASHTO	American Association of State Highway and Transportation Officials
AISC	American Institute of Steel Construction
ASCE	American Society of Civil Engineers
ASTM	American Society for Testing and Materials
AWS	American Welding Society
COE	Corps of Engineers
EC	Engineering Circular
EM	Engineering Manual
ER	Engineering Regulation
ETL	Engineering Technical Letter
PCA	Portland Cement Association
PTI	Post-Tensioning Institute
TM	Army Technical Manual

EXECUTIVE SUMMARY

This document identifies all the major design criteria to be applied in the development of the concept design of the Atlantic Lock Project. The original Panama Canal design presented criteria that was later used or modified to be used in many navigational projects throughout the United States by the Corps of Engineers. The application of the criteria has resulted in robust and safe designs. This document presents the Corps of Engineers design criteria for many features of the "Concept Design of the Atlantic Lock". The application of this criterion will fulfill expectations of high quality, durability, consistent operability, predictable performance, reduced scheduled maintenance, extended project longevity, and safety. This same philosophy was applied in the design of the original Panama Canal system and the benefits are readily apparent. The Panama Canal was named as one of the "Seven Wonders of the Modern World" by the American Society of Civil Engineers (ASCE) in its January 1997 issue. A quote from the article; "The society sought nominations from the civil engineering experts from around the globe. From their consensus emerged The Seven Wonders which were judged on such factors as pioneering of design and construction contributions to humanity and engineering challenges overcome". Although the existing lock structures are 88 years old, they are in good condition and will likely remain in service well into the twenty-first century. It is expected that the "state-of-art" criteria contained herein will result in equal or even improved performance for the Post Panamax Atlantic Lock.

These criteria have also been refined to reflect economic pressures to reduce initial project costs for major civil works projects without affecting the "built to last" philosophy. Recently, the Corps of Engineers experienced external pressure to reduce project costs for navigational projects. In response, several notable developments have emerged in the area of innovative design and construction. Resources were dedicated to develop and advance emerging technology and provide new engineering guidance. Specific work included research activities at laboratories (Innovative Navigation Projects), formation of special teams (Regional Navigation Design Team), and fast track development of design criteria. These coordinated advancements specifically address known maintenance and operability issues and resulting cost savings and have been incorporated into the proposed criteria for this project.

There is an increased risk that commercial and/or industrial criteria alone will not result in a product equal to the standards of high quality, durability, maintainability, operability, and safety set by the existing Panama Canal. Application of commercial and/or industrial criteria can result in a lower first cost, however the consequences will likely be significantly higher life cycle costs. Trading for low first costs, by accepting higher life cycle costs, is generally unacceptable for business with long-term perspectives.

In conclusion, the criteria presented herein is best suited for a civil works project that will be in service for a long time, a project similar to the existing Panama Canal Locks. We sincerely believe this to be consistent with ACP interests and are presenting that criteria herein.

1. INTRODUCTION

1.1. Purpose

The purpose of this report is to summarize the design criteria and methodology for the design of the Third Lane for the Panama Canal.

1.2. Project Description

This study represents a sub-set of efforts by the Panama Canal Authority (ACP) to supplement capacity of the existing Panama Canal system. This report is focused on general concept level design for two lock configurations at the Atlantic Ocean side of the Panama Canal. The two lock systems considered in this report include a new Triple-Lift single lock structure and a new Double-Lift lock structure. These two locks structures are designed to a concept level for the purpose of establishing basic configuration and budgetary funding needs. Two alternate filling and emptying systems are presented for each lock configuration. Water saving basins are provided for each design arrangement to conserve water use. Provisions for a Fourth Lane to the east of the Third Lane will be considered when developing concepts for the Third Lane as stated in the scope of work. The concept designs are based on the design criteria contained within this report.

1.3. Scope of Analyses and Design

The design criteria presented in this report will be applied to the design of the following features and study tasks:

- Alignment Studies
- Site Usage and Development
- Filling and Emptying Systems
- Water Saving Features
- Masonry Structures
- Lock Gates
- Valves and Bulkheads
- Cofferdams
- Mechanical Operating Equipment
- Electrical and Control Systems
- Operations Building

1.4. Related Documents

The following materials were used to obtain information on the project features:

Table A-1-1 Project Feature Document/References

Document/Reference
Available survey and geotechnical data
Harza Alignment Report including electron version of site maps
Coordination with appropriate Panamanian agencies and other associated Contract work as necessary.
Site geotechnical data and test results
Locations of preferred alignments
Moffat & Nichol Water Saving Basin report and/or information
Texas A & M Vessel Positioning Report
Tide and Gatun Lake data
1940 report of new lock hydraulic model study
Report on Locks pressure testing and wave run-up
Locks Squat Study of vessel movements
List of Gate Types (PIANC)
Electronic Mapping of A-2 Alignment
ACP Post-Panamax Workshop Documentation
Geologic Mapping
Requested Items: Drawing Border, Electronic CAD files of Plan with modified A-2 alignment, Electronic CAD files of Geologic Mapping, Lock Profiles with 50% WSB

2. HYDRAULIC DESIGN

2.1. References

Navigation locks will be designed in accordance with the criteria and guidance furnished in portions of the Corps of Engineers manuals for engineering and design, industry standards and in consideration of recent advancements and technology. A partial list of technical references follows.

Engineering Manuals:

- EM 1110-2-1602 — Hydraulic Design of Reservoir Outlet Works, 15 October 1980.
- EM 1110-2-1604 — Hydraulic Design of Navigation Locks, 30 June 1995.
- EM 1110-2-1610 — Hydraulic Design of Lock Culvert Valves, 15 August 1975.
- EM 1110-2-1611 — Layout and Design of Shallow-Draft Waterways, 31 December 1980.
- EM 1110-2-1613 — Hydraulic Design Guidance for Deep-Draft Navigation Projects, 8 April 1983.
- EM 1110-2-2602 — Planning and Design of Navigation Locks, 30 September 1995.
- EM 1110-2-2701 — Vertical Lift Gates, 30 November 1997.
- EM 1110-2-2702 — Design of Spillway Tainter Gates, 1 January 2000.
- EM 1110-2-2703 — Lock Gates and Operating Equipment, 30 June 1994.
- EM 1110-2-2902 — Conduits, Culverts, and Pipes, 31 March 1998.

Engineering Regulations:

- ER 1110-2-1150 — Engineering and Design for Civil Works Projects, 31 August 1999.
- ER 1110-2-1200 — Plans and Specifications for Civil Works Projects, 30 October 1993.
- ER 1110-2-1404 — Hydraulic Design of Deep-Draft Navigation Projects, 31 January 1996.
- ER 1110-2-1458 — Hydraulic Design of Shallow-Draft Navigation Projects, 30 April 1998.

Technical Publications:

- Hydraulic Design Criteria – Volumes 1 and 2, USACE, November 1987.

- Conceptual Design Study of Locks Water Saving Basins for Proposed Post-Panamax Locks at the Panama Canal, Moffatt & Nichols Engineers, undated.
- "Measurement of Pressures Related to Vessel Movement within Miraflores Upper West Lock", Pittsburgh District, USACE, 30 June 1999.
- Technical Note, "In-Chamber Longitudinal Culvert Design for Lock Filling and Emptying Systems", Preliminary Design Guidance, John E. Hite, ERDC, USACE, undated.
- Miscellaneous Paper H-75-7, "Lock Design, Sidewall Port Filling and Emptying System", Waterways Experiment Station, USACE, July 1975.
- Miscellaneous Paper H-76-13, "Lock Filling and Emptying – Symmetrical Systems", Waterways Experiment Station, USACE, 1976.
- Technical Note, "Modeling Surges in Navigation Lock Approaches", Richard Stockstill, ERDC/CHL CHETN-IX-2, USACE, 2000.
- Technical Note, "Simulation of Flow In Hydraulic Structures using ADH", Stockstill and Berger, ERDC/CHL CHETN-IX-4, USACE, 2000.
- Technical Note, "Modeling Navigation Conditions at Lock Approaches", Richard Stockstill, ERDC/CHL CHETN-IX-6, USACE, 2001.
- Technical Report INP-CHL-1, "Innovative Lock Design", Waterways Experiment Station, USACE, 1998.
- Technical Report ERDC/CHL TR-00-24, "New McAlpine Lock Filling and Emptying System, Ohio River, Kentucky", Engineer Research and Development Center, USACE, 2000.
- Technical Report ERDC/CHL TR, "Inner Harbor Navigation Canal Replacement Lock Filling and Emptying System, Louisiana", Engineer Research and Development Center, USACE, In Preparation.
- Technical Report HL-87-3, "Safe Navigation Speeds and Clearance at Lower Sill, Temporary Lock 52, Ohio River", Waterways Experiment Station, USACE, 1987.
- Technical Report HL-96-13, "Navigation Lock for Bonneville Dam, Columbia River, Oregon", Waterways Experiment Station, USACE, 1996.
- Technical Report ERDC/CHL TR-00-13, "Effects of Lock Sill and Chamber Depths on Transit Time of Shallow Draft Navigation", Engineer Research and Development Center, USACE, 2000.

<u>Miscellaneous References:</u>

- Concrete Pipe Handbook, American Concrete Pipe Association, Vienna, Virginia, 1981.

- Concrete Pipe Design Manual, American Concrete Pipe Association, Vienna, Virginia, 1980.
- Handbook of Hydraulics, Brater, E.F. et al, McGraw-Hill Book Company, 1996.
- Open-Channel Hydraulics, Chow, V.T., McGraw-Hill Book Company, 1959.
- Hydraulic Design of Navigation Locks, Davis, J.P., USACE, September 1989.
- ETL 1110-2-223, Navigation Lock Sill Depths and Hydraulic Loads on Gates, USACE, 1977.
- Navigation Locks for Push Tows, Kooman, C., Government Printing Office, The Hague-Netherlands, 1973.
- Internal Flow Systems, Miller, D.S., British Hydromechanics Research Association, 1978.
- The Locking of Ships with High Blockage Factors, Report to the National Ports Council by the British Transport Docks Board Research Station, National Ports Council, Southall, Middlesex, 1980.
- Final Report of the International Commission for the Study of Locks, Permanent International Association of Navigation Congresses, Brussels, 1986.
- User's Manual for LOCKSIM: Hydraulic Simulation of Navigation Lock Filling and Emptying Systems, Schohl, G.A., USACE, January 1999.
- Swirling Flow Problems at Intakes, Hydraulic Structures Design Manual, Knauss, J. (Ed.), International Association for Hydraulic Research, Rotterdam, Netherlands, 1987

2.2. Design Parameters

2.2.1. Design Ships

The design ships presented in Table A-2-1 were selected to be consistent with the deadweight specified in the terms of reference.

Table A-2-1 Design Ships

Vessel Type	Length [m]	Beam [m]	Draft [m]	Block Coefficient	Deadweight [t]	Displacement [t]
Container	385.7	54.9	15.2	0.65	125 000	209 000
Bulk Carrier	290.0	45.0	15.2	0.85	135 000	169 000

2.2.2. Design Water Surface Elevations

The design water surface elevations presented in Table A-2-2 were selected to be consistent with the range specified in the terms of reference.

Table A-2-2 Pool Operating Range

Range	Gatun Lake (m PLD)	Atlantic Ocean (m PLD)
Minimum	26.670	0.564
Average	25.908	0.061
Maximum	23.927	-0.381

2.2.3. Properties of Water

The properties of water presented in Table A-2-3 were selected based upon data published in previous studies prepared by Moffatt and Nichol Engineers for ACP.

Table A-2-3 Properties of Water

Location	Class	Salinity (PPT)	Density (kg/m^3)	Unit Weight (N/m^3)	Dynamic Viscosity ($N\text{-}s/m^2$)	Kinematic Viscosity (m^2/s)	Temperature (°C)
Gatun Lake and Upper Lock	Fresh	0	997.3	9783.5	9.16×10^{-4}	9.19×10^{-7}	24
Middle Lock	Lower Brackish	1	998.1	9790.9	9.18×10^{-4}	9.20×10^{-7}	24
Lower Lock	Low Brackish	4.5	1000.7	9817.1	9.26×10^{-4}	9.26×10^{-7}	24
Lower Approach	Brackish	10	1004.9	9858.4	9.39×10^{-4}	9.34×10^{-7}	24
Atlantic Ocean	Salt	33	1022.9	10034.5	9.92×10^{-4}	9.70×10^{-7}	24

2.3. Project Features

2.3.1. Lock Chambers

- Lock dimensions: 60.960 m wide, 426.720 m useable chamber length between inner gates, 457.200 m effective maximum useable chamber length, length between gates to be determined based upon type of gate
- Minimum operating sill clearance: 18.288 m
- Minimum hydraulic freeboard: 1.311 m
- Top of walls: Minimum hydraulic freeboard will be applied to the maximum operating water surface. Additional wall height may be required to accommodate structural and/or mechanical requirements.

2.3.2. Filling and Emptying Systems

- Target equalization time: 8-10 minutes for single lift of Triple-Lift configuration. Proportioned to 12-15 minutes for single lift of Double-Lift configuration.
- Culverts: Sized to meet target equalization times with use of water saving basins.
- Conduits: Sized to meet target equalization times with use of water saving basins. Size and number of conduits to be determined based upon compatibility with lock culverts, equalization times, and symmetry of filling and emptying system.
- Culvert valves: In general, same total area as lock culverts. Bifurcation of lock culvert valves may be recommended for certain filling and emptying systems.
- Conduit valves: Same size as water saving basin conduits
- Valve Loads: Hydrostatic head under normal conditions. Factor of 1.5 applied to static head to account for hydrodynamic effects under unusual conditions. The proposed valve loads are presented in Table A-2-4.

Table A-2-4 Valve Design Head Differential

Valve Location	Maximum Head Differentials Normal Operating Conditions [m]	
	Hydrodynamic	Hydrostatic
Upper lock fill valves	20.288	13.525
Valves between locks	40.576	27.051
Lower lock empty valves	20.288	13.525
Water saving basin valves	10.144	10.144

- Valve Schedule: Selected to meet target equalization times while satisfying hawser force criteria and mechanical requirements. In general, the valve schedule presented in Table A-2-5 will be used.

Table A-2-5 Valve Schedule

Valve Location	Operating Time Without WSBs (minute)	Operating Time With WSBs (minute)
Upper Lock Fill Valves	2	1
Valves Between Locks	4	2
Lower Lock Empty Valves	2	1
Water Saving Basin Valves	n/a	1

- Valve control: Lock culvert valves will provide one-way flow control. Water saving basin conduit valves will provide two-way flow control.
- Hawser forces: Consideration of existing conditions for Miraflores Locks and international criteria. Maximum allowable end-to-end water surface slope predicted by Locksim with an unoccupied chamber of 0.5/1000.
- Recycling: Provide provisions for recycling water from lower lock to upper lock.
- Redundancy, Lock Gates: Provide double set downstream
- Redundancy, Culvert Valves: Provide slots for future installation of backup valves

2.3.3. Water Saving Basins

- Configuration: Two basins per lock, two compartments per basin with closable connection
- Target water savings: 50%
- Target equalization time: 8-10 minutes for single lift of Triple-Lift configuration. Proportioned to 12-15 minutes for single lift of Double-Lift configuration.
- Surface area: Approximately equal to surface area of lock
- Floor elevations: To be determined based upon water savings and cost
- Minimum hydraulic freeboard: 1.000 m
- Minimum top of walls: Minimum hydraulic freeboard will be applied to the maximum operating water surface. Additional wall height may be required to accommodate structural and/or mechanical requirements.

3. GEOTECHNICAL FEATURES

3.1. References

The Geotechnical and Soil Mechanics features of this project shall be designed in accordance with the criteria and guidance furnished in portions of the Corps of Engineers manuals for engineering and design, industry standards and other technical references as follows:

Engineering Manuals:

- EM 1110-1-1804 — Geotechnical Investigations, 1 January 2001
- EM-1110-1-1904 — Settlement Analysis, 30 September 1990
- EM-1110-1-1905 — Bearing Capacity of Soils, 30 October 1992
- EM 1110-1-2907 — Rock Reinforcement, 15 February 1980
- EM 1110-1-2908 — Rock Foundations, 30 November 1994
- EM 1110-1-3500 — Chemical Grouting, 31 January 1995
- EM 1110-2-1421 — Groundwater Hydrology, 28 February 1999
- EM-1110-2-1901 — Seepage Analysis and Control for Dams, 30 April 1993
- EM-1110-2-1902 — Stability of Earth and Rock Fill Dams, 1 April 1970
- EM-1110-2-1906 — Laboratory Soils Testing, 20 April 1986
- EM-1110-2-1908 — Instrumentation of Embankment Dams and Levees, 30 June 1995
- EM-1110-2-2200 — Gravity Dam Design, 30 June 1995
- EM 1110-2-2301 — Test Quarries and Test Fills, 30 September 1994
- EM 1110-2-2302 — Construction With Large Stone, 20 October 1990
- EM-1110-2-2502 — Floodwalls and Retaining Walls, 29 September 1989
- EM 1110-2-2602 — Planning and Design of Navigation Locks, 30 September 1995
- EM 1110-2-2901 — Tunnels and Shafts in Rock, 30 May 1997
- EM 1110-2-2906 — Design of Pile Foundations, 15 January 1991
- EM 1110-2-3506 — Grouting Technology, 30 January 1984
- EM 1110-2-3800 — Systematic Drilling and Blasting for Surface Excavations, 1 March 1972
- EM 1110-2-6000 — Selection of Design Earthquake and Ground Motions,

Engineering Technical Letters:

- ETL 1110-2-282 Rock Mass Classification Data Requirements for Rippability, 30 June 1983
- ETL 1110-2-286 Use of Geotextiles under RipRap, 25 July 1984
- ETL 1110-2-310 Stability Criteria for Existing Concrete Navigation Structures on Rock Foundations, 17 December 1987
- ETL 1110-2-334 Design and Construction of Grouted RipRap, 21 August 1992
- ETL 1110-2-544 Geotechnical Analysis by the Finite Element Method, 31 July 1995

Engineering Regulations:

- ER 1110-2-1150 Engineering and Design for Civil Works Projects, 31 August 1999
- ER 1110-2-1802 Reporting Earthquake Effects, 25 July 1979
- ER 1110-2-1806 Earthquake Design & Evaluation of Civil Works Projects, 31 July 1995

Technical Publications:

- ASTM D420, Site Characterization for Engineering Design and Construction Purposes
- ASTM D653, Standard Terminology Relating to Soil, Rock and Contained Fluids
- TM 5-818-1, Soil & Geology Procedures for Foundation Design of Structures
- TM 5-818-4, Backfill for Subsurface Structures

Miscellaneous References:

- NAVFAC U.S. Naval Facilities Design Memorandum (DM-7)
- Historical data Construction of Gatun Locks and Dam
- Technical data 1940's Third Lock Project

3.2. Methodology

Geotechnical design analyses are based on the proposed project features, and the following design methodology.

3.2.1. Site Characterization

The engineering properties of the overburden and rock strata within the areas of the proposed third lock area shall be determined from interpretation and correlation of existing subsurface boring logs and testing data provided by the ACP.

3.2.2. Geotechnical Features

Geotechnical design features and technical requirements such as instrumentation, seepage cut-offs, granular filters, stability berms, and erosion and scour protection, shall be identified during the concept study.

3.2.3. Conceptual Design

Engineering analysis of stability, settlement, seepage, seismic loadings, foundation requirements, and the proposed geotechnical features, will be based on existing data, and available information. Presumptive values will be derived from empirical data when actual site data is not available. Engineering analyses shall be performed using detailed computations, computer aided design software, and shall be in accordance with the aforementioned Corps of Engineers design standards. References and sources of all geotechnical data used to assign soil and rock properties in this conceptual design study will be identified.

3.2.4. Final Design Requirements

Recommendations for the verification or collection of additional geotechnical data and laboratory testing which may be required to support the final design effort will be identified in the concept study, in addition to any reanalyses of project features that may be required.

3.3. Design Aids:

The concept design shall utilize the following Corps of Engineers (COE) proprietary software, in addition to commercially owned software* which is licensed to the Pittsburgh District.

Geotechnical Design Feature	Proposed COE Software
Slope Stability Analysis	UTEXAS4, STABL6
Seepage Analysis	CSEEP, CFRAG(W)
Settlement Analysis	CSET(W), CSANDSET(W)
Consolidation	FD31
Bearing Capacity Analysis	CBEAR
Pile Analysis	COM624G, CPGA, FB-Pier
Soil-Structure Interaction	CWALSSI(W)
Design Computations	Excel, MathCAD

- Golden Software Surfer Program – To be used for contouring and 3d mapping, incorporating data from Gemcom and gINT programs as provided by the ACP.
- Golden Software Grapher Program – To be used to produce working copies of geologic profiles for later refinement with Insitu CADD package

- Insitu Geologic CADD Drawing Software – Used to incorporate CADD files from the other CADD software into the format used by Microstation.

4. CONCRETE MATERIALS

4.1. References

Concrete will be designed to meet the project requirements, maximize service life, control thermal effects, optimize economy, and facilitate constructability. Concrete materials will be selected to achieve these objectives. Concrete and concrete material recommendations will be in accordance with the criteria and guidance furnished in portions of the Corps of Engineers publications, industry standards and other technical references as listed below. Corps of Engineers Engineering Manuals will provide basic guidance. Various sections of the American Concrete Institute "Manual of Concrete Practice", and Portland Cement Association "Design and Control of Concrete Mixtures" will also be utilized for guidance as appropriate. The primary sections for consideration are shown below. In addition, current state-of-the-art practices will be utilized.

Engineering Manuals:

- EM 1110-2-2000 — Standard Practice for Concrete for Civil Works Structures, 1 February 1994, Rev. 31 March 2001
- EM 1110-2-2006 — Roller-Compacted Concrete, 15 January 2000
- EM 1110-2-2102 — Waterstops and Other Preformed Joint Materials for Civil Works Structures, 30 September 1995

Engineering Technical Letters:

- ETL 1110-2-365 — Nonlinear Incremental Structural Analysis of Massive Concrete Structures, 31 August 1994
- ETL 1110-2-542 — Thermal Studies of Mass Concrete Structures, 30 May 1997

Engineering Regulations :

- ER 1110-1-2002 — Cement, Slag, and Pozzolan Acceptance Testing, 30 September 1998

Technical Publications:

- ACI 201.2R-01 — Guide to Durable Concrete
- ACI 207.1R-96 — Mass Concrete
- ACI 207.2R-95 — Effect of Restraint, Volume Change, and Reinforcement on Cracking of Mass Concrete
- ACI 207.5R-99 — Roller Compacted Mass Concrete
- ACI 209R-92 — Prediction of Creep, Shrinkage, and Temperature Effects in Concrete Structures (Reapproved 1997)

- ACI 211.1-91 — Standard Practice for Selecting Proportions for Normal, Heavyweight, and Mass Concrete (Reapproved 1997)
- ACI 212.3R-91 — Chemical Admixtures for Concrete
- ACI 212.4R-93 — Guide for the Use of High-Range Water-Reducing Admixtures (Superplasticizers) in Concrete
- ACI 229R-99 — Controlled Low Strength Materials (CLSM)
- ACI 318R-02 — Building Code Requirements for Structural Concrete
- ACI 336.1R-01 — Standard Specifications for the Construction of Drilled Piers
- ACI 336.3R-93 — Design and Construction of Drilled Piers
- ACI 504R-90 — Guide to Sealing Joints in Concrete Structure (Reapproved 1997)
- ADSC — ADSC Standards and Specifications for the Foundation Drilling Industry (1999)
- ASTM S169C — Concrete and Concrete-Making Materials (1994)
- PCA — Design and Control of Concrete Mixtures (13th Ed.)

4.2. Material Considerations

Concrete material considerations will be reviewed based on available information from previous Third Lock Studies, current industry information, and information provided by the Panama Canal Authority (ACP). Points of contact in the ACP organization, as well as local contacts will be the basis for this review. These items include:

- Discussion of potential sources of cement and coarse and fine aggregate.
- Quality of canal water and groundwater with respect to concrete exposure.
- Water quality for concrete mixing and curing.
- Cement, pozzolan and ground-granulated blast-furnace slag restrictions based on material properties and availability.
- Aggregate restrictions based on reactivity, durability, quality and availability.
- Availability of chemical admixtures.
- Availability and quality of mineral admixtures.
- Concrete mixture designs, required properties and material information on existing structures.

5. STRUCTURAL FEATURES

5.1. References

The structures are designed in accordance with the criteria and guidance furnished in portions of the Corps of Engineers manuals for engineering and design standards, industry standards and other technical references as follows:

Engineering Manuals:

- EM 385-1-1 Safety and Health Requirements Manual, 3 September 1996
- EM 1110-1-1905 Bearing Capacity of Soil, 30 October 1992.
- EM 1110-1-2907 Rock Reinforcement, 15 February 1980.
- EM 1110-1-2908 Rock Foundations, 30 November 1994.
- EM 1110-2-1604 Hydraulic Design of Navigation Locks, 30 June 1995.
- EM 1110-2-1610 Hydraulic Design of Lock Culvert Valves, 15 August 1975.
- EM 1110-2-1611 Layout and Design of Shallow-Draft Waterways, 31 December 1980.
- EM 1110-2-2000 Standard Practice for Concrete for Civil Works Structures, 1 February 1994, Rev. 31 March 2001
- EM 1110-2-2100 Draft - Stability Analysis of Concrete Structures, May 2001
- EM 1110-2-2102 Waterstops and Other Preformed Joint Materials, 30 September 1995.
- EM 1110-2-2104 Strength Design for Reinforced Concrete Hydraulic Structures, 30 June 1992.
- EM 1110-2-2105 Design of Hydraulic Steel Structures, 31 May 1994.
- EM 1110-2-2200 Gravity Dam Design, 30 June 1995.
- EM 1110-2-2502 Retaining and Flood Walls, 29 September 1989.
- EM 1110-2-2503 Design of Sheet Pile Cellular Structures, Cofferdams and Retaining Structures, 29 September 1989.
- EM 1110-2-2504 Design of Sheet Pile Walls, 31 March 1994.
- EM 1110-2-2602 Planning and Design of Navigation Locks, 30 September 1995.
- EM 1110-2-2608 Navigation Locks Fire Protection Provisions, 28 February 1994.

- EM 1110-2-2703 Lock Gates and Operating Equipment, 30 June 1994.
- EM 1110-2-2902 Conduits, Culverts and Pipes, 31 October 1997, Rev. 31 March 1998.
- EM 1110-2-2906 Design of Pile Foundations, 15 January 1991.
- EM 1110-2-3001 Planning and Design of Hydroelectric Power Plant Structures, 30 April 1995.
- EM 1110-2-3400 Painting: New Construction and Maintenance, 30 April 1995.

Engineering Technical Letters:

- ETL 1110-2-256 Sliding Stability for Concrete Structures, June 1981.
- ETL 1110-2-307 Flotation Stability Criteria for Concrete Hydraulic Structures, 20 August 1987.
- ETL 1110-2-310 Stability Criteria for Existing Concrete Navigation Structures on Rock Foundations, 17 December 1987.
- ETL 1110-2-338 Barge Impact Analysis, 30 April 1993.
- ETL 1110-2-343 Structural Design Using the Roller-Compacted Concrete Construction Process, 31 May 1993.
- ETL 1110-2-365 Nonlinear, Incremental Structural Analysis of Massive Concrete Structures, 31 August 1994.

Engineering Circulars:

- EC 1110-2-291 Stability Analysis of Concrete Structures, 31 October 1997.
- EC 1110-2-6052 Structural Design of Precast and Prestressed Concrete for Offsite Prefabricated Construction of Hydraulic Structures, 1 January 2001

Engineering Regulations:

- ER 1110-2-1150 Engineering and Design of Civil Works Projects, 31 August 1999.
- ER 1110-2-1200 Plans and Specifications for Civil Works Projects, 30 October 1993.
- ER 1110-2-1458 Hydraulic Design of Shallow Draft Navigation Projects, 30 April 1998.
- ER 1110-2-1806 Earthquake Design and Evaluation for Civil Works Projects, 31 July 1995.
- ER 1110-2-8152 Planning and Design of Temporary Cofferdams and Braced Excavations, 31 August 1994.

• ER 1110-2-8157	Responsibility for Hydraulic Steel Structures, 31 January 1997.

Industry Codes and Recommendations:

• AASHTO	American Association of State Highway and Transportation Officials, Guide Specification and Commentary for Vessel Collision Design of Highway Bridges, 16th Ed., Interim 1998.
• AASHTO	American Association of State Highway and Transportation Officials, Standard Specifications for Highway Bridges, 16th Edition, Interim 1998.
• ACI 318M-99.	American Concrete Institute, Building Code Requirements for Reinforced Concrete.
• ACI 224R-90	American Concrete Institute, Control of Cracking in Concrete Structures.
• ACI 350R-89	American Concrete Institute, Environmental Engineering Concrete Structures.
• AISC ASD	American Institute of Steel Construction, Manual of Steel Construction, Allowable Stress Design, 9th Edition, 1989.
• AISC LRFD	American Institute of Steel Construction, Manual of Steel Construction, Load & Resistance Factor Design, 1994.
• ANSI/ASCE 7-95	American Society of Civil Engineers, Minimum Design Loads for Buildings and Other Structures.
• AWS-D 1.1-98	American Welding Society, Structural Welding Code, Steel.
• PTI	Post-Tensioning Institute, Post Tensioning Manual, 5th Edition.
• PTI	Post-Tensioning Institute, Recommendations for Prestressed Rock and Soil Anchors, 1996.

Miscellaneous References:

- Technical Manual TM-5-809-1 Structural Design Criteria for Loads, June 1992.
- REMR-CS-41Comparison of Cast-in-Place Concrete Versus Precast Stay-in-Place Forming Systems for Lock Wall Rehabilitation, October 1993.
- ERDC/ITL TR-01-6, Tri-Service CADD/GIS Standards, Release 2.0
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5.2. Project Features

5.2.1. Lock Gates

An initial screening of various gate types will be performed based on the referenced design criteria, engineering judgment, and adoption of features from similar projects. A separate design report will be developed to summarize findings. Specific recommendations will be presented and coordinated with the ACP to define the direction of following study efforts. Upon completion and concurrence from the ACP on the gate selection study the narrative discussions provided in this section will be replaced with project specific design criteria.

The basic considerations to be applied in the gate selection process include:

- Continuity of service – Gates with the least amount of underwater mechanical roller systems are desirable. Where underwater mechanical systems are used the degree of effort to perform service must be minimized. They must be serviceable for maintenance and repair from damage and easily replaced or changed out within a reasonable period of time. Due to the size and mass of the gates, handling and transport to maintenance facilities should be minimized.
- Preserving Gatun Lake storage – Although not as critical at other locations within the system, the Lake gates must be able to maintain lake levels under all conditions. They should be able to close under full discharge through the lock unless other emergency closure systems or double gate operations are provided. Gates at the Atlantic Ocean side should tolerate reverse loading from tidal variations.
- Reliability and durability – As discussed in the Scope of Work, an evaluation of fabrication processes will be performed to determine if welded, bolted, or riveted construction is preferred.
- Integration with lock masonry features – Each gate type has a determinable ineffective length, that is, clear operational chamber space where vessels can not occupy and required gate arrangement features that include gate sills, monolith or housing structures. Cursory cost estimating will be performed on lock closure structures, operating equipment, concrete and excavation to evaluate the implication of gate type on overall economy. Impacts to water demands will be addressed through narrative discussions.

A cursory level of design and analysis will be performed in the gate selection study. However, primary members will be sized to prove feasibility of concepts. The basic sources of loading will include the following for strength and serviceability analysis:

- Hydraulic Loads
- Gravity Loads
- Operating Loads
- Ship Impact
- Earthquake

5.2.1.1. Miter Gates Load And Resistance Factor Design Criteria

This section presents the criteria used in the concept design of the mitering lock gates proposed for the Panama Locks Third Lane Concept Design for the Atlantic Ocean Side Locks. The basic structural systems of the lock gates are shown schematically on Report Drawings Series ACP-R-21. The criteria include specifications of applied loads, load cases, load combinations, and analysis methods used in the design process. Design methods described herein are in accordance with those given in references. The design equations presented are in English Units. Conversion to Metric Units was avoided for maintain consistency with establish Corps criteria which exists only in English Units. Minimum plate width and thickness have been converted to metric.

5.2.1.1.1. Design Basis

Load and Resistance Factor Design (LRFD) is a method of proportioning structures in such a manner that no applicable limit state is exceeded when the structure is subjected to all appropriate design load combinations. The basic safety check in LRFD may be expressed mathematically as $\Sigma\gamma_i Q_{ni} \leq \alpha\ \varphi R_n$ where Q_{ni} represents nominal (code-specified) load effects, and γ_i denotes the load factors that account for variability in the loads to which they are assigned. The expression $\Sigma\gamma_i Q_{ni}$ is the required strength. Load factors and load combinations for miter gate design are listed later in this chapter. R_n is the nominal resistance, and φ is a resistance factor that reflects the uncertainty in the resistance for the particular limit state and, in a relative sense, the consequence of attaining the limit state. The factor α is a reliability factor of 0.9 for Hydraulic Steel Structures (HSS).

5.2.1.1.2. Gate Properties

a General Geometry

The general geometry of the proposed horizontally framed miter gates is divided into two groups: basic structural elements that include the skin plate, intercostals, and plate girders that comprise the majority of the weight in the gate; and detailed structural elements that include the tapered end section, end diaphragms, quoin post, thrust diaphragm, and diagonals (function of diagonals is replaced with plating in the case of the double skin gate alternative).

b Structural Steel

The lock gates would be constructed of low-alloy steels with a yield points of 345 and 450 MPa.

5.2.1.1.3. Loads And Reactions

The principal loads applicable to the Panama miter gate design would be gravity, hydraulic, operating, ship impact, and earthquake loads. Reactions and load cases are generally considered in two categories: gate in the open or intermediate position with no pool differential; and with leaves mitered and supporting the full hydrostatic load.

a Vertical loads

Vertical loads consist of the dead load of the gate, live loads applied to the walkway or bridge at the top of gate and an allowance for mud or excess water in buoyancy chambers.

b Hydraulic loads

Hydraulic loads consist of static water load on the gate produced by the pool differential or temporal hydraulic loads (see Report Drawings ACP-R-21/1 to 3). The effect of temporal loads due to waves, surges, etc., on the gate has been evaluated with appropriate conditions selected. A minimum temporal hydraulic load of 0.23 m acting from the full submergence elevation down to the sill, with a period exceeding thirty seconds (thus considered static), is specified for the gate structural systems, temporal hydraulic loads for gate operating equipment is address in Mechanic Section.

c Operating loads

Q is the maximum load exerted by the operating machinery (obtained from the mechanical design calculations) and is considered for cases in which the gate is held by a submerged obstruction.

d Ship impact loads

Ship impact load is a dynamic load applied to the gate when struck by a vessel. The impact load is usually specified as a point load and it is applied to the girders above the pool level in the downstream direction at the miter point (symmetrical impact); and anywhere in the girder span at which an impact is possible (unsymmetrical impact). This location is anywhere in the span as defined for possible impact conditions. The impact load is for this project is unknown, therefore, back calculations are have been performed to determine the maximum impact force associated with the design shown.

The equations used to find the axial and flexure loads are:

i. Symmetrical impact

$$P = \frac{5I}{\sqrt{10}};$$

$$M = -Pe$$

ii. Unsymmetrical impact

$$P = \frac{(4x + a)I}{\sqrt{10}\, a}$$

$$M = \left[\frac{Ix(a - x)}{a}\right] - Pe$$

Where:

P = Axial loads

M = Flexure load

I = Impact load

x = Distance between the quoin contact point and the unsymmetrical impact load position.

a = Distance between quoin contact point and miter contact.

e = Distance between work line and neutral axis of the girder.

e Earthquake loads

Design loads are specified based on an operational basis earthquake (OBE) having 50 percent chance of being exceeded in 100 years. This translates to a probability of annual exceedance of 0.0069, or approximately 145 years mean recurrence interval. The earthquake load is based on inertial hydrodynamic effects of water moving with the structure. This is determined based on Westergaard's equation:

$$p = \frac{7}{8} \gamma_w a_c \sqrt{Hy}$$

Where **p** is the lateral pressure at a distance **y** below the pool surface, γ_w is the unit weight of water, **H** is the pool depth, and $\mathbf{a_c}$ (expressed as a fraction of gravitational acceleration **g**) is the maximum acceleration of the supporting lock wall due to the selected ground motion. For typical Corps projects, an Operational Basis Earthquake is usually used. However, because of concerns with continuing operating of the lock following a major earthquake, a Maximum Design Earthquake has been adopted for the concept design. The lock wall is assumed to be rigid in determination of $\mathbf{a_c}$, and the assumed direction of $\mathbf{a_c}$ is parallel to the lock centerline. The inertia forces resulting from the mass due to structural weight are not included in the earthquake load because the magnitudes of these loads are insignificant compared to the hydrodynamic loads obtained from Westergaard's equation.

f Load Combinations

The miter gates are proportioned to have design strengths at all sections at least equal to the required strengths calculated for the factored loads and forces in the following load combinations:

$$1.4 H_s + 1.0 I$$

$$1.4 H_s + 1.0 H_t$$

$$1.2D + 1.6M + 1.0 H_t$$

$$1.2D + 1.6M + 1.2Q$$

$$1.2 H_s + 1.0E$$

where:

D = dead loads

Q = maximum operating load

E = earthquake load

I = impact load

M = mud load

$\mathbf{H_s}$ = hydrostatic loads

$\mathbf{H_t}$ = temporal hydraulic load

g Load Cases

The following load cases are used with the appropriate loading combinations:

i. Case 1: Mitered Condition

Loads include hydrostatic loads due to upper and lower pools and impact or temporal hydraulic loads. Loads *D*, and *M* act when the gate is in the mitered position. However, in the mitered position the effects of *D*, and *M* will not control the member sizes. Loads *D*, and *M* are accounted for in load Case 2 in which they may be controlling.

(a) Above Pool. The first load combination is applicable to the girders located above pool (upper pool elevation for the upper gate and lower pool for the lower gate) where minor impact may occur. The skin plate and intercostals are not design for ship impact. For design of skin plate and intercostals located above pool, a minimum hydrostatic head of 2 m is assumed.

(b) Below Pool. The upper gate is designed assuming that the lock is dewatered. Loads include hydrostatic loads due to upper pool only (load case 2 with $H_t = 0$). The lower gate is designed considering normal upper and lower pool elevations including temporal hydraulic load H_t. H_t is applicable only to the submerged part of the gate.

ii. Case 2: Gate Torsion

Loads include gravity loads (*M*, and *D*) and operating equipment load *Q* or temporal hydraulic load H_t . In this condition there are no differential hydrostatic loads.

(a) Temporal Condition. The third load combination is applied to consider gate leaf torsion with the temporal hydraulic load acting on the submerged part of leaf (the temporal hydraulic load may act in either direction).

(b) Submerged Obstruction. The fourth load combination is applied to consider leaf torsion that may be caused by a submerged obstruction. For this case, it is assumed that the bottom of the leaf is held stationary by a submerged obstruction while *Q* is applied causing the gate leaf to twist.

iii. Case 3: Earthquake

The last load combination is applied if the gate is mitered and hydrostatic loads due to upper and lower pools are acting. The earthquake acceleration is to be applied in the direction parallel to the lock centerline. Elastic structural analysis is performed with no allowance for ductility. The maximum design earthquake should be investigated for inelastic behavior in following studies where some plastic deformation is tolerable provided it is localized and does not result in failure of the gate systems.

5.2.1.1.4. Analysis Procedure

The Corps of Engineer's proprietary lock gate design software, CMITER-LRFD was used in the design of the lock gates. Several modifications were made to accommodate various aspects of this project. The structural design specifications, analysis assumptions, and procedures used to analyze and investigate the major structural elements of a horizontally framed miter gate are as follows:

a Skin plate

Skin plate is analyzed as a rectangular flat plate with all edges fixed. A uniform load equal to the water pressure at the center of the panel under consideration is assumed to act over the entire surface.

The skin plate is to be sized using: (1) the yield stress criteria, where the maximum calculated stress should be less than the yield limit state of $\alpha\Phi_b F_y$ (where **α** is a reliability factor for HSS and has a value of 0.9 as defined earlier, **Φ_b** is the bending resistance factor equal to 0.9 as defined earlier, and **F_y** is the yield stress); (2) deflection criteria, where the maximum deflection allowable is **0.4t**, where **t** is the skin plate thickness; and (3) fatigue criteria, where the maximum calculated stress range shall be less than the fatigue limit **Fr.** The stress and deflection are calculated using the following equations:

$$F_N = \frac{0.5W\,b^2}{t^2\left[1+.623\left(\frac{b}{a}\right)^6\right]}$$

$$\delta = \frac{0.0284\,Wb^4}{\left[1+1.056\left(\frac{b}{a}\right)^5\right]Et^3}$$

where **F_N** is the nominal stress ($\leq \alpha\Phi_b F_y$ for yield stress criteria and **F_N** $\leq F_r$ for fatigue criteria), **W** is the factored uniform load (**W_u**) for yield stress criteria and unfactored uniform hydrostatic loads (**W**) for fatigue criteria, **t** is the plate thickness, **a** is the larger plate dimension, **b** is the smaller plate dimension, **E** is the modulus of elasticity, and **δ** is the deflection (**$\delta\langle\delta_{max}$=0.4t).** The minimum size of the skin plate above the pool level should be determined using an assumed hydrostatic head of 2 m.

b Intercostals

Intercostals would be flat plates or T sections, sized in such a manner that the maximum calculated moment is less than the nominal bending strength $\alpha\Phi_b M_n$. Intercostals are configured as vertical fixed-end beams with supports at the centerline of girder webs. The intercostals may be designed as simple or fixed-end beams. An effective width of the skin plate is assumed to act with the intercostal plate or T section, producing a T section when interacting with a plate and an I section when interacting with a T section. The effective width of the skin plate on each side of the plate or T section is calculated using the width-to-thickness ratio of $\frac{95}{\sqrt{F_y}}$. The loading, assuming average water pressure, begins at the edge of the girder flange or a maximum of 6 in. from the centerline of the girder web. The assumed loading is applied over an area with the boundary lines being 45 degrees from the point at the edge of the flange (or 6 in.), and half the distance between intercostals. These boundary lines intersect with the load line boundaries from adjacent panels at a point midway between intercostals, thereby forming an effective load area of two triangular areas, one at each end, usually with a rectangular section in the center. The bending strength of the intercostals is the plastic moment for a simply supported beam since the skin plate supports the compression flange continuously. For fixed and pin ends, the bending strength can be found by use of the following equations:

a. For T beams loaded in the plane of symmetry and bending about the major axis, with flange and web slenderness ratios less than the corresponding values of λ_r.

$$M_n = M_{cr} = \frac{C_b \pi \sqrt{E I_y G J}}{L_b}\left[B + \sqrt{1 + B^2}\right] \le M_y$$

where

$$B = \pm 2.3\left(\frac{d}{L_b}\right)\sqrt{\frac{I_y}{J}}$$

C_b = Bending coefficient dependent upon moment gradient. A unit value is used for intercostal design to represent the most severe loading case.

E = Modulus of elasticity of steel (29,000 ksi)

I_y = Moment of inertia about y-axis, in.^{4}

G = Shear modulus of elasticity of steel, ksi

J = Torsional constant, in.^{4}

L_b = Laterally unbraced length

M_y = Initial yield bending moment, kip-in.

d = stem height

The plus sign for B applies when the stem is in tension, and the minus sign applies when the stem is in compression.

b. For doubly and singly symmetrical I-shaped beams bending about the major axis, the nominal flexure strength M_n is the lowest value obtained according to the limit state of: (a) lateral torsional buckling (LTB); (b) flange local buckling (FLB); and (c) web local buckling (WLB).

The nominal flexural strength shall be calculated using the criteria discussed earlier.

c Horizontal Girders

The horizontal girders are, in effect, a series of three hinged arches that transmit the water pressures to the lock walls through the quoin blocks. Because they are subject to combined bending and axial loads, they should be designed through use of the beam-column criteria.

The following is a list of criteria and assumptions that are used in the design of horizontal girders:

i. Girder Analysis. The basic procedure for girder analysis is to assume that each girder is isolated as an individual member. Each member is designed as beam-column elements using the criteria discussed in this chapter.

ii. Axial, flexure, and shear strength. The axial, flexure, and shear strengths are calculated using the criteria discussed in this chapter with the following assumptions:

(a) Upstream and downstream girder flanges are braced continuously by skin plate.

(b) The length of girders considering buckling about the major axis (in the plane of the web) is the distance between the quoin block and miter block (l_x). The ends are assumed pinned, and the value of K is 1.0.

(c) The length of girders considering buckling about the minor axis is the distance between intermediate diaphragms (l_y). The ends are assumed fixed with K=0.65.

(d) The design strength of compression members whose elements have width-to-thickness ratios less than λ_r of Table A-5-1 is $\alpha\Phi_c P_n$

Table A-5-1 Width to Thickness Ratios

Member	Compact Section	Noncompact Section	Slender Section
Flanges	$\frac{b}{t} - \frac{65}{\sqrt{F_y}}$	$\frac{b}{t} - \frac{106}{\sqrt{F_{yw} - 16.5}}$	$\frac{b}{t} > \frac{106}{\sqrt{F_{yw} - 16.5}}$
Web	$\frac{h_c}{t_w} - \frac{253}{\sqrt{F_y}}$	Not applicable	$\frac{h_c}{t_w} > \frac{253}{\sqrt{F_y}}$
Skin plate	$\frac{b}{t} - \frac{65}{\sqrt{F_y}}$	$\frac{b}{t} - \frac{106}{\sqrt{F_{yw} - 16.5}}$	$\frac{b}{t} > \frac{106}{\sqrt{F_{yw} - 16.5}}$

$\Phi_c = 0.85$

$P_n = A_g F_{cr}$

for $\lambda_c \leq 1.5$

$$F_{cr} = (0.658^{\lambda_c^2}) F_y$$

for $\lambda_c \rangle 1.5$

$$F_{cr} = \left[\frac{0.877}{\lambda_c^2}\right] F_y$$

where:

Φ_c = resistance factor for compression, Φ = 0.85

P_n = nominal compressive strength, kips

F_{cr} = critical stress, ksi.

A_g = gross area of member, in.^{2}

F_y = specified yield stress, ksi

$$\lambda_c = \frac{Kl}{r\pi}\sqrt{\frac{F_y}{E}}$$

where

E = modulus of elasticity, ksi

K = effective length factor

l = unbraced length of member, in.

r = governing radius of gyration about plane of buckling, in.

(e) The nominal flexure strength M_n is the lowest value obtained according to the limit state of lateral torsional buckling (LTB), flange local buckling (FLB) and web local buckling (WLB). The flexure design strength is $\alpha\Phi_b M_n$, and the nominal flexural strength M_n shall be determined as follows for each limit state:

for $\lambda \le \lambda_p$:

$$M_n = M_p$$

For $\lambda_p \langle \lambda \le \lambda_r$ (for limit state of lateral torsional buckling)

$$M_n = C_b\left[M_p(M_p M_r)\left(\frac{\lambda \lambda_p}{\lambda_r \lambda_p}\right)\right] \le M_p$$

(for limit state of flange and web local buckling)

$$M_n = M_p(M_p M_r)\left(\frac{\lambda \lambda_p}{\lambda_r \lambda_p}\right)$$

for $\lambda \rangle \lambda_r$:

$$M_n = M_{cr} = SF_{cr}$$

Where:

$$\lambda = \frac{L_b}{r_y} \text{ (for lateral torsional buckling)}$$

$$\lambda = \frac{b}{t} \text{ (for flange local buckling)}$$

$$\lambda = \frac{h_c}{t_w} \text{ (for web local buckling)}$$

$$\lambda_p = \frac{300}{\sqrt{F_{yf}}} \text{ for LTB}$$

$$\lambda_p = \frac{65}{\sqrt{F_{yf}}} \text{ for FLB}$$

$$\lambda_p = \frac{640}{\sqrt{F_{yf}}} \text{ for } WLB$$

$$\lambda_r = \frac{X_1}{(F_{yw} F_r)} \sqrt{1 + \sqrt{1 + X_2 (F_{yw} F_r)^2}} \text{ for } LTB$$

$$X_1 = \frac{\pi}{S_x} \sqrt{\frac{EGJA}{2}}$$

$$X_2 = 4 \frac{C_w}{I_y} \left(\frac{S_x}{GJ} \right)^2$$

$$\lambda_r = \frac{106}{\sqrt{F_{yw} 16.5}} \text{ for } FLB$$

$$M_r = (F_{yw} F_r) S_{xc} \le F_{yf} S_{xt} \text{ for } LTB$$

$$M_r = (F_{yw} F_r) S_{xc} \text{ for } FLB$$

$$M_r = R_e F_{yf} S_x \text{ for } WLB$$

$$R_e = 1.0 \quad 0.1(1.3 + a_r)(0.81 \quad m) \le 1.0$$

$$F_{cr} = \frac{C_b X_1 \sqrt{2}}{S_{xc} \lambda} \sqrt{1 + \frac{X_1^2 X_2}{2 \lambda^2}} \text{ for } LTB$$

$$F_{cr} = \frac{11{,}200}{\lambda^2} \text{ for } FLB$$

The terms used in the above equations are:

M_n = nominal flexural strength, kip-in.

M_p = plastic moment, kip-in.

M_{cr} = buckling moment, kip-in.

M_r = limiting buckling moment, kip-in.

λ = controlling slenderness parameters

λ_p = largest value of λ for which $M_n = M_p$

λr = largest value of λ for which buckling is inelastic

C_b = bending coefficient dependent upon moment gradient. (A unit value is used for girders designed to represent the most severe loading case.)

S = section modulus, in^3.

L_b = laterally unbraced length, in.

r_y = radius of gyration about minor axis, in.

A = cross section area, in^2.

F_r = compressive residual stress in flange, ksi.

F_y = specified minimum yield strength, ksi.

F_{yf} = yield strength of the flange, ksi.

F_{yw} = yield strength of the web, ksi.

J = torsional constant, in^4.

S_{xc} = section modulus of the outside fiber of the compression flange, in^3.

S_{xt} = section modulus of the outside fiber of the tension flange, in^3.

b = flange width, in.

d = overall depth, in.

h_c = twice the distance from the neutral axis to the inside face of the compression flange less the fillet or corner radius, in.

t_f = flange thickness, in.

t_w = web thickness, in.

R_e = hybrid girder factor.

a_r = ratio of web area to compression flange area.

m = ratio of web yield stress to flange yield stress or F_{cr}

(f) The interaction of flexure and compression in symmetrical shapes shall be limited by the following formulas:

$$for \frac{P_u}{\alpha \phi_c P_n} < 0.2$$

$$\frac{P_u}{\alpha\phi_c P_n}+\frac{8}{9}\left(\frac{M_{ux}}{\alpha\phi_b M_{nx}}+\frac{M_{uy}}{\alpha\phi_b M_{ny}}\right)\le 1.0$$

$$for\frac{P_u}{\alpha\phi_c P_n}\ge 0.2$$

$$\frac{P_u}{2\alpha\phi_c P_n}+\left(\frac{M_{ux}}{\alpha\phi_b M_{nx}}+\frac{M_{uy}}{\alpha\phi_b M_{ny}}\right)\le 1.0$$

where:

P_u = required compressive strength, kips

M_u = required flexural strength, Kip-in.

$M_u = B_1\ M_{nt}$

M_{nt} = required flexural strength in member assuming there is no lateral translation of the frame, kip-in.

M_n = nominal flexural strength, Kip-in.

Φ_b = resistance factor for flexure = 0.90

α = reliability factor for HSS

$$B_1=\frac{C_m}{\left(1\frac{P_u}{P_e}\right)}\ge 1$$

$$P_e=\frac{A_g F_y}{\lambda_c^{\ 2}}$$

where:

C_m = 1.0

A_g = gross area of member, in.^{2}

λ_c = as defined in section to calculate axial strength of compression members.

(g) The design shear strength for webs is $\alpha\Phi_v V_n$, where α = 0.9, Φ_v = 0.9, and the nominal shear strength V_n is determined as follows:

$$for\ \frac{h}{t_w} \leq 187\sqrt{\frac{K}{F_{yw}}}$$

$$V_n = 0.6\ F_{yw} A_w$$

$$for\ \sqrt{\frac{K}{F_y}} < \frac{h}{t_w} \leq 234\sqrt{\frac{K}{F_y}}$$

$$V_n = 0.6\ F_{yw} A_w \frac{187\sqrt{\frac{K}{F_{yw}}}}{\frac{h}{t_w}}$$

$$for\ \frac{h}{t_w} > 234\sqrt{\frac{K}{F_y}}$$

$$V_n = A_w \frac{26{,}400K}{\left(\frac{h}{t_w}\right)^2}$$

The web plate buckling coefficient K is given by

$$K = 5 + \frac{5}{\left(\frac{a}{h}\right)^2}$$

except that K shall be taken as 5 if a/h exceeds 3 or $[260/(h/t_w)]^2$. When stiffeners are not required, K=5. In unstiffened girders, h/t shall not exceed 260

where:

A_w = cross section area of the web, in^2.

F_{yw} = yield strength of the web, ksi.

t_w = web thickness, in.

h = clear distance between flanges, in.

a = clear distance between transverse stiffeners.

iii. Equations to find the axial, flexure, and reaction loads are:

$$P2 = Wt$$

$$P = P1 + P2$$

$$P1 = \frac{WL}{2}\cot\theta$$

$$M_x = \frac{W}{2}[LxLacot\theta + (ta)^2 a^2 x^2]$$

$$N = \frac{WL}{2}$$

where:

P1 = axial loads due to the reaction at the lock wall

P2 = axial loads due to the uniform load

W = uniform loads

L = girder length

θ = miter angle

t = distance between work line and the outside face of the upstream flanges

P = maximum resultant compression load

a = distance between the work line and the neutral axis

M_x = flexure load at distance x from the miter contact point

N = girder reaction perpendicular to the girder axial axis (maximum shear load)

iv. The minimum flange widths are to be 200 mm for upstream flanges and 300 mm for downstream flanges. The minimum thickness as are specified as 10 mm for webs and 13 mm for flanges.

v. The downstream flanges are limited to 610 mm or $24t_f$ (t_f is the flange thickness), reducing the possibility of the flange being undesirably wide and thin. The maximum change in flange width on the same edge of a girder web is 150 mm, with a 75 mm differential on each edge of the flange except for the downstream flange of the bottom girder, where the total 150 mm of differential may be on the upper edge of the flange. This applies between the sections at the centerline of a girder, where the upstream flange is maximum width and the downstream flange is minimum width. This also applies at a section at the end of the girder where the upstream flange is a minimum width and the downstream flange is a maximum width. The bottom girder is a special case in which the downstream flange is a maximum of 300 mm and a minimum of 220 mm, with an extension below the centerline of the web of 75 mm to provide additional clearance between the bottom girder and the sill. For the end section of the bottom girder, where the downstream flange is heavier, the upper portion of the flange is a maximum of $15t_f$ above the centerline of the girder web.

vi. The upstream flange of the bottom girder should extend 150 mm below the center line of the girder web, from end to end of the girder, with the skin plate 13 mm above the lower edge of the flange. A minimum of 100 mm should be used above the center line of the web, thus making a minimum width of 250 mm for the upstream flange of the lower girder.

vii. The maximum extension of the skin plate above the centerline of the top girder web is 200 mm, to prevent interference with the operating strut. The maximum extension of the skin plate above the top flange is 13mm, limiting the maximum width of the upstream flange of the top girder to 300 mm.

viii. An effective portion of the skin plate is assumed to act with the upstream flange. The effective width of skin plate next to each edge of the upstream girder flange is based on a width-to-thickness ratio consistent with design assumptions (assumption of compact or noncompact section).

ix. Webs are designed using requirements for uniformly compressed stiffened elements. The use of slenderness parameters for webs in combined flexure and axial compression elements should be avoided since these criteria were developed for rolled shape beam columns and may not apply for deep girder sections.

x.The ratio values for compact, noncompact, and slender sections are shown in Table A-5-1.

d Tapered End Section

The tapered end sections of girders are analyzed in a manner different from that used to analyze girder sections with full web depths. The moment is determined assuming a cantilever section of length Z' with a uniform water load plus the moment created by the girder reaction being eccentric from the centroid of the section.

The critical point is at a distance Z' from the end of the web. Z' is equal to one half the smaller span between adjacent girders (above and below) to the web under consideration minus eight times the thrust diaphragm thickness. If eight times the thrust diaphragm thickness is greater than one half the smaller adjacent span, the value of Z' becomes negative, and the minimum section for the tapered web is taken as the web width at the end of the web, with one half the height of the trust diaphragm and the appropriate upstream and downstream girder flanges. The maximum width of flanges shall be $24t_f$, reducing the possibility of the flange being undesirably wide and thin.

e End Diaphragm

End diaphragms, often called quoin and miter diaphragms, are designed as a panel acting as a skin plate, with the effective panel being between the stiffener angle and the next lower girder. The stiffener is at midpoint between girders. Design loads are the hydrostatic head at the center of the effective panel and the reactions produced by the diagonal prestress loads. Each end diaphragm, between girders, is divided into four panels subjected to the water pressure. The girder webs, vertical flange at the skin plate, the thrust diaphragm, and the horizontal end diaphragm stiffener surround four panels. The panels are assumed fixed on all four edges and designed by the same formulas as applied to the skin plate. The end diaphragm stiffeners are assumed fixed at the upstream end and at the centerline of the thrust diaphragm. The diaphragm load is triangular on the ends, with the boundary lines at 45 degrees from the vertical at the point of the triangle, this point being the center line of the thrust diaphragm and the upstream ends of the stiffener. A part of the end diaphragm shall be assumed effective with the stiffener, with the effective width equal to $\frac{95}{\sqrt{F_y}}$ on each side of the angle.

f Thrust Diaphragm

The thrust diaphragm is tangent to the thrust curve of the gate at the contact points and is approximately in line with the thrust curve between the contact points and the end diaphragm, which is the limit of the thrust diaphragm. The thrust diaphragms will distribute the reactions of the girder web into the quoin block and also serve as a damming surface between the end plate and the end diaphragm. Part of the thrust diaphragm is also considered effective in the quoin post, making it subject to bearing, skin plate, and column action stresses. Shear stress is to be checked also, but is not combined with the listed forces. The analysis is based on combined axial load from the girder reaction and bending from water damming pressure, which has to be less than or equal to the lesser value of the yield stress or the elastic limit stress. The flexure stress produced by the damming water pressure can be determined using the skin plate equations, and the axial load is determined as follows:

i. Determine V_a.

θ = zeta

$$V_a = W_u L \cos\theta$$

ii. Determine H_a using summation of moments in contact point equal ($\Sigma M_{cp}=0$).

$$H_a = \frac{V_a L \cos\theta \frac{W_u L^2}{2}}{L\cos\theta}$$

iii . Determine axial load (R)

$$R = V_a \sin 2\theta + H_a \cos 2\theta$$

The elastic limit can be found by assuming that the panel under consideration is clamped on all edges and that equal uniform compression exists on two opposite edges, with the critical stress equal to

$$K\frac{E}{1\mu^2}\left(\frac{t_d}{b}\right)^2$$

where:

a = longer dimension of panel

b = shorter dimension of panel

μ = Poisson's ratio

t_d = thrust diaphragm thickness

K = 7.7 when a/b = 1.0

K = 6.7 when a/b = 2.0

K = 6.4 when a/b = 3.0

g Quoin Post

A section of the thrust diaphragms, vertically from top to bottom girders and horizontally from the contact plate to a point $\frac{95}{\sqrt{F_y}}$ beyond the thrust diaphragm stiffener plate, forms a column to support the dead weight of the leaf plus any live loads. The end plate and two vertical stiffeners form one flange of the column. A plate perpendicular to the thrust diaphragm forms the other flange, with vertical stiffeners on the outside edges.

The axial load of the quoin post consists of the dead weight of the leaf plus the ice and mud load. Due to the eccentricity of the pintle and gudgeon pin with respect to the centroid of the quoin post, the quoin post is subjected to axial and bending loads and to the skin plate action of the thrust diaphragm. The following symbols and formulas are used to find the combined loads in the quoin post:

$$M_x = Pe_y$$

$$M_y = Pe_x$$

$$\sigma = \frac{P}{A} + \frac{M_x}{I_x}C_y + \frac{M_y}{I_y}C_x$$

P = total axial load (dead weight)

ex, ey, ez = eccentricity distance from centroid of entire quoin post cross section to (inclined) action line of pintle reaction Cx and Cy = extreme fiber distance

h Diagonals

This section is only applicable to single skin plate miter gate design. For double skin gates torsion is generally not a significant design consideration due to the inherent torsional resistance provided by the multi-closed cell construction. Finite element analysis should be performed to verify adequate strength and serviceability of double skin gates. The primary load cases that would be considered are normal operation of the gate (partially submerged in water) and with the stall force of operating equipment applied when the gate is lodged on an obstruction at the sill.

A gate is a deep cantilever girder with a relatively short span. The skin plate is the web of the girder. If the ordinary formulas for the vertical deflection of a cantilever submitted to shearing and bending stresses are applied, the values obtained will be low. This happens because the skin plate imparts such a great vertical stiffness to the leaf.

For single skin plate gates, diagonals are provided to function with the skin plate for torsional resistance to applied forces. For double skin plate gates, the torque tubes formed by the horizontal girders and skin plate offer significant resistance to torsional loads. The discussion that follows described the diagonal system used for the single skin plate gate alternative.

The stresses in the diagonals are a function of only the torsional (twisting) forces acting upon the leaf. These forces produce a considerable torsional deflection when the gate is being opened or closed.

The shape of the twisted leaf is found geometrically. Then the work done by the loads is equated to the internal work realized by the structure. From this, the resistance offered by each diagonal to twisting of the leaf is calculated as a function of the torsional deflection of the leaf and the dimensions of the structure.

The procedure and equations required to design the diagonal elements are as follow:

i. Evaluate the stiffness of the leaf in deforming the diagonal (A'). Until more test data are available, it is suggested that A' be taken as the sum of the average cross sectional areas of the two vertical and horizontal girders which bound a panel times 1/8 for welded horizontally framed leaves with skin of flat plates.

ii. Evaluate the elasticity constant of the leaf without diagonals (Q_o).

$$Q_o = 4\,E_s \sum\left(\frac{J}{H} + \frac{J}{v}\right)$$

where:

E_s = shearing modulus of elasticity

J = modified polar moment of inertia of the horizontal and vertical members of the leaf

H = distance between top and bottom girder

v = distance from center line of the pintle to extreme miter end of the leaf

iii. Location of shear center

$$X = \frac{b}{I}\Sigma a\, y\, y_n$$

$$Y = \frac{\Sigma I_n y}{\Sigma I_n}$$

where:

b = distance from the center line of the skin plate to the downstream flange of a horizontal girder

I = moment of inertia of the gate leaf about the vertical axis

I_n = moment of inertia of any horizontal girder about its vertical centroid axis

a = cross section area of that part of the horizontal girder that lies outside the midpoint between the skin plate and the downstream flange

y = distance to any horizontal girder from the horizontal centroid axis of a vertical section through the leaf

y_n = distance to any horizontal girder from the horizontal shear center axis of a vertical section through a leaf

iv. Load torque areas.

v. Calculate the ratio of change in length R_o of diagonal to deflection of leaf when diagonal offers no resistance. R_o is positive for positive diagonals and negative for negative diagonals.

$$R_o = \frac{2wt}{v\sqrt{(w^2+h^2)}}$$

where:

w = width of the panel inclosing diagonals

t = distance from center line of skin plate to center line of diagonal

v = distance from center line of the pintle to extreme miter end of the leaf

h = height of panel inclosing diagonals

vi. Required size of diagonals.

$$A = \frac{\sum Tz}{sR_o hv}$$

where:

A = cross section area of diagonal

Tz = torque area (product of the torque, T, of an applied load and the distance, z, to the load from the pintle; z is measured horizontally along the leaf. Tz is positive if the load produces a positive deflection).

s = design strength for tension members ($s=\alpha\Phi Pn$), which is the lower value of the following:

1. For yield in the cross section, $\alpha=0.9$ and $\Phi=0.9$

   $Pn = F_y A_g$

2. For fracture in the net section, $\alpha=0.9$ and $\Phi_t=0.75$

   $Pn = F_u A_e = F_u(UA_g)$

vii. Evaluate the ratio (R) of the actual change in length of diagonal to deflection of the leaf. R is positive for positive diagonals and negative for negative diagonals.

$$R = \frac{A'}{A + A'} R_o$$

viii. Evaluate the elasticity constant of a diagonal (Q).

$$Q = \frac{RR_o EAhv}{L}$$

where:

E = bending modulus of elasticity

L = length of a diagonal, center to center of pins

ix. Deflection of leaf.

$$\Delta = \frac{\Sigma Tz}{Q_o + \Sigma Q}$$

x. Prestress deflection in diagonals (D_{max}). D is the deflection of the leaf required to reduce the stress in a diagonal to zero. D is always positive for positive diagonals and negative for negative diagonals.

$$D = \frac{sL}{RE} + \Delta$$

$$s = \frac{RE}{L}(D\Delta)$$

xi. Stresses during normal operation of the gate. The value of D must be between the minimum and maximum value.

$$s = \frac{RE}{L}(D - \Delta)$$

5.2.2. Culvert Valves

Wheel gate valves have been selected for use with both filling and emptying systems for the Double-Lift Lock Concept Design. The valves have been designed using the Allowable Stress Design (Type-A structure) criteria presented in EM 1110-2-2105, Design of Hydraulic Steel Structures. Allowable stresses will be 0.75 times those permitted by AISC (1989). Valves will be designed to resist 1.5 times the normal operational head. See Appendix G – Valve and Valve Bulkheads for additional information.

5.2.3. Bulkheads

Bulkheads would be designed in accordance with EM 1110-2-2105, Design of Hydraulic Steel Structures. Allowable stresses will be 1.1 times those permitted by AISC (1989). Bulkheads will be designed to resist expected pool elevations for dewatering applications. See Appendix G – Valve and Valve Bulkheads for additional information.

5.2.4. Masonry Features

The purpose of these standards is to provide guidance for the preliminary design and analysis of lock masonry structures. These standards establish and standardize the stability criteria for use in the preliminary design and evaluation of the lock masonry structures according to common Corps of Engineers civil works projects. The term "stability" used in this document applies to external global stability of the structures — sliding, rotation, and bearing. Guidance is provided on load classification, loadings, design and analysis computations, earth pressure computations for both static and earthquake conditions, hydrostatic and hydrodynamic forces, strength design for reinforced concrete hydraulic structures, and drafting standards.

The following criteria will be applied to general investigations of lock walls, water saving basins, and entrance wall selection. This criteria will tailored throughout the study to reflect the selection of features.

Essential requirements for operational and structural performance of the lock and entrance walls include:

- Limiting global displacements and deflections to a tolerable range.
- Minimizing interference with the lock's inlet opening.
- Limiting misalignments and tolerances satisfying the navigational purpose.
- Providing for the ease and economy of construction.

- Economy of structural and functional monitoring and inspections.
- Durability of materials and hydraulic performance of the structure.
- Providing for an appropriate level of ultimate structural strength exceeding the factored capacity-demand.
- Ability to survive unusual and extreme events such as OBE, MDE, and impact by a vessel.

5.2.4.1. Scope

The design and analysis of the lock masonry structures and the preparation of drawings shall conform in general to the requirements set forth by these standards and all subsequent revisions. Industry and COE design standards, manuals and recommendations shall be referenced and followed in the course of analyzing, designing and preparing drawings for the lock masonry structures. For special features, supplementary design standards shall be developed and included as an appendix to this document.

5.2.4.2. Applicability

These standards shall apply to the preliminary design and analyses of all lock masonry structures for the Third Lane Atlantic Locks. Discretion and judgment must be applied in the use, interpretation, and applicability of the guidance provided within this document. Consideration should be given to unusual conditions or service not addressed in this document. When a significant departure is required or the need arises for specific guidance not covered by these standards, the project manager must be notified for guidance. The project manager will notify the responsible design team member(s) for guidance. The independent technical review team (ITRT) will determine the adequacy of the guidance developed by the responsible design team member(s). The developed design guidance shall be incorporated into this document for future reference and use.

5.2.4.3. Structural Computations

Concept designs shall be developed for the following representative lock walls and entrance walls: valve monolith, miter gate monolith, typical lock monoliths for the upper and lower chambers and the Atlantic Ocean and Gatun Lake entrance wall monoliths. For the concept design of the lock monoliths, stability analyses and reinforced concrete design shall be provided in the computations. For the purpose of consistency and uniformity in the presentation, arrangement and filing of computations, the following instructions are to be used.

5.2.4.3.1. Stability Analysis

Stability analyses for the lock masonry structures at the foundation-structure interface shall be conducted according to the recommendations and provision provided in EM 1110-2-2100, Draft Manual - Stability Analysis of Concrete Structures. Stability shall also be assessed on selected surfaces within the structure in accordance with the methods presented in EM 1110-2-2200, Gravity Dam Design. The stability analyses are conducted to assure that the hydraulic concrete structures meet minimum safety and performance objectives. The stability of the structures is assessed in terms of overturning, sliding and bearing.

If the stability requirements of the draft manual conflict with those in other COE Engineering Manuals (EM) or Engineering Technical Letters (ETL), the requirements of the draft manual shall govern. The draft manual defines the types and combination of applied loads, including uplift forces due to hydrostatic pressures in the foundation material.

The objective of a stability analysis is to maintain horizontal, vertical and rotational equilibrium of the structure. Geotechnical information is needed to properly define and perform a realistic stability analysis. Possible failure modes and planes of weakness must be determined from onsite conditions, material strengths and uplift forces. Stability is ensured by:

- Providing an adequate factor of safety against sliding at all possible failure planes.
- Providing specific limitations on the magnitude of the foundation bearing pressure.
- Providing constraints on the permissible location of the resultant force on any plane.

Satisfying the above provisions does not ensure stability if the structure experiences significant loss of foundation material due to erosion or piping, or if there is an internal failure due to inadequate strength of the structural materials. Stability is just one of the requirements necessary to ensure adequate structural performance.

a Sliding Stability

Sliding stability shall be assessed at/or near the foundation-structure interface. Where a shallow weak seam exists below a structure's contact with the foundation, deep-seated sliding may occur. The lock monolith shall be evaluated for both modes of failure, if applicable.

b Overturning Stability

The assessment of overturning stability shall be based on the resultant location of the effective normal force on the base of the structure. Conformance with the resultant location requirements ensures that the structure is safe from rotational failure. For some load condition categories, the resultant is allowed to fall outside the middle-third of the base (kern). In these instances, it is assumed that the structure-foundation interface has no capability for resisting tensile stresses; therefore, part of the structure's base is assumed to lose contact with the foundation resulting in changes to the uplift pressure acting on the base. For a monolith base not in 100 percent compression, the uplift pressure for the portion of the base not in compression will equal to the maximum pressure head at the base of the heel of the monolith.

c Bearing

Safety against a foundation bearing failure shall be based on the foundation bearing pressure and bearing capacity. The effective normal force on the base of the structure and its resultant location are used to determine the foundation bearing pressure. The allowable bearing capacity is defined as the maximum pressure that can be permitted on a foundation soil or rock mass giving consideration to all pertinent factors, with adequate safety against rupture of the soil or rock mass, or settlement of the foundation of such magnitude as to jeopardize the performance and safety of the structure. Increases in allowable bearing capacity are permitted for unusual and extreme load conditions over those required for usual load conditions.

5.2.4.4. Allowable Stresses for Concrete

5.2.4.4.1. Mass Concrete

Mass concrete monoliths must meet the allowable stress requirements of EM 1110-2-2200 as shown for the following load conditions:

a **Allowable Compressive Stress**

Usual $0.3 f'_c$
Unusual $0.5 f'_c$
Extreme $0.9 f'_c$

b **Allowable Tensile Stress (psi)**

Usual 0
Unusual $0.6 f'c^{2/3}$
Extreme $1.5 f'c^{2/3}$

5.2.4.5. Compressive Strength

5.2.4.5.1. Mass Concrete

f'_c = 18 MPa at 90 days

5.2.4.5.2. Tremie Concrete

f'_c = 28 MPa at 90 days

5.2.4.5.3. Precast Concrete

f'_c = 35 MPa at 28 days

5.2.4.5.4. Cast-In-Place and Structural Concrete:

f'_c = 28 MPa at 28 days

Non-Shrink Grout f'_c = 35 MPa

5.2.4.5.5. Reinforcement:

Reinforcement Bars (fy = 420 MPa):

Welded ASTM A-706M, Not Welded ASTM A-615M

5.2.4.5.6. Post-Tension/Pre-stress Tendons and Bars:

Tendon ASTM A-416M, Grade 1860, 15.2-mm diameter, Stress Relieved, Low Relaxation (f_{pu} = 1861 MPa):

Threaded Bars ASTM A-722M, Grade 1030 (fpu = 1030 MPa)

5.2.4.6. Concrete Strength Design

The masonry lock structures are considered mass concrete structures. However, when a section (construction joint) of a monolith does not satisfy the internal stability requirements of EM 1110-2-2200 and reinforcement is required, or a portion of the monolith carries significant load (such as the toe stub of a monolith or a concrete slab overlying a large recess area) and concrete reinforcement is required, the structures will be treated as a reinforced-concrete hydraulic structures and shall be designed according to the requirements of EM 1110-2-2104, Strength Design for Reinforced-Concrete Hydraulic Structures and ACI 318M-99, Building Code Requirements for Reinforced Concrete. The lock walls shall be designed according to the strength design method, using appropriate load factors, hydraulic load factor, and strength reduction factors when appropriate. The concept design computations will provide the size, spacing and location of the principal concrete reinforcement, shrinkage and temperature steel, and the location of expansion and construction joints.

Reinforced concrete hydraulic structures shall be designed using the strength design method in accordance with ACI 318 except as specified in EM 1110-2-2104.

5.2.4.6.1. Hydraulic Factor, H_f

In the strength design method the hydraulic factor is applied to the overall load factor equations for obtaining the required nominal strength. The hydraulic factor is used to improve crack control in hydraulic structures by increasing reinforcement requirements, thereby reducing steel stresses at service load conditions. This is similar to the approach taken in ACI 350R-89. According to ACI Codes for hydraulic structures in a state of flexure.

H_f equals 1.3, except for members in direct tension H_f equals 1.65.

This factor is applied for the determination of required nominal strength for all combinations of axial load and moment as well as for shear.

ACI 350R-89 recommends applying the H_f on the excess shear that must be carried by the shear steel. This application will be modified to the entire shear demand.

For earthquake effects, H_f will not be applied in the case of OBE and MDE.

With respect to the application of the hydraulic factor, H_f, and using the nomenclature of the EM 1110-2-2104 noted previously, the load combination for the ultimate state design will take the form: $U = H_f (1.4D + 1.7L)$, where H_f equals 1.3 for application under the Usual Condition.

5.2.4.6.2. Demand Reduction Factor

When resistance to wind or other forces that constitute short duration loads with low probability of occurrence are considered, then a reduction in the factored load combination may be utilized. The reduction factor has been termed the Demand Reduction Factor and is applied to the unusual load combinations. The factor is not applied to the extreme load combinations since the basic ultimate load factors for this condition equal 1.0 and should not be reduced further.

5.2.4.6.3. Basic Ultimate Factors

Two methods are provided for by ACI 318 and EM 1110-2-2104 codes for determining the factored moments, shear, and thrusts for designing hydraulic structures by the strength method. They are the single load factor method and the method based on the ACI 318 Code. The ACI 318 method will be adopted.

The load factors prescribed in ACI 318 are applied with two modifications. First, the load factor for lateral fluid pressure is taken as 1.7, and second, the factored load combination shall be further multiplied by the hydraulic factor H_f. When dead or live loads have a relieving effect on the factored load combination, the combination of factored loads shall be investigated using an ultimate factor of 0.90 toward either the dead or live loads.

5.2.4.6.4. Minimum Clear Cover

The minimum concrete clear cover as outlined in EM 1110-2-2104 will be used to increase corrosion protection. The cover is correlated to susceptibility of corrosion or erosion as determined by its exposure rating.

5.2.4.6.5. Temperature and Shrinkage Reinforcement

In concrete slabs and walls where the main reinforcement extends in one direction only, reinforcement in the direction normal to the main reinforcement is provided to minimize the effect of cracking due to temperature and shrinkage. The main reinforcement is at least the amount required for temperature and shrinkage control. Temperature and shrinkage reinforcement is provided in accordance with EM 1110-2-2602, with limits established by EM 1110-2-2104.

5.2.4.7. Design Loads

The masonry lock structures shall be designed to carry the maximum dead, live, impact, ship-positioning, construction and earthquake loads to which the structures may be subject. The lock structures shall also be designed for special loads envisioned for the structures and the loadings from appurtenance structures. The development of the dead, live and construction loads shall be based on EM 1110-2-2602, Planning and Design of Navigation Locks and where applicable, ANSI/ASCE 7-95, Minimum Design Loads for Buildings and Other Structures. Impact and ship-positioning loads shall be developed by COE and provided to the appropriate designers. The earthquake loads shall be developed by the COE Seismic Analysis Team (SAT), ERDC and will be provided to the designers.

5.2.4.7.1. Load Definitions

a Dead Loads

Dead loads are produced by the weight of building construction materials and fixed equipments.

b Live Loads

Live loads are produced by occupancy of buildings, construction and maintenance loads, moving equipment such as cranes and locomotives of the ship positioning system and impact loads.

c Seismic Loads

Forces on the structure are caused by earthquake motion at the base of the structure. The resultant earthquake forces represent the inertial affects that are attributed to the lock wall mass, surrounding soil (increased earth pressure) and the water in the chamber (hydrodynamic force). The following design earthquakes shall be considered in the preliminary design of the lock walls:

- Operational basis earthquake (OBE). The OBE is considered to be an earthquake that has a 50 percent chance of being exceeded in 100 years (or a 144-year return period).

- Maximum design earthquake (MDE). The MDE is the maximum level of ground motion for which a structure is designed or evaluated. Generally, the probabilistically determined MDE for normal structures is an earthquake that has a 10 percent chance of being exceeded in a 100-year period (or a 950-year return period). For critical structures the MDE is the same as the maximum credible earthquake (MCE).

- Maximum Credible Earthquake. The MCE is defined as the greatest earthquake that can reasonably be expected to be generated on a specific source, on the basis of seismological and geological evidence. The MCE is based on a deterministic site hazard analysis.

The seismic coefficient method shall be used in the preliminary analysis and design of the lock and entrance walls. Although it fails to account for the true dynamic characteristics of the soil-water-structure interaction, it is an acceptable method for evaluating structural stability and conducting preliminary designs. The seismic coefficient used for the preliminary seismic stability evaluation of critical concrete hydraulic structures should be equal to the effective peak ground acceleration (EPGA) expressed as a decimal fraction of the acceleration of gravity. For non-critical structures, seismic coefficient used for the preliminary seismic stability evaluation of critical concrete hydraulic structures should be equal to 2/3 times the effective peak ground acceleration.

5.2.4.7.2. Loading Conditions

a Usual (Service) condition

Constitutes the structural performance and adequacy of the structure for the intended use. This condition is associated with the ordinary operation of the facility. The design objective is to preclude all but minor damage (for example, loss of concrete cover in the confined area of the impact) to the structure in this performance category. This is the normal operating condition and accounts for the most severe loads during a complete lockage cycle and for the most severe stability requirements experienced during normal conditions.

b Unusual Condition

Refers to structural performance under severe environmental loads or forces whose magnitude is substantially higher than loads associated with the normal operation of the facility. The design objective is to preclude major damage to the structure in this performance category. The conceded damage should be repairable without substantial interference (short construction-periods) with the operations of the facility.

Under these conditions, the design should account for unusual loads that can occur at normal lock sites, such as vessel impact, unusual saturation levels, scheduled pool drawdowns, low-water levels and operating basis earthquakes (OBE). The dewatered condition of the lock is also considered in this case with loads corresponding with expected water levels and uplift conditions. Construction conditions are also considered as an unusual load case. Under these conditions, full earth load, moist or saturated, with or without uplift, according to construction plans, possible use as part of a cofferdam, and all construction surcharge loads should be considered.

c Extreme Condition

Denotes the structural performance during and after uncommon and severe occurrences such as a maximum credible earthquake (MCE) event, an impact by an uncontrolled vessel.

5.2.4.7.3. Load Cases

After the individual loads acting on the representative lock walls and entrance walls are determined, the possible combinations of such loads that will determine the most adverse condition needs to be established. Conditions and combinations of loadings that require consideration are provided in the following tables. The individual loads should be combined appropriately to determine the critical conditions for global and internal stability and internal wall stresses. An independent check is also required for each monolith to determine whether these conditions are adequate for determination of the most critical loading. Table A-5-2 presents the load cases that shall be considered in the stability analyses and walls. For (4) categories of loading are considered: construction, normal operation, maintenance and earthquake.

Table A-5-2 Load Combinations for Lock and Entrance Wall Monoliths

Load Case	Description	Load Type	Requirements
Lock Chamber East Wall Monoliths			
Case 1	Normal Operating Condition	Usual	(1) Backfill to a predetermined height. (2) Saturation line to an assumed level. (3) Surcharge due to sloped fill. (4) Loading surcharge if any. (5) Ship-positioning loads. (6) Lower pool in lock chamber. (7) Uplift as defined by water elevations.
Case 2A	Normal Operating Condition	Unusual	The same requirements as for Case 1 except for the condition that the water saving basins leak and the saturation elevation of the backfill is equivalent to the maximum pool elevation of the water saving basin.
Case 2C	Maintenance Condition	Unusual	The same requirements as for Case 1 are included except for the following conditions: (1) Lock chamber dewatered to a predetermined level. (2) No ship-positioning load.
Case 2D	Normal Operation with OBE	Unusual	The same requirements as for Case 1 except for the condition of an operating basis earthquake (OBE) load added in the most critical direction.
Case 2G	Normal Operation with MCE	Extreme	The same requirements as for Case 1 except for the condition of a maximum credible earthquake (MCE) load added in the most critical direction
Case 3A	Construction Condition	Unusual	The requirements include: (1) Moist backfill to a predetermined height. (2) Permanent or construction surcharge. (3) Wind as applicable. (4) No uplift.
Case 3B	Construction Condition with Cofferdam	Unusual	The same requirements as for Case 3 A are included except for the condition that hydrostatic forces are active instead of moist earth in accordance with construction and cofferdam plans.

Load Case	Description	Load Type	Requirements
Miter Gate Bays			
Case 1A	Normal Operating Condition	Usual	The requirements are as follows: (1) Gates mitered. (2) Upper pool upstream of gates. (3) Lower pool downstream of gates. (4) Applicable wall loadings as defined in paragraphs C-3, C-4, and C-5.
Case 1B	Normal Operating Condition	Usual	The requirements are as follows: (1) Gates swinging free in approximate mitered position. (2) For upper gate bay, upper pool in gate bay. (3) For lower gate bay, lower pool in lock chamber. (4) Applicable wall loadings as defined in paragraphs C-3, C-4, and C-5.
Case 2E	Maintenance Condition	Unusual	The same requirements as for Case 1 B are included except for the following conditions: (1) Lock chamber unwatered to a predetermined level. (2) Uplift as defined by water elevations added in most critical direction.
Cases 2F & 2A	Normal Operating Condition with OBE	Unusual	The same requirements as for Cases 1 A and 1 B are included except for the condition of OBE earthquake loads added in most critical direction.
Case 2G	Normal Operation with MDE	Extreme	The same requirements as for Cases 1 A and 1 B are included except for the condition of MDE earthquake loads added in most critical direction
Case 3A	Construction Condition	Usual	The requirements are as follows: (1) Moist backfill to a predetermined height. (2) Permanent or construction surcharge. (3) Wind as applicable. (4) No uplift. (5) Gates swinging free in appropriate mitered position.
Case 3B	Construction Condition with Cofferdam	Unusual	The same requirements as for Case 3 A are included except for the condition that hydrostatic forces are active instead of moist earth in accordance with construction and cofferdam plans.

Load Case	Description	Load Type	Requirements
Upper and Lower Entrance Walls			
Case 1	Normal Operating Condition	Usual	The requirements are as follows: (1) Upper or lower pool on face of wall as applicable. (2) Saturated fill. (3) Upper or lower pool on back face of wall as applicable. (4) Boat impact on face of wall at most critical angle of incidence and force. (5) Ship-positioning pull force away from face of wall as applicable.
Case 2D	Normal Operating Condition with OBE	Unusual	The same requirements as Case 1 are included except for the following conditions: (1) OBE Earthquake in most critical direction. (2) No impact or ship-positioning forces.
Case 2G	Normal Operating Condition with MDEE	Extreme	The same requirements as Case 1 are included except for the following conditions: (1) MDE Earthquake in most critical direction. (2) No impact or ship-positioning forces.
Case 3	Construction Condition	Unusual	The requirements are as follows: (1) Moist backfill. (2) Permanent or construction surcharge. (3) Wind as applicable. (4) No uplift. (4) Applicable gate loads for vertically framed miter gates and rolling gates. (5) Uplift and vertical water loading as defined by water elevations.
Culvert Valve Monoliths			
Load cases the same as with the Chamber Monoliths, except with differential head loads from culvert valves added.			
Culvert Bulkhead Monoliths			
Load cases the same as with the Chamber Monoliths, except with differential head on the culvert bulkhead added in the Maintenance Condition.			

5.2.4.7.4. Loads

a Unit weights

The assumed average values for weights of materials are as follows:

Table A-5-3 Project Feature Document/References

ITEM	Weight (kg/m^3)	Unit Weight (lb/ft^3)
Water	1000	62.5
Reinforced Concrete	2400	150.0
Steel	7850	490.0
Backfill, existing	2020	126.0 (moist)
	2080	130.0 (saturated)
Rock	1925	120.0

b Equipment Loads

The loads from operating machinery are determined following computational procedures prescribed in EM 1110-2-2703, "Lock Gates and Operating Equipment". Operating loads and support structures consider gates movement through water, self weight, stall loads, closure against obstacles, and wave and surge effects of water on gates.

Operating loads for valves include self weight and friction from wheels and rollers.

Loads from ship positioning equipment is described in Section 5.2.4.8.4.f.

c Hydrostatic Loads

i. Lateral and Uplift Hydrostatic Forces

Lateral and uplift hydrostatic forces on the structure are determined according to the pool levels in the given load case. Uplift forces on the structure base are in accordance with EM 1110-2-2602. Concrete internal hydrostatic pore pressure is also in accordance with EM 1110-2-2602. Note that this applies to the RCC, GRF and the conventional concrete. However, for the GRF, internal planes will also be checked for internal hydrostatic pore pressures varying linearly from 100% of the head on the high water side to 100% of the head on the low water side. The results of these analyses will be compared to extreme load case allowable valves.

ii. Saturation Level of Soil Behind Lock Wall

The saturation level of the soil behind the wall will be determined for Normal, Seismic, and Maintenance Conditions:

d Lateral Earth Loads

Lateral earth pressures on the structure are calculated in accordance with EM 1110-2-2502. At-rest lateral earth pressures are used.

e **Vessel Load**

Impact loads for the lock and entrance walls are established from the ship impact study. The impact energies were calculated based on the ship weight, velocity and angle of approach. The results are presented in the ship collision study section.

f **Ship Positioning System Loads**

The vertical and lateral loads for the new the ship positioning system is based the current requirement of a 203 000 t ship and scaling the loads for the current ship positioning system and ship size. For the current 140 000 t ship, the locomotive weight is 490 kN and the total line pull is 312 kN. The resultant lateral and vertical loads that shall be considered in the stability analysis of the lock and entrance walls are:

- Lateral force: 0.486 MN / Winch
- Vertical force: 1.530 MN

g **Wind Load**

Wind loads are usually small in comparison to other forces which act on civil works structures, and shall not be considered in the preliminary lock and entrance wall monolith design

h **Construction Load**

A uniform load equal to 9.58 kPa shall be applied to the tops of the monoliths for the construction load case.

i **Debris and Ice Loads**

Not Applicable

j **Wave Hindcasting**

The wave field develops over time, and is dependent on the water depth, fetch geometry, air-water temperature difference and the strength and duration of the wind. The wave height and period can be hindcast if these variables are known or estimated.

k **Seismic Loads**

The following table provides the design seismic coefficients and velocities that shall be in the seismic stability analyses and design of the lock and entrance wall monoliths. The values provided are to be used for both yielding and non-yielding backfill conditions. The OBE values will be provided by the Seismic Analysis Team (SAT), ERDC, COE when the United States Geologic Survey (USGS) study is completed for the Gatun area. For the preliminary design and stability analyses, the vertical acceleration and vertical seismic coefficient kv shall be assumed equal to zero. Normally, vertical accelerations are considered in seismic stability evaluations. However, since simplified conservative engineering procedures will be used in the preliminary analyses, vertical accelerations do not be included in these preliminary calculations. When the seismic engineering evaluations proceed to the final design stage, more sophisticated modal response- or time history-based engineering procedures will be used to assess the potential for damage to the lock features. In addition, when the USGS site-specific earthquake hazard results become available, vertical accelerations shall be recommended for the final design and stability analyses.

Table A-5-4 Design Seismic Coefficients and Velocities

	Seismic Coefficient k_h
OBE	0.140
MDE	0.313

i. Acceleration Coefficient

Acceleration coefficients will be determined for the operating basis earthquake (OBE) and maximum design earthquake (MDE) in accordance with new guidelines for the earthquake design and evaluation for civil works projects (ER1110-2-1806) using the information in the seismic studies provided by the ACP. Under the level of the OBE ground motion, the structure is expected to remain functional with little or no damage, and without interruption of function. The maximum design earthquake (MDE) is the maximum level of the ground motion for which a structure is designed or evaluated. The associated performance requirement is that the project performs without catastrophic failure, although severe damage or economic loss may be tolerated: For critical features, the MDE shall be the same as the maximum credible earthquake (MCE). For all other features, the MDE shall be selected as a lesser earthquake than the MCE that provides economic designs meeting appropriate safety standards.

ii. Inertia Forces

The magnitude of the inertia forces is the product of the structural mass and seismic coefficient. In accordance with EM 1110-2-2200, the following formula is used to compute the inertia force for the structure:

$$P_e = \alpha \bullet W$$

where P_e = horizontal or vertical force

α = seismic coefficient

and W = weight of structure

The resultant forces are assumed to act at their respective centers of gravity.

iii. Static Earth Pressure

In the seismic load cases, the static earth pressure may be calculated in accordance with the General Wedge Method in EM 1110-2-2502, using a strength mobilization factor (SMF). The SMF used shall be as follows:

OBE Condition 1/1.7

MCE Condition 1/1.3

l Dynamic Earth Pressure

The dynamic earth pressure may be calculated in accordance with the General Wedge Method in EM 1110-2-2502, using an SMF as shown above.

m Hydrodynamic Forces

Hydrodynamic forces are determined using the Westergaard formula as given in EM 1110-2-2200.

n Sloshing Effect

Sloshing effects are determined and applied by the Housner's method as cited in a paper by Epstein (1976).

o Thermal Loads

A nonlinear, incremental structural analysis (NISA) is beyond the scope of work.

5.2.4.7.5. Site Condition

A stability analysis typically requires the following information: the potential planes of weakness beneath the structure (deep-seated sliding), the strength of the materials along potential planes of weakness, uplift forces that occur at the foundation or within the structure, the backfill material properties, and all loads and load conditions to which the structure may be subjected. Geologic information is important in defining seepage conditions and uplift pressures within the foundation rock mass. The stability safety factors shall be applied for the case where the soil and rock properties are well defined. The reasons for this decision is as follows:

- Available records of construction, operation, and maintenance are available for Gatun Locks.
- Foundation strengths will be established with a high level of confidence.
- The governing load conditions can be established with a high level of confidence.
- Uplift pressures will be controlled with a planned seepage cutoff wall.

The material properties that shall be used for the rock, soil and engineering backfill are provided in the geotechnical section of the report.

5.2.4.8. Stability Safety Factors And Criteria

The lock walls and a portion of the entrance walls are to be considered as critical structures; the outer reaches of the entrance walls that may displace laterally a significant amount during an earthquake and only temporarily disrupt navigation are to be considered normal structures. The definition of critical structure for the preliminary design of the typical lock and entrance wall monoliths is that the failure (greater than no lateral displacement or a small pre-selected lateral displacement) of any of these monoliths during an earthquake would prevent the use of the Third Lane for an extended period of time and would result in severe economic loss. The definition of normal structure is that the failure (large displacement) of any of these monoliths during an earthquake would only temporarily disrupt the use of the Third Lane and the economic impact would be minor. The safety factors provided in these standards are based on the assumption that for critical and normal structures, the strength of the materials of the foundation and structures has been conservatively established through explorations and testing. The monoliths are checked for global stability and internal stability. For global stability, the requirements of EM 1110-2-2100 are used. For internal stability, sliding stability must be assessed on selected surfaces within the structure in accordance with the methods presented in EM 1110-2-2200

5.2.4.8.1. Sliding Factor of Safety

A factor of safety is required in sliding analyses to provide a suitable margin of safety between the loads that can cause instability and the strength of the materials along potential failure planes that can be mobilized to prevent instability. The factor of safety for sliding is defined by the following equation:

$$FS_S = \frac{N' \cdot \tan\phi + c \cdot L}{T} \qquad \textbf{(1)}$$

where N' = effective force acting normal to the sliding failure plane under the structural wedge,

ϕ = angle of internal friction of the foundation material under the structural wedge,

c = cohesive strength of the foundation material under the structural wedge,

L = length (effective base) of the structural wedge in contact with the foundation

and T = shear force acting parallel to the base of the structural wedge.

The required factors of safety for sliding stability for the critical structures are:

- Usual Load Condition: 1.7
- Unusual Load Condition: 1.3
- Extreme Load Condition 1.1

5.2.4.8.2. Overturning Stability — Limits on Resultant Location X_R

Overturning stability is evaluated by determining the location of the resultant of all applied forces (including uplift) with respect to the potential failure plane. This location can be determined through static equilibrium analysis. Limits on the location of the resultant are provided in the following table and are applicable for both normal and critical structures. The entire base must be in compression for the usual load condition, to maintain full contact between the structure and the foundation, so there is no chance for higher uplift pressures to develop in a crack. This helps ensure linear behavior for common loading conditions. For the unusual load case, higher uplift pressures may develop in a relatively short crack, but this would cause only minor nonlinear behavior. For extreme load conditions on typical civil works projects, a shear or bearing failure will occur before overturning could occur. Therefore, the resultant is permitted to be anywhere within the base, and safety is ensured by the safety factor requirements for sliding and by the limits on allowable bearing stresses.

Table A-5-5 Limits on Resultant Location

Usual	Unusual	Extreme
100% of Base in Compression	75% of Base in Compression	Resultant Within Base

The equation for determining the resultant location X_R of the effective normal force N' is

$$X_R = \frac{\Sigma(M_R - M_O)}{N'} \tag{2}$$

where M_R = the resisting moment, MN-m/m,

and Mo = the overturning moment, MN-m/m.

5.2.4.8.3. Uplift Effects on the Resultant Location

The resultant location X_R is dependent on the uplift force due to hydrostatic pressure acting on the base of the monolith. The uplift pressure acting on the base will be assumed a straight-line distribution for a base in 100 percent compression (X_R is within the middle third of the base [kern]). The water pressure at the toe of a monolith will be equal to the chamber pool pressure head; the water pressure at the heel will be equal to the ground water pressure head. For the case where X_R is outside the kern, a crack will form at the heel of the monolith and the water pressure in the crack region is assumed to equal the pressure head at the heel. As a result, the uplift force increases in proportion to the width of the base not in compression. The effective base in compression and the uplift may be solved for simultaneously using a closed-form solution. The closed-form solution for the effective base in compression B_{eq} is a function of the overturning and resisting moments, uplift pressures at the toe and heel and the resultant effective vertical load. The effective base in compression is calculated using the equation

$$B_{eq} = \frac{3 \cdot (\sum M_R - \sum M_o - p_w \cdot B/2)}{\sum F_V - p_w \cdot B} \tag{3}$$

where F_R = the vertical force, MN/m,

and p_w = the water pressure at the heel of the monolith, kPa.

If B_{eq} is greater than or equal to the monolith base width B, then B_{eq} is equal to B. Once B_{eq} is established, the uplift force and overturning moment due to hydrostatic uplift pressure may be calculated and the resultant location and stability analyses may be completed.

5.2.4.8.4. Allowable Bearing Capacity

The allowable bearing capacity value is defined as the maximum pressure that can be permitted on the soil or rock giving consideration to all pertinent factors with adequate safety against rupture of the soil or rock mass, or movement of the foundation of such magnitude that the structure is impaired. Bearing failure is related to the relative compressibility of the foundation materials, the loading conditions, the geometry of the structure base, and the strength of the foundation and concrete at the structure-foundation interface. Bearing capacity may be related to the shear capacity of the foundation materials or to the deformability of the foundation. Information on foundation bearing analysis can be found in EM 1110-1-1905 for soils, and EM 1110-1-2908 for rock. Safety against bearing failure is generally expressed in terms of an allowable compressive stress for concrete and an allowable bearing capacity for foundation materials. These allowable values include an adjustment that represents a factor of safety. The allowable compressive stress and allowable bearing capacity values are established by testing performed by materials engineers and geotechnical engineers. The allowable compressive stress and bearing capacity values established for usual load conditions can be increased for the unusual and extreme load conditions. A 15% increase is permitted for unusual loading conditions and a 50% increase is permitted for extreme loading conditions.

The following equations provide the peak base bearing pressure for the resultant location X_R located within the middle third of the base

$$q_{max,min} = \left(\frac{N'}{B}\right) \cdot \left(1 \pm \frac{6 \cdot e}{B}\right) \tag{4}$$

and outside the middle third of the base

$$q_{max} = \left(\frac{4}{3}\right) \cdot \left(\frac{N'}{B - 2 \cdot e}\right) \tag{5}$$

where e = the distance of the resultant location X_R from the base centerline, m,

and B = base width, m.

5.2.4.8.5. Seismic Stability

The seismic coefficient method will be used to evaluate the preliminary design and stability of the lock and entrance walls subjected to earthquake ground motions. The preliminary seismic design of the lock and entrance walls shall follow the recommendations of ITL-92-11, The Seismic Design of Waterfront Retaining Structures, February 1992. In order to achieve economy of design, small displacements of the structures will be established to permit the use of active earth pressure coefficients during an earthquake loading.

5.2.4.9. Static And Dynamic Force Computations

The magnitude of the static and dynamic earth pressures acting on a wall is dependent on the amount of lateral and/or rotation movement of the wall during loading. The interdependence between wall deformation and the static and dynamic earth pressure forces acting on a wall is well established and the Table A-5-6 provides the wall movement required for active or passive earth pressures to be mobilized for various densities of cohesionless material.

The term "yielding backfill" will be used for walls that undergoing sufficient displacement to permit active or passive soil pressure to be mobilized. The term "non-yielding backfill" will be used for walls that are not permitted to displace or their displacement is insufficient for active or passive soil pressure to be mobilized. For the non-yielding backfill condition, at-rest soil pressures will be used. For the case of yielding backfill, active or passive soil pressure will be used.

Table A-5-6 provides guidance to the type of earth pressure analysis that will be conducted for yielding and non-yielding backfill for the cases of both static and earthquake loads.

5.2.4.9.1. Static Active Earth Pressure and Force

The lock and entrance walls shall be designed using conservative criteria. The monoliths will be stiff, monolithic structures. In addition, they will be founded predominantly on rock or possibly on drilled shafts for deep soil deposits. As a result, the monoliths that will not yield sufficiently to develop active pressures since the shear strength of the soil is not fully mobilized. However, for an earthquake, the monoliths may be designed for a predetermined movement, due to sliding along the monolith-rock interface or displacement of the monoliths founded on drilled shafts. For this case, the predetermined movement should exceed the movement required for the minimum active earth pressure to mobilize. The amount of movement required is dependent on the height of the backfill retained by a monolith. For active earth pressure computation, Coulomb's equation will be used.

Table A-5-6 Approximate Magnitudes of Wall Movement for Active/Passive Pressure Condition to Mobilize (From Clough & Duncan, 1991)

Type of Backfill	Values of Y/H[1]	
	Active Pressure	Passive Pressure
Dense Sand	0.001	0.01
Medium-Dense Sand	0.002	0.02
Loose Sand	0.004	0.04
[1] Y is equal to the amount of lateral and or tilting movement of the wall at the soil surface elevation to reach a minimum active or maximum passive pressure. H is equal to the height of the retained soil.		

Table A-5-7 Stability Analysis Guidance for Static and Seismic Cases.

	Load Case	
	Static	Seismic
Backfill Non-Yielding	1. At-Rest Soil Pressure: • Use Jaky's Eq. for level backfill • Use the Danish Eq. for sloped backfill 2. Downdrag – account for backfill settlement	1. At-Rest Soil Pressure: • Use Jaky's eq. for level backfill • Use the Danish Code Eq. for sloped backfill 2. Downdrag – account for backfill settlement 3. Lateral Seismic Force Use Wood's simplified procedure
Yielding Backfill	1. Active Soil Pressure: • Use Coulomb's Eq. with angle of interface friction	1. Active Soil Pressure • Use Coulomb's Eq. with angle of interface friction 2. Lateral Seismic Force a. Use one of the following: • Horizontal seismic coefficients values provided for OBE and MDE • Displacement Controlled Approach – seismic coefficient based upon allowable permanent horizontal wall displacement. b. Use Mononobe-Okabe Eq.
Hydrostatic Forces	1. Assume Static Pressure	1. Assume Static Pressure
Hydrodynamic Forces	-------	1. Use Westergard's Eq.

$$K_A = \frac{\cos^2(\phi-\theta)}{\cos^2\theta \cdot \cos(\theta+\delta) \cdot \left[1+\sqrt{\frac{\sin(\phi+\delta)\cdot\sin(\phi-\beta)}{\cos(\delta+\theta)\cdot\cos(\beta-\theta)}}\right]^2} \qquad (6)$$

where ϕ = soil internal friction parameter,

δ = angle of interface friction,

and β = top surface slope angle, positive when slope is upward when moving away from the structure.

The active earth pressure force P_A is computed using the following equation and is oriented at an angle δ to the normal along the back of the wall at a height equal to H/3 above the heel for soil without the presence of groundwater (Figure A-5-1):

$$P_A = \frac{1}{2} \cdot K_A \cdot \gamma \cdot g_{SI} \cdot H^2 \qquad (7)$$

$$P_{Ax} = P_A \cdot \cos(\delta+\theta) \qquad (8)$$

$$P_{Ay} = P_A \cdot \sin(\delta+\theta) \qquad (9)$$

where γ = moist or saturated soil density, kg/m^3,

g_{SI} = acceleration due to gravity, 9.80665 m/s^2,

and H = height moist or saturated backfill.

5.2.4.9.2. Surcharge Loadings

To approximate the additional lateral earth pressure on a wall due to surface loading such as strip loads, line loads, and ramp loads, use the elastic solutions provided in ITL-92-11, p. 45-46. For uniform surcharge loading, apply an effective vertical soil pressure equal to the magnitude of the surcharge.

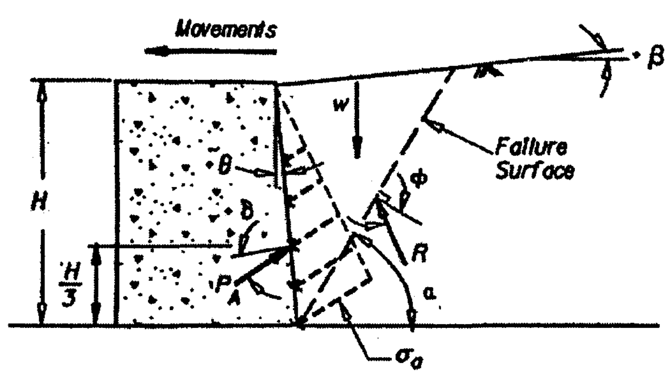

Figure A-5-1 Coulomb active earth pressure for inclined backfills and walls.

5.2.4.9.3. Static At-Rest Pressure

Jaky's equation and the Danish Code equation will be used to calculate the at rest pressure coefficient K_o for both the driving side and resisting side of the backfill. For horizontal and sloped backfill, the following equation may be used, respectively:

$$K_o = \begin{cases} 1-\sin\phi \\ (1-\sin\phi)\cdot(1+\sin\beta) \end{cases} \tag{10}$$

The at-rest earth pressure force P_o is computed using the following equation at a height equal to H/3 above the heel for soil without the presence of a water table (Figure A-5-1):

$$P_o = \frac{1}{2}\cdot K_o \cdot \gamma \cdot g_{SI} \cdot H^2 \tag{11}$$

5.2.4.9.4. Groundwater within Backfill – Static Case

For the case where groundwater is within the retained backfill, the soil is partitioned into two zones – moist or saturated soil above the water table and the submerged soil. The following equation provides the total soil force exerted on the wall (Figure A-5-2):

$$P = \begin{cases} P_1 + P_2 + P_3 \\ \frac{1}{2}\cdot K \cdot \gamma \cdot g_{SI} \cdot h_m^2 + K \cdot \gamma \cdot g_{SI} \cdot h_m \cdot h_s + \frac{1}{2}\cdot K \cdot (\gamma_{sat} - \gamma_w) \cdot g_{SI} \cdot h_s^2 \end{cases} \tag{12}$$

where K = K_A or K_o,

γ_{sat} = moist or saturated soil density, kg/m^3,

h_m = height moist or saturated backfill, m,

and h_s = height moist or saturated backfill, m.

The resultant location of the total soil earth force is:

$$h = \frac{\left(\frac{1}{6}\cdot K \cdot \gamma \cdot g_{SI} \cdot h_m^3 + h_s\right) + \frac{1}{2}\cdot K \cdot \gamma \cdot g_{SI} \cdot h_m \cdot h_s^2 + \left(\frac{1}{6}\cdot K \cdot (\gamma_{sat} - \gamma_w) \cdot g_{SI} \cdot h_s^3\right)}{P} \tag{13}$$

The equations are valid for the static at-rest and active soil loading conditions. For more information, refer to Draft EM 1110-2-2100 and ITL-91-11.

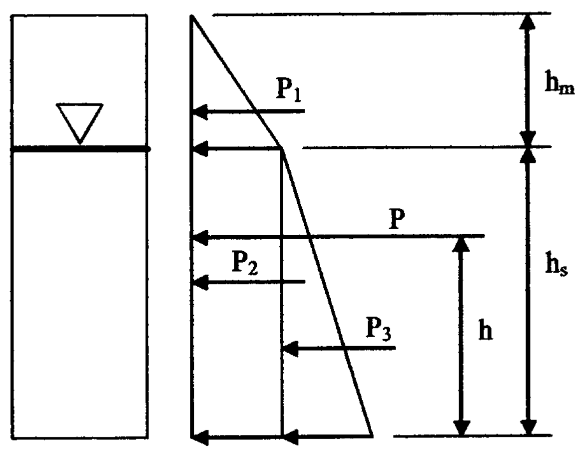

Figure A-5-2 Horizontal earth pressure and force diagram for groundwater case.

5.2.4.9.5. Seismic Active Earth Force

The Mononobe-Okabe theory is an extension of Coulomb's theory of static active and passive earth pressures and includes the effects of dynamic earth pressures on earth retaining structures. The theory accounts for the effects of vertical and horizontal earthquake motion through the use of vertical and horizontal acceleration coefficients - k_v and k_h, respectively. The inertial forces of the vertical and horizontal ground accelerations (opposite in direction of the ground motion direction) are equal to $k_v \cdot W$ and $k_h \cdot W$, respectively, were W is the weight of the structure

The Mononobe-Okabe theory provides both the static and dynamic earth forces. The Mononobe-Okabe equation for dynamic earth force of backfill without the presence of groundwater is

$$P_{AE} = \frac{1}{2} \cdot K_{AE} \cdot \gamma \cdot (1-k_v) \cdot H^2 \tag{14}$$

and acts at an angle δ from the normal to the back of the wall of ground height H. The back the wall may be sloped an angle of θ from a vertical plane, as shown in Figure A-5-1. The resultant forces in the x-y plane are provided by equations 4 and 5. The equation for the dynamic earth pressure coefficient is

$$K_{AE} = \frac{\cos^2(\phi-\psi-\theta)}{\cos\psi \cdot \cos^2\theta \cdot \cos(\psi+\theta+\delta) \cdot \left[1+\sqrt{\frac{\sin(\phi+\delta)\cdot\sin(\phi-\psi-\beta)}{\cos(\delta+\psi+\theta)\cdot\cos(\beta-\theta)}}\right]^2} \tag{15}$$

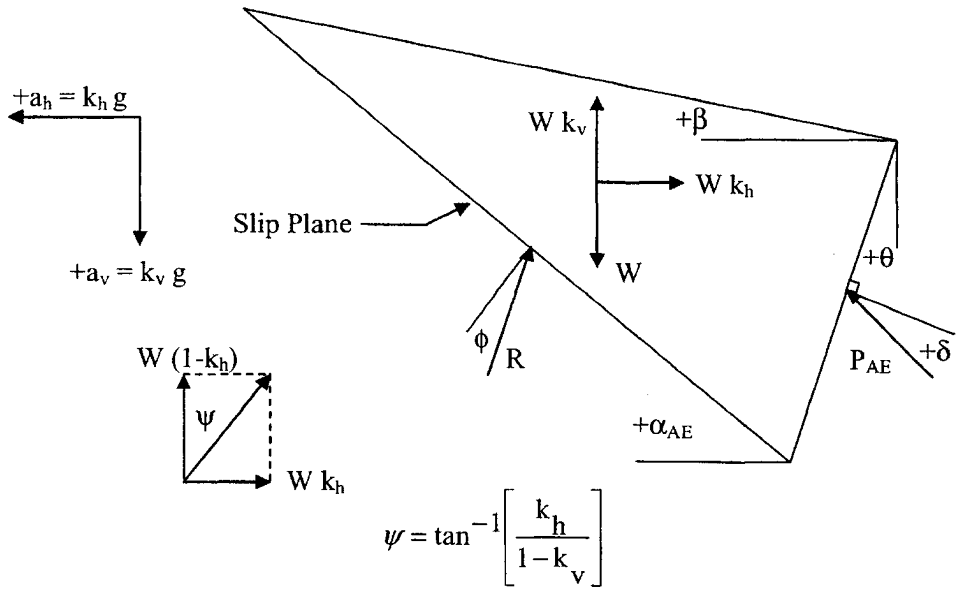

Figure A-5-3 Seismic wedge and force.

and the seismic inertial angle is equal to

$$\psi = \tan^{-1}\left[\frac{k_h}{1-k_v}\right] \tag{16}$$

The earth pressure force PAE provides both the dynamic active earth pressure force and the static earth pressure force. As a result, PAE is equal to

$$P_{AE} = P_A + \Delta P_{AE} \tag{17}$$

The resultant location of ΔP_{AE} is 0.6·H, where H is the total height of the backfill. The resultant location of P_{AE} is

$$Y_{AE} = \frac{P_A \cdot Y_A + \Delta P_{AE} \cdot (0.6 \cdot H)}{P_{AE}} \tag{18}$$

5.2.4.9.6. Displacement Controlled Method

For the displacement controlled method, pre-selected wall movements are used in the seismic stability analysis of an earth retaining structure. The seismic coefficient k_h is calculated from the pre-selected lateral displacement d_r for the monolith. Once the seismic coefficient is established, the stability of the monolith is calculated using the Mononobe-Okabe equations. A factor of safety is based on the lateral displacement selected for the structure versus the predicted displacement; the factor of safety is not based on the ratio of the structure's shear capacity and the resultant shear force. The maximum transmissible acceleration N* that may be applied to the structure by the earthquake ground motion is used to estimate the seismic coefficient k_h. The maximum transmissible ratio is provided by the equation

$$N^* = A \cdot \left\{0.66 - \frac{1}{9.4} \cdot \ln\left[\frac{d_r \cdot (A \cdot g)}{V^2}\right]\right\} \tag{19}$$

where $A \cdot g$ = base acceleration, in/sec^2 (equation is derived using U.S. customary units),

V = base velocity , in/s,

d_r = the pre-selected displacement, in,

and g = acceleration due to gravity, 386 in/s^2.

The factor of safety for sliding FS_S is used to calculate a new maximum transmissible ratio N^*_{new} using the equation

$$N^*_{new} = FS_S \cdot N^*_{old} \tag{20}$$

The new value for the maximum transmissible ratio N^*_{new} is used to calculate a new lateral displacement value for the monolith using the equation

$$d_{r\text{-}new} = \left[\frac{495 \cdot V^2}{(A \cdot g_I)}\right] \cdot \exp^{\left(-9.4 \cdot \frac{N^*_{new}}{A}\right)} \tag{21}$$

The new value is substituted into equation 15 to calculate a new estimate for maximum transmissible ratio N* and the process is repeated until d_r converges.

5.2.4.9.7. Groundwater Within the Backfill – Seismic Case

For the preliminary design and stability analyses of the lock and entrance walls, the assumption shall be made that the pore water pressure does not change during an earthquake and that an excess pore water condition does not exist. The seismic angle to be used for partially submerged backfill is

$$\psi_{e1} = \tan^{-1}\left[\frac{\gamma_e \cdot k_h}{\gamma_b}\right] \tag{22}$$

where $\gamma_b = (\gamma_{sat} - \gamma_w)$

and

$$\gamma_e = \left(\frac{h_1}{h}\right)^2 \cdot \gamma_1 + \left[1 - \left(\frac{h_1}{h}\right)^2\right] \cdot \gamma_{sat} \tag{23}$$

Equation 16 should be substituted into equation 11 for the case of a partially submerged backfill (Figure A-5-4)

5.2.4.9.8. Seismic At-Rest Earth Force

A monolith that is massive and stiff does not move (rotational and/or lateral) a sufficient amount for the shear strength of the backfill to fully mobilize. As a result, the earth pressures are at the at-rest condition. For the seismic case, Wood's equation shall be used to calculate the seismic at-rest earth force. The equation is

$$F_{sr} = k_h \cdot \gamma \cdot H^2 \tag{24}$$

where γ = γ_m, γ_{sat} or γ_e (depending on the presence of groundwater), kg/m^3,

and H = total height of the backfill, m.

The resultant force acts at a height equal to 0.63·H above the back of the wall. Equation 20 does not account foe the static at-rest earth force. The at-rest earth force must be applied to the structure in the stability analysis in addition to the seismic at-rest earth force.

5.2.4.9.9. Structural Inertial Force

The inertial force for the lock and entrance wall monoliths is provided by the equation:

$$F = k_h\, W. \tag{25}$$

The inertial force is assumed to act in a horizontal direction at the center of mass of the structure, and in the opposite direction of the ground motion.

5.2.4.9.10. Hydrodynamic Water Pressure

Water that is above the ground surface and adjacent to a structure will increase the inertial forces that the structure will experience during an earthquake. As the structure is displaced during an earthquake, it moves through the surrounding water. This movement causes hydrodynamic forces to occur on the structure.

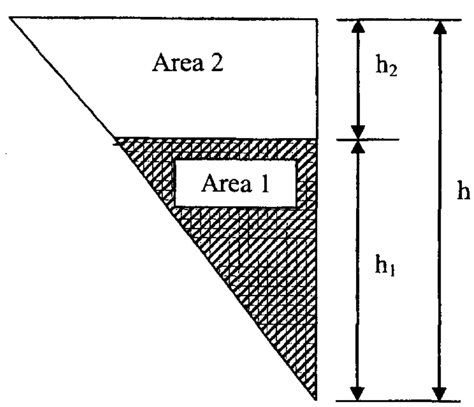

Figure A-5-4 Effective unit weight for partially submerged backfills

The water inside (culvert) and surrounding the structure (lock chamber and water saving basin) is a source of hydrodynamic forces. The hydrodynamic force of the water during an earthquake shall be calculated using the following Westergaard formula:

$$PE = (7/12)\ k_h\ \gamma_w\ g_{st}\ h \qquad (26)$$

where PE = hydrodynamic force per unit length,

and h = depth of water in culvert, lock chamber and/or stilling basin.

The hydrodynamic force can either increase or decrease the water force and is added to or subtracted from the static water pressure force to get the total water force on the structure. The hydrodynamic force acts in the opposite direction of the earthquake ground motion. The line of action for the force PE is 0.4 h above the ground surface.

5.2.4.10. Spreadsheet Calculations

Spreadsheet calculations demonstrating the use of the equations in calculating static and dynamic earth forces and water forces and conducting the stability analyses with respect to sliding, overturning and bearing are provided in Appendix A. Two examples are provided. The first provides the seismic stability analysis for a non-yielding structure. The second provides a seismic stability analysis for a yielding structure undergoing a predetermined lateral displacement.

5.2.4.11.Rock/Soil Anchors:

5.2.4.11.1. Rock and Soil Anchors

Tie-backs will be designed according to the recommendations of the Post Tensioning Institute, “Recommendations for Prestressed Rock and Soil Anchors, 1996”.

5.2.4.11.2. Cement Grout for Bond Zone:

Cement: Cement grout shall be made from either Type I, II, III, or V Portland cement conforming to ASTM C 150 specification.

Grout Mix Design: A neat cement grout made with a w/c ratio of 0.4 to 0.45 by weight and Type I cement.

5.2.4.11.3. Bond:

Length: The bond length is not less than 4.6 m for ASTM A 416M strand and 3.0 m for ASTM A722M Type II bars.

Ultimate Bond Stress: The ultimate bond stress between the rock and the anchor grout can be approximated by using the value of 10% of the unconfined compressive strength of the rock, up to a maximum value of 3.1MPa.

5.2.4.11.4. Free Stressing Length:

The free stressing length for rock and soil anchors shall not be less than 4.6 m for strand tendons and 3 m for bar tendons. The minimum stressing length recommended is to prevent significant subsequent reductions in transfer load due to seating losses. Since wedges on epoxy coated strand cannot be powerseated and must bite through the epoxy coating, they have significantly higher seating losses than wedges on bare strand.

5.2.4.11.5. Anchor Spacing in Bond Length:

Center-to-center spacing between bond lengths shall be at least 4 times their nominal diameter and normally should be greater than 1.2 m. The intent of the minimum separation of bond length is to prevent both anchor interaction and intersection due to drilling deviations.

5.2.4.11.6. Drill Hole Diameter:

The drill hole diameter for multiple-element tendons shall be large enough so that the area of prestressing strand within the drill hole does not exceed 15% of the total area of the hole. The drill hole shall be sized to provide a minimum of 13 mm grout cover over the tendon. For bar anchors, the grout cover requirement only governs.

5.2.4.11.7. Drilling:

The drill bit or casing crown shall not be less than 3 mm smaller than the specified hole diameter.

5.2.4.11.8. Hole Alignment and Tolerances:

The anchor hole may be located within 300 mm (either direction) of its plan location. The deviation of the hole's entry angle from its specified inclination shall be no more than ±3 degrees.

5.2.5. Water Saving Basin Walls, and Entrance Walls

5.2.5.1. Introduction

A multidisciplinary seismic analysis team of Corps of Engineers (COE) technical personnel was assembled to specify appropriate values of seismic coefficients for the Atlantic Third Lane Locks and appurtenant structures, and to provide technical assistance in the concept design of the lock walls, entrance walls, and WSB walls. The seismic analysis team consists of Robert M. Ebeling, Robert L. Hall, Enrique E. Matheu, and Donald E. Yule, Engineering Research and Development Center (ERDC).

In preparing these recommendations the team reviewed the state-of-the-knowledge regarding the seismic hazard for Panama and more specifically the Gatun Lock vicinity, the recent seismic evaluation studies of the Canal's existing structures, and the proposed structural performance goals. Additionally, as a frame of reference, COE policy was assumed as the model basis for these initial recommendations.

5.2.5.2. Review of Seismic Hazard for Panama

There are two efforts that have significantly contributed to the state-of-the-knowledge of regional seismicity of the Republic of Panama. These are studies performed by the U.S. Geological Survey (USGS) for support of the analysis of existing Panama Canal structures and regional seismic hazard studies performed jointly by the University of Panama and the Research Council of Norway (NORSAR) in support of national seismic hazard mitigation initiatives. These studies are cited in the Reference section.

5.2.5.2.1. Background

The general seismic setting for the region that includes the Republic of Panama is a moderately active seismic region controlled by the close proximity to a complicated set of tectonic plate boundaries. Panama, itself, is located on the Panama Block microplate surrounded by four major plates which are in the process of accommodating their relative individual driving forces and resulting deformations. It is interesting to note that one of the reasons supporting selection of the canal's location was the region's apparent a-seismicity. Current technical opinion now recognizes Panama as a more seismically active region. Although, recent seismicity is concentrated on the borders with Costa Rica and Columbia there is evidence for consideration of seismicity closer to the canal based on a significant historical earthquake in 1882 offshore, northeast of Colon, with a 7.7 estimated magnitude. This earthquake is associated with the North Panama Deformed Belt (NPDB) seismotectonic province that parallels the Caribbean coast, offshore of Panama (Camacho et al, 1994). The recent 1991 Limon earthquake offshore of Costa Rica, with a 7.5 magnitude, represents a significant earthquake associated with the NPDB that demonstrates this source's seismic capacity. Therefore, considering the current state-of-knowledge of the region's seismic setting and historical and instrumental seismic data, credible and significant seismic hazards should be considered in any engineering design.

5.2.5.2.2. Regional Seismic Hazard Studies

There has been considerable effort to develop seismic hazard maps for the region. These regional studies have culminated in the 'NORSAR' probabilistic seismic hazard curves for selected major cities and national maps of peak ground accelerations (PGA) for a suite of return periods (Rojas et al, 1993; Camacho et al, 1994). The results of these studies are presented in Table A-5-8 and Figure A-5-5 for Colon, Panama, which is near the Gatun site. The table and accompanying figure also compare the Colon PGA hazard curve with the Maximum Credible Earthquake (MCE) determined by the USGS for the Gatun vicinity from source #1. The MCE is discussed in the next Section. The comparison of the return periods for the MCE level PGA is also shown for these two studies. For example, a PGA for a return period of 1,000 years is 0.232g from NORSAR hazard curve. If we extrapolate the median MCE PGA of 0.313g on this hazard curve it corresponds to a 2,700-year return period.

5.2.5.2.3. Site-Specific Seismic Hazard for Gatun Dam and Vicinity

In support of the seismic stability analysis of Gatun Lock and Dam the ACP contracted the USGS to conduct seismic hazard assessment studies and then develop earthquake ground motions for design (Schweig et al, 1998; Joyner, 1998). This effort continues in that currently the USGS is developing equal hazard spectra for use in analysis of the existing Canal structures to augment their deterministic results and the probabilistic PGA maps developed by NORSAR.

The USGS study presented three earthquake sources that comprise the seismic hazard for the Gatun vicinity (Schweig, 1998). Source #1, associated with the NPDB, was determined to be the most critical for seismic safety analysis. This source provided an MCE event with a magnitude 7.7 at a closest distance of 35 km with an estimate of recurrence from 330 to 1,000 years. This recurrence estimate is based on assigning a slip rate from a general relationship between earthquake magnitude and slip and then using the convergence rate of the regional plates to arrive at a recurrence interval. The other zones were less well constrained with one zone having an estimated recurrence of 10,000 – 20,000 years and the third source was derived from some evidence that small earthquakes (magnitude 5 to 6) were possible within 2 km of the site. This study was reviewed during the 8th Meeting of the Geotechnical Advisory Board (GAB), which recommended use of a MCE from source #1, and to investigate in a probabilistic context consideration of earthquakes from sources #2 and #3. Based on these peer-reviewed seismic sources, the USGS developed design ground motions for an MCE for source #1 (Joyner, 1999), with resulting mean and mean plus one standard deviation PGA values of 0.313g and 0.544g. The design ground motions are further discussed in the next section.

It appears that comparison of the USGS deterministically derived design ground motions and the NORSAR probabilistic derived hazard maps produce inconsistent hazard estimates. However, some very important points must be considered. Both approaches recognize a maximum magnitude of 7.7 for the sources. However, the NORSAR probabilistic hazard estimate shows the seismic hazard to be higher on the west and east boundaries of Panama. This is a result of the current earthquake catalog that shows increased seismicity in these two areas with respect to central Panama. Also, their seismic source zonation for Panama provides for a single zone with a resultant uniform hazard for a region that contains the Canal. The MCE deterministic scenario approach used for the analysis of the existing Gatun Locks recognizes the continuity of the structural geologic setting which places the seismic source capability evidenced by recent seismicity, the magnitude 7.5 Limon Earthquake at the western end of the NPDB to be possible along the entire length of this structure. This geologic-based source continuity and seismic capacity of the NPDB is further supported by the magnitude 7.7 1882 earthquake located near the midpoint of this structure. The deterministic approach considered the reasonable upper bound hazard at the closest distance, whereas the probabilistic approach is based on regionally convolved recent patterns of seismicity and provides a mean estimate of the seismic hazard. To keep these hazard estimates in the right perspective, one should note that the ground motion attenuation relationship used for these studies contains a factor of two for one standard deviation from the mean estimate, which provides some measure of the uncertainty in the results.

5.2.5.3. Previous Seismic Evaluation Studies for Existing Canal Structures

The increased concerns about the seismicity of the region motivated the initiation of a comprehensive evaluation of the Gatun Dam, Spillway and Locks to investigate the response of these structures under seismic loading. Initial efforts focused on the Gatun Dam System, aimed to control the water of the Chagres, Trinidad, and Gatun Rivers to form the Gatun Lake. This lake serves a most critical function since it provides all the water needed for operations of the Panama Canal. Additional seismic evaluation studies were also performed for the Madden Dam, a 223-ft high concrete gravity dam located 19.3 km upstream from the Panama Canal on the Chagres River, and the Miraflores Spillway, near the Pacific end of the Canal.

Technical personnel of the Geotechnical Branch of the Panama Canal Authority carried out the seismic evaluation studies, which were initially focused on the Gatun concrete spillway. Dr. Robert L. Hall (ERDC-GSL) provided overall technical assistance for this effort. Prof. Anil K. Chopra (University of California, Berkeley) was part of the review process, which included a meeting in January 1999 at the Waterways Experiment Station with participation of technical staff from Panama. The preliminary evaluation studies were completed in June 2000 in the form of a series of technical reports (Abrego and DePuy, 2000; Abrego, 2000a; Abrego, 2000b). A general review meeting was held in July 2001 at the University of California, Berkeley, and in addition to Prof. Chopra and Dr. Hall, the ad-hoc review team included Dr. Yusof Ghanaat (Quest Structures), and Dr. Enrique Matheu and Mr. Don Yule (ERDC-GSL).

As presented in the previous section, the earthquake source zones capable of affecting the Gatun Dam System were discussed at the 8th Meeting of the Geotechnical Advisory Board in July 1998. The Geotechnical Advisory Board consisted of the following members: Drs. James M. Duncan, Robert L. Schuster, Norbert R. Morgenstern, Robert L. Wesson and William F. Marcuson III. The Board anticipated that the analyses would be dominated by ground motions associated with source #1, corresponding to a magnitude 7.7 at a distance of 35 km with a recurrence interval of 330~1,000 years. The final definition of the seismic loading used for the seismic evaluation studies was based on the recommendations from Dr. William B. Joyner (Joyner, 1999). The design ground motion was selected based on data from the 1985 Chile earthquake, and mean and mean plus one standard deviation values of 0.313g and 0.544g were recommended, respectively, for the peak ground acceleration (PGA). These were specified as rock motions. These values were reviewed and accepted by the Geotechnical Advisory Board during the 9th Meeting in September 1999. The Board also accepted the Pichilemu record (1985 Chile earthquake) suggested by Joyner to be used as input in the seismic evaluations. The Board, however, suggested the additional inclusion of the Siquirres record (1991 Limon, Costa Rica, earthquake) among the records used in the analyses. The records were scaled to a PGA level of 0.544g, which corresponds to the mean plus one standard deviation value and it is representative of the MCE.

Seismic stability analyses were carried out to evaluate the dynamic performance of the Gatun concrete spillway using a simplified procedure for the monolith section, and a finite element procedure for the analysis of the piers. The simplified procedure used for these studies yields lateral forces associated with the fundamental vibration mode of the monoliths, according to the methodology described in the WES

Technical Report SL-89-4. The finite element analyses were carried out using the finite-element software SAP2000.

Core samples were obtained to characterize the material properties of the concrete for both Gatun and Miraflores Spillways. No concrete material testing was performed for the Madden Dam and existing information was used to estimate the values of the corresponding material parameters. Available results from extensive previous studies performed by the technical personnel from the Waterways Experiment Station were used to determine some of the foundation parameters for the evaluation of the Gatun Spillway (Yule 1996).

The dynamic stresses computed for the Gatun Spillway were below the dynamic tensile strength of the concrete, and therefore it was concluded that the main cross-section of the spillway has sufficient capacity to withstand the design ground motions. Global stability analyses (overturning and sliding) were also performed using the traditional pseudo-static approach based on a seismic coefficient value of 1/3 of the PGA. In the analysis a range of conditions was assumed to represent the shear strength characteristics of the interface between the concrete section and the foundation. In addition, sliding block analyses were also performed to estimate the magnitude of the resulting displacement. The results indicated that the spillway is structurally stable against overturning and sliding along the base.

The seismic analyses performed for spillway and non-overflow sections of the Madden Dam revealed stresses below the estimated tensile strength of the concrete material. It was concluded that the dam has sufficient capacity to withstand the design ground motion at a 0.544g level, even under the conservative assumption of a rigid foundation. Global stability analyses (overturning and sliding) were also performed using the traditional pseudo-static approach based on a seismic coefficient value of 1/3 of the PGA. For the condition in which no cohesion is assumed at the dam-rock interface, the sliding safety factors are below the minimum values required by Corps of Engineers guidance documents. However, it is noted that, based on 10-year data, the level assumed for the Alajuela Lake was overly conservative. Consideration of the normal operating conditions for this lake reveals safety factors that are above the minimum threshold most of the time.

The conclusions for the stress analyses of the main section of the Miraflores Spillway were similar to those for the Gatun case, and it was estimated that the spillway would perform satisfactorily under the specified seismic loading. A global stability evaluation was also performed following the same methodology used for the other analyses. The results indicated that the spillway is structurally stable against overturning and sliding along the base.

At its July 2001 meeting (University of California, Berkeley), the ad-hoc technical review team agreed with the basic methodology used for the dynamic stress analyses and the interpretation of their results. However, specific recommendations were made towards the need to use a generalized (smooth) design spectrum for further studies. These smooth spectra are developed from large data sets that are analogous for specific seismic settings and provide more robustness in estimating spectra. The review team indicated that there are several peer-reviewed earthquake strong motion spectral attenuation relationships that could be used to generate these spectra. In addition, since a site specific PGA had been already determined, it was suggested to use this design parameter to anchor the spectra developed from the selected attenuation relationships. The review team also provided specific recommendations regarding the global stability evaluations. In particular, it was recommended that additional stability analyses should be carried out with an increased value of seismic coefficient, equal to 2/3 of the PGA selected for the MCE. This value should be used in the additional studies to be added to the other load cases presented in the reports. It should be noted that no substantial change in the nature of the conclusions was expected for this new load case if the stabilizing effects provided by downstream structures such as aprons were included in the calculations. Further studies were recommended to establish the dynamic performance of the piers in both the Gatun and Miraflores Spillways.

5.2.5.4. Recommendations for preliminary seismic design of the new structures

The unprecedented criticality of the new lock system translates in the need for very severe performance criteria under seismic loading. The current COE regulation (ER 1110-2-1806) establishes two levels of seismic loadings and corresponding performance conditions: Maximum Design Earthquake (MDE) and Operating Basis Earthquake (OBE). The MDE represents the maximum level of ground motion for which a structure must be designed to perform without catastrophic failure, although severe damage or economic loss may be tolerated. For critical features, the MDE is set equal to the Maximum Credible Earthquake (MCE), defined as the greatest earthquake that can be generated by a specific source. For all other features, the MDE is typically selected as a lesser earthquake than the MCE; this provides more economical designs and still meets appropriate safety standards. The OBE represents an earthquake that can reasonably be expected to occur within the service life of the structure, which is expected to exhibit little or no damage, and no operational interruption. Typically, critical projects are those whose failure during or immediately following an earthquake could result in loss of life.

The new Panama locks will differ from the typical Corps of Engineers experience with locks because it could be categorized as a critical project from an economic point of view. Because of the exceptional circumstances around this project, loss of operation for an extended period of time is economically equivalent to catastrophic structural failure. Therefore, the MDE level of seismic demand should be set equal to the corresponding MCE level for those project components and subsystems that are essential for continuity of operation.

In addition, the unparallel importance of the new locks in Panama's economic structure reduces drastically the amount of damage that can be tolerated under extreme seismic loading conditions. Any damage expected under these conditions should not cause long-term loss of operation and should be rapidly repairable. Damage levels severe enough to cause loss of operation for an extended or indefinite period of time should not be accepted, even if the global structural integrity is preserved. Therefore, any damage that is considered during the MDE event (that for the critical structural/operational components of the project should be set equal to the corresponding MCE level, as discussed before) should be carefully assessed for its impact on impeding operations of the new lock system.

The seismic analysis team determined an appropriate value of the peak ground acceleration (PGA) to be used in preliminary design and feasibility studies. Since the existing Gatún locks and other structural features have been recently subject of seismic evaluation studies, it is reasonable to consider the use of the same ground motions for the preliminary analyses associated with the concept design of the new lock system. These ground motions for the Gatún site were based on a deterministic approach for a MCE, as the Gatún locks constitute a critical structure. These ground motions were developed by the USGS, with a MCE characterized by a magnitude Mw = 7.7 at a distance d = 35 km. The corresponding PGA value considered for these seismic evaluation studies is 0.544g, which represents a mean plus one standard deviation level for the MCE event. The PGA value for the corresponding mean value for this same earthquake and return period is 0.313g.

The team members are aware that the USGS will be under contract in the near future to perform a probabilistic study that will enable developing equal hazard spectra. This complete definition of the seismic ground motions is critical before addressing a detailed structural design. The critical nature of the new Panama Canal Locks, the information from past seismic studies, and the estimated return period of 330~1,000 years (Schweig, 1998) would seem to support the use of the 0.544g mean plus one standard deviation. However, the estimate of the corresponding return period based on previous studies by Camacho raises some doubt about the use of a mean plus one standard deviation value. The adoption of such a severe ground motion will require the use of detailed analysis procedures and well-defined performance criteria. The Corps of Engineers guidance (ER 1110-2-1806) allows for the selection of the mean ground motion for the MDE. Since detailed performance criteria have not developed and the efforts required for detailed analysis using refined numerical procedures would be excessive at this stage, this team supports the use of a mean ground motion of 0.31g. This level of ground motion can be used with simplified tools and assuming mostly elastic response of the structure. The use of 0.31g mean values should be reevaluated when the results of current and future seismic ground investigations are available.

The OBE conditions represent an earthquake that can reasonably be expected to occur within the service life of the project, that is, with a 50-percent probability of exceedence during the service life. This corresponds to a return period of 144 years for a project with a service life of 100 years. For the new lock system, a service life equal to 100 years is recommended. The team will recommend an OBE event based on a service life of 100 years. A final estimate for the OBE can be determined once the USGS probabilistic studies are concluded and their results are provided. Until this study is complete, using the existing data, Camacho 1997 study, an extrapolation of their seismic hazard curve to a return period of 144 yrs yields approximately 0.14g's for a rock site.

As recommended for the preliminary stability evaluations of existing concrete structures not in contact with earth, a seismic coefficient equal to 2/3 of the PGA could be used for this type of concrete structures. However, in the case of earth retaining structures that are allowed to experience some movement during the earthquake, the displacement-controlled approach incorporates wall movements explicitly in the stability analysis of earth retaining structures (Ebeling and Morrison, 1992). It is, in effect, a procedure for choosing a seismic coefficient based on upon an explicit choice of an allowable permanent displacement. Having selected the seismic coefficient, the usual stability analysis against sliding is performed, including the use of Mononobe-Okabe equations. No safety factor is applied to the required weight of wall evaluated by this approach; the appropriate level of safety is incorporated into the step used to calculate the horizontal seismic coefficient.

Normally, vertical accelerations are considered in seismic stability evaluations. However, so long as simplified conservative engineering procedures are used in preliminary analyses, vertical accelerations need not be included in these preliminary calculations. When the seismic engineering evaluations proceed to the design memorandum stage and more sophisticated modal response- or time history-based engineering procedures are used to assess the potential for damage to the lock features and the USGS site-specific earthquake hazard results become available, vertical accelerations shall be included in the analyses. The multidisciplinary seismic analysis team at ERDC can provide assistance in the determination of an appropriate vertical PGA values as well as the selection of horizontal and vertical time histories to be used for time history-based seismic analyses.

5.2.5.5. Recommendations

1) A design service life for the lock equal to 100 years is recommended. The impact of a 100-year versus a 50-year life is that the seismic loading design requirement for an OBE event will increase.

2) A PGA value of 0.31g is supported for the preliminary design of the new lock system for the MDE event.

3) A PGA value of 0.14g is recommended for the preliminary design of the new lock system for the OBE event. This current recommendation should be reviewed based on future results of ongoing studies in Panama by the USGS.

4) The corresponding seismic coefficient should be determined based on the choice of an allowable permanent displacement. For typical values of displacement, the seismic coefficient is normally about 2/3 of the PGA.

Table A-5-8 PGA Seismic Hazard Curve for Gatun Locks and Comparison to MCE

Probabilistic Seismic Hazard Curves							
Location	Longitude	Latitude					
Colon	-79.901	9.355					
	Camacho et al: 1997						
	Extrapolated			Reported			
	Mean values						
Return Period (years)	31	63	125	250	500	1000	
PGA (g) soil	0.123	0.148	0.177	0.212	0.256	0.305	
rock	0.094	0.112	0.135	0.161	0.194	0.232	
	Camacho et al: 1994						
Return Period (years)	2	10	50	100	250	500	1000
PGA (g) soil							
rock	0.031	0.061	0.107	0.127	0.168	0.209	0.250
MCE (USGS)							
	Mean		M+1stdev		Recurrence	USGS	
Source #1	0.313		0.544		330 to 1,000		

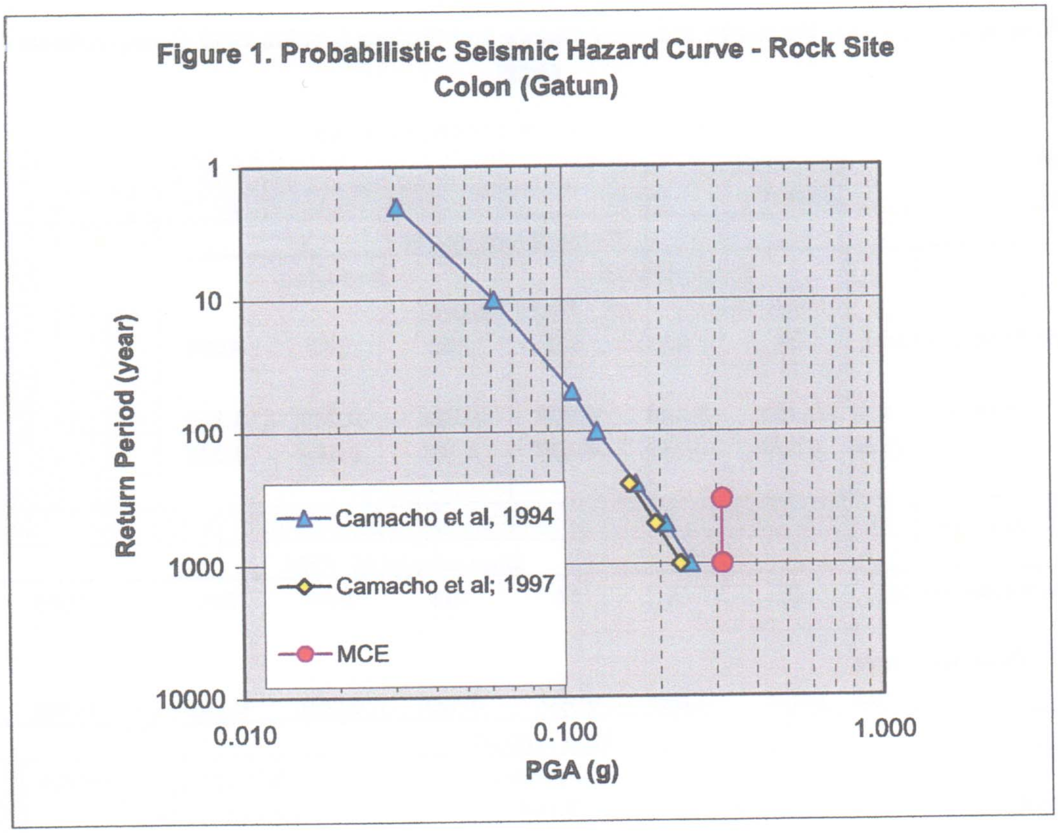

Figure A-5-5 Probabilistic Seismic Hazard Curve

5.2.6. Cofferdams

Layout and design of temporary cofferdam structures will developed to dewater the construction site to build portions or the entire Third Lane Lock structure. A cellular cofferdam was anticipated as a required project feature. However, it was determined that earthen dam construction would be used for the Double-Lift Configuration. The criteria presented would be modified as required to reflect the actual cofferdam arrangement selected for implementation.

Sheet pile cells will be analyzed with the program CCELL. The program CCELL is based on EM 1120-2-2503, Design of Sheet Pile Cellular Structures. For a coffer cell on a rock or soil foundation, the program analyzes six modes of failure: sliding stability; shear failure along the center line of the cell; horizontal shear (Cummings Method); interlock failure; pullout of outboard sheeting and bearing capacity of the rock foundation. Seismic loading conditions will be developed to reflect the risk of damage and the temporary nature of the cofferdam. Factors of Safety based on EM 1120-2-2503 will be followed as a guide with justification provided for any recommended deviations.

Table A-5-9 Cofferdam Required Factor of Safety

Loading Conditions			
{PRIVATE }	**Normal**	**Temporary**	**Seismic**
Bursting (Interlock Tension)	2.00	1.50	1.3
Slip Failure along Vertical Center Plane	1.50	1.25	1.1
Horizontal Shear (Cumming's Method)	1.50	1.25	1.1
Pullout of Outboard Sheeting	1.50	1.25	1.1
Bearing Failure of the Foundation	3.00	3.00	1.5
Sliding on the Base	1.50	1.50	1.3

5.2.6.1. Loading Conditions

Several loading conditions will be developed and evaluated for cofferdam construction and temporary use. The limit states identified above will be checked. The design will consider excavation, filling condition, drain construction, normal conditions, tidal and lake elevation variations, and seismic events.

5.2.6.2. Steel Sheet Pile

Material selections will be based on market availability, environment conditions, and site-specific conditions such as drivability.

5.2.6.3. Allowable Foundation Bearing Capacity

The foundation bearing pressure will be estimated based on requirements of EM 1110-2-2200.

5.2.6.4. Steel Design Criteria For Cofferdams

5.2.6.4.1. Structural Shapes, Plates and Bolts

a General:

Steel bracing was designed using the allowable stress design method in accordance with the AISC (1989) as modified by Engineer Manual 1110-2-2105, "Design of Hydraulic Steel Structures".

b Material Properties:

The following material properties are assumed for the structural steel components (for sheet piles and appurtenant metals see below):

Plates, Bars and Structural Shapes:	ASTM A 572M
H-Piles:	ASTM A 572M
Bolts:	ASTM A 490M
	and
	ASTM A 325M
Welds:	E70XX Electrodes
Stainless Steel:	AISI Type 304

c Allowable Stresses:

The bracing is considered as a Type B HSS, with allowable stresses equal to 0.83 times that allowed by AISC. This is deemed appropriate due to the long-term duration of the cofferdam and to have the proper degree of conservatism for the cofferdam, which is a major and critical component of the new lock work.

5.2.6.4.2. Sheet Piles

a General:

The allowable stress design method will be used for design of the cofferdam cell sheet piles. All designs will be in accordance with EM 1110-2-2504, Engineering Design of Sheet Pile Walls.

b Material Properties:

Z Piling:	Sections conforming to ASTM A 690M
Flat Web Piling:	Sections conforming to ASTM A 328M

Appurtenant Materials:

Plates, Bars, Structural Shapes:	ASTM A 588M
Bolts, Nuts and Washers:	ASTM A325M, Type 3

c Allowable Stresses:

The allowable stresses were assumed as follows:

Bending: $f_b = 0.5\ f_y$

Shear: $f_v = 0.33\ f_y$

Structural Design Feature Proposed COE Software

Design Computations	MS Excel*, MathCad*

5.3. Design Aids

The concept design shall utilize the following Corps of Engineers (COE) proprietary software, in addition to commercially owned software* which is licensed to the Pittsburgh District.

5.3.1. Lock Structures

• STAB.	Concrete general stability analysis on 2-D section.
• GTSTRUDL 26.	Georgia Tech Research Corporation – is a general-purpose finite element analysis program that provides nonlinear, spectral, and time history analysis.
• STAAD/Pro	General purpose structural analysis.
• Concrete Columns/Shafts	PCA Col*
• Concrete Mat	PCA Mats
• Pile Analysis	COM624G, CPGA, FB Pier*

5.3.2. Lock Appurtenances.

• CMITER	Structural design of miter gates.
• STAAD/PRO	2-D Structural frame analysis.

5.3.3. Lock Entrance Walls.

• CSLIDE	Sliding stability analysis of concrete structure.
• STAB	Concrete general stability analysis on 2-D section.

5.3.4. Construction Cofferdam.

• CCELL	Design/Analysis of sheet pile cells
• CWALSSI	Design/Analysis of sheet pile walls
• UTEXAS3	Deep Seated Sliding and Slope Stability Analysis

6. MECHANICAL SYSTEMS

6.1. References

Mechanical systems and components are designed in accordance with the criteria and guidance furnished in portions of the Corps of Engineers manuals for engineering and design, industry standards and other technical references as follows:

Engineering Manuals:

- EM 1110-2-2602 Planning and Design of Navigation Locks, 30 September 1995.
- EM 1110-2-2610 Gate Operating Equipment for Navigation Locks and Spillways
- EM 1110-2-2703 Lock Gates and Operating Equipment, 30 June 1994.

Engineering Regulations:

- ER 1110-2-1150 Engineering and Design for Civil Works Projects, 31 August 1999
- ER 1110-2-1200 Plans and Specifications for Civil Works Projects, 30 October 1993.

Miscellaneous References:

- ANSI
- ASTM
- ASME
- MSS
- NFPA
- NFLPA
- SAE

6.2. Lock Gate Operating Machinery

The lock gate operating machinery will be the most advanced equipment available that is appropriate for the gate type selected. An initial screen of various gate operating systems for each gate type will be performed based on the referenced design criteria, engineering judgment, and adoption of features from similar projects. A separate report will be developed to summarize findings. Specific recommendations will be presented and coordinated with the ACP to define the direction of following study efforts. Upon completion and concurrence from the ACP on the gate machinery selection, the narrative discussions provided in this section will be replaced with project specific design criteria.

The basic considerations to be applied in the gate machinery selection process include:

- Direct connected cylinders – As discussed in the Scope or Work, direct connected hydraulic cylinders will be considered to the extent possible for the application.
- Maintenance – The gate machinery will have low maintenance requirements, with emphasis on ease of service and repair.
- Durability – The machinery selected will be very reliable and unaffected by the heat and humidity of a tropical environment.

A cursory level of design and analysis will be performed in the gate machinery selection. However, primary components will be sized to prove feasibility of concepts. The analysis will consider the following:

- Operating Loads
- Ship Impact
- Wave Action

6.3. Filling and Emptying Valve Operating Machinery

The valve operating machinery will be the most advanced equipment available that is appropriate for the valve type selected. An initial screen of various valve operating systems for each valve type will be performed based on the referenced design criteria, engineering judgment, and adoption of features from similar projects. A separate report will be developed to summarize findings. Specific recommendations will be presented and coordinated with the ACP to define the direction of following study efforts. Upon completion and concurrence from the ACP on the valve machinery selection, the narrative discussions provided in this section will be replaced with project specific design criteria.

The basic considerations to be applied in the valve machinery selection process include:

- Direct connected cylinders – As discussed in the Scope or Work, each valve actuator will be either directly connected to the valve or utilize an appropriate valve linkage, depending on the lock wall and valve arrangement used.
- Maintenance – The valve machinery will have low maintenance requirements, with emphasis on ease of service and repair
- Durability – The machinery selected will be very reliable and unaffected by the heat and humidity of a tropical environment
- A cursory level of design and analysis will be performed in the valve machinery selection. However, primary components will be sized to prove feasibility of concepts. The analysis will consider the following for strength and serviceability analysis:
- Operating Loads
- Hydrodynamic loads
- Surge effects

7. ELECTRICAL SYSTEMS

7.1. References

Electrical systems and components will be designed in accordance with the criteria and guidance furnished in portions of the Corps of Engineers manuals for engineering and design, industry standards and other technical references as follows:

Engineering Manuals:

- EM 1110-2-2602 Planning and Design of Navigation Locks, 30 September 1995.
- EM 1110-2-2610 Gate Operating Equipment for Navigation Locks and Spillways
- EM 1110-2-2703 Lock Gates and Operating Equipment , 30 June 1994

Engineering Regulations:

- ER 1110-2-1150 Engineering and Design for Civil Works Projects, 31 August 1999

Technical Publications:

- ANSI C2 National Electrical Safety Code
- NFPA 70 National Electrical Code
- NFPA 79 Electrical Standard for Industrial Machinery
- NFPA 101 Life Safety Code
- NFPA 110 Emergency and Standby Power Systems
- Underwriter Laboratories, Incorporated
- Illuminating Engineering Society (IES) Lighting Handbook

Miscellaneous References :

- TM 5-811-1 Electric Power Supply and Distribution
- TM 5-811-14 Coordinated Power Systems Protection

7.2. Lock Controls

Conceptual control system architecture drawings will be developed to illustrate the control system. A state-of-the-art, truly open, distributed control system configuration will be shown. A network architecture diagram will be provided to illustrate the connections and flow of information from the operating machinery. Careful consideration shall be given to expandability and redundancy. All operating machinery conditions will be displayed on personal computers and data trending will provide the operators with an operating history to aid in preventative maintenance. Proposed control system software and operator interface software will be indicated on the control system architecture drawing.

7.3. Electrical Distribution Systems

Conceptual drawings will be developed to meet the power needs of the proposed operating machinery and lighting for the locks. Single line drawings, which identify how power will be distributed throughout the lock electrical system will be illustrated. Electrical diagrams will indicate the various components of the lock distribution system including switchgear, motor control centers, transformers, power panels and motors. Preliminary information such as voltage levels, bus capacities, breaker ratings, and other information, which will aid in describing the lock distribution system will be shown. Conceptual design will satisfy the minimum safety requirements of the National Electrical Code (NFPA 70). In addition, plans showing general locations of major components will be developed.

7.4. Lighting Systems

Lock chamber lighting will be provided by fixtures mounted on high mast poles. Various lighting alternatives will be evaluated to determine the optimum arrangement, spacing, and most suitable lighting fixtures. Design considerations will include uniform and adequate illumination levels, (22 lux at low pool), minimizing negative effects of glare and shadows, cost, and ease of maintenance. Conceptual drawings will indicate recommended fixture locations, types, and other details required to satisfy the canals lighting objectives.

7.5. Emergency Power

Provisions for emergency power will be incorporated into the power distribution system. Standby power will be provided for critical loads to maintain lockages during power outages.

8. CIVIL/SITE AND RELOCATIONS

8.1. References

Civil/Site and project relocations are designed in accordance with the criteria and guidance furnished in portions of the Corps of Engineers manuals for engineering and design, industry standards and other technical references as follows:

Engineering Manuals

- EM 1110-2-2602 — Planning and Design of Navigation Locks, 30 September 1995.
- EM 1110-1-2909 — Accuracy Standards for Engineering, Construction, and Facility Management Surveying and Mapping, Chapter 11, Change 2, 1 July 1998

Engineering Regulations

- ER 1110-2-1150 — Engineering and Design for Civil Works Projects, 31 August 1999
- ER 1110-2-1200 — Plans and Specifications for Civil Works Projects, 30 October 1993

Technical Publications

- EM 385-1-1 — Safety and Health Requirements Manual, US Army Corps of Engineers, 3 September 1986
- AASHTO — American Association of State Highway and Transportation Officials, Guide Specification for Highways

Miscellaneous References

- Harza Alignment Report, "Evaluation of Lock Channel Alignments", August 2000
- Topographic Mapping, ACP furnished data file, containing north, east, elevation data for the site (5 m and 1 m contour accuracy)

8.2. Lock Alignment

The civil/site development effort requires the creation of digital terrain models (DTM's) to define material layers. Once DTM's have been generated (existing ground topography, top of firm rock layer), corridor modeling can begin. The Scope of Work defines "Task 3" as "Optimization of the Lock Alignment". Various positions of horizontal lock alignment have been investigated, all of which define an alignment at a specific distance and orientation (azimuth), east of the existing Gatun locks. The optimization task is an iterative process which seeks to establish the best "fit" of lock alignment, given the position of the Gatun east wall and need to maintain lock operations at all times throughout the construction phases, navigational safety, WSB geometry, rock DTM, excavation requirements, construction logistics, efficiency, economics and future plans for Fourth Lane expansion.

Harza Engineering evaluated alignments and presented results in the "Evaluation of Lock Channel Alignments", August 2000. The Harza report concluded that two alignments on the Atlantic side merit further consideration and refinement, A1 and A2. A1 is defined thru the 1939 excavation and excluded from this Scope of Work. A2 is defined immediately to the east, and parallel to, the Gatun east wall. Being parallel to the existing locks, the A-2 alignment forces traffic to make a "turn" (course correction) at some PI point, north of the locks. Given the necessity of a turn, A-2 was initially positioned to intersect with a Point of Intersection (PI), located approximately (1 300 m north of the northern end of the middle wall), then rotated 9.75 degrees, to Azimuth 177-53'-10". This rotational shift in A2 (A2 prime) was found to provide a safe navigational alignment to facilitate lock operations during construction. The alignment will be optimized, to provide the best fit, given the site geometry (location of hillside to the east), geotechnical data (location of rock strata, type and quality of overburden for cut slope stability), location and dimensions of WSB's, and excavation requirements. Disposal area locations are a consideration (equipment needed, whether single or double handling will be necessary, method of transportation). The disposal area is assumed to be located approximately 2.4 km to the north and east of the center of the construction area. Quantities and character of material (soil, rock) must be determined before the superior economic plan for removal can be finalized. Large, land-based equipment is anticipated for excavation requirements.

Harza estimated that excavation costs may comprise half or more of the total project costs (Alignment A2 had an estimate of 12 100 000 m^3 of cut, at a cost of $180,000,000). A variety of excavation techniques could be utilized to construct the new alignment.

8.3. Site Access

Temporary site access (during construction) will be addressed. Permanent site access across the Third Lane (bridge, tunnel, etc.) will be by others. Construction (off road) equipment access to the lock construction area, including batch plant location(s) and means of transporting materials to the site will be addressed. Construction of the Water Saving Basins (WSB) will be positioned between the existing and proposed lock lanes, which will create concrete walls (vertical offsets) between the plateau elevations. Personnel and equipment access to the lockwalls and WSB's will be provided. Existing roadways serving the town of Gatun will be relocated, as required. Circulation roadways in the Gatun area will be removed (as will accompanying buildings) which fall within the excavation limits for Third Lane construction along the 9.75 degree alignment. Site access is anticipated to remain unchanged (from the east side), to approach the lock site. Numerous building, roadway and utility relocations will be required and addressed. Real Estate acquisition is not anticipated to be a problem, since the lands falling within the Third Lane corridor are ACP owned. Buildings requiring relocation will be identified by building number. WSB geometry, proposed lock wall and culvert valve geometry, filling/emptying culverts, WSB conduits, etc. will all play a part in laying out site access.

The following assumptions will be made for the Concept Level Design:

i. Existing buildings located along the Gatun esplanade will remain in tact, and are not to be affected by Third Lane construction. The lock will remain fully operational during all phases of construction.

ii. Access to the existing Gatun locks will be addressed, but will have to be tailored to conform to the proposed site, given location and geometry of the WSB's. This access will likely remain in the approximate location as existing (from the northeast, across the 1939 excavation "plug").

iii. The Maintenance Facility buildings (and electrical building) may require relocation, depending on the manner in which the overburden cut or fill slopes tie back into existing ground. Affected buildings would need to be removed prior to initiating the site clearing contract.

iv. Access to the west (Dam) side of Gatun lock is assumed to remain unchanged. No changes to the access road or circulation roads will be made. Access across the existing lock gates will remain unchanged. Access across the new lock will be studied by others.

v. Existing circulation roads located adjacent to the existing locks (east side) will be utilized to the extent possible. The WSB's will require perimeter access, both during construction as well as post-construction. Once the final WSB positions are plotted, with cut/fill slopes and tie ins, geometric clearances will be determined. The northern-most WSB could pose a horizontal clearance problem, if the shape remains rectangular (encroachment on the Gatun east wall). If this is found to be the case, some means of stabilizing the east wall (driven sheet piles) may be necessary, to allow sufficient space for WSB access.

8.4. Esplanade

Fourth Lane planning and project development is anticipated to follow Third Lane construction. Hence, construction of any site features along the east side of the Third Lane will be "temporary" (removed at the time of Fourth Lane construction). A concrete esplanade area would normally be provided adjacent to the east wall, to "house" maintenance and operations buildings, control shelters, etc. These facilities will be located between the existing Gatun locks and the Third Lane, to avoid service disruption or need to demolish at some future date.

8.5. Utility and Building Relocations

The Third Lane Alignment will be positioned approximately midway between the existing Gatun locks and the 1939 excavation. As such, numerous buildings and utilities must be removed/relocated prior to executing the site clearing or excavation contracts. Known utilities serving the Gatun locks are power lines, water lines and telephone lines. Known buildings falling within the excavation limits number approximately 120. The maintenance buildings (No. 24 and 26) will not be disturbed, and will remain during and following construction.

8.6. Lock Safety and Signage

Lock safety criteria (hand railing, etc) will be met., along lockwalls, entrance walls and WSB perimeter. All required signage will be commensurate with existing ACP criteria. Navigation-based signs (large and readily visible with greatly reduced lighting levels, recently required by Corps of Engineers criteria for US navigation projects) will not be proposed.

8.7. Design Aids

INROADS Software: InRoads is a commercially available, MicroStation based, Bentley software package, used commonly for civil/site development projects. This software has the capability of modeling topography, horizontal and vertical alignments, typical cross section templates, corridors with superelevations or transitions. Once the 3-D surface (terrain) has been created, InRoads models or "pushes" a defined template along a defined path, thru the 3-D terrain, generating excavation volumes. Alignments must be defined. Volumes calculated could be rock volumes, overburden volumes or any volume in which boundaries dividing material layers has been defined by a 3-D digital terrain model. InRoads SelectCAD version 8.2 is the latest available module, and will be used for the Gatun topo modeling. InRoads will be utilized to import the ASCII data file (ACP supplied, containing topographic mapping data by Northing, Easting, Elevation in .txt format) and to generate the 3-D Digital Terrain Model (DTM). The number of templates required will depend on site geometry. Initially one template will be used to capture the horizontal requirements of the walls and WSB, with profile adjustments made for the three elevations of the chambers (3 lift lock) or two elevations of chambers (2 lift lock). Template refinements will be made following submission of the Draft Alignment Report, in June 2002, to adjust and correct for further development in design of walls, culverts, WSB's.

9. CONTROL BUILDINGS

The concept design of the operations building will include space for lock controls (new and existing locks) and administrative staff. The design will be based on input from the ACP and tailored to their requirements including provisions for the handicapped. Numerous attempts were made throughout the design process to seek input from the ACP on design requirements for the control building. However, no data was provided. No provisions for handicap access were incorporated into the design.

9.1. References

Control building will be designed in accordance with the criteria and guidance furnished in portions of the Corps of Engineers manuals for engineering and design, industry standards and other technical references as follows:

- U.S. Army Corps of Engineers, Architectural and Engineering Instructions (AEI) Design Criteria, July 1995.
- Life Safety Code (NFPA 101) 1995, for exiting and means of egress as referenced by MIL-HDBK 1008B.
- Uniform building Code (UBC) for fire resistance requirements and area limitations as referenced by MIL-HDBK 1008B.
- Military Handbook Fire Protection for Facilities, Engineering Design and Construction, MIL-HDBK 1008C, 15 Jan 1994.
- National Standard Plumbing Code, 1995

10. COST ESTIMATING AND CONSTRUCTION SCHEDULING

10.1. References

The following references will be used to assist in the preparation of the project cost estimate and the construction schedule. These references will be considered for guidance only. The project team will work with the ACP in developing the format for the cost estimate and construction schedule.

Engineering Manual

- EM 111-2-1304 — Civil Works Construction Cost Index System, 3 September 2002

Engineering Regulations

- ER 1110-1-1300 — Cost Engineering Policy and General Requirements, 26 March 1993
- ER 1110-2-1302 — Civil Works Cost Estimating, 31 March 1994
- ER 1-1-11 — Process, Schedules, and Network Analysis Systems, 15 June 1995

Engineering Instructions

- EI 01D010 — Construction Cost Estimating

Engineering Pamphlets

- EP 1110-1-8 — Equipment Handbook
- EP 415-1-4 — Network Analysis Systems Guide

Engineering Circular

- EC 1110-2-538 — Civil Works Project Cost Estimating – Code of Accounts

Miscellaneous References

- R.S. Means Heavy and Highway
- TRACES Unit Price Book
- List of Prices of Materials of Construction, Panama (CAPAC)
- Mobile District Guidance for Estimating Projects in Panama

10.2. Criteria

The concept design level total project cost estimates will include construction cost (broken down into the various items of work), anticipated design cost, and construction management cost. Relevant notes on construction methods, and other assumptions will be included to assist in review.

The construction costs estimate will be presented in unit price format and will be based upon historical information (from ACP, and COE), subcontractor quotes, unit price books, and when appropriate, the development of a detailed estimate for specific items. Cost information for work in Panama (labor and equipment rates, material cost, freight, taxes, fuel costs, etc.) will be used to develop factors to apply to historic unit prices from lock and dam construction projects in the United States. An appropriate level of contingency, as determined by the cost engineer with input from the designer, will be applied to each item on a case-by-case basis.

Design cost and Construction Management cost will be derived by developing percentages to apply to the construction cost. These percentages will be developed with support from the design team, the Pittsburgh District Construction Branch, as well as the ACP.

Several less detailed cost comparison estimates will be developed in support of the design team (Gate Study, Alignment Optimization, etc.). For these early cost comparisons, adjusted historic unit prices will be the main source for costs.

The construction schedule will be prepared with guidance from the design team as well as construction professionals within the COE. The schedule will focus on the main tasks associated with the construction of the lock.

PANAMA CANAL CONCEPT DESIGN

Atlantic Locks Structure

Third Lane Lock

Appendix B

Gate Selection Study

Prepared for

Canal Capacity Projects Office

By

US Army Corps of Engineers®

Final Report

23 July 2003

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1. GATE SELECTION REPORT SUMMARY

This report screens possible gate alternatives to select the most efficient lock gate type for use in the Concept Design of the Atlantic Ocean side Canal Locks. The gates were designed to be compatible with the proposed chamber configuration of 61 m wide, 426.8 m long (face to face, inner gates), double sets of gates at each lock location, and 18.3 m of water depth over the sills. The gate type selected herein was developed to a concept design level of detail and integrated into the Concept Design of the Atlantic Locks. The factors that were considered for this lock gate type selection are as follows; (1) maintainability, (2) total cost, including bulkheads if required and gate monoliths/walls, (3) compatibility with the filling and emptying systems, (4) compatibility with water saving basins, (5) lockage water consumption, (6) compatibility with the ship positioning system, (7) possible use for access, (8) reliability, and (9) safety. A complete discussion of these factors is contained in Chapter 7.

The screening process for the gate type selection began with the Initial Team Meeting that was held in Pittsburgh, PA, USA, on 17-19 April 2002. The Structural Design Team, lead by Mr. Andy Harkness (Pittsburgh Office), experienced production team members in gate design from various Corps Offices, and expert Independent Technical review (ITR) members spent the better part of two days in defining and making a preliminary evaluation of the possible gate types. The ACP Canal Capacity Projects Office Locks Team attended this meeting and participated in the preliminary discussions. The gate options initially considered were (1) miter gates, (2) multi-leaf vertical lift gates, (3) rolling gates and rolling dams, (4) sector gates, (5) submersible tainter type gates, and (6) drum gates. After 2 ½ days of technical analysis/evaluation and discussion of the selection criteria, the Structural Design Team decided to proceed with the conceptual level design of the two most probable gate types, rolling gates and miter gates, as the most efficient gate type for use in the Atlantic Locks. The initial subjective evaluation, without design calculations, did not identify a clear superiority between miter gates and rolling gates. The remainder of gate types all showed lesser benefits of use and using the established evaluation criteria, ranked much lower in the overall rating matrix.

This report section presents the results from the initial screening process and Initial Screening Report as presented to the ACP and the continuing evaluation and feasibility design level development of the two most probable gate types, rolling gates and miter gates. The continued equal design level was necessary to show the full relative merits of the two gate types and provide a clear decision for the recommendation of the gate type for use in the Atlantic Locks Concept Design. This short-list evaluation also includes a sector gate, since a ship lock using sector gates is in final design in one of the Corps Districts and data is available to analyze and validate the initial team meeting conclusions. The example used is that of the Inner Harbor Navigation Canal Lock in New Orleans, Louisiana, USA. Concept level design of the three gate types was prepared and evaluated in accordance with the selection criteria noted above. Drawings and calculations (calculations are provided in electronic format on accompanying CD-Rom disk) are included herein. The final evaluation of the gate designs and related information showed miter gates to be the recommended type of gate for the Atlantic side Canal Locks. The major advantages for this gate type are summarized as follows:

CAPITAL COSTS

- Mitering lock gate offer a $145,000,000 savings over rolling gates in initial construction cost. This cost saving considers construction of new Maintenance Facility Infrastructure.

- Miter gates have an advantage in the fabrication cost due to their lower weight. Total gate weight for gate sets at six locations are 19 000 t for miter gates, 27 000 t of structural steel plus 2 500 t of removable lead ballast for rolling gate installation and removal, and 24 000 t for sector gates. These total weights are for the double lift lock structure and include the upper, center, and lower gates with inclusion of the anchorage systems, contact blocks, sill beams, rolling gate carriages, and other required accessory or embedded metals. Similar weight comparisons can be expected for the triple lift lock.
- Miter gates require approximately 414 000 m^3 less rock excavation than rolling gates because they do not require large separate gate recess structures. Figure B-1-1 depicts gate monoliths and sill with the recess structure that is required to house a rolling gate. Figure B-1-2 shows the cross section of a miter gate bay. Both alternatives have similar excavation cross-sections over the length of the gate bay, however, miter gates do not require excavation for separate recess structures.
- Miter gates require approximately 550 000 m^3 less concrete construction than rolling gates. The large difference in concrete volume is attributed to the construction of large concrete recess structures for the rolling gate as shown in Figure B-1-1. This is a significant cost advantage of miter gates.

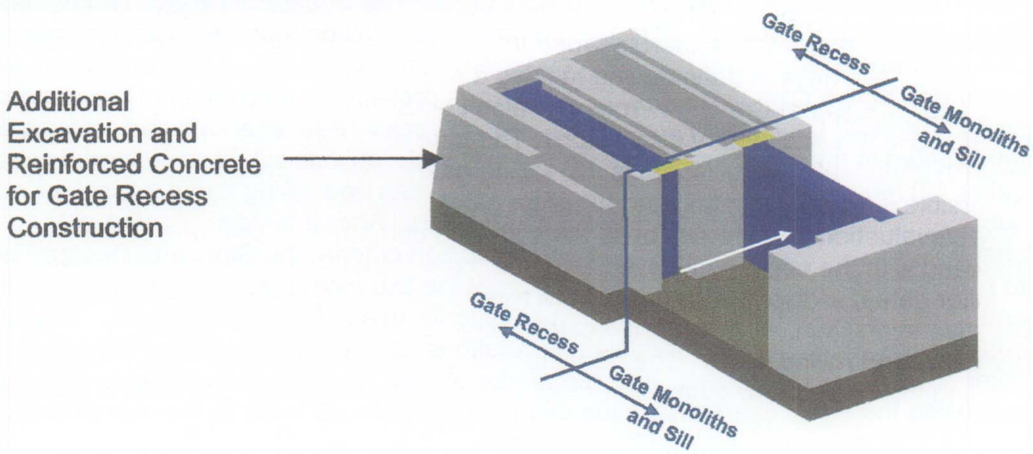


Figure B-1-1, Rolling Gate Bay Schematic

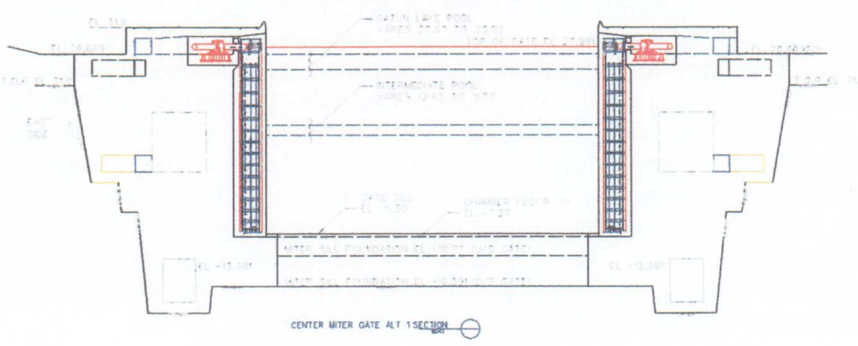


Figure B-1-2, Section Through a Miter Gate Bay

WATER CONSERVATION

- Lockage water consumption for miter gates is somewhat less than rolling gates with only a 0.6 % difference in water use. Sector gates use 12% more water than rolling gates.

SYSTEM COMPATIBILITY

- Miter gates are more compatible with the layout of distributed filling and emptying systems because no there are no interferences with culverts and gate recess structures. The lack of space requirements at the back of the lock wall as needed for rolling gates presents a more compatible and flexible arrangement when integrating water saving basins into the design. This allows the basins to be longer along the lock wall length while minimizing the width requirements from the back of the wall.

SAFETY

- Miter gates are very reliable in lock operations based on their long history of use on navigation projects. Although they have not been used in locks of this size, they are easily adaptable for the larger lock size when there is no need to accommodate reverse water loads. Miter gates damaged by ship impact would be easier to repair as the arch shape and design would minimize ship impact damage. Equal impact to rolling gates may result in significant deflections and warpage, which may not permit the gates to be retracted into the gate slot after ship impact. The weight of one rolling gate is similar to a one pair of miter gates, or one miter gate leaf weighs approximately one-half that of a rolling gate. The heavier weight of a rolling gate would not permit the rolling gates to be easily lifted if they lost their buoyant capacity, thus putting the lock out of service for an extended period of time. In comparing salvage operations in response to a catastrophic accident where loss of buoyancy occurred, two lighter miter gate leafs would be easier to clear from the lock chamber than one heavy rolling gate structure. In USACE's opinion, repair of moderate damage to a rolling gate within the confines of a slot will be difficult considering access and scaffolding needs. Work on a miter gate lying horizontally and close to the ground allows better access to repairs and facilitates equipment and material delivery. Floating plant equipment is needed to service either a miter or rolling gate. For rolling gates, floating plant is needed to set a caisson in the recess slot and for miter gates, floating plant is needed to lift the gate off the pintle and set it afloat.

MAINTENANCE INVESTMENTS

- Rolling gate mechanical features (underwater rollers and drive systems) requires more frequent adjustments and maintenance than hydraulic actuated miter gate machinery.

A comparison of various lock gate types was made in Stahlbau, Volume 12, December 2000, "Miter Gates in Some Recent Projects in the Netherlands" by Rysgard A. Daniel. In this individual analysis of existing lock gate systems in the Netherlands, where rolling gates are used, it is apparent that the use of miter gates is more advantageous in many areas. The following table is taken from that report.

Table 1. Global evaluation results, example
Tabelle 1. Ergebnisse der Globalbewertung, Beispiel

gate type / criterion	Mitre gate	Vertical gate	Rolling gate
Construction costs	good	fair	poor
Navigation conditions	good	fair	good
Operation & maintenance	good	good	fair
Lock layout conditions	good	good	poor
Esthetics	good	poor	good
Environmental analysis	good	fair	fair
global	good	fair	fair

Figure B-1-3 Independent Gate Selection Study

It is very interesting to note the similarities in the evaluation criterion from the cited report and that which we are independently using for this Study. Although the Study was conducted for gates, which are not of the size that would be used in the Panama Canal, the results have been verified by our detailed analysis as extendable to larger gates. When comparing the results shown in the table above to our detailed evaluation results summarized in the below and further explained at the end of this Appendix, the findings are essentially identical.

Evaluation Criteria and Rating								
Feature Alternative	**First Cost**	**Ease and Frequency of Maintenance**	**Risk to Extented Navigation Closure Resulting from Significant Accidents**	**Access for Crossing Over Lock**	**History of Reliable Service or Precedent**	**Compatibility with Water Saving Basin and Filling and Emptying System**	**Water Conservation**	**WEIGHTED EVALUATION**
Miter Gates	3	3	3	2	3	3	3	2.86
Rolling Gate	1	2	2	3	3	3	3	2.43
Sector Gates	2	1	2	1	1	2	1	1.43
Procedure included ranking the individual alternative by individual criteria to consider relative comparison. 3 is the highest, 1 is the lowest								

Figure B-1-4 Final Ranking of Gate Alternatives.

In addition to the findings in the above cited paper, the gate study performed by the Panama Canal Company in the 1940's for the design of a 140-foot wide Third Locks showed that they also reached the conclusion to use mitering type lock gates as the most favorable gate type. This study also considered the additional cost of gate recess structures in the evaluation process. As the results of this study proceeded, the advantages of using mitering lock gates continually surfaced.

Therefore, based on the results of the above discussion, detailed gate study with appropriate analyses performed for each gate type including sizing of operating equipment and development of required masonry structures, and evaluation in accordance with the developed criteria, we are recommending miter gates as the most favorable gate type for the Atlantic Locks.

2. INTRODUCTION

This report contains the initial gate selection studies and following detailed design studies that lead to the selection of the mitering type gate with conclusions and recommendations on the various gates studied, gate arrangements, and lock gate monolith configurations for the Third Lane on the Atlantic Ocean side of the Canal. The study and decision on this feature was considered to be one of the most important in the design of this project as it impacted the selection of several features such as monolith type, layout of the filling and emptying system, and maintenance requirements. All of the gate types considered have been developed to a stage sufficient to indicate their individual feasibility, advantages and disadvantages, and relative costs. The three most favorable gate types, mitering gates, rolling gates and sector gates, have all been developed to an almost equal level of design to support the determination of the relative merits and reliability. The importance of reliability and longevity under all conditions has been emphasized in all evaluations along with evaluation to the full criterion.

Rolling gates were used on many low lift (3 m) 33.5 m wide locks on the Ohio River by The Corps of Engineers until they were replaced with miter gates in higher lift configurations circa 1930's. Miter gates are generally accepted as having preferred characteristic and economic benefits for higher lift locks. Rolling gates generally only have application in tidal areas where resistance to minor reverse load is a consideration. Berendrecht Lock, Belgian Ministry of Public Works, has the largest rolling gates in service to date. This 68 m wide ship lock is located between the Scheldt River and the B2-B3 Canal Dock in Belgium. The gates are 22.6m tall. The Tidal River Scheldt pool elevation varies from +7.5 m to –1.00 m and the water level in the docks is constant at 4.25m. The minimum head applied to these gates is 3.25 m and the maximum head is 5.25 m. The 13 m lift for the Third Lane 2-Lift Lane Locks is about 2.5 times higher (with load proportional to head, the load on the Third Lane gates would be about 6 time higher) than Berendrecht. The size of the lock gates required for the Third Lane Locks exceeds either the lift height or width of any lock gate currently in service today.

2.1. Relevant Projects

A review of the gate types on the existing Panama Canal locks and other comparable locks, with the reasons for their adoption is considered relevant to this study. Two of the four projects listed below, Inner Harbor Navigation Canal Lock (33.5 wide), New Orleans, Louisiana, USA and Bremerhaven Ship Lock, Bremerhaven (33.5 wide), Germany are considered to reflect state of the art practice for their intended application. A trip was made in May 2002 to the Bremerhaven Lock to become more knowledgeable of its design and operational considerations. In addition, consideration was given to the operation of the large Belgium locks in formulating final conclusions.

Available information is presented below.

2.1.1. Panama Canal Locks, Panama

The existing locks are 33.5 m in width. In addition to the features required for normal lockages, certain additional and redundant features were incorporated to provide for the safety and dependability of operation and conservation of water. These additional features included safety gates, guard gates, intermediate chamber gates, emergency dams, guard chains for safety and floating caissons for servicing the gates.

2.1.1.1. Safety Gates

The outer gate at the lower end of the two upper lock chambers is referred to as a safety gate. Safety gates are placed in locations that separate upper and lower pools. The philosophy for having these gates is based on preventing disastrous results should a gate be rammed and fail into the chamber allowing the water to pass freely through the open lock chamber.

2.1.1.2. Guard Gates

The first gate at each end of each lock is referred to as a guard gate. The guard gate at the lower end of the locks is a reversed gate that can act as a barrier to vessels moving into the lower lock chamber. Because the lower end gate is installed in a reverse orientation, it can also be used to dewater the lock chamber for maintenance work.

2.1.1.3. Intermediate Gates (short chamber gates)

These gates are located at an intermediate position along the lock length between the normal service gates and permit lockages of up to 198 m without using the full 305 m lock chamber length. The intent of this feature was to conserve water usage during dry seasons by using shorter chamber lengths but presents problems in the lockage operation and is not efficient.

2.1.1.4. Emergency Dams

Each lock was provided with an emergency dam to close off uncontrolled flow of water through the lock chamber if failure of a service gate occurred. However, there is no record of service gate failures. The emergency dams have been removed or made inoperative and are no longer a project feature in the Panama Canal system. In light of the potential for terrorist act these structures may become a valuable feature upon further security evaluation.

2.1.1.5. Chain Fenders (Guard Chains)

A fender chain was installed upstream of each gate or pair of gates whose failure might result in unobstructed flow of water through the locks and at the entrance to the lower end of each lock. The chains were installed as a second precaution to prevent damage to vital areas of the lock system. As the size of ships increased, these chains become less reliable and have not been used in lockage operations for some time.

2.1.1.6. Caissons

Floating caissons are placed at the upper and lower ends of each lock. The placement of caissons permits the lock chamber or lane to be dewatered for maintenance.

2.1.1.7. Evaluation of Present System

The successful operation of the present locks has proven the adequacy of the above features in combination with the ship positioning system to safely and reliably transit the ships through the locks. Based on the history of use of these features, the following evaluation is made for use in the new lock structure:

i. The chain fenders (Guard Chains) are no longer used. The use of this system or a device to perform a similar function is not anticipated to be pertinent to the Third Lane Project considering the size and design shapes of these much larger ships.

ii. Although emergency closure systems seem relevant for the Canal system, historically they have not been important to the project operation. However, their use will be investigated and a recommendation provided for or against their inclusion into this design.

2.1.2. St. Mary's Fall Canal, Sault Ste. Marie, Michigan, USA

The Sault Ste. Marie Locks, located in Michigan, USA, are shown in Figure B-2-1. The largest of four locks at the Sault Ste. Marie Locks Project is the Poe Lock. The Poe Lock was constructed in 1968 and is 366 m long, 33.5 m wide and 9.7 m deep. Panamax vessels used the Poe Lock on a daily basis. The Poe Lock uses double sets of horizontally framed miter gates as service gates. An additional set of gates is provided on the lower end in reverse orientation and is used to dewater the lock chamber. A guard cable is used to protect the lock gates as shown in Figure B-2-2.

Figure B-2-1 Soo Locks, (Sault Ste. Marie Locks)

Figure B-2-2 Articulated Fender Boom in Front of Miter Gates

2.1.3. IHNC Canal Lock, New Orleans, Louisiana, USA

The existing Inner Harbor Navigation Canal Lock is a vital link in the nation's inland waterway navigation system. It connects the Mississippi River, the Gulf Intracoastal Waterway (GIWW), the Mississippi River-Gulf Outlet (MRGO), the Industrial Canal (also known as the Inner Harbor Navigation Canal), and Lake Pontchartrain. The Port of New Orleans completed the existing lock in 1921. Growth in waterway traffic over the years has made the Industrial Canal Lock one of the United States most congested locks with an average wait of 10 hours, but often as much as 24-36 hours. The basic problem is that the current lock is simply too small to accommodate the volume of existing and future river traffic. The lock is 22.86 m wide by 195 m long and 9.6 m deep.

Following traffic and feasibility studies, design of a replacement lock is nearly complete. A new lock is being designed that will be 33.5 wide with a usable chamber 366 m long, and a have a water depth over-the-sill of approximately 11 m. The new lock will provide continued deep-draft access to the Industrial Canal and an almost three-fold increase in lock chamber capacity.

Selection of lock gates was a critical issue for this project. Miter gates were originally selected and designed. One extra set of miter gates, installed in a reverse orientation, was provided for reverse head conditions. Because they are more durable than miter gates and a single set of gates can take both a direct head and a reverse head, the use of sector gates is being be investigated. Final selection of a gate type for the Inner Harbor Navigation Canal is pending a detailed review of cost and functional requirements.

2.1.4. Bremerhaven Ship Lock, Bremerhaven, Germany

The Bremerhaven ship lock is located on the shores of the North Sea in Bremerhaven Germany. The 33 m wide lock is fitted with rolling gates and has a lift height of approximately 4 m. The lock filling and emptying system is self-contained within the rolling gate structure, complete with four sets of service/emergency valves and hydraulic operating equipment. A typical gate mounted slide gate is shown in Figures B-2-8 and B-2-9. No other supplemental filling and emptying systems are provided. The gate is moved by two large rolling type chains (Figure B-2-10), each powered by a hydraulic motor/gear reducer/sprocket assembly (Figure B-2-11). A single hydraulic power unit serves both chain drives (Figure B-2-12)

Figure B-2-3 Bremerhaven Ship Lock, Germany

Figure B-2-4 View Looking Across Upstream Gates.
Note Moveable Curved Roadway Section (See Figure B-2-5)

Figure B-2-5 Rolling Gate In Closed Position With Roadway Raised

Figure B-2-6 Spare Rolling Gate in Mooring Location

The gates are designed to float in and out of the gate recess structure for replacement or major repair. The project is provided with a spare gate for use in a five-year maintenance/service cycle. The gate recesses can be dewatered with installation of caissons for routine inspection and minor maintenance such as bearing lubrication. Major maintenance on the spare gate is performed while the gate is stored at the docking location (Figure B-2-13). Shore power is provided at the docking location for testing and exercising the operating systems (Figure B-2-14). After a gate is removed, the rolling assemblies are removed in the wet after the gate is removed, and are reconditioned and stored on the same maintenance cycle as the lock gates. The two rolling truck assemblies for the stored spare gate are shown in Figures B-2-7 and B-2-15.

Figure B-2-7 Spare Gate Rolling Truck Assembly

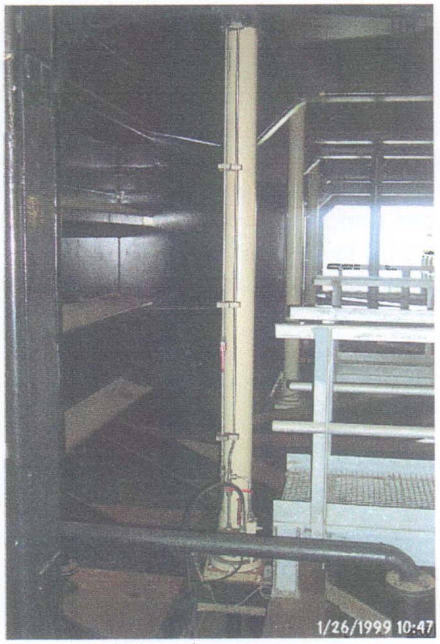


Figure B-2-8 Gate Mounted Filling/Emptying Valve Cylinders

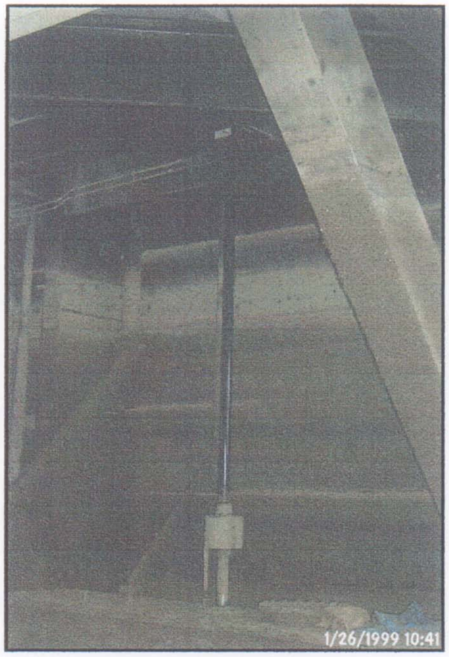


Figure B-2-9 Typical Bonnet and Cylinder Rod for Filling And Emptying Valves

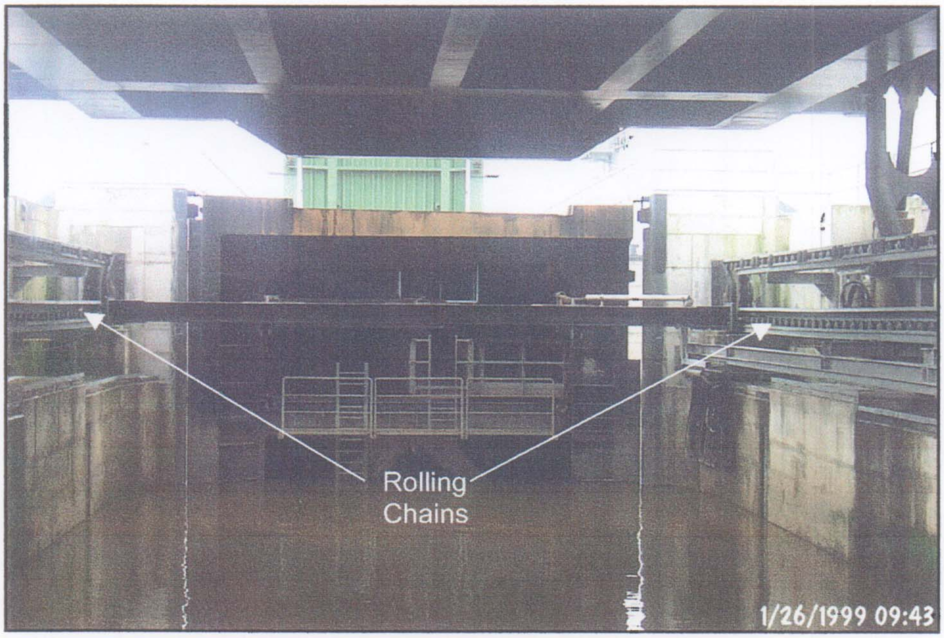


Figure B-2-10 Rolling Chains, Guides and Gate Linkage

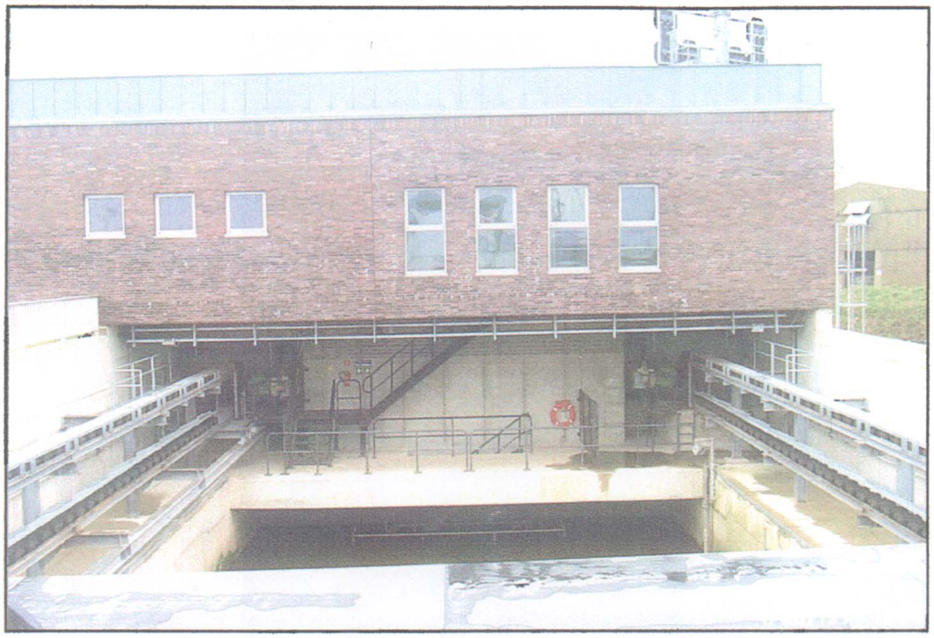


Figure B-2-11 Reducer/Sprocket Assemblies For Hydraulic Chain Drive System

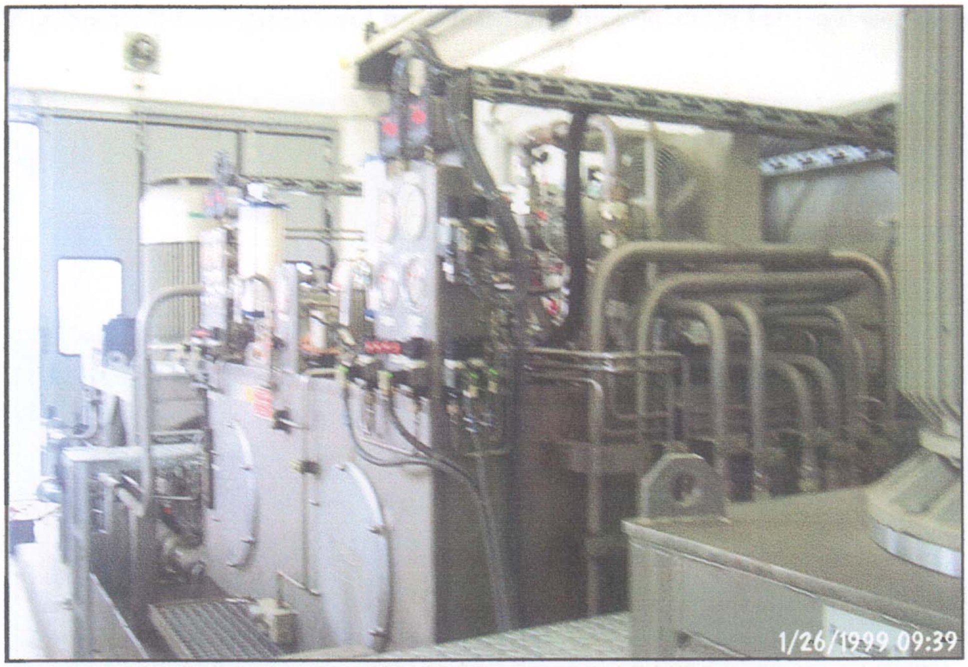


Figure B-2-12 Hydraulic Power Unit for Chain Drive System

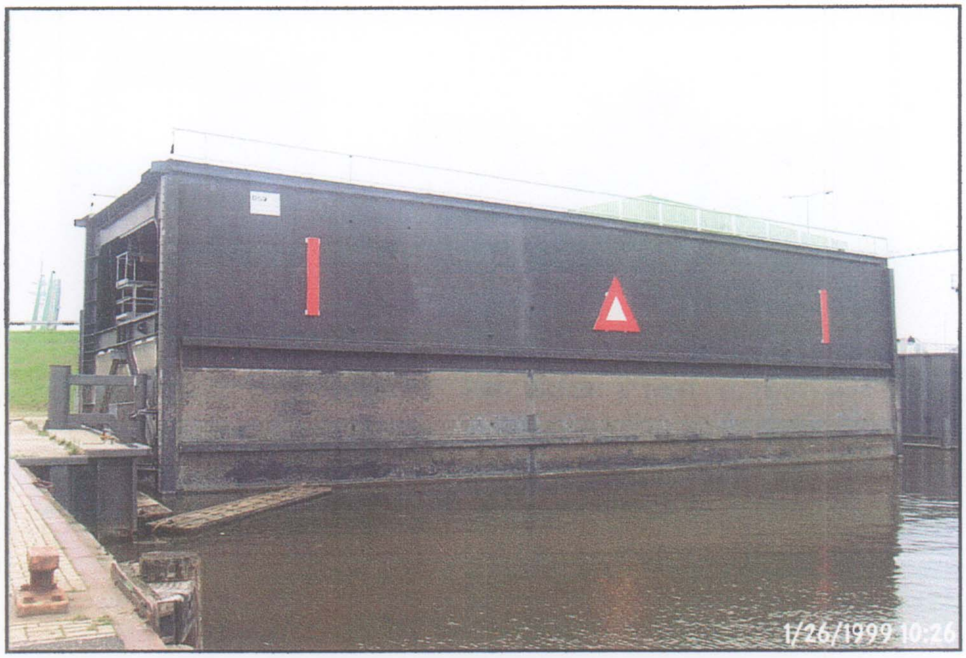


Figure B-2-13 Spare Gate at Maintenance Docking Location

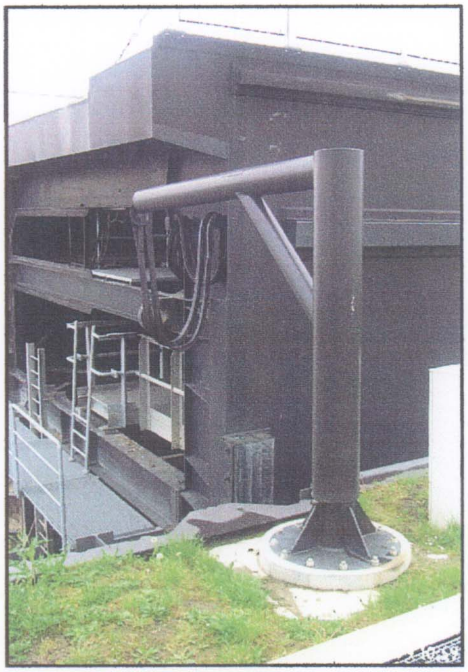


Figure B-2-14 Shore Power At Maintenance Docking Location

Figure B-2-15 Gate Rolling Truck Assemblies

3. ALTERNATIVES CONSIDERED

At the Initial Project Team Meeting held in Pittsburgh on 17-19 April 2002, a list of possible gate types was developed and a preliminary screening was made for application of the gates in the Atlantic Lock Structure, Concept Level Design Project. Based on the results of this screening and progressive screening studies that were conducted, the three most probable gate types were selected and designs were developed for the lock gates, gate operating equipment, and their associated masonry features to a concept design level for the purpose of evaluating compatibility in the lock system, fabrication, maintenance, operation, reliability, and cost. The gate designs were based on preliminary lock layout information for the center gate bay of a double lift lock system. Minor discrepancy exists in some dimensions and elevations used in development of alternatives due to refinements occurring as the study progressed. These variations are noted in the report; however, the findings of the Gate Selection Study are not sensitive to these minor refinements in geometry and operating pool elevations. Costs for the upstream and downstream gate arrangements were based on comparative ratios developed for the corresponding gate heights.

4/18/2002	Evaluation Criteria and weights									
Feature Alternative	Impacts to Navigation for Maintenance and Repair	Initial Fabrication Cost including bulkheads if required and MasonryHigh rating for low cost)	Compatibility with Thru The Sill F&E	Compatibility with Wall F&E	Access for crossing over lock	Proven History/Precedent	Compatibility with WSB	Water Conservation	Compatibility with locomotive	Weighted Evaluation
	0.2	0.2	0.075	0.075	0.05	0.1	0.1	0.1	0.1	
Miter Gates	5	9	10	10	7	6	10	2	9	7.4
Multi-leaf Lift Gates	1	1	10	10	1	3	10	5	9	4.7
Rolling Gate and Rolling Dam	8	5	10	1	9	9	5	9	6	6.8
Sector	3	4	10	3	3	3	6	1	1	3.6
Submersible Tainter	1	4	1	10	1	5	10	2	9	4.5
Drum	1	4	1	10	1	8	10	3	9	4.9

Procedure included ranking the individual alternatives by columns to consider relative comparisons.

Figure B-3-1 Initial Gate Short Listing

The final evaluation of miter gates, rolling gates, and sectors gates is discussed in Section 7 of the this Appendix Report.

3.1. General Criteria

All gate alternatives were designed for the upper and corresponding lower bound of pool elevations associated with normal operation of a double-lift lock. Where appropriate, a one-meter hydraulic surge effect was added to the upper pool elevation. Any effects from intrusion of saltwater were not considered in this study. Based upon ACP-furnished seismic studies and an evaluation of seismic activity probable for the region, a seismic ground motion of 0.313g maximum credible earthquake (MCE) has been established for the Atlantic Structures. The gates are designed with a 1/3 reduction in the MCE or 0.21g; see Appendix A for criteria on earthquake design. The two-thirds reduction is appropriate considering that there is a minimal thickness of soil backfill behind the walls has no significant influence on the lock walls for ground motions in the longitudinal axis of the lock.

3.1.1. Loading Conditions

3.1.1.1. Loading I

Gatun Lake	25.91 m
Intermediate Pool	15.05 m
Seismic coefficient	0.21g (2/3 of the MCE)

3.1.1.2. Loading II

Intermediate Pool	15.05 m
Atlantic Ocean Low Tide	-0.38 m
Seismic coefficient	0.21g (2/3 of the MCE)

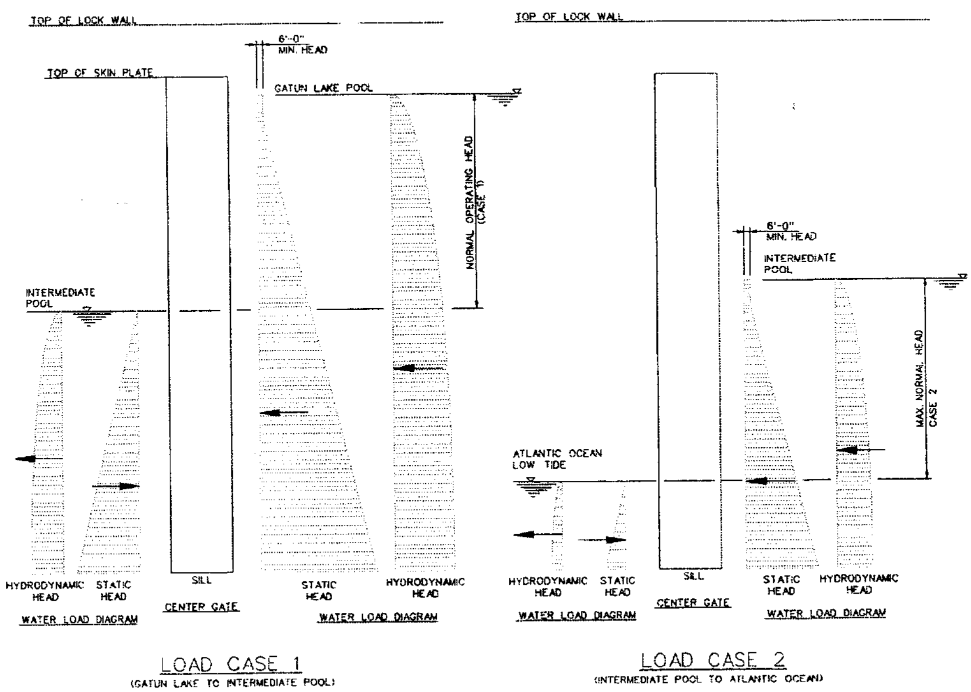


Figure B-3-2 Loading Diagram for the Center Gates

3.1.2. Geometric Layout Considerations

For preliminary assessments, all gates were assumed to be 36.4 m tall and the design was based on the center gates with the sill at elevation (-7.77 m). The top of gate was set at elevation 28.63. The top of wall elevation in the areas of the gates varies with the gate alternative used based on the required structural depth for bridges provided over the gates for ship positioning equipment and clearance and access for operating equipment. The wall geometry and gate spacing vary based on operating mechanisms, gate anchorage, gate recesses, and minimum wall thickness adjacent to filling and emptying culverts. The water usage was found to vary with gate alternatives based on their associated geometric layout. Water consumption is discussed in more detail in Section 7, Alternative Evaluation.

4. MITER GATE ALTERNATIVE

The original miter gates constructed for the Panama Canal are still in operation after 87 years of service and have served the Canal well. Miter gates are also used on a large percentage of moderate- to high-lift locks in the United States and other countries. These gates are relatively simple in construction and operation and can be operated, however, they cannot be used to close-off flow in emergency situations or tolerate reverse heads. Provisions to accommodate closure in limited-flow conditions can be implemented if warranted, however, this was not included in the scope of this study. A cantilevered support system can be designed and constructed to carry ship positions systems over the gate recess.

The primary deficiency present in the existing lock gates is leakage into ballast chambers and difficulty in achieving tight tolerances when setting new contact blocks. Both of these are addressed in the concept design. Adjustable quoin and miter blocks are provided to allow rapid and accurate adjustment. The adjustment process can use the methods the ACP has developed to transfer as-built dimensions into the maintenance process. The adjustable contact block detail is developed around an extensive materials research project conducted by USACE. The adjustable contact blocks are in service at McAlpine Lock and Dam. Material selection is based on a history of use and results in avoidance of material freeze up and binding to ensure long-term service. Leakage is not viewed as a significant design issue. With proper attention to detail, selection and fabrication processes, leakage can be avoided.

There are 88 hanging and 2 spare gates in the existing Panama Canal System. These gates are currently all of original construction except for minor repairs. They were fabricated using riveted construction and employ double skin plate framing which provides strong torsional resistance to operating loads and allows them to be ballasted and transported great distances by floating as a barge. In the history of the Canal there has been several accidental impacts to lock gates, however, none these impacts has caused damage that distorted the relative alignment between the miter and quoin ends of the gate. Several conclusions can be made from this set of historic data as listed below:

- The double skin plate system provides flexibility in gate movements from the repair yard to their point of installation.
- The general structural system is well suited to the operating requirements.
- Fatigue and fracture issues are not problematic.
- Double skin plate gates do not require periodic adjustment of the relative quoin/miter alignment.

Two structural systems have been considered for miter gates, a single skin plate gate with diagonals and a double skin plate gate. These two alternatives are compared to evaluate total gate weight and identify maintenance and infrastructure requirements.

The single skin plate gate is common on inland navigation project and usually results in the most efficient use of materials (least structural steel). These gates are installed and removed with a crane and cannot be floated. The single skin plate gates have a past history of fatigue and fracture failures. Several studies have been conducted to address fatigue and fracture failures in single skin plate miter gates. The conclusion of these studies attributes shortened life to misalignment, improperly tensioned diagonals, residual stress effects from welding, and selection of connection joint details. Some of these issues are common to all gate types and some are unique to the single skin plate design.

Double skin plate gates are heavier in the dry but lighter when submerged in water due to buoyancy from use of air chambers. These gates are typically installed with assistance of a crane however they can be proportioned for unassisted righting for installation without the use of a crane. Gates constructed with double skin plates require less maintenance because periodic diagonal stressing operations is not required. The double skin plate design offers a uniform shear flow distribution in the structural system and avoids special reinforcement for concentrated loads resulting from diagonals used in single skin plate gates systems.

The primary load path from hydrostatic pressure is the same for both single and double skin plate designs. The thrust line for the tapered end miter gates designed in this study is shown in Figure B-4-1. Both gate designs are based on common loading conditions.

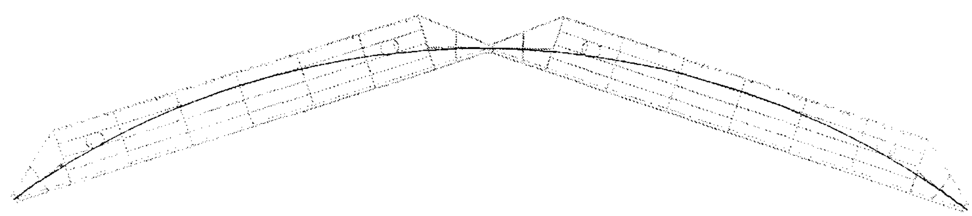


Figure B-4-1 Thrust Line of Tapered End Miter Gate

Based on the advantages of the double skin plate gate over a single skin plate gate, a double skin plate miter gate is recommended as the preferred miter gate configuration.

4.1. Gate Structural Systems and Design

Plates 1 to 4 show the general assembly of the center gate bay miter gates. These gates are design as three-hinge arches with a rise of 1/3 of the span (miter angle 1 to 3). The front face (upstream face when gate is mitered) of each gate leaf is composed of three vertical surfaces. The rear face (downstream face when the gate is mitered) is a vertical plane. At the center portion, the gate leaf would be 4 m wide and tapers at the quoin and miter ends. The front face is watertight and acts as a pressure carrying system. The skin plate of the rear face (double skin plate design) is vented except for sections covering buoyancy chambers and inspection compartments. The leaves are entirely of welded construction made up of plate sections. Bolted sections may be used to facilitate use of spare gate body parts. At points of high load concentration, and elsewhere for general bracing, plate elements are used to stiffen skin plates. Horizontal girders are spaced approximately at 2 200 mm. Seven vertical plate diaphragms run top to bottom of each gate leaf.

4.1.1. Horizontal Girder Spacing

Horizontal girders are spaced vertically for structural efficiency and are far enough apart to permit a person to easily stand up and move about for ease of inspection and maintenance of interior compartments of the gates. Intercostals in the form of plate elements are used to stiffen the skin plate spanning between girders. Two ladders are provided, one at each end of the gate to facilitate access. Provisions for delivery of fresh air and exhaust are needed but not detailed. Drain holes, sump pumps, and ballasting features are also required but not detailed.

4.1.2. Anchorage System

Buoyancy chambers are used to reduce loads on the top anchorage and pintle ball. The top anchorage and pintle are designed to support the entire weight of the gate during dewatering or if the buoyancy chamber is full of water.

4.1.3. Gudgeon Pin and Pintle Location

The gudgeon pin is located above the top girder. The connection area is stiffened such that the load can be shared between the top two girders. Likewise, the pintle is moved as high into the gate as possible, so that the bottom two girders can carry the imposed horizontal reactions.

4.1.4. Strut Arm Location

Structurally, the preferred location for the strut arm is approximately at the mid to lower third of the gate height to reduce torsion during operation. However, for serviceability, a connection point above upper pool is desirable. This allows the strut arm to be removed with ease.

4.1.5. Diagonals

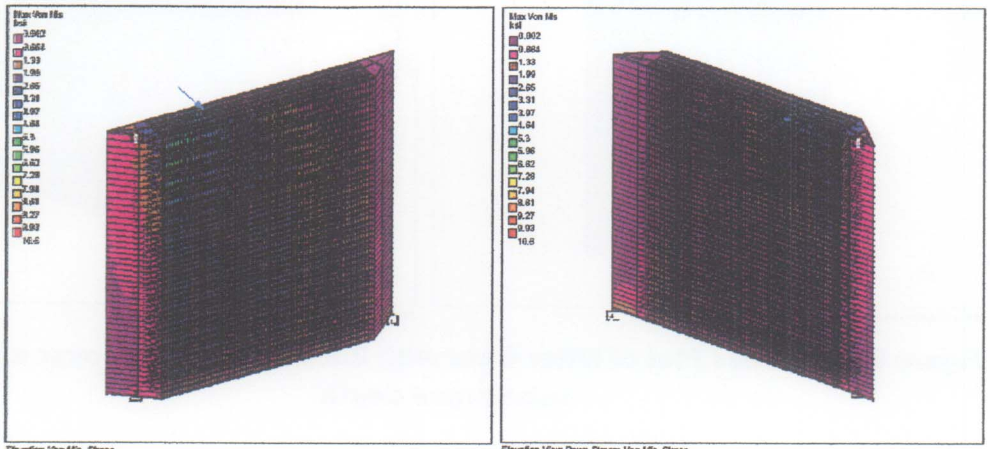
A miter gate undergoes much flexing during its movement due to the resistance of the water through which it must pass. This cause torsional flexing of the gate that must be resisted by a double skin plate system or diagonal bars on the downstream side of the gate. If the gate is provided with buoyancy and inspection chamber areas, these chambers would greatly reduces the need for diagonals.

4.1.6. Gate Anchorage

Typical gate anchorage designs use a three-pin linkage on one anchorage to allow loads to pass efficiently to the gudgeon pin. This arrangement is modified because of instability resulting from axial compression on the three-pin linkage during operation of the gate. A retractable linkage assembly has been developed to facilitate gate installation and removal while addressing load reversal. The linkage and support structure are design to support the gate hanging in a dewatered lock, operating loads, and earthquake loads plus gate self-weight with buoyancy. A post-tensioned anchorage system is used to support the embedded anchorage frames. This system was selected to minimize the distance from a downstream miter gate to the miter gate monolith joint.

4.1.7. Design Methodology

The miter gates were designed using the Corps of Engineer's computer program, "Design and Investigation of Horizontally Framed Miter Gates", using the Load and Resistance Factor Design Criteria (CMITERW-LRFD), Windows Version. This program is an excellent tool to properly proportion a structural system for miter gates and verify that code defined strength requirements are met. Some modifications were made to enhance the software design capability for the Panama miter gates. Updates were made in areas of weight calculations, buoyancy input and readout, and skin plate design. This program has been used and validated on many previous Corps miter gate designs and its use is sufficient at this level of design. A finite element model was used to evaluate the serviceability aspects of the miter gate during movement in water and with the bottom edge of the gate restrained and the full operating load of the hydraulic operating equipment applied to the gate. The stress and deflection plots for these two loading conditions are shown in Figures B-4-2 through B-4-5.



Elevation Von Mis. Stress

Elevation View Down Stream Von Mis. Stress

Figure B-4-2 Stress Plot With Machinery Stall Loads Applied to Miter Gates.

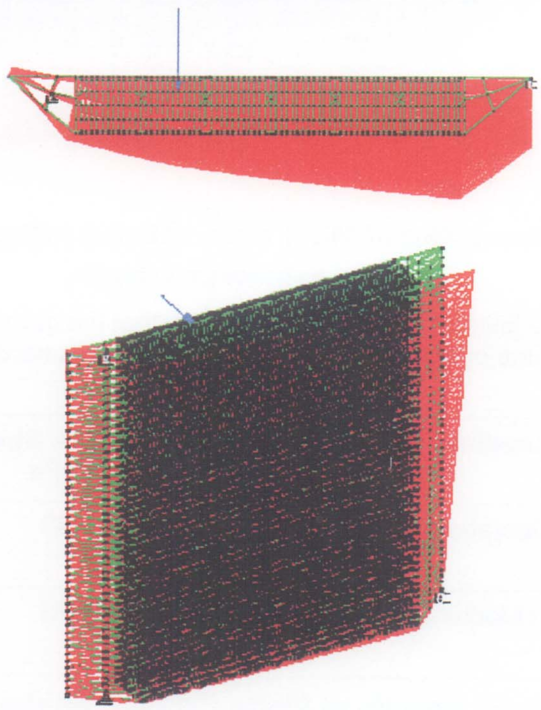


Figure B-4-3 Deflected Shape of Miter Gate with Machinery Stall Loads Applied.

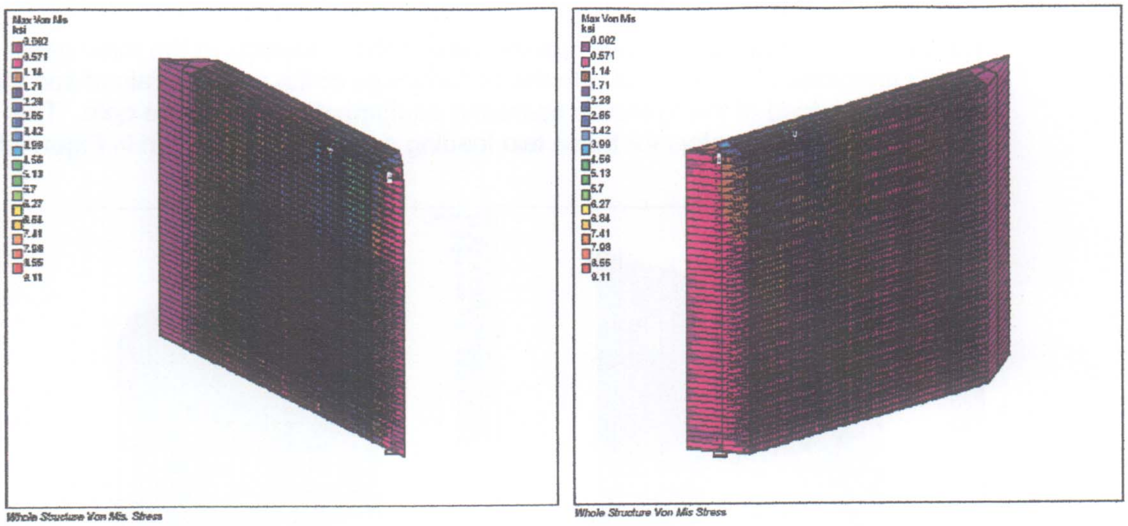


Figure B-4-4 Stress Plot of Miter Gate with 2.9 kN/m2 applied over a 18.3 m submerged depth

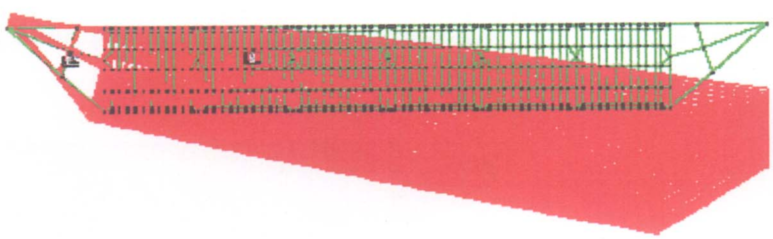


Figure B-4-5 Stress Plot of Miter Gate with 2.9 kN/m2 applied over a 18.3 m submerged depth

The results of the finite element modeling show that the gate performs well under both normal and extreme operating conditions. Table 1 summarizes the finite element analysis results.

Loading Condition	Von Mis. Stress, MPa	Maximum Deflection, mm
Normal Operation or Movement in Water	63	35
Maximum Machinery Load Applied to Gate	73	25

Table B-4-1 Results of Finite Element Analysis of Miter Gates

Additional finite element analysis and design would be required if the design is continued beyond this concept design level.

4.2. Gate Monolith Design

This work represents a preliminary assessment of loads and loading combinations, structural analyses, and concrete reinforcement design for the center gate bay. A contract modification was issued that eliminated a requirement to consider one wall serving as either a middle wall or portions of a cofferdam for a new fourth lane lock, a U-Frame Miter-Gate Monolith would be used for this configuration. For economic reasons, the Third Lane lock would be designed with a clear separation to a future four lane. Gravity masonry structures are more economical than U-Frame structures and are recommended as the preferred method of construction for the revised constraints. This is based on previous experiences on other Corps projects as providing the most sufficient and economical design. Sufficient design was performed for the three most probable gate types to provide a reasonable comparison of requirements.

4.2.1. Loads / Combinations

Loads including gravity, lateral backfill and water pressures, uplift, miter gate reactions, earthquake, and operational forces have been included in the various analyses performed as applicable and are defined below. Load case 2G, as described in Appendix A – Project Design Criteria was used to proportion the gate monoliths.

4.2.1.1. Data/Assumptions Used:

- Concrete Wt. = 2 400 kg/m^3
- Water = 1 000 kg/m^3
- Lateral Earth Pressure Coefficient (at rest) = 0.5
- Earthquake Loads
- MDE Pseudo-static EQ Coefficient = 0.313
- Hydro-dynamic effects using Westergard Formulations
- "Dynamic" earth pressures calculated using Mononobe-Okobe formulas
- Wt. of Ship Positioning System Units = 1 070 kN, applied as a distributed load of 326 N/m along monolith at line of rails.
- Lateral Line Pull from Ship Positioning System Units = 667 kN, applied as a distributed load of 239 N/m at elevation 28.8 when it is conservative to do so.

The full range of load cases and load combinations is listed in Appendix A-Project Design Criteria. Load case 2G – Normal Operation with a maximum design earthquake was used to proportion the miter gate monoliths for the following reasons:

- The rock elevation in the vicinity of gate monoliths is substantially higher than the founding elevation. Since the miter gate monoliths are cast against rock over a large percentage of their height there is adequate resistance to the lateral loads imposed by arching action of the miter gates.
- The seismic acceleration is high and previous work demonstrated that seismic loading controlled the design.

4.2.2. Cross Sections and Configurations

The selected cross section considered the location of the filling and emptying system, galleries, and equipment rooms. A reinforced toe at the founding elevation which extends into the miter gate sill was incorporated to optimize the gate monolith cross section. Concrete is placed against the rock cut lines to minimize the use of formwork and backfill material.

4.2.3. Geometric Considerations

When considering all aspects of the functional requirements of the intermediate miter gate monoliths, there are many dimensional provisions required regardless of analyses optimization. With a 5 m recess to accommodate the miter gate, and with a direct connected cylinder arrangement at the top of the wall and 6 m wide filling and emptying culverts wrapping around the gate recesses, it was determined that a monolith wall width of 20 m would be a minimum. Additionally, given the elevation differences in subsequent downstream monoliths, a transitional reach of the gate sill would need to be thickened to "plug" the step from the upper chamber to the lower chamber. (Note the downstream miter gate monolith and sill would have a lower founding elevation to serve as a "plug".) With these geometric considerations in mind, the dimensions of 3.5 m and 12 m for the gate sill (upstream and downstream center gates respectively) thickness and 20 m for the wall thickness were chosen as minimum requirements. The sill length was set at approximately 91 m to accommodate two sets of miter gates and their anchorage systems. The final geometry of the center miter gate bay is shown on Attached Plate 1.

4.2.4. Analysis & Design

Conventional stability analysis computations were performed to determine the required monolith proportions. Bearing failure on rock controlled the design.

4.3. Mechanical Systems

Plate 5 shows the general arrangement of the recommended miter gate machinery for a gate connection point on the miter gate above upper pool elevation. The machinery would be of the direct-connected, hydraulic cylinder type, similar to that being installed on the existing Gatun locks. The cylinder would be supported in the miter gate monolith machinery recess by a trunnion/cardon ring assembly (or gimbal) with the piston rod connected to the gate with a spherical bearing type clevis. The cylinder would have a serrated carbon steel piston rod with a CERAMAX type corrosion resistant ceramic overlay as part of a CIMS type positioning system. This system would provide the required precision and be compatible with the length of stroke necessary for this application. Each cylinder would be equipped with scrapers, wipers, zero leakage seals, and stop tubes as required. Greaseless bearings would be used for the cylinder trunnion, cardon ring, and rod clevis. All miter gate cylinders would be identical to minimize spare part requirements. Based on preliminary analysis, the cylinders would have a 762 mm bore, 559 mm rod, and 10 634 mm stroke.

A separate hydraulic power unit (HPU) would operate each cylinder. The power units and most control components would be located in the machinery tunnel adjacent to each gate. Interconnecting hydraulic piping to the cylinders would be stainless steel. Each power unit would include a stainless steel reservoir, two independent motor-pump groups, relief valves, check valves, flow control valves, filters, and gages. A sealed breather system utilizing a desiccant dryer and accumulator would be required to prevent water contamination from the operation in a humid environment. The hydraulic system would have a design pressure of 17.2 MPa with a peak allowable of 24.1 MPa. Using a 4-minute operating time it is anticipated that a 108 kW motor would be required to drive the hydraulic pump.

Access for inspection and maintenance of the bearings, hoses, seals and position transducer would be provided. Environmental protection measures would include: pressure switches to automatically shut off the working pumps (and oil flow) in case of hose or pipe rupture; zero leakage cylinder rod seals with protective scraper/wiper; self lubricating bearings to eliminate the need for grease and an oil containment system for each HPU.

4.4. Gate Installation

This section presents three methods of operation to install the 4.15-m wide miter gates. Three methods of gate installation are provided to demonstrate a range of possible solutions for consideration. Each alternative has advantages and disadvantages that should be considered in more detail as the Third Lane Lock Project evolves. The alternatives considered include purchasing a high capacity crane that would find many other uses on the canal system, a heavy lift barge, and using the existing Titan barge crane. The least cost technically feasible alternative is to use the Titan as it is existing infrastructure and maintenance staff is trained in this installation method.

The water levels to be considered are:

- Minimum water level at installation = 18.35 m. This water level is tied in with the position of the pintle at the bottom of the lock (where the bottom of the gate would engage).
- Maximum water level available = 30.00 m. This water level is available nearby to bring the gate to a position near vertical, i.e. using water ballast to fill the bottom chambers of the gate.
- Maximum water level during operation = 21.64 m. This water level is the highest level of water on both sides of the gate during operation of the gate at the locks. The level of water that the gate needs to control on one side of the gate may be higher.

4.4.1. Installation by Heavy Lift

Installation and removal by heavy lift would require a 2000 (+/-) t capacity crane as shown conceptually in Figure B-4-9, Gate Lifter Concept. A crane with a 2000MT capacity would find other beneficial uses in the Panama Canal. Of primary interest is emergency response to a severe gate damage incident where a gate leaf had to be removed from a lock chamber to allow continued service. Other possible uses include lifting tugboats out of water for service, possible uses during construction of the Third Lane Locks, consideration as a supplement or replacement for the syncrolift system. The cost to purchase a crane fitted as required to remove and install the miter gates would cost approximately $80 to 100 million dollars. Two cranes are shown in Figures B-4-7 and B-4-8, both are comparable in size (one smaller & one larger) to what is required to handle new Panama Canal lock gates.

The Yoshida Go No. 60 (1,700 MT), shown in Figure B-4-3, is the most powerful full-revolving crane vessel in Japan. This is a conventional fixed counterweight crane with hull dimensions of 115x45 and is equipped with thrusters. A fixed counterweight crane type would need sponsons to provide adequate hull beam to lift the required loads. The main hull would be limited to about 32 m beam to fit through the existing locks.

The Stanislav Yuden pictured below in Figure B-4-4 is somewhat larger than required for a single skin plate gate design. The Stanislav Yuden utilizes an active ballast transfer system in lieu of a fixed counterweight and has a maximum capacity of 2 500 t. The active ballast system is desirable from an operational perspective for the Panama Canal because the active ballast vessel could function as the crane for all of the lock gates, new and old, as it would be fully capable at a hull beam of around 32 m. A fixed counterweight crane type would need sponsons to provide adequate hull beam to lift the gates for the proposed Third Lane. The main hull would be limited to about 32 m beam to fit through the existing locks. The active ballast transfer system would allow the use of a hull that would fit through the existing locks, and not need any sponsons to make the lift. However the hull would be comparable in length to the Stanislav Yuden, or about 185 m.

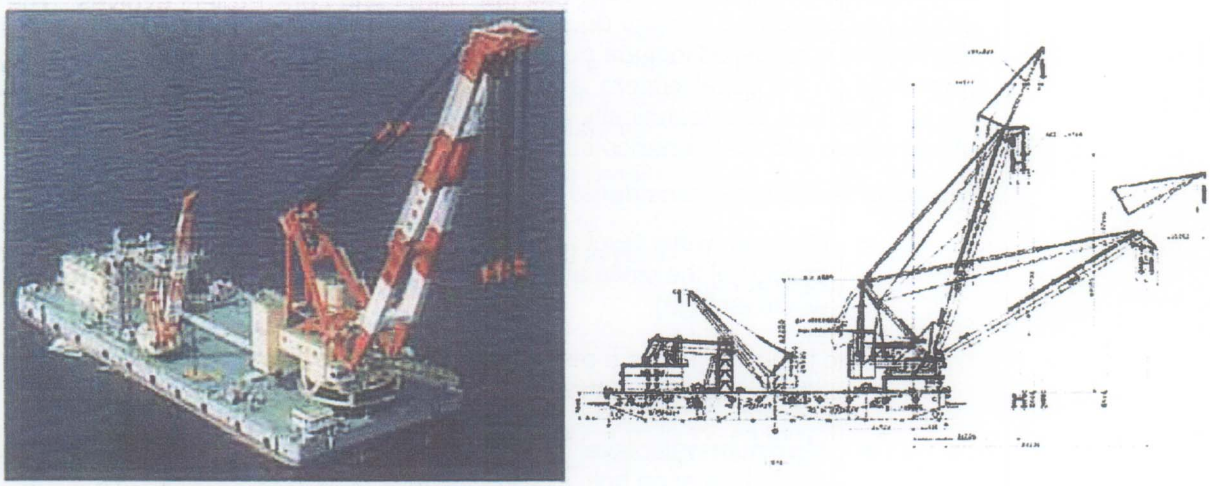


Figure B-4-7 The Yoshida Go No. 60 (1,700 MT)

Gates would be serviced while stored in a vertical position with this alternative. The feasibility of servicing gates in this orientation has only been evaluated to a concept level of design. The gates would be transported by floating plant to a storage area. The storage area would be equipped with a suitably proportioned foundation with walls built to simulate lock walls. The storage site would be equipped with pintles and gate anchorages to support the lock gates. A multi-level platform would be built into the site to safely access either side of the gate leafs. Infrastructure for painting, welding, machining, etc. would be provided to establish repairs. A general concept of this storage structure is shown in Report Drawing ACP-R-21/16 and 21/17. The ACP has developed expertise in transferring field alignment of exist embedded quoin block alignment into the realignment process of new contact blocks. The adjustable contact block detail proposed for mitering lock gates would greatly ease contact block setting and adjustment process. This detail is shown on drawing ACP-R-21/18 and ACP-R-21/19.

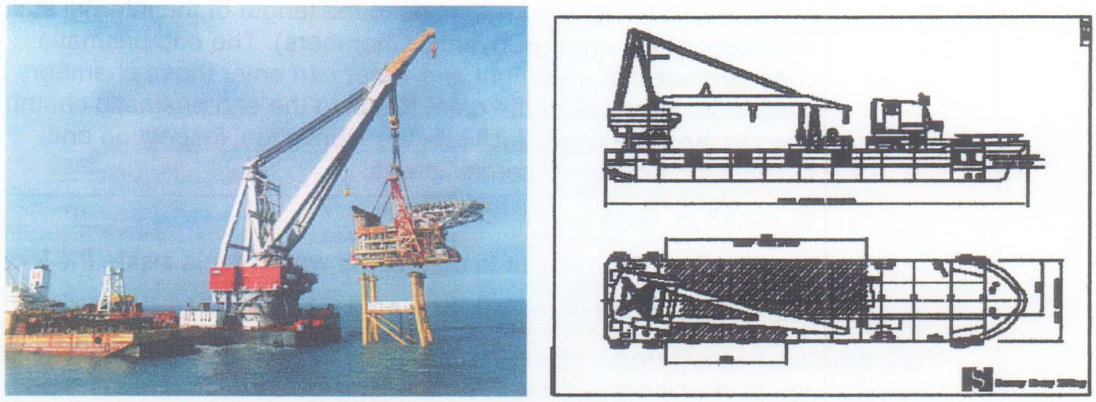


Figure B-4-8 The Stanislav Yuden (2,500 MT) Crane

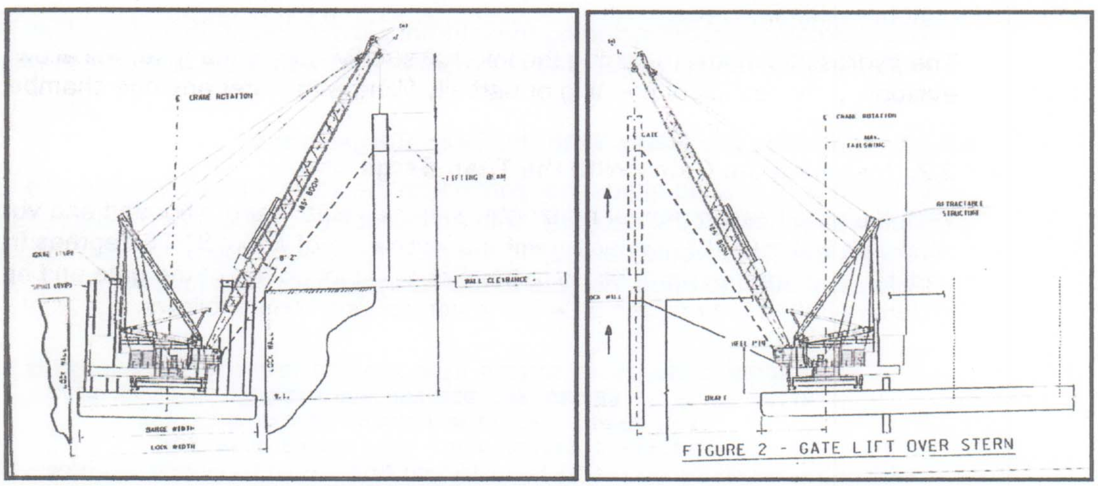


Figure B-4-9 Gate Lifter Concept

4.4.2. Use of Existing Methods to Install and Remove Miter Gates

The method for removal and installation of miter gates on the existing Panama Locks involves a ballasting sequence to control sinking and floating of gates with the assistance of a crane. This same approach can be used to remove and install the larger gates for the Third Lane Lock. The gates can be transported to and from a centralized maintenance facility.

4.4.2.1. Floating Stability Evaluation Of Miter Gates

The basic dimensions of the new miter gates in the vertical position are as follows:

- L* = 35.46 m. This is the overall length of the longer sidewall. This dimension is tied with the width dimension of the locks.

- L = 27 m. This length corresponds to the length of the internal subdivision chambers of the gate (Buoyancy Chambers). The end prismatic chambers are not watertight and water can enter those chambers from the shorter side wall of the gate. Keeping the end prismatic chambers open to the exterior will facilitate the fabrication, inspection and maintenance of these "corner" areas.
- W = 4.15 m. Gate width.
- H = 35.65 m. Gate height to control the water levels inside the lock.
- The weight of one gate (with W = 4.15 m) is 1 770 t.

The gate model considered in this report includes the internal subdivision of the gate. The gate internal chambers (Voids) are numbered from the bottom up. Void1 corresponds to the row of chambers from the bottom of the gate to 2.2 m above the bottom; Void 2 corresponds to the row of chambers from the top of Void1 to 4.4 m above the bottom of the gate etc... Void4 extends from 6.6 m to 8.8 m above the bottom of the gate, etc.

The hydrostatic model including the internal subdivision of the gate, will allow evaluating the results of flooding or partially filling with water any one chamber at a time.

4.4.2.2. Installing the Gates With the Titan Barge Crane

With the gate floating almost horizontal with the voids 1 and 2 flooded and void 3 filled with water to 40% capacity, with an inclination of about 81.75 degrees from the vertical, the Titan would hook up to the pick up point on top of the gate and apply a lift force of 300 t to bring the gate to the vertical position as follows:

```
        MITER GATES 35.65 METERS HIGH AND 300 METRIC TON CRANE LIFT
                       WEIGHT and DISPLACEMENT STATUS
                  BPL draft: 18.319 @ 13.50f, 18.319 @ 13.50a
                   Trim: 0.000/27.000,  Heel: Stbd 0.35 deg.
Part----------------------------Weight(MT)----LCG-----TCG-----VCG
LIGHT SHIP                        1,770.00   0.000   0.300s 15.250
Crane Lift                         -300.00   0.000   1.750s 35.650
  Total Fixed--------->           1,470.00   0.000   0.004s 11.087
               Load-----SpGr-----Weight(MT)----LCG-----TCG-----VCG------RefHt
VOID1.C       1.000    1.000        242.79   0.000   0.000   1.100
VOID2.C       1.000    1.000        242.79   0.000   0.000   3.300
VOID3.C       0.400    1.000         97.10   0.000   0.010s  4.840     -5.280
  Total Tanks--------->             582.68   0.000   0.002s  2.640
  Total Weight-------->           2,052.68   0.000   0.003s  8.689
                                  Displ(MT)----LCB-----TCB-----VCB
HULL                   1.000      2,052.35   0.000   0.000   9.159    -18.318
-----------------------------------------------------------------------------
               Righting Arms:                0.000   0.000s
Distances in METERS.---------------------------------------------------------

                          HYDROSTATIC PROPERTIES
            Trim: 0.000/27.000, Heel: Stbd 0.35 deg., VCG = 8.689
 Origin  Displacement    Center  of Buoyancy
  Depth----Weight(MT)----LCB-----TCB-----VCB-----WPA-----LCF------BML-----BMT
 18.318     2,052.35   0.000   0.000   9.159   112.0   0.000     0.05   0.001
```

```
  LCF   Displacement   Buoyancy-Ctr.  Weight/          Moment/
 Draft----Weight(MT)----LCB-----VCB-------CM-----LCF---CM trim----GML-----GMT
18.319      2,052.35   0.000   9.159      1.12  0.000      0.40    0.52   0.471
Distances in METERS.-----Specific Gravity = 1.000.-----------Moment in M.-MT.
                             Trim is per 27.00M.
Draft is from BPL.                                     True Free Surface included.
```

The gate is floating stable with the assistance of the crane with a heel of 0.35 degrees to starboard. The metacenter is still above the VCG, and GMT= 0.471 m, which translates in positive and sufficient stability.

The bottom of the gate is at 18.319 m below the surface, and the crane can now lower the gate on the pintle, 18.35 m below the surface.

This operation to upright the miter gate from the free floating position with 81.75 degrees inclination from the vertical (almost horizontal) when voids 1 and 2 are flooded and void 3 is filled to 40% capacity, requires a depth of water of about 19 m, and a crane lift of 300 t which is well within the rated capacity of the Titan. The installation procedure is shown conceptually in Figure B-4-10.

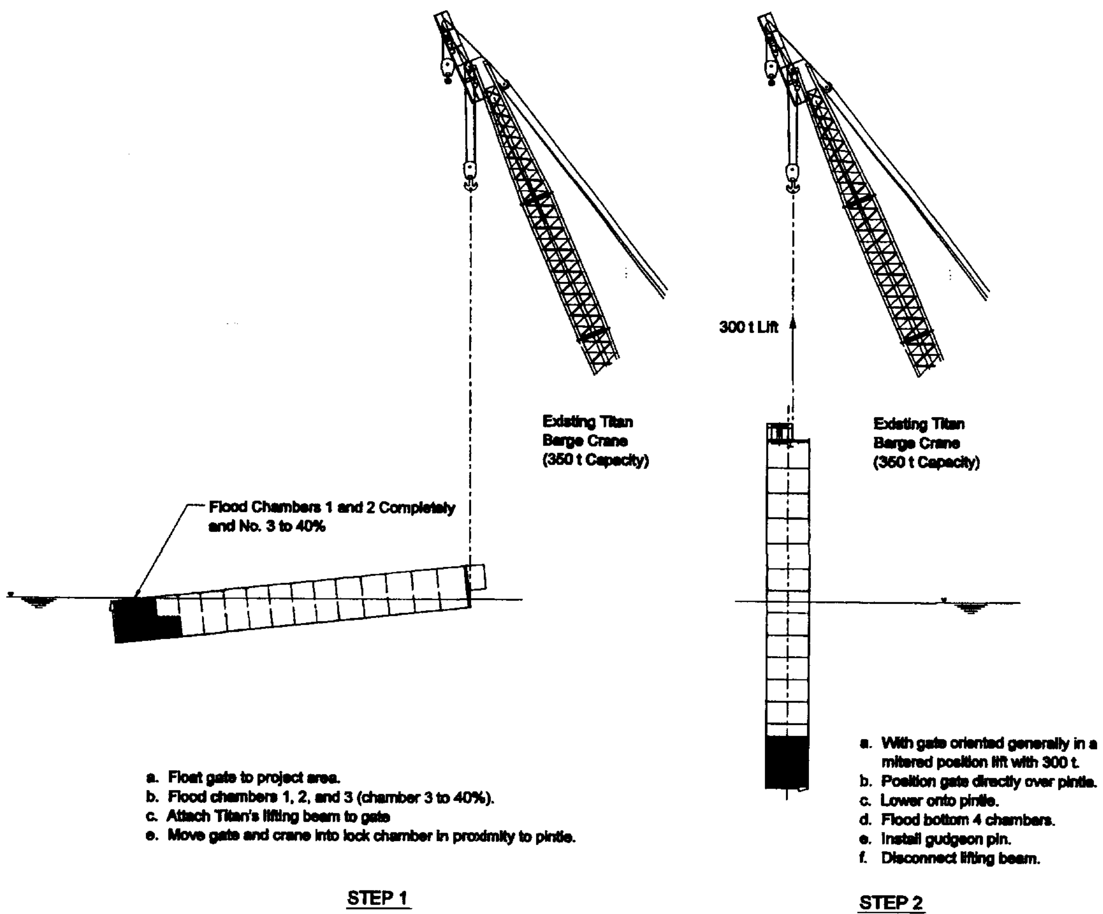


Figure B-4-10 Gate Installation with the Titan Barge Crane

4.4.2.3. Installing the 4.15 m Miter Wide Gate With a Heavy Lift Barge

A heavy lift barge (HLB) could be equipped with a transfer system to receive gates upon floating off the pintle and then rotated the gates to a horizontal position. This barge system can also be used to set gates after they have be floated to their installation point. The gates would be righted as shown in Figure B-4-12. With the gate positioned horizontally on the front of a barge two option are available for service. First, floating plant infrastructure could be built to service the gates afloat. There are twelve lock gates; all gates could be serviced in one year with one-month work time on each gate. The gates could also be delivered to land based maintenance facilities without the use of a syncrolift system.

4.4.2.3.1. Setting Gates on the Pintle

For this operation, the **HLB** will submerge to pick up the gate that shall be free floating almost horizontal (88.19 degree inclination), with all interior chambers dry.

The HLB ballasts to submerge and is placed under the gate:

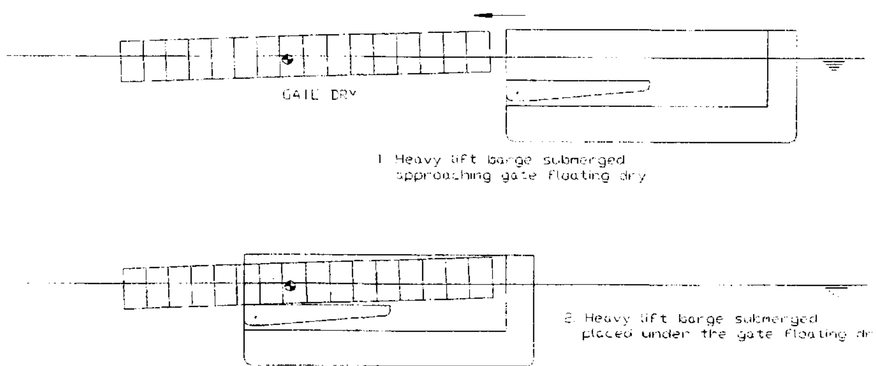


Figure B-4-11 – Heavy Lift Barge Retrieving Gate

The gate is then placed in the upright position using a combination of ballast inside the gate and hydraulic jacks to operate the 7 (seven) "positioning levers" provided on the deck of the **HLB**.

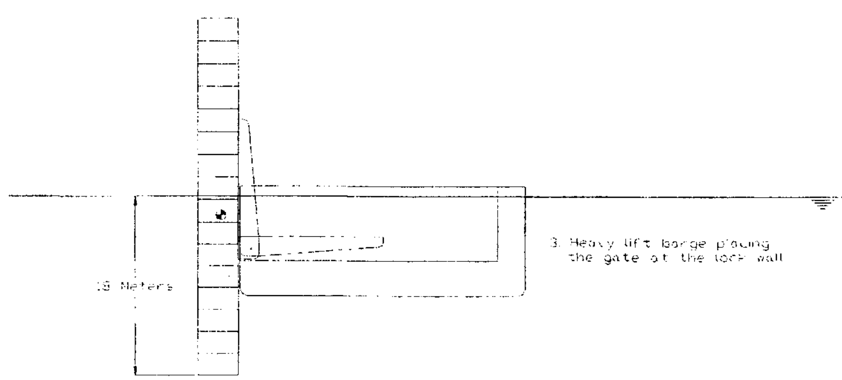


Figure B-4-12 – Heavy Lift Barge Setting Gate on Pintle

Proper adjustment of the ballast inside the gate and on the **HLB** will permit installation on the pintle base at the proper water level on the lock wall. Gates can be removed in the reverse order then transported as illustrated in Figure B-3-14.

4.4.2.3.2. Gate Transport

For this operation, the barge will submerge to pick up the gate that shall be free floating almost horizontal (88.19 degree inclination), with all interior chambers dry.

The Barge ballasts to submerge and is placed under the gate:

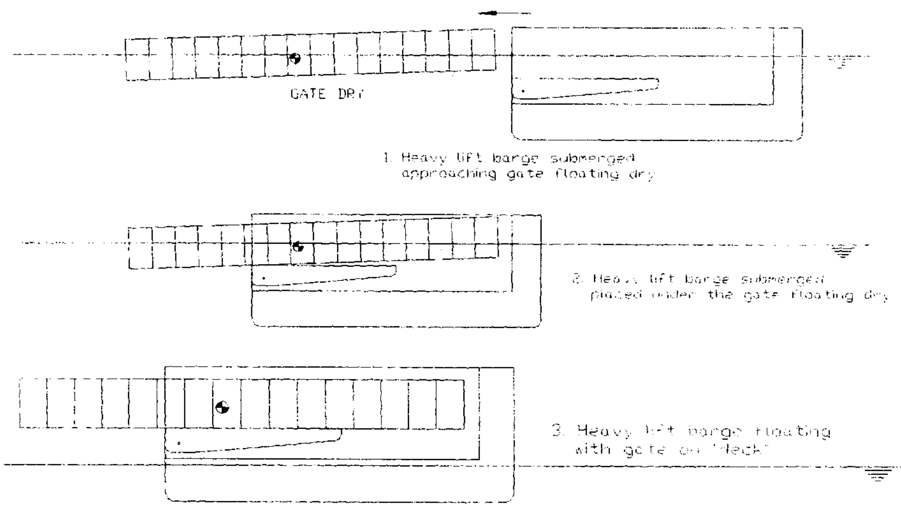


Figure B-4-13 – Heavy Lift Barge Transport Sequence

After the gate is positioned on the **HLB**, the **HLB** can start pumping out ballast to lift the gate out of the water. The gate will place itself horizontal on the "lever arms" or any blocking pre-positioned on the deck of the **HLB**, because the CG of the gate is located inboard of the side of the **HLB**.

Another barge can be brought alongside the **HLB**, to support the gate overhang. This second barge can accommodate the crew that will be involved in the repair and maintenance work on the gate.

4.5. Maintenance And Maintenance Facilities

It is noted that the miter gates can be fitted with hydraulically articulated ramps to permit small pickup trucks to cross over the lock chamber.

A detailed evaluation of installation and removal procedures is beyond the scope of this screening study. However, installation and removal of miter gates would be with the minimal use of cranes. This would be accomplished with the use of buoyancy chambers divided into compartments separated by vertical watertight diaphragms. Control of ballast in the compartments would be necessary for setting and removing the gate leaves by flotation. Water level in buoyancy chambers would be controlled by the use of compressed air and valves. Supplemental gate handling and positioning devices would be embedded in the gate monolith to facilitate positive and accurate gate placement onto the pintle.

A separate maintenance facility is recommended for the use of miter gates due to a lifting capacity limitation on existing syncrolift systems. A concept is presented in Figures B-4-14 to B-4-18 for a maintenance facility located on the shores of Gatun Lake or the Atlantic Ocean approach to the locks. The underwater skid shown in Figure B-4-14 could be eliminated with a barge based deliver system as shown conceptually in Figures B-4-15.

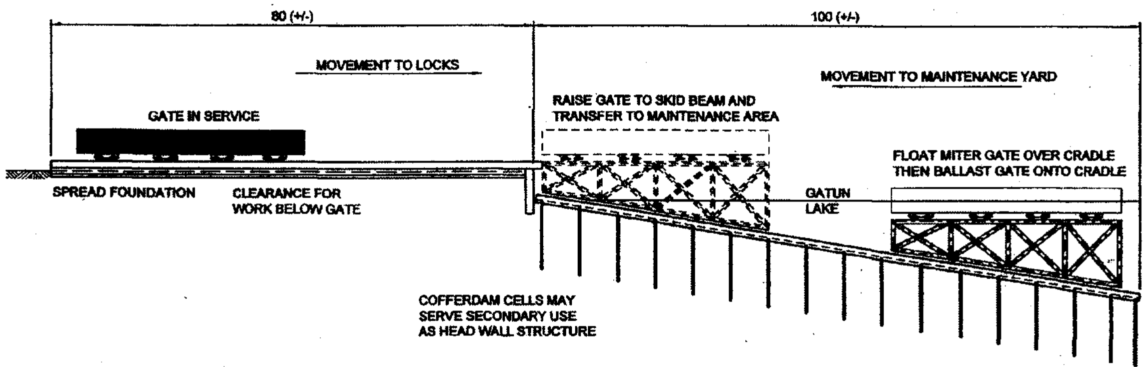


Figure B-4-14 Conceptual Lock Gate Maintenance Facility

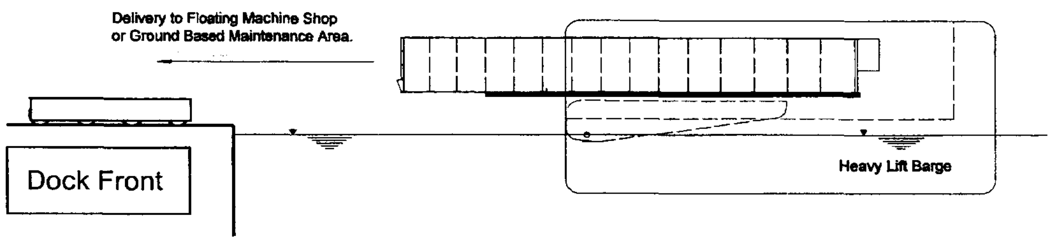


Figure B-4-15 Barge Delivery to Gate Maintenance Facility

The maintenance facilities at Mt. Hope would be expanded to service the new gates. This would provide reuse of significant maintenance infrastructure, reducing costs and consolidating repair work to one site. The necessary work would include clearing of existing grounds, construction of a new dock front and installation of a rail based transfer system. The arrangement of the dock front coupled with the ballasting capability of the delivery barge would accommodate vertical alignment for off-loading miter gates in consideration of Atlantic Ocean tidal variations. Maintenance activities would be easier and safer with the gate in a horizontal orientation. Scaffolding, associated rigging and handle schemes would not be necessary, as it would be required to work on or within a 35 m tall structure.

Figure B-4-16 Possible Development at Mt. Hope

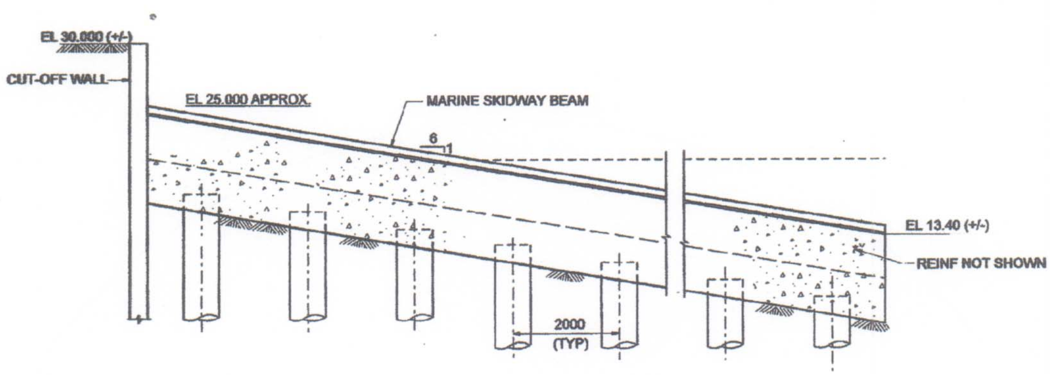


Figure B-4-17 Concrete Skid Beam

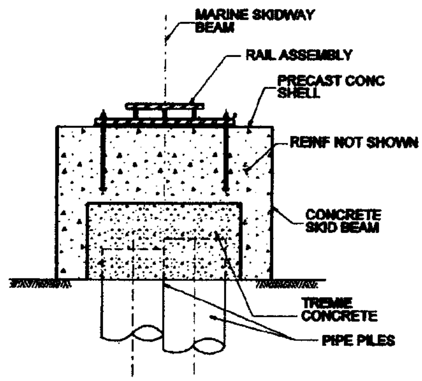


Figure B-4-18 Section Through Concrete Skid Beam

4.6. Miscellaneous Comments

An arched miter gate could be considered in continuing studies for further gate weight saving should the ability to float lock gates be determined unwarranted by the ACP. An arched miter gate would be efficient with the mitering end moved further upstream relative to that for the tapered miter gates (miter angle greater than 1:3). This change in geometry would require a longer chord length from contact block to contact block. A wider gate leaf would likely result in higher operating loads due to greater submerged area. The gate bay length would increase with the greater spacing between pintles. The miter gate monolith lock walls may increase in width to accommodate deeper gate recesses. The water use for each lockage would increase due to the increase of ineffective lock chamber length. The water saving basins would also need to be upsized for the longer chambers. In all, water use, lock layout, maintenance infrastructure, filling and emptying times, and gate weights are inter-related and would need to be reevaluated in following studies. The thrust line of an arch gate is shown in Figure B-4-19.

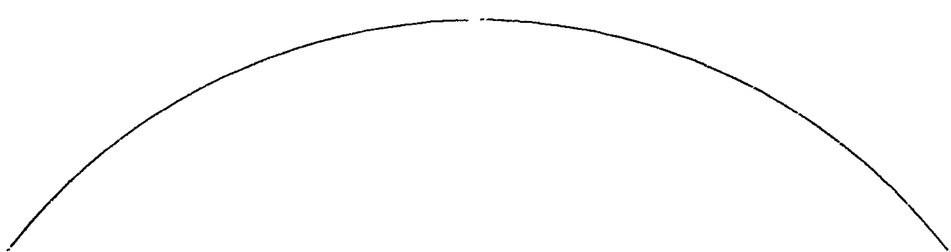


Figure B-4-19 Thrust Line of Arched Miter Gate

5. ROLLING GATE ALTERNATIVE

The second alternative under consideration for the Atlantic Locks Structure, Third Lane Expansion Project, was the use of Rolling Gates. A study was performed to determine the feasibility and cost associated with this alternative similar to that for the miter gate. This alternative has been redesign from the draft Gate Selection Study. Ballast Chambers have been added to permit the gate to be removed and installed by floatation and to reduce operating loads on the roller assemblies. The end sections are open where possible to reduce hydraulic drag during operational movements and the general arrangement of supports has been modified to reflect a "wheel barrow" configuration with additional stability provided by a rolling train support in the recess chamber. Open grating on the gate ends and 10 mm skin plating on the downstream face is provide to keep debris out of the gate body. Approximately 2500 t of removable lead would be required at the bottom of the gate to provide adequate floating stability when moving the gate out of the gate recess into open water. Lead ballast is not required for normal operation of the gate. The combined weight of the gate and buoyancy chambers would be approximately 5 100 t without lead ballast.

In order to determine a reasonably accurate estimate, it was necessary to develop a conceptual layout for such a gate system and perform a conceptual level structural design. The analytical approach taken was to develop the gate's major structural system, its member sizes, and the necessary shell plate thickness. This information was used to estimate the gross material weight of structural steel and plate work.

5.1. Geometry

The clear span between adjacent lock walls is set at 60.96 m. It was determined that, in order to distribute gate reactions and properly seal the structure, the gate would project beyond the wall surface into a recess. The overall length of the rolling gate was set at 67.1 m. A range of gate widths varying from 6 to 12.2 m was examined to evaluate gate weights (associated cost) and predicted deflections (serviceability). The optimum structure depth was found to be 10 m. The gate weight increased as the depth increased to 12 m as expected. The gate recess was sized to accommodate the rolling gate and operating equipment with clearances set to allow routine maintenance. The base elevation of the gate foundation was set at –7.77 m.

5.2. Method of Analysis

The rolling gate was designed using a stiffness/finite element structural analysis program (STAAD/Pro). Initially an acceptable plate thickness and stiffener system was determined for the maximum hydrostatic pressures. This was accomplished by developing a series of finite element models with varying plate thickness and stiffener spacing. After a selected plate and stiffener spacing was set, the gate was modeled as a three-dimensional truss consisting of horizontal and vertical trusses spaced at 6.1 m intervals. The upstream face was framed with rolled sections from truss to truss and covered with a finite element plate mesh to serve as a damming surface. The structural beam stiffeners serve to support the exterior plate elements (spaced at 1 m) and carry the hydrostatic pressures to the main space frame system.

A consideration for reducing the size of the horizontal/vertical truss top and bottom cords was studied by increasing the overall width of the gate. The gate was modeled at a width of 9.1 m and 12.2 m. The results indicate a reduction in some member sizes at 12.2 m but the overall weight of steel increased. There was also an increase in cost for the concrete recess, sill, ship positioning bridge, and total gate steel.

A consideration for reducing the gate sill foundation cost was studied by eliminating the lateral support at the base and transferring the lateral forces into the lock walls. The gate was modeled in the first case with lateral support along the sill and vertically along the lock walls. This resulted in a 4 100 t structure with large lateral loads on the gate sill. The geometry of the gate sill would require a steel grillage installed in slot to distribute forces down into the sill. Inclined rock anchors may be required to resist sliding when uplift pressures are considered. Biaxial tension loads exist in the downstream framing members of the gate when supports are provided along the wall and sill. This state of stress would require special connection treatment when considering fracture mechanics and establishing permissible initial weld flaw size for non-destructive testing requirements. The second model was developed with no lateral support along the sill, lateral support was taken at the walls only. A rubber seal and embedded stainless steel plates would be used to provide a seal under the rolling gate. When no lateral support was used at the base the gate weight increases by 910 t to 5 000 t and the deflection was acceptable at 150 mm to 180 mm. A stress plot of the gate with hydrostatic and hydrodynamic water load from earthquake is shown in Figure B-5-1.

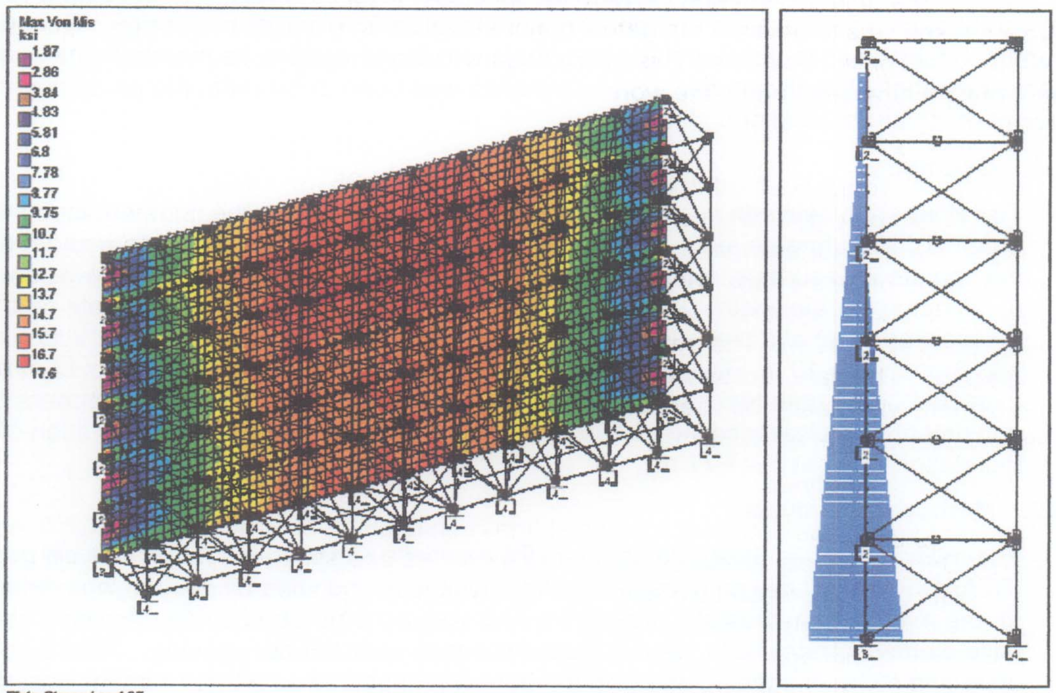


Plate Stress Load 27

Figure B-5-1 Stress Plot of Rolling Gate with Earthquake Loading.

5.3. Materials

The thickness of plate in the lower 12 m would be approximately 32 mm thick. The thickness of plate in the central 12 m would be approximately 16 mm inch thick. The thickness of the plate in the top 12 m would be approximately 13 mm inch thick. The rolled sections and plate work would be of Grade 345 material.

5.4. Buoyancy and Floating Stability Considerations

5.4.1. General

The basic dimensions of the proposed new gate (in the vertical position), is as follows:

- L = 64.00 m. This is the overall length of the gate. This dimension is tied with the width dimension of the lock, and shall not be modified.
- W = 10.36 m. Gate width. This dimension can be different if necessary to provide better stability.
- H = 35.36 m. Gate height to control the water level inside the locks.
- The weight of one gate is 5 079 t.

5.4.2. Gate Floating "Vertical" In 16.5 Meters Of Water

The hydrostatic model dimensions are:

- Length = 10.36 m (the width of the gate),
- Beam = 64.00 m (the length of the gate), and
- Depth = 35.36 m (the height of the gate).

The position of the center of gravity (CG) of the weight (5 079 t), with the hydrostatic model in the vertical configuration above described, is:

- X = 1.22 m forward midships (midships is the origin for X coordinates at the mid-point of the gate width). The forward end of the gate model is –5.18 m from the origin, and the aft end is 5.18 m from the origin for a total length of 10.36 m. X is negative forward.)
- Y = 0 on centerline. Y is positive to starboard.
- Z = 15.24 m above the bottom of the gate. The bottom of the gate is the origin for Z. Z is positive upward.

In this configuration, the Hydrostatic Properties of the roller gate are:

HYDROSTATIC PROPERTIES

No Trim, No Heel

Origin Depth	Displacement Weight (MT)	Center of Buoyancy LCB	TCB	VCB	WPA	LCF	BML	BMT
1.000	663.04	0.008f	0.000	0.500	663.0	0.000	8.94	341.334
2.000	1,326.07	0.004f	0.000	1.000	663.0	0.000	4.47	170.667
3.000	1,989.11	0.003f	0.000	1.500	663.0	0.000	2.98	113.778
4.000	2,652.15	0.002f	0.000	2.000	663.0	0.000	2.24	85.333
5.000	3,315.19	0.002f	0.000	2.500	663.0	0.000	1.79	68.267
6.000	3,978.22	0.001f	0.000	3.000	663.0	0.000	1.49	56.889
7.000	4,641.26	0.001f	0.000	3.500	663.0	0.000	1.28	48.762
7.500	4,972.78	0.001f	0.000	3.750	663.0	0.000	1.19	45.511
8.000	5,304.30	0.001f	0.000	4.000	663.0	0.000	1.12	42.667

8.500	5,635.82	0.001f	0.000	4.250	663.0	0.000	1.05	40.157
9.000	5,967.34	0.001	0.000	4.500	663.0	0.000	0.99	37.926
9.500	6,298.86	0.001	0.000	4.750	663.0	0.000	0.94	35.930
10.000	6,630.38	0.001	0.000	5.000	663.0	0.000	0.89	34.133
10.500	6,961.89	0.001	0.000	5.250	663.0	0.000	0.85	32.508
11.000	7,293.41	0.001	0.000	5.500	663.0	0.000	0.81	31.030
11.500	7,624.93	0.001	0.000	5.750	663.0	0.000	0.78	29.681
12.000	7,956.45	0.001	0.000	6.000	663.0	0.000	0.75	28.444
12.500	8,287.97	0.001	0.000	6.250	663.0	0.000	0.72	27.307
13.000	8,619.49	0.001	0.000	6.500	663.0	0.000	0.69	26.256
13.500	8,951.00	0.001	0.000	6.750	663.0	0.000	0.66	25.284
14.000	9,282.52	0.001	0.000	7.000	663.0	0.000	0.64	24.381
14.500	9,614.04	0.001	0.000	7.250	663.0	0.000	0.62	23.540
15.000	9,945.56	0.001	0.000	7.500	663.0	0.000	0.60	22.756
15.500	10,277.08	0.001	0.000	7.750	663.0	0.000	0.58	22.022
16.000	10,608.60	0.001	0.000	8.000	663.0	0.000	0.56	21.333
16.500	10,940.12	0.000	0.000	8.250	663.0	0.000	0.54	20.687
17.000	11,271.63	0.000	0.000	8.500	663.0	0.000	0.53	20.078
17.500	11,603.15	0.000	0.000	8.750	663.0	0.000	0.51	19.505
18.000	11,934.67	0.000	0.000	9.000	663.0	0.000	0.50	18.963
18.500	12,266.19	0.000	0.000	9.250	663.0	0.000	0.48	18.450
19.000	12,597.71	0.000	0.000	9.500	663.0	0.000	0.47	17.965
19.500	12,929.23	0.000	0.000	9.750	663.0	0.000	0.46	17.504
20.000	13,260.75	0.000	0.000	10.000	663.0	0.000	0.45	17.067
20.500	13,592.27	0.000	0.000	10.250	663.0	0.000	0.44	16.650
21.000	13,923.79	0.000	0.000	10.500	663.0	0.000	0.43	16.254
21.500	14,255.30	0.000	0.000	10.750	663.0	0.000	0.42	15.876
22.000	14,586.82	0.000	0.000	11.000	663.0	0.000	0.41	15.515
22.500	14,918.34	0.000	0.000	11.250	663.0	0.000	0.40	15.170
23.000	15,249.86	0.000	0.000	11.500	663.0	0.000	0.39	14.841

Distances in METERS.-----Specific Gravity = 1.000.-------------------------

From this table, the weight of the gate, 5 079 t, can be floated in about 8 m of water, provided that the gate remains vertical. With the weight of the gate placed at the Center of Gravity CG previously defined, the gate alone is unstable in the vertical position, and would find equilibrium positioned at an angle of 86.79 degrees with the vertical:

```
                          WEIGHT and DISPLACEMENT STATUS
                  Trim: Fwd 86.79 deg.,  Heel: Port 0.08 deg.
Part-----------------------------Weight(MT)----LCG-----TCG-----VCG
WEIGHT                             5,079.00   1.220f  0.000  15.240
                      SpGr------Displ(MT)----LCB-----TCB-----VCB------RefHt
HULL                  1.000        5,080.78   3.984f  0.052p 15.086      1.942
-----------------------------------------------------------------------------
                Righting Arms:                0.001f -0.003s
Distances in METERS.---------------------------------------------------------
```

During installation of the gate, it is required that the gate remains stable in the vertical position, to be positioned in 16.5 m of water. From the table of Hydrostatic Properties, this corresponds to a total weight of the gate equal or less than 10 940 t, which allows for ballast equal or less than 5 861 t.

The following is a determination that adding water ballast alone would not suffice to make this gate stable in the vertical position:

```
                    WEIGHT and DISPLACEMENT STATUS

                         Origin Depth: 10.328
                    Trim: Fwd 54.21 deg.,  Heel: zero
Part----------------------------Weight(MT)----LCG-----TCG-----VCG
LIGHT SHIP                        5,079.00   1.220f  0.000  15.240
WATERBALLAST                      6,630.00   0.000   0.000   5.000
  Total Weight-------->          11,709.00   0.529f  0.000   9.442
                         SpGr-----Displ(MT)----LCB-----TCB-----VCB------RefHt
HULL                     1.000   11,708.99   0.703f  0.000   9.317    -10.328
-----------------------------------------------------------------------------
               Righting Arms:                0.000   0.000
Distances in METERS.---------------------------------------------------------
```

In the above calculation, water ballast equivalent to 10 m of water inside the gate was placed with a VCG (Vertical Center of Gravity) at 5 m above the bottom of the gate. With this much water the gate is still inclined 54.20 degrees from the vertical, and the total weight exceeds the maximum weight of 10 940 t to float in 16.5 m of water.

The conclusion is that solid ballast is required to make this gate stable in the vertical position, while floating with a draft of 16.5 m or less for the installation to be feasible.

Next, 5 750 t of solid ballast shall be considered, placed at the bottom of the gate with an offset to balance the center of gravity of the gate itself, and with a VCG of 0.5 m above the bottom of the gate (this requires the specific gravity of the solid ballast to be 9 or greater):

```
                     WEIGHT and DISPLACEMENT STATUS

                     Baseline draft: 16.330 @ Origin
                     Trim: Aft 0.05 deg.,  Heel: zero
Part----------------------------Weight(MT)----LCG-----TCG-----VCG
LIGHT SHIP                        5,079.00   1.220f  0.000  15.240
SOLIDBALLAST                      5,750.00   1.080a  0.000   0.500
  Total Weight-------->          10,829.00   0.001a  0.000   7.413
                         SpGr------Displ(MT)----LCB-----TCB-----VCB---
---RefHt
HULL                     1.000   10,828.95   0.001   0.000   8.166
-16.332
---------------------------------------------------------------------
--------
               Righting Arms:                0.000   0.000
Distances in METERS.-------------------------------------------------
--------
```

From the above results, the conclusion is that with 5 750 t of solid ballast (specific gravity 9 or greater to obtain a VCG for the solid ballast of 0.5 m above the bottom of the gate or less), placed at the bottom of the gate, with an offset to balance the center of gravity of the gate itself, the gate would float in about 16.33 m of water with practically no inclination from the vertical (Trim = 0.05 degrees).

The following calculations determine the minimum amount of solid ballast material that combined with water ballast would maintain the gate stable in the vertical position floating with a draft of 16.5 m of water or less. Lead ballast with specific gravity of 11.3 is assumed.

```
                    WEIGHT and DISPLACEMENT STATUS

                    Baseline draft: 16.500 @ Origin
                     Trim: 0.00 deg.,  Heel: zero
Part-----------------------------Weight(MT)----LCG-----TCG-----VCG
LIGHT SHIP                         5,079.00  1.220f  0.000  15.240
SOLIDBALLAST                       3,098.00  2.000a  0.000   0.250
WATERBALLAST                       2,763.00  0.000   0.000   2.600
  Total Weight-------->           10,940.00  0.000   0.000   7.803
                     SpGr------Displ(MT)----LCB-----TCB-----VCB---
---RefHt
HULL                 1.000        10,940.00  0.000   0.000   8.250
-16.500
-----------------------------------------------------------------
--------
               Righting Arms:                0.000   0.000
Distances in METERS.---------------------------------------------
--------
```

From the above results, the conclusion is that with 3 098 t of solid ballast, placed inside the gate to obtain a ballast VCG of 0.25 m above the bottom of the gate or less, and placed with an offset to balance the center of gravity of the gate itself (2 m after midships), the gate would still float upright in about 16.5 m of water, provided that water ballast in the amount of 2 763 t is added. There is practically no inclination from the vertical (Trim = 0.00 degrees).

Since the VCG of the total weight is at 7.803 m above the bottom of the gate and the VCB (vertical center of buoyancy) is higher at 8.25 m, another try at reducing the amount of solid ballast follows. 1 500 t of lead is assumed:

```
                      WEIGHT and DISPLACEMENT STATUS

                       Baseline draft: 16.500 @ Origin
                      Trim: Aft 21.53 deg.,  Heel: zero
Part-----------------------------Weight(MT)----LCG-----TCG-----VCG
LIGHT SHIP                         5,079.00  1.220f  0.000  15.240
SOLIDBALLAST                       1,500.00  4.500a  0.000   0.250
WATERBALLAST                       4,361.00  0.000   0.000   4.000
  Total Weight-------->           10,940.00  0.051a  0.000   8.704
                     SpGr------Displ(MT)----LCB-----TCB-----VCB------RefHt
HULL                 1.000        10,939.98  0.214a  0.000   8.292   -15.348
--------------------------------------------------------------------------
               Righting Arms:                0.001   0.000
Distances in METERS.------------------------------------------------------
```

```
                      WEIGHT and DISPLACEMENT STATUS

                       Baseline draft: 16.497 @ Origin
                      Trim: Fwd 10.45 deg.,  Heel: zero
Part-----------------------------Weight(MT)----LCG-----TCG-----VCG
LIGHT SHIP                         5,079.00  1.220f  0.000  15.240
SOLIDBALLAST                       1,500.00  4.000a  0.000   0.250
WATERBALLAST                       4,361.00  0.000   0.000   4.000
  Total Weight-------->           10,940.00  0.018f  0.000   8.704
                     SpGr------Displ(MT)----LCB-----TCB-----VCB------RefHt
HULL                 1.000        10,938.18  0.100f  0.000   8.258   -16.224
--------------------------------------------------------------------------
               Righting Arms:                0.000   0.000
Distances in METERS.------------------------------------------------------
```

The two calculations above show that 1 500 t of solid ballast is not adequate. In the first calculation, the ballast is placed 4.5 m after from midships and the gate is inclined 21.53 degrees from the vertical to the after side. In the second calculation, the ballast is placed 4.0 m aft from midships and the gate is inclined 10.45 degrees from the vertical to the forward side. The vertical equilibrium is unstable with only 1 500 t of lead ballast.

Moving up to consider 2 000 t of lead ballast:

```
                        WEIGHT and DISPLACEMENT STATUS

                         Baseline draft: 16.500 @ Origin
                         Trim: Aft 9.50 deg.,  Heel: zero
Part-----------------------------Weight(MT)----LCG-----TCG-----VCG
LIGHT SHIP                          5,079.00   1.220f  0.000  15.240
SOLIDBALLAST                        2,000.00   3.500a  0.000   0.250
WATERBALLAST                        3,861.00   0.000   0.000   3.500
  Total Weight-------->            10,940.00   0.073a  0.000   8.356
                           SpGr------Displ(MT)----LCB-----TCB-----VCB------RefHt
HULL                       1.000   10,940.00   0.091a  0.000   8.258   -16.274
-----------------------------------------------------------------------------
               Righting Arms:                  0.001   0.000
Distances in METERS.---------------------------------------------------------
```

```
                        WEIGHT and DISPLACEMENT STATUS

                         Baseline draft: 16.497 @ Origin
                         Trim: Fwd 2.35 deg.,  Heel: zero
Part-----------------------------Weight(MT)----LCG-----TCG-----VCG
LIGHT SHIP                          5,079.00   1.220f  0.000  15.240
SOLIDBALLAST                        2,000.00   3.000a  0.000   0.250
WATERBALLAST                        3,861.00   0.000   0.000   3.500
  Total Weight-------->            10,940.00   0.018f  0.000   8.356
                           SpGr------Displ(MT)----LCB-----TCB-----VCB------RefHt
HULL                       1.000   10,938.32   0.022f  0.000   8.249   -16.483
-----------------------------------------------------------------------------
               Righting Arms:                  0.000   0.000
Distances in METERS.---------------------------------------------------------
```

The two calculations above show that 2 000 t of solid ballast is not adequate. In the first calculation, the ballast is placed 3.5 m aft from midships and the gate is inclined 9.5 degrees from the vertical to the aft side. In the second calculation, the ballast is placed 3.0 m aft from midships and the gate is inclined 2.35 degrees from the vertical to the forward side. The vertical equilibrium is unstable with only 2 000 t of lead ballast.

Moving up to consider 2 200 t of lead ballast:

```
                    WEIGHT and DISPLACEMENT STATUS

                     Baseline draft: 16.498 @ Origin
                    Trim: Fwd 2.79 deg.,  Heel: zero
Part-----------------------------Weight(MT)----LCG-----TCG-----VCG
LIGHT SHIP                         5,079.00   1.220f  0.000  15.240
SOLIDBALLAST                       2,200.00   2.700a  0.000   0.250
WATERBALLAST                       3,661.00   0.000   0.000   3.500
  Total Weight-------->           10,940.00   0.023f  0.000   8.297
                          SpGr------Displ(MT)----LCB-----TCB-----VCB------RefHt
HULL                      1.000   10,938.62   0.026f  0.000   8.250    -16.478
-------------------------------------------------------------------------------
               Righting Arms:                 0.001   0.000
Distances in METERS.-----------------------------------------------------------
```

```
                    WEIGHT and DISPLACEMENT STATUS

                     Baseline draft: 16.497 @ Origin
                    Trim: Aft 4.25 deg.,  Heel: zero
Part-----------------------------Weight(MT)----LCG-----TCG-----VCG
LIGHT SHIP                         5,079.00   1.220f  0.000  15.240
SOLIDBALLAST                       2,200.00   3.000a  0.000   0.250
WATERBALLAST                       3,661.00   0.000   0.000   3.500
  Total Weight-------->           10,940.00   0.037a  0.000   8.297
                          SpGr------Displ(MT)----LCB-----TCB-----VCB------RefHt
HULL                      1.000   10,938.32   0.040a  0.000   8.250    -16.452
-------------------------------------------------------------------------------
               Righting Arms:                 0.000   0.000
Distances in METERS.-----------------------------------------------------------
```

The above results are better, however, the stability around the vertical position is still difficult to control, in part due to the fact that the VCG of the total weight is 8.297 m above the bottom of the gate and above the 8.25 m for the VCB position. The result clearly shows that the gate would try to find the equilibrium position a few degrees to one side or the other from the vertical, being very sensitive to the offset of the solid ballast.

Next, 2 500 t of lead ballast is assumed:

```
                    WEIGHT and DISPLACEMENT STATUS

                     Baseline draft: 16.500 @ Origin
                    Trim: Fwd 3.16 deg.,  Heel: zero
Part-----------------------------Weight(MT)----LCG-----TCG-----VCG
LIGHT SHIP                         5,079.00   1.220f  0.000  15.240
SOLIDBALLAST                       2,500.00   2.300a  0.000   0.250
WATERBALLAST                       3,361.00   0.000   0.000   3.000
  Total Weight-------->           10,940.00   0.041f  0.000   8.054
                          SpGr------Displ(MT)----LCB-----TCB-----VCB------RefHt
HULL                      1.000   10,939.93   0.030f  0.000   8.251    -16.475
-------------------------------------------------------------------------------
               Righting Arms:                 0.000   0.000
Distances in METERS.-----------------------------------------------------------
```

WEIGHT and DISPLACEMENT STATUS

Baseline draft: 16.500 @ Origin
Trim: Aft 0.38 deg., Heel: zero

Part	SpGr	Weight (MT)	LCG	TCG	VCG	RefHt
LIGHT SHIP		5,079.00	1.220f	0.000	15.240	
SOLIDBALLAST		2,500.00	2.500a	0.000	0.250	
WATERBALLAST		3,361.00	0.000	0.000	3.000	
Total Weight-------->		10,940.00	0.005a	0.000	8.054	
	SpGr	Displ (MT)	LCB	TCB	VCB	RefHt
HULL	1.000	10,940.32	0.004a	0.000	8.250	-16.500
Righting Arms:			0.000	0.000		

Distances in METERS.

The two calculations above show that 2 500 t of solid ballast is adequate. In the first calculation, the ballast is placed 2.3 m aft from midships and the gate is inclined 3.16 degrees from the vertical to the forward side. In the second calculation, the ballast is placed 2.5 m aft from midships and the gate is inclined 0.38 degrees from the vertical to the after side. The vertical equilibrium is stable with 2 500 t of lead ballast.

The solid ballast chamber would be the bottom chamber, used for solid ballast only. The solid ballast chamber height can extend from the bottom of the gate to about 0.5 m above the bottom, to be filled with lead of specific gravity 11.3. As an alternative, the solid ballast chamber can have a chamber height greater than 0.5 m and allow water ballast to share the chamber with the solid ballast. Adjustment of the gate draft at installation would be made with the right amount of water ballast.

5.4.3. Gate Remaining In Place In 22 Meters Of Water

The gate would remain in place when the water level rises during operation of the locks to a maximum water level of about 22 m. To accomplish this, more water would be allowed inside the gate.

HYDROSTATIC PROPERTIES

No Trim, No Heel

Origin Depth	Displacement Weight (MT)	Center of Buoyancy LCB	TCB	VCB	WPA	LCF	BML	BMT
4.000	2,652.15	0.002f	0.000	2.000	663.0	0.000	2.24	85.333
5.000	3,315.19	0.002f	0.000	2.500	663.0	0.000	1.79	68.267
6.000	3,978.22	0.001f	0.000	3.000	663.0	0.000	1.49	56.889
16.000	10,608.60	0.001	0.000	8.000	663.0	0.000	0.56	21.333
16.500	10,940.12	0.000	0.000	8.250	663.0	0.000	0.54	20.687
17.000	11,271.63	0.000	0.000	8.500	663.0	0.000	0.53	20.078
21.000	13,923.79	0.000	0.000	10.500	663.0	0.000	0.43	16.254
21.500	14,255.30	0.000	0.000	10.750	663.0	0.000	0.42	15.876
22.000	14,586.82	0.000	0.000	11.000	663.0	0.000	0.41	15.515
22.500	14,918.34	0.000	0.000	11.250	663.0	0.000	0.40	15.170
23.000	15,249.86	0.000	0.000	11.500	663.0	0.000	0.39	14.841

Distances in METERS.-----Specific Gravity = 1.000.

From the table of Hydrostatic Properties, at the 22 m draft, the total displacement of the gate is 14 586 t. The total weight of the gate would be greater than 14 586 t to stay in place. This is equivalent to ballasting the gate with about 11 m of water inside the gate, which would add about 7 293 t to the weight of the gate for a total of 14 872 t (=5 079 gate weight +2 500 lead weight + 7 293 water ballast).

The VCG of the 7 293 t of water ballast would be about 6 m above the bottom of the gate.

WEIGHT and DISPLACEMENT STATUS

Baseline draft: 22.430 @ Origin
Trim: Aft 0.06 deg., Heel: zero

Part	Weight (MT)	LCG	TCG	VCG
LIGHT SHIP	5,079.00	1.220f	0.000	15.240
SOLIDBALLAST	2,500.00	2.500a	0.000	0.250
WATERBALLAST	7,293.00	0.000	0.000	6.000
Total Weight-------->	14,872.00	0.004a	0.000	8.189

	SpGr	Displ (MT)	LCB	TCB	VCB	RefHt
HULL	1.000	14,872.00	0.000	0.000	11.215	-22.430
Righting Arms:			0.000	0.000		

Distances in METERS.

The draft is now 22.43 m. The gate remains in place and in equilibrium in the vertical position, until it would float again if the water level is 22.43 m above the bottom of the gate or higher, or the water ballast inside the gate is emptied.

5.4.4. Gate With 2 500 Metric Tons Of Solid Ballast Only

With the interior of the gate dry, (no ballast water), the gate with solid ballast would float out of the vertical as follows:

WEIGHT and DISPLACEMENT STATUS

Origin Depth: 2.926
Trim: Aft 74.07 deg., Heel: zero

Part	Weight (MT)	LCG	TCG	VCG
LIGHT SHIP	5,079.00	1.220f	0.000	15.240
SOLIDBALLAST	2,500.00	2.500a	0.000	0.250
Total Weight-------->	7,579.00	0.007a	0.000	10.295

	SpGr	Displ (MT)	LCB	TCB	VCB	RefHt
HULL	1.000	7,579.00	2.439a	0.000	9.601	-2.926
Righting Arms:			0.000	0.000		

Distances in METERS.

The gate with solid ballast of 2 500 t, and without water ballast, would float with an inclination of 74.07 degrees from the vertical. However, adding water ballast would make the gate stable in the vertical position for installation in 16.5 m of water, and for operation at higher water levels as previously determined.

5.5. Rolling Gate Monolith and Recess

5.5.1. Geometric Considerations

When considering all aspects of the functional requirements of the intermediate rolling gate structure monoliths and gate recesses, several dimensional provisions would be required regardless of analyses optimization. The minimum recess width would be established based on the gate width, operating clearances, and clearance allowance for maintenance activities. It would be desirable to minimize the width of the recess slots since water use is related to the plan area of this feature. The overall length of the recess would be established on similar criteria except there would be added needs for a bridge to carry the ship positioning systems over the recess and added length of the recess on top for space for operating equipment. Adequate top of wall space would be required for a point of access to the locks, machinery closures, and maintenance activities all around the gate areas.

The minimum space between adjacent rolling gates would be established by the minimum width of the gate reaction monolith, width for stability of the intermediate wall with one recess dewatered for maintenance activities. A practical limitation on space would be established to facilitate maintenance and access. The minimum dimension for this consideration would be approximately 15 m when machinery recesses are considered.

The overall length of the sill was set at 63 m, however, additional studies would be required to verify that the upstream wall monolith adjacent to the upstream recess can be shortened to the extent shown in this study.

To accommodate the rolling gate recesses, culverts for the filling and emptying systems would be either located longitudinally in the lock itself, or in the lock wall below the gate recesses.

If the rolling gate side culvert is lowered to pass under the gate recess, the opposite culvert should be lowered as well to maintain symmetry. Assuming thick gravity walls would be constructed, the culverts could be lowered without severely affecting the design of the walls. Also, lowering the culverts would not degrade hydraulic performance of the filling and emptying system. One disadvantage, though, would be that it makes the valves more difficult to access. The amount of lowering necessary varies depending on location and type of filling system. For the major portion of the system, culverts would need to be lowered up to 6 m below where they otherwise might be placed. However, at the downstream rolling gates of the lower chamber, the tops of culverts would need to dive approximately 20 m below ocean level to clear the recesses or pass completely around the gate recess. Depending on the filling system, this is approximately 3 to 17 m lower than would otherwise be necessary

5.5.2. Cross Section and Configuration

Several cross sections have been analyzed for the gate recess including gravity walls, tieback systems, and counterfort walls. W-Frame walls were selected for the rolling gate recess slots based on applied operating equipments loads and dewatering conditions for maintenance. The east wall gate monolith, opposite from the recess, would be a gravity structure. A steel grillage system would be provided to in the bearing slots to support gate reaction forces, this support system has not been fully designed but is included in estimated costs based on engineering assumptions and adaptation from other Corps projects.

5.5.3. Design Loads /Combinations

The loading conditions used for the analyses of the lock walls follows the U.S. Army Corps of Engineers' criteria for gravity structures. The Army Corps of Engineers criteria for the stability of lock structures used for the analysis and design are as follows:

The specific loads that were applied to the gate monolith wall sections are as follows:

Dead Loads - The dead loads included the weight of concrete, backfill, and gate self weight.

Lateral Earth Loads - Lateral earth pressures on the structure were calculated in accordance with EM 1110-2-2502 – Retaining and Flood Walls. At-rest lateral earth pressures were used.

Gate Reaction Loads - Hydrostatic forces on gate structures are determined according to the pool levels in the given load case.

Hydrostatic Loads - Lateral and uplift hydrostatic forces on the structure are determined according to the pool levels in the given load case.

Surcharge - Vehicular and miscellaneous surcharge due to the water saving basins and other surcharges equal to 4.5 m of earth was assumed behind the wall.

Maximum Design Earthquake (MDE) - The analysis performs a pseudo-static seismic analysis with computed dynamic forces. These dynamic forces include hydrodynamic soil and water forces and structure inertia forces. Earthquake loading is treated as an inertial force applied statically to the structure. The earthquake acceleration might take place in any direction; the analysis was performed for the most unfavorable direction.

5.5.3.1. Concrete Inertia Forces

The magnitude of the inertia forces is the product of the structural mass and seismic coefficient. In accordance with EM 1110-2-2200 – Gravity Dam Design, the following formula is used to compute the inertia force for the structure:

$$Pe = \alpha W$$

where Pe = horizontal or vertical force

α = seismic coefficient

W = weight of structure

The resultant forces are assumed to act at their respective centers of gravity.

5.5.3.2. Hydrodynamic Water Pressure

Hydrodynamic water pressures were computed using the Westergaard procedure.

5.5.3.3. Dynamic Earth Pressures

Woods procedure was used to compute the dynamic earth pressures acting along the back of walls.

5.5.3.4. Load Cases for Gate Monoliths

All load cases considered that a fourth lane would not be constructed adjacent to the third lane.

Case 1A & 1B - Normal Operating Condition

Case 1C & 1D - Normal Operating Condition

Case 2A & 2B – Unusual Condition

Case 2A & 2B – Unusual Condition

Case 3A & 3B - Maximum Credible Earthquake/Maximum Design Earthquake Condition

Case 3C & 3D - Maximum Credible Earthquake/Maximum Design Earthquake Condition

5.5.3.5. Load Cases for Gate Recess Walls

Case 1A & 1B - Normal Operating Condition

Upper gate recess water elevation at 26.67 m & 15.05 m.

Lower gate recess water elevation at 15.05 m & 26.67 m.

Backfill and surcharge behind upper and lower recess wall.

Case 2A & 2B Unusual Condition

Upper gate recess water elevation at 26.67 m and lower gate recess water elevation at –0.381 m.

Upper gate recess water elevation at 23.93 m and lower gate recess water elevation at –0.381 m.

Backfill and surcharge behind upper and lower recess wall.

Case 2C & 2D Maintenance Condition

Upper gate recess water elevation at 26.67 m and lower gate recess dewatered.

Upper gate recess dewatered and lower gate recess water at 26.67 m.

Backfill and surcharge behind upper and lower recess wall.

Case 3A, 3B, 3C, & 3D Maximum Credible Earthquake/Maximum Design Earthquake Condition

Same as Case 1A & 1B, except with MCE/MDE in both directions for each case.

Note: Load cases 1A, 1B, 2A, and 2B are not critical and therefore were not examined extensively in these analyses. Other specific load cases were not examined if by inspection it was determined that they would not govern.

5.5.4. Analysis and Design

The analysis of gate recess slots was accomplished using a stiffness/finite element structural analysis program (STAAD/Pro). This system was modeled as a W-Frame structure.

The analysis performed within was an attempt to model and capture the worst-case load scenarios that the wall would see over its service life. The loads computed were established based on conservative assumptions known at the time of the analysis that may be able to be refined to produce a more economical design. A refined analysis would further investigate the foundation parameters, rock strata, reinforcement efficiency, deflection limitations, and refined geometry of the sections.

5.5.5. Operating Machinery

Plate 8 shows the general arrangement of the recommended rolling gate machinery. The rolling gate would be moved by two large rolling-type chains attached to a cross-linked carriage assembly near the top of the recess end of the gate. The drive sprocket for each chain would be attached to the output shaft of a hydraulic motor driven gear reducer. For redundancy, a second, backup motor/reducer would be mounted to the opposite side of the sprocket. A fixed idler sprocket for each chain would be attached to the concrete at the opposite end of the gate recess. Every other link pin of the rolling type chain would incorporate a pair of wheels that ride in horizontally supported guide tracks spanning between the drive and idler sprockets. A single hydraulic power unit, with two pump-motor groups, would serve both chain drives.

Using conventional wheel bearings and with a 5-minute operating time it is anticipated that a 300 kW motor would be required to drive each hydraulic pump. Design hydraulic operating pressures would be 17.2 MPa with a peak allowable of 24.1 MPa.

5.5.6. Maintenance And Maintenance Facilities

Routine maintenance and inspection would be performed with while the gate is dewatered in the gate recess. Floating plant would be necessary to set and install recess bulkheads. Due to limited access for personnel and service equipment, as complexity and severity of repair work increases, the value of using the gate recess diminishes to the point where a loss of feasibility occurs for major repairs. The primary load-carrying members would be located with the skinned structure below rows of horizontal and vertical members. Due to the weight of primary members, with limited access for hoists and limited room for diagonal movement or rotating of large members over 5 m long, major repair work would be extremely difficult to perform.

A closure caisson would be needed to seal the recess opening along the face of lock wall prior to dewatering for inspection and maintenance. This structure would be designed to float, however, the naval architecture has not been performed. The recess opening would be approximately 10.5 m wide. Some arrangement of stacking caissons would be desirable to maximize the use of like features. For the purpose of estimating costs, two caissons were assumed, one approximately 36 m tall and one 23.0 m tall. The 35.5 m caisson would be used on the center gate bay recesses and the downstream gate recesses. The 23.0 m caisson would be used on the upstream gate recesses and the last downstream recess. The caissons would be ballasted approximately vertically and just off the sill. The rolling gate and its operating equipment would be used to move the caisson into position against the slot.

6. SECTOR GATE ALTERNATIVE

A sector gate design is presented as a third alternative because of its ability to open or close under head, which enables it to stop flowing water and it may be used as an emergency gate. It may be advantageous to provide one sector gate at each end of the lock structure to insure against the loss of Gatun Lake level and for maintenance purposes. The gate also has the capability to act as the filling and emptying system for the locks. However, it would present an increase in hawser forces due to its point source filling and emptying by cracking open the gates and in this specific application, is not suitable for this use.

Sector gates are typically used where differential heads are 6 m or less or there are reverse heads, however, there are no technical reasons that prohibit their use in conditions with higher heads. Sector gates are more durable compared to a miter gate or other types of gates in that they can be operated under a differential head (flowing water) and can withstand a reverse head. They are commonly used in tidal areas where they would experience a reverse head. To date, the largest Corps sector gate was the one model tested for use in the Inner Harbor Navigation Canal (IHNC) Lock. Application of sector gates in the Atlantic Locks structure would result in gate heights significantly higher than IHNC, but its use appears to be practical. A general arrangement of the Atlantic Lock with sector gates is shown on Plate 10. It should be noted that these gates are larger than other types and require more in-wall recess space, similar to the rolling gates, but they have the added benefit of being able to be closed under some head of flowing water. They would also require a longer bridge to support locomotive loads should they be used in the Canal Locks as the overall wall opening is much longer. Each sector gate leaf would weigh approximately 2 200 000 kg including a chamber side fendering system and walkway. The size of the gate leaf would require added supports at the pintle column and wheels at the skin plates that are not commonly found on most existing sector gates. The recess walls, thrust blocks, and machine blocks were designed as pure cantilever walls. For a double-lift lock system, the preferred location for filling and emptying culverts would be in the lock walls. Special details would be required to depress the culvert below the gate recess. In a triple-lift lock, in-chamber culverts are deemed feasible and would eliminate a technical problem. The base thickness was based on engineering judgment and geometric requirements. Additional engineering would be required should this alternative be selected for further study. The gate bay geometry shown accommodates the gate shape, but the out-to-out monolith dimensions must be determined by an iterative process to match the foundation design.

6.1. Sector Gate Structure

6.1.1. General.

The structural design is in accordance with Corps engineering guidance and applicable industry standards. The Corps criteria are specified in EM1110-2-2105 and EM 1110-2-2703. Buoyant chambers and bottom rollers (casters) were investigated to counter the deflection and couple-force induced by the tremendous gate weight. Since existing Panama Canal lock gates contain buoyant chambers; this was considered first. The added weight needed to construct the closed chambers and the higher cost of maintaining confined spaces made the use of rollers more attractive. Although, the buoyant tanks could be made removable and disposed of in lieu of maintaining them, the rolling option was favored for this study. The rollers also have drawbacks. The rollers could freeze up and induce a large force on the gate machinery. The New Orleans District is now using stainless steel rollers with a high degree of success on existing structures. Redundant rollers can be added and placed into service as problematic ones are taken out of service. For this design, rollers were used, however, both alternatives need to be considered as the dead load couple-force adds an axial load of 3 600 kN to the bottom chords.

6.1.1.1. Geometry.

The sector-type gates would be of welded construction. The gate would include wheel supports at the skin plate. The gates have a central angle of 70 degrees. The radius to the outside of the horizontal girder would be 33.7 m. The height of the gate is 36.1 m. Each gate has four vertical trusses that would carry the loads to the hinge and pintle. A gear would operate the gate; the gear rack would be located 1.22 m below the top of gate at the same elevation as the top frame. The bottom frame would permit approximately 225 mm of clear space to the floor. A 36.6 m span bridge would be located over the gate recess to allow use of ship positioning equipment. The estimated structure depth is 2.4 m, which raises the top of wall elevation to 31.33 m.

6.1.1.2. Method of Analysis.

The gate was designed using the Working Stress Design (WSD) method. Pipes were used as the truss and frame members as the circular shape has better axial properties. The allowable stresses are in accordance with Chapter 4 of EM1110-2-2105, "Design of Hydraulic Steel Structures". Joint design was done in accordance with the American Petroleum Institute (API) Recommended Practice 2A-WSD (RP 2A-WSD). The gate framing would consist of four vertical trusses and five horizontal frames. The main chords were assumed fixed and the secondary braces were pinned. The horizontal curved girder was designed with the arc load projected onto a straight continuous beam; axial loads and effects of curvature were discounted. The skin plate horizontally spans the intercostals; the skin plate also served as the flange of the vertical intercostals. The dead load of the structure was borne on the wheels and at the pintle ball. The hydrostatic load was directed into the monolith thrust block at the hinge, pintle, and three intermediate support points. The intermediate supports would only resist lateral load, similar to the miter gate quoin.

Two load cases were considered to govern this design, normal operation and loss of the tail water pool. The normal operation case has a headwater at El 26.7 and the tail water at El. 15.1. The gate members were designed for 0.83 percent of the applicable industry standard for this case. The loss of tail water was considered as an overstress condition; stresses were increased to 1.10% of AISC allowable stress values per EM 1110-2-2105. Given the magnitude of the load and size of the gate, a larger overstress would be investigated in future designs. The loss of tail water case governed the design. A ship impact of 1 780 kN was applied to the top two channel side truss panel points. Ship impact on the horizontal girder was included as a 5 m water head; future designs would include a ship impact point load. The impact case would be verified in further studies.

6.1.1.3. Materials.

The skin plate thickness was increased 1.5 mm for corrosion loss. The skin plate would be constructed from material conforming to ASTM A 572/A 572M Grade 345 steel. All rolled sections would also be constructed with Grade 345 material. The chords would be constructed from steel pipe conforming to ASTM A-500 Grade B or API 5L Grade X42. To prevent internal corrosion, as done on offshore pipe structures, the inside surface would be coated and then sealed. Pipe joints, requiring cutting templates, could be manufactured stateside and later spliced to chord lengths in Panama.

6.1.1.4. Hinge and Pintle.

The gate frames are supported at the top by a hinge and at the bottom by a pintle. In order to assure good pintle and hinge alignment, a spherical pin would be used in the hinge to compliment the spherical pintle. Horizontal reactions would be transferred to the lock wall through the bronze bushings and through quoin blocks at the intermediate supports. Anchor bolts for the hinge anchorage would be designed for a maximum tensile stress of 230 MPa using steel with a minimum yield of 415 MPa. In order to insure firm contact between the movable and the fixed hinge castings under all normal conditions, the anchor bolts would be pre-tensioned by tightening the nuts sufficiently to induce a stress of approximately 200 MPa in the bolts. The pre-tension force would be based on the pre-tension stress acting on the bolt root area.

6.1.2. Concrete Monolith and Recess

The reinforced concrete gate bay walls are designed using Ultimate Strength Design, in accordance with ACI 318 and EM 1110-2-2104. To minimize the potential for cracking, the stresses at service loads are checked and compared to the allowable of ACI 357. Additional guidance has also been obtained from the AASHTO Standard Specifications for Highway Bridges and EM 1110-2-2703. Both the walls and the thrust blocks would be designed using a concrete compressive strength of 35 MPa. Backfill placed behind walls would be a granular, free draining material. The water table is assumed to be close to that of the Atlantic Ocean Stage (used El 1.5).

The thrust blocks would be roughly 12 m wide by 14 m long, pure cantilever walls, with a geometry varying to accommodate the sector gates. The overall length of the gate sill was set at 114 m.

If further studied, a more economical design would be considered. The backfill height may be reduced and /or a lightweight, porous aggregate could be used as backfill.

6.1.3. Machinery

Plate 17 shows the general arrangement of the recommended sector gate machinery. There are numerous methods for operating a lock sector gate. Two principal methods are hydraulic cylinder and low speed high torque (LSHT) hydraulic motor with a drive gear attached to the motor and gate rack gear. Drive loads depend on bottom seal friction, hinges and pintle friction, wheel friction, if used, and contingencies. On sector gates used in chamber openings of 33.5 m and less the operating machinery of choice has been LSHT hydraulic motor with a drive gear driving a gear rack mounted to the skin plate near the top of the gate. This method has proven to be trouble free. The drive gear and rack on a large gate such as that proposed for the Panama Canal would have a diametric pitch of approximately 0.75. Because of the very low speed of operation the rack gear would be cut inexpensively using a precision abrasive water jet machine. The drive gear would be approximately 1 to 1.5 m in pitch diameter and would be cut by conventional means. The recommended drive motor would be either a Hagglunds Series 63 or 84 depending on the drive gear arrangement and operating loads. The attached plate shows a Hagglunds 63 Series motor with a 1.0 m pitch diameter gear with a gear width of 100 mm. This machine with the proper hydraulic power unit would also be capable of operating the gate if flotation support is used in lieu of wheels.

The hydraulic power unit (HPU) for driving the LSHT would have an electronically control proportional valve for providing smooth acceleration and de-acceleration. The hydraulic pump would be a horsepower limited, variable speed pump, which has a controlling that changes the flow volume versus pressure requirements to utilize the full horsepower of the motor without exceeding it. The pump would also be set to a maximum flow volume to prevent the gate from over-speeding. The advantage of this pump is that the gate would automatically slow down if there is an unexpected obstruction and then either speed up when passed the obstruction or stop if the obstruction is too much for the gate machinery to power through. This would all be done smoothly and without overloading the pump driver or the machinery and without intervention from the lock operator.

It is anticipated that a 30 kW motor would be required to drive the hydraulic pump. The design tangential rack force for the case above would be approximately 90 kN. Design hydraulic operating pressures would be 17 MPa with a peak allowable of 24 MPa. At 24 MPa the LSHT would provide a rack force of 145 kN.

6.1.4. Maintenance And Maintenance Facilities

The gates would require repainting at an interval not to exceed 20 years. At the time of painting, the hinge and pintle bushings would be changed out. Maintenance other than lubrication and minor repairs cannot be performed in place, the gate would need to removed and transported to a maintenance facility. The size of the gate would the use of a dry dock. For convenience of transportation, installation, and removal, the sector gate would need to be fitted with buoyancy chambers.

7. ALTERNATIVE EVALUATION

All of the three gate type alternatives identified in the previous section are suitable for the Atlantic Locks Project. However, to select the most favorable alternative, further evaluation was required. This section describes this process, evaluation criteria, and properties of gate systems to form the basis for the recommended gate type for use in the Canal Atlantic Locks concept design.

7.1. General Requirements

The alternatives described in the previous section have been evaluated generally on the following criteria:

7.1.1. Compatibility in system

This criterion is based on how well the gate type integrates with other primary project features including: layout of the filling and emptying systems, water saving basins, and ship positioning system. The primary focus of this study is evaluation of the gate type for use in a double lift lock arrangement. Comments are provided when a particular gate type is better suited for a triple lift lock.

7.1.2. Fabrication

A general discussion related to the complexity and issues of fabrication is provided for each gate type. For simplicity, all gates were assumed to be of welded construction except as noted.

7.1.3. Maintenance

The maintenance facility that supports gate service is important not only in direct cost but in the intangible elements of reliability and effect on the capacity of the Canal. Both tangible and intangible elements are addressed. A general discussion of routine and preventative maintenance requirements of the mechanical operating system for each gate type is provided.

7.1.4. Operability

Operational characteristics and performance history of similar (though somewhat smaller) gates of each type are discussed.

7.1.5. Reliability

A general discussion of predicted reliability based on the complexity of gate and machinery components and on the operational history of similar systems is provided.

7.1.6. Water Use

Since continuous and future operation of the Panama Canal is dependent on a limited supply of water with increasing demand, a comparison of water usage is important. The relative water use is determine for each gate alternative and reported below.

Preliminary cost estimates are developed to include the gates, operating equipment, masonry, excavation for gate block monoliths and sills, and ancillary features such as machinery rooms and maintenance facilities. A summary of this data is presented in Paragraph 7.5, Figure B-7-4.

7.2. Miter Gate

This gate type is similar in appearance and operation to the gates currently in service in the Panama Canal. The primary difference is in the greater size and method of fabrication.

7.2.1. Compatibility in system

Of all lock gates considered for further studies, miter gates offer the most flexibility in the lock structure. There are no interferences with the layout of the filling and emptying culverts or water saving basins, and additional large recesses and structures are not required to accept the gate. It does however, increase the length of the lock chamber but shows an overall lesser cost than the other gate types finally evaluated. Approximately 40% less concrete is required to build the gate bays (all gate monoliths and sills) for a double-lift lock when compared to similar features in a double-lift lock fitted with rolling gates.

7.2.2. Fabrication

Shop-welded fabrication is anticipated with some assembly on-site. Numerous welds can be made using automated welding equipment, which generally improves quality control and reduces fabrication time. Proper joint detail selection, strict quality control requirements, and strong quality assurance during fabrication would result in a durable and minimum maintenance structure. Aside from strict dimensional tolerances, miter gates are relatively simple to fabricate and install. At this size of gate, entry into and movement inside the gate for fabrication and painting operations are very easy.

Direct-connect hydraulic cylinders are proposed to operate the gates. These systems are shipped assembled and shop tested before shipment. Fabrication is straightforward. Installation and removal of the hydraulic cylinder can be arranged such that the entire assemble rolls out on a track system to an accessible area on the back of the lock wall.

7.2.3. Maintenance

Miter gates generally require minor service every 5 years with reconditioning in dry conditions every 15 to 25 years. Generally, the anodes are replaced, gudgeon pin bushings are inspected or replaced, anchorage is inspected and serviced, and paint systems are touched-up or redone as required. This is maintenance that is normally accomplished in-place. For a gate of this size, maintenance to interior surfaces is not difficult as the size provides adequate access to all compartment areas without the need to install scaffolding or other access planking. The direct-connected type miter gate machinery has proven to be a simple system to maintain, repair and replace.

If the gate would need to be taken out of the chamber for service off-site, the miter gate would floated off the pintle by adding air into the air chambers, lifting with the Titan crane, and rotated to a horizontal position. It would be floated to a work site. If a heavy lift barge is used, the gate would be loaded horizontally on top of the barge as shown in Fig B-4-13, and transported to the work site such as Industrial Division. At this site, the barge would be positioned against the dock head and the gate would be slid off of the barge onto a work carriage as is now done with the syncrolift, see Fig B-4-15. The advantage of this operation is that all of the maintenance facilities are already in place and the operations are readily available. Access to the horizontally positioned gate would require only a little more construction of access facilities than now exist and the gate would also not be required to be accessed in a vertical position over a height of 35 m. The maintenance would be performed as is now done and not require transporting men and equipment to a remote site where repair facilities are nonexistent. The reinstallation would require the reverse of this procedure. All of these procedures are well known with the existing maintenance operations.

7.2.4. Operation

Miter gates operated quickly (opened or closed in 5-minutes) and have a proven track record for reliability on locks throughout the world and in the Panama Canal. In a seismic event, miter gates would maintain a mitered position with differential head as the static head exceeds the opposing hydrodynamic head. The direct connected machinery is very common in Europe and is becoming common in the U.S., where it is now used on lock widths of 17.1 m, 25.6 m, and 33.5 m. This machinery provides smooth, reliable, and controlled gate movement. This type of system has been used on Corps 33.5 m wide locks since the 1960's and has proven to be very reliable. Direct connect hydraulic cylinders of size comparable to those proposed for the Third Lane are in operation on civil works projects owned and operated by the Corps of Engineers.

7.2.5. Reliability

Miter gates have a long proven track record on the existing Canal system and on projects around the world. Although the miter gate design for the Atlantic Locks Project is larger than any currently in service, similar performance is expected. The direct-connected miter gate machinery has been very successful and has proven to be reliable and trouble free.

7.2.6. Water Use

The water surface shaded in Figure B-7-1 is raised and lowered during lockage of normal traffic when double miter gates are used. This area is directly related to water usage. For the miter gate alternative, this surface is 32 140 m^{2} and accounts for the gate recesses and increased length of lock chamber The miter gate configuration would use approximately 0.6 % less water than is needed for use with rolling gates.

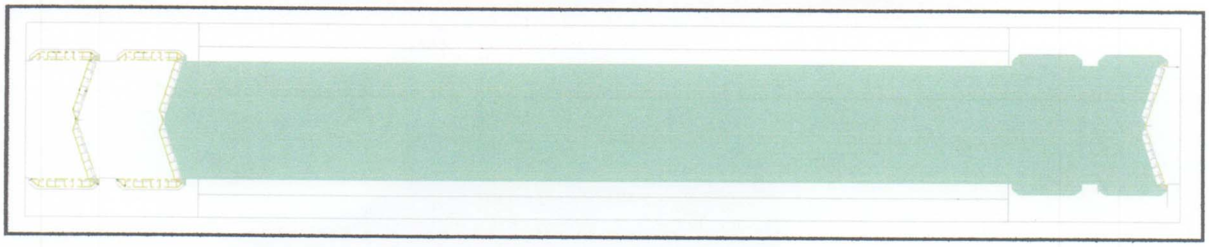


Figure B-7-1 Miter Gate Lockage Surface Area

7.2.7. Advantage and Disadvantages

7.2.7.1. Advantages

i. Lowest first cost, lightest gate weight.
ii. Long record of dependable operation
iii. Would be designed to minimize maintenance.

iv. Minimizes the need for submerged mechanical components.
v. Would be made readily removable for repairs.
vi. Great potential for interchanging gates without unwatering.
vii. Would be designed for floating transport without pontoons.
viii. Off-site fabrication is feasible.
ix. Maintenance requirements are similar to existing operations.

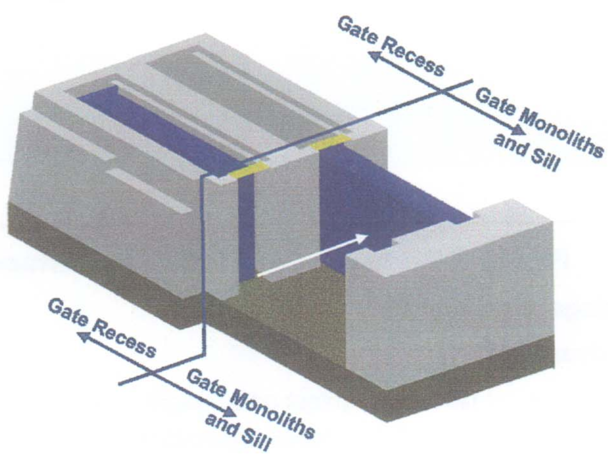
7.2.7.2. Disadvantages

i. Cannot be closed in flowing water.
ii. Lock closure may be required for maintenance of sills and possibly gates if not made removable in the wet. If it is desired to not dewater the locks for maintenance of underwater features, a caisson or other such structure can be used to provide this capability, however, the lock would have to be closed while this maintenance is being performed.

7.3. Rolling Gates

7.3.1. Compatibility in system

Hydraulic engineering studies indicate that rolling gates are compatible with feasible arrangements of filling and emptying systems considered. There would be some additional costs to lower the culverts in proximity to the Atlantic gate bay. Details would need to be developed transition the elevation of the culvert over several monoliths. The transition would complicate construction. The large additional gate recesses, equipment platform, and need to deliver materials for maintenance creates access issues to the top of lock walls, especially with the use of water saving basins. Although safety handrails can be provided, differences in elevation present additional opportunities for falls. A significant amount of additional excavation and concrete volume is required to build the rolling gate alternative compared with a miter gate solution. Over 1,000, 000m^3 of concrete is needed to build three gate recess structures, gate monoliths, and sills for a double-lift lock with rolling gates. This is a significant increase compared to 600,000m^3 of concrete needed to build all gate monoliths and sills for a double-lift lock equipped with miter gates.



7.3.2. Fabrication

Rolling gates are heavy compared to other gate types considered in this study. The structure is made up of heavy rolled sections and plate elements that are necessary to restrain the large lift heights. The rolling process of these large shapes presents potential for manufacturing defects and difficulty in fabricated connections. There are numerous complex joints formed between these heavy structural shapes that would require placement of heavy weldments. Nearly all of the joints would require pre- and post- heat treatment to control fatigue and fracture issues. With additional transportation provisions, the gates could be fabricated off-site and floated in. Shop fabrication is preferred to produce a high quality gate that is necessary to have the reliability in this service. The total fabricated weight of all rolling gates for a double-lift lock including embedded metals, carriage systems, bearing plates, etc, is 27,000 t. The total fabricated weight of all miter gates for a double-lift lock including embedded A-frames, contact blocks, pintle ball, sill, etc, is 19,500 t. Rolling gate structures have a disadvantage from a material usage perspective when compared to miter gates. This is due to the structural efficiency of the miter gates' arching action and the inefficiency of the simple beam action of the rolling gate design. Should a rolling gate be proportioned to float without ballast, additional inefficiencies are built into the design because a wider structure would be required for floating stability that results in additional material usage.

7.3.3. Maintenance

The submerged mechanical features associated with the rolling gate will require regular maintenance. The gate can be serviced in the dry within the gate recess. However, it is recognized that this is an increased demand to the efforts for regular maintenance resources. The rolling gate has significant surface area to maintain with a paint system. Painting would be expensive due to complications associated with general access, the height of this gate requiring much scaffolding and interior access, numerous members with surfaces that are difficult to reach (back side of W-Shapes) and confinement working within a recessed pit. A chain drive system would require periodic maintenance in the form of lubrication and a cable drive system would require frequent adjustment to accommodate moving this large weight. Roller bearings would need to be lubricated on a five-year cycle.

Minor maintenance of the rolling gates would be accomplished in the dry and necessitate the setting of the dewatering caisson across the gate recess opening. This would require the use of the ACP floating plant to be on site for setting the caisson and dewatering of the recess much as is needed for maintenance of the miter gates. The shops would also be needed to provide maintenance support as for any on-site maintenance. In addition, a crane would be needed to remove the upper wagon for maintenance and to provide access for placing scaffolding and equipment inside the recess. Due to the configuration of having water saving basins with the locks, access to these recess areas is a concern as access along the top of walls is limited. Access to the back of the recess would place the crane at a distance below the top of wall and present a need for a high lift and pass over the gate operations building.

Once inside the recess, the scaffolding and equipment would need to be passed over the 61 m length of the gate and have a need to scaffold 35 m vertically along the exterior of the gate at multiple sides. Access to the interior of the gate would be significantly more difficult with the air chambers located in the gate. Working conditions would be humid and wet and be much different from working on the gates in the controlled conditions at Industrial Division. Access at the side of the gates would not be possible because of the crane rails required for the upper wheeled wagon and configuration of the structures.

At the heavier weight, the rolling gate would not be readily lifted with a crane. The long span and space truss design render the rolling gate more susceptible to damage from impacts of the large Post-Panamax vessels and have more risk in this application. The single gate as opposed to the two leaves of the miter gates would not be removed easily from the chamber with damage from a serious impact. Problems with the lower wheeled wagon may cause difficulties in servicing the gate if it is not moveable.

Major maintenance of the rolling gate would have to be accomplished at the dry dock in Balboa and need to be serviced in the vertical position. This involves floating the gate and transporting it vertically for service. As this is not an ACP facility, service facilities may be limited. The ACP fleet would have to be positioned at the dry dock to provide adequate service facilities.

7.3.4. Operation

Rolling gates in service operate smoothly with ramp up and ramp down cycles. However, most existing rolling gates are used on low lift locks that require a gate design capable of handling a small differential head and reverse head loading from both upstream and downstream directions. For this reason, rolling gates are ideal for locks subject to tidal changes and storm surges. Providing smooth, reliable operation of the massive rolling gate required for a large high lift lock would be more difficult. While positional accuracy would not be an issue, accelerating and decelerating a massive rolling gate would require precision feedback and control. Time of operation would be about 5 minutes. Ship impacts or obstructions can "lock" the wheels and make moving the gate impossible.

7.3.5. Reliability

Rolling gates have traditionally been driven by either a wire rope and drum type drive system or by a chain type drive system. A disadvantage of the wire rope and drum mechanism is that the ropes tend to lose tension with use, thereby requiring frequent inspection and adjustment. Also, because wire rope can break unexpectedly, periodic replacement is usually required as preventative maintenance. Rolling chain type drive systems are more reliable, so long as lubrication and maintenance requirements are followed. With either system, the submerged wheel bearings on the gate must also be maintained properly to insure reliable operation.

7.3.6. Water Use

The water surface shaded in Figure B-7-2 is raised and lowered during lockage of normal traffic when double gates are used. This area is directly related to water usage. For the rolling gate alternative, this surface is 32 340 m^2. The rolling gate configuration would use approximately 0.6 % more water than for miter gates considering the increased chamber length needed between gates and water in the recesses.

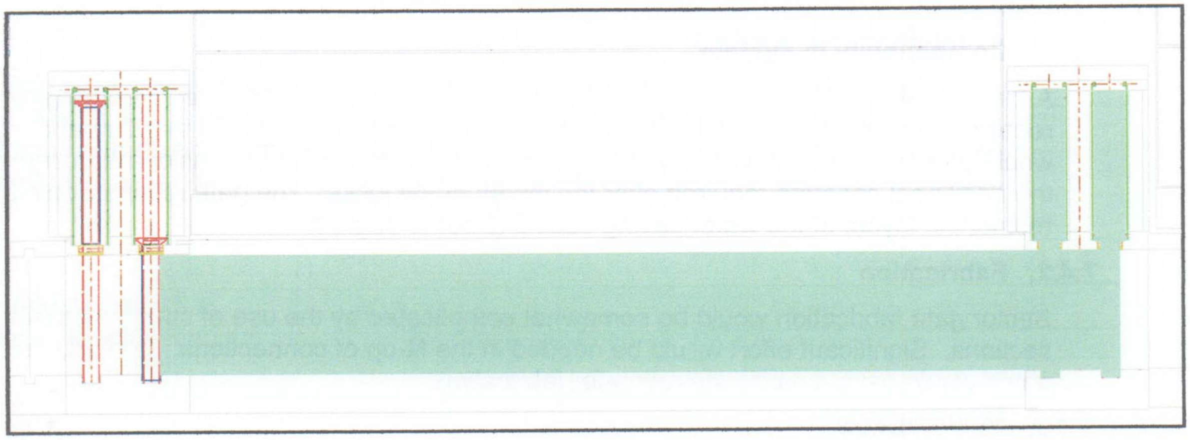


Figure B-7-2 Rolling Gate Lockage Surface Area

7.3.7. Advantage and Disadvantages

7.3.7.1. Advantages

i. Gates can be readily inspected and minor service performed in the gate recess. Tall but narrow caissons are required to bulkhead the gate recess before dewatering can occur.
ii. Fabrication does not require significantly tight dimensional tolerance.
iii. Easy to design.

7.3.7.2. Disadvantages

i. Gate can easily become an obstacle in the lock chamber should the gate be struck by a vessel. This would close the third lane until the gate is repaired to a point where it can be moved into the gate recess. Repair of a gate outside the gate recess would be difficult.
ii. Should the floatation chambers fail to function and the gate drop to the chamber or recess floor, the large weight would be very difficult to raise and move if necessary. Equipment of the size needed is not available in Panama. Access to the recesses or chamber would be near impossible for this large equipment.
iii. Fatigue and fracture is a concern due to the large number of thick plate connections used in the design.
iv. Maintenance occurs below grade in a pit where access and clearances are limited. Access to the recesses is difficult for personnel and equipment.
v. Periodic preventative maintenance required for chain or wire rope driven components and submerged wheel assemblies.
vi. Requires construction of additional large recess.
vii. Layout of the water saving basins is somewhat limited to width increases.
viii. Safety issues associated with more exposure to differential grade elevations.

7.4. Sector Gate

7.4.1. Compatibility in system

A long bridge would be required for the ship positioning system to travel over the gate recess. The structure depth of this bridge would be approximately 2.7 to 3.0 m and would control the top of wall elevation. This would be a significant consideration due to the additional concrete required over the length of the locks. The gate recess would also restrict the layout of the water saving basins to some degree.

7.4.2. Fabrication

Sector gate fabrication would be somewhat complicated by the use of structural tube sections. Significant effort would be needed in the fit-up of connections. Overall, sector gates would have a moderate rating for fabrication.

7.4.3. Maintenance

Sector gates cannot be serviced in their gate recess. They would require removal and transport to a maintenance facility for maintenance. Unless they are design to float, this would be an unrealistic maintenance procedure. The rack and pinion mechanism is mainly used on lock gates or gates that have a high frequency of use. Once the rack and pinion is aligned there would be no further adjustments required until wear in the gate's hinge and pintle eventually manifests itself in the tightening of the gear mesh, but the gate bushings are usually replaced before this becomes a problem. The exposed rack and pinion gears would also require frequent lubrication. This would be the most difficulty of the three alternatives to service on or off-site.

7.4.4. Operation

The operating equipment used to open and close the gates has a history of good performance. No significant issues exist in this area.

7.4.5. Reliability

The rack and pinion type sector gate machinery has proven to be reliable and trouble free.

7.4.6. Water Use

The water surface shaded in Figure B-7-3 is raised and lowered during lockage of normal traffic when double sector gates are used. This area is directly related to water usage. For the sector gate alternative, this surface is 36 177 m^2 and includes increased chamber length and recess areas. The sector gate configuration would use approximately 12% more water than that for rolling gates.

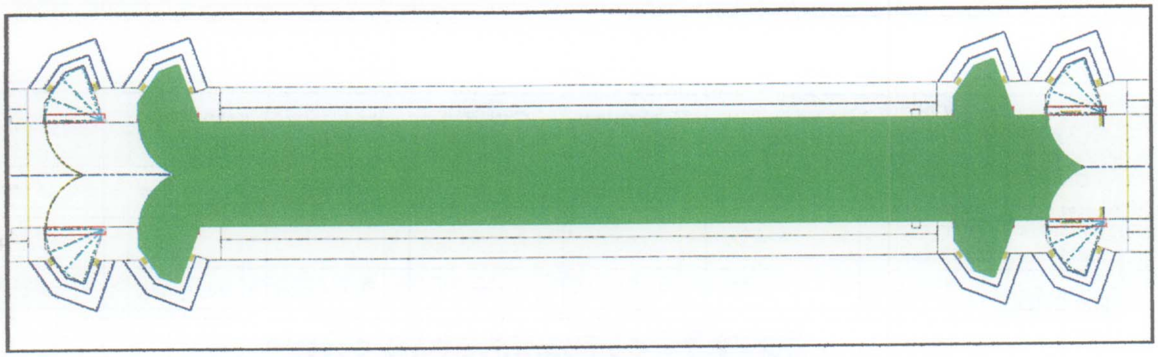


Figure B-7-3 Sector Gate Lockage surface area

7.4.7. Advantage and Disadvantages

7.4.7.1. Advantages

i. The major advantage of the sector gate, similar to the tainter gate, is that it can be operated against a head. Sector gates can open or close under the head differential between upper and lower normal operating pool elevations. It can also be designed to operate under minor reverse heads.

ii. The gates are more durable than miter gates. A slight ship impact, from any direction, would not cause the gates to open, as is the case with a reverse force on a miter gate.

iii. The sector gates can be used for end-filling the lock chamber, however, this is not recommended for use in the Panama Locks.

iv. Sector gates can be closed if temporally opened during an earthquake event.

7.4.7.2. Disadvantages

i. The major disadvantage would be the required large recess monolith. A bridge would be required to span the recess as a provision for ship positioning equipment. The bridge depth would be approximately 2.7 to 3 m, which raises the top of wall elevation approximately 1.2 m higher than the miter gate and rolling gate alternatives.

ii. Except for general lubrication, the gates cannot be serviced in place.

7.5. Cost Analysis

Preliminary cost estimates have been prepared to a level of accuracy commensurate with screening level studies. Any relative error is assumed to be common to all alternatives and does not affect the ranking of alternatives based on cost.

Gate Type	Maximum Gate Weight, t	Cost for All Lock Gates	Mechanical Systems	Gate Bay Construction	Excavation Cost	Associated Features	Total Cost
Miter Gates (2 Leafs)	1737 Ea, 3475 Set	$ 127,022,838	$ 10,800,000	$ 57,725,100	$ 6,918,985	$ 6,732,514	$ 209,199,436
Sector Gates (2 Leafs)	2200 Ea 4400 Set	$ 153,016,482	$ 3,600,000	$ 178,753,500	$ 5,312,000	$ 9,792,514	$ 350,474,495
Rolling Gates	5100 Ea	$ 178,378,304	$ 9,000,000	$ 152,397,196	$ 10,505,650	$ 3,852,975	$ 354,134,125

Figure B-7-4 Comparative Gate Costs

7.6. Final Alternative Rating and Ranking

Based on a design of each alternative which included the structural design of gates, operating machinery, concrete gate housing and support structures, excavation, and associated features and progressive screening studies that have been conducted, the three most probable gate types were reevaluated with the results shown in Figure B-7-5, Final Ranking of Gate Alternatives. A scoring of 3 indicates the highest rating and 1 indicates the least favorable rating. Based on this rating miter gates are the preferred lock gate. The miter gate lock gate solution has the least first cost and is the easiest to maintain.

Evaluation Criteria and Rating								
Feature Alternative	First Cost	Ease and Frequency of Maintenance	Risk to Extented Navigation Closure Resulting from Significant Accidents	Access for Crossing Over Lock	History of Reliable Service or Precedent	Compatibility with Water Saving Basin and Filling and Emptying System	Water Conservation	WEIGHTED EVALUATION
Miter Gates	3	3	3	2	3	3	3	2.86
Rolling Gate	1	2	2	3	3	3	3	2.43
Sector Gates	2	1	2	1	1	2	1	1.43
Procedure included ranking the individual alternative by individual criteria to consider relative comparison. 3 is the highest, 1 is the lowest								

Figure B-7-5 Final Ranking of Gate Alternatives.

8. RECOMMENDATIONS

A technical evaluation of three lock gate type configurations based on alternate lock gate systems was performed. The following criterion was used in the evaluation process: compatibility in the lock system, ease of fabrication, maintenance, operability, reliability, and water use. The results of this study are conclusive that mitering lock gates are preferred. This recommendation is based on the following observations:

- Miter gates are the least first cost solution, even when considering provisions for new maintenance infrastructure.
- Miter gates are more compatible with the layout of the filling and emptying systems as well as with the use of water saving basins.
- Water usage is not a significant issue when selecting between miter gates and rolling gates as lock systems equipped with either gate use about the same water volume. Miter gates use approximately 0.6% less water than rolling gates. Although the lock chamber length is increased when using mitering lock gates, the recess is very compact and minimizes ineffective space. The length of a lock chamber equipped with rolling gates is minimized but the recess required to house the gate is very large and offsets water savings associated with a shorter chamber length.
- Miter gates use simple operating equipment that requires minimal maintenance.
- Miter gates have minimal underwater mechanical systems.
- Miter gates can be more easily removed from the lock chamber if damaged by ship impact. Recovery from significant damage to a rolling gate may require extended lock closures and loss of revenue.
- There are many intangible benefits associated with using common gate types on the lock system as a whole. Maintenance staffs are familiar with operating and maintenance procedures.

Additional recommendation for the miter gates design include:

- Use a double skin plate system on the miter gates.
- Use the same gate height for the center gates and lower gates.
- Develop in the wet gate installation and removal procedures.
- Use direct connect hydraulic cylinders as operating equipment.
- Conduct life cycle cost analysis of all installation options.
- Develop considerations for emergency response situations.

9. REFERENCES

- 17-19 April 2002 Initial Project Team Meeting Summary Report
- "The Panama Canal, The Third Locks Project, Final Report on Modified Third Locks Project, Part II – Design, Chapter 8 – Lock Gates", August 1943, Department of Operation and Maintenance, Special Engineering Division, Balboa Heights, Canal Zone.
- ER 1110-2-1150 Engineering and Design for Civil Works Projects, 31 Aug. 1999
- ETL 1110-2-355 Structural Analysis and Design of U-Frame Lock Monoliths, 31 Dec. 1993
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- EM 1110-2-2105 Design of Hydraulic Steel Structures, Change 1, 31 May 1994
- EM 1110-2-2200 Gravity Dam Design, 30 June, 1995
- EM 1110-2-2502 Retaining and Flood Walls, 29 Sept. 1989
- EM 1110-2-2602 Planning and Design of Navigation Locks, 30 Sept. 1995
- EM 1110-2-2703 Lock Gates and Operating Equipment, 30 June 1994
- AASHTO Standard Specifications for Highway Bridges
- ACI 318/318R (1999) Building Code Requirements for Structural Concrete and Commentary
- ACI 357R-84 Guide for Design and Construction of Fixed Offshore Concrete Structures (Reapproved 1997)
- ER 1110-2-1806 – Earthquake Design and Evaluation for Civil Works Projects
- ITL-92-11 – The Seismic Design of Waterfront Retaining Structures

10. ATTACHMENT 1 – SUMMARY OF QUANTITIES

The quantities provided are for all gates and gate bay monoliths for a double-lift lock. For this screening level investigation all other gate quantities are pro-rated based on the height ratio.

Note: The $90/m³ unit price for concrete is an average of $60/m³ for roller compacted (RCC) and 120/m³ for cast-in-place (CIP) concrete. An estimate of 50-50 split of RCC and CIP concrete was used for pricing purposes.

Miter Gate Alternative

FEATURE	ITEM	UNIT	QUANTITY	UNIT COST	TOTAL
Miter Gates & Embedded Metals	Miter Gates Fabricated (incl. Sill embedded metals, quion $ miter blocks, etc), Delivered, and Installed	t	19,542	$ 6,500.00	**$127,022,838**
Mechanical	Operating Machinery	ea	12	$ 900,000.00	**$10,800,000**
Masonry Construction	Mass Concrete (1/2 CIP, 1/2 RCC)	m3	587,390	$ 90.00	$52,865,100
	Reinforcing	kg	5,400,000	$ 0.90	$4,860,000
				Sub Total	**$57,725,100**
Excavation	Rock Excavation	m3	864,873	$ 8.00	**$6,918,985**
Associated Features - Maintenance Facility	Precast Concrete	m3	326	$ 400.00	$130,560
	Tremie Concrete	m3	227	$ 300.00	$68,000
	CIP Concrete	m3	765	$ 150.00	$114,750
	Soil Excavation	m3	2,901	$ 10.00	$29,013
	8" Concrete Paving	m3	600	$ 300.00	$180,000
	Reinforcing	kg	61,223	$ 1.25	$76,528
	Steel Piling (100,000 kg)	m	1,000	$ 250.00	$250,000
	Hoist Systems	ea	1	$ 300,000.00	$300,000
	Cradle (Str. Steel)	t	194	$ 8,000.00	$1,551,662
Recess Covers & Bridge	Structural Steel	t	576	$ 7,000.00	$4,032,000
				Sub Total	**$6,732,514**
				TOTAL	**$209,199,436**

Roller Gate Alternative

FEATURE	ITEM	UNIT	QUANTITY	UNIT COST	TOTAL
Roller Gates & Embedded Metals	Roller Gate (Str. Steel), Track, Rollers, Carriage, etc	t	27,212	$ 6,500.00	$176,878,304
	Lead Ballast	t	2,500	$ 600.00	$1,500,000
				Sub Total	**$178,378,304**
Mechanical	Operating Machinery	ea	6	$ 1,500,000.00	**$9,000,000**
Masonry Construction	Mass Concrete (1/2 CIP, 1/2 RCC)	m3	1,051,890	$ 90.00	$94,670,137
	Reinforcing	kg	64,141,176	$ 0.90	$57,727,059
				Sub Total	**$152,397,196**
Excavation	Rock	m3	1,313,206	$ 8.00	**$10,505,650**
Associated Features - Bridge over Recess	Bridge Superstructure	ea	6	$ 200,000.00	$1,200,000
Associated Features - Bulkheads for Recess	Str. Steel	kg	408	$ 6,500.00	$2,652,975
				Sub Total	**$3,852,975**
				Total	**$354,134,125**

Sector Gate Alternative					
Sector Gates	**ITEM**	**UNIT**	**QUANTITY**	**UNIT COST**	**TOTAL**
6 Sector Gates & Embedded Metals	Sector Gate (Str. Steel), Rollers, Track, etc	t	23,541	$ 6,500.00	**$153,016,482**
Mechanical	Operating Machinery	ea	12	$ 300,000.00	**$3,600,000**
Gate Bay Masonry Construction	Mass Concrete (1/2 CIP, 1/2 RCC)	m3	1,007,000	$ 90.00	$90,630,000
	Reinforcing	kg	97,915,000	$ 0.90	$88,123,500
				Sub Total	**$178,753,500**
Excavation	Rock	m3	664,000	$ 8.00	**$5,312,000**
Associated Features - Maintenance Facility	Precast Concrete	m3	326	$ 400.00	$130,560
	Tremie Concrete	m3	227	$ 300.00	$68,000
	CIP Concrete	m3	765	$ 150.00	$114,750
	Soil Excavation	m3	2,901	$ 10.00	$29,013
	8" Concrete Paving	m3	600	$ 120.00	$72,000
	Reinforcing	kg	61,223	$ 1.25	$76,528
	Steel Piling (100,000 kg)	m	1,000	$ 250.00	$250,000
	Mechanical Systems	ea	1	$ 300,000.00	$300,000
	Cradle (Str. Steel)	t	194	$ 8,000.00	$1,551,662
Associated Feature - Bridge over Recess	Bridge Structure	ea	12	$ 600,000.00	$7,200,000
				Sub Total	**$9,792,514**
				Total	**$350,474,495**

11. ATTACHMENT 2 – REPORT PLATES

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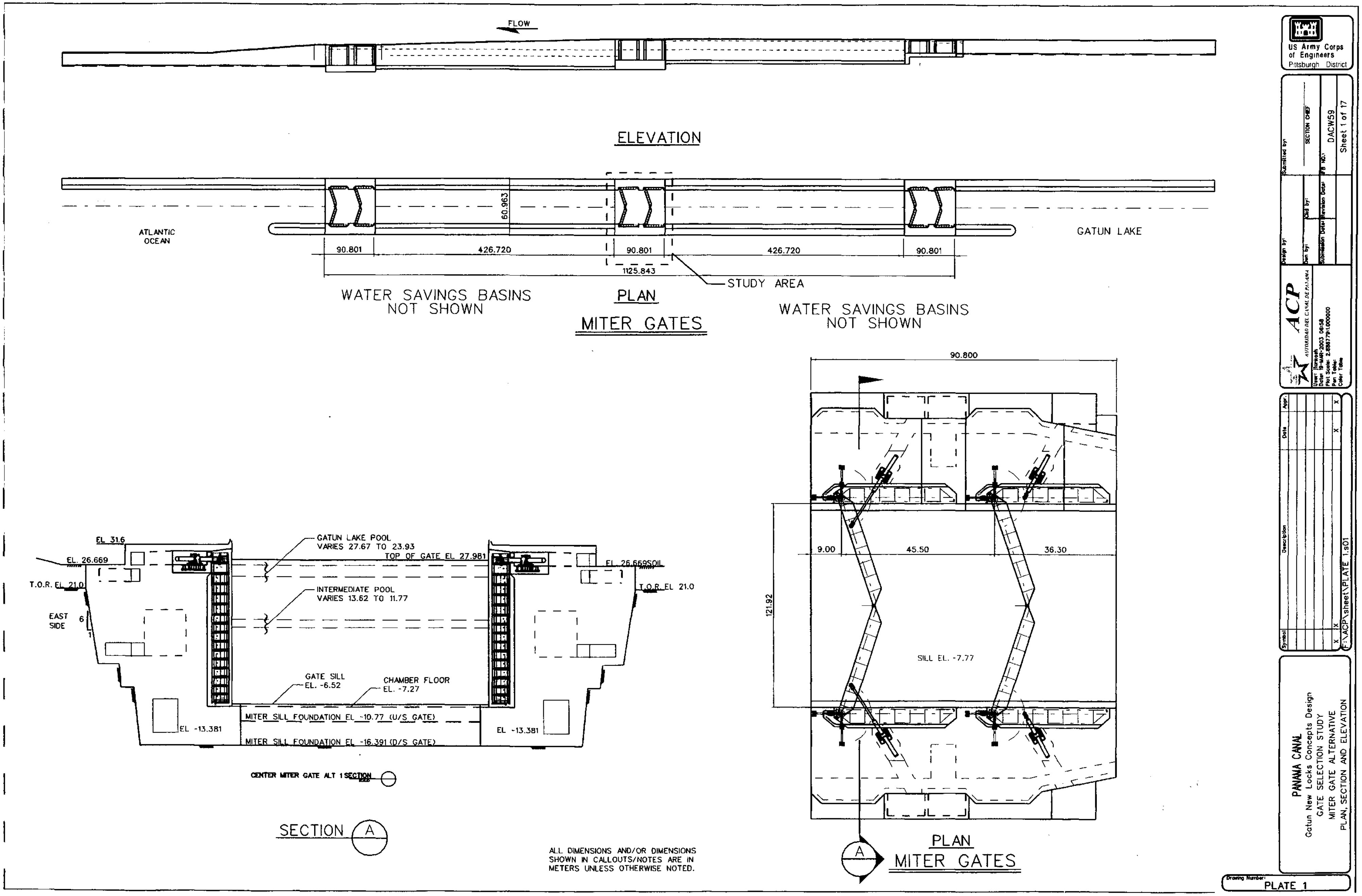
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TOP OF GATE EL 27.981
EL 26.669SOIL
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T.O.R. EL 21.0
INTERMEDIATE POOL VARIES 13.62 TO 11.77
EAST SIDE
GATE SILL EL. -6.52
CHAMBER FLOOR EL. -7.27
MITER SILL FOUNDATION EL -10.77 (U/S GATE)
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Sheet 1 of 17
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Gatun New Locks Concepts Design
GATE SELECTION STUDY
MITER GATE ALTERNATIVE
PLAN, SECTION AND ELEVATION
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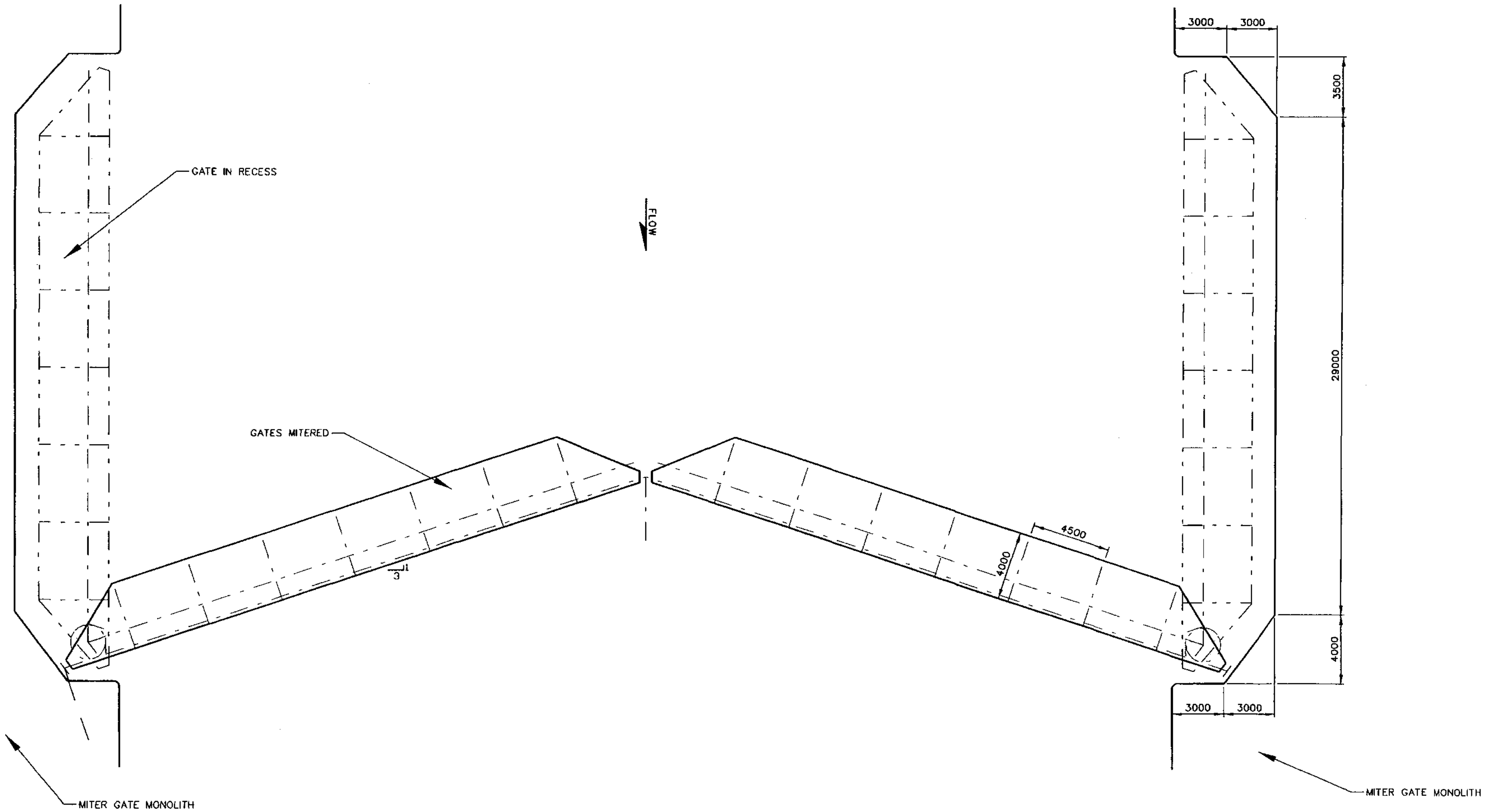
PLATE 1



PLAN

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Pittsburgh District

Submitted by:
SECTION CHIEF

FB NO.: DACW59
Sheet 2 of 17

Design by:
Dwn by:
Ckd by:
Submission Date:
Revision Date:

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Gatun New Locks Concept Design
GATE SELECTION STUDY
MITER GATE ALTERNATIVE
PLAN

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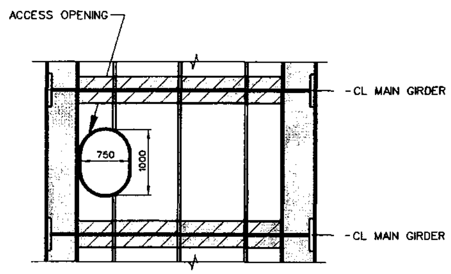
PLATE 2



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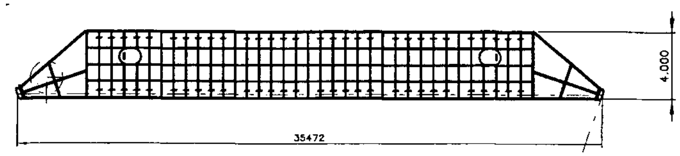

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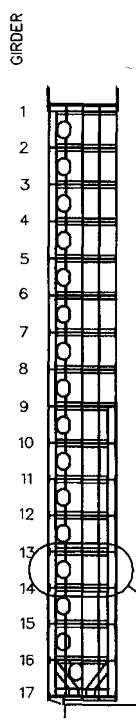

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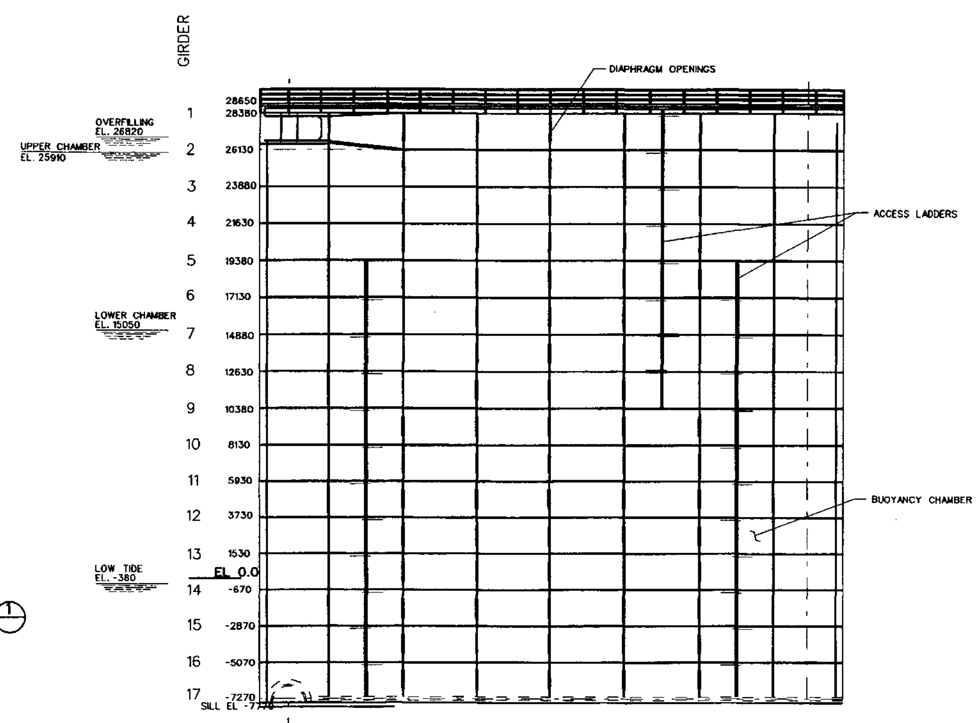

SECTION C

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DACW59
Sheet 3 of 17

PANAMA CANAL
Gatun New Locks Concept Design
GATE SELECTION STUDY
MITER GATE ALTERNATIVE
SECTIONS & DETAIL

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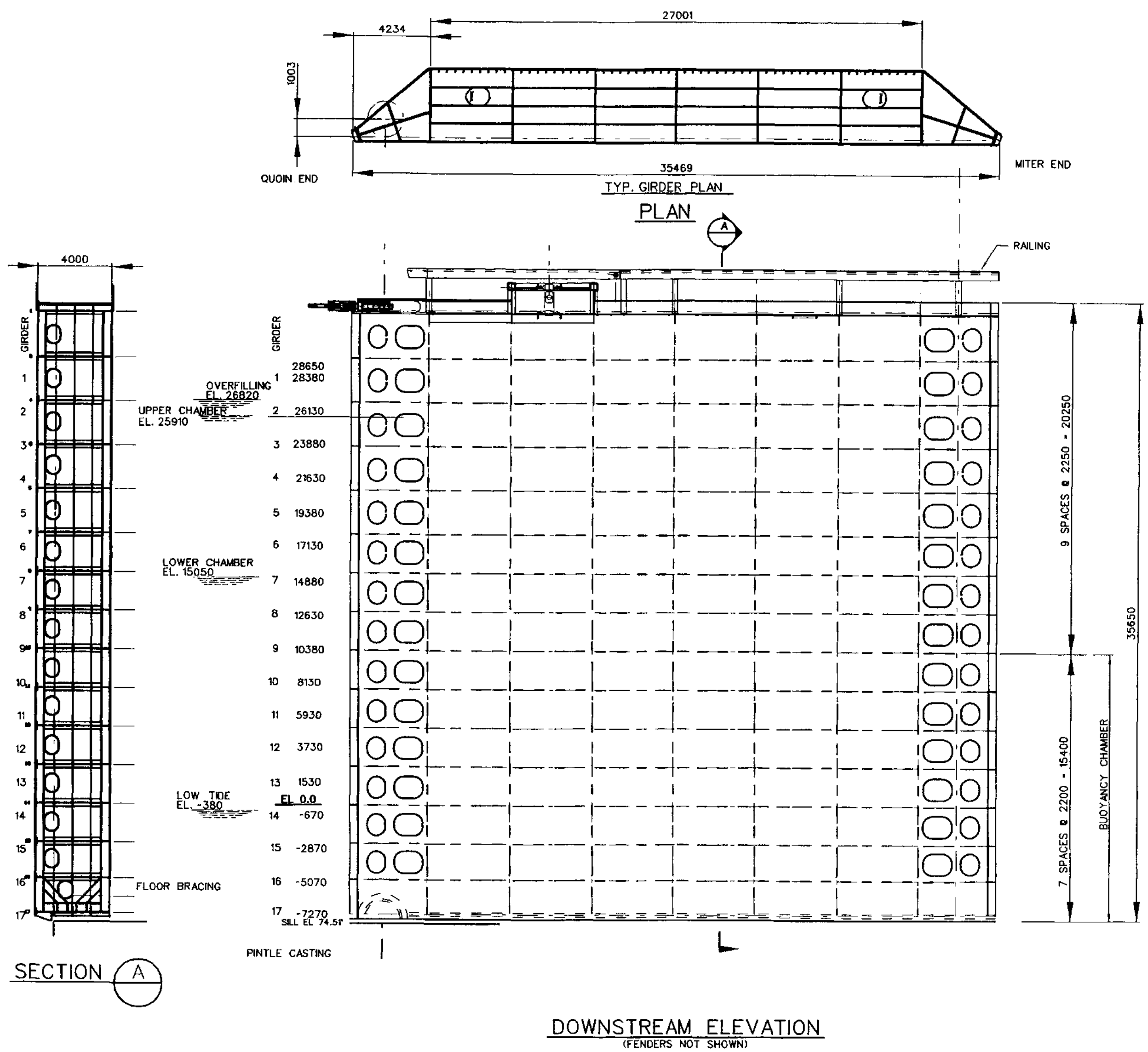
PLATE 3



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Sheet 4 of 17

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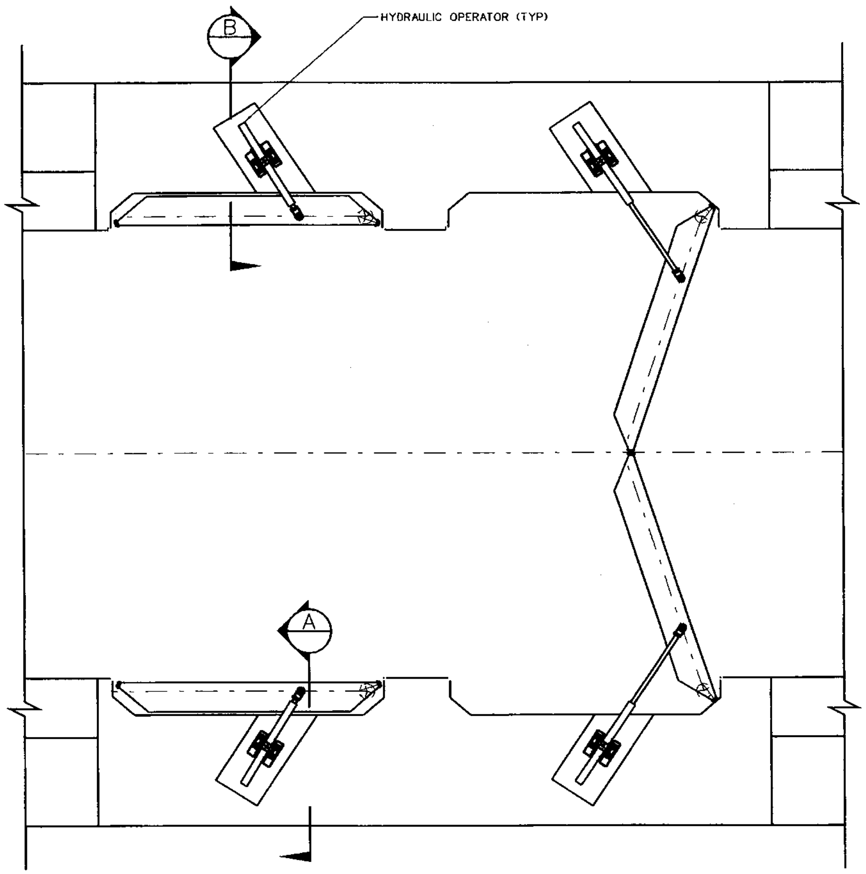
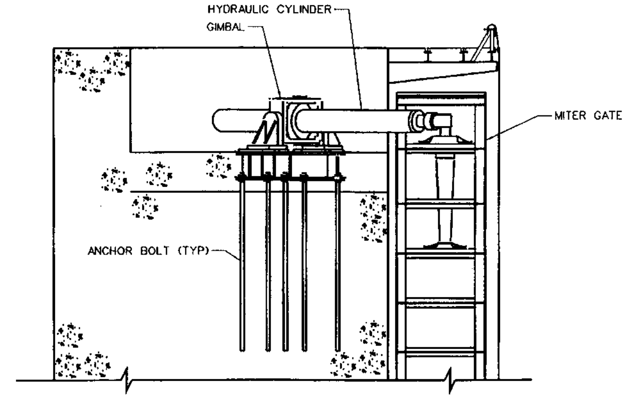
PANAMA CANAL
Gatun New Locks Concept Design
GATE SELECTION STUDY
MITER GATE ALTERNATIVE
DOWNSTREAM ELEVATION, PLAN & SECTION

PLATE 4



MITER GATE MACHINERY ARRANGEMENT



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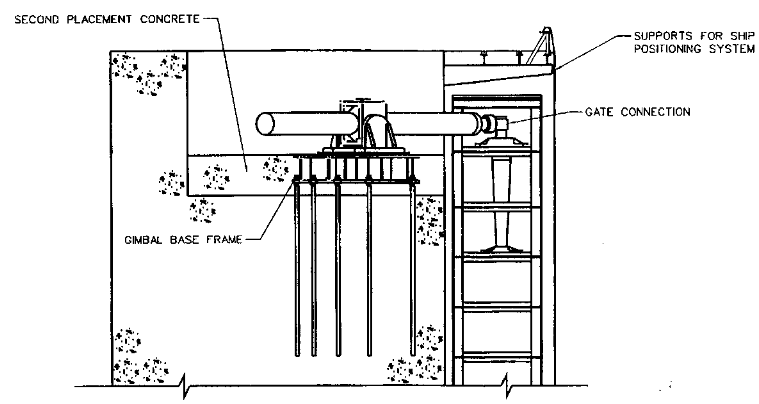

SECTION B

US Army Corps of Engineers
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Sheet 5 of 17

PANAMA CANAL
ATLANTIC LOCKS CONCEPT DESIGN
MITER GATE MACHINERY
DIRECT CONNECT HYDRAULIC CYLINDER
PLAN AND SECTION

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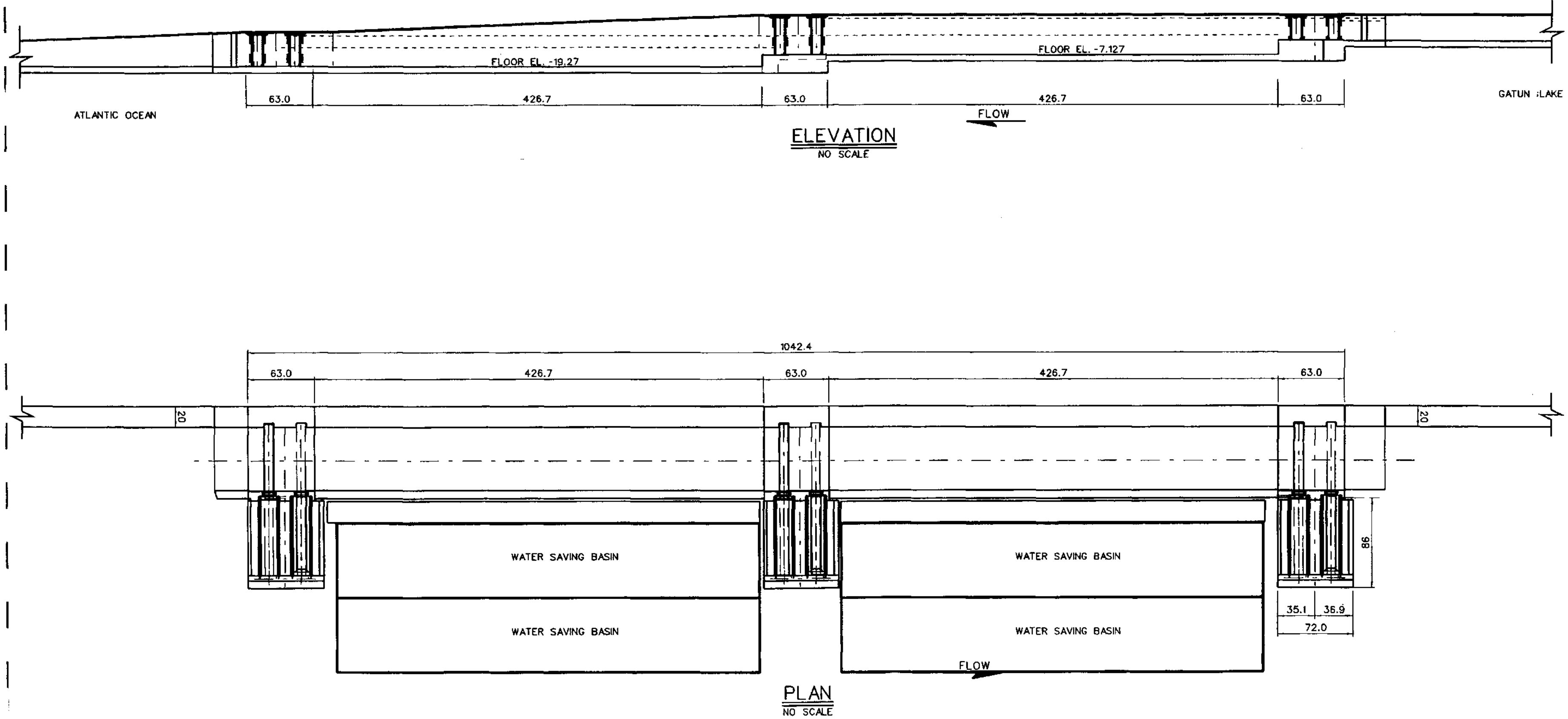
PLATE 5



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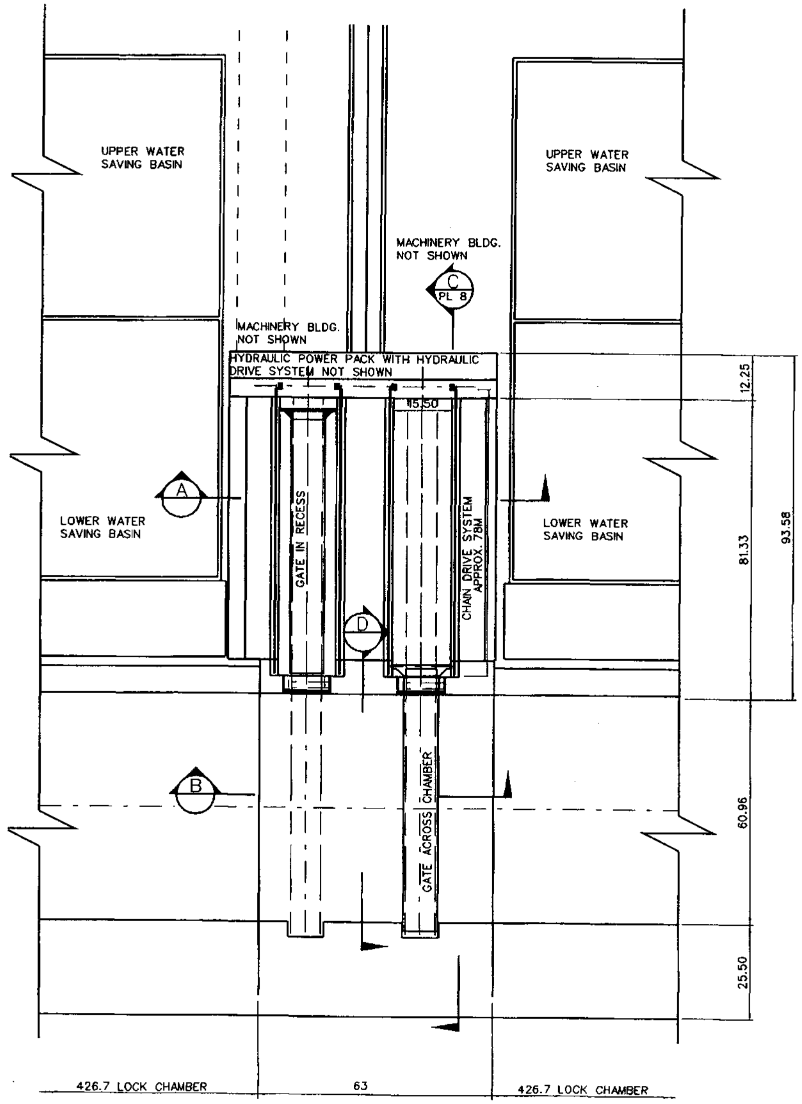
AUTORIDAD DEL CANAL DE PANAMA

Sheet 6 of 17

PANAMA CANAL
ATLANTIC LOCKS CONCEPT DESIGN
GATE SELECTION STUDY
ROLLER GATE ALTERNATIVE
PLAN & ELEVATION

Drawing Number: PLATE 6



ROLLER GATE ARRANGEMENT
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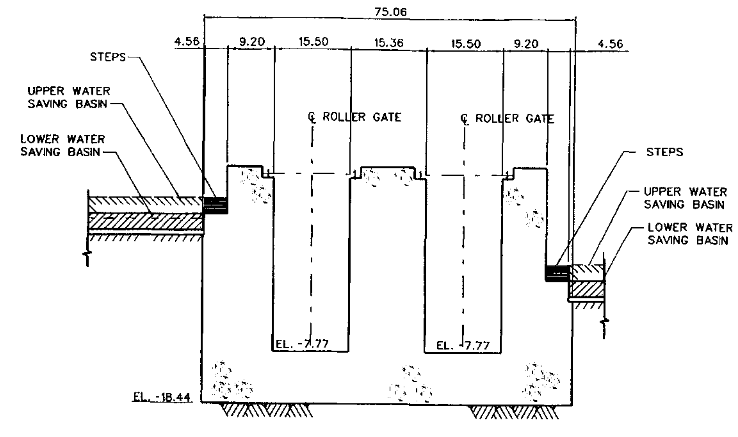

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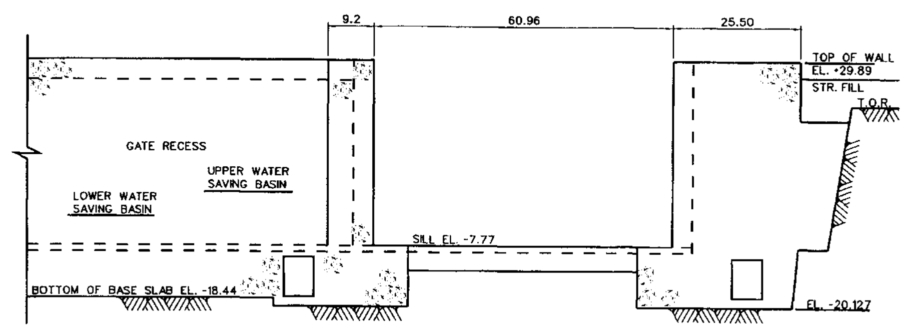

SECTION D
GATE MONOLITH

NOTE: GATE, BRIDGE AND DRIVE SYSTEM NOT SHOWN FOR CLARITY

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Date: 18-MAR-2003

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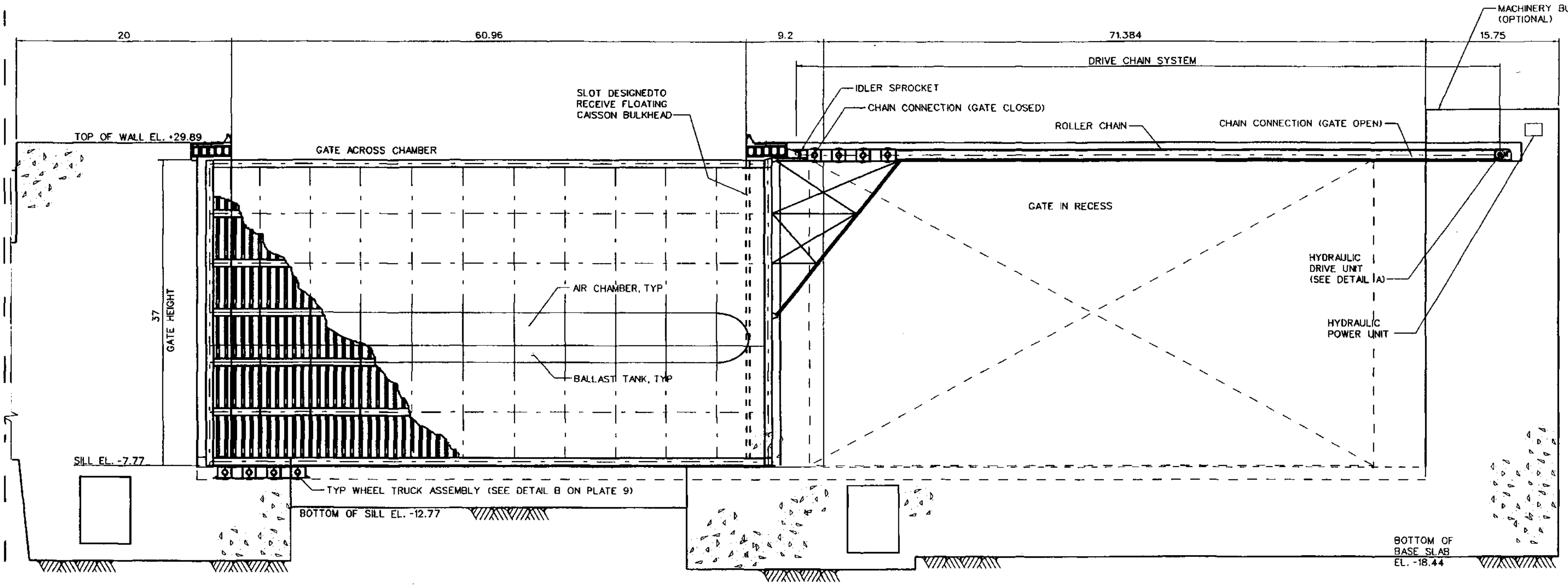
PANAMA CANAL
ATLANTIC LOCKS CONCEPT DESIGN
GATE SELECTION STUDY
ROLLER GATE ALTERNATIVE
ARRANGEMENT & SECTION

Drawing Number:
PLATE 7



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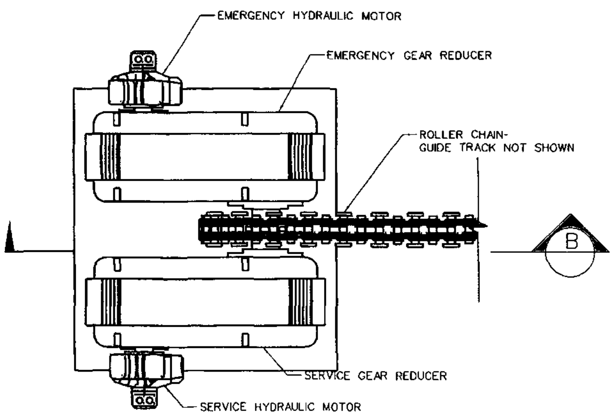

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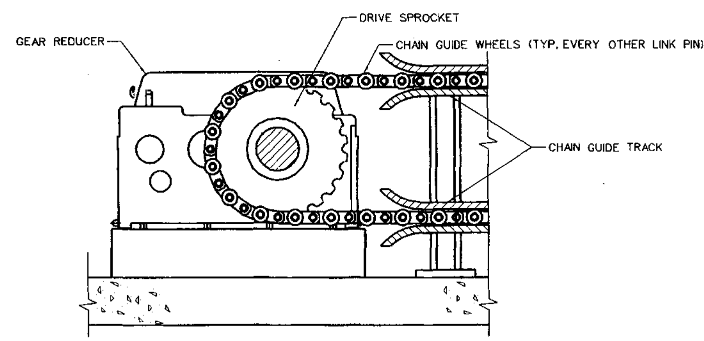

SECTION B
(ELEVATION)
NO SCALE

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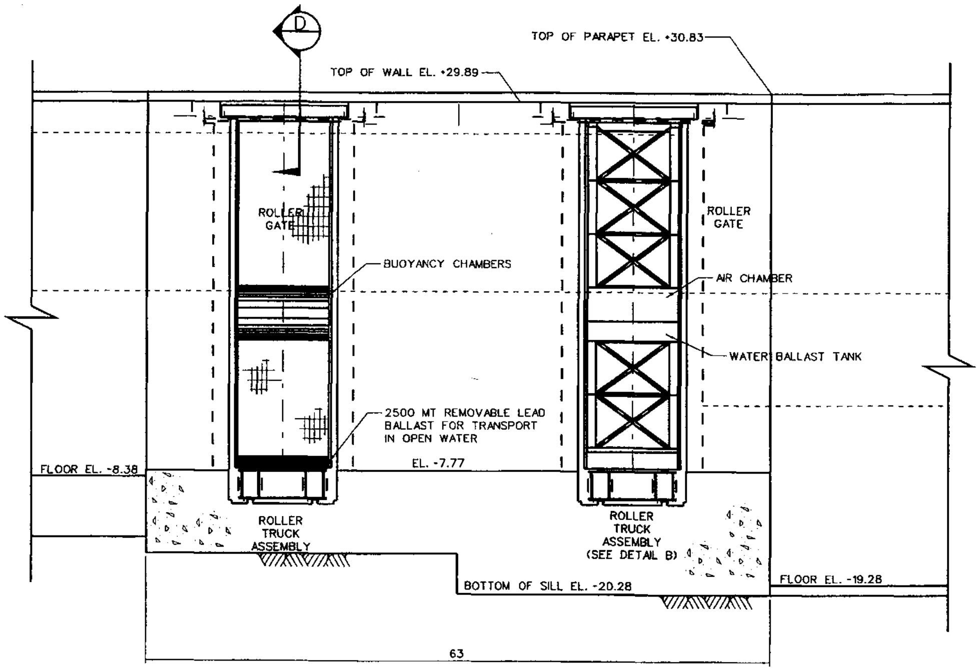
PANAMA CANAL
ATLANTIC LOCKS CONCEPT DESIGN
GATE SELECTION STUDY
ROLLER GATE ALTERNATIVE
SECTION

PLATE 8



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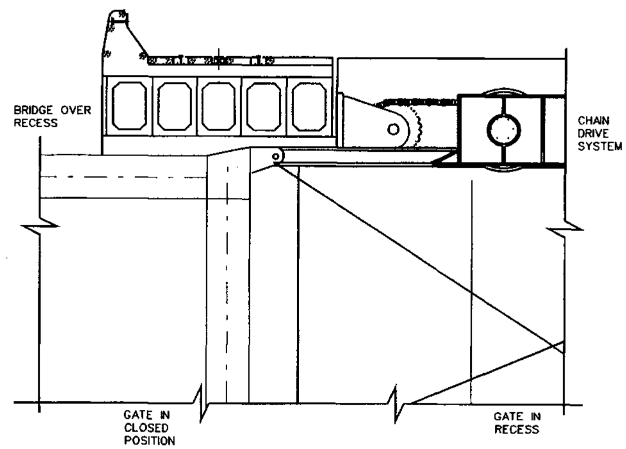

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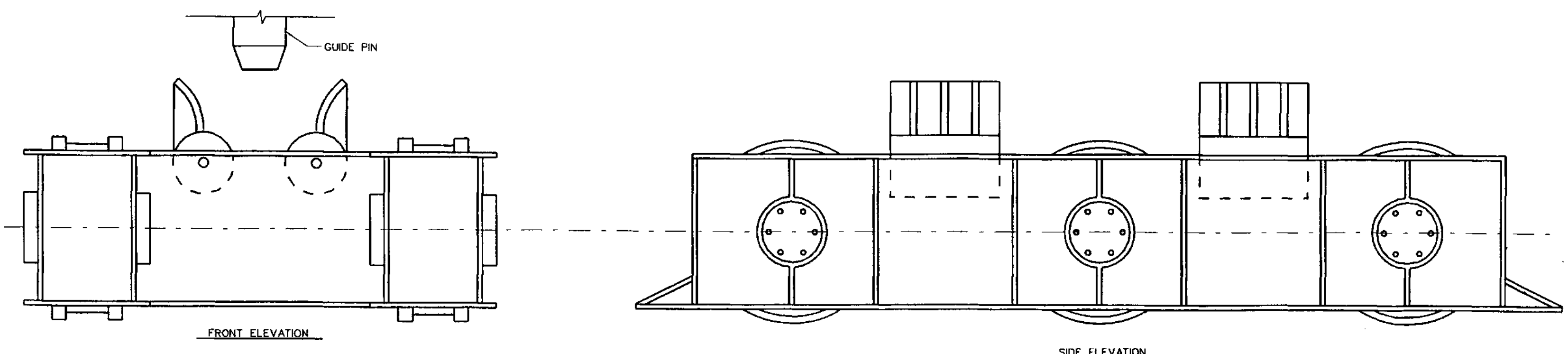

DETAIL (B)
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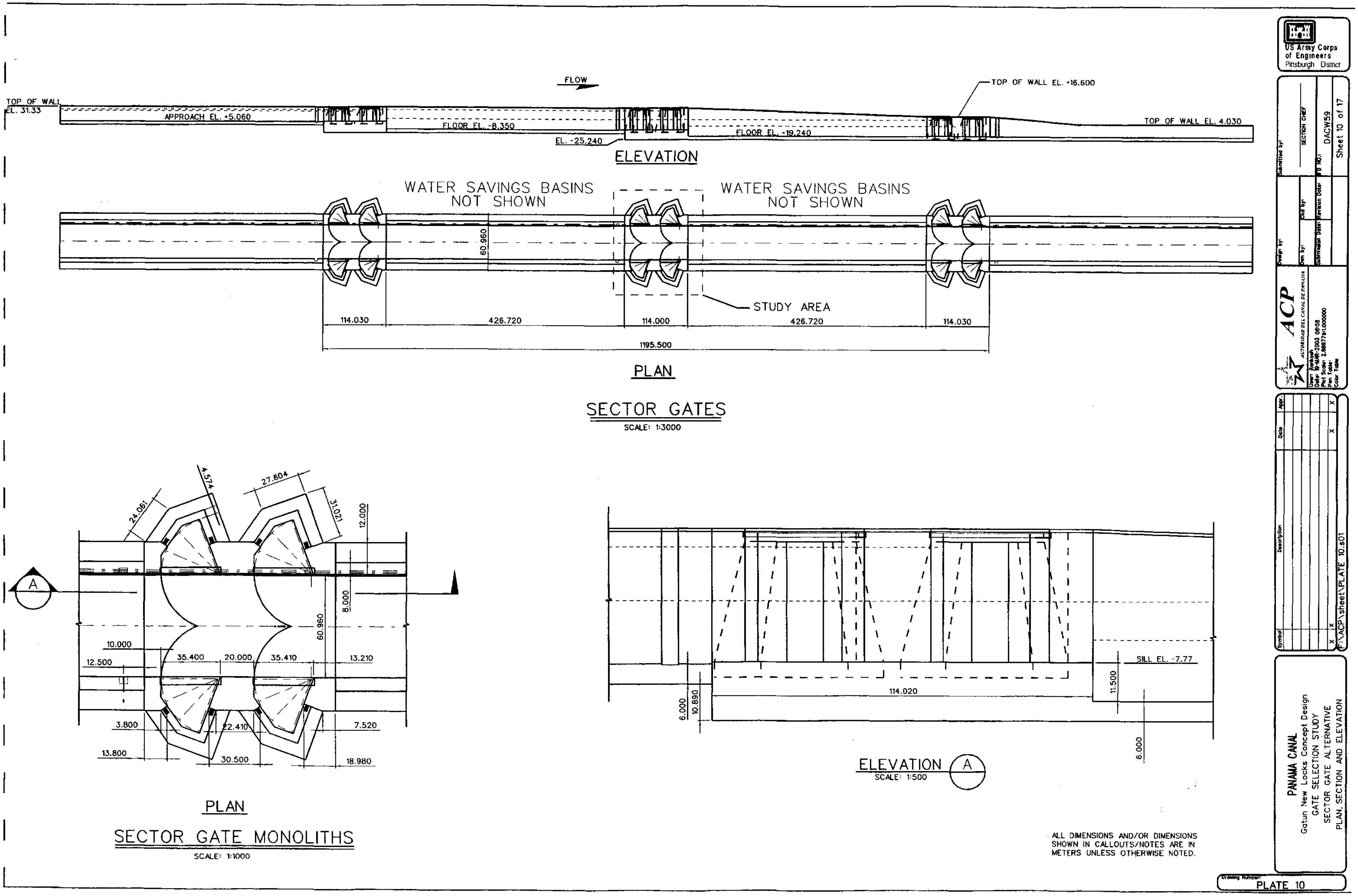
PANAMA CANAL
ATLANTIC LOCKS CONCEPT DESIGN
GATE SELECTION STUDY
ROLLER GATE ALTERNATIVE
SECTIONS

PLATE 9



FLOW
TOP OF WALL EL. 31.33
APPROACH EL. +5.060
FLOOR EL. -8.350
EL. -25.240
FLOOR EL. -19.240
TOP OF WALL EL. +16.600
TOP OF WALL EL. 4.030
ELEVATION
WATER SAVINGS BASINS NOT SHOWN
WATER SAVINGS BASINS NOT SHOWN
60.960
STUDY AREA
114.030
426.720
114.000
426.720
114.030
1195.500
PLAN
SECTOR GATES
SCALE: 1:3000
24.061
4.574
27.604
31.021
12.000
8.000
60.960
10.000
12.500
35.400
20.000
35.410
13.210
3.800
22.410
7.520
13.800
30.500
18.980
A
PLAN
SECTOR GATE MONOLITHS
SCALE: 1:1000
SILL EL. -7.77
114.020
11.500
6.000
10.890
6.000
ELEVATION A
SCALE: 1:500
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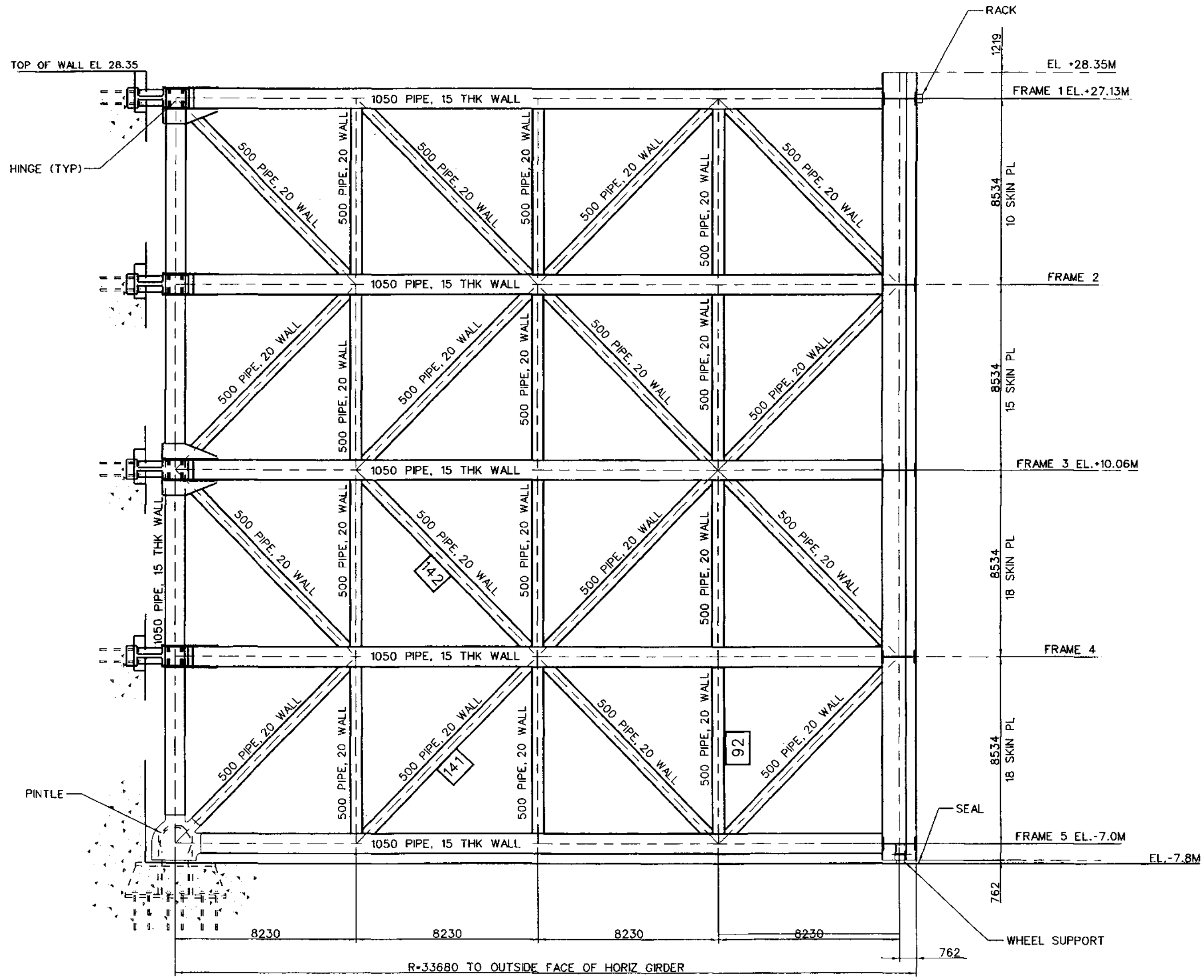
Gatun New Locks Concept Design
GATE SELECTION STUDY
SECTOR GATE ALTERNATIVE
PLAN, SECTION AND ELEVATION
PLATE 10



B CENTER TRUSS

FRAME PLAN

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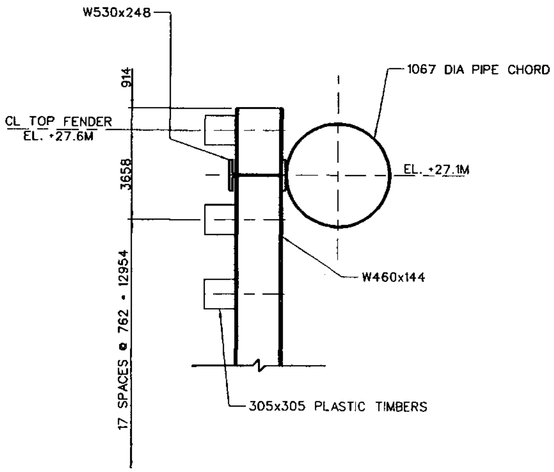
PANAMA CANAL
ATLANTIC LOCKS CONCEPT DESIGN
GATE SELECTION STUDY
SECTOR GATE ALTERNATIVE
B - FRAME PLAN

PLATE 11



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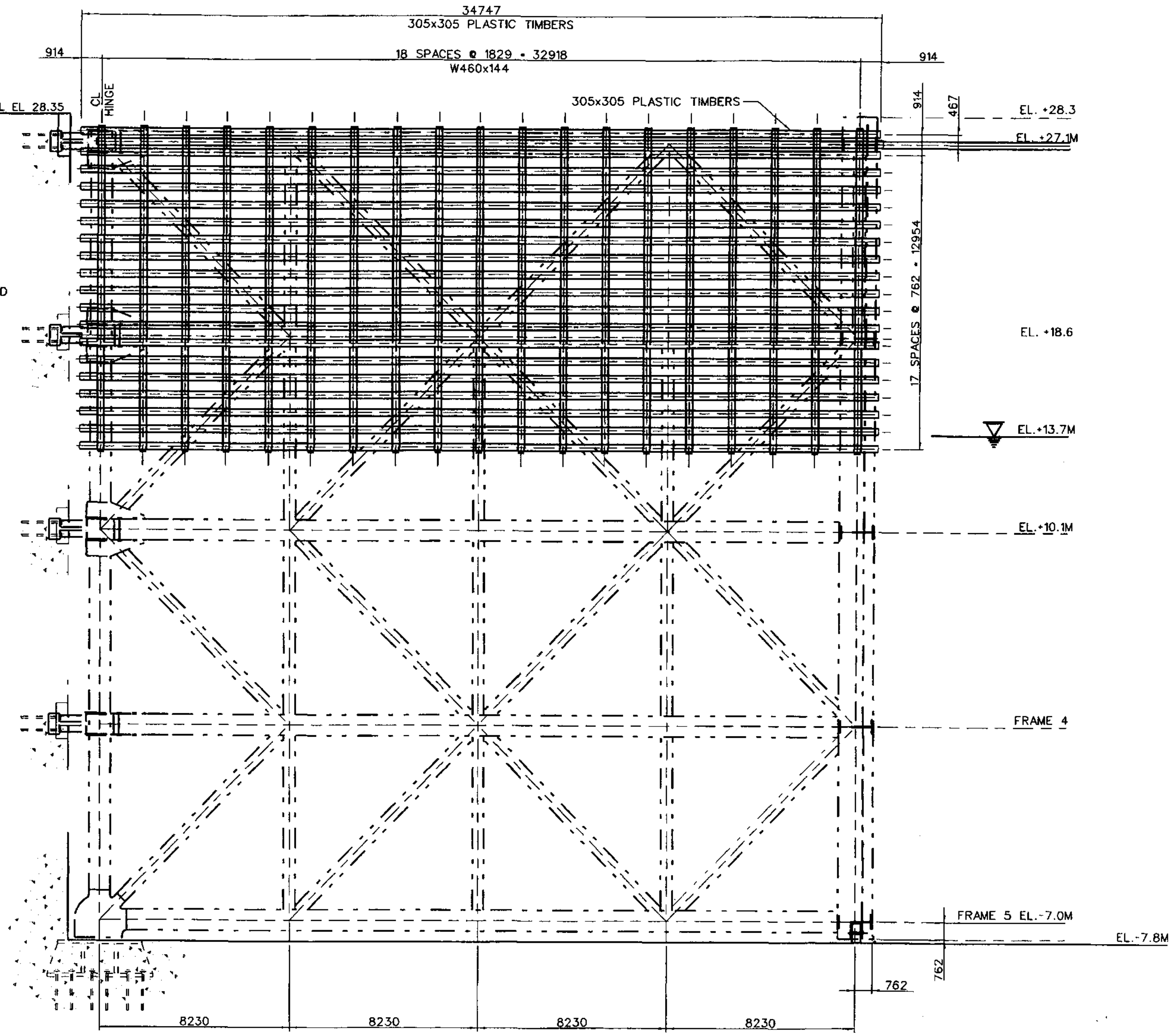

GATE ELEVATION

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PANAMA CANAL
ATLANTIC LOCKS CONCEPT DESIGN
GATE SELECTION STUDY
SECTOR GATE ALTERNATIVE
ELEVATION & DETAIL

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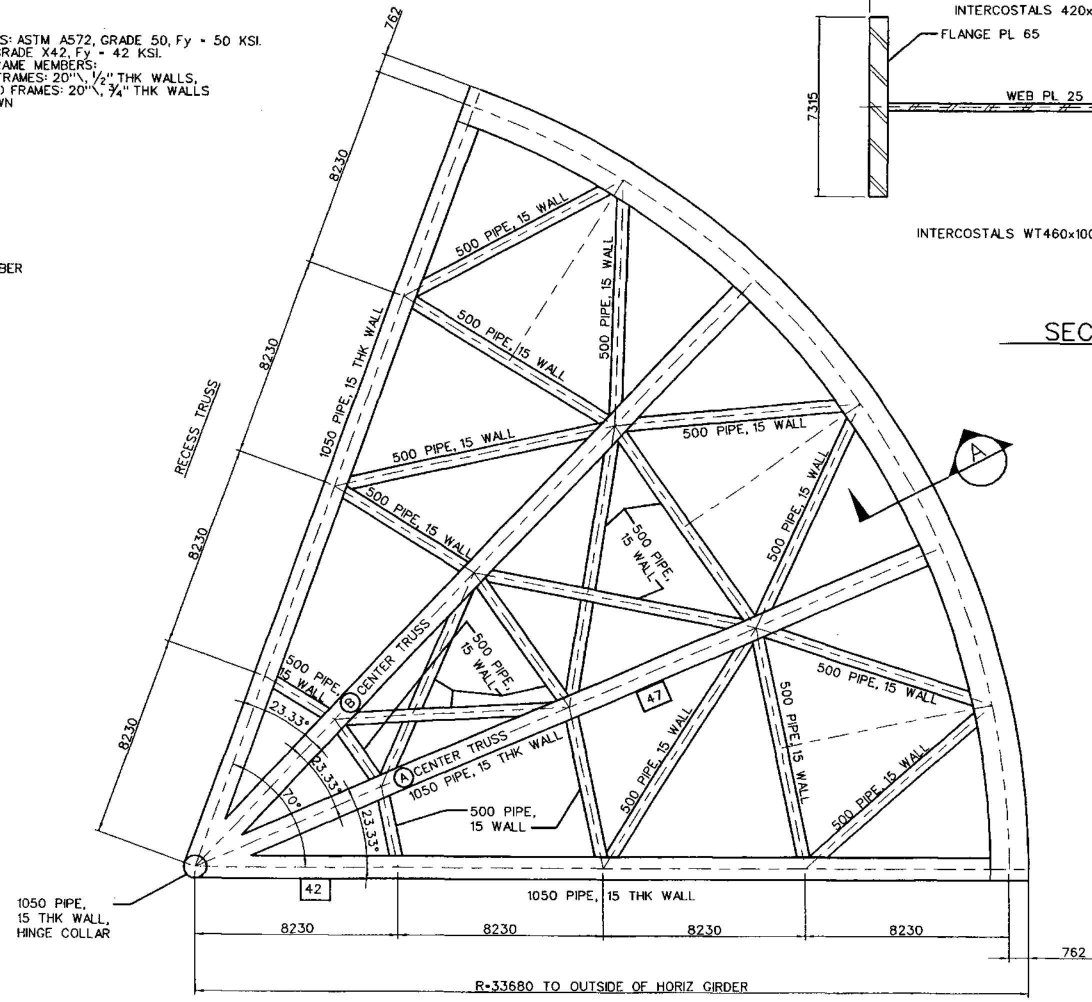
Drawing Number:
PLATE 12

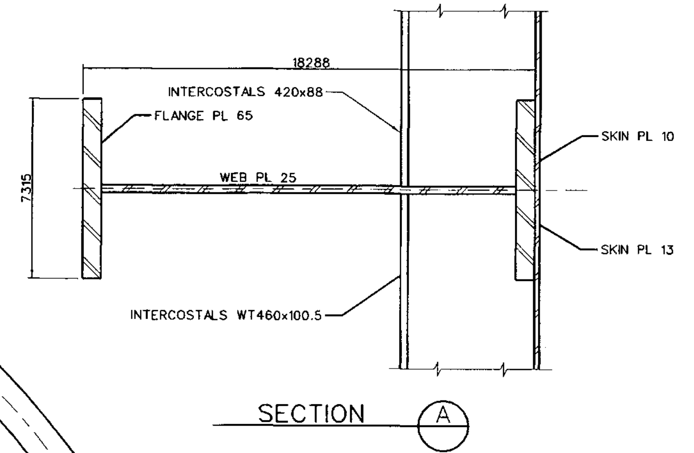
NOTES:
PLATES AND SHAPES: ASTM A572, GRADE 50, Fy = 50 KSI.
PIPE: API SPEC 5L, GRADE X42, Fy = 42 KSI.
ALL HORIZONTAL FRAME MEMBERS:
UPPER TWO (2) FRAMES: 20"\, 1/2" THK WALLS,
LOWER THREE (3) FRAMES: 20"\, 3/4" THK WALLS
EXECPT AS SHOWN

42 MEMBER NUMBER



CHANNEL TRUSS

FRAME 1&2 PLAN



SECTION A

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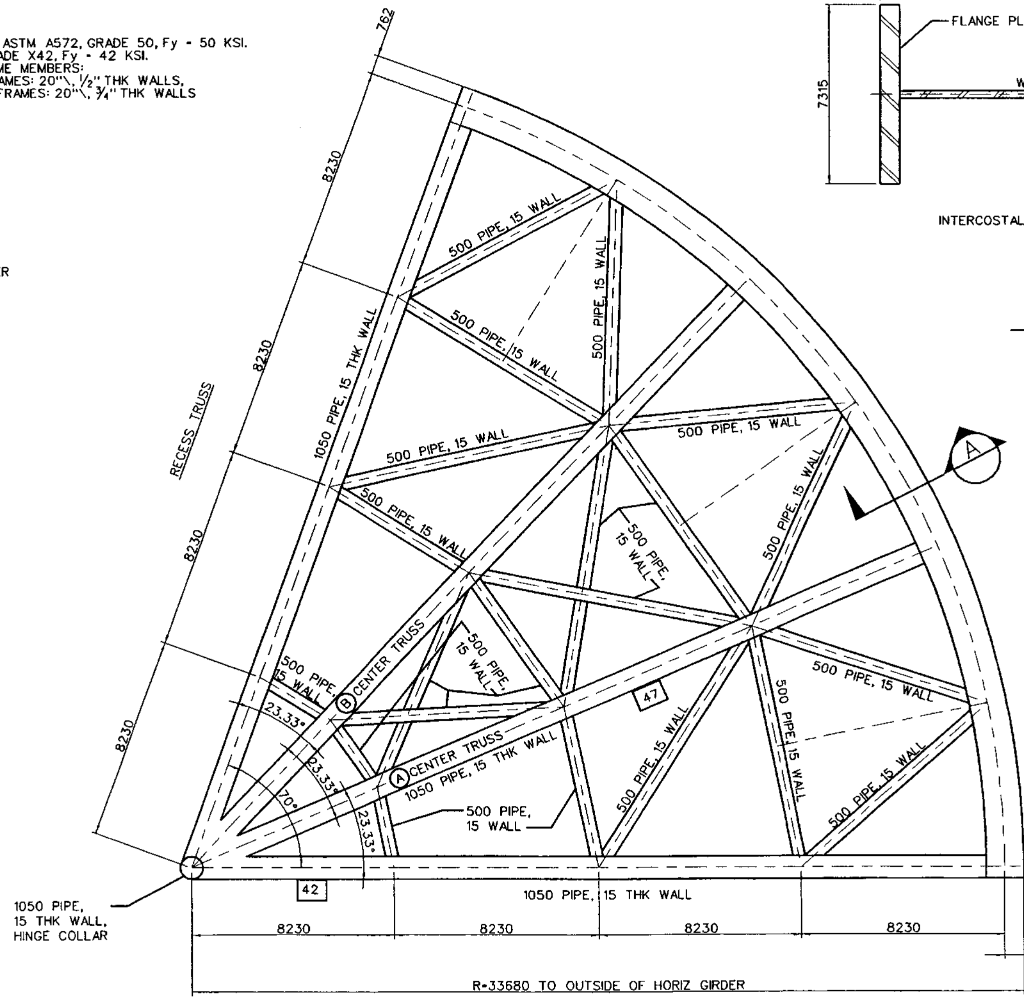
PANAMA CANAL
ATLANTIC LOCKS CONCEPT DESIGN
GATE SELECTION STUDY
SECTOR GATE ALTERNATIVE
FRAME 1 & 2 - PLAN & SECTION

F:\ACP\sheet\PLATE 13.s01

Drawing Number: PLATE 13

NOTES:
PLATES AND SHAPES: ASTM A572, GRADE 50, Fy = 50 KSI.
PIPE: APISPEC 5L, GRADE X42, Fy = 42 KSI.
ALL HORIZONTAL FRAME MEMBERS:
UPPER TWO (2) FRAMES: 20"\, ½" THK WALLS,
LOWER THREE (3) FRAMES: 20"\, ¾" THK WALLS
EXECPT AS SHOWN

42 MEMBER NUMBER



CHANNEL TRUSS

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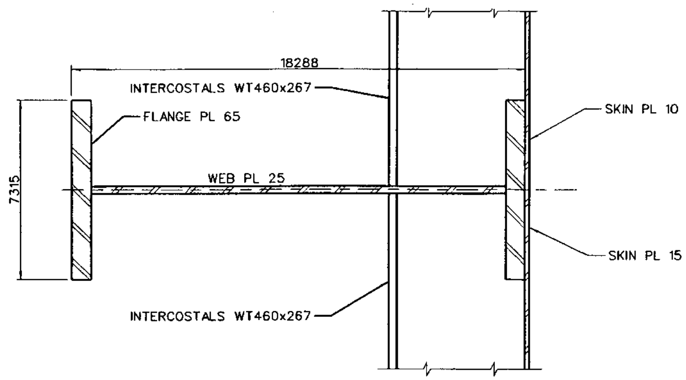

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PANAMA CANAL
ATLANTIC LOCKS CONCEPT DESIGN
GATE SELECTION STUDY
SECTOR GATE ALTERNATIVE
FRAME 3 - PLAN & SECTION

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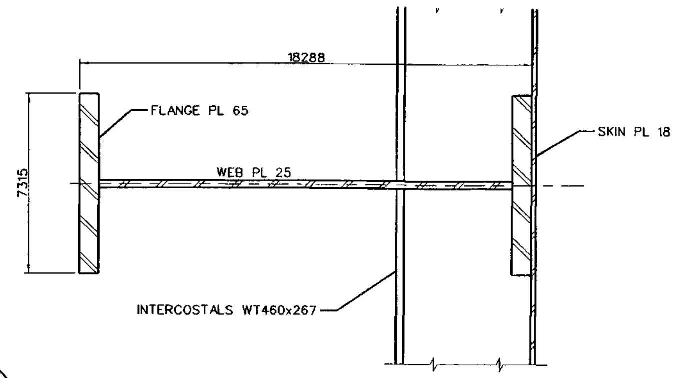
PLATE 14

NOTES:
PLATES AND SHAPES: ASTM A572, GRADE 50, Fy = 50 KSI.
PIPE: APISPEC 5L, GRADE X42, Fy = 42 KSI.
ALL HORIZONTAL FRAME MEMBERS:
UPPER TWO (2) FRAMES: 20"\, 1/2" THK WALLS,
LOWER THREE (3) FRAMES: 20"\, 3/4" THK WALLS
EXECPT AS SHOWN

42 MEMBER NUMBER



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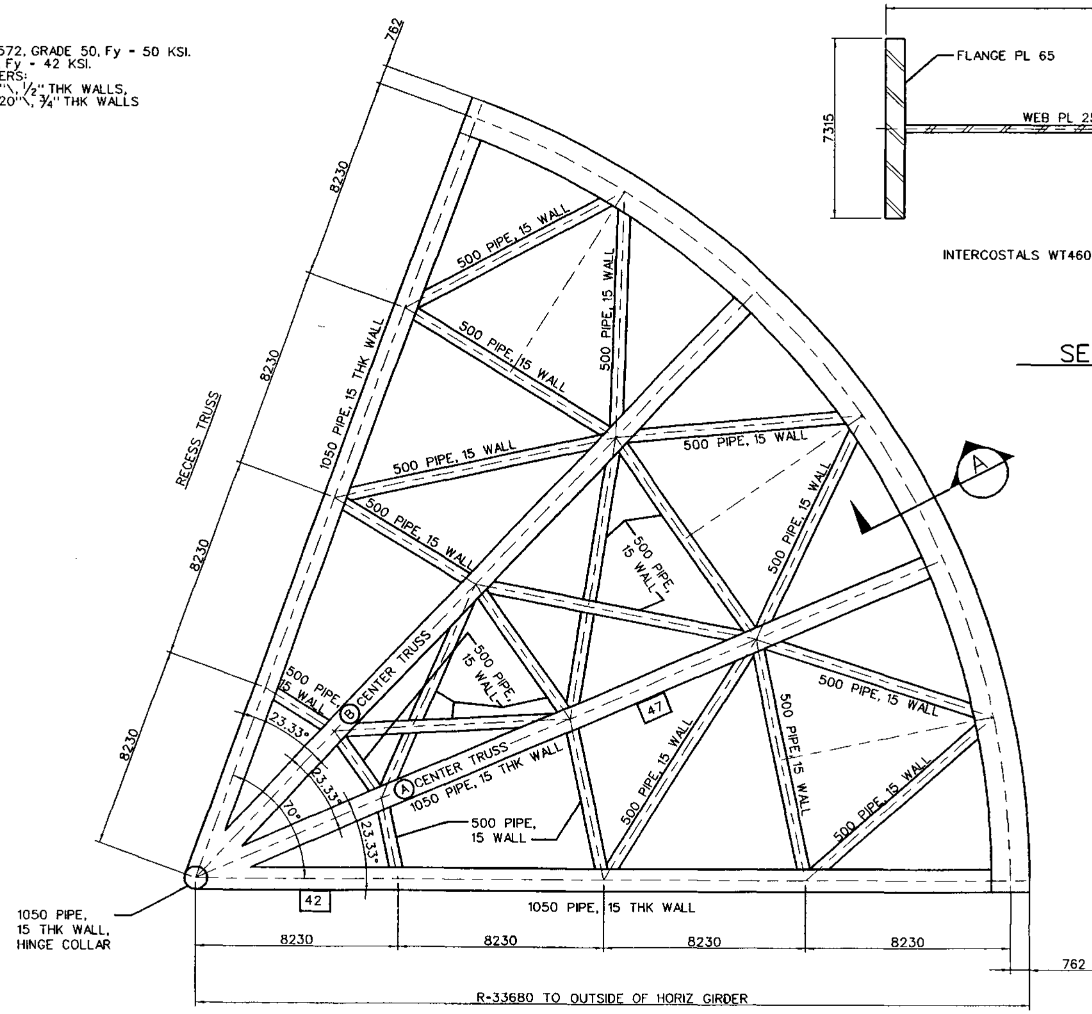

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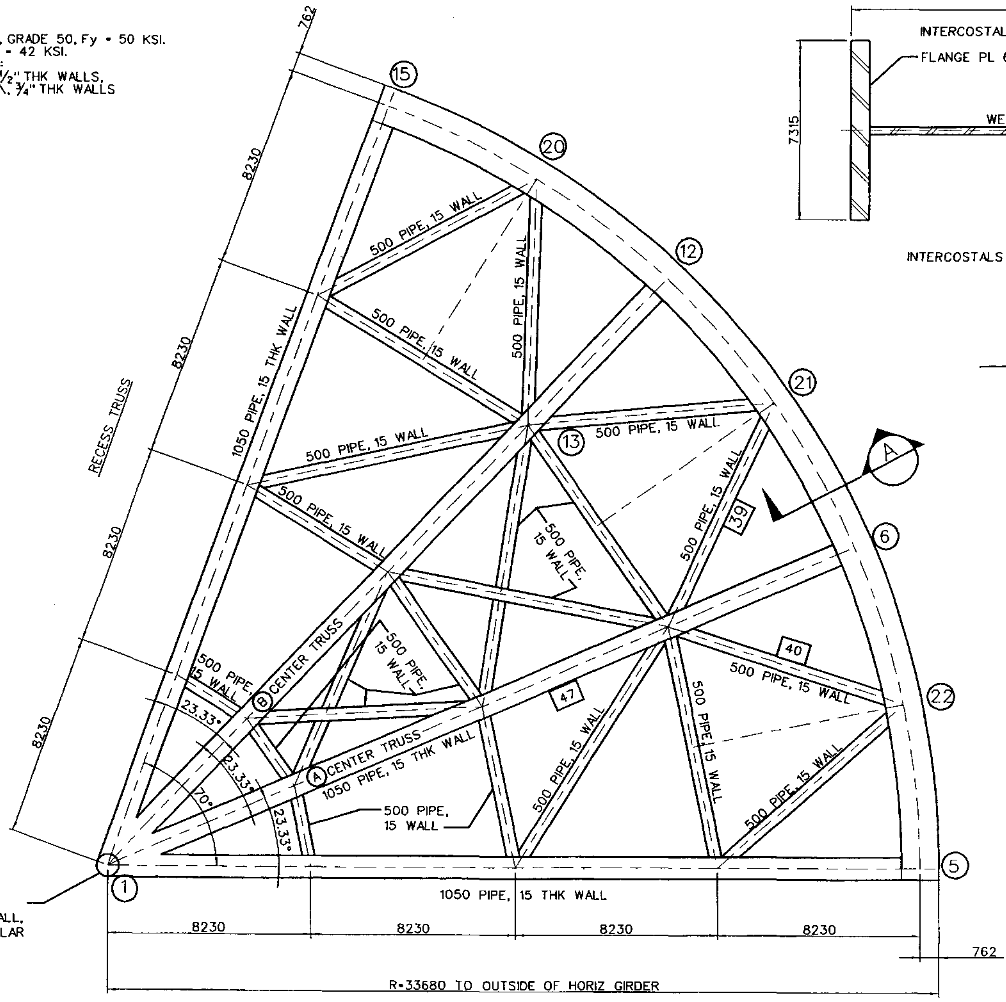
PANAMA CANAL
ATLANTIC LOCKS CONCEPT DESIGN
GATE SELECTION STUDY
SECTOR GATE ALTERNATIVE
FRAME 4 - PLAN & SECTION

Drawing Number: PLATE 15

NOTES:
PLATES AND SHAPES: ASTM A572, GRADE 50, Fy = 50 KSI.
PIPE: APISPEC 5L, GRADE X42, Fy = 42 KSI.
ALL HORIZONTAL FRAME MEMBERS:
UPPER TWO (2) FRAMES: 20"\, 1/2" THK WALLS,
LOWER THREE (3) FRAMES: 20"\, 3/4" THK WALLS
EXECPT AS SHOWN

42 MEMBER NUMBER



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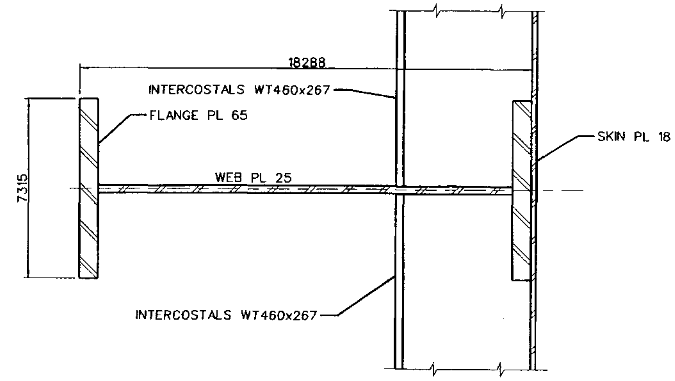

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ATLANTIC LOCKS CONCEPT DESIGN
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SECTOR GATE ALTERNATIVE
FRAME 5 - PLAN & SECTION

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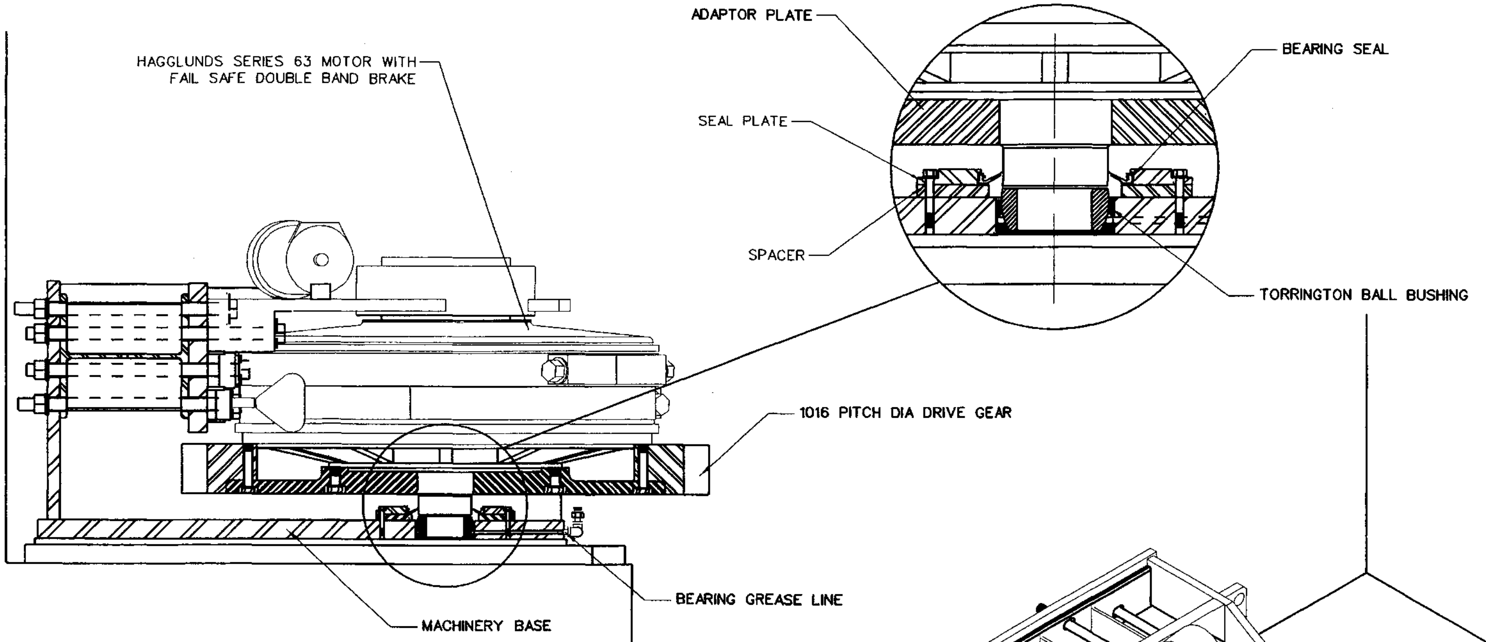
PLATE 16



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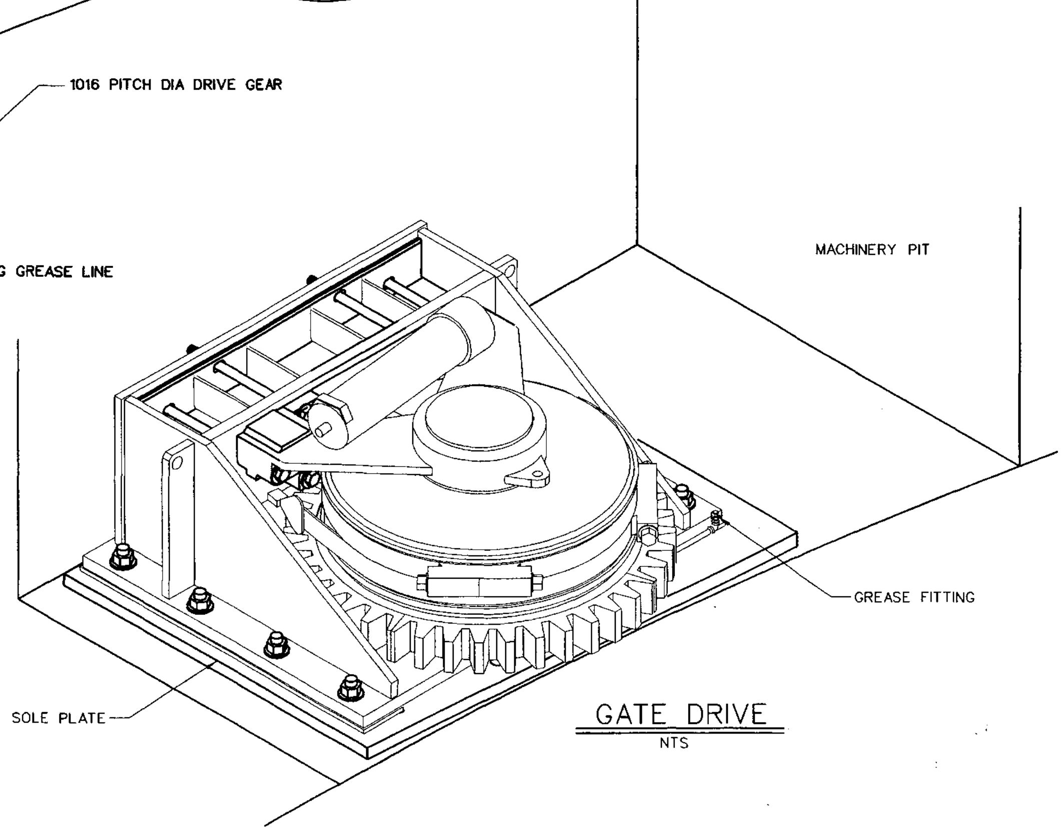

GATE DRIVE

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PANAMA CANAL
ATLANTIC LOCKS CONCEPT DESIGN
GATE SELECTION STUDY
SECTOR GATE ALTERNATIVE
MACHINERY - ISOMETRIC & SECTION

PLATE 17

PANAMA CANAL CONCEPT DESIGN

Atlantic Locks Structure

Third Lane Lock

Appendix C

Geotechnical Investigations, Analyses and Designs

Prepared for

Canal Capacity Projects Office

By

US Army Corps of Engineers®

Final Report

23 July 2003

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Exhibit C-I-3 ACP Direct Shear Test Report

Exhibit C-I-4 Excerpts from 1963 Shannon and Wilson Report on Foundation Investigations at Trinidad Arm of Lake Gatun

Exhibit C-I-5 Table 2.1 from Seepage, Drainage, and Flownets by H. Cedegren

Exhibit C-I-6 Excerpts from 1943 PCC Report, Part II, Chapter 5

Exhibit C-I-7 Excerpts from 1941 Outline Report on Canal Zone Geology in Relation to the Third Locks Project by D. MacDonald

Exhibit C-I-8 Table 4-8 from Foundation Analysis and Design by J. Bowles

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Geology Exhibit

Exhibit C-II-1 Final Report on Modified Third Locks Project, Part II - Design, Chapter 5 – Foundations and Slopes, December 1943

Part I - Soils

1. BACKGROUND

1.1. General

Numerous geotechnical investigations and studies have been conducted in support of design and construction activities for the Panama Canal, the Gatun Locks, and most recently, the Third Lane Lock studies. Data compiled in connection with these studies has been documented in the form of Design Reports and Drawing Folio's, Governor Reports, Special Reports by the Panama Canal Commission (PCC), and more recent investigations by the Panama Canal Authority (ACP). A complete listing of the geotechnical reports, publications, and texts used in conjunction with the preparation of this Geotechnical Appendix for the 2002 Concept Design Study is presented in Exhibit C-I-1.

1.2. Purpose

The purpose of this soils investigations was to provide technical information and engineering support for structural design of the lock features, geotechnical requirements for the site design, and engineering guidance for construction planning. The information presented in this Geotechnical Appendix was derived from the results of previous studies, published data, empirical data, data provided by the ACP as outlined in the Scope of Work, and Corps of Engineers concept design efforts. A complete summary of the published design parameters and soils data that was obtained during the document search, is presented in Table C-I-1.

2. SUBSURFACE INVESTIGATIONS

Subsurface investigations in support of the previous Third Lane studies were conducted in the 1930's and in 2001, and advanced along two separate alignments. The 1930's borings were advanced during preliminary design for the 1940's channel excavation. A total of 43 borings were drilled for the 2001 alignment study. Sixteen borings were advanced for Alignment A-1, and twenty-seven borings were advanced for Alignment A-2. The subsurface information collected from these studies was interpolated and extrapolated using digital terrain modeling software, and used as the design basis for this study. A borings plan, rock profiles, detailed descriptions of the previous subsurface investigations, and the results from the laboratory testing of rock, are presented in more detail in Part II of the Geotechnical Appendix

3. DESCRIPTION OF SUBSURFACE CONDITIONS

A plan drawing showing the topography of the site and the 9.45 degree alignment is presented on Plate C-I-1. A soil profile was developed along this alignment to show the top of ground and top of rock elevations and the subsurface soil conditions of the site. Borings data that was within close proximity of the alignment was added to the profile to provide insight into the subsurface conditions and identify subsurface conditions or hazards that may be located within the study area. Soil profiles along the 9.45 degree alignment are presented on Plates C-I-2, C-I-3, and C-I-4. For the purpose of generalizing the subsurface conditions at the site for concept design, the soil types were grouped into three major soil categories. The boring information suggests that the ground surface is covered in most places by a thin layer of random fill material ranging in depth from 5 to 10 m. This material was derived from the uncontrolled placement of overburden materials excavated during construction of the existing Gatun Locks. These areas appear to have been later used for land development around the project. Random fill is also found at significant depths in areas that were used for upland disposal during the lock construction and the 1940's channel excavation. A review of the SPT results indicate that the random fill deposits consist of organic and inorganic clays and silts of high plasticity. The material has a medium to soft consistency at the surface, and becomes softer and weaker with depth.

Underlying the random fill materials are soils commonly referred to as Atlantic Muck, which consists primarily of organic clays and silts of high plasticity. The Atlantic Muck is not present everywhere within the limits of excavation, but exist as interconnected masses of irregular shape and variable depth deposits that lie below sea level within the eroded valleys of the Gatun sandstone formation. The Atlantic Muck soils were recognized from the beginning of explorations within the Canal Zone as being problematic, and the original efforts by the French and later by the Americans revealed the treacherous nature of these materials. Several slides involving the Atlantic Muck occurred during excavation of the existing Gatun Locks and approach channels. Although these slides were not of the magnitude and concern caused by the Guilliard cut, they proved to be very difficult to remediate, and were identified as a cause for the delayed completion of the Gatun Locks construction project.

The Gatun Sandstone formation has the widest aerial extent of any of the sedimentary formations present within the canal zone, and is overlain by a maximum of 40 feet of weathered overburden that is the product of its weathering.. The Gatun Soils are derived from the weathering of these sedimentary rocks, and consist of moderately stiff cohesive clays, sandy clays, silty clays, and iron stained, red weathered rock that lies directly above the weathered Gatun sandstone formation. The transition of the red clay overburden to the weathered rock is reported to be so gradual that it precludes the establishment of a sharp dividing line. Past studies found it nearly impossible to separate the weathered rock from the red-clay derivative, so the two types of materials were identified as a single unit for slope design and the delineation of lock wall founding elevations.

4. LABORATORY TESTING OF SOIL

Laboratory data and testing results provided by the ACP were used in this concept design study to determine soil properties and design strengths. Subsurface investigations were conducted along the ACP designated Alignment A-2 in 2001 for a previous Third Lane study. During this investigation, all three soil types described in paragraph 3 were sampled and tested in the ACP Laboratory to determine soil classifications, Atterberg limits, gradations, specific gravity, and moisture contents. All testing was performed on disturbed samples. Shear testing was performed on 2 remolded samples of the Gatun overburden. According to the ACP, the samples were compacted to a density that approximated the insitu density at the sample locations, and the samples where tested in direct shear and Bromhead Ring Shear test apparatus. Tests performed on these recompacted samples were used to estimate the insitu, drained strength for the Gatun overburden. No shear testing data was provided by the ACP on the Atlantic Muck soils or the Random Fill soils that are located within the concept study alignment. Shear strengths for the Atlantic Muck and Random Fill soils materials were estimated from laboratory data provided by the ACP that was collected from other project sites within the canal zone, from geotechnical reports and correspondence presented in Plate C-I-1, and from correlations with the SPT data taken from the boring logs.

5. EVALUATION OF DATA

5.1. General

Soil test data from samples obtained throughout the project site were grouped into the same three major categories of soil that are commonly referenced by the ACP and other historical reports and literature . Where specific soil properties were not provided, and no published values were found, properties were estimated using correlations with existing data and engineering judgment.

5.2. Density

Soil densities used in this concept design study were calculated as saturated unit weights from data provided by the ACP, or were estimated from empirical data using the descriptions and properties presented in the laboratory data as the design basis.

5.2.1. Gatun Soils

The saturated unit weight of each direct shear test sample performed on remolded samples of the Gatun silty sand (SM) was calculated using the dry unit weight, and the initial void ratio of each sample. Samples were assumed to be 100% saturated for the saturated unit weight calculation. As shown in Figure C-I-5-1, the data points for the saturated unit weights were plotted as a function of depth. Most data points were less than one standard deviations from the median value, therefore the median value of γ_{Gatun} = 16 kN/m^3 was selected for concept design.

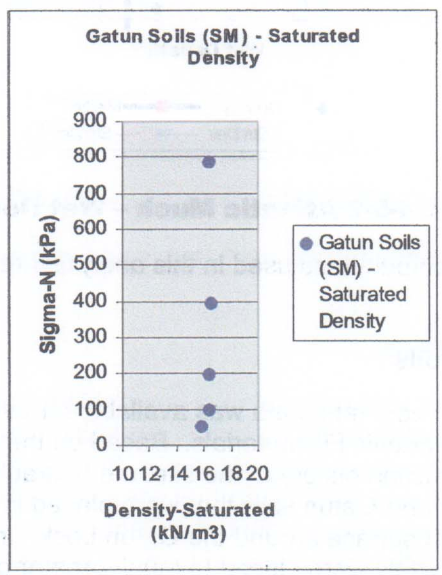


Figure C-I-5-1 Gatun Soils Saturated density

Data values and computations used in this analysis are presented in Table C-I-2.

5.2.2. Atlantic Muck Soils

The saturated unit weight of the Atlantic Muck soils (ML) along the proposed 3rd lane alignment were adopted from the laboratory testing results of similar Atlantic Muck Soils (CL) collected at the Trinidad Dam site, which is located in the vicinity of the Gatun Lock. This data was provided by the ACP for use in the concept design study. As shown in Figure C-I-2, the saturated unit weight data points were plotted as a function of depth, and most data points were less than one standard deviations from the median value, therefore the median value of γ_{Muck} = 14 kN/m^3 was selected for concept design.

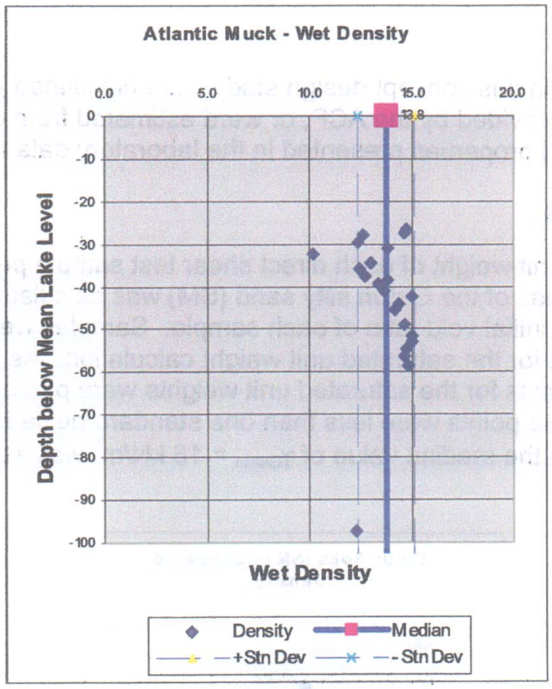


Figure C-I-5-2 Atlantic Muck – Wet Density

Data values and computations used in this analysis are presented in Table C-I-3.

5.2.3. Hydraulic Fill Soils

No dry unit weight or void ratio data was available for estimating the saturated unit weight of the Hydraulic Fill materials. Based on the soil classifications, SPT data, and the construction history of the site, the hydraulic fill soils are a random mix of Atlantic Muck and Gatun soils that were placed in layers ranging from 5-10 m to raise the ground surface around the Gatun Lock. In more remote locations, the hydraulic fill materials were placed to much greater depths (10-15 m). The unit weights of this material was therefore assumed for the concept design purposes as γ_{Random} = 15 kn/m^3. This value falls between design densities of the Gatun and Atlantic Muck soils, and is believed to be a reasonable value for concept design purposes.

5.3. Shear Strengths

The development of soil shear strengths was essential for estimating temporary cut slopes for lock construction, and evaluating the suitability of the overburden for reuse as construction material. The shear strengths of the insitu soils were evaluated using laboratory testing data provided by the ACP, and correlations with the SPT results reported on the boring logs. Since correlations are generally unreliable, existing shear test data was given precedence over correlations when lab data was available. The criteria and methodology used to derive the concept level design shear strength data for the overburden soils is provided below. The criteria and methodology used to derive the design shear strength data for rock is provided in Appendix C, Part II, Geology,

5.3.1. Gatun Soils

Direct shear tests were performed at the ACP soils lab on remolded samples of the Gatun Overburden following procedures outlined in ASTM 3080. Five specimens were prepared by compacting the disturbed samples in a 2.5" direct shear device until the desired unit weight was achieved. The samples were consolidated then sheared at a constant strain rate of 0.003 mm/min, at normal stresses of 50, 100, 200, 400, and 800 kPa, until the samples reached 15 mm of displacement. The slow rate of displacement was to allow for the total dissipation of pore pressure that may be induced in the samples during loading. Torsional ring shear tests were performed under drained conditions to determine residual shear strengths. Test results from the ACP Laboratory are presented in Exhibit C-I-2 and C-I-3. No undisturbed samples or triaxial shear testing of Gatun soils was available for use in this study, therefore Unconsolidated Undrained (Q), Consolidated Undrained (R), and Consolidated Drained (S) shear strengths were assumed, based on the results of the direct shear and ring shear testing. A summary of Direct Shear Test and Ring Shear Test results are shown in Figure C-I-5-3.

			Laboratory Results					
			Direct Shear		Ring Shear			
			Drained		Drained		Drained	
		Unified	Peak		Peak		Residual	
		Soil	c	ϕ	c	ϕ	c	ϕ
Insitu Soil Type	Description	Classification	kN/m^2	Degrees	kN/m^2	Degrees	kN/m^2	Degrees
Gatun Overburden	Silty Sand; Residual material derived from Gatun Sandstone	SM	21	31	3	29	9	20

Figure C-I-5-3 Direct Shear Test and Ring Shear Test

The assumed Q, R, and S (R-bar) strengths used for concept design are presented in Figure C-I-5-4.

			Concept Design Strengths					
			Unconsolidated Undrained		Consolidated Undrained		Consolidated Drained	
		Unified Soil Classification	Q -Strength		R - Strength		"S" (R-bar) Strength	
			c	ϕ	c	ϕ	c	ϕ
Insitu Soil Type	Description		kN/m^2	Degrees	kN/m^2	Degrees	kN/m^2	Degrees
Gatun Overburden	Silty Sand; Residual material derived from Gatun Sandstone	SM	30	0	21	29	21	31

Figure C-I-5-4 Concept Design Strengths

5.3.2. Atlantic Muck Soils

An extensive amount of triaxial shear testing was performed in 1962 on undisturbed samples of Atlantic Muck for preliminary design of an embankment dam on the Trinidad Arm of Lake Gatun. The report by Shannon and Wilson (Exhibit C-I-4) presented a significant amount of test data and results. The testing data used in this study is presented in Exhibit C-I-4. Although actual ranges of test results were provided, the Shannon and Wilson report did not identify or recommend specific design values for use in design analysis. The Atlantic Muck soils tested at the Trinidad embankment site were classified primarily as CL materials containing silts and peat, whereas the soil samples collected at the Gatun Lock site were classified primarily as ML materials containing some clay. Because the SPT results and index properties of these materials where similar, and both were generally categorized as Atlantic Muck, the shear test data provided in the Trinidad report was provided by the ACP for use in this study, and is believed to be appropriate for estimating concept level shear strengths values for the Atlantic Muck materials at the Gatun site.

In order to consolidate the diverse shear strength data provided in the Shannon & Wilson Report and aid in the selection of concept design parameters, plots of "p" versus "q" were made from the results of each triaxial test at failure. In accordance with Corps of Engineers EM 1110-1-1902, straight lines were drawn on the p-q plots in such a way that one-third of the data points fell on or below the line. The p-q plots for the drained and undrained test data of the Atlantic Muck are shown in Figure C-I-5-5 and Figure C-I-5-6.

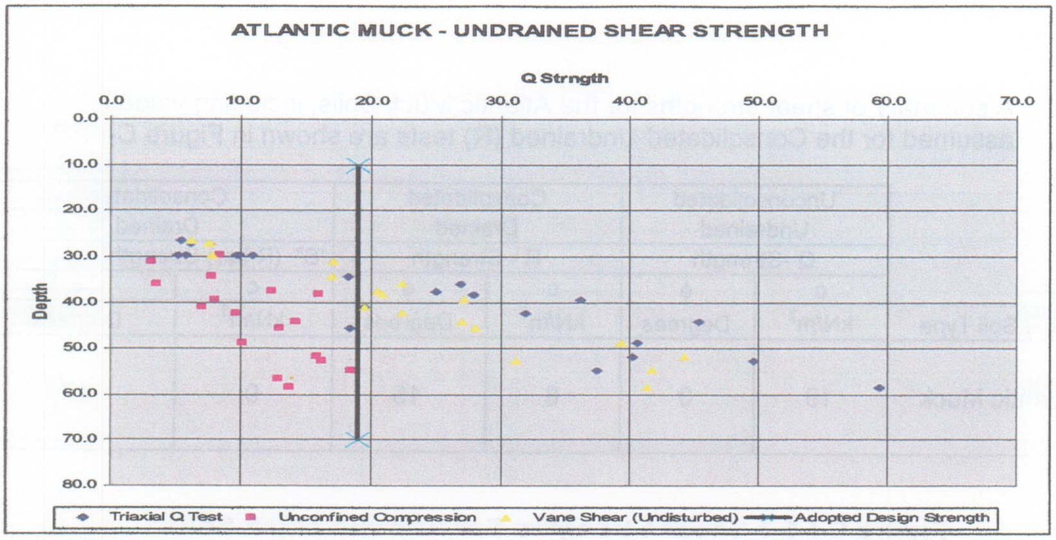


Figure C-I-5-5 Atlantic Muck Undrained Shear Strength

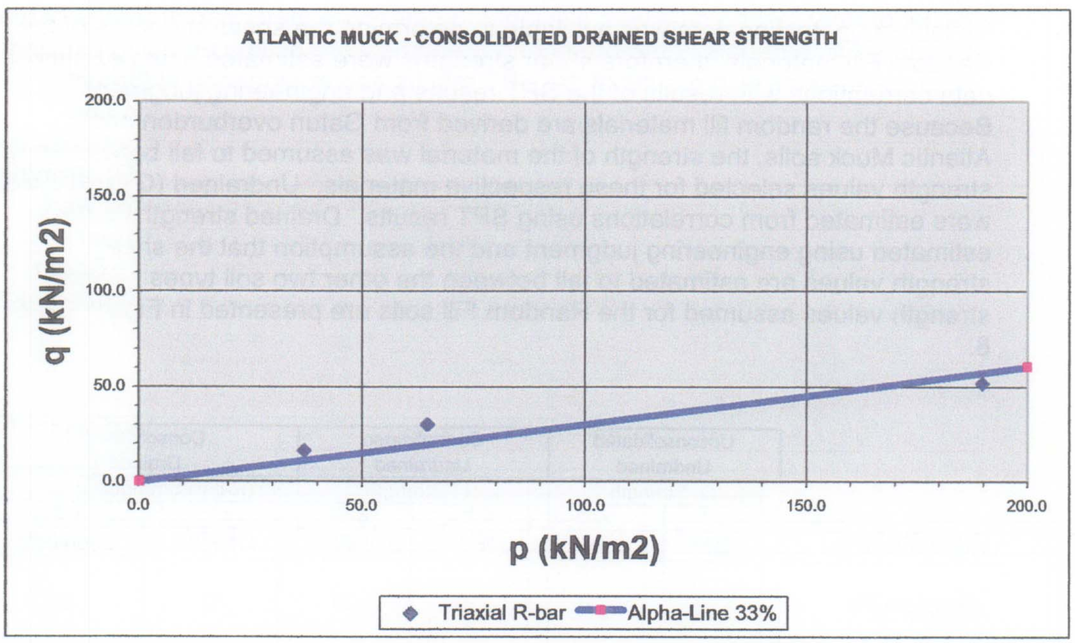


Figure C-I-5-6 Atlantic Muck Consolidated Drained Shear Strength

A summary of shear strengths for the Atlantic Muck Soils, including values assumed for the Consolidated-Undrained (R) tests are shown in Figure C-I-5-7.

	Unconsolidated Undrained Q -Strength		Consolidated Drained R - Strength		Consolidated Drained "S" (R-bar) Strength	
	c	ϕ	c	ϕ	c	ϕ
Insitu Soil Type	kN/m^2	Degrees	kN/m^2	Degrees	kN/m^2	Degrees
Atlantic Muck	19	0	8	15	0	17

Figure C-I-5-7 Shear Strengths For Atlantic Muck Soils

The laboratory data used for estimating the Unconsolidated-Undrained (Q) strengths, and Consolidated-Drained (R-bar) strengths, are presented in Tables C-I-4 and C-I-5 respectively.

5.3.3. Random Fill

No laboratory testing data was available to determine the shear strengths of the Random Fill materials, therefore shear strengths were estimated using existing data correlations with results of the SPT results and engineering judgment. Because the random fill materials are derived from Gatun overburden and Atlantic Muck soils, the strength of the material was assumed to fall between the strength values selected for these respective materials. Undrained (Q) strengths were estimated from correlations using SPT results. Drained strengths were estimated using engineering judgment and the assumption that the shear strength values are estimated to fall between the other two soil types. Shear strength values assumed for the Random Fill soils are presented in Figure C-I-5-8.

	Unconsolidated Undrained Q -Strength		Consolidated Undrained R - Strength		Consolidated Drained "S" (R-bar) Strength	
	c	ϕ	c	ϕ	c	ϕ
Insitu Soil Type	kN/m^2	Degrees	kN/m^2	Degrees	kN/m^2	Degrees
Random Fill	24	0	18	24	0	27

Figure C-I-5-8 Shear Strengths for Random Fill Soils

5.4. Permeability

Soil permeability's were estimated from Cedgren (Exhibit C-I-5) using the soil descriptions and gradations provided by the ACP, and data provided from the Shannon and Wilson Report. Permeability values for the Atlantic Muck were provided in the 1943 PCC report (Exhibit C-I-6). That data is consistent with the assumed values. Permeability coefficients selected for use in the concept design are presented in Figure C-I-5-9.

Insitu Soil Type	Description	Unified Classification System	Permeability k m/sec
Atlantic Muck	Silty Clay, Sandy Clay, and Fat Clay, with high organic content	CL, ML, CH	1.00E-11
Gatun Overburden	Silty Sand; Residual material derived from Gatun Sandstone	SM	1.00E-07
Disposal Fill	Clay with trace Sand and Silt; material derived from canal excavations	CL,SM,SC	1.00E-08

Figure C-I-5-9 Permeability Coefficients

5.5. Piezometric Data

Although several piezometers were installed during the 2001 subsurface exploration program for the A-2 alignment, a data collection program had not yet been established, and no piezometric data was available for use in the concept design study. Phreatic levels within the soil zones were estimated and extrapolated from 24 hour groundwater elevation readings as noted on the boring logs. None of the boring logs indicated the presence of artesian conditions, however, until specific subsurface zones are monitored and the potential uplift is quantified, conservative uplift pressures in the more permeable rock zones should be anticipated for design of the lock walls, lock floor slabs, and water saving basins.

5.6. Bearing Capacity

Presumptive Bearing Capacity values were selected for the Structural Backfill, Random Rockfill, and Random Backfill construction materials based on empirical data provided in the Uniform Building Code, as presented on EXHIBIT C-I-8. Adopted parameters used in this concept design study are presented in TABLE C-I-6. Values for the rockfill were provided specifically for the design analysis of the water savings basins where the top of rock is below the footing depth. Due to the relatively shallow depth of rock in this area, it was specified that the existing overburden would be removed and replaced with compacted rock fill prior to construction of the water savings basins. Bearing Capacity values for the other backfill materials were provided for design purposes where the respective backfill types were specified.

6. DESIGN PARAMETERS

6.1. Insitu Materials

The design parameters selected for the three major soil types that are found within the vicinity of the Gatun Locks are summarized in Table C-I-6. These soil properties were used for all concept level structural and geotechnical design analysis.

6.2. Construction Materials

Design values were taken from other reports and documents when available, or were selected from published data displaying similar material properties when site specific data was not available. The insitu soil and rock material properties were examined to determine their suitability for use as random, impervious, pervious, and structural fills. This project will require off-site disposal of a significant quantity of excavated materials derived from the lock construction. Major cost savings can be realized if these insitu material can be reused and the amount of handling and hauling is minimized using staged construction techniques. An assessment of the insitu materials and their suitability for reuse as construction materials is discussed below.

6.2.1. Atlantic Muck

This material is unsuitable for use as engineered fill. The material is soft and wet, and construction activities in regards to these materials are anticipated to be troublesome. The concept design assumes these materials will be spoiled off site.

6.2.2. Random Fill

This material has a wide range of material properties depending on the source of the original fill materials that were spoiled during construction of the existing Gatun Locks and 1940's channel excavation. Use of select materials from this soil mass may be possible, but for the purposes of this study, the material was assumed to be unsuitable for use as engineered fill, and will be spoiled off site.

6.2.3. Gatun Overburden

This material appears to be suitable for use as impervious and semi-impervious fill. Shrinkage and swell properties should be further evaluated in final design, as well as tests to determine optimum moisture content, and the soils permeability coefficients at various levels of compaction or density.

6.2.4. Gatun Rock

This material will produce the largest quantity of spoil on the site, and also appears to be the most suitable material for use as random construction fill and semi-pervious backfill. The 1943 Report (Exhibit C-I-6) addressed the suitability of the Gatun Rock excavation spoil for use as pervious fill behind the lock walls. Although experiments found considerable breakdown took place in abrasion tests, the materials would be suitable for pervious backfill if handled properly. Based on review of the core samples and the tendency of this material to weather and breakdown rapidly, it is assumed that the Gatun rock excavation will not meet current ASTM or Corps of Engineers standards for use as a satisfactory free draining aggregates behind the lock walls or for use as cell fill.

6.2.5. Manufactured Aggregates

Free draining and durable structural backfill aggregate requirements for this project were assumed to be imported from local and regional quarries that manufacture limestone and basalt aggregates. A summary of the material properties and design values selected for the construction aggregates and random fill materials are presented in Table C-I-6. These values were used in all concept level geotechnical and structural design computations.

7. SLOPE DESIGN

7.1. General.

In accordance with the scope of work, the construction load case is to be analyzed for recommendations on temporary cutslopes in overburden at two critical locations. However, both the End of Construction (short-term) and Fully-Drained (long-term) load cases were analyzed for concept design because it is difficult to predict the length of time required for the soils in the shear zone of the cut slope to reach the fully drained condition, and the end of construction load case may not be the critical loading that controls the slope design. Additionally, the project site has three different soil types, each being predominant within certain reaches of the project alignment. In order to more accurately estimate the limits of excavation and the excavation quantities, stability analyses were performed for the undrained (short-term) and drained (long term) conditions, at three critical locations, to establish a design basis for safe excavation slopes in the different overburdens materials that are present along the proposed alignment.

7.2. Method of Stability Analysis

Minimum factors of safety for each loading condition analyzed were calculated using the UTEXAS4 slope stability program, Version No.4.0.0.3.[Revised:6-29-00]. This software was developed specifically for the Corps of Engineers by Professor Stephen G. Wright of the University of Texas, and meets all Corps of Engineers criteria for slope stability analysis. The TEXGRAF software package was used as the interface for post processing illustrations. The analysis was performed using Spencer's method, which is a two-dimensional limit equilibrium method of slices procedure which calculates a factor of safety satisfying both moment and force equilibrium. The Spencer's method also yields the angle of the interslice forces. Search procedures for both circular and non-circular shear surfaces with various initial positions and search modes were used. Water filled, vertical cracks were specified as necessary to avoid tensile forces in the upper slice data. Only non-circular shear surfaces were analyzed for the concept design. A sufficient number of failure points were included in the analysis such that the non-circular surface could assimilate a circular shear surface if the critical surface was in fact circular. Due to the proposed gradual slopes and layered soil strata at the site, the non-circular shear surfaces were able to search for and follow the base of the weaker layers, resulting in wedge-type failures that were more representative of the types of failures typically seen in layered strata. Circular searches should be conducted in future studies when analysis is being performed in the northern navigation approach, and in other reaches where deeper and more uniform deposits of Atlantic Muck may be found.

7.3. Safety Criteria

Because the Corps of Engineers has no specific Factor of Safety requirements for the stability of cut slopes, the minimum factors of safety for the various loading conditions were adopted from those listed in EM 1110-2-1902 for earth and rock fill embankment dams. In accordance with EM 1110-2-1902, the minimum required Factor of Safety for each load case analyzed are shown in Figure C-I-7-1:

Stability Load Case	Analysis	F.S. $_{min}$
End of Construction	Undrained	1.3
Long Term	Drained	1.4

Figure C-I-7-1 Factors of Safety

In situations where significant uncertainty exists in the site conditions or the laboratory shear strength data, EM 1110-2-1902 recommends that the minimum factor of safety criteria be adjusted to values that are commensurate with the level of risk and uncertainty that has been introduced into the analysis The Factor of Safety for the Long Term Case may be reduced to F.S=1.3 when the cutslope excavations are temporary and will be backfilled prior to completion of Lock construction.

7.4. Design Conditions

7.4.1. End of Construction Case – Total Stress Analysis

This case is intended to represent the short-term, undrained loading condition, and is based on the assumption that positive pore pressures are induced because the soil cannot consolidate readily under an imposed load during the construction period. In cut slopes however, an unloading occurs which would induce lowered or negative pore pressures. Despite this difference between the design assumptions and field conditions, this case was used for design of the cutslopes, and is consistent with Corps of Engineers guidance and design practices. A total stress analysis was performed for this load case using saturated unit weights and no pore pressures. The inclusion of water filled, vertical tensile cracks, were required to avoid negative shear (tensile) forces. The depth of the crack was computed using Rankine's earth pressure theory, and the computed crack depths are shown in Figure C-I-7-2.

Material	Tensile Crack Depth (meters)
Random Fill	3.2
Atlantic Muck	2.7

Figure C-I-7-2 Tensile Crack Depth

Computations to determine the tensile crack depths are shown in Table C-1-7.

7.4.2. Long Term Case – Effective Stress Analysis

This case is intended to represent the long-term, fully drained loading condition, and is based on the assumption that the soils have had sufficient time to fully consolidated. An effective stress analysis was performed for this load case using saturated unit weights. The piezometric surface is based on the 24 hour water level readings obtained from the boring logs at each representative section analyzed.

7.5. Surcharge Effects

7.5.1. Construction

The stability analysis for temporary cut slope design did not include surcharges induced from construction equipment or stockpiles that may be located at the top of slope. Based on the relative uncertainty of the soil shear strength parameters and the limited locations that were analyzed, surcharge loading were not considered. Additional refinement to the slope stability analysis should be performed in future design studies to include these types of surcharge loadings.

7.5.2. Seismic

The stability analysis for temporary cut slope design did not include a seismic surcharge loading. Whereas seismic loads are normally recommended for the design of navigation approach slopes where satisfactory long-term performance is preferred, seismic loads are not normally considered for temporary cut slope design where stability requirements are short term and the consequences of failure and clean-up will be incidental to the construction activity.

7.6. Sections Analyzed

Three representative cross-sections were evaluated to determine the recommended temporary construction slopes behind the lock and entrance walls. As described in the preceding sections, the site is composed of three predominant soil types. Because excavations in the Random Fill and the Atlantic Muck may occur to depths in excess of 5-10 m, representative section locations were selected so as to analyze cut slopes at locations having deposits of these materials to these depths. An additional section was analyzed behind the Water Saving Basins to determine if the concept design slopes in this reach provides sufficient clearance between the top of the temporary cut and the existing Gatun Lock infrastructure. The location of the cross-sections used for the stability analysis are shown on Plate C-I-1.

7.6.1. Project Station 12+200 – East Slope

The geotechnical cross-section shown on Plate C-I-5 represents the subsurface conditions at this location where temporary excavation slopes will be made in the Atlantic Muck. According to the soil profile presented in Plate C-I-3, this section represents a typical project reach where the deeper deposits of the Atlantic Muck are located. The stability model used to analyze this geotechnical cross-section was developed using the ground surface and rock surface elevations interpolated from the use of digital terrain modeling software. Boring data was superimposed on the section, and the soil zones were extrapolated from the boring data. Due to limited boring data on the east side of the project alignment, the soil profile was conservatively extrapolated such that the Atlantic Muck soils are the predominant soil type and control stability. The model used for the slope stability analysis is presented in Plate C-I-5. A summary of the computed Factors of Safety for this project station are shown in Figure C-I-7-3.

Station 12+200 East - 1v:8h Slopes in Atlantic Muck Soils			
Stability Load Case	Analysis	$F.S._{min}$	$F.S_{Computed}$
End of Construction	Undrained	1.3	2.6
Long Term	Drained	1.4	1.3*
Long Term w/ Berm	Drained	1.4	3.2

Figure C-I-7-3 Factors of Safety – Sta. 12+200 East

Results of the stability analysis performed for the End of Construction (Total Stress Analysis) and Long Term (Effective Stress Analysis) showing the location of the critical potential failure surface are shown on Plates C-I-6 thru C-I-9. Computations, including input and output files, are presented in the Computation Section of this Appendix. As noted in Figure C-I-7-3, the computed factor of safety does not meet the minimum criteria for the long-term load case. As noted previously, Factor of Safety Criteria for the long term case on temporary cutslopes can be reduced to $FS_{min} = 1.3$, provided that the temporary excavations are backfilled before the slopes have adequate time to fully drain. The construction schedule and the insitu soil properties should be further evaluated to in future studies in an attempt to use the lower Factor of Safety value. To meet the criteria for this concept level design, a rock fill construction bench is proposed at the toe of slope and will be placed there concurrent with the rock excavation activities. The rock filled stability bench proposal, as shown on Plate C-I-5, offers several advantages. The rockfill bench will reduce the surface sloughing that is typical of cut slopes in soft soils, and it will improve the overall long term stability of deep seated failures. A second benefit of the rock fill bench is that it reduces the amount of rock spoil that must be hauled to the disposal site or temporary stockpile locations. The rock fill buttress will also provide an improved ground surface for use by heavy equipment that will otherwise be impeded when traversing the Muck soils. Lastly, this rock filled stability bench will serve as a transition zone between the exposed soil cut slopes and the pervious structural backfill zone, significantly reducing the quantity of required structural backfill.

7.6.2. Project Station 12+750 – East Slope

The geotechnical cross-section shown on Plate C-I-9 represents the subsurface conditions at this location wheretemporary excavations must be made in the Random Fill deposits. According to the soil profile presented in Plates C-I-2, C-I-3, and C-I-4, this section represents a typical project reach where the deeper deposits of the Atlantic Muck are located. The stability model used to analyze this geotechnical section was developed using the ground surface and rock surface elevations interpolated from the use of digital terrain modeling software. Boring data was superimposed on the section, and the soil zones were extrapolated from the boring data. Due to limited boring data on the east side of the project alignment , the soil profile was conservatively extrapolated such that the Random Fill materials were the predominant soil type and would be the controlling soil type in the slope stability analysis. The subsurface model used for the slope stability analysis is presented in Plate C-I-9. A summary of the computed Factors of Safety for this project station are shown in Figure C-I-7-4.

Station 12+750 East - 1v:6h Slopes in Random Fill Soils			
Stability Load Case	Analysis	F.S. $_{min}$	F.S $_{Computed}$
End of Construction	Undrained	1.3	1.7
Long Term	Drained	1.4	1.8

Figure C-I-7-4 Factors of Safety - Sta. 12+750 East

Results of the stability analysis performed for the End of Construction (Total Stress Analysis) and Long Term (Effective Stress Analysis) showing the location of the critical potential failure surface are shown on Plate C-I-10 and Plate C-I-11. Computations, including input and output files, are presented in the Computation Section of this Appendix. Although not required for stability, a rock fill construction bench will be placed along the toe of the slope to improve long term stability, reduce haulage and disposal costs, and provide a transitional zone of materials between the exposed soil slopes and the free draining structural back fill.

7.6.3. Project Station 12+750 – West Slope

The geotechnical cross-section shown on Plate C-I-12 represents the subsurface conditions at this location where temporary excavations will be made in the Gatun soils and Random Fill materials behind the Water Saving Basins. The stability model was developed using the DTM ground surface and rock surface. Boring data was superimposed on the section, and the soil zones were extrapolated from the boring data. Sufficient boring data existed at this cross-section such the subsurface profile is believed to be representative of the actual conditions. The subsurface model used for the slope stability analysis is presented in Plate C-I-12. Results of the stability analysis performed for the End of Construction (Total Stress Analysis) and Long Term (Effective Stress Analysis) showing the location of the critical potential failure surface are shown on Plates C-I-13 thru C-I-15.A summary of the computed Factors of Safety for this project station are shown in Figure C-I-7-5.

Station 12+750 West - 1v:4h Slopes Behind Water Savings Basins			
Stability Load Case	Analysis	F.S. $_{min}$	F.S $_{Computed}$
End of Construction	Undrained	1.3	1.8
Long Term - Deep	Drained	1.4	1.8
Long Term- Shallow	Drained	1.4	1.6

Figure C-I-7-5 Factors of Safety – Sta. 12 + 750 West

Computations, including input and output files, are presented in the Computation Section of this Appendix. Although not required for stability, a rock fill construction bench will be placed along the toe of the slope to improve long term stability, reduce haulage and disposal costs, and provide a transitional zone of materials between the exposed soil slopes and the free draining structural back fill.

7.7. Results of Stability Analysis

As illustrated in Figures C-I-7-3, C-I-7-4, and C-I-7-5, the computed factors of safety for the concept design slopes exceed the minimum required factor of safety for each load case, with the exception of project station 12+200 east, where a rock fill stability bench is required to improve long term stability. Although the temporary cut slopes could theoretically be steepened and still meet the minimum factor of safety criteria, the limited soil test data and boring data within the potential zones of shear failure introduce significant uncertainty into the analyses. Because of these uncertainties, higher Factors of Safety were believed to be more appropriate for concept level slope design and construction planning purposes..

7.8. Comparison of Results

Due to the uncertainty of the shear strength data used in the analysis, and the excessive distances that borings and subsurface data was extrapolated to develop the stability models, the results of the stability analysis were compared with previous report findings, field data, and empirical data, in an attempt to confirm the validity of these results.

7.8.1. Existing Slopes

The existing slopes within the existing 1942 Third Lane channel excavation were evaluated using the top of ground elevations from the 2001 DTM and the lock floor excavation elevations as shown on the 1943 folio of drawings. The contract drawings label the excavation limits as "Payment Line for Excavation", and specify that a 1v:4h cut slope shall be made in the deeper soil deposits. These proposed slopes appear to be consistent with the guidance provided in the Stability Charts that were developed by MacDonald, Cassagrande, and the Panama Canal Commission, and presented in the 1941 report (Exhibit C-I-7). However, an interpretation of the 2001 DTM contours along the northern reach of the 1943 canal excavation where deep deposits of the Atlantic Muck were encountered suggest that the existing slopes are much flatter. The existing slopes were calculated to be approximately 1v:8h using the rock excavation limits shown on the contract drawings, and the 2001 DTM topography. Further review of the topography and general alignment of the contours near the top of the cut in this area suggest that slope movement or massive failure may have occurred sometime after the 1943 channel excavation work was suspended.

- The existing Random Fill slopes were calculated from the DTM at locations identified on the 1940 contract drawings as upland disposal sites. These slopes were fairly uniform and consistently graded at approximately 1v:2h to 1v:3h. Although these soils were placed during construction by undocumented methods, lift thicknesses, and compaction requirements, and it is unknown if the final slopes were re-graded after the material had dried, the assessment still provides a unique insight into the long term stability of this soil mass.
- The existing slopes in the Gatun soil mass located in the northern approach at the existing Gatun lock were observed during a May 2002 field inspection. These slopes were standing at approximately a 1v:2h slope near the top of bank, and became steeper near the toe of slope where overburden consisted of a shallow colluvial layer that covered the weathered Gatun rock formation.

7.8.2. Published Data

The amount of data available for this study on soil properties of the Gatun soils and the Random Fill Soils is very limited. The stability of the Atlantic Muck soil slopes has been of interest and concern to the designers and builders of the Panama Canal since the projects inception. A significant amount of research and analysis has been performed over the years on Atlantic Muck soils with most conclusions in agreement that stability of open excavations in this type of soil is typically problematic. Stability charts comparing the depth of cut with the cotangent of the slope were developed by MacDonald , Cassagrande and Middlebrook, and the PCC Engineering Division. (Exhibits C-I-6 and C-I-7) A comparison of the chart solution slopes for typical slope heights along the 9.75-degree alignment suggests that the slopes proposed for the 2002 study are somewhat conservative. Because very little documentation exists on the method of analysis, the shear strength data, and the factor of safety criteria used to develop of the 1943 chart solutions, the stability charts were not utilized for the 2002 concept design. Furthermore, review of existing slopes as presented in section 7.8.1, suggest that the design slopes proposed for the 1943 canal work may have underestimated the long term stability of the Atlantic Muck soils. There was no published data available for this study which addressed the overall stability of the Gatun soils or Random Fill soils slopes.

7.9. Recommended Temporary Construction Slopes

Based on the results of the slope stability analysis, and review of the exiting documentation and field conditions at the Gatun project site, the recommended cutslopes for excavations made to top of rock are summarized by soil type in Figures C-I-7-6.

Generalized Summary of Temporary Cut-Slopes by Material Type		
Soil Type	Deposit	Slope
Atlantic Muck	All	1v:8h
Random Fill	>10 meters	1v:6h
Random Fill	<10 meters	1v:4h*
Gatun Overburden	All	1v:2h
*Proposed Cutslopes behind Water Savings Basins		

Figure C-I-7-6 Summary of Temporary Cut Slopes by Material Type

Similarly, the recommended cut slopes are summarized by project location and are presented in Figure C-I-7-7.

Generalized* Summary of Temporary Cut-Slopes By Project Stationing				
East Slope			West Slope	
Slope	Material	Station	Material	Slope
1v:8h	Muck	10+000	Muck	Remove
1v:8h	Muck	11+000	Muck	Remove
1v:6h	Random	12+000	Gatun	1v:4h
1v:6h	Random	13+000	Random	1v:4h
1v:8h	Muck	14+000	Muck	
*Actual cut slope limits are project station dependant, and the slope transitions are reflected on the Site Plan Drawings				

Figure C-I-7-7 Summary of Temporary Cut Slopes by Project Stationing

The temporary construction slopes were estimated using engineering judgment following an evaluation of existing cut slopes within the vicinity of the project site, and discussions with ACP Geotechnical staff representatives. The proposed slopes were then analyzed using the available soils data to confirm that they met the minimum Factor of Safety criteria. Since the optimum project alignment was selected using clearance requirements for the flatter and more conservative cut slopes, the risks of requiring unforeseen building and utility relocations during final design should be minimized. Due to the highly variable geotechnical conditions at the site, the lack of specific soils data within the vicinity of the proposed project alignment, and the slope clearance issues with the existing infrastructure, it is believed that the use of the flatter slopes and respective higher factors of safety, are appropriate for this level of design.

8. SEEPAGE, DRAINAGE, AND DEWATERING

Based on the limited subsurface data available for the southern limits of the project alignment, the installation of a steel sheet pile seepage cut-off wall is recommended to reduce the quantity of seepage and uplift into the excavation from Lake Gatun. With an effective seepage cut-off wall installed between the Gatun Lake and the lock excavation, the required dewatering system must be adequate in size to remove all remaining interior drainage from within the excavation. Potential sources of seepage include infiltration from the overburden and rock, precipitation from storm events, and overland storm flow run-off.

High uplift pressures are also a major concern, especially in the upstream areas of the excavation where Gatun Lake will have a major influence on the groundwater table. Depending on the effectiveness of the seepage cut-off wall and the rock strata in controlling seepage, a pressure relief system may be required to control uplift pressures within the excavation, and reduce the potential for boils and heaving of the soil slopes.

Based on Corps of Engineers experience with coffered excavations on rock, seepage infiltration from the above sources has been typically managed using a series of collection ditches, sumps, and pumps. Localized seeps in the overburden are normally mitigated with sand and gravel filter blankets to reduce the potential for piping, and upland diversion ditches are normally placed near the top of slope to route surface swales and upland drainage channels to the northern Atlantic approach.

Additional investigations such as insitu permeability tests, pump tests, and a comprehensive piezometric monitoring program are required during future geotechnical studies to evaluate groundwater conditions at the site and identify the most effective dewatering system that will relieve uplift pressures and minimize seepage into the excavation.

9. LIQUIFACTION POTENTIAL

Insufficient soils data exists to perform any type of meaningful analysis on the liquefaction potential of the Atlantic Muck, Gatun, and Random Fill soils. A review of the borings and soils data provided for this report suggest that the physical properties and gradations of the soils within the proposed alignment are not typically susceptible to liquefaction.

These findings are consistent with published literature and studies conducted by the United States Geological Survey (USGS). In a letter sent to the Panama Canal Commission in 1988, the USGS claims to have found no evidence of paleoliquifaction within Lake Gatun and Limon Bay areas. The formal report, which was referenced in the letter, could not be located, and was unavailable for inclusion in this report.

The results of this 1988 USGS study must be located and used as the basis for developing future studies. The results of the USGS study should be compared to the specific soil conditions of the 3rd lane alignment to confirm that this location is representative of the USGS study areas, and that the results of the USGS study are applicable to the 3rd Lane Alignment at Gatun Lock. Additional liquifaction studies should be conducted once the final lock alignment has been established. Subsurface investigations and laboratory testing programs developed for the final alignment design should include the collection of data that will allow for the liquification potential of these soils to be determined more conclusively.

10. FUTURE GEOTECHNICAL DESIGN RECOMMENDATIONS.

The subsurface conditions developed for use in this concept design study were based on borings data collected from previous alignment studies, therefore the number and location of the available subsurface borings and test data covered a very broad area within the vicinity of the Gatun Locks. To best utilize the available data for this alignment study, the borings data was interpolated and extrapolated to develop a representative subsurface model along the proposed alignment. Based on the highly variable conditions at this site, extensive subsurface investigations will be required along the final project alignment to ensure that the project design is based on actual site conditions.

With a project of such large magnitude, a thorough understanding of the subsurface conditions and the soil and rock properties is required to develop a subsurface model that accurately represents the site conditions. Any geologic hazards that may be detrimental to the construction or long term performance of the structure must be identified early in the design stage so that potential design and construction challenges can be addressed.

A significant amount of subsurface exploration, drilling and sampling, insitu testing, and laboratory testing, will be required to ascertain the subsurface conditions. Future investigations should include, but not be limited to the following:

- Geophysical Testing. Geophysical testing should be conducted with its primary focus on determining the highly variable top of rock contours that can not be readily identified from interpolation and extrapolation of the individual borings. Subsurface anomalies identified from the geophysical results should be further explored with test borings.

- Subsurface Drillling. An extensive subsurface drilling program should be conducted following the geophysical work. The borings plan should be developed to confirm the results of the geophysical testing, collect disturbed and undisturbed material samples at the most critical locations and depths, and investigate any subsurface anomalies.

- Soil Sampling. Disturbed and undisturbed soil samples should be collected within and beyond the limits of the temporary slope excavations. The locations of these samples should be obtained at the depths and locations where potential failure planes were identified in the stability analyses. Additional samples should be collected within the limits of excavation to characterize the materials and provide the construction contractor with sufficient data to develop the excavation work plan and select the most efficient earthmoving equioment for the job. Undisturbed samples should be obtained at the approximate depths where potential failure planes may develop. It is likely that the Atlantic Muck and the hydraulic fill soils will be difficult to sample due to their very soft consistency .The Atlantic Muck soils are very sensitive to disturbance, therefore the sampling and collection procedures must be developed to minimize disturbance and compensate for insitu conditions that may not be accurately reflected in the laboratory results. Denison Sampling Tubes may be required to obtain samples of the Gatun Soils.

- Laboratory Testing. Laboratory testing should be performed on numerous disturbed samples to determine the physical and index properties of each soil type. Lab testing should include determination of void ratio, dry density, specific gravity, shear strengths, permeability, and consolidation parameters. Statistical analysis of the laboratory data will be required to assist with the selection of appropriate design parameters. Both drained and undrained triaxial shear testing should be performed on undisturbed soil samples using the appropriate ASTM or Corps of Engineers testing procedures. Consolidation testing is also recommended to obtain a better understanding of the soils load history and behavior under load, permeability, and rate of consolidation.

- Insitu Testing. Due to the sensitivity of the Atlantic Muck soil and the potential difficulty of obtaining quality undisturbed samples, insitu testing of the Muck using a Vane Shear device should also be pursued. The results of the insitu testing should be compared to the results of the undrained triaxial tests.

- Groundwater Monitoring. Open standpipe piezometers or automated vibrating wire systems should be installed in both the soil and rock strata at select elevations to accurately assess the groundwater conditions at the site. It is essential that a thorough understanding of the groundwater and potential uplift conditions be determined, and that the maximum groundwater elevations be identified for final design. A data collection and observation schedule must be developed with sufficient frequency to address any seasonal changes in to the groundwater elevation that may result from climatic conditions or pool fluctuations within Lake Gatun. Hydrostatic uplift and artesian conditions that may result from Lake Gatun impoundment should also be investigated and monitored thru the use of piezometers installed to sense certain zones such as the more permeable soil layers, the weathered rock zone, and the deeper rock strata.

The subsurface investigations should be planned and conducted in several phases as the need for additional data is identified during the final design. Because the Atlantic Muck was reported to be very sensitive, undisturbed sampling and testing procedures must be conducted in such a manner to address or compensate for sample disturbance.

Due to the size and magnitude of the project site, and the volume of data that will be generated from the subsurface investigations, the use of digital terrain models which incorporate boring data and subsurface conditions into the modeling software will be very valuable in reviewing subsurface cross-sections and profiles at various locations during the design analysis. Proficient use of such computerized modeling systems will allow for rigorous evaluations and analysis and assure a more comprehensive study of the highly variable subsurface conditions at this project site.

11. LISTING OF TECHNICAL REFERENCES

- Final Report on Modified Third Locks Project, Part II – Design, Chapter 3 – Geology; Department of Operation and Maintenance, Special Engineering Division, Balboa Heights, Canal Zone. December 1943
- Final Report on Modified Third Locks Project, Part II – Design, Chapter 5 – Foundations and Slopes; Department of Operation and Maintenance, Special Engineering Division, Balboa Heights, Canal Zone. December 1943
- National Highway Institute, Publication No. FHWA HI 97-013; November 1998
- Ismithian Canal Studies; Third Locks Project, Appendix 3, Report of the Governor of the Panama Canal; 1947
- Geotechnical Advisory Board, Meeting No. 9 – Gatun Dam Seismic Studies, Letter Report; Panama Canal Commission; September 1999
- Design Earthquakes for the Evaluation of Seismic Hazard at the Gatun Dam and Vicinity – Final Report; U.S. Geological Survey; August 1999
- Geotechnical Advisory Board, Meeting No. 8 – Gatun Dam Seismic Studies, Letter Report; Panama Canal Commission, July 1998
- Soil Liquifaction Website, Department of Civil Engineering, University of Washington, U.S.A.
- Correlations of Soil Properties; Carter and Bentley, Pentech Press, 1991
- Report on Foundation Investigations – Proposed Rock and Earth Fill Embankment Across Trinidad Arm of Lake Gatun, Panama Canal Zone, for Tudor Engineering Co; Shannon and Wilson, Inc, Seattle Washington, March 1963

Section II - Geology and Rock Foundations

12. SUBSURFACE INVESTIGATIONS

12.1. 1930's Drilling

An extensive drilling program was undertaken in the 1930's for the design of the new third lock. Over 400 borings were drilled along the alignment, which is now called Alignment A-1 in the current studies. These borings are numbered as a G- series. Information from these borings was available only as typewritten logs or as stick logs on construction cross sections.

12.2. 2001 Investigation

The borings drilled in 2001 were drilled by ACP forces and by a contract drilling company, Swissboring. The ACP provided inspectors for their borings and Swissboring used contract geologists to inspect their borings.

The borings drilled in 2001 were concentrated along two alignments. Alignment A-1 followed the third lock excavation of the 1940's and Alignment A-2 was adjacent and parallel to the existing locks. A total of 43 borings were drilled 16 for Alignment A-1 and 27 for Alignment A-2

The numbering system used was:

- Alignment A-1 ACP borings - TA1-1 to TA1-5
- Alignment A-1 Swissboring borings TA1C-1 to TA1C-11
- Alignment A-2 ACP borings - TA2-1 to TA2-10
- Alignment A-2 Swissboring borings TA2C-1 to TA2C-17

(TA is abbreviation for *Tercer Atlantico* or Third Atlantic)

Table C-II-GG-1 is a summary of the 2001 borings. Copies of all the 2001 boring logs will be included as an exhibit in the final April 2003 submittal.

The modification of Alignment A-2 that is being used in this report lies between the original Alignments A-1 and A-2 so only a few of the borings actually are within a reasonable proximity of the revised Alignment A-2. A considerable amount of interpolation was necessary to use this information in the current study.

The locations of the 2001 borings in relation to the proposed modified Alignment A-2 lock footprint are shown on drawing ACP-R-10/1.

13. TESTING

13.1. 1930's Testing

There was a large amount of rock testing done for the original third lock design in the 1930's. Available reports only summarize the data. Table C-II-GG-2 has appeared in several of the 1930's Third Lock reports and in the 1947 sea level canal study. It summarizes the general rock strength parameter recommendations that resulted from the testing. No specific 1930's laboratory testing results from the Gatun Formation were available for this report.

13.2. 2001 Investigation

Rock testing done by the ACP during the 2001 program consisted of rock properties and unconfined pressure strength testing. Sixty-four test samples from 19 borings were tested for density, moisture content, modulus of elasticity and unconfined compression. Table C-II-GG-3 is a summary of the detailed data from the program. Table C-II-GG-4 organizes the data by rock type. The results of this testing have also been plotted on rock profiles along the baseline (Figures C-II-GG-6 to C-II-GG-8). These profiles show the test values plotted at the actual elevation and Northing location projected on the Alignment A-2 baseline.

Some pressure testing for rock permeability was also done. Tests were completed on nine borings with 49 successful tests out of 85 attempts. Table C-II-GG-7 summarizes the pressure testing.

14. SITE CHARACTERIZATION

14.1. Boring Data Used

The logs of the 2001 drilling program provide the most complete and reliable data and were used as the primary source of information in evaluating the site. Information from the 1930's drilling was available only in old typed boring logs and as graphical logs in the old construction drawings. This information was used only to help establish the Top-of-Sound Rock (TSR) surface and the elevation of the surface of the conglomerate strata used as marker bed as described below. Since conglomerate is a distinctive rock type we had confidence that the classification of strata as conglomerate in these older borings was accurate. If information from the older borings seemed vague or contradicted the information from the 2001 drilling logs it was not used,

14.2. Methodology

The 1930's borings program concentrated on the present Alignment A-1 and the 2001 borings concentrated on Alignments A-1 and A-2 so there are not a significant number of borings ideally located on the modified Alignment A-2 being studied in this report. In order to determine a top of sound rock surface and evaluate the subsurface conditions, information from the borings on the two alignments had to extrapolated across to the revised Alignment A-2.

The extrapolation was accomplished using the Surfer modeling program from Golden Software Co. The program was used to establish a TSR surface across the site (Figure C-II-GG-3).

The Surfer program can use several different mathematical algorithms to create surfaces. Since the program is purely a mathematical operation, the data was compiled using several of the algorithms to derive a TSR surface that seemed most reasonable, based on our geological knowledge of the site.

There are several methods of interpolation available in the program. The Surfer literature states that Kriging is one of the more flexible methods and is useful for gridding almost any type of data set. The literature recommends Kriging as the default gridding method. The contour surfaces in this report were produced using the Kriging gridding method.

The data was also analyzed with some of the other gridding methods and produced results consistent with Kriging within the lock area. Results did vary in the southern approach where boring information is more widely spaced. To produce a surface that seemed topographically and geologically reasonable in this area, points of elevation were added to the TSR data file. These points caused the program to adjust the rock surface to more closely conform to the topography in the south approach area.

In addition, the Surfer program was used to develop a contour surface on top of the conglomerate strata used as a marker bed to analyze the structural geology of the site (Figure C-II-GG-4).

To further the analysis, information from the Surfer program was used in the associated Grapher program to develop profiles and cross sections. The conglomerate marker bed was used as an aid in correlating information from widely spaced borings and projecting it on to the baseline profile.

Table C-II-GG-9 is a table of the metadata used by the Surfer program. The order in which the borings appear in this table is generally the order of importance in which the boring information was used. The process was iterative, first only the 2001 borings were considered and then groups of the 1930's boring were considered. As can be seen in the table, less useful information was gathered from the older G series borings toward the bottom of the table.

The TSR contour information was also used in the InRoads program to develop rock excavation quantities. In order to create the surface Surfer first produces a file of grid points, called a grid file (*.grd). The Surfer program then uses information from the grid file to produce a contour surface file (*.srf).

The Surfer grid file was converted into a file format compatible with InRoads. The Inroads program then used this grid file to recreate the TSR surface using its own surface creation feature. The TSR surface produced by InRoads closely matched the one produced by the Surfer program.

14.3. Physiography

The top of rock surface at the site is variable and does not trend with the top of ground as is typical in most areas.

There were four separate geologic phases to the development of the top of rock surface at Gatun:

- The bedrock (Gatun Formation) was formed by the consolidation of volcanic detritus that had been deposited in a shallow marine environment
- The sea level receded and bedrock became subject to erosion, forming a typical surface topography of small hills separated by valleys. The bedrock weathered forming a layer of residual soil that conformed to the surface of the rock (Gatun Overburden).
- The sea again encroached, again submerging the ground surface in a shallow marine environment. Sediment deposition occurred again. The material deposited was a soft highly organic, highly plastic clay and silt (Atlantic Muck). This deposition resulted in a relatively flat ground surface that covered the original hill and valley topography.
- The land surface again emerged above sea level and erosion processes continue resulting in the present topography.

These geologic processes have resulted in a rock surface consisting of high 'knobs' of rock separated by erosion channels that have been filled in by the Atlantic muck creating the highly variable top of rock elevation presently seen at the site.

14.4. Stratigraphy

The rock at the site is all part of the Gatun Formation. In general, as described in previous reports, the Gatun is made up of argillaceous and tuffaceous sandstones, volcanic tuffs and conglomerates. The strata were formed in a marine environment but the primary source of material from deposition was the sporadic volcanic eruptions that marked the period.

Figure C-II-GG-1 is a regional geologic column used in the 1930's Third Locks studies that shows the Gatun Formation in relation to overall Canal Zone stratigraphy.

For the purposes of this report a general geologic column has been developed (Figure C-II-GG-2) for the project area based, primarily, on the 2001 subsurface investigations.

The strata have separated into 7 units, which are described on Figure C-II-GG-2.

- Unit 1 is the recent soil overlying the older material. It consists of residual silt, clay and spoil from the Third Lock excavation.
- Unit 2 is the Atlantic Muck, which was deposited when the sea inundated the site. It is present only at elevation very near sea level. Drawing ACP-R-10/3 is a plan produced by the ACP showing the areal distribution of the Atlantic Muck.
- Unit 3, called the Gatun Overburden, is a regolith of residual soil that had formed above the underlying rock before the sea inundated the site.
- Units 4, 5, 6, and 7 are part of the Gatun Formation bedrock.
- Unit 4 is most recent bedrock at the site; it consists of tuffaceous sandstone. Generally the bedding is massive but there are areas of finely laminated deposits within the strata. These laminar deposits have been designated Unit 4A.
- Unit 5 is the conglomerate that was used as a marker bed in determining stratigraphy and structure.
- Unit 6 is a pumiceous sandstone that grades into Unit 7, a pumiceous tuff.

Drawing ACP-R-10/2 is a geologic profile along the baseline. Since creating the profile involved considerable interpolation from distant borings the details are general in nature. The intent of this profile is to provide a general framework of the stratigraphic conditions.

Older general columns from the Third Lock design also numbered the stratigraphic units. The custom at that time was to number the units with the lowest number at the bottom increasing the numbers upward to the youngest. In these columns the Unit 5 conglomerate is designated Unit 10. The Unit 4 tuffaceous sandstone appears to have been separated into two units, Units 11 and 12 on the old columns.

14.5. Structural Geology

The structure of the strata at the site is relatively simple. As indicated by the dip of the conglomerate marker bed on the geologic profile, Drawing ACP-R-10/2, there is a homoclinal dip to the north. The elevation of the conglomerate bed drops from an elevation of about 28 m at the south end to an elevation of -32 m at the north end to. The dip is at it's greatest between N 1,024,800 and N 1,026,000. In this area the dip is to the north-northwest at about 2.5°.

An anomalous feature described as an 'erosional' surface' occurs at the south, Lake Gatun, end of the alignment. This feature made correlation of boring information at this end of the alignment difficult. The area will require more investigation in future studies

14.6. Phreatic Conditions

The only available information on water levels at the site are water level readings taken in the 2001 borings. These readings were taken at the completion of the drilling so they do not give an accurate representation of groundwater conditions.

However, these readings have been used to construct the contours of a phreatic surface (Figure C-II-GG-5). The phreatic surface profile is also shown on Drawing ACP-R-10/2. As expected these readings closely follow the ground surface.

15. ROCK ENGINEERING CHARACTERISTICS

15.1. Summary of 2001 Testing

As noted previously, 64 test samples from 19 borings were tested for density, moisture content, modulus of elasticity and unconfined compression. Table C-II-GG-4 is a summary of the results separated by rock type. Figures C-II-GG-6 to C-II-GG-8 show these results plotted against the geologic profile. Examination of these figures indicates that there is no trend in rock engineering properties and no correlation between the geologic units and rock strength. The distribution of test values on these profiles confirm the heterogeneous nature of the Gatun formation as described in the 1930's geological reports.

Tables C-II-GG-5 and C-II-GG-6 are summaries of the all the testing for sandstone and conglomerate respectively. The small graphs on these tables were developed to look for any correlation between elevation of sample and strength values. The graphs show there is no strong correlation between these two properties.

15.2. 1930's Testing

A considerable amount of testing, both insitu and in the laboratory, was done in for a 1938 study. The work is summarized in the "Final Report on Modified Third Locks Project, Part II Design, Chapter 5 – Foundation and Slopes", Dec. 1943". Exhibit C-II-GG-A is an excerpt from the report covering "Foundation Studies".

15.3. Shear Strength

One direct shear test on rock was performed as a part of the 2001 testing. The test was run on remolded tuffaceous sandstone. Figure C-II-GG-10 shows the results of this test.

Some insitu testing done in the 1930's is described in Exhibit C-II-GG-A

Table C-II-GG-8 summarizes information on the 1930's testing as well as the one rock shear test performed after the 2001drilling program.

15.4. Bearing Capacity

The only information available on bearing capacity is in summaries of test results in some of the 1930's design reports. These narratives indicate that a bearing capacity of 1.9 MPa is a reasonable value for the Gatun formation rock. This value was included in the summary in Table C-II-GG-2.

In the report "An Outline of Canal Zone Geology in Relation to the Third Lock Project" published by Donald F. MacDonald in 1941 there is a description of insitu load bearing tests done on the Gatun Formation in 1908 for the existing lock. Following is an excerpt from MacDonald's report; metric conversion of units has been added:

> "Though comparatively soft, the Gatun formation is quite strong enough to support any lock structure that may be built upon it. About 1908 loading tests were made by leveling off one square foot, then loading it. A settlement of 0.132 of an inch [3.35mm] was noted after 37,128 pounds [16 841 kg] of load had been added. This was believed due to crushing of loose surface grains. The final load of 77,350 pounds (38 2/3 tons) per square foot [3.7 MPa] produced no further measurable settlement. Several individual test pieces failed from 250 to 1625 pounds per square inch (18 to 117 tons per square foot) [1.71 to 11.2 MPa]. If these test pieces had been embedded in the solid rock from which they were taken their resistance to load would have been much higher. Recent field and laboratory compression tests on the various beds of this formation have been made by the Soils Mechanics Laboratory, which confirm this statement."

Additional information on bearing tests done in the 1930's is included in Exhibit C-II-GG-A

15.5. Permeability

Results of the pressure testing described above indicate the bedrock has a relatively high coefficient of permeability (K). Test values ranged from a high of K= $1.17E^{-3}$ to a low of K= $4.42E^{-6}$. Table C-II-GG-7 is a summary of the test results. Figure C-II-GG-9 is a plot of the test values on the geologic profile. The plot does not show any areas of high permeability trends that cross the entire site. There are some indications of a local area of high permeability in the laminated areas of the Unit 4 tuffaceous sandstone.

16. ROCK DESIGN ROCK PARAMETERS

16.1. Discussion

Rock strength parameters were derived from an examination of the recent laboratory test results performed in 2001 and descriptions of previous laboratory and field-testing performed in the 1930's. Engineering judgement had to be used in interpreting this data since some of the available information does not include the type of tests that would be ideal for design. The values discussed below are appropriate for this level of study, but will need early confirmation in the next phase of design.

The bearing capacity was determined to be 1.9 MPa. This is the same value used previously in the 1930's third lock design. As described in the excerpt from the 1943 report, Exhibit C-II-GG-A, it was noted that the plate bearing tests of the 1930's sustained this load for several days. Also, lab test results of Gatun sandstone taken from borings closest to the proposed lock (TA2C-3 and 4) had an average unconfined compressive strength (UCS) of 5.1 MPa.

To determine the design Young's Modulus, a modulus ratio was determined from the plate bearing tests. A design Young's Modulus was determined to be 344.7 MPa. This value is a reasonable estimate for design when compared with the average laboratory value of 1317 MPa based on the engineering considerations discussed below.

The guidelines in paragraph 4-24 'Selection of Design Moduli' in EM 1110-1-2908; Rock Foundations were used in Modulus selection. These guidelines state "...the moduli values used for design purposes are selected rather than determined. The selection process requires sound engineering judgment by an experienced team of field and office geotechnical professionals..." and that "...the selection process should involve an integrated approach in which a number of methods are used..."

There are two sources of data that could be used to in the modulus determination: the plate bearing tests on insitu rock from the 1930's (described in Exhibit C-II-1 on page 19) and the laboratory testing of rock core from the 2001 drilling program.

Since the 1930's tests involved insitu plate bearing testing with plates up to a diameter of 1.02 m, results from these tests seemed the most appropriate to use. To determine the design Young's Modulus, a modulus ratio was determined from the plate bearing tests. The modulus ratio was determined to be 180, which is typical for soft rock. After multiplying this value by the bearing capacity (180 x 1.9 MPa), the design Young's Modulus was determined to be 344.7 MPa.

The results of the core testing from the 2001 drilling were also considered. Results from Borings TA2C-3 and 4 were studied since these are on or very near the proposed alignment. An empirical assessment based on engineering judgment of which samples were most representative of the foundation rock resulted in an average laboratory E value of 1317 MPa. If this type of laboratory data is the only available information standard engineering practice is to use a reduced value for design. The reduction factor is based on rock mass characteristics of the strata and influence of scale between the small test sample and the characteristics of a large volume of rock that will be reacting to a mass concrete structure.

Typical reductions are 50% to 80% of the laboratory value or:

E_{rock_mass}/E_{lab} = 0.20 to 0.50 for fair quality rock

Using this equation to compare the recommended E value to the laboratory E value yields:

E_{rock_mass}/E_{lab} = 344.7MPa/1371MPa = 0.26

This comparison shows that the recommended value of 344.7 MPa is conservative but is reasonable when compared to the laboratory values.

The average unit weight from the lab samples of borings TA2C-3 and 4 was determined to be 1912 Kg/m^3.

For sliding friction, only one direct shear test on Gatun sandstone was performed as part of the 2001 testing (Figure C-II-GG-10). This test yielded an angle of internal friction of Phi =31 degrees and a cohesion of C=21 KPa.

To account for the large-scale roughness of an actual lock monolith foundation, the cohesion was raised to 70 KPa. The increase was based on a process of engineering judgment:

- Typically when no actual shear test data is available an estimate of peak cohesion can be made by using 10% to 15% of the unconfined compressive strength (UCS).

- As shown in Table C-II-5 the average UCS strength was 6.51 MPa. However, to keep the values conservative, the estimate was based on the lower end of the range of all the UCS test values – 2MPa. 10% of this value is 0.2MPa or 200 KPa.

To increase the level of conservatism only 25% of this value, 50 KPa was added to the test value of 21 KPa. The reduction to 25% is based the assumption that only 25% of the sliding surface is intact rock and the remainder of the surface consists of fractured rock, weak seams or other alterations. This total equals 71 KPa. Since 71 infers a level of precision that is not warranted 70 KPa will be used.

The resulting design shear strength thus becomes: Phi=31 degrees and C=70 KPa.

16.2. Recommended Rock Design Parameters

Table C-II-5-1 Sliding Friction

Phi ϕ	Cohesion	Cohesion
degrees	KPa	psi
31°	70	10

E_m	344.7 MPa
Bearing Capacity	1.9 MPa
Unit weight	1900 kg/m^3

16.3. Slopes

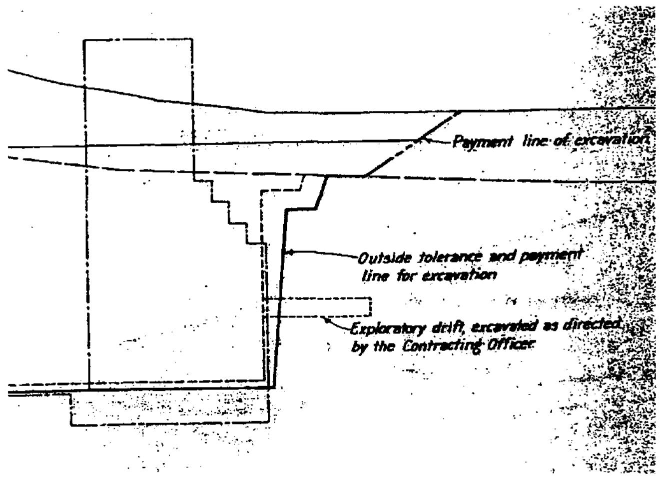
As discussed in the previous section above, there is not a lot of subsurface information along the revised Alignment A-2 baseline. Analysis of the available engineering characteristics from the 2001 testing indicates that the rock is heterogeneous and there are no strata that exhibit unique engineering properties that would require special benching or other variations in cut slope design.

The recommended slopes within the lock excavation are approximately the same design as the slopes used in the excavation for the 1940 Third Lock cut. The current study and previous studies verify that the geology along both Alignment 1 and 2 is very similar. Since the cut design along the 1940's excavation is based on much more geologic information and analysis, and the fact that these cuts are still stable after 50 years, we feel it is appropriate to use the 1940's slope designs for the current study.

The lock cut design shown in Figure GG-11 is based on the typical sections shown in

Folio 1 of 2
Panama Canal
Third Locks Project
Drawings for Construction
New Gatun Locks

One of these typical sections is shown on the figure below.



These sections show a payment line for excavation that the slope design in Figure GG-11 is based on. The sections also show a dashed vertical slope line, this line is the inside tolerance for excavation. Paragraph 4-03 (3) of Specifications for Excavation, New Gatun Locks Structure, 1940 states:

> *(3) Class B Excavation.- Where Class B excavation is made in sound rock, the contractor will be permitted to excavate vertically a distance of 60 feet. In all such cuts in excess of 60 feet, above the 60 foot elevation, the contractor shall maintain of each lift approximately parallel to, but in no case steeper than, the slopes of the finished cross-section as shown on the drawings. This slope may consist of vertical cuts and horizontal berms arranged in a manner approved by the contracting officer. The slopes of the finished excavation shall conform to the slopes shown on the drawings.*

The reason these cuts within the lock excavation can be built at a steeper slope is that they are essentially temporary cuts. After the lock concrete is completed the area between the back of the lock structure and the face of the cut is backfilled. In the interest of economics, the slope is kept as steep as feasible to reduce the amount of material needed later to backfill the area.

The schedule for the current project anticipates that the lock excavation will be open for about three to four years. After completion of the lock concrete the area between the back of the lockwalls and the face of the cut will be backfilled with Controlled Low Strength Material (CLSM).

The typical section shows a 3 m wide horizontal berm, 16 m above the bottom of the excavation. The purpose of this berm is to keep any material eroded from the weathered rock near the top of the cut from eroding and sloughing into the excavation. The height and width of these berms are usually adjusted during construction.

In view of these factors and the level of this study, two basic typical slope designs are recommended, one design for the area within the lock excavation and one for the approach areas. These designs are shown on Figures C-II-G-11 and C-II-G-12.

These typical sections are similar to the design in the Third Lock construction drawings from 1940.

The design within the lock excavation has been modified to lower the berm height to 8 m as recommended by the ACP.

In future studies, when more subsurface information is available, the design may be modified to account for variations in the strata.

17. RECOMMENDATIONS FOR FUTURE INVESTIGATIONS

17.1. Additional subsurface investigations

If the study of this alignment advances to the feasibility stage an additional program of subsurface investigation will be needed. Areas of particular interest would be:

- More complete coverage of the proposed alignment concentrating on lock walls, particularly gate monoliths.
- The areas within the Unit 4 tuffaceous sandstone where highly laminated strata were encountered
- The erosional surface encountered at the south end of the alignment.

17.2. Laboratory Testing

In conjunction with the additional subsurface investigation more laboratory rock testing will be needed. Undisturbed samples through planes of weakness such as joints and the clay filled laminations will be especially important. Shear strength along these planes, in many cases; determines the stability of the entire rock mass.

17.3. Geophysical Investigations

Since the TSR is highly variable it cannot be accurately delineated by core borings alone. Geophysical exploration techniques in conjunction with additional borings would be able to better define the TSR.

It appears that the seismic velocities of the rock would vary from the overlying soil so seismic refraction or reflection techniques should be effective. Other geophysical methods such as resistivity, acoustic soundings or ground penetrating radar may also be effective.

The scope of any contract for geophysical investigation should be written so that choosing the appropriate geophysical methods of exploration is the responsibility of the contractor. The scope should only state the goals of the investigation.

17.4. Periodic water readings at well points

Most of the 2001 borings have had well points installed in them to determine ground water levels. These well points should be read on a regular basis to determine average phreatic conditions and seasonal variations. Since the borings are spread over the whole site knowledge of these conditions will be important for Alignment A-2 or any alignment proposed for future study

Appendix C – Geotechnical Investigations, Analyses and Designs
Part I – Soils
Tables

Table C-I-1	Summary of Soils Data from Previous Geotechnical Studies
Table C-I-2	Density - Gatun Overburden Soils
Table C-I-3	Density – Atlantic Muck Soils
Table C-I-4	Consolidated Undrained Shear Strength – Atlantic Muck Soils
Table C-I-5	Consolidated Drained Shear Strength – Atlantic Muck Soils
Table C-I 6	Tabular Summary of Concept Design Parameters

US Army Corps of Engineers Pittsburgh District

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3rd Lane - Concept Design Study

Summary of Published Soils Data and Document Sources

Computed By: D. Carlson Date Jul-02

Checked By: CES

Soil Description	USC	Sample Type	Unit Wt. Dry	Unit Wt. Sat	Permeability	"Q" Triaxial Shear Unconsolidated-Undrained		"R-bar" Triaxial Shear Consolidated-Undrained		"S" Direct Shear Consolidated-Drained		Bromhead Ring Shear Peak		Bromhead Ring Shear Residual		Unconfined Compression	In-situ Vane-Shear	Reference Number	Date	Data Source		
Symbol			γ dry	γ sat	k	c	Ø	c	Ø	c	Ø	c	Ø	c'	Ø'	q_u	c'					
Unit			kN/M³	kN/M³	cm/sec	kN/m²	Degrees	kpa	Degrees	kN/m²	Degrees	kN/m²	Degrees	kN/m²	Degrees	kN/m²	kN/m²					
					Range	Range			Range													
Atlantic Muck	CH	Undisturbed				4.6 - 106.7	0°	0	16° - 18°								180 - 580		1963	Shannon-Wilson		
Black Muck	CH								17°										1947	Isthmian Canal Study	A-12	S-IV
Atlantic Muck	CH					4 - 54	0°	16	28°										1998	Geotech Advisory Board	Letter	Mtg 8
Atlantic Muck					1.00E-09																App H	
Hydraulic Fill						5	0°	21	27°										1998	Geotech Advisory Board	Letter	Mtg 8
Gatun Silty Sand	SM	Remolded								21	31°	3	29°	9	20°				2002	ACP - Geotechnical Lab	Test Result Summary	

US Army Corps of Engineers
Pittsburgh District

Panama Canal Authority
Atlantic Locks
3rd Lane - Concept Design Study

Dry, Moist, and Saturated Densities
Gatun (SM) Overburden Soils

Computed By: D. Carlson Jul-02

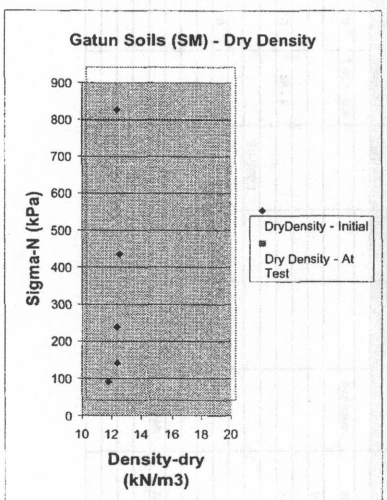
Checked By: CES

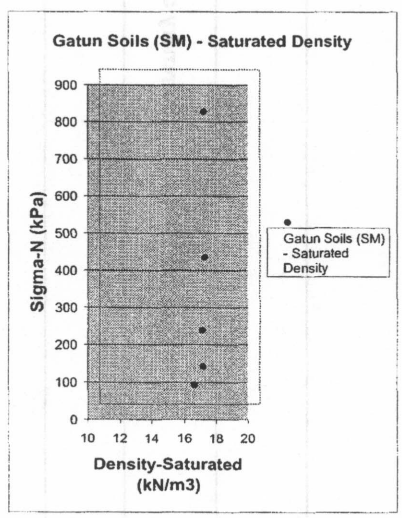
Boring:	TA2-4
Sample Type:	Disturbed - Compacted to approximate insitu density
Sample Depth:	28 - 30 m
Formation:	Gatun Tuffaceous Sandstone
Soil Type:	Residual
Description:	Silty Sand
Unified Soil Classification:	SM

$$\gamma\,sat = \frac{G + S\,e}{1 + e}\,\gamma\,water$$

Normal Stress	kpa	σ n	50.29	99.47	197.29	393.47	785.3		
INITIAL TEST	**Units**	**Symbol**	**1**	**2**	**3**	**4**	**5**	**Median**	**Stn Dev**
Water Content	%	ω	45.6	45.5	44.5	44.3	41	44.5	1.9
Density (dry)	kN/m^3	γ dry	11.5	12.1	12.0	12.3	12.1	12.1	0.3
Saturation * (Before Test)	%	S	107	117	112	115	106	112.0	4.8
Saturation** (100%)			100	100	100	100	100		
Void Ratio	-	e	1.1	0.93	0.95	0.9	0.92	0.9	0.1
Specific Gravity	-	G	2.3	2.3	2.3	2.3	2.3	2.3	0.0
Unit Weight - Water	kN/m^3	γ water	9.8	9.8	9.8	9.8	9.8	9.8	0.0
computed									
Density (Saturated)	*kN/m^3*	*γ_{moist}*	*15.9*	*16.4*	*16.3*	*16.5*	*16.4*	*16.4*	*0.3*

Saturation * - actual "initial" and "at test" values exceeded 100%; vaues are suspect
Saturation ** - 100% assumed for Saturated Unit Weight Computation





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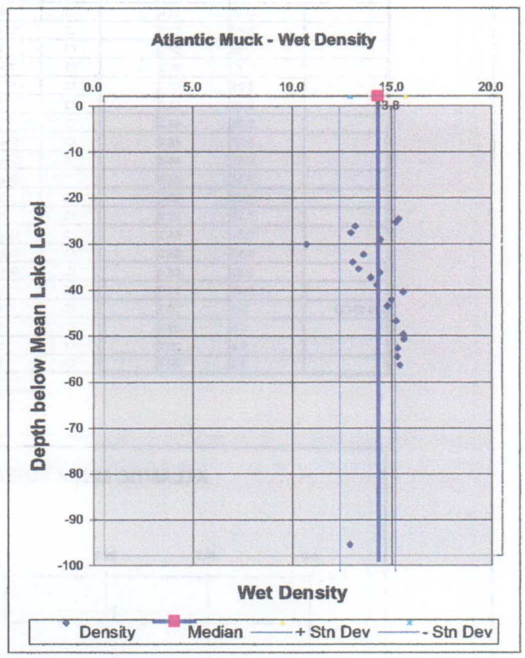
3rd Lane - Concept Design Study

Density
Atlantic Muck Soils

Computed By: D. Carlson Jul-02
Checked By: CES

a and parameters below were summarized from Shannon & Wilson (x); Table A-1

Boring	Sample No.	Sample Depth (BOH)	Sample Elevation	USC	Color	Sample Density (Ave) γ sat
		meters	msl			kN/m³
A-25-AXU	S-3	26.6	No Data	CL	Gray-Brown	14.8
	S-4	27.4		CL	Gray-Brown	14.7
	S-5	28.2		CL	Gray	12.6
	S-7	29.7		CL	Gray-Brown	12.4
	S-9	31.2		CL	Tan/Gry-Brn	13.9
	S-10	32.1		CL		10.2
	S-13	34.4		CL	Brown	13.0
	S-15	36.0		CL	Brown	12.5
	S-17	37.5		CL	Brown-Gray	12.8
	S-18	38.3		CL	Gray	13.8
	S-19	39.3		CL	Tan - Gry	13.4
	S-21	41.0		CL	Tan	13.7
	S-23	42.5		CL	Tan	15.0
	S-25	44.2		CL	Gray	14.4
	S-27	45.6		CL	Gray-Brown	14.2
	S-31	48.8		CL	Gray-Brown	14.7
	S-34	51.7		CL	Gray-Brown	15.0
	S-35	52.8		CL	Brown	15.0
	S-51	54.7		CL	Gray-Brown	14.7
	S-53	56.5		CL	Gray-Brown	14.7
	S-55	58.4		CL	Gray-Brown	14.9
UP-25-CU	S-6	108.5		CL	Gray-Black	10.3
DN-25-CU	S-2	97.5		CL	Black	12.3
	S-2	97.5		CL	Black	12.3
	S-2	97.5		CL	Black	12.3
	S-2	97.5		CL	Black	12.3



γ sat	**Median**	13.8	kN/m³
	Std Deviation	1.4	
	Max	15.0	
	Min	10.2	

Panama Canal Authority
Atlantic Locks
3rd Lane - Concept Design Study

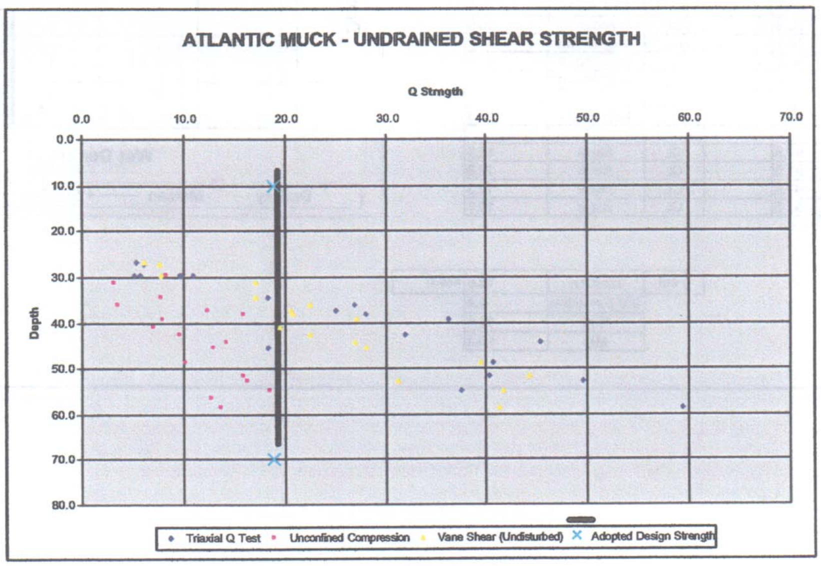
COMPUTATION

Determination of Unconsolidated Undrained "Q" Shear Strength
Atlantic Muck Soils

Computed By: D. Carlson Date:
Checked By: Date:

Test data and parameters below were summarized from Shannon & Wilson Report (x); Table A-1 and A-3

Boring	Sample No.	SampleDepth (BOH)	Sample Elevation	USC	Color	Unconsolidated Undrained	Unconfined Compression	Unconfined Compression	Vane Shear
						Undisturbed	Disturbed	Disturbed	Undisturbed
		0.0	23.7744			Q	qu	Q	Q
		meters	meters			kN/m²	kN/m²	=qu/2	kN/m²
A-25-AXU	S-3	26.6		CL	Gray-Brown	5.4			6.1
	S-4	27.4		CL	Gray-Brown	6.1			7.6
	S-7	29.7		CL	Gray-Brown	10.9	16.9	8.4	7.8
	S-9	31.2		CL	Tan/Gry-Brn		6.3	3.2	17.1
	S-13	34.4		CL	Brown	18.3	15.6	7.8	17.0
	S-15	36.0		CL	Brown	27.0	7.1	3.5	22.4
	S-17	37.5		CL	Brown-Gray	25.0	24.8	12.4	20.6
	S-18	38.3		CL	Gray	28.0	32.1	16.0	20.9
	S-19	39.3		CL	Tan - Gry	36.2	16.1	8.1	27.2
	S-21	41.0		CL	Tan	19.6	14.1	7.0	19.5
	S-23	42.5		CL	Tan	31.9	19.3	9.6	22.4
	S-25	44.2		CL	Gray	45.4	28.4	14.2	27.0
	S-27	45.6		CL	Gray-Brown	18.4	26.0	13.0	28.1
	S-31	48.8		CL	Gray-Brown	40.6	20.3	10.2	39.4
	S-34	51.7		CL	Gray-Brown	40.2	31.8	15.9	44.1
	S-35	52.8		CL	Brown	49.6	32.6	16.3	31.2
	S-35	52.8		CL	Brown	49.6	32.6	16.3	31.2
	S-51	54.7		CL	Gray-Brown	37.4	37.2	18.6	41.7
	S-53	56.5		CL	Gray-Brown		25.7	12.9	
	S-55	58.4		CL	Gray-Brown	59.4	27.6	13.8	41.3
	S-63	65.7		CL	Gray				
DN-25-CU	S-2	29.7		CL	Black	9.9			
	S-2	29.7		CL	Black	9.7			
	S-2	29.7		CL	Black	5.7			
	S-2	29.7		CL	Black	5.1			



US Army Corps of Engineers Pittsburgh District

Panama Canal Authority
Atlantic Locks
3rd Lane - Concept Design Study

Determination of Consolidated Drained "R-bar" Shear Strength
Atlantic Muck Soils

Computed By: D. Carlson July-02

Checked By: CES

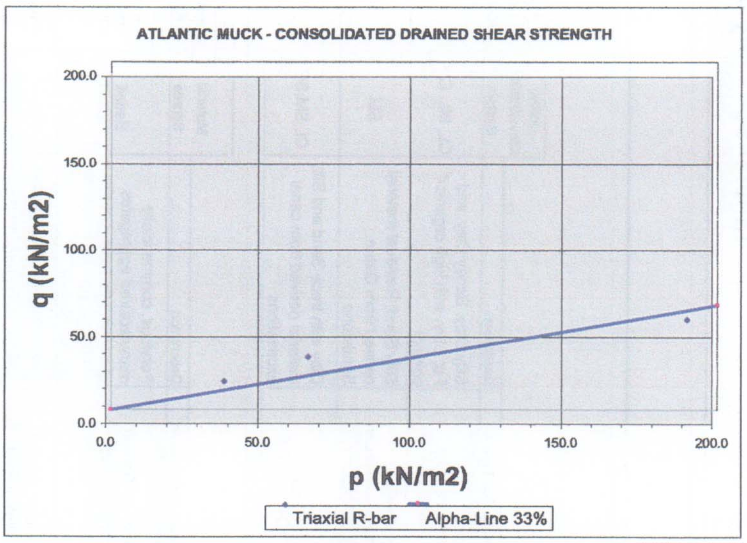
Test data and parameters below were summarized from Shannon & Wilson (x): Table A-1 and A-3

Boring	Sample No.	SampleDepth (BOH)	Elevation	USC	Color	Sample Density (Ave)	Consolidated Drained	Confining Pressure	Pore Water Pressure	Deviator Stress at Failure	p	q	α	α
							Undisturbed							
		0				γ sat	R-bar	σ_3	μ_0	$\sigma_1-\sigma_3$	p	q	alpha	Alpha
		meters	msl			kN/m³	kN/m²	kN/m²	kN/m²	kN/m²	kN/m²	kN/m²	radians	Degrees
A-40-AXU	S-3	26.0604		CL	Gray-Brown	15.5	114.0	262.4	0	227.4	376.1	113.7	0.293616	16.8
	S-7	29.1084		CL	Gray-Brown	14.4	51.9	20.7	0	32.8	37.1	16.4	0.416667	23.9
	S-7	29.1084		CL	Gray-Brown	15.3	16.4	137.9	0	103.9	189.8	51.9	0.267105	15.3
	S-9	30.6324		CL	Black	11.6	29.0	34.5	0	60.3	64.6	30.2	0.436627	25.0

Plot Data - Calculated at 33rd Percentile			
p	q	Alpha	Alpha
0	0	0.293351	16.8
200.0	60		

Relationship of C and Phi to K_f Line on p-q Diagram
$\sin \phi = \tan \alpha$
$c = \dfrac{a}{\cos \phi}$

R-bar Strength				
a	α	c	ϕ	ϕ
kN/m²	Deg	kN/m²	Deg	Radians
0.0	16.8	0.0	17.6	0.3069



US Army Corps of Engineers Pittsburgh District

Panama Canal Authority
Atlantic Locks
3rd Lane - Concept Design Study

Tabular Summary of Concept Design Parameters

Computed By: D. Carlson   Date Aug-02
Date   Checked By: CES

	Insitu Soil Type	Description	Unified Classification System	Average Density γ_{dry} kN/m³	Average Density γ_{sat} kN/m³	Unconsolidated Undrained Q-Strength c kN/m²	Unconsolidated Undrained Q-Strength ϕ Degrees	Consolidated Undrained R-Strength c kN/m²	Consolidated Undrained R-Strength ϕ Degrees	Consolidated Drained "S" (R-bar) Strength c kN/m²	Consolidated Drained "S" (R-bar) Strength ϕ Degrees	Permeability k m/sec
Insitu Soils	Atlantic Muck	Silty Clay, Sandy Clay, and Fat Clay, with high organic content	CL, ML, CH	7	14	19	0	8	15	0	17	1.00E-11
	Gatun Overburden	Silty Sand; Residual material derived from Gatun Sandstone	SM	12	16	30	0	21	29	21	31	1.00E-07
	Disposal Fill	Clay with trace Sand and Silt; material derived from canal excavations	CL,SM,SC	10	15	24	0	18	24	0	27	1.00E-08

	Construction Material	Description	Material Source	Gradation Aggregate Size % Passing	Compacted Density γ_{moist} kN/m³	Compacted Density γ_{sat} kN/m³	Relative Density Dr	Shear Strength c kN/m²	Shear Strength ϕ Degrees	Wall Friction δ Degrees	Presumptive Bearing Capacity Mpa	Permeability k m/sec
Construction Materials	Structural Backfill; Well graded	Pervious, commericially manufactured aggregates obtained from local quarries. (1-3 cm diameter)	Basalt	3 cm 100% 1 cm 60% #100 <10%	19	20	0.8	0	35	24	0.28	1.00E-03
	Random Rockfill; Well graded	Semi-pervious, compacted random rockfill, obtained from rock excavation for 3rd Gatun Lock	Gatun Sandstone	Max Size 10 cm Min Size Varies	17	19	0.8	0	33	17	0.28	1.00E-06
	Random Backfill	Impervious, unclassified, compacted random soil fill obtained from overburden excavation for 3rd Gatun Lock	CL,SM,SC	100% passing # 40	13	16	n/a	0	30	14	0.01	1.00E-08

Notes:
1. Adopted parameters derived from limited ACP Lab Data, Geotechnical Reports, Emperical Data, Correlations, and Engineering Judgement.
2. Parameters derived from sources other than appropriate testing procedures are unreliable and should only be used for Concept Design purposes.

US Army Corps of Engineers
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Panama Canal Authority
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3rd Lane - Concept Design Study

Stability Analysis
Theoretical Depth of Tensile Crack

Computed By: D. Carlson Date Jul-02
Checked By: CES

Depth of Crack estimated from Rankine's Earth Pressure Theory for active earth pressure behind a horizontal ground surface.

For Total Stress Analysis:

Crack Depth $Z(t) = \frac{2\,Cm}{\gamma} \times \text{Tan}\left(45 + \frac{\phi}{2}\right)$

For Random Fill

$C = 24\ kN/m^2$

$\phi = 0$ Deg

$\gamma = 15\ kN/m^3$

Z (t) Random Fill = 2 (24) x tan (45 + 0/2)

Z (t) Random Fill = 3.2 m

For Atlantic Muck

$C = 19\ kN/m^2$

$\phi = 0$ Deg

$\gamma = 14\ kN/m^3$

Z (t) Random Fill = 2 (24) x tan (45 + 0/2)

Z (t) Atlantic Muck = 2.7 m

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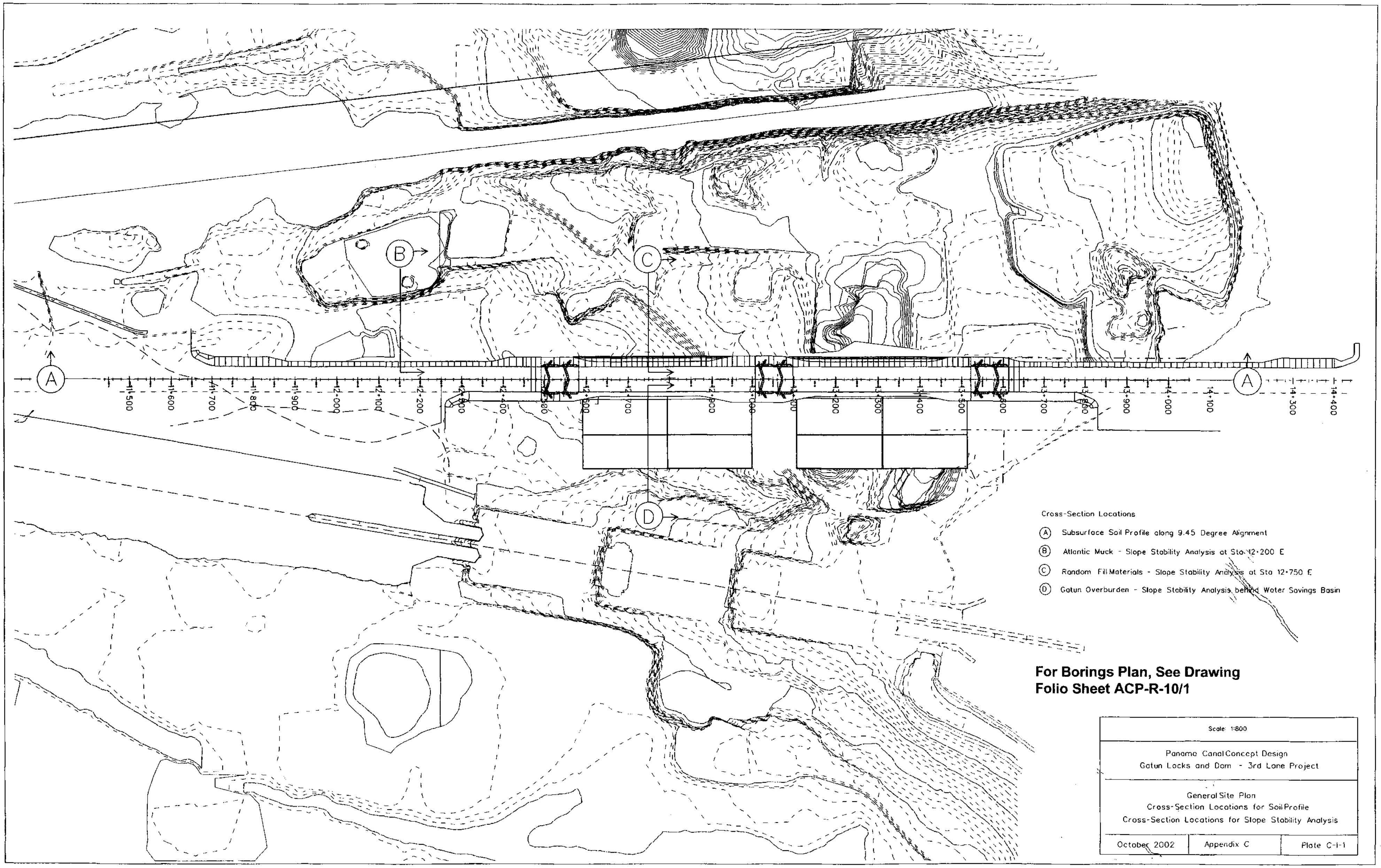
Appendix C – Geotechnical Investigations, Analyses and Designs

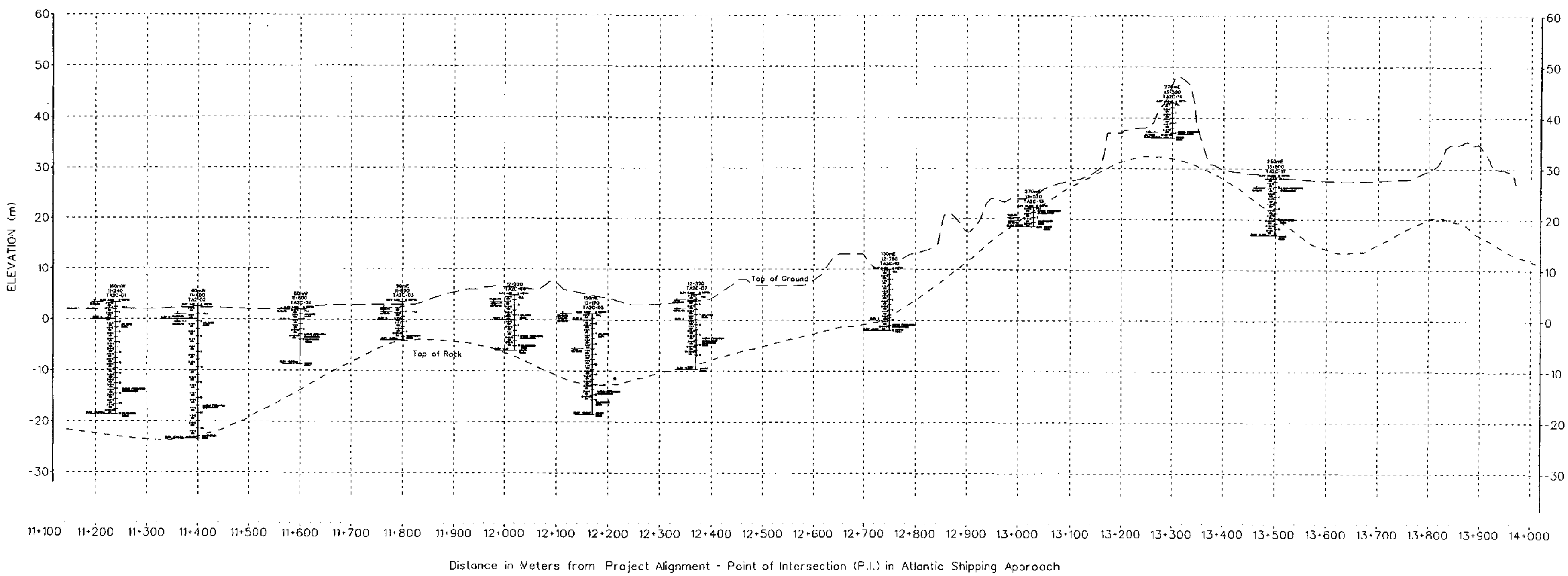
Part I – Soils

Plates

Plate C-I-1	General Site Plan
Plate C-I-2	Soil Profile – Station 11+100 to Station 14+000
Plate C-I-3	Soil Profile – Station 11+100 to Station 12+800
Plate C-I-4	Soil Profile – Station 12+000 to Station 13+600
Plate C-I-5	Cross-Section – Station 12+200
Plate C-I-6	Station 12+200 – Slope Stability Analysis Results
Plate C-I-7	Station 12+200 – Slope Stability Analysis Results
Plate C-I-8	Station 12+200 – Slope Stability Analysis Results
Plate C-I-9	Cross-Section – Station 12+750 East
Plate C-I-10	Station 12+750 East – Slope Stability Analysis Results
Plate C-I-11	Station 12+750 East – Slope Stability Analysis Results
Plate C-I-12	Cross-Section – Station 12+750 West
Plate C-I-13	Station 12+750 West – Slope Stability Analysis Results
Plate C-I-14	Station 12+750 West – Slope Stability Analysis Results
Plate C-I-15	Station 12+750 West – Slope Stability Analysis Results



A
B
C
D
A
11+500
11+600
11+700
11+800
11+900
12+000
12+100
12+200
12+400
12+700
12+900
13+200
13+300
13+400
13+500
13+700
13+900
14+000
14+100
14+300
14+400
Cross-Section Locations
(A) Subsurface Soil Profile along 9.45 Degree Alignment
(B) Atlantic Muck - Slope Stability Analysis at Sta 12+200 E
(C) Random Fill Materials - Slope Stability Analysis at Sta 12+750 E
(D) Gatun Overburden - Slope Stability Analysis behind Water Savings Basin
For Borings Plan, See Drawing
Folio Sheet ACP-R-10/1
Scale: 1:800
Panama Canal Concept Design
Gatun Locks and Dam - 3rd Lane Project
General Site Plan
Cross-Section Locations for Soil Profile
Cross-Section Locations for Slope Stability Analysis
October 2002
Appendix C
Plate C-I-1



Soil Profile Along Project Baseline
9.45 Degree Alignment
Project Station 11+100 to 14+000

For Enlarged View of Soils Data, See Figures 3 & 4

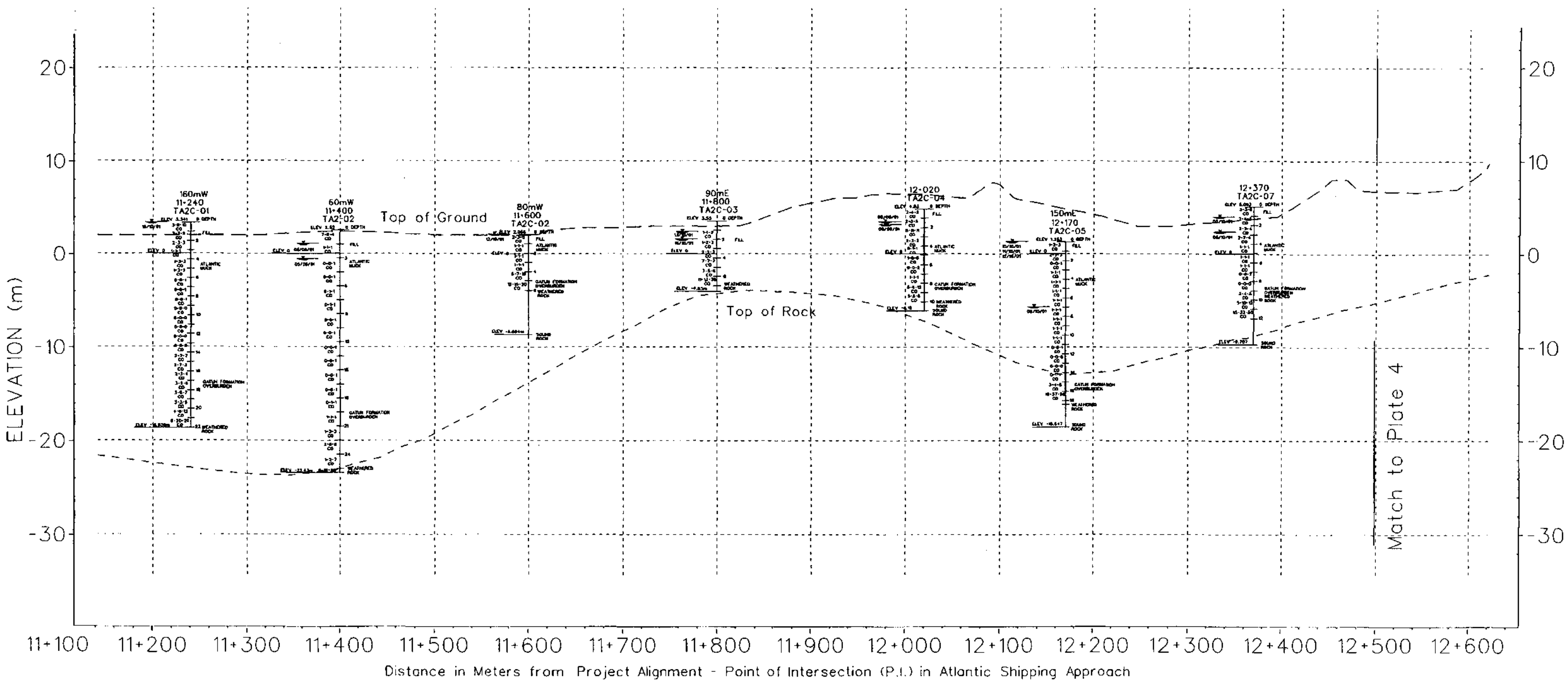
NOTE: TOP OF GROUND AND TOP OF ROCK ELEVATIONS ALONG PROJECT CENTERLINE INTERPOLATED AND EXTRAPOLATED BY DIGITAL TERRAIN MODELING SOFTWARE. BORING DATA WAS SUPERIMPOSED ON DTM SURFACES AT ACTUAL ELEVATIONS.

-SCALE-
Vertical: 1:800 Horizontal: 1:8000

Panama Canal Concept Design
Gatun Locks and Dam - 3rd Lane Project

Soil Profile Along Project Baseline
9.45 Degree Alignment
Project Station 11+100 to 14+000

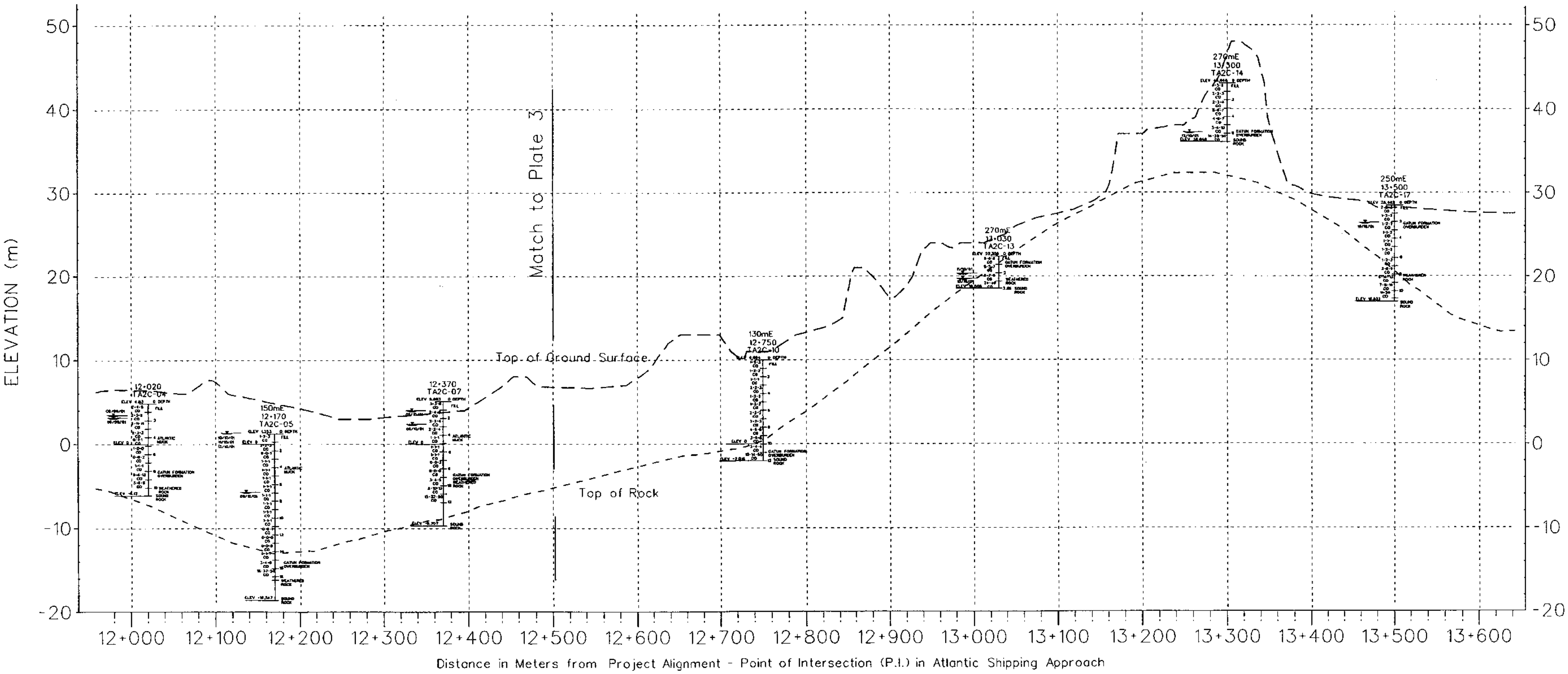
October 2002	Appendix C	Plate C-I-2



Soil Profile Along Project Centerline
9.45 Degree Alignment
Project Station 11+100 to 12+800

NOTE: TOP OF GROUND AND TOP OF ROCK ELEVATIONS ALONG PROJECT CENTERLINE INTERPOLATED AND EXTRAPOLATED BY DIGITAL TERRAIN MODELING SOFTWARE. BORING DATA WAS SUPERIMPOSED ON DTM SURFACES AT ACTUAL ELEVATIONS.

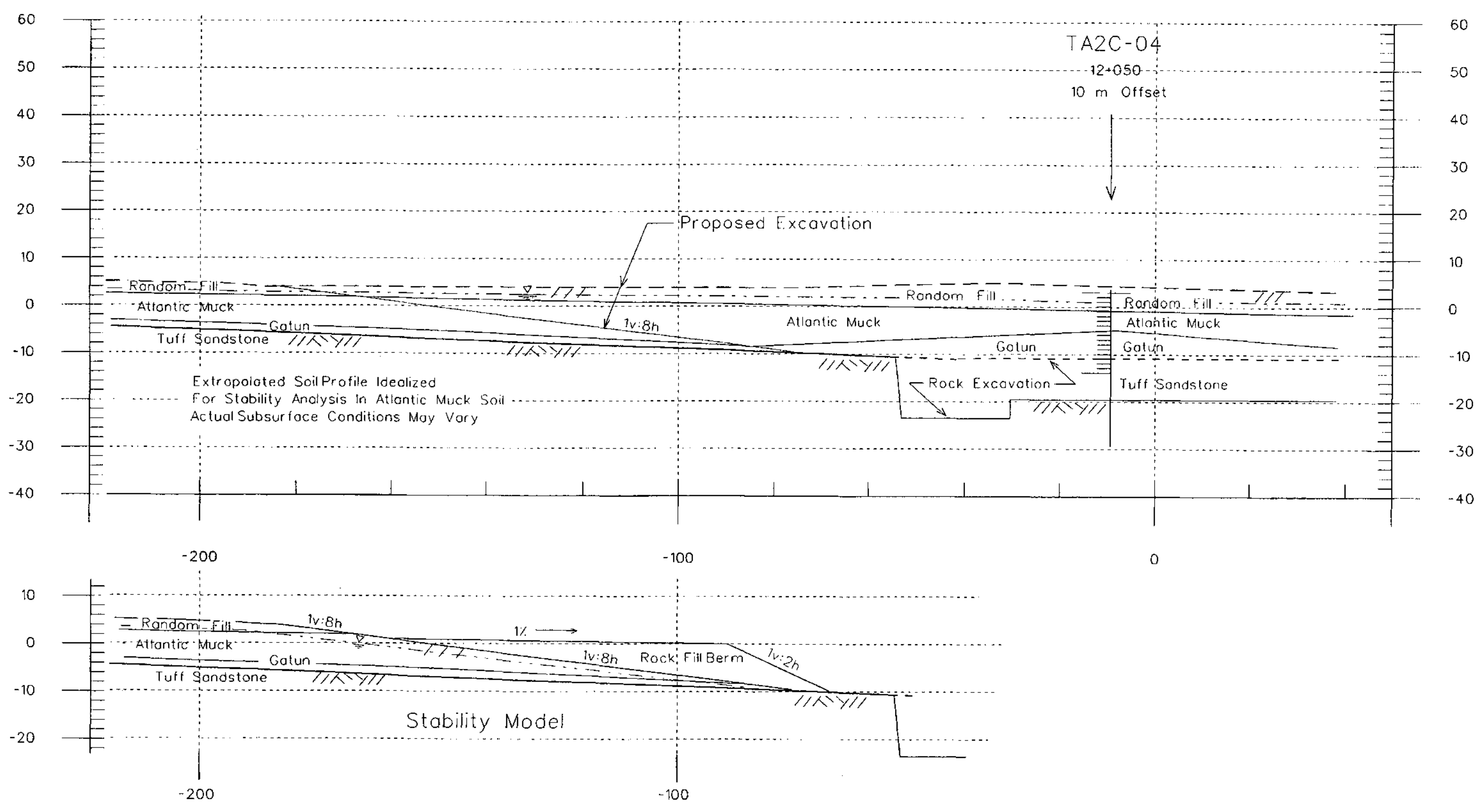
-SCALE-		
Vertical: 1:500		Horizontal: 1:5000
Panama Canal Concept Design Gatun Locks and Dam - 3rd Lane Project		
Soil Profile Along Project Centerline 9.45 Degree Alignment Project Station 11+100 to 12+800		
October 2002	Appendix C	Plate C-I-3



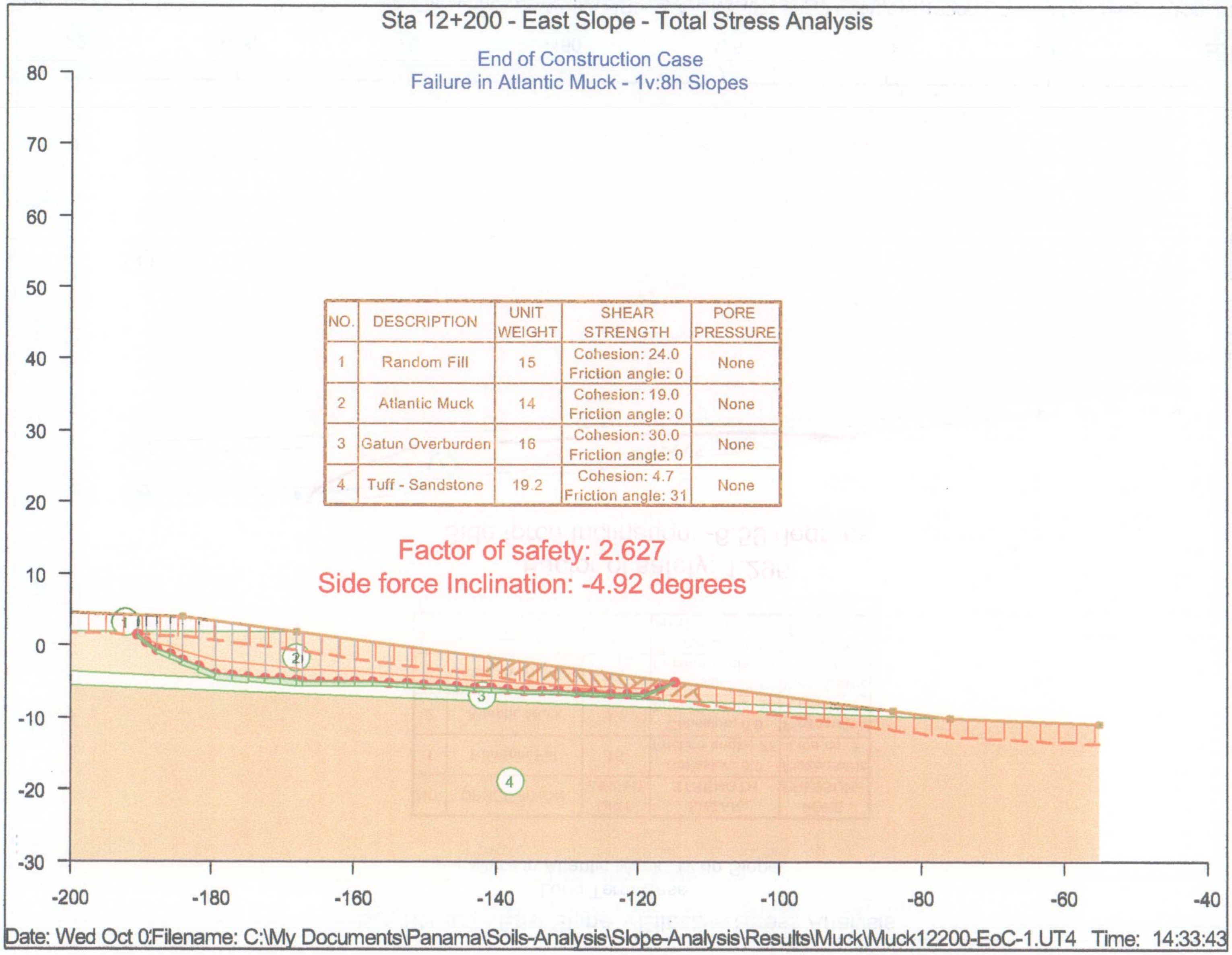
Soil Profile Along Project Baseline
9.45 Degree Alignment
Project Station 12+000 to 13+600

NOTE: TOP OF GROUND AND TOP OF ROCK ELEVATIONS ALONG PROJECT CENTERLINE INTERPOLATED AND EXTRAPOLATED BY DIGITAL TERRAIN MODELING SOFTWARE. BORING DATA WAS SUPERIMPOSED ON DTM SURFACES AT ACTUAL ELEVATIONS.

-SCALE-		
Vertical: 1:500		Horizontal: 1:5000
Panama Canal Concept Design Gatun Locks and Dam - 3rd Lane Project		
Soil Profile Along Project Baseline 9.45 Degree Alignment Project Station 12+000 to 13+600		
October 2002	Appendix C	Plate C-I-4

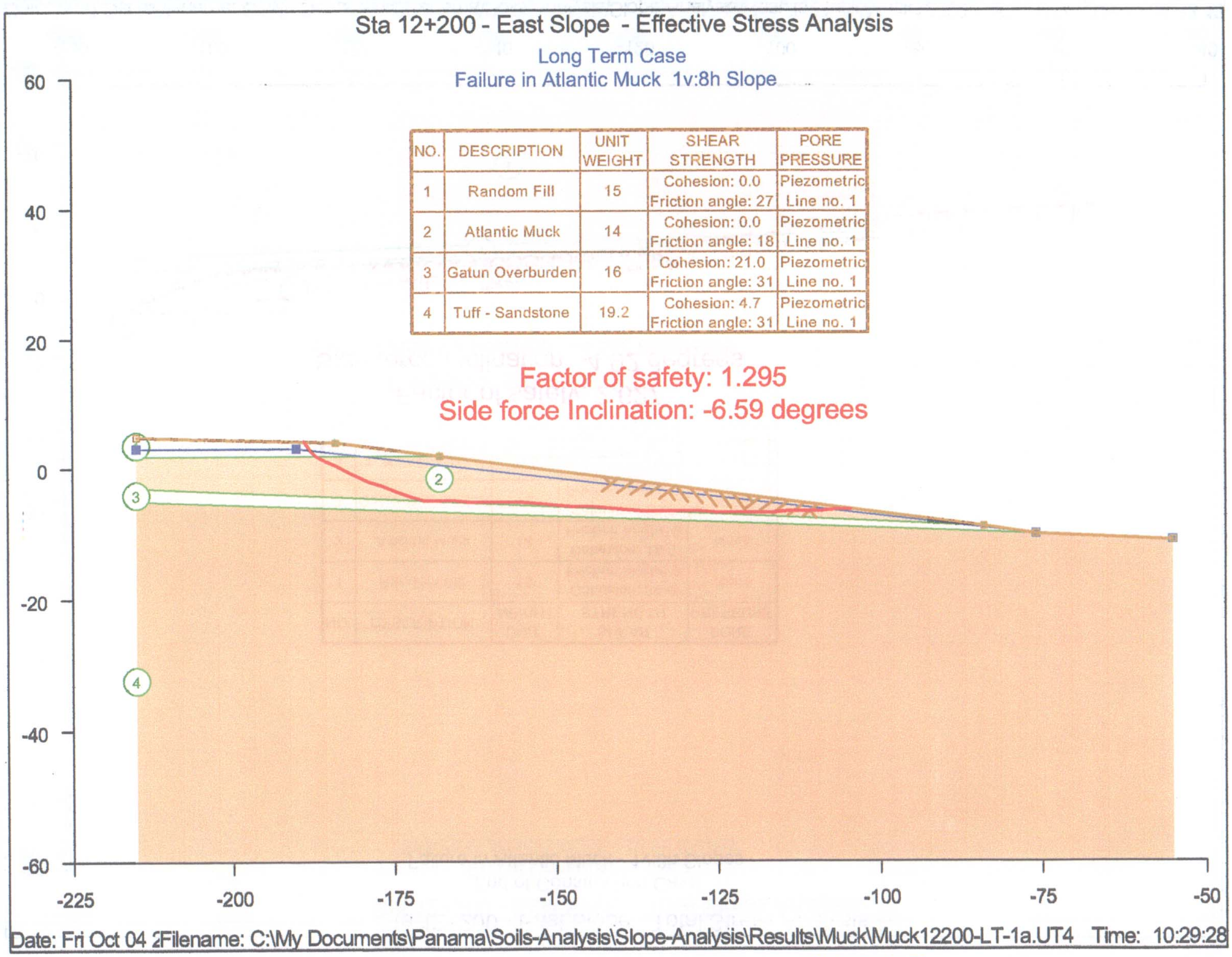


Scale: 1:100
Panama Canal Concept Design Gatun Locks and Dam - 3rd Lane Project
Slope Stability Analysis Model Project Station 12+200 East
October 2002 · Appendix C · Plate C-I-5



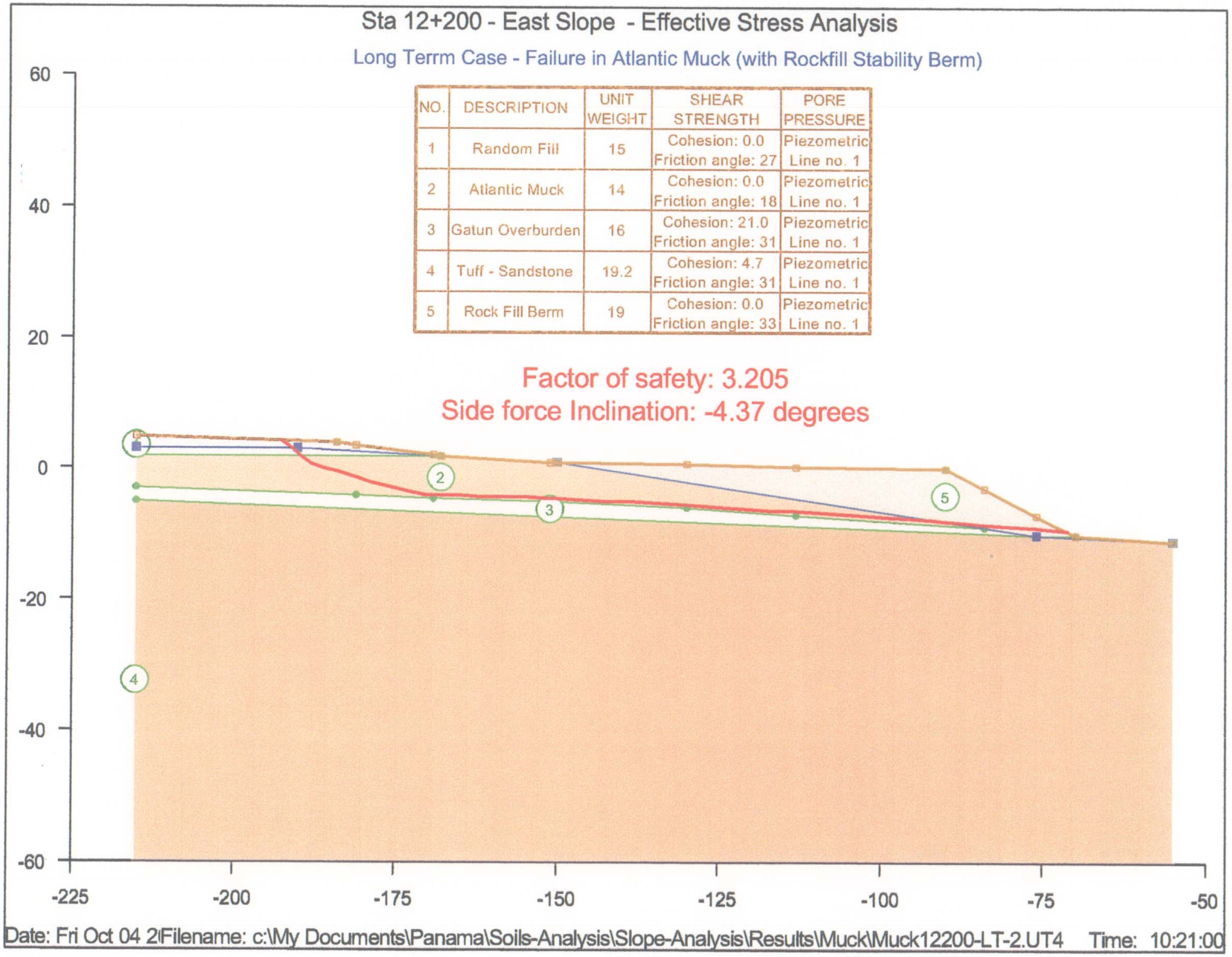
NO.	DESCRIPTION	UNIT WEIGHT	SHEAR STRENGTH	PORE PRESSURE
1	Random Fill	15	Cohesion: 24.0 Friction angle: 0	None
2	Atlantic Muck	14	Cohesion: 19.0 Friction angle: 0	None
3	Gatun Overburden	16	Cohesion: 30.0 Friction angle: 0	None
4	Tuff - Sandstone	19.2	Cohesion: 4.7 Friction angle: 31	None

Plate C - I - 6

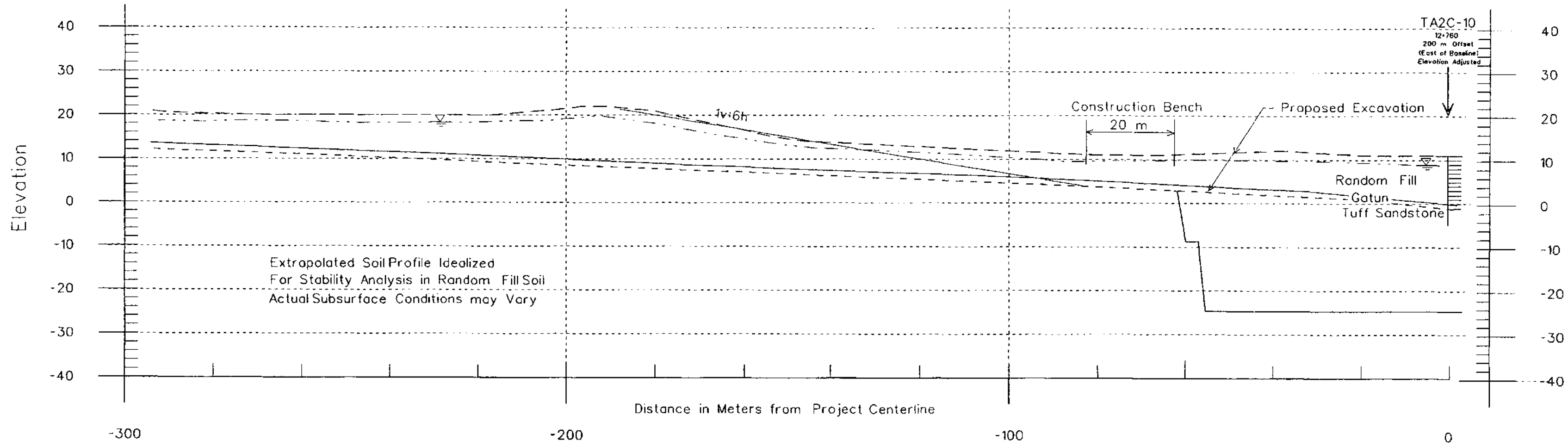


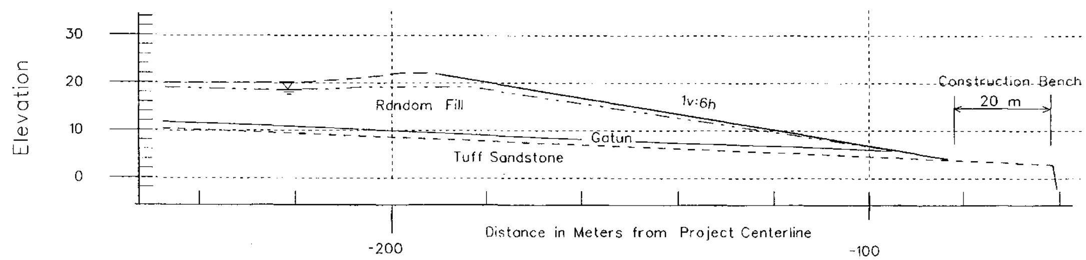
Sta 12+200 - East Slope - Effective Stress Analysis
Long Term Case
Failure in Atlantic Muck 1v:8h Slope
NO. | DESCRIPTION | UNIT WEIGHT | SHEAR STRENGTH | PORE PRESSURE
1 | Random Fill | 15 | Cohesion: 0.0 Friction angle: 27 | Piezometric Line no. 1
2 | Atlantic Muck | 14 | Cohesion: 0.0 Friction angle: 18 | Piezometric Line no. 1
3 | Gatun Overburden | 16 | Cohesion: 21.0 Friction angle: 31 | Piezometric Line no. 1
4 | Tuff - Sandstone | 19.2 | Cohesion: 4.7 Friction angle: 31 | Piezometric Line no. 1
Factor of safety: 1.295
Side force Inclination: -6.59 degrees
60
40
20
0
-20
-40
-60
-225
-200
-175
-150
-125
-100
-75
-50
2
3
4
Date: Fri Oct 04 2Filename: C:\My Documents\Panama\Soils-Analysis\Slope-Analysis\Results\Muck\Muck12200-LT-1a.UT4 Time: 10:29:28

Plate C - I - 7



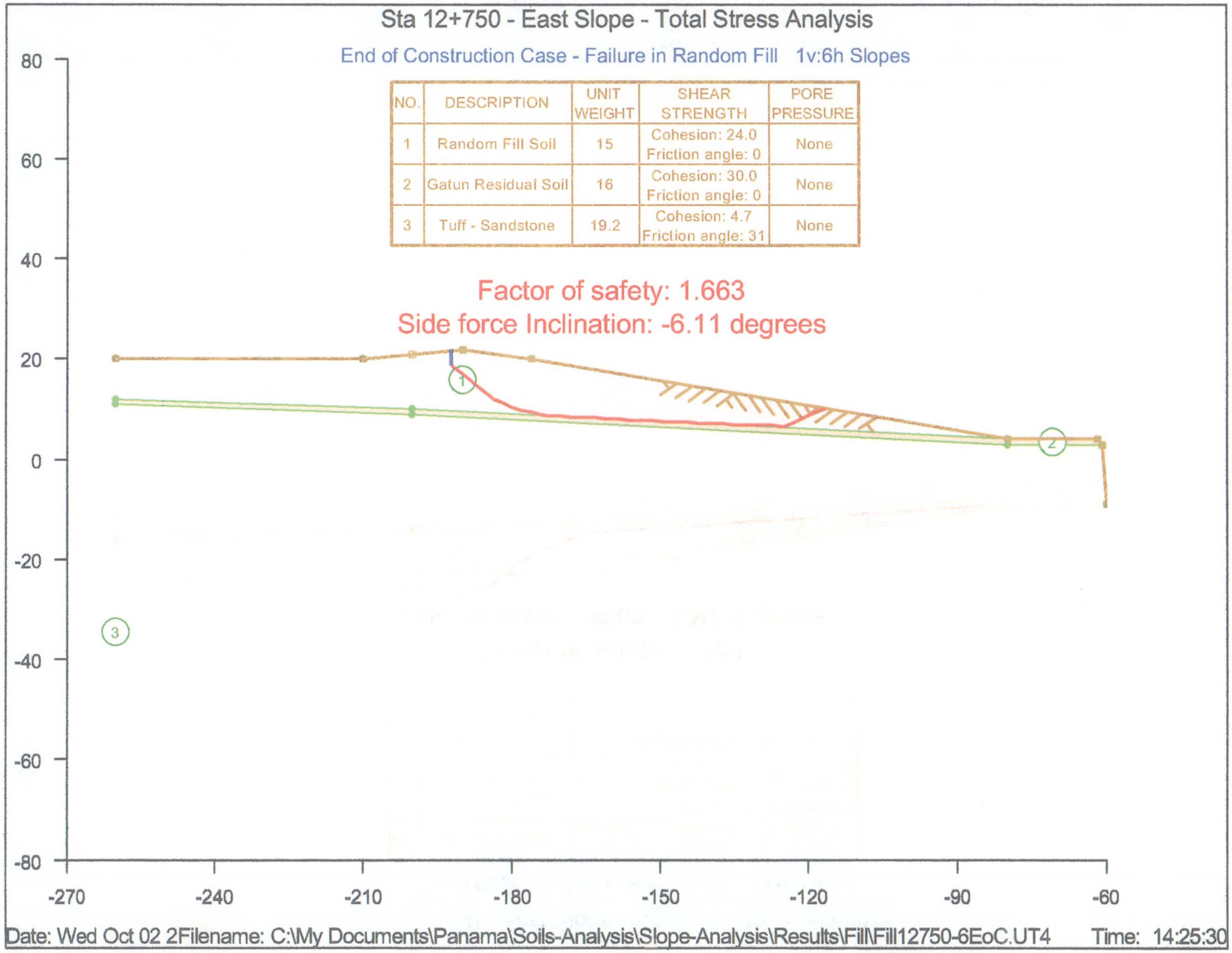
NO.	DESCRIPTION	UNIT WEIGHT	SHEAR STRENGTH	PORE PRESSURE
1	Random Fill	15	Cohesion: 0.0 Friction angle: 27	Piezometric Line no. 1
2	Atlantic Muck	14	Cohesion: 0.0 Friction angle: 18	Piezometric Line no. 1
3	Gatun Overburden	16	Cohesion: 21.0 Friction angle: 31	Piezometric Line no. 1
4	Tuff - Sandstone	19.2	Cohesion: 4.7 Friction angle: 31	Piezometric Line no. 1
5	Rock Fill Berm	19	Cohesion: 0.0 Friction angle: 33	Piezometric Line no. 1





Stability Model

Scale: 1:1000		
Panama Canal Concept Design Gatun Locks and Dam - 3rd Lane Project		
Slope Stability Analysis Model Project Station 12+750 East		
October 2002	Appendix C	Plate C-1-9



NO.	DESCRIPTION	UNIT WEIGHT	SHEAR STRENGTH	PORE PRESSURE
1	Random Fill Soil	15	Cohesion: 24.0 Friction angle: 0	None
2	Gatun Residual Soil	16	Cohesion: 30.0 Friction angle: 0	None
3	Tuff - Sandstone	19.2	Cohesion: 4.7 Friction angle: 31	None

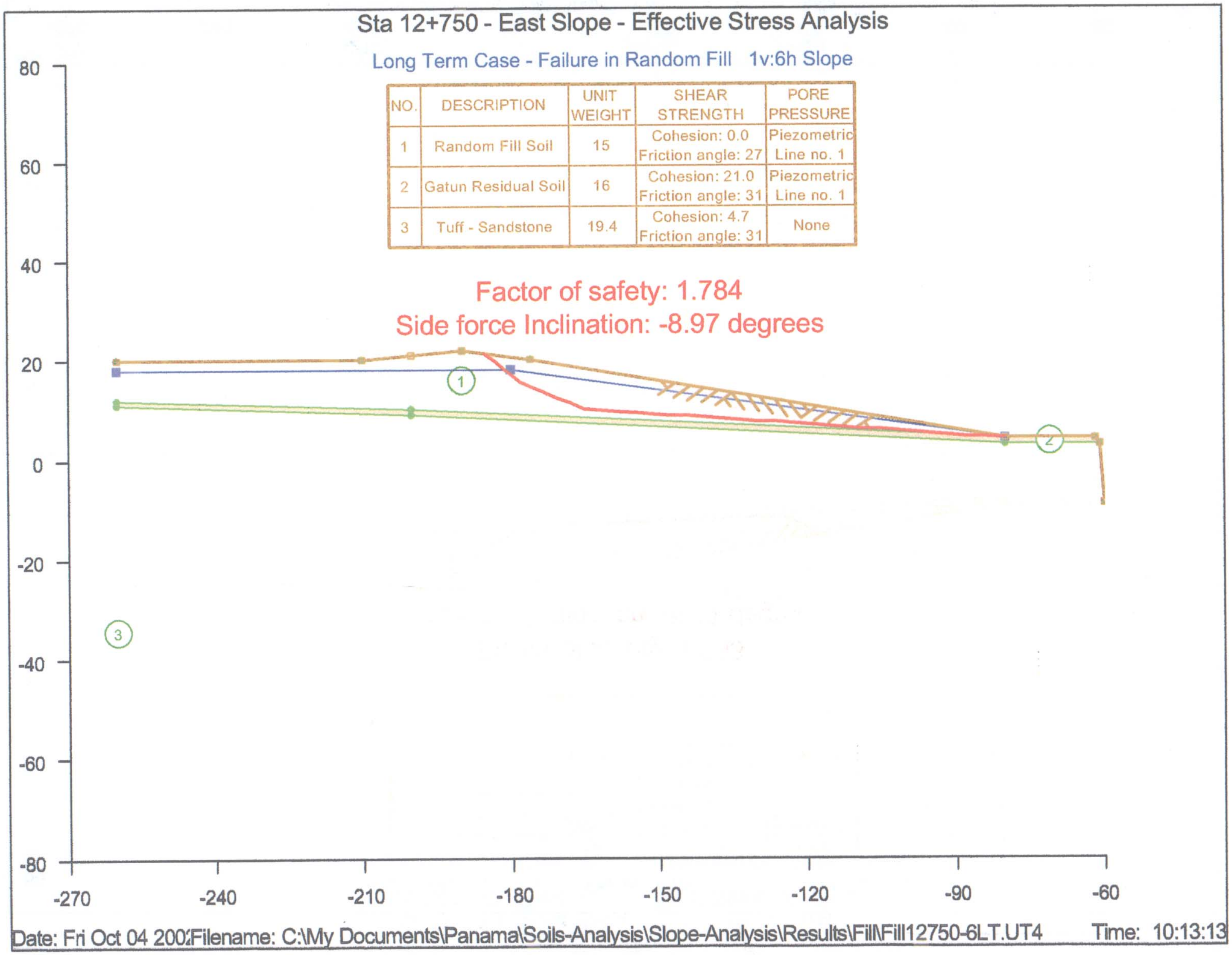
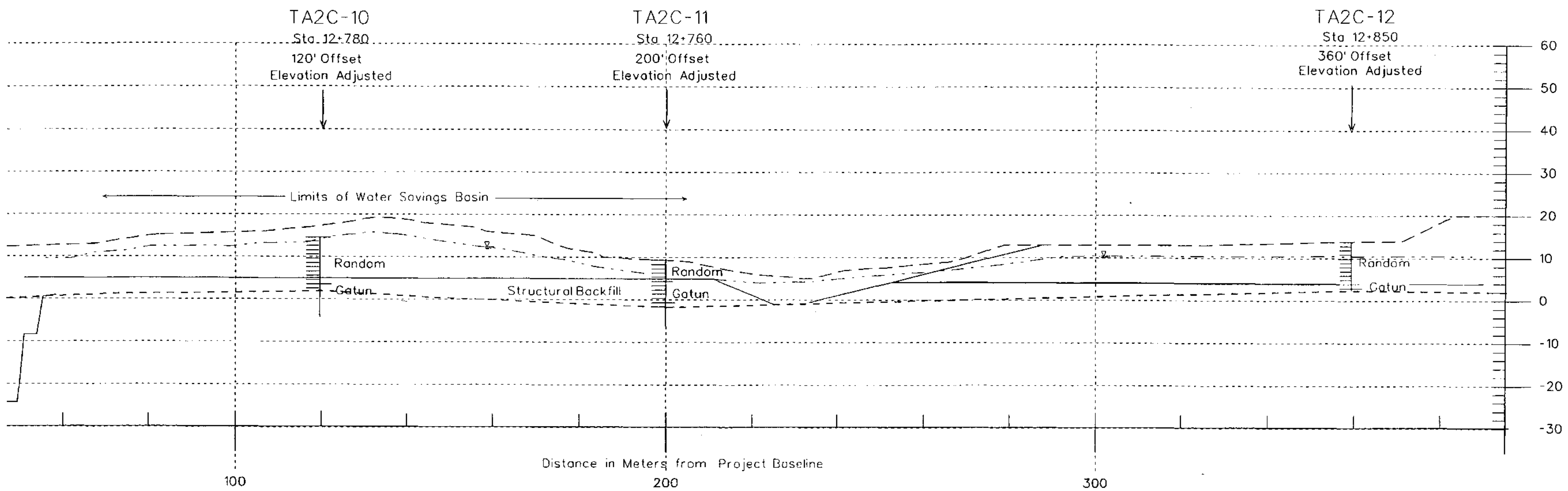
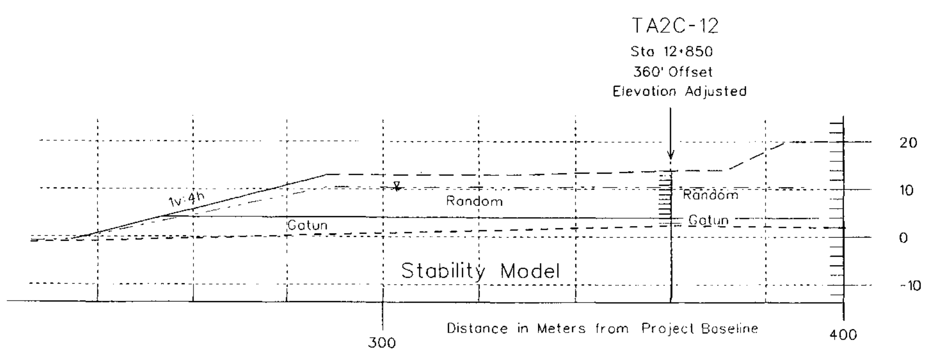

Sta 12+750 - East Slope - Effective Stress Analysis
Long Term Case - Failure in Random Fill 1v:6h Slope
NO. | DESCRIPTION | UNIT WEIGHT | SHEAR STRENGTH | PORE PRESSURE
1 | Random Fill Soil | 15 | Cohesion: 0.0 Friction angle: 27 | Piezometric Line no. 1
2 | Gatun Residual Soil | 16 | Cohesion: 21.0 Friction angle: 31 | Piezometric Line no. 1
3 | Tuff - Sandstone | 19.4 | Cohesion: 4.7 Friction angle: 31 | None
Factor of safety: 1.784
Side force Inclination: -8.97 degrees
80
60
40
20
0
-20
-40
-60
-80
1
2
3
-270
-240
-210
-180
-150
-120
-90
-60
Date: Fri Oct 04 200:Filename: C:\My Documents\Panama\Soils-Analysis\Slope-Analysis\Results\Fill\Fill12750-6LT.UT4
Time: 10:13:13

Plate C - I - 11





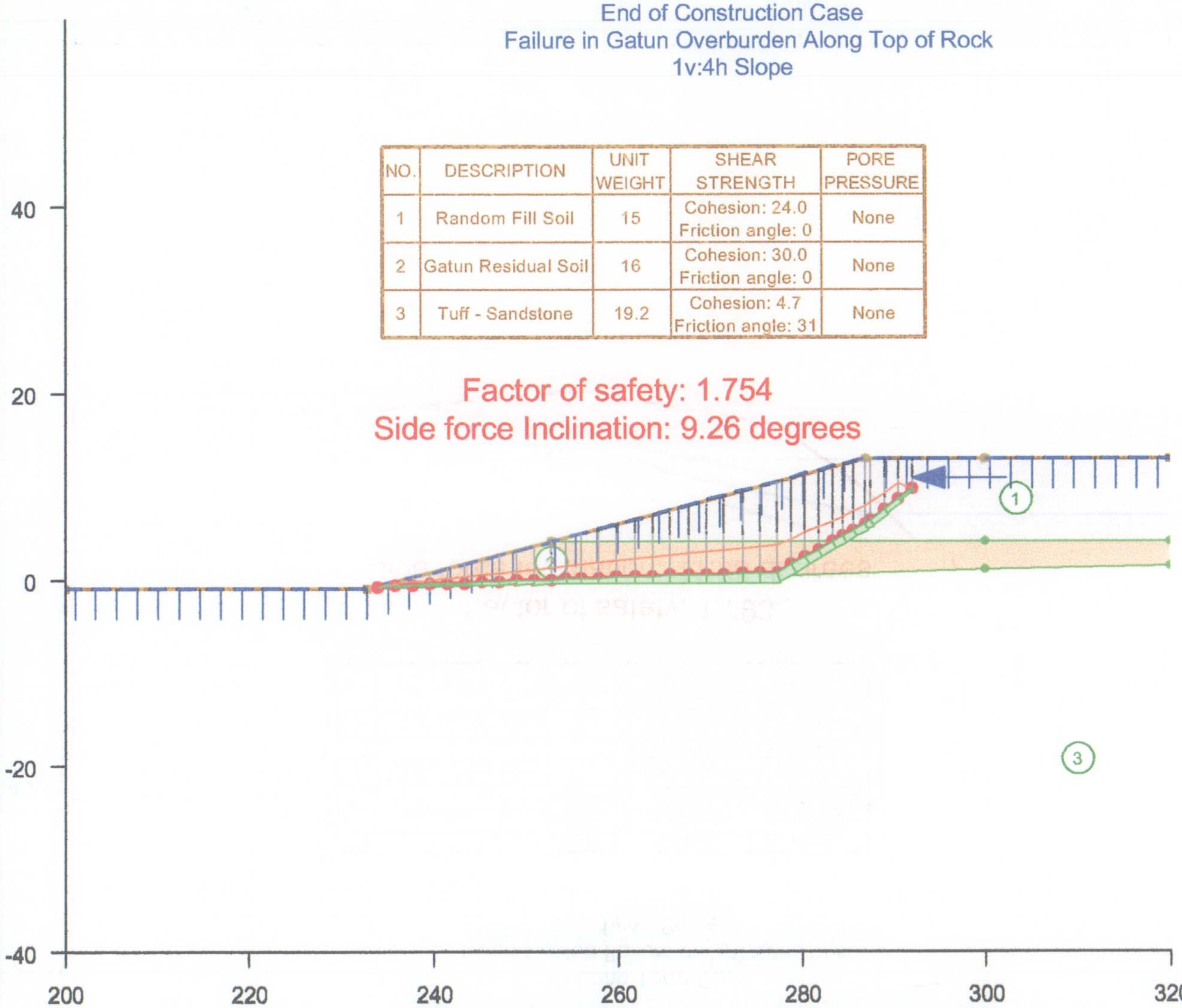
Scale: 1:1000		
Panama Canal Concept Design Gatun Locks and Dam - 3rd Lane Project		
Slope Stability Analysis Model Project Station 12+750 West		
October 2002	Appendix C	Plate C-I-12

Sta 12+750 - West Slope

End of Construction Case
Failure in Gatun Overburden Along Top of Rock
1v:4h Slope

NO.	DESCRIPTION	UNIT WEIGHT	SHEAR STRENGTH	PORE PRESSURE
1	Random Fill Soil	15	Cohesion: 24.0 Friction angle: 0	None
2	Gatun Residual Soil	16	Cohesion: 30.0 Friction angle: 0	None
3	Tuff - Sandstone	19.2	Cohesion: 4.7 Friction angle: 31	None

Factor of safety: 1.754
Side force Inclination: 9.26 degrees



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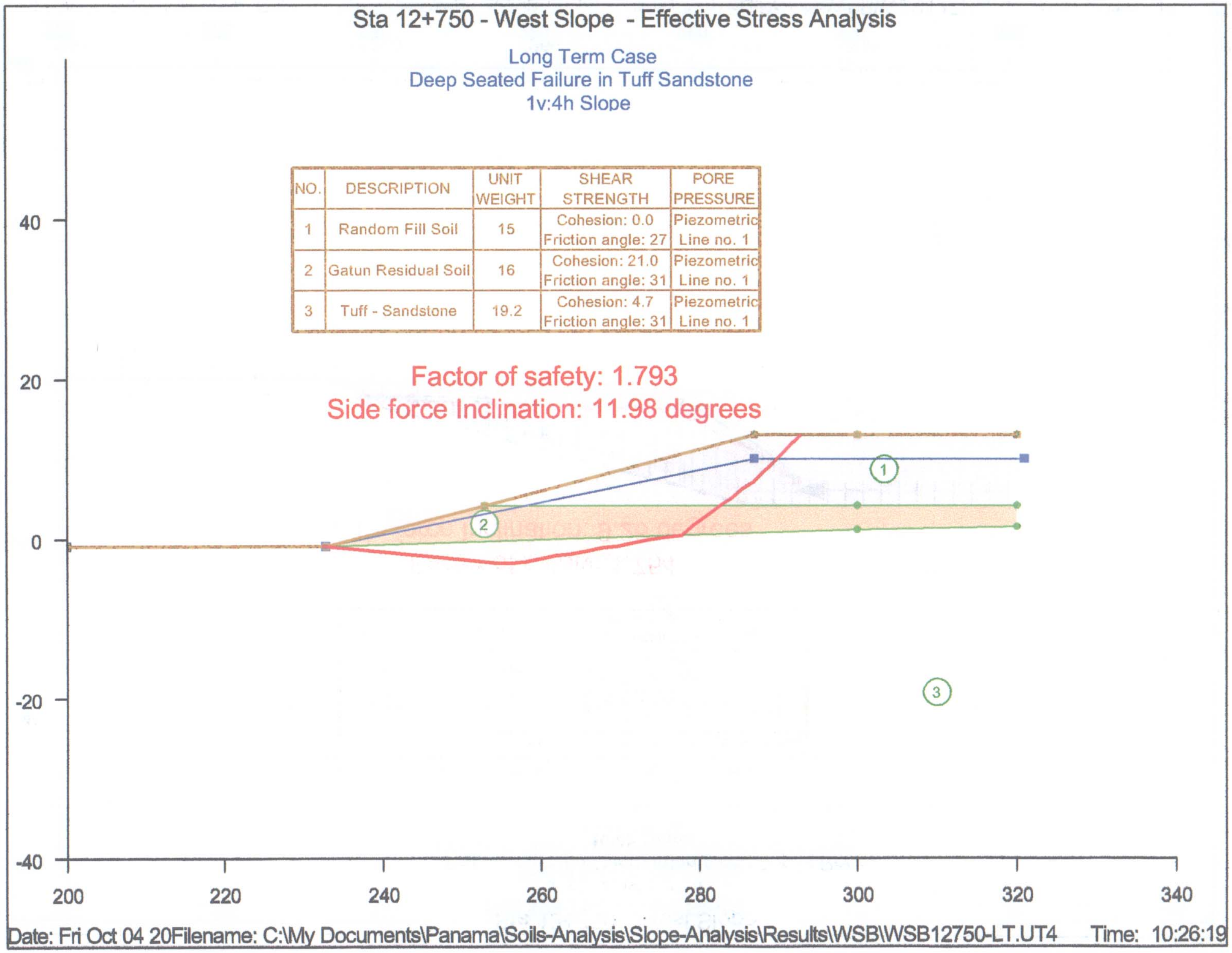
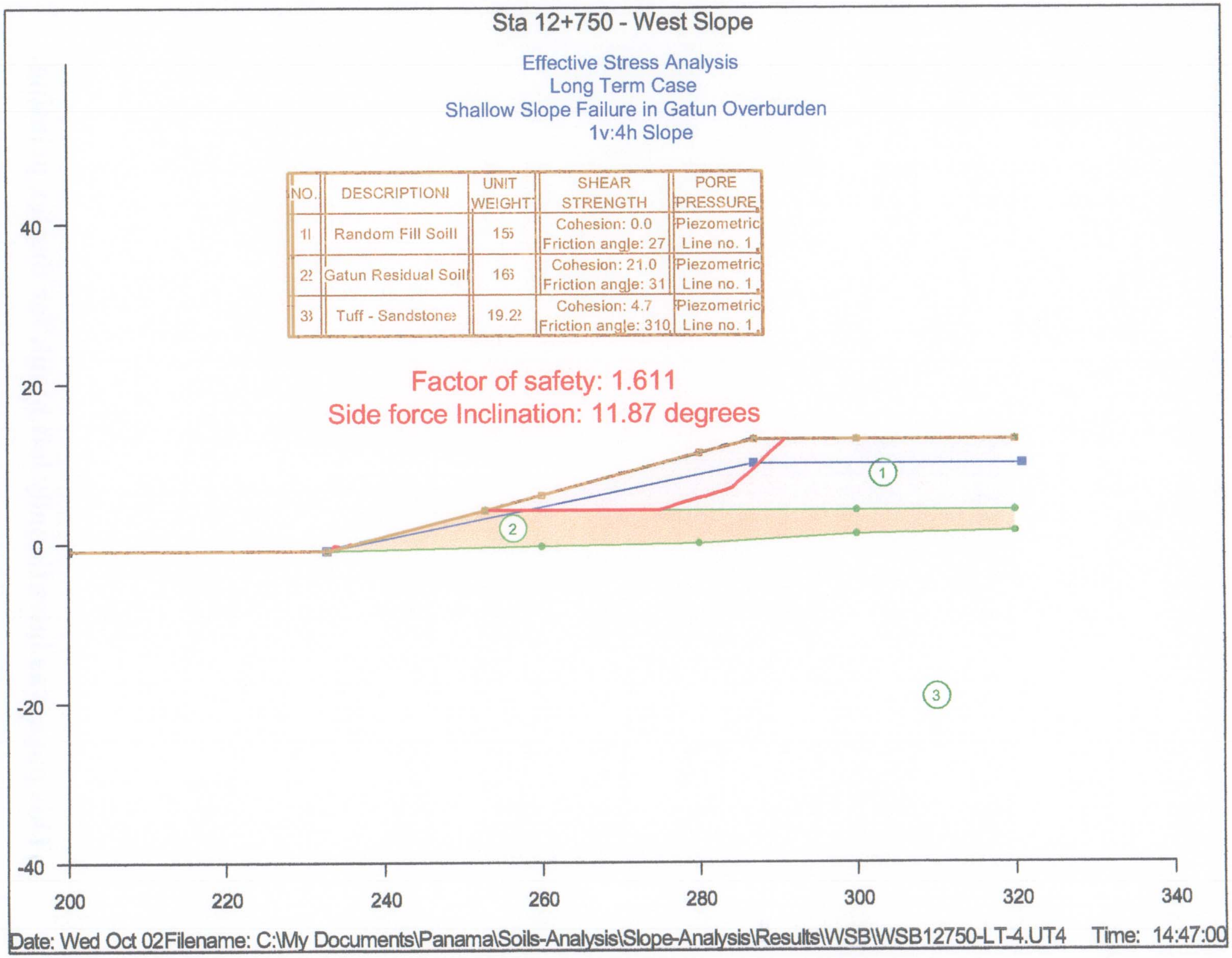


NO.	DESCRIPTION	UNIT WEIGHT	SHEAR STRENGTH	PORE PRESSURE
1	Random Fill Soil	15	Cohesion: 0.0 Friction angle: 27	Piezometric Line no. 1
2	Gatun Residual Soil	16	Cohesion: 21.0 Friction angle: 31	Piezometric Line no. 1
3	Tuff - Sandstone	19.2	Cohesion: 4.7 Friction angle: 31	Piezometric Line no. 1

Plate C - I - 14



Sta 12+750 - West Slope
Effective Stress Analysis
Long Term Case
Shallow Slope Failure in Gatun Overburden
1v:4h Slope
NO.
DESCRIPTION
UNIT WEIGHT
SHEAR STRENGTH
PORE PRESSURE
1
Random Fill Soil
15
Cohesion: 0.0
Friction angle: 27
Piezometric Line no. 1
2
Gatun Residual Soil
16
Cohesion: 21.0
Friction angle: 31
Piezometric Line no. 1
3
Tuff - Sandstone
19.2
Cohesion: 4.7
Friction angle: 31
Piezometric Line no. 1
Factor of safety: 1.611
Side force Inclination: 11.87 degrees
40
20
0
-20
-40
200
220
240
260
280
300
320
340
1
2
3
Date: Wed Oct 02Filename: C:\My Documents\Panama\Soils-Analysis\Slope-Analysis\Results\WSB\WSB12750-LT-4.UT4
Time: 14:47:00

Plate C - I - 15

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Appendix C – Geotechnical Investigations, Analyses and Designs

Part I – Soils

Exhibits

Exhibit C-I-1	Detailed Listing of Technical References
Exhibit C-I-2	ACP Bromhead Ring Shear Report
Exhibit C-I-3	ACP Direct Shear Test Report
Exhibit C-I-4	Excerpts from 1963 Shannon and Wilson Report on Foundation Investigations at Trinidad Arm of Lake Gatun
Exhibit C-I-5	Table 2.1 from Seepage, Drainage, and Flownets by H. Cedegren
Exhibit C-I-6	Excerpts from 1943 PCC Report, Part II, Chapter 5
Exhibit C-I-7	Excerpts from 1941 Outline Report on Canal Zone Geology in Relation to the Third Locks Project by D. MacDonald
Exhibit C-I-8	Table 4-8 from Foundation Analysis and Design by J. Bowles
Exhibit C-I-9	Table 4.1 from Correlations of Soil Properties by Carter & Bentley

EXHIBIT C-I-1 LISTING OF TECHNICAL REFERENCES

1) Final Report on Modified Third Locks Project, Part II – Design, Chapter 3 – Geology; Department of Operation and Maintenance, Special Engineering Division, Balboa Heights, Canal Zone. December 1943

2) Final Report on Modified Third Locks Project, Part II – Design, Chapter 5 – Foundations and Slopes; Department of Operation and Maintenance, Special Engineering Division, Balboa Heights, Canal Zone. December 1943

3) National Highway Institute, Publication No. FHWA HI 97-013; November 1998

4) Ismithian Canal Studies; Third Locks Project, Appendix 3, Report of the Governor of the Panama Canal; 1947

5) Geotechnical Advisory Board, Meeting No. 9 – Gatun Dam Seismic Studies, Letter Report; Panama Canal Commission; September 1999

6) Design Earthquakes for the Evaluation of Seismic Hazard at the Gatun Dam and Vicinity – Final Report; U.S. Geological Survey; August 1999

7) Geotechnical Advisory Board, Meeting No. 8 – Gatun Dam Seismic Studies, Letter Report; Panama Canal Commission, July 1998

8) Soil Liquifaction Website, Department of Civil Engineering, University of Washington, U.S.A.

9) Correlations of Soil Properties; Carter and Bentley, Pentech Press, 1991

10) Report on Foundation Investigations – Proposed Rock and Earth Fill Embankment Across Trinidad Arm of Gatun Lake, Panama Canal Zone, for Tudor Engineering Co; Shannon and Wilson, Inc, Seattle Washington, March 1963

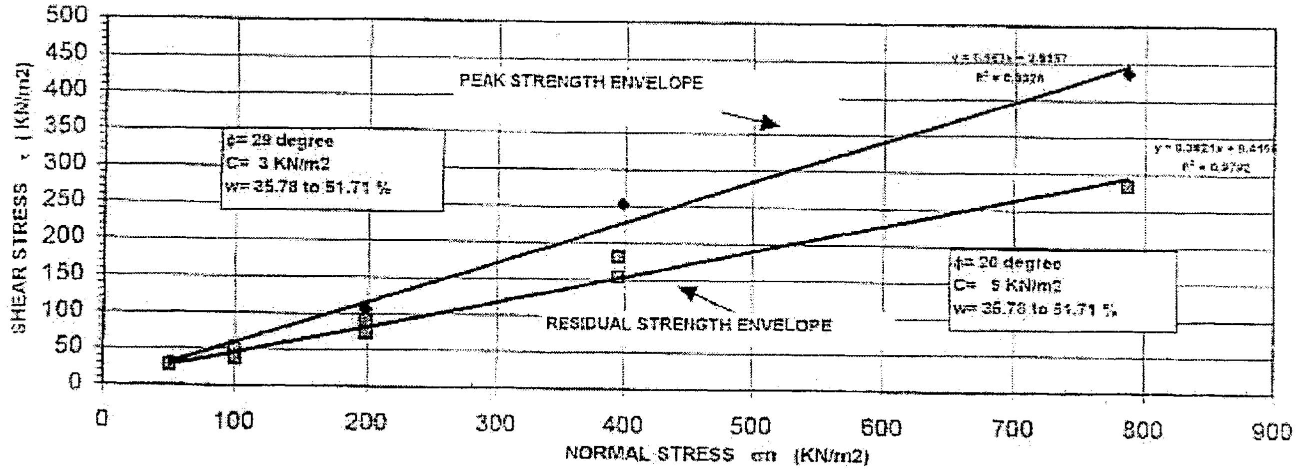
11) Memorandum C 42, An Outline Report on Canal Zone Geology in Relation the Third Lock Project; The Panama Canal Special Engineering Division Third Locks Project; Donald F. MacDonald, March 1941

12) Presumptive Bearing Capacities from Indicated Building Codes, Page 230, Table 4-8, Foundation Analysis and Design, 4th Edition, by J.E. Bowles; McGraw-Hill Publisher,1988.

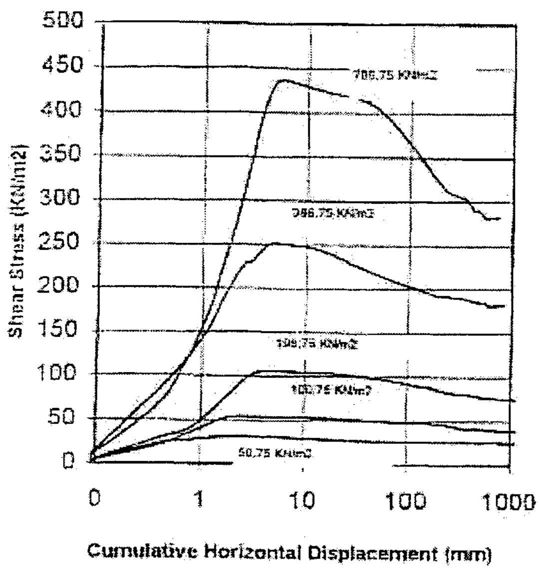
13) Typical Permeability Values for Soil, Page52, Table 4.1, Correlations of Soil Properties, by M.Carter and S. Bently; Pentech Press Publisher, 1991

14) Comparison of Flow and Seepage Through Soils and Aggregates, Page 23, Table 2.1, Seepage, Drainage, and Flownets, 3rd Edition, by H. Cedergen, John Wiley & Sons Publisher, 1989.

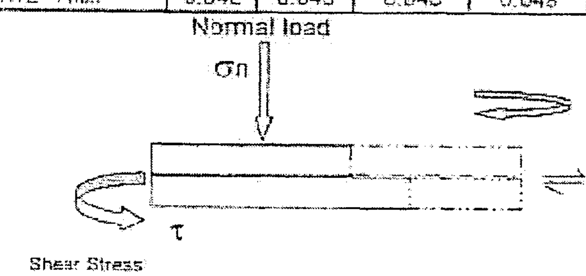
ACP-GEOTECHNICAL LABORATORY
FAILURE ENVELOPE
DRAINED BROMHEAD RING SHEAR TEST



Residual Shear Stress



TEST No	1	2	3	4	5
NORMAL STRESS (kPa)	51.75	100.75	198.75	394.75	786.75
SHEAR STRESS (kPa)	31.2	53.6	105.9	252.6	436.3
STRAIN (mm)	3.3	4.1	4.8	5.2	5.9
RESIDUAL STRESS (kPa)	25.5	47.93	89.62	153.4	282.4
	28.1	39.16	71.1	181.4	
		52	74.9		
		36.4			
SPEED RATE °/ min	0.048	0.048	0.048	0.048	0.048



Annular plane of relative rotary motion

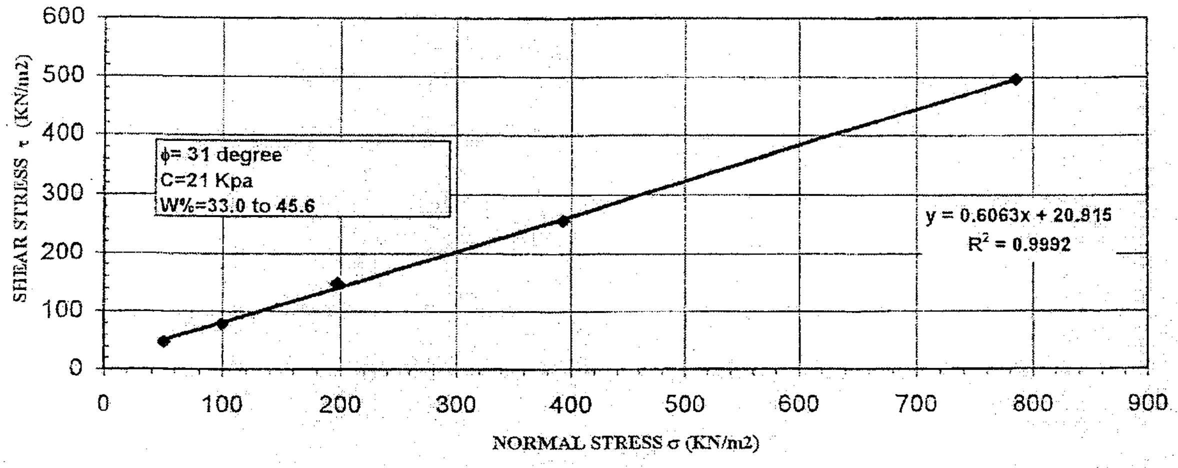
OJECT: Third Set of Locks
CATION: Atlantic
RING: TA2-4
PTH: 28.60-30.00 m
RMATION: Tuffaceous sandstone , Gatun Fom.
SCRIPTION (UCS): SM silty sand
LOR: grayish brown
MPLE TYPE: Remolded
STED BY: G. GUERRA 10/19/01

INDEX PROPERTIES					
LL=	48	LP=	34	IP=	14
Gs=	2.3	sieve No. 200=	34%		
Clay Fraction					
0.005mm=	14%			0.002mm=	10%
Water content=	35.78 to 51.71%				
Shear Strength Parameters					
C peak=	3	Kpa	C residual=	9	Kpa
ϕ peak=	29	degree	ϕ residual=	20	degree

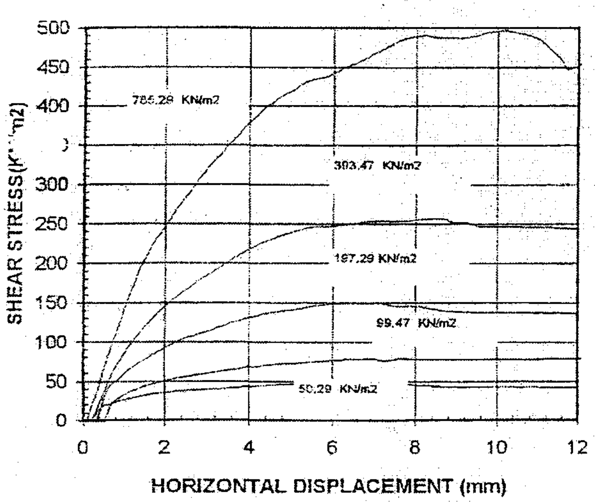
BROMHEAD RING SHEAR REPORT

EXHIBIT C-I-2

ACP- GEOTECHNICAL BRANCH
FAILURE ENVELOPE
DIRECT SHEAR TEST



DIRECT SHEAR TEST



TEST No	1	2	3	4	5
INITIAL TEST					
Water Content, %	45.6	45.5	44.5	44.3	41
Dry Density, pcf	73.3	77	76.7	78	77
Saturation, %	107	117	112	115	106
Void Ratio	1.1	0.93	0.95	0.9	0.92
AT TEST					
Water Content, %	41.7	40.6	41.7	35.2	33
Dry Density, pcf	81.2	87	81	87	88.8
Saturation, %	121	136	121	117	115
Void Ratio	0.81	0.72	0.81	0.72	0.68
NORMAL STRESS (kPa)	50.29	99.47	197.29	393.47	785.3
SHEAR STRESS (kPa)	47.8	78.9	149.4	256.5	496
STRAIN (mm)	6.5	7.8	6.7	8.6	10.2
SPEED RATE mm/ min	0.003	0.003	0.003	0.003	0.003
DIAMETER (mm)	63.3	63.3	63.3	63.3	63.3

PROJECT: Third Locks
LOCATION: Gatun
BORING: TA2-4
DEPTH: 28.60m @ 30.0m
FORMATION: Gatun (tuffaceous sandstone)
DESCRIPTION (UCS): SM silty sand
COLOR: Gray
SAMPLE TYPE: Remolded
TESTED BY: G. GUERRA 6/7/01

INDEX PROPERTIES

LL= 48 LP= 34 IP= 14

Gs= 2.3 sieve No. 200= 34%

Clay Fraction (passing sieve No. 10)

0.005mm= 14% 0.002mm= 10%

Water content= 33.0 to 45.6%

Shear Strength Parameters

C peak= 21 Kpa

ϕ peak= 31 degree

DIRECT SHEAR TEST REPORT

AUTORIDAD DEL CANAL DE PANAMA

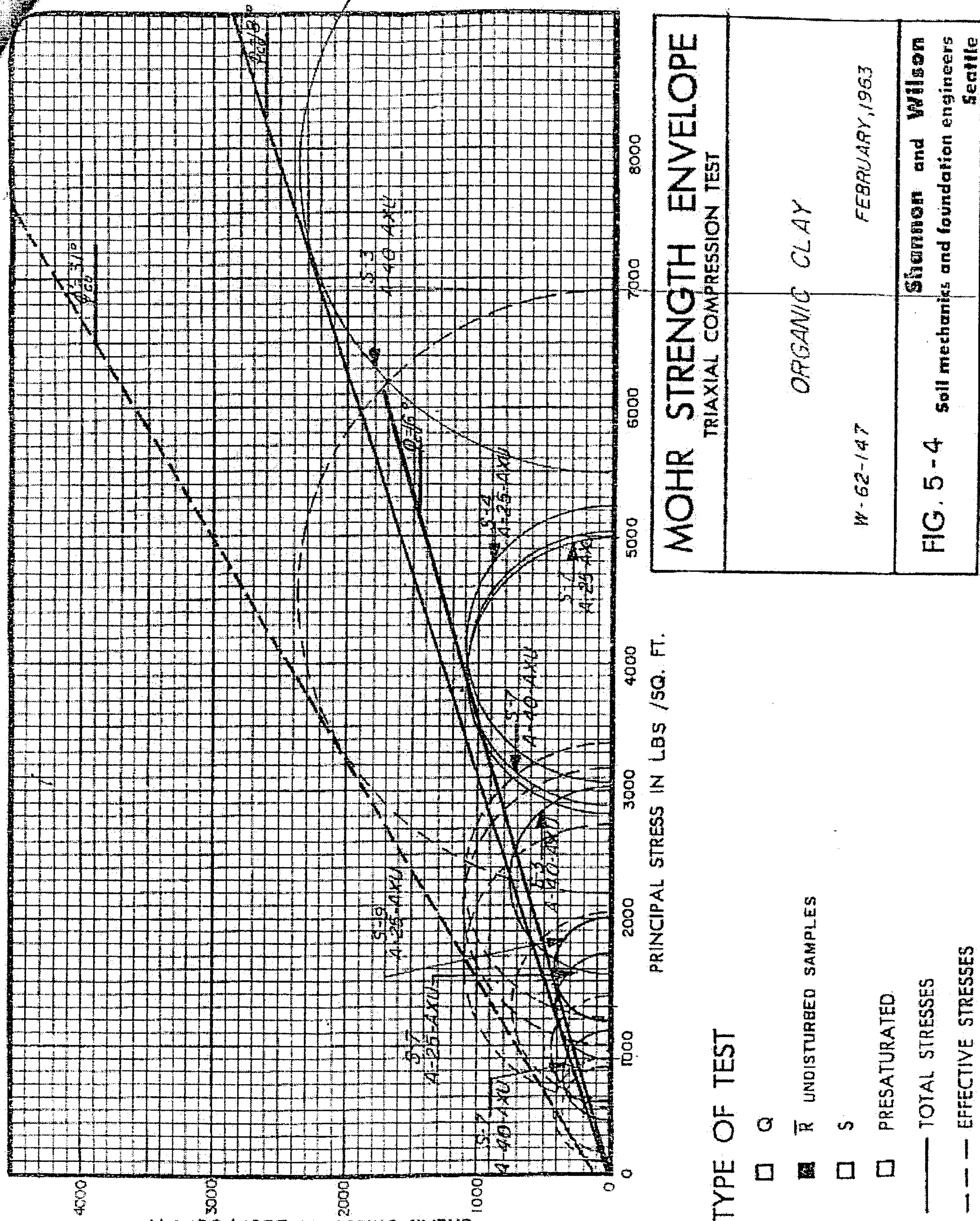


EXHIBIT C - I - 4
Excerpt from 1963 Shannon & Wilson Report on Foundation Conditions at Trinidad Arm of Gatun Lake
Reference 10

TABLE A-I SUMMARY OF SEATTLE LABORATORY TEST DATA

Boring No.	Sample No.	Depth, feet	Wet Density pcf	Natural Water Content, percent	Atterberg Limits LL	PL	PI	Other Tests	Sample Classification
A-23-AXII	1	77.3-79.8	60.5	558 565 574 349 793	337	165	172	$^{*}V=213$ $V_R=37$ $S_t=5.57$ $\bar{Q}_s=119$ C	Very soft, gray-brown, leafy and fibrous PEAT with wood fragments, rootlike roots and very sparse clots of gray clayey silt (1/8 in. diameter).
	2	80.6-83.2	64.5 66.0	264 250 215	219	86	133	$\bar{R}_s=215$ $\bar{Q}_s=92$ $V=141$ $V_R=34$ $S_t=4.47$ C S.G. = 1.57	Brown fibrous to finely divided PEAT with some leaves and wood fragments.
	3	84.3-87.3	91.2	72.5 73.2				S.G. = 2.64 $V=128$ $V_R=50$ $S_t=2.56$ $\bar{Q}_s=117$	Soft, gray-brown organic CLAY with scattered rusty yellow-brown spots and sparse light gray concretions 1/16 in.-3/4 in. diameter.
	4	87.3-89.8	93.1	91.8 83.4 82.1 114.7	89	44	45	C $V=158$ $V_R=30$ $S_t=5.27$ $\bar{Q}_s=127$	Soft, gray-brown, organic CLAY, mottled with yellow-brown streaks. (Gray fine sand layers to 1/2 in. at top of sample).
	5	89.3-92.4	88.3	124.6 135.0 114.5 85.0	123	53	70	$U_s=50$ $V=102$ $V_R=44$ $S_t=2.3$	Very soft, dark gray, organic CLAY with organic planes scattered throughout.
	7	94.9-97.8	83.2 85.1 58.6	141 143 136 146 306	142	64	78	$\bar{Q}_s=228$ $\bar{R}_s=423$ C $V=161$ $V_R=28$ $S_t=5.74$ $U=352$	Soft, gray to gray-brown, organic CLAY, mottled with yellow and black carbonized organic materials. Soft, dark brown, fibrous PEAT with wood fragments.
	9	95.8-101.3	74.8 102	183 167 151 171 184 182 182	174	78	96	$U_s=171$ C $V_1=258$ $V_{R1}=42$ $S_{t1}=6.11$ $\bar{R}_s=338$ $V_2=234$ $V_{R2}=55$ $S_{t2}=4.1$	Soft, light tan to gray-brown, mottled organic CLAY with abundant organic material. Layered with thin partings of fibrous, leafy to woody peat. Organic clay is bleached in 1/8 in.-3/4 in. zones above peat partings.
	10	102.8-105.4	64.7	202 271				$\bar{R}_s=1208$	Interbedded soft organic CLAY and firm fibrous PEAT.
								*See Sheet 9 for Legend.	
A-25-AXII	11	105.4-107.9	70.6 65.3	302 288 214 300 282	333	214	119	$\bar{Q}_s=394$ $U_s=66$ C $V=522$ $V_R=152$ $S_t=3.43$	Dark brown clayey PEAT. Soft, light gray, organic CLAY with trace of organic material. Dark brown PEAT with trace of thin (1/8 in.) clay layers. Peat is horizontally stratified with leafy material.
	13	130.4-132.8	35.8 41.6 50.5	117.3 120.6 117.0 89.9 101.3 121.8 126.2 142	139 145	51 60	88 85	$V_1=313$ $V_{R1}=92$ $S_t=3.4$ C $V_2=336$ $V_{R2}=64$ $S_t=4.2$ $U_s=126$ $\bar{Q}_s=303$	Soft, brown, organic CLAY with organic matter scattered throughout. Some pieces of wood evident.
	15	115.8-118.1	75.8 78.3	141 126 126 155 128 140 124	155	61	94	$V_1=408$ $V_{R1}=107$ $S_{t1}=4.72$ $\bar{Q}_s=363$ $V_2=468$ $V_{R2}=110$ $S_{t2}=4.26$ C $U_s=140$	Fibrous PEAT. Soft, brown, organic silty CLAY with partings (1/32 in.) of dark organic material.
	17	120.6-123.1	80.9 81.6	133 131 88.0 95.3 89.0 98.4 135	123	57	66	$\bar{Q}_s=522$ $V=430$ $V_R=86$ $S_t=4.94$ C S.G.=2.36 $U_s=519$	Fibrous PEAT. Soft dark brown-gray, mottled, organic silty CLAY with sparse organic material. Soft, brown, organic CLAY with streaks of tan organic clay. Layers composited organic material and wood.
	18	173.1-175.6	84.6 83.8 90.9 83.3 91.0 89.1	107.2 123.4 126.8 106.1 107.5 102.5 103.1 101.0 112.1				$V_1=117$ $V_{R1}=40$ $S_{t1}=3.5$ $\bar{Q}_1$=Void $\bar{Q}_{12}$(Med.)=566 $\bar{Q}_{13}$(Slow)=572 Q_{1a} Fast=605 $V_2=436$ $V_{R2}=100$ $S_{t2}=4.36$ $U_{1s}=670$ $U_{2s}=550$ C	Soft to medium, dark gray organic silty CLAY with yellow pockets of silty clay.
	19	126.8-129.1	85.2 85.1	137 106 110 154 103.8 107	162	69	93	$U_s=137$ $U=549$ $V_R=98$ $S_t=5.6$ C $\bar{Q}_s=765$ (partially consolidated)	Mottled tan-gray organic silty CLAY with irregular voids to 1/8 in. diameter.
	21	131.8-134.3	86.6 87.4	100 101 57 90 94.7 99.1 93	103	41	60	$V_1=361$ $V_{R1}=70$ $S_{t1}=4.69$ $U_s=294$ (Low-Fracture) C $V_2=408$ $V_{R2}=78$ $S_t=5.25$ $\bar{Q}_s=109$	Mottled olive tan to olive, organic silty CLAY, with very sparse organic material (stippled pattern).

EXHIBIT C - I - 4
Excerpt from 1963 Shannon & Wilson Report on Foundation Conditions at Trinidad Arm of Gatun Lake
Reference 10

EXHIBIT C - I - 4
Excerpt from 1963 Shannon & Wilson Report on Foundation Conditions at Trinidad Arm of Gatun Lake
Reference 10

TABLE A-1 SUMMARY OF SEATTLE LABORATORY TEST DATA

Boring No.	Sample No.	Depth, feet	Wet Density pcf	Natural Water Content percent	Atterberg Limits LL	PL	PI	Other Tests	Sample Classification
A-25-AXU	23	137.0-139.5	95.6	76.7 74.9 67.6 73.0 65.6 70.8	85	40	45	$V_l=401$ $V_R=61$ $S_t=7.77$ $U_s=401$ $V_2=401$ $V_{R2}=20$ $S_{t2}=9.34$ C $\bar{Q}_s=666$	Mottled olive-tan to olive organic silty CLAY with occasional light yellow concretions, thin (less than 1/32 in.) silt partings with leaf impressions. Some organic fragments and rust color spots.
	25	142.5-145.0	92.7 91.0	62.1 81.0 80.7 79.0 82.9 83.3 80.4 83.2 79.5	93	44	53	C $V=364$ $V_R=116$ $S_t=4.31$ $\bar{Q}_R=118$ $U_s=354$	Medium, gray to light gray, organic CLAY with yellow very dense silt layers and layers and partings of fine sand. Some peat and other organic materials. Some brown organic CLAY.
	27	147.5-149.5	90.6 92.1	88.5 85.0 82.4 85.4 81.7				$\bar{Q}_s=305$ $V=386$ $V_R=86$ $S_t=4.20$ $U_s=344$	Gray brown silty organic CLAY containing scattered yellow siltstone nodules (max. 3/4 in.) and scattered organic fragments. Some yellow streaks and pockets similar to nodules but not hard.
	31	157.6-160.1	91.1 91.7	94.0 82.3 80.9 79.4	116	46	70	$U_s=425$ $\bar{Q}_s=840$ $V=827$ $V_R=126$ $S_t=6.56$ C	Gray-brown silty organic CLAY with stratified 1/8-1/4 in. thick yellow, soft to locally hard "concretionary" streaks and layers. Thin gray to yellow-gray silt streaks, pockets, and partings.
	34	167.5-169.5	94.3 96.3	83.4 80.1 70.2 70.9 70.5	88	52	36	$V=932$ $V_R=182$ $S_t=5.00$ $U_s=654$ $\bar{Q}_s=840$ C	Gray-brown silty organic CLAY with very fine sand partings and thin stiff peat laminations. Gray-brown silty organic CLAY with occasional silt to very fine sand streaks and partings, and occasional yellow streaks of soft silty clay. Sparse organic streaks.
	35	175.8-177.3	95.0 95.5	81.8 81.3 77.1 82.6 81.6 79.8 79.3 75.1 114.6	111	51	60	$U_{s1}=780$ $U_{s2}=581$ C $V=652$ $V_R=106$ $S_t=351$ $\bar{Q}_{s1}=1155$ $\bar{Q}_{s2}=1035$	Medium, dark brown, organic CLAY with organic matter scattered throughout; slight odor, occasional thin (1/2 to 1/4 in.) layer of yellow silt and siltstone and trace of very thin seams or pockets of fine sand.
	51	177.0-179.3	94.2 99.5	81.1 76.1 72.9 79.4	112	47	65	C $\bar{Q}_s=791$ $V=876$ $V_R=152$ $S_t=5.73$ $U_s=776$	Gray-brown, slightly organic silty CLAY with thin very fine sand-silt partings and occasional yellow alterations. Occasional thin (1/32 in.) peat parting. Yellow portion is very sensitive clay.
	53	183.0-185.3	91.7	75.3 76.8 72.6				$U_s=537$	Brown-gray silty CLAY with very fine sandy silt. Some organic matter and siltstone nodules.

TABLE A-1 SUMMARY OF SEATTLE LABORATORY TEST [DATA]

Boring No.	Sample No.	Depth, feet	Wet Density pcf	Natural Water Content percent	Atterberg Limits LL	PL	PI	Other Tests	Sample Classification
A-25-AXU	55	188.0-190.5	91.4 95.2	73.2 70.2 74.6 75.4 74.2				$U_s=577$ $V=862$ $V_R=178$ $S_t=4.84$ $\bar{Q}_s=1240$	Gray-brown silty CLAY with very fine sandy partings and pockets. Occasional pockets and thin partings of organic material. Yellow alterations, generally soft or slightly harder than sample, occasionally concretionary and hard.
	58	201.0-203.5		60.1 59.2 49.0 56.2					Gray silty fine SAND with thin organic clay partings. Some thin peat layers (1/8 to 1/4 in.). Trace of shells. Occasional yellow silty clay layers.
	59	204.0-206.5	95.5	63.8 63.9 50.0				$\bar{Q}_s=2050$	Medium, layered brown-gray fine SAND and gray organic silty CLAY to organic clay.
	61	213.0-215.5	82.8	111.2 112.0 114.7	178	102	76	C $\bar{Q}_s=2100$	Stiff, dark brown, irregularly layered, woody PEAT with gray silty clay.
	62	223.0-225.5		49.6 41.7					Gray-brown silty fine to medium SAND.
	60	227.2-229.7							Medium gray and brown, organic stained silty medium to fine SAND.
	69	230.2-232.7	97.3	41.3 60.8 18.1				$\bar{Q}_s=779$	Dense gray-brown silty fine to medium SAND with some organic material.
UP-35-C[illegible]	B-6	106.0-108.5	15.7	219 178				$\bar{R}_t=1450$	Very soft gray organic CLAY with organic inclusions. Firm black fibrous PEAT. Same as top.

EXHIBIT C - I - 4
Excerpt from 1963 Shannon & Wilson Report on Foundation Conditions at Trinidad Arm of Gatun Lake
Reference 10

TABLE A-1 SUMMARY OF SEATTLE LABORATORY TEST DATA

Boring No.	Sample No.	Depth, feet	Wet Density pcf	Natural Water Content percent	Atterberg Limits LL	PL	PI	Other Tests	Sample Classification
UP-40-CU	S-1	78.5-80.0		156 277 344				$V_1=77$ $V_{R_1}=16$ $S_{t_1}=4.8$ $V_2=62$ $V_{R_2}=16$ $S_t=3.9$	Very soft decayed PEAT with pockets of very soft gray organic clay.
	S-2	85.0-87.5 (5" Shelby)	 77.2 78.5 75.4 75.4	317 264 198 204 212 208 138				$\bar{Q}_{s_1}$ = Void $\bar{Q}_{s_2}=195$ $\varepsilon=0.62$ $\bar{Q}_{s_3}=200$ $\varepsilon=2.69$ $\bar{Q}_{s_4}=182$ $\varepsilon=0.29$ $\bar{Q}_{s_5}=170$ $\varepsilon=0.12$	Very soft black to dark brown PEAT. Very soft brown-gray organic CLAY with pockets of clayey peat.
	S-3	83.5-85.0		225 178 170 355				$V=150$ $V_R=27$ $S_t=5.0$	Very soft black-brown decayed PEAT with layers of very soft gray organic peaty clay. Very soft gray organic CLAY with layers of black peat.
	S-5	88.5-90.0		130 132 108 133				$V=152$ $V_R=31$ $S_t=3.6$	Soft gray-mottled-yellow organic CLAY with pockets and layers of black peat.
	S-7	93.5-95.0		159 182 406 310				$V=367$ $V_R=52$ $S_t=7.0$	Soft gray organic CLAY with pockets of black peat. Soft black PEAT with some peaty CLAY. Soft black PEAT.
	S-9	98.5-100.0		315 154 430 388 335				$V_1=212$ $V_{R_1}=25$ $S_{t_1}=8.1$ $V_2=284$ $V_{R_2}=44$ $S_{t_2}=6.7$	Very soft brown organic CLAY with pockets of organic material. Brown decayed fibrous PEAT. Very soft brown organic CLAY with pockets of organic material.
	S-11	103.5-105.0		304 104 368 384 455				$V_2=483$ $V_{R_2}=50$ $S_{t_2}=8.3$ $V_1=440$ $V_{R_1}=41$ $S_{t_1}=10.7$	Firm brown fibrous PEAT with occasional layers of organic clay.
	S-13	108.5-110.0		381 650 522 568 545				$V_1=556$ $V_{R_1}=64$ $S_{t_1}=8.5$ $V_2=628$ $V_{R_2}=68$ $S_{t_2}=9.1$	Firm black decayed fibrous PEAT.

TABLE A-1 SUMMARY OF SEATTLE LABORATORY TEST D

Boring No.	Sample No.	Depth, feet	Wet Density pcf	Natural Water Content percent	Atterberg Limits LL	PL	PI	Other Tests	Sample Classification
UP-40-CU	S-14	112.5-114.8		365 590 503 485				$V=380$ $V_R=134$ $S_t=2.7$	Firm dark brown decayed fibrous PEAT.
	S-15	117.5-119.8		158 141 161				$V_2=311$ $V_{R_2}=62$ $S_t=5.3$ $V_1=424$ $V_{R_1}=54$ $S_t=7.9$	Soft gray organic CLAY with pockets of peat.
	S-16	122.5-124.8		145 144 195 190 181				$V_1=206$ $V_{R_2}=46$ $S_{t_1}=6.2$ $V_2=388$ $V_{R_2}=50$ $S_{t_2}=7.8$	Very soft gray-brown to dark gray organic CLAY with pockets of peat.
	S-17	127.5-129.8		142 140 147 152 144 145				$V_2=350$ $V_{R_2}=36$ $S_{t_2}=5.4$ $V_1=380$ $V_{R_1}=58$ $S_{t_1}=6.7$	Very soft brown and gray organic peaty CLAY with roots, leaves, twigs, and peat pockets.
DR-40-CU	S-9	122.0-124.3		54.4 53.3 53.0					Medium light brown SAND Soft dark gray organic CLAY with leaves and stems. Soft blue-gray CLAY with siltstone nodules and occasional organics.
UP-15-CU	S-1	90.0-92.5	84.9 91.5	106 104 93.0 96.0 42.0 85.5 81.0	115	37	80	C Triaxial Dissipation Test (See Table A-4) Special Atterberg Limit Tests (See Table A-4) Triaxial Dissipation Test (See Table A-4)	Very soft purple-gray organic CLAY.
	S-11	101.0-103.3		210					Interbedded PEAT and peaty CLAY.
UP-50-CU	S-5	57.0-59.3		Sand 57.1 Clay 70.5					Interbedded brown and gray silty fine SAND and gray organic CLAY with occasional black organic inclusions.

TABLE A-1 SUMMARY OF SEATTLE LABORATORY TEST DATA

Boring No.	Sample No.	Depth, feet	Wet Density pcf	Natural Water Content percent	Atterberg Limits LL	PL	PI	Other Tests	Sample Classification
DF-25-CU	S-13	106.0-108.5 (5" Shelby)		140 321 131					Very soft gray organic CLAY with organic inclusions. Firm black fibrous PEAT. Same as top.
DN-25-CU	S-2	95.0-97.5 (5" Shelby)	78.4	161 162 188 219 210				$\bar{Q}_{s1}=206$ $S_R=2.60$ $\bar{Q}_{s2}=202$ $S_R=1.38$ $\bar{Q}_{s3}=113$ $S_R=0.47$ $\bar{Q}_{s4}=107$ $S_R=0.05$	Very soft black organic peaty CLAY.
	S-3	71.0-81.5		574 650 680				Photo - See fig. 3-4	Very soft brown loose decayed fibrous PEAT.
	S-6	85.0-87.5		94.5 179 244				Photo - See fig. 3-4	Very soft blue-gray CLAY. Very soft gray peaty organic CLAY. Very soft dark gray organic CLAY interbedded with PEAT.
	S-7	91.0-93.5		137 155 143				Photo - See fig. 3-4	Very soft brown and brown-gray organic CLAY with peat inclusions.
	S-9	97.0-99.5		171 184 148				Photo - See fig. 3-4	Very soft gray organic CLAY interbedded with brown fibrous PEAT.
	S-11	103.0-105.5		186 274 118				Photo - See fig. 3-5	Very soft dark gray organic CLAY interbedded with dark brown peat and wood.
	S-13	109.0-111.5		216 237 309				Photo - See fig. 3-5	Very soft dark gray organic CLAY laminated with brown compact PEAT.

TABLE A-1 SUMMARY OF SEATTLE LABORATORY TEST D

Boring No.	Sample No.	Depth, feet	Wet Density pcf	Natural Water Content percent	Atterberg Limits LL	PL	PI	Other Tests	Sample Classification
DN-25-CU	S-16	118.0-120.5		108 110 112				Photo - See fig. 3-5	Very soft gray-brown organic CLAY with occasional silt pockets.
	S-17	124.0-126.5		103 89.0 101				Photo - See fig. 3-5	Very soft brown and dark gray organic CLAY with scattered siltstone nodules.
A-40-AXU	S-11	87.5-90.0 (5" Shelby)		66.7 68.5 66.0 98.3 80.7 84.1					Very soft very decayed PEAT and light blue-gray CLAY Very soft light blue-gray slightly organic CLAY.
	S-3	83.0-85.5	95.8 90.7	56.9 61.5 65.0 70.5 67.1				$\bar{R}_{s1}$ = 2300 ($\bar{\sigma}_c$ = 5400) $\bar{R}_{s2}$ = 745 ($\bar{\sigma}_c$ = 1550)	Very soft gray-brown organic silty CLAY stratified with thin layers of organic material.
	S-7	93.0-95.5	91.8 97.5	97.0 80.1 88.0 86.3 81.5				$\bar{R}_{s1}$ = 1083 ($\bar{\sigma}_c$ 2000) $\bar{R}_{s2}$ = 341 ($\bar{\sigma}_c$ = 420)	Very soft blue gray organic CLAY with organic pockets and occasional layers of clayey peat to 1 inch thick.
	S-9	98.0-100.5	73.5	115 148 130 140 131				$\bar{R}_s$ = 695	Soft black PEAT interbedded with soft organic CLAY. Soft dark to light gray organic CLAY with layers of black organic material.
	S-11	103.0-105.5	60.1	231 253 184				$\bar{R}_s$ = 670	Black slightly clayey fibrous PEAT.
	S-13	108.0-110.5		346 337 424					Compact black and brown PEAT with large wood fragment 0.5 feet thick.

EXHIBIT C - I - 4
~~Excerpt from 1963 Shannon & Wilson Report on Foundation~~
Conditions at Trinidad Arm of Gatun Lake
Reference 10

TABLE A-1: SUMMARY OF SEATTLE LABORATORY TEST DATA

Boring No.	Sample No.	Depth, feet	Wet Density pcf	Natural Water Content percent	Atterberg Limits			
					LL	PL	PI	

LEGEND

C Consolidation Test

LL Liquid Limit

PL Plastic Limit

PI Plasticity Index

$\bar{Q}_s$ Shear Strength, psf, determined from undrained triaxial test. Bar indicates test conducted with pore pressure measurements.

$\bar{R}_s$ Shear Strength, psf, determined from consolidated undrained triaxial test. Bar indicates test conducted with pore pressure measurements.

S.G. Specific Gravity

ϵ_R Strain Rate, percent strain per minute.

S_t Sensitivity (V/V_R)

U_c Shear Strength, psf, determined from unconfined compression test.

V Undisturbed Laboratory Vane Shear Strength, psf.

V_R Remolded Laboratory Vane Shear Strength, psf.

σ_c Consolidation Pressure

NOTE

Refer to Table A-3 for detailed summary of triaxial test results.

EXHIBIT C - I - 4
Excerpt from 1963 Shannon & Wilson Report on
Foundation Conditions at Trinidad Arm of Gatun Lake
Reference 10

TABLE 2.1 Comparison Between Flow in Open Channels and Pipes and Seepage Through Soils and Aggregates

Flow medium	Typical coefficients of permeability, ft/day	Effective channel diameter	Hyd. gradient	Discharge velocity, ft/sec	Q, cfs	Q, gpm	Square feet needed for discharge of 2 in. pipe[a]
Smooth channel		80 ft = 2R	0.01	35	450,000		
Smooth pipe		8 ft = d	0.01	15	750		
		1 ft = d	0.01	4.5	3.5		
		2 in. = d	0.01	0.5	0.018	8	2 in. pipe
$1\frac{1}{2}$–1 in. gravel	140,000	0.2 in.	0.01	0.015	0.015[b]	6.8[b]	1.2
1–$\frac{1}{2}$ in. gravel	50,000	0.1 in.	0.01	0.005	0.005[b]	2.3[b]	3.5
$\frac{1}{8}$"–#4 gravel	8,000	0.03 in.	0.01	0.0008	0.0008[b]	0.36[b]	22
Coarse sand	800	0.01 in.	0.01	0.0001	0.0001[b]	0.045[b]	180
Fine sand, or graded filter agg.	1.0	0.002 in.	0.01	1×10^{-6}	1×10^{-6}[b]	4.5×10^{-4}[b]	18,000
Silt	0.001	0.00024 in.	0.01	1×10^{-8}	1×10^{-8}[b]	4.5×10^{-7}[b]	18,000,000
Fat clay	0.00001	0.00004 in.	0.01	1×10^{-11}	1×10^{-11}[b]	4.5×10^{-9}[b]	2,000,000,000

[a] $S = 0.01$.

[b] Q per square foot area.

EXHIBIT C - I - 5
Table 2.1 from Seepage, Drainage, and
Flownets by H. Cedegren
Refernce 14

sandstone;[18] consequently the tests for the determination of the shearing strength of the dark-grey sandstone are considered to be applicable to all the foundation rocks of the new Gatun Locks.

Backfill

5-73. The first design of the lock walls was based on the assumption that the backfill behind the walls would be free-draining, permitting the assumption of a low water table behind the wall. The soft and friable character of the rock found in the core drilling led to some apprehension as to the assumption of a free-draining backfill. After excavation for the lock structure had begun, it was apparent that the rock broke down considerably upon handling, and it was decided to investigate the problem more thoroughly.

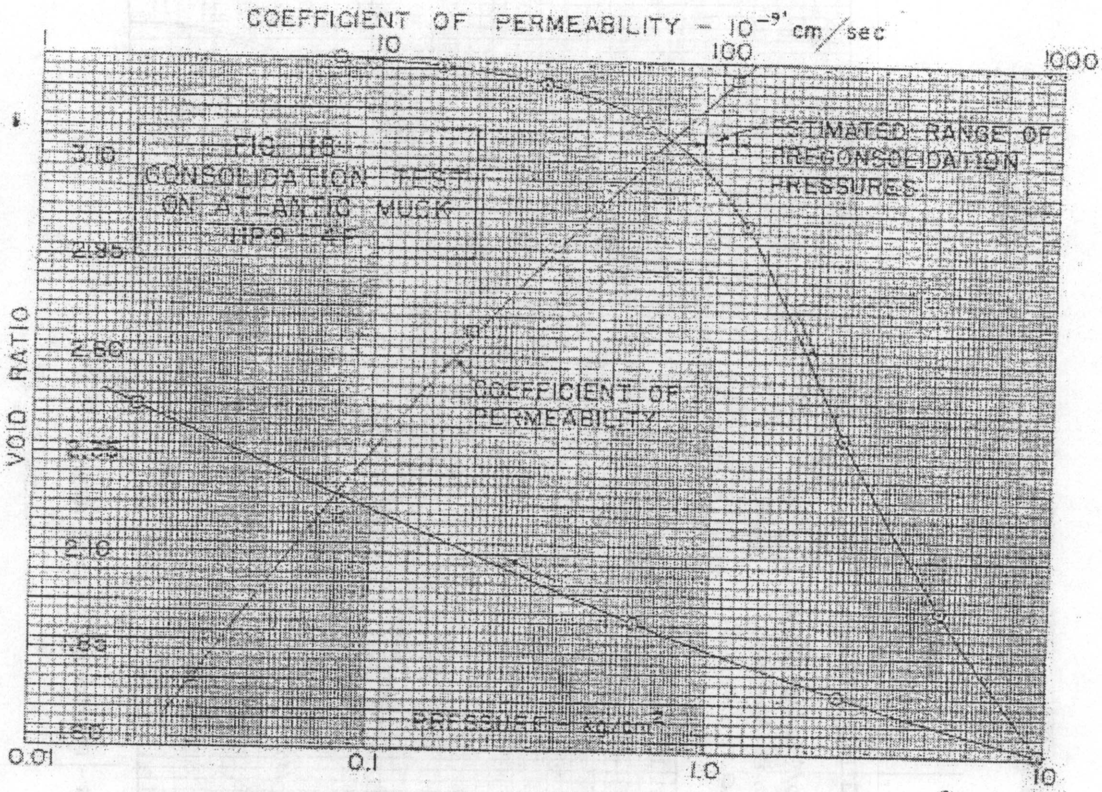
5-74. Experiments were made in the cut to determine the relative resistance to breakdown and the permeability of the fines resulting from breakdown. Considerable breakdown of the different materials took place in abrasion tests. It was found that the fines would be suitable for pervious backfill if handled properly.[20]

5-75. Using various methods of compaction, field tests were made on the rock from the excavation.[21] These tests showed that the method of compaction provided by the backfill specifications, which requires layers 1 foot thick, compacted by careful routing of loading equipment over the layers, will result in a dense, strong backfill. γ_{dry} * The dry density of the compacted material will be about 85 pounds per cubic foot with a moisture content of 30 per cent of the dry weight. However, the backfill will probably be quite impervious and may have a high water table. Accordingly, the lock walls were designed for a water table varying in a straight line from Gatun Lake level (elevation 87) at the upstream end to sea level at the lower end.

5-76. The angle of internal friction for the backfill was assumed as 32 degrees for use in obtaining the earth pressures. ϕ * Tests on the fine material resulting from breakdown of the Gatun rocks indicated angles of approximately 32 to 33 degrees, and since the fines will be the weakest material in the mass of the backfill, the design assumption was verified as being satisfactory.[21]

Foundation Drainage

5-77. The thickness of the floor of the upper flight of the existing Gatun Locks was increased, and the floor was anchored by means of steel rails grouted into the foundation rock, so that it would resist full hydrostatic head. This precaution was taken since part of the floor covers a pervious stratum of tuffaceous sandstone, which



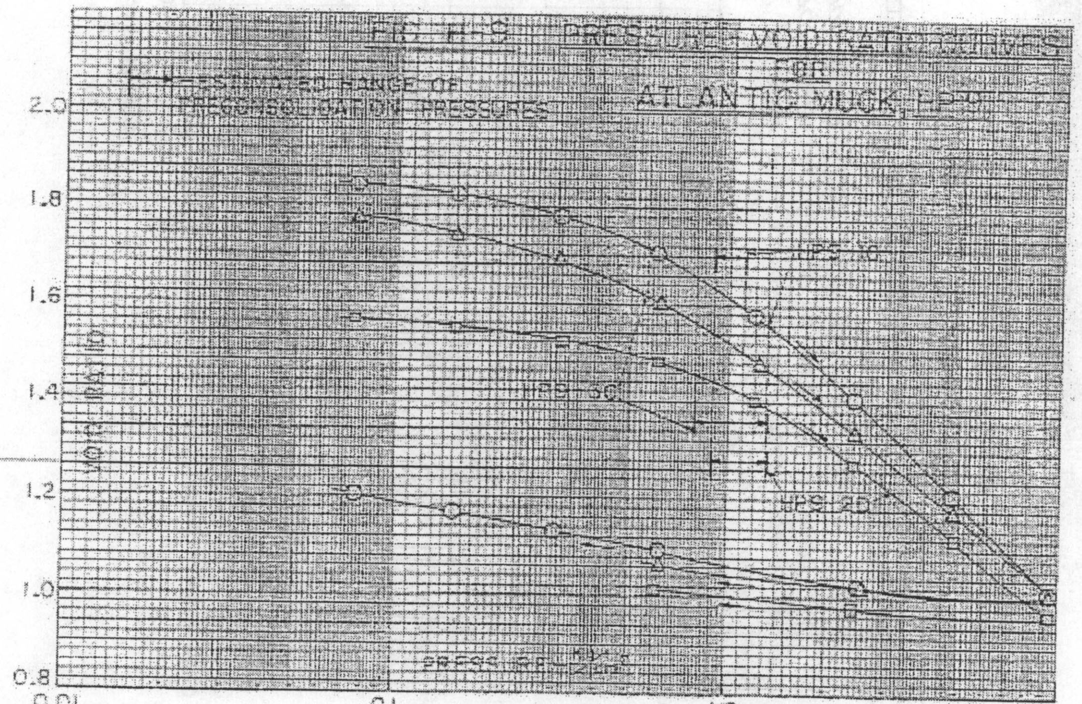
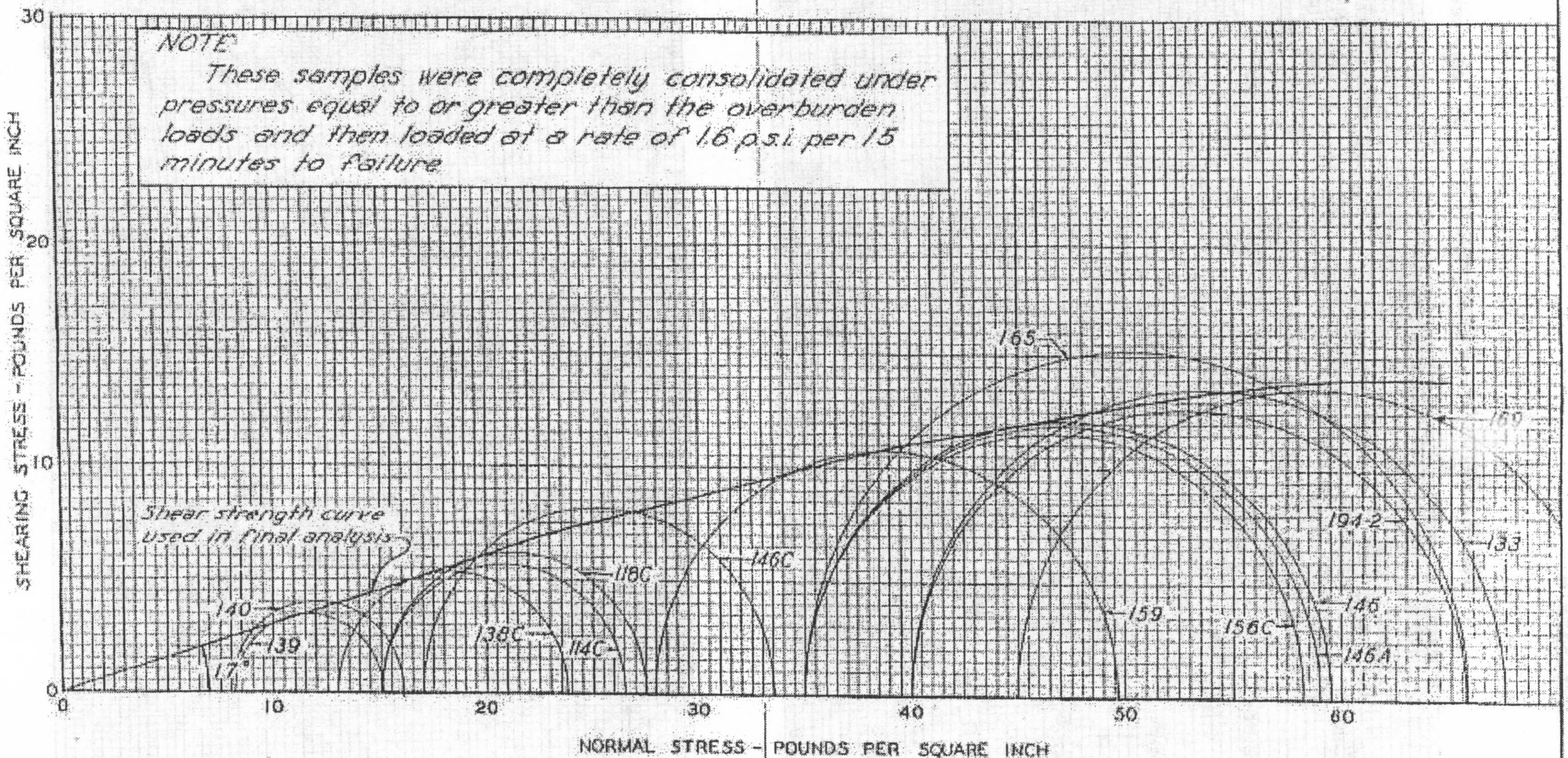


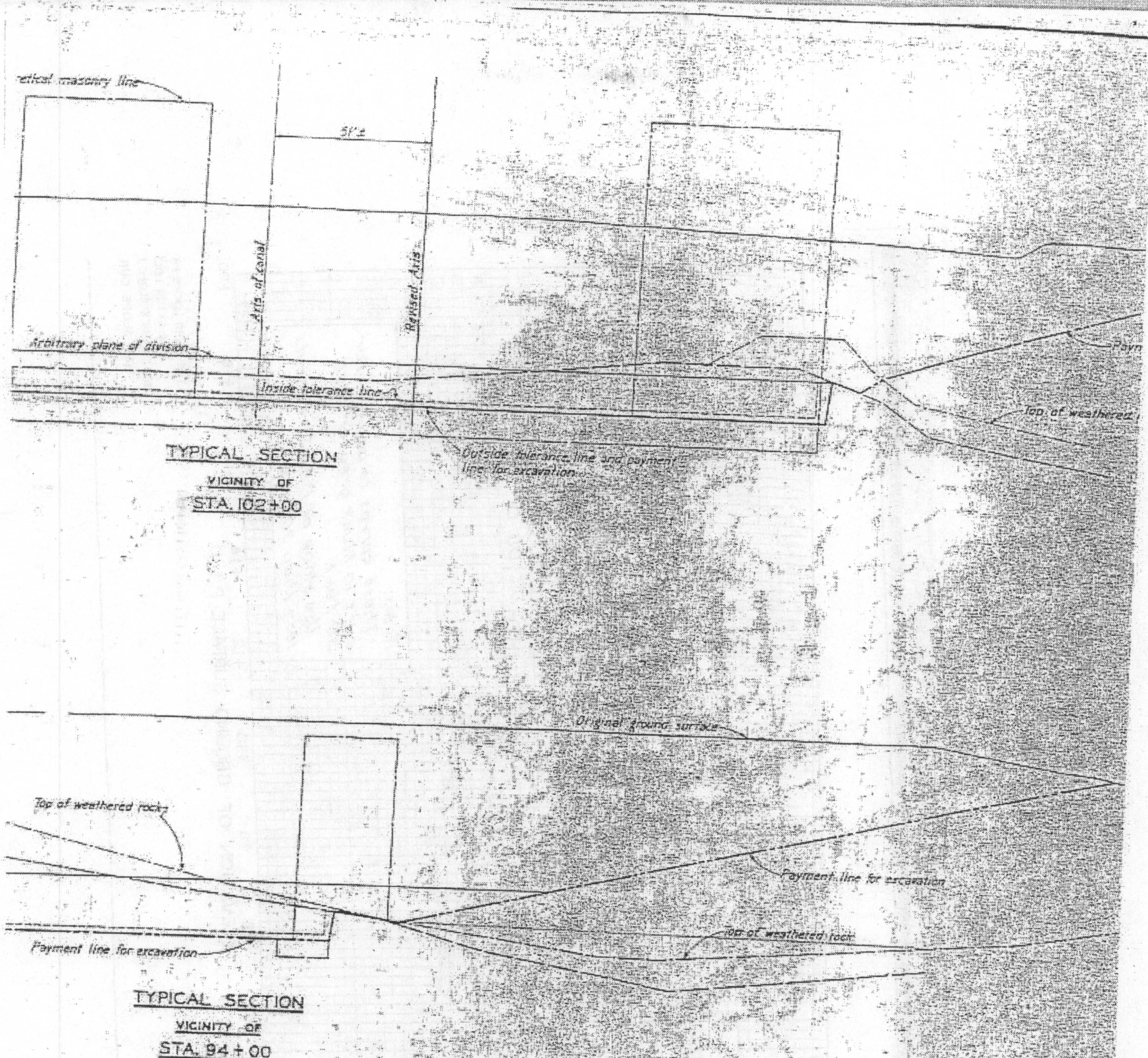
EXHIBIT C - I - 6
Excerpt from 1943 PCC Report
Reference 2



THE THIRD LOCKS PROJECT
NEW GATUN LOCKS
BLACK MUCK SLOPE DESIGN
MOHR'S CIRCLES

FIGURE 5-27

EXHIBIT C - I - 6
Excerpt from 1943 PCC Report
Reference 2



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Excerpt from 1943 PCC Report
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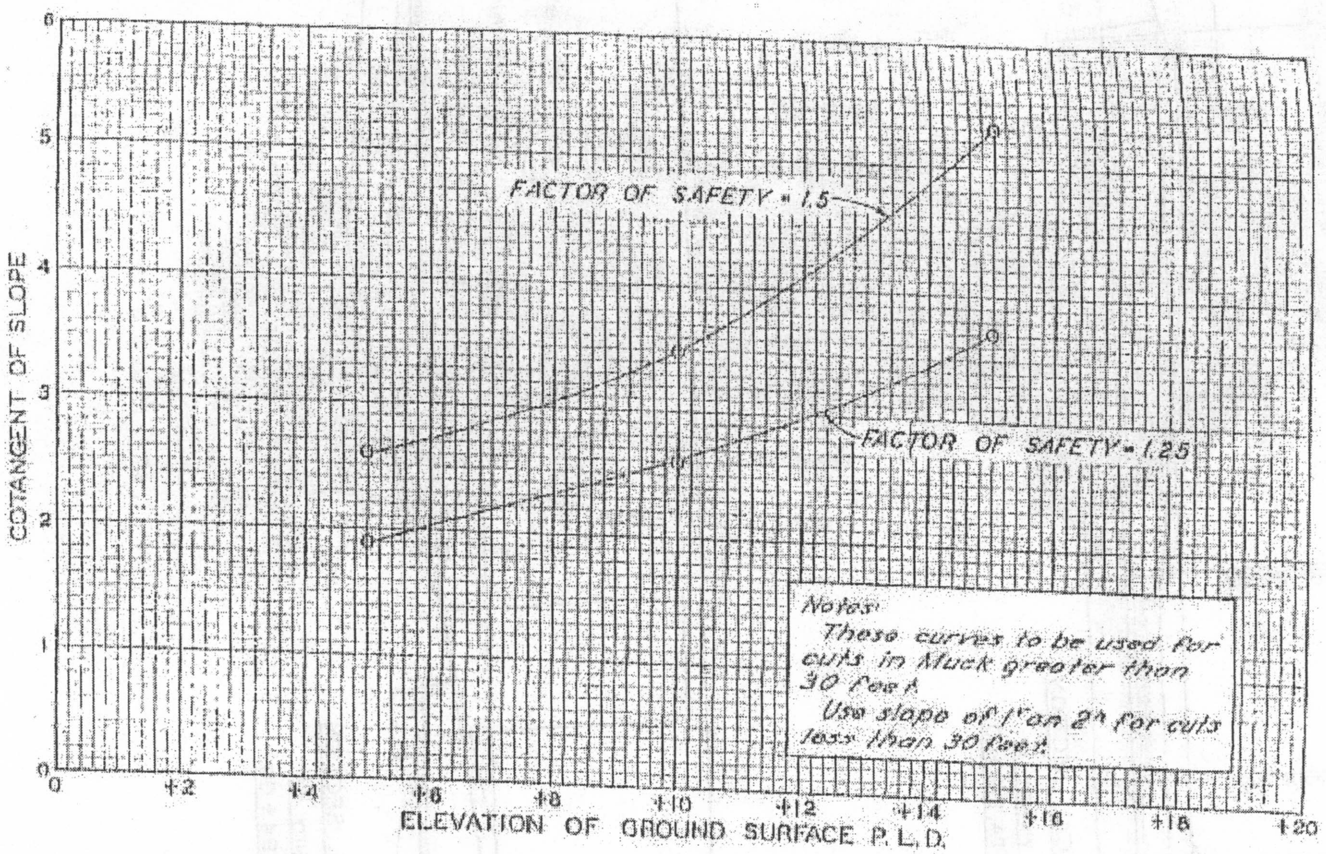


THE THIRD LOCKS PR
NEW GATUN LOC
BLACK MUCK SLOPE
WET SLOPE CUR

FIGURE 5-23

FIGURE 5-1

EXHIBIT C-1-6
Excerpt from 1943 PCC Report
Reference 2

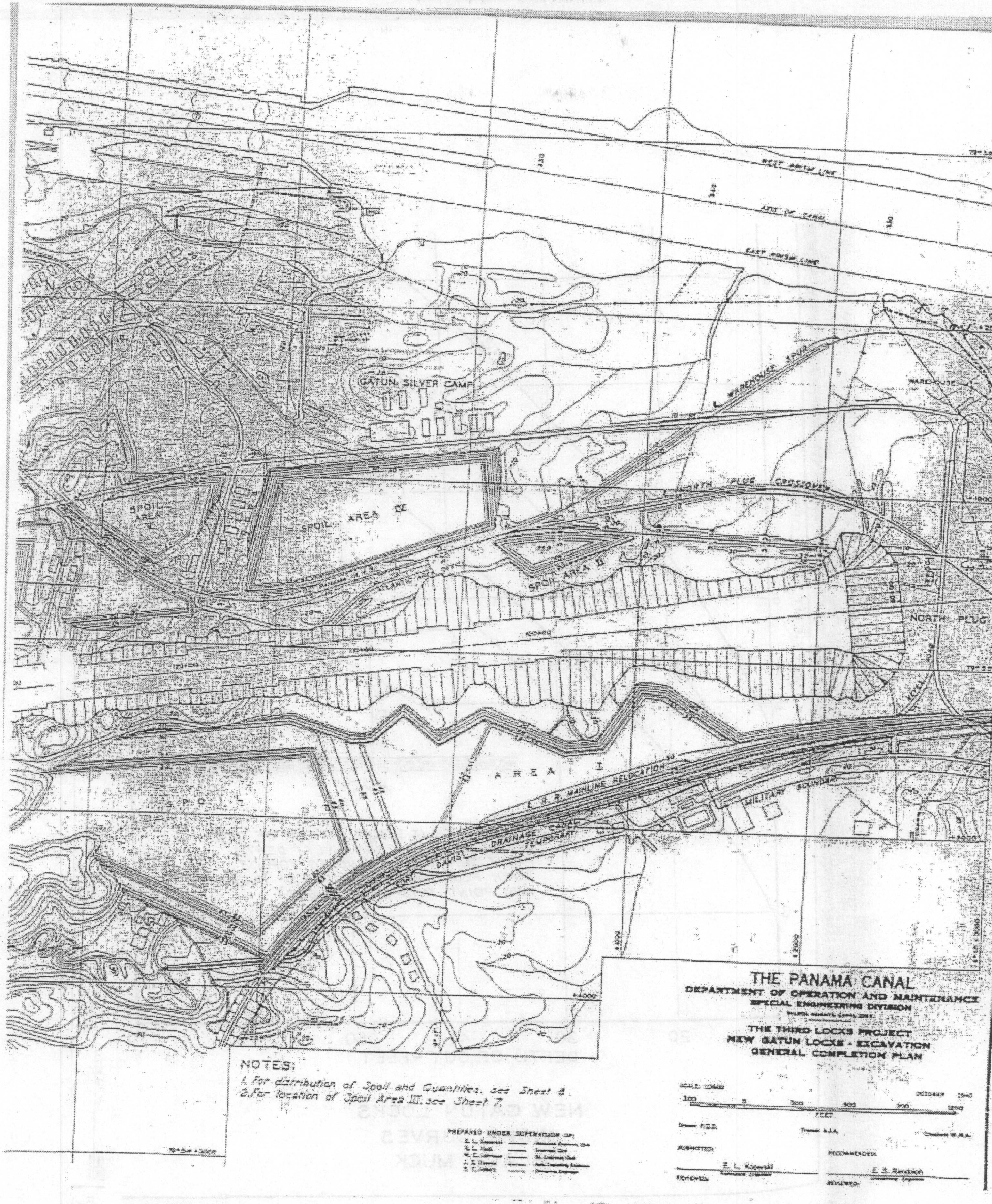
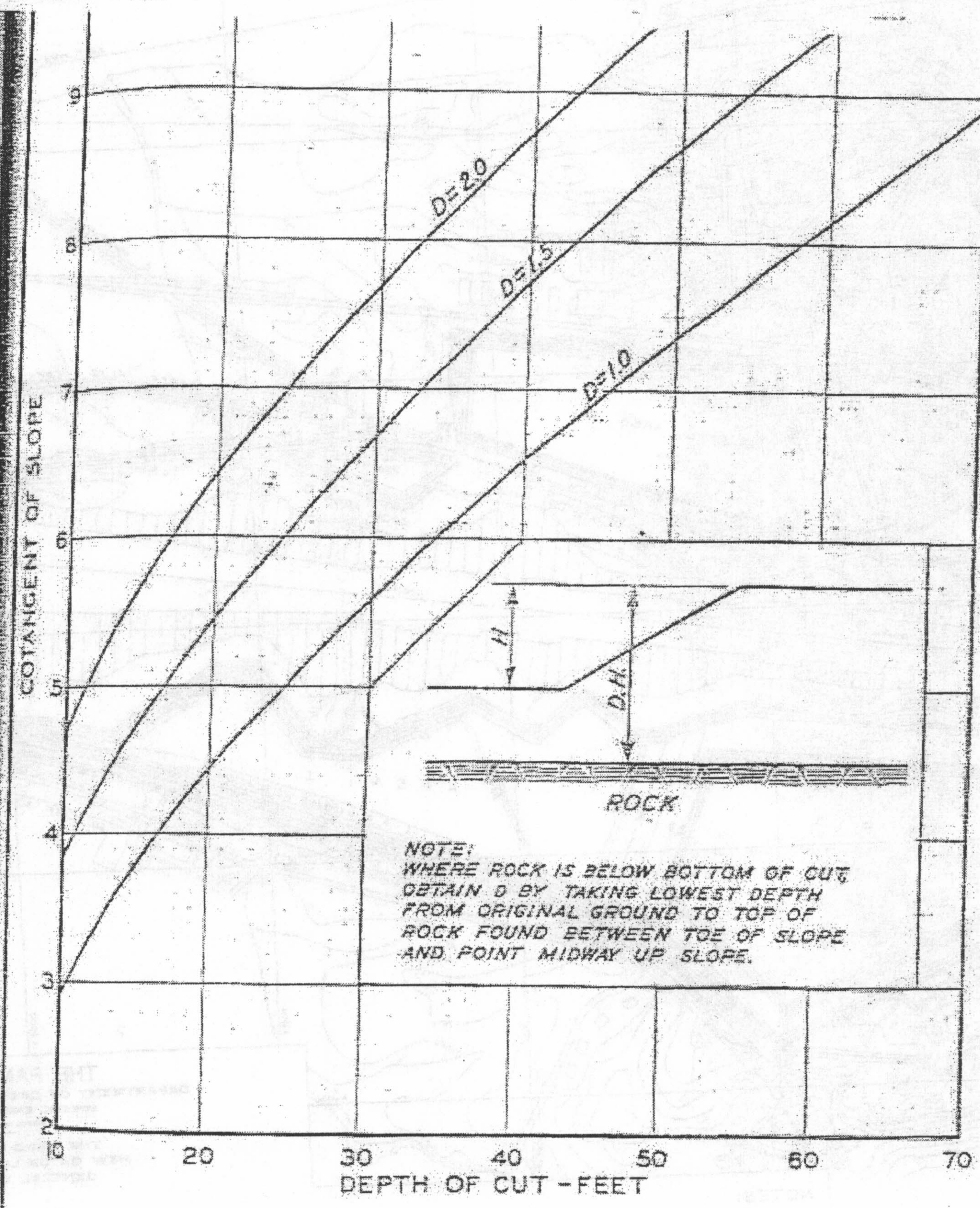


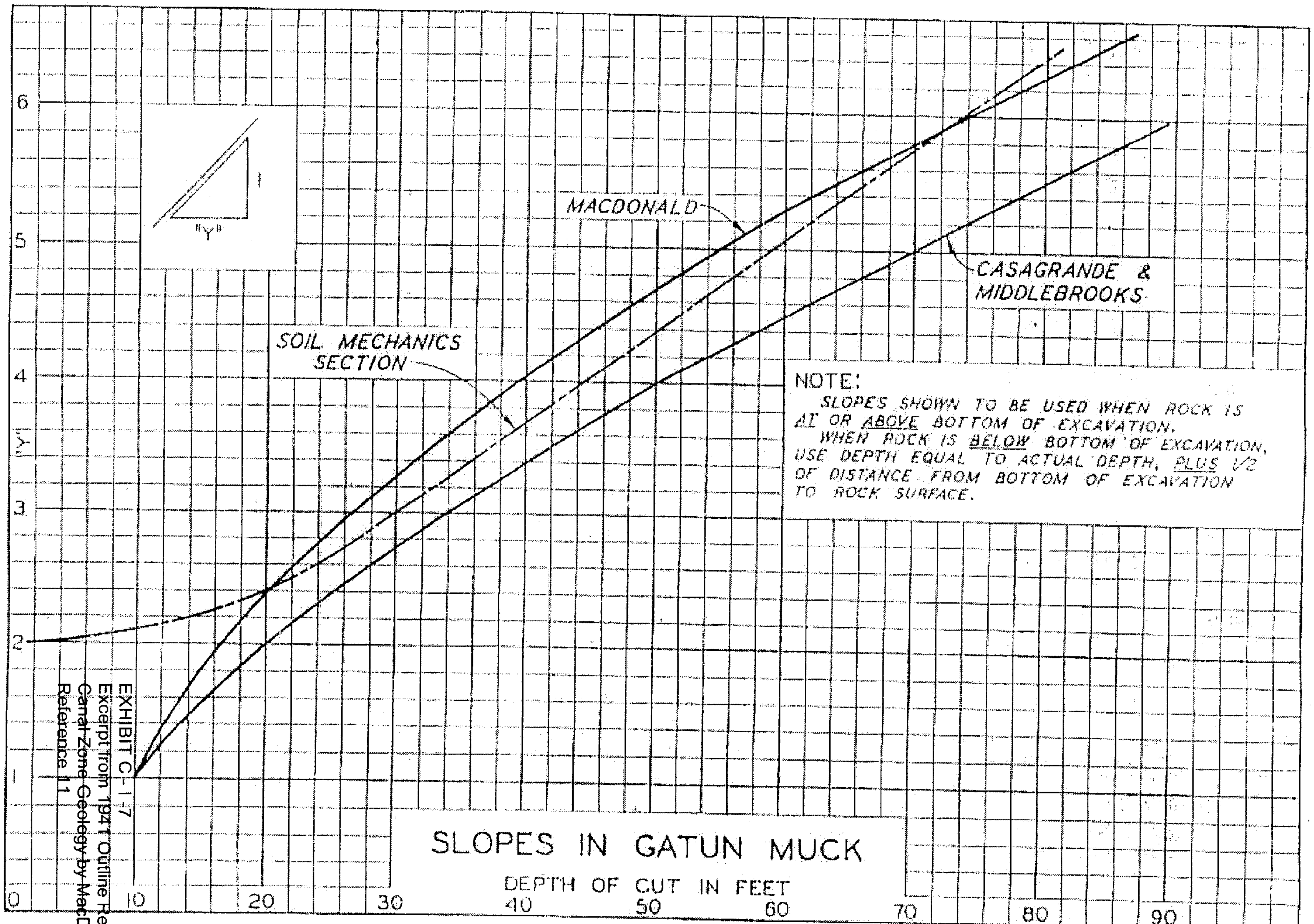
EXHIBIT C - I - 6
Excerpt from 1943 PCC Report
Reference 2



NEW GATUN LOCKS
SLOPE CURVES
BLACK MUCK

FIGURE 5-2

EXHIBIT C - I - 6
Excerpt from 1943 PCC Report
Reference 2



TO ACCOMPANY GEOLOGIC REPORT BY MACDONALD MAR. 31, 1941 FIGURE 10

EXHIBIT C-1.7
Excerpt from 1941 Outline Report
Canal Zone Geology by MacDonald
Reference 11

TABLE 4-8 **Presumptive bearing capacities from indicated building codes, psf**
Soil descriptions vary widely between codes—following represents author's interpretations

Soil description	Chicago, 1986	Natl. Board of Fire Underwriters 1976	Atlanta† 1973	BOCA‡ 1984	Uniform Bldg. Code, 1976§ *	
Clay, very soft	500					
Clay, soft	1 500	2 000	1 000	3 000	2 000	
Clay, ordinary	2 500					
Clay, medium stiff	3 500	2 000	2 000		2 000	= 0.01 MPa RANDOM BACKFILL
Clay, stiff	4 500		3 000	4 000		
Clay, hard	6 000				4 000	
Sand, compact and clean	5 000		6 000	12 000		
Sand, compact and silty	3 000					
Inorganic silt, compact	2 500					
Sand, loose and fine				4 000	4 500	
Sand, loose and coarse, or sand-gravel mixture, or compact and fine		3000 to 8000		8 000	6 000	
Gravel, loose and compact coarse sand	6 000			8 000	6 000	
Sand-gravel, compact			8 000	12 000	6 000	= 0.28 MPa STRUCTURAL BACKFILL
Hardpan, cemented sand, cemented gravel	12 000	20 000		20 000		
Soft rock				20 000		
Sedimentary layered rock (hard shale, sandstone, siltstone)		30 000		50 000	30 000	
Bedrock	200 000	200 000		200 000	200 000	

† Use of presumptive pressures limited to structures not over four stories.
‡ Building Officials and Code Administrators International, Inc.
§ Author interpretation.

4-15 SAFETY FACTORS IN FOUNDATION DESIGN

Buildings are designed on the basis of determining the service loads and obtaining a suitable ratio of material strength to these loads termed a safety factor SF. None of the quantities in this ratio are precisely known, so that codes or experience are relied upon to develop the ratio as, hopefully, a lower-bound value—the real value is this or something larger. Engineering materials such as steel and concrete are "manufactured" with strict quality control; nevertheless, in strength design for concrete the effective ultimate strength is taken as 85 percent of the unconfined compressive strength. The yield stress for steel and other metals is a "lower-bound" value—in the case of steel on the order of 10 to 20 percent less than the general range of measure yield strengths. Thus a "safety factor" of sorts is already applied.

Code values used to develop live and other loads are a compromise between upper and near upper bound. Building self-weight, or dead load, is reasonably

Exhibitt C - I - 8
Table 4-8 from Foundation Analysis and Design by Bowles
Reference 12

EXHIBIT C - I - 9
Table 4.1 from Correlations of Soil Properties
By Carter and Bently
Reference 13

Table 4.1 TYPICAL PERMEABILITY VALUES FOR SOILS

Coefficient of permeability (log scale)	m/s: 10^{-11}, 10^{-10}, 10^{-9}, 10^{-8}, 10^{-7}, 10^{-6}, 10^{-5}, 10^{-4}, 10^{-3}, 10^{-2}, 10^{-1}, 1
	cm/s: 10^{-9}, 10^{-8}, 10^{-7}, 10^{-6}, 10^{-5}, 10^{-4}, 10^{-3}, 10^{-2}, 10^{-1}, 1, 10, 100
	ft/s: 10^{-10}, 10^{-9}, 10^{-8}, 10^{-7}, 10^{-6}, 10^{-5}, 10^{-4}, 10^{-3}, 10^{-2}, 10^{-1}, 1
Permeability:	Practically impermeable / Very low / Low / Medium / High
Drainage conditions:	Practically impermeable / Poor / Good
Typical soil groups:	GC→ GM→ SM SW→ GW→
	CH SC MH MC–CL SM–SC SP→ GP→
Soil types:	Homogeneous clays below the zone of weathering / Silts, fine sands, silty sands, glacial till, stratified clays / Clean sands, sand and gravel mixtures / Clean gravels
	Fissured and weathered clays and clays modified by the effects of vegetation

Note: the arrow adjacent to group classes indicates that permeability values can be greater than the typical value shown.

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Appendix C – Geotechnical Investigations, Analyses and Designs

Part I – Soils

Computations

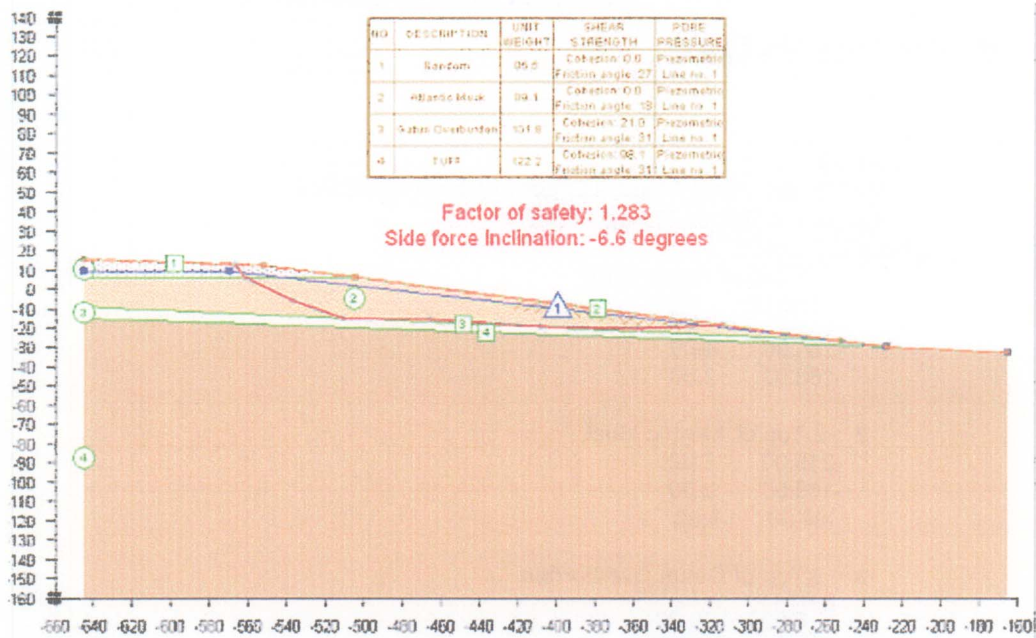
Input and output files from UTEXAS4 Stability Analysis Software

QUALITY CHECK

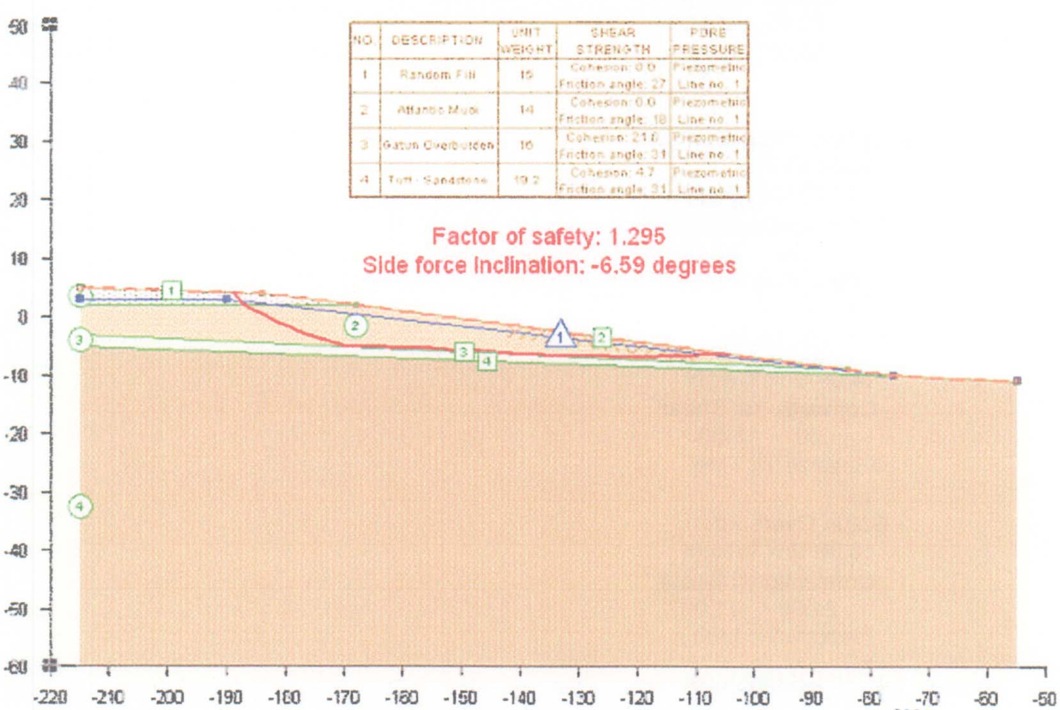
Comparison of Results of UTEXAS4 Software

Metric -vs- English Units

UTEXAS4 Results Using English (ft-lb) Units



UTEXAS4 Results Using Metric (kg-m) Units



INPUT & OUTPUT FILES
MOST CRITICAL DESIGN SLOPE
RESULTS OF UTEXAS4 SOFTWARE
STABILITY ANALYSIS

UTEXAS4 DATA INPUT FILE

```
HEADING
   Sta 12+200 - East Slope  - Effective Stress Analysis
   Analysis of Slopes In Atlantic Muck
PROFILE LINES
       1    1 Top of Random Fill
          -215.00     5.00
          -184.00     4.00
          -168.00     2.00

       2    2 Top of Atlantic Muck
          -215.00     2.00
          -168.00     2.00
           -84.00    -9.00

       3    3 Top of Gatun Overburden
          -215.00    -3.00
           -84.00    -9.00
           -76.00   -10.00

       4    4 Top of Tuff - Sandstone
          -215.00    -5.00
           -76.00   -10.00
           -55.00   -11.00


MATERIAL PROPERTIES
    1 Random Fill
        15.00 Unit Weight
        Conventional Shear
             .00    27.00
        Piezometric Line
        1
    2 Atlantic Muck
        14.00 Unit Weight
        Conventional Shear
             .00    18.00
        Piezometric Line
        1
    3 Gatun Overburden
        16.00 Unit Weight
        Conventional Shear
           21.00    31.00
        Piezometric Line
        1
    4 Tuff - Sandstone
```

```
      19.20 Unit Weight
      Conventional Shear
          4.70     31.00
      Piezometric Line
      1
PIEZOMETRIC LINES
     1       9.80 Piezo Line 1
        -215.00       3.00
        -190.00       3.00
          -76.00    -10.00
          -55.00    -11.00


ANALYSIS/COMPUTATION
   Noncircular Search
       -193.00       4.00
       -188.00       1.00
       -179.00      -2.00
       -170.00      -3.00
       -155.00      -4.50
       -136.00      -5.00
       -119.00      -6.00
       -102.00      -6.00

         1.00       .50     40.00
SINgle-stage Computations
SHOrt-form output
CRAck
          .00          D
SORt radii
CRItical
PROcedure for computation of Factor of Safety
SPENCER

GRAPH
COMPUTE
```

UTEXAS4 OUTPUT FILE

TABLE NO. 1
COMPUTER PROGRAM DESIGNATION: UTEXAS4
Originally Coded By Stephen G. Wright
Version No. 4.0.0.3 - Last Revision Date: 06/29/2000


```
****************************************************************
* RESULTS OF COMPUTATIONS PERFORMED USING THIS SOFTWARE       *
* SHOULD NOT BE USED FOR DESIGN PURPOSES UNLESS THEY HAVE     *
* BEEN VERIFIED BY INDEPENDENT ANALYSES, EXPERIMENTAL DATA    *
* OR FIELD EXPERIENCE. THE USER SHOULD UNDERSTAND THE ALGORITHMS *
* AND ANALYTICAL PROCEDURES USED IN THIS SOFTWARE AND MUST HAVE *
* READ ALL DOCUMENTATION FOR THIS SOFTWARE BEFORE ATTEMPTING  *
* TO USE IT. NEITHER SHINOAK SOFTWARE NOR STEPHEN G. WRIGHT   *
* MAKE OR ASSUME LIABILITY FOR ANY WARRANTIES, EXPRESSED OR   *
* IMPLIED, CONCERNING THE ACCURACY, RELIABILITY, USEFULNESS   *
* OR ADAPTABILITY OF THIS SOFTWARE.                           *
****************************************************************
```

UTEXAS4 - Version: 4.0.0.3 - Latest Revision: 06/29/2000
Licensed for use by: David E. Carlson, U. S. Army Corps of Engineers
Time and date of run: Fri Oct 04 10:16:42 2002
Name of input data file: C:\My Documents\Panama\Soils-Analysis\Slope-Analysis\Results\Muck\Muck12200-LT-1a.dat
Sta 12+200 - East Slope - Effective Stress Analysis
Analysis of Slopes In Atlantic Muck

TABLE NO. 3

```
****************************************************************
* NEW PROFILE LINE DATA *
****************************************************************
```

----- Profile Line No. 1 - Material Type (Number): 1 -----

Description: Top of Random Fill

Point	X	Y
1	-215.00	5.00
2	-184.00	4.00
3	-168.00	2.00

----- Profile Line No. 2 - Material Type (Number): 2 -----

Description: Top of Atlantic Muck

Point	X	Y
1	-215.00	2.00
2	-168.00	2.00
3	-84.00	-9.00

----- Profile Line No. 3 - Material Type (Number): 3 -----

Description: Top of Gatun Overburden

Point	X	Y
1	-215.00	-3.00
2	-84.00	-9.00
3	-76.00	-10.00

----- Profile Line No. 4 - Material Type (Number): 4 -----

Description: Top of Tuff- Sandstone

Point	X	Y
1	-215.00	-5.00
2	-76.00	-10.00
3	-55.00	-11.00

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Analysis of Slopes In Atlantic Muck

TABLE NO. 4

* NEW MATERIAL PROPERTY DATA - CONVENTIONAL/FIRST-STAGE COMPUTATIONS *

------------------ DATA FOR MATERIAL NUMBER 1 ------------------

Description: Random Fill
Unit weight of soil (material): 15.0
CONVENTIONAL (ISOTROPIC) SHEAR STRENGTHS
Cohesion - - - - - - - - 0.0
Friction angle - - - - - 27.00 (degrees)

Pore water pressures are defined by a piezometric line.
Piezometric line number: 1
Negative pore water pressures are NOT allowed - set to zero.

------------------ DATA FOR MATERIAL NUMBER 2 ------------------

Description: Atlantic Muck
Unit weight of soil (material): 14.0
CONVENTIONAL (ISOTROPIC) SHEAR STRENGTHS
Cohesion - - - - - - - - 0.0
Friction angle - - - - - 18.00 (degrees)

Pore water pressures are defined by a piezometric line.
Piezometric line number: 1
Negative pore water pressures are NOT allowed - set to zero.

------------------ DATA FOR MATERIAL NUMBER 3 ------------------

Description: Gatun Overburden
Unit weight of soil (material): 16.0
CONVENTIONAL (ISOTROPIC) SHEAR STRENGTHS
Cohesion - - - - - - - - 21.0
Friction angle - - - - - 31.00 (degrees)

Pore water pressures are defined by a piezometric line.
Piezometric line number: 1
Negative pore water pressures are NOT allowed - set to zero.

------------------ DATA FOR MATERIAL NUMBER 4 ------------------

Description: Tuff - Sandstone
Unit weight of soil (material): 19.2
CONVENTIONAL (ISOTROPIC) SHEAR STRENGTHS
Cohesion - - - - - - - - 4.7
Friction angle - - - - - 31.00 (degrees)

Pore water pressures are defined by a piezometric line.
Piezometric line number: 1
Negative pore water pressures are NOT allowed - set to zero.

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Analysis of Slopes In Atlantic Muck

TABLE NO. 6

* NEW PIEZOMETRIC LINE DATA - CONVENTIONAL/FIRST-STAGE COMPUTATIONS *

-------------- Piezometric Line Number 1 -------------

Description: Piezo Line 1
Unit weight of fluid (water): 9.8

Point	X	Y
1	-215.00	3.00
2	-190.00	3.00
3	-76.00	-10.00
4	-55.00	-11.00

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Sta 12+200 - East Slope - Effective Stress Analysis
Analysis of Slopes In Atlantic Muck

TABLE NO. 16

* NEW ANALYSIS/COMPUTATION DATA *

Coordinates of points on shear surface which are to be shifted

Point	X	Y	Shift Angle
1	-193.00	4.00	angle to be computed - moveable
2	-188.00	1.00	angle to be computed - moveable
3	-179.00	-2.00	angle to be computed - moveable
4	-170.00	-3.00	angle to be computed - moveable
5	-155.00	-4.50	angle to be computed - moveable
6	-136.00	-5.00	angle to be computed - moveable
7	-119.00	-6.00	angle to be computed - moveable
8	-102.00	-6.00	angle to be computed - moveable

Initial distance for shifting points on shear surface = 1.000
Final distance for shifting points on shear surface = 0.500
Maximum steepness permitted for toe of shear surface = 40.00
Conventional (single-stage) computations will be performed.
Automatic search output will be in short form.
Depth of crack: 0.000
Radii for each grid point will be sorted in the order of increasing radius.
Critical circles for grid points will be output in the order of increasing factor of safety.
Search will be continued after the initial mode to find a most critical circle.
Procedure of Analysis: Spencer

The following represent default values or values that were prevously defined:
Maximum increment for slice subdivision: 30
There is no water in a crack.
Seismic coefficient: 0.000
Unit weight of water (or other fluid) in crack: 62.4
Maximum number of passes for noncircular search: 50
No restrictions exist on the lateral extent of the search.
No shear surfaces other than the most critical will be saved for display later.
Neither slope face was explicitly designated for analysis.
Standard sign convention used for direction of shear stress on shear surface.
Iteration limit: 100
Force imbalance: 1.000000e-005 (fraction of total weight)
Moment imbalance: 1.000000e-005 (fraction of moment due to total weight) Minimum weight required for computations to be performed: 100 Initial trial factor of safety: 3.000 Initial trial side force inclination: 17.189 (degrees)

TABLE NO. 26

▪▪

* NEW, COMPUTED SLOPE GEOMETRY DATA *

▪▪

These slope geometry were generated from the Profile Lines.

Point	X	Y
1	-215.00	5.00
2	-184.00	4.00
3	-168.00	2.00
4	-84.00	-9.00
5	-76.00	-10.00
6	-55.00	-11.00

Left end point on noncircular shear surface adjusted to:
X: -193.51, Y: 4.31
Adjustment was made to put end point at bottom of crack.
Right end point on noncircular shear surface adjusted to:
X: -106.91, Y: -6.00
Adjustment was made to put end point on the slope.
Noncircular Shear Surface Points After End Point Adjustment
Coordinates of points on shear surface which are to be shifted

Point	X	Y	Shift Angle
1	-193.51	4.31	angle to be computed - moveable
2	-188.00	1.00	angle to be computed - moveable
3	-179.00	-2.00	angle to be computed - moveable
4	-170.00	-3.00	angle to be computed - moveable
5	-155.00	-4.50	angle to be computed - moveable
6	-136.00	-5.00	angle to be computed - moveable
7	-119.00	-6.00	angle to be computed - moveable
8	-106.91	-6.00	angle to be computed - moveable

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Analysis of Slopes In Atlantic Muck

TABLE NO. 40

* Short-Form Output Table for Search with Noncircular Shear Surfaces *

Shift Distance	Factor of Safety	Point	X	Y	Point	X	Y
1.000	1.465	1	-193.51	4.31	5	-155.00	-4.50
		2	-188.00	1.00	6	-136.00	-5.00
		3	-179.00	-2.00	7	-119.00	-6.00
		4	-170.00	-3.00	8	-106.91	-6.00
End of Trial: 1							
1.000	1.328	1	-192.51	4.27	5	-155.06	-5.50
		2	-187.84	1.34	6	-136.04	-6.00
		3	-179.02	-2.09	7	-119.03	-7.00
		4	-170.10	-3.99	8	-107.90	-5.87
End of Trial: 2							
0.500	1.328	1	-192.51	4.27	5	-155.06	-5.50
		2	-187.84	1.34	6	-136.04	-6.00
		3	-179.02	-2.09	7	-119.03	-7.00
		4	-170.10	-3.99	8	-107.90	-5.87
End of Trial: 3							
0.500	1.313	1	-192.01	4.26	5	-155.05	-5.26
		2	-187.75	1.52	6	-136.06	-6.50
		3	-179.08	-2.31	7	-119.03	-6.79
		4	-170.18	-4.49	8	-107.40	-5.94
End of Trial: 4							
0.500	1.313	1	-191.51	4.24	5	-155.04	-5.07
		2	-187.74	1.55	6	-136.05	-6.26
		3	-179.14	-2.46	7	-119.02	-7.29
		4	-170.26	-4.98	8	-106.91	-6.00
End of Trial: 5							
0.500	1.308	1	-191.01	4.23	5	-155.05	-5.57
		2	-187.71	1.60	6	-136.04	-6.02
		3	-179.31	-2.93	7	-119.03	-7.05
		4	-170.22	-4.74	8	-106.41	-6.06
End of Trial: 6							
0.500	1.302	1	-190.51	4.21	5	-155.04	-5.33
		2	-187.73	1.56	6	-136.06	-6.52
		3	-179.14	-2.46	7	-119.03	-6.83
		4	-170.19	-4.49	8	-105.92	-6.13
End of Trial: 7							
0.500	1.302	1	-190.01	4.19	5	-155.03	-5.09
		2	-187.44	1.97	6	-136.05	-6.28
		3	-179.13	-2.43	7	-119.02	-7.33
		4	-170.26	-4.99	8	-105.42	-6.19
End of Trial: 8							
0.500	1.299	1	-189.51	4.18	5	-155.05	-5.59
		2	-187.43	1.99	6	-136.04	-6.05
		3	-179.16	-2.51	7	-119.02	-7.09
		4	-170.19	-4.50	8	-104.93	-6.26
End of Trial: 9							
0.500	1.295	1	-189.01	4.16	5	-155.04	-5.35
		2	-187.35	2.09	6	-136.06	-6.55
		3	-178.98	-2.04	7	-119.02	-6.86
		4	-170.26	-4.99	8	-104.43	-6.32
End of Trial: 10							

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Sta 12+200 - East Slope - Effective Stress Analysis Analysis of Slopes In Atlantic Muck

TABLE NO. 41

* Critical Noncircular Shear Surface *

***** CRITICAL NONCIRCULAR SHEAR SURFACE *****

X:	-189.01	Y:	4.16
X:	-187.35	Y:	2.09
X:	-178.98	Y:	-2.04
X:	-170.26	Y:	-4.99
X:	-155.04	Y:	-5.35
X:	-136.06	Y:	-6.55
X:	-119.02	Y:	-6.86
X:	-104.43	Y:	-6.32

Minimum factor of safety: 1.295
Side force inclination: -6.59
Time required to find most critical surface: 2.0 seconds
Number of passes required to find most critical surface: 10
Total number of shear surf aces attempted: 170
Total number of shear surfaces for which factor of safety was successfully calculated: 170

Pass	Shift Distance	Pt.	Max. Dist. Moved	Minimum F	n Tried	n Computed
1	1.0000	7	1.000	1.3278	17	17
2	1.0000	4	1.000	1.3278	34	34
3	0.5000	4	0.500	1.3134	51	51
4	0.5000	4	0.500	1.3132	68	68
5	0.5000	3	0.500	1.3082	85	85
6	0.5000	6	0.500	1.3024	102	102
7	0.5000	7	0.500	1.3023	119	119
8	0.5000	5	0.500	1.2988	136	136
9	0.5000	4	0.500	1.2950	153	153
10	0.5000	2	0.500	1.2950	170	170

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Name of input data file: C:\My Documents\Panama\Soils-Analysis\Slope-Analysis\Results\Muck\Muck12200-LT-1a.dat
Sta 12+200 - East Slope - Effective Stress Analysis
Analysis of Slopes In Atlantic Muck

TABLE NO. 43

* Coordinate, Weight, Strength and Pore Water Pressure *
* Information for Individual Slices for Conventional *
* Computations or First Stage of Multi-Stage Computations. * * (Information is for the critical shear surface in the *
* case of an automatic search.) *

Slice No.	X	Y	Slice Weight	Matl. No.	Cohesion	Friction Angle	Pore Pressure
	-189.01	4.16					
1	-188.45	3.46	12	1	0.0	27.00	0.0
	-187.88	2.76					
2	-187.62	2.42	14	1	0.0	27.00	3.0
	-187.35	2.09					
3	-187.26	2.04	6	1	0.0	27.00	6.3
	-187.16	2.00					
4	-186.37	1.61	58	2	0.0	18.00	9.6
	-185.58	1.22					
5	-184.79	0.83	74	2	0.0	18.00	15.5
	-184.00	0.44					
6	-182.75	-0.18	146	2	0.0	18.00	23.1
	-181.49	-0.80					
7	-180.24	-1.42	178	2	0.0	18.00	32.4
	-178.98	-2.04					
8	-177.89	-2.41	175	2	0.0	18.00	39.5
	-176.80	-2.78					
9	-175.71	-3.15	189	2	0.0	18.00	44.3
	-174.62	-3.52					
10	-173.53	-3.88	202	2	0.0	18.00	49.1
	-172.44	-4.25					
11	-171.35	-4.62	216	2	0.0	18.00	53.8
	-170.26	-4.99					
12	-169.13	-5.02	227	2	0.0	18.00	55.2
	-168.00	-5.04					
13	-166.70	-5.08	251	2	0.0	18.00	53.1
	-165.41	-5.11					
14	-164.11	-5.14	241	2	0.0	18.00	50.8
	-162.81	-5.17					
15	-161.52	-5.20	231	2	0.0	18.00	48.5
	-160.22	-5.23					
16	-158.93	-5.26	220	2	0.0	18.00	46.2
	-157.63	-5.29					
17	-156.33	-5.32	210	2	0.0	18.00	43.9
	-155.04	-5.35					
18	-153.68	-5.44	211	2	0.0	18.00	42.1
	-152.33	-5.52					
19	-150.97	-5.61	204	2	0.0	18.00	40.8
	-149.61	-5.70					
20	-148.26	-5.78	197	2	0.0	18.00	39.4
	-146.90	-5.87					
21	-145.55	-5.95	190	2	0.0	18.00	38.0
	-144.19	-6.04					
22	-142.84	-6.12	183	2	0.0	18.00	36.7
	-141.48	-6.21					
23	-140.12	-6.29	176	2	0.0	18.00	35.3
	-138.77	-6.38					
24	-137.41	-6.46	169	2	0.0	18.00	34.0
	-136.06	-6.55					
25	-134.64	-6.57	167	2	0.0	18.00	31.9
	-133.22	-6.60					
26	-131.80	-6.63	154	2	0.0	18.00	29.3
	-130.38	-6.65					
27	-128.96	-6.68	142	2	0.0	18.00	26.6
	-127.54	-6.71					
28	-126.12	-6.73	129	2	0.0	18.00	24.0
	-124.70	-6.76					
29	-123.28	-6.78	116	2	0.0	18.00	21.3
	-121.86	-6.81					

30	-120.44	-6.84	104	2	0.0	18.00	18.7
	-119.02	-6.86					
31	-117.85	-6.82	74	2	0.0	18.00	15.6
	-116.68	-6.78					
32	-115.50	-6.73	61	2	0.0	18.00	12.1
	-114.33	-6.69					
33	-113.16	-6.65	48	2	0.0	18.00	8.7
	-111.99	-6.60					
34	-110.82	-6.56	35	2	0.0	18.00	5.2
	-109.64	-6.52					
35	-108.47	-6.47	22	2	0.0	18.00	1.7
	-107.30	-6.43					
36	-106.58	-6.40	7	2	0.0	18.00	0.0
	-105.87	-6.38					
37	-105.15	-6.35	2	2	0.0	18.00	0.0
	-104.43	-6.32					

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Time and date of run: Fri Oct 04 10:16:42 2002
Name of input data file: C:\My Documents\Panama\Soils-Analysis\Slope-Analysis\Results\Muck\Muck12200-LT-1a.dat
Sta 12+200 - East Slope - Effective Stress Analysis
Analysis of Slopes In Atlantic Muck

TABLE NO. 44

****

* Seismic Forces and Forces Due to Distributed Loads for *
* Individual Slices for Conventional Computations or the *
* First Stage of Multi-Stage Computations. *
* (Information is for the critical shear surface in the *
* case of an automatic search.) *

****

There are no seismic forces or forces due to distributed loads
for the current shear surface
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Name of input data file: C:\My Documents\Panama\Soils-Analysis\Slope-Analysis\Results\Muck\Muck12200-LT-1a.dat
Sta 12+200 - East Slope - Effective Stress Analysis
Analysis of Slopes In Atlantic Muck

TABLE NO. 47

****

* Information for the Iterative Solution for the Factor of *
* Safety and Side Force Inclination by Spencer's Procedure *

****

Allowable force imbalance for convergence: 5.0421e-002
Allowable moment imbalance for convergence: 8

```
     Trial   Trial
    Factor  Side Force   Force    Moment            Delta
Iter-  of   Inclination Imbalance Imbalance  Delta-F   Theta
ation Safety (degrees)   (lbs.)   (ft.-lbs.)       (degrees)

  1  3.00000  -17.1887 -3.410e+002 -1.924e+004
First-order corrections to F and Theta ............ -3.7821    2.0625
Reduced values - Deltas were too large ............ -0.5000    0.2727

  2  2.50000  -16.9161 -2.867e+002 -1.627e+004
First-order corrections to F and Theta ............ -2.1989    2.3042
Reduced values - Deltas were too large ............ -0.5000    0.5239

  3  2.00000  -16.3921 -2.048e+002 -1.186e+004
First-order corrections to F and Theta ............ -1.0056    2.8033
Reduced values - Deltas were too large ............ -0.5000    1.3939

  4  1.50000  -14.9982 -6.830e+001 -4.783e+003
First-order corrections to F and Theta ............ -0.2080    4.5345
Reduced values - Deltas were too large ............ -0.1314    2.8648

  5  1.36861  -12.1335 -2.122e+001 -1.934e+003
First-order corrections to F and Theta ............ -0.0703    4.3393
Reduced values - Deltas were too large ............ -0.0464    2.8648

  6  1.32219   -9.2687 -6.715e+000 -7.501e+002
First-order corrections to F and Theta ............ -0.0265    2.4556
Second-order corrections to F and Theta ........... -0.0272    2.6713

  7  1.29499   -6.5974 5.446e-003 -1.092e+000
First-order corrections to F and Theta ............ -0.0000    0.0059
Second-order corrections to F and Theta ........... -0.0000    0.0059

  8  1.29497   -6.5915 -8.132e-011 -1.286e-009
First-order corrections to F and Theta ............ -0.0000   -0.0000
```

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Sta 12+200 - East Slope - Effective Stress Analysis
Analysis of Slopes In Atlantic Muck

TABLE NO. 55

* Check of Computations by Spencer's Procedure (Results are for the *
* critical shear surface in the case of an automatic search.) *

Summation of Horizontal Forces: 1.90736e-013
Summation of Vertical Forces: 3.57936e-013
Summation of Moments: 1.44382e-011
Mohr Coulomb Shear Force/Shear Strength Check Summation: 1.47438e-013
UTEXAS4 - Version: 4.0.0.3 - Latest Revision: 06/29/2000
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Time and date of run: Fri Oct 04 10:16:42 2002
Name of input data file: C:\My Documents\Panama\Soils-Analysis\Slope-Analysis\Results\Muck\Muck12200-LT-1a.dat
Sta 12+200 - East Slope - Effective Stress Analysis
Analysis of Slopes In Atlantic Muck

TABLE NO. 58
* Final Results for Stresses Along the Shear Surface * * (Results are for the critical shear surface in the case of a search.) *

SPENCER'S PROCEDURE USED TO COMPUTE THE FACTOR OF SAFETY
Factor of Safety: 1.295 Side Force Inclination: -6.59
------- VALUES AT CENTER OF BASE OF SLICE --------

Slice No.	X-Center	Y-Center	Total Normal Stress	Effective Normal Stress	Shear Stress
1	-188.45	3.46	6.5	6.5	2.5
2	-187.62	2.42	16.8	13.9	5.5
3	-187.26	2.04	26.4	20.1	7.9
4	-186.37	1.61	32.6	23.0	5.8
5	-184.79	0.83	41.9	26.4	6.6
6	-182.75	-0.18	52.4	29.3	7.4
7	-180.24	-1.42	64.2	31.8	8.0
8	-177.89	-2.41	75.4	35.9	9.0
9	-175.71	-3.15	81.3	37.0	9.3
10	-173.53	-3.88	87.2	38.2	9.6
11	-171.35	-4.62	93.2	39.3	9.9
12	-169.13	-5.02	101.1	45.9	11.5
13	-166.70	-5.08	97.4	44.3	11.1
14	-164.11	-5.14	93.5	42.7	10.7
15	-161.52	-5.20	89.6	41.1	10.3
16	-158.93	-5.26	85.7	39.5	9.9
17	-156.33	-5.32	81.8	37.8	9.5
18	-153.68	-5.44	77.8	35.7	9.0
19	-150.97	-5.61	75.2	34.5	8.6
20	-148.26	-5.78	72.6	33.2	8.3
21	-145.55	-5.95	70.1	32.0	8.0
22	-142.84	-6.12	67.5	30.8	7.7
23	-140.12	-6.29	64.9	29.6	7.4
24	-137.41	-6.46	62.3	28.4	7.1
25	-134.64	-6.57	59.4	27.5	6.9
26	-131.80	-6.63	54.9	25.6	6.4
27	-128.96	-6.68	50.4	23.8	6.0
28	-126.12	-6.73	45.9	21.9	5.5
29	-123.28	-6.78	41.4	20.1	5.0
30	-120.44	-6.84	36.9	18.2	4.6
31	-117.85	-6.82	32.3	16.7	4.2
32	-115.50	-6.73	26.7	14.6	3.7
33	-113.16	-6.65	21.1	12.4	3.1
34	-110.82	-6.56	15.5	10.3	2.6
35	-108.47	-6.47	9.9	8.1	2.0
36	-106.58	-6.40	5.3	5.3	1.3
37	-105.15	-6.35	1.8	1.8	0.4

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Sta 12+200 - East Slope - Effective Stress Analysis
Analysis of Slopes In Atlantic Muck

TABLE NO. 59
* Final Results for Side Forces and Stresses Between Slices * * (Results are for the critical shear surface in the case of a search.) *

Slice No.	X-Right	Side Force	Y-Coord. of Side Force Location	Fraction of Height	Sigma at Top	Sigma at Bottom
	VALUES AT RIGHT SIDE OF SLICE					
1	-187.88	6	3.39	0.466	3.6	5.5
2	-187.35	15	2.79	0.349	0.7	13.7
3	-187.16	16	2.73	0.346	0.6	14.1
4	-185.58	32	2.02	0.282	-3.5	25.9
5	-184.00	54	1.38	0.265	-6.2	36.5
6	-181.49	101	0.43	0.276	-7.7	52.5
7	-178.98	161	-0.49	0.286	-8.3	67.5
8	-176.80	197	-1.07	0.290	-8.6	75.3
9	-174.62	237	-1.65	0.294	-8.8	83.1
10	-172.44	281	-2.23	0.297	-9.0	90.9
11	-170.26	328	-2.81	0.300	-9.1	98.8
12	-168.00	308	-2.93	0.300	-8.7	95.5
13	-165.41	285	-3.07	0.301	-8.2	91.8
14	-162.81	263	-3.21	0.302	-7.6	88.0
15	-160.22	241	-3.35	0.303	-7.0	84.2
16	-157.63	221	-3.48	0.305	-6.3	80.3
17	-155.04	201	-3.62	0.307	-5.6	76.2
18	-152.33	190	-3.83	0.309	-5.0	74.0
19	-149.61	179	-4.05	0.311	-4.5	71.9
20	-146.90	169	-4.27	0.313	-4.0	69.8
21	-144.19	159	-4.49	0.315	-3.5	67.8
22	-141.48	150	-4.70	0.317	-3.0	65.9
23	-138.77	141	-4.93	0.319	-2.6	64.0
24	-136.06	132	-5.15	0.321	-2.3	62.3
25	-133.22	115	-5.29	0.323	-1.8	58.4
26	-130.38	100	-5.44	0.325	-1.3	54.6
27	-127.54	85	-5.59	0.328	-0.9	50.7
28	-124.70	72	-5.74	0.331	-0.4	46.8
29	-121.86	60	-5.89	0.334	0.1	43.0
30	-119.02	49	-6.04	0.338	0.6	39.1
31	-116.68	36	-6.08	0.340	0.7	34.2
32	-114.33	25	-6.12	0.341	0.7	29.4
33	-111.99	16	-6.17	0.341	0.6	24.5
34	-109.64	9	-6.22	0.338	0.3	19.3
35	-107.30	3	-6.27	0.341	0.3	11.8
36	-105.87	1	-6.27	0.454	2.2	3.9
37	-104.43	-0	-6.32	1.000	0.0	0.0

Read end-of-file on input while looking for another command word.
End of input data assumed - normal termination.

Appendix C – Geotechnical Investigations, Analyses and Designs

Part II – Geology

Tables

Table C-II-GG-1	Summary of 2001 Borings
Table C-II-GG-2	Summary of 1930's Testing
Table C-II-GG-3	Summary of 2001 Testing General Data
Table C-II-GG-4	Summary of 2001 Testing - Table of Results
Table C-II-GG-5	Summary of 2001 Testing- Summary of Sandstone Testing
Table C-II-GG-6	Summary of 2001 Testing- Summary of Conglomerate Testing
Table C-II-GG-7	Summary of Rock Permeability Testing
Table C-II-GG-8	Rock Shear Strength Testing
Table C-II-GG-9	Surfer Metadata

Table C-II-GG-1
Summary of Boring Information for 2001 Program

Boring Number	Northing	Easting	Top of Boring	Top of Sound Rock	Bottom of Boring	Completion Date
TA1-01	1,028,285.9	619,036.3	2.91	-3.5	-37.47	6/4/2001
TA1-02	1,026,949.8	619,258.4	4.06	-11.3	-30.49	5/5/2001
TA1-03	1,025,857.4	619,179.6	10.34	4.2	-29.96	5/10/2001
TA1-04	1,024,869.0	619,563.0	29.13	14.6	-22.27	5/14/2001
TA1-05	1,023,898.8	619,659.5	30.66	28.3	-19.95	5/17/2001
TA1C-01	1,027,710.0	618,865.0	13.71	10.3	-36.29	10/18/2001
TA1C-02	1,027,395.0	619,195.0	1.135	-10.8	-33.865	10/13/2001
TA1C-03	1,026,920.0	619,075.0	3.158		-31.842	10/14/2001
TA1C-04	1,026,505.0	619,410.0	4.89	-14	-35.11	10/17/2001
TA1C-05*	1,026,062.0	619,407.0	1.875	-8.8	-38.125	10/20/2001
TA1C-06	1,025,615.0	619,460.0	13.216	3.4	-36.784	10/18/2001
TA1C-07*	1,025,585.0	619,260.0	13.427	10.1	-26.573	10/23/2001
TA1C-08	1,025,305.0	619,430.0	18.896	18.9	-31.104	10/22/2001
TA1C-09*	1,024,840.0	619,355.0	32.06	19.8	-7.94	10/20/2001
TA1C-10	1,024,415.0	619,565.0	27.643	21.6	-27.907	10/22/2001
TA1C-11	1,024,145.0	619,750.0	63.714	62	-16.286	10/19/2001
TA2-01	1,026,794.2	618,642.5	1.34	-21.6	-28.75	5/21/2001
TA2-02	1,026,573.7	618,833.5	2.52	-25	-29.48	6/6/2001
TA2-03	1,026,389.5	618,614.3	0.868	-19.6	-29.862	4/27/2001
TA2-04	1,025,984.5	618,649.9	4.72	-4.6	-46.5	4/30/2001
TA2-05	1,025,468.0	618,697.3	4.4	-5.8	-36.02	5/4/2001
TA2-06	1,024,953.0	618,436.0	14.9	2.6	-35.83	6/13/2001
TA2-07	1,024,680.7	618,487.2	28.57	25.1	-22.43	6/1/2001
TA2-08	1,024,400.0	618,430.0	16.7	9.4	-23.6	7/4/2001
TA2-09	1,024,190.0	618,450.0	7.6	-20.1	-34.8	7/8/2001
TA2-10	1,024,050.0	618,385.0	6.5		-37.4	7/7/2001

Boring Number	Northing	Easting	Top of Boring	Top of Sound Rock	Bottom of Boring	Completion Date
TA2C-01	1,026,740.0	618,920.0	3.341	-19.4	-26.859	10/16/2001
TA2C-02	1,026,355.0	618,870.0	2.056	-9.2	-28.444	10/12/2001
TA2C-03*	1,026,170.0	618,715.0	3.55	-3.8	-36.45	10/15/2001
TA2C-04	1,025,950.0	618,820.0	4.83	-6.1	-35.17	9/8/2001
TA2C-05*	1,025,780.0	618,670.0	1.253	-18	-48.947	10/12/2001
TA2C-06	1,025,795.0	618,550.0	2.468	-13.4	-37.732	10/5/2001
TA2C-07	1,025,595.0	618,794.0	5.093	-9.7	-24.907	10/6/2001
TA2C-08	1,025,590.0	618,595.0	2.75	-0.3	-37.25	10/4/2001
TA2C-09*	1,025,405.0	618,509.0	4.045	-8.7	-25.955	10/10/2001
TA2C-10	1,025,220.0	618,745.0	9.984	-1.9	-20.216	10/5/2001
TA2C-11*	1,025,237.0	618,664.0	14.055	3.2	-36.195	10/9/2001
TA2C-12	1,025,135.0	618,525.0	6.084	-6.8	-44.016	10/7/2001
TA2C-13*	1,024,925.0	618,620.0	22.355	18.5	-32.745	10/11/2001
TA2C-14	1,024,665.0	618,886.0	43.048	36.1	12.6	10/13/2001
TA2C-15*	1,024,600.0	618,400.0	28.784	12.9	-11.216	10/15/2001
TA2C-16	1,024,485.0	618,555.0	34.978	11.9	-15.322	10/16/2001
TA2C-17	1,024,445.0	618,675.0	28.482	17.2	-1.718	10/19/2001

Table C-II-GG-1

ENGINEERING PROPERTIES — CANAL ZONE ROCK UNITS

FORMATION	DESCRIPTION	GEOLOGICAL CLASSIFICATION OF RELATIVE HARDNESS, Average	RANGE IN UNIT HARDNESS	UNIT WEIGHT, pounds per cu. ft.	SHEAR STRENGTH Ø, degrees	SHEAR STRENGTH c, pounds per sq. in.	COMPRESSIVE STRENGTH,* pounds per sq. in.	ASSIGNED ALLOWABLE BEARING CAPACITY, tons per sq. ft.
OVERBURDEN**	Silts, sands, clays, gravels, etc.	Soft to very hard overburden	OH-1 to OH-5	Variable	Variable		Variable	Variable
ATLANTIC MUCK	Poorly consolidated clays and silts	Soft to medium soft overburden	OH-1 to OH-2	*90±	*17	*0	0.3-18.1	Very low
PACIFIC MUCK	Poorly consolidated clays and silts	Soft to medium soft overburden	OH-1 to OH-2	90±	17	0		Very low
CUCARACHA	Largely dense greenish-gray clay shales highly slickensided within certain horizons. Black carbonaceous shales, sandstones, and conglomerates in subordinate proportions	Soft rock	RH-1 to RH-3	*135±	*10	*16†	1.3-356	***6-15
CULEBRA	Medium-hard sandstones, soft sandy and carbonaceous shales	Soft rock	RH-1 to RH-3	*135±	*11	*168	69-4020	15
LA BOCA	Dense silty or sandy dark gray shales with intercalated sandstone beds sporadically present	Soft rock	RH-1 to RH-3	*140±			75-3940	15
PANAMA TUFF	Coarse- to fine-grained acid tuffs, sandstones, and soft clay shales	Soft rock	RH-1 to RH-3	140±	Medium high strength			15
LAS CASCADAS	Agglomeratic tuff and tuff-breccia consisting of angular fragments of hard dark gray andesite in a clayey dark gray to light green altered tuff matrix	Medium hard rock	RH-2	140±			34-5922	15+
CAIMITO	Coarsely bedded medium- and fine-grained medium-hard limy sandstones and tuffs	Medium hard rock	RH-2 to RH-3	*135±			113-8470	20
GATUN	Fine-grained argillaceous and calcareous sandstones with interbedded dense tuffs and conglomerates	Medium hard rock	RH-2	*120±	*17	*220	470-940	20
CHAGRES SANDSTONE	Generally fine-grained medium-hard massively and tightly bedded dense gray friable sandstone	Medium hard rock	RH-2 to RH-3	120-125	High strength			20
TORO LIMESTONE	Slightly cemented shell- and coral-fragment limestone with minor intercalated lenses of medium- and coarse-grained sandstone	Medium hard rock	RH-2	130±	High strength			20
BOHIO CONGLOMERATE	Subangular to rounded pebbles, cobbles, and boulders up to two feet in diameter in a dark gray or brown, generally coarse, friable tuffaceous sand matrix	Hard rock	RH-3	*150±			334-2185	20+
EMPERADOR LIMESTONE	Algal and coralline reef limestone of local extent and spotty distribution	Hard rock	RH-3	170±	Very high strength			30+
PEDRO MIGUEL AGGLOMERATE	Hard light to dark gray fine- to coarse-textured agglomerates and tuffs	Hard rock	RH-3 to RH-4	*155±			4240-6970	50+
BAS OBISPO	Largely hard subangular to angular fragments of andesite and basalt in a sandy hard matrix of the same composition	Hard rock	RH-3 to RH-4	*155±			2248-6956	50+
BASALT	Hard columnar-jointed basalt flows and intrusives	Very hard rock	RH-4	*165±			2710-26150	50+
MISCELLANEOUS RHYOLITE AND ANDESITE	Generally very hard moderately jointed intrusions (dikes, etc.) and flows	Very hard rock	RH-4	160-170	Very high strength			50+

* Data determined by laboratory tests. Similar data for other rock units are estimates only, based upon lithologic similarity of the respective formations to those for which test data are available or upon local experience and field investigations.

** Includes weathered rock.

*** Data determined by field bearing test.

GEOLOGICAL HARDNESS SCALE

Overburden and Weathered Rock

Code	Term	Description
OH-1	Soft	Easily squeezed through the fingers. Consistency of fresh putty. (Muck, some clays.)
OH-2	Medium Soft	Cannot be squeezed readily through the fingers, but is easily indented with finger point at moderate pressure.
OH-3	Medium Hard	Nonpenetrable at moderate finger pressure. A pencil point can be readily pushed into sample.
OH-4	Hard	Difficult to take drive sample. Difficult to push pencil point into sample.
OH-5	Very Hard	Material of near-rock character.

Sound Rock

Code	Term	Description
RH-1	Soft	Harder than OH-5 and cannot be crumbled between the fingers but can be easily picked with geology hammer. (Some shales and uncemented sandstones.)
RH-2	Medium Hard	Can be picked with moderate blows of geology hammer. Can be cut with knife.
RH-3	Hard	Cannot be picked with geology hammer but can be chipped with moderate blows of hammer.
RH-4	Very Hard	Chips can be broken off with heavy blows of geology hammer.

Table GG-2

Table GG-3
Summary of 2001 Testing General Data

Boring	Sample No.	Depth Top (m)	Depth Bottom (m)	Elevation Top (m)	Elevation Bottom (m)	Unit Weight Kglm3	Elastic Modulus E (Mpa)	Unconf. Comp. c (Mpa)	Specific Gravity	Water cont. (%)	Rock Type	Formation	Northing	Easting	TOH
TA1C-01	4-1	14.11	14.40	4.79	4.50	1890	537	7	2.3	13.77	Conglomerate	Gatun	1,025,305.0	619,430.0	18.90
TA1C-01	4-2	14.11	14.40	4.79	4.50	1903	828	18	2.3	12.93	Conglomerate	Gatun	1,025,305.0	619,430.0	18.90
TA1C-01	5-1	15.23	15.54	-10.40	-10.71	1899	409	7	2.3	36.43	Conglomerate	Gatun	1,025,950.0	618,820.0	4.83
TA1C-01	5-2	15.23	15.54	-10.40	-10.71	1874	552	6	2.3	31.98	Conglomerate	Gatun	1,025,950.0	618,820.0	4.83
TA1C-02	3-1	19.74	19.87	-17.27	-17.40	2396	3241	4	2.3	33.95	Conglomerate	Gatun	1,025,795.0	618,550.0	2.47
TA1C-02	6-1	34.64	34.85	-31.89	-32.10	1990	1583	3	2.3	16.36	Conglomerate	Gatun	1,025,590.0	618,595.0	2.75
TA1C-04	1-1	20.68	21.00	-16.63	-16.95	2049	774	5	2.3	29.31	Conglomerate	Gatun	1,025,405.0	618,509.0	4.05
TA1C-04	1-2	20.68	21.00	-14.60	-14.92	1978	784	9	2.2	11.25	Conglomerate	Gatun	1,025,135.0	618,525.0	6.08
TA1C-05	8-1	47.20	47.45	-41.12	-41.37	1677	1533	10	2.2	20.40	Conglomerate	Gatun	1,025,135.0	618,525.0	6.08
TA1C-05	8-2	47.20	47.45	-42.37	-42.62	1678	1724	5	2.2	22.72	Microconglomerate	Gatun	1,025,950.0	618,820.0	4.83
TA1C-06	8-1	43.18	43.40	-38.09	-38.31	1787	2638	2	2.2	24.13	Microconglomerate	Gatun	1,025,595.0	618,794.0	5.09
TA1C-07	8-1	28.47	28.70	-18.49	-18.72	1905	517	3	2.2	42.94	Microconglomerate	Gatun	1,025,220.0	618,745.0	9.98
TA1C-07	9-1	34.10	34.36	-20.39	-20.65	1745	1011	5	2.0	30.76	Sandstone	Gatun	1,027,710.0	618,865.0	13.71
TA1C-07	9-2	34.10	34.36	-20.39	-20.65	1746	876	10	2.0	29.11	Sandstone	Gatun	1,027,710.0	618,865.0	13.71
TA1C-08	6-1	25.81	26.09	-12.10	-12.38	1811	580	4	2.0	32.52	Sandstone	Gatun	1,027,710.0	618,865.0	13.71
TA1C-08	6-2	25.81	26.09	-12.10	-12.38	1784	724	5	2.0	32.52	Sandstone	Gatun	1,027,710.0	618,865.0	13.71
TA1C-08	10-1	40.57	40.90	-39.43	-39.76	2043	1164	25	2.4	9.91	Sandstone	Gatun	1,027,395.0	619,195.0	1.14
TA1C0-8	10-2	40.57	40.90	-39.43	-39.76	2184	4127	11	2.4	33.17	Sandstone	Gatun	1,027,395.0	619,195.0	1.14
TA1C-09	3-1	20.27	20.50	-15.38	-15.61	1621	907	5	2.0	24.01	Sandstone	Gatun	1,026,505.0	619,410.0	4.89
TA1C-09	4-1	23.02	23.28	-18.13	-18.39	1641	2483	5	2.0	26.46	Sandstone	Gatun	1,026,505.0	619,410.0	4.89
TA1C-09	4-2	23.02	23.28	-21.14	-21.40	1713	3065	10	2.0	44.36	Sandstone	Gatun	1,026,062.0	619,407.0	1.88
TA1C-10	7-1	26.04	26.24	-24.16	-24.36	1668	595	11	2.0	36.36	Sandstone	Gatun	1,026,062.0	619,407.0	1.88
TA1C-10	9-1	32.91	33.15	-19.69	-19.93	1926	711	15	2.0	34.37	Sandstone	Gatun	1,025,615.0	619,460.0	13.22
TA1C-11	7-1	28.20	28.44	-14.77	-15.01	1600	852	3	2.1	27.97	Sandstone	Gatun	1,025,585.0	619,260.0	13.43
TA1C-11	17-1	74.46	74.65	-61.03	-61.22	2118	567	5	2.1	34.14	Sandstone	Gatun	1,025,585.0	619,260.0	13.43
TA2C-01	2-1	20.83	30.14	-16.40	-16.71	1932	1705	5	2.1	40.98	Sandstone	Gatun	1,025,585.0	619,260.0	13.43
TA2C-01	2-2	29.83	30.14	-10.93	-11.24	1916	2355	4	2.0	31.50	Sandstone	Gatun	1,025,305.0	619,430.0	18.90
TA2C-02	5-1	29.71	29.98	-10.81	-11.08	1894	401	5	2.0	39.89	Sandstone	Gatun	1,025,305.0	619,430.0	18.90
TA2C-02	5-2	29.71	29.98	2.35	2.08	1966	1314	6	2.0	54.13	Sandstone	Gatun	1,024,840.0	619,355.0	32.06
TA2C-03	5-1	34.24	34.53	-6.60	-6.89	1817	544	5	2.0	54.02	Sandstone	Gatun	1,024,415.0	619,656.0	27.64
TA2C-03	5-2	34.24	34.53	-6.60	-6.89	1929	743	2	2.0	29.96	Sandstone	Gatun	1,024,415.0	619,656.0	27.64
TA2C-03	6-1	38.97	39.35	24.74	24.36	1901	840	6	2.0	55.51	Sandstone	Gatun	1,024,145.0	619,750.0	63.71
TA2C-03	6-2	38.97	39.35	24.74	24.36	1898	1008	3	2.0	17.34	Sandstone	Gatun	1,024,145.0	619,750.0	63.71
TA2C-04	4-1	25.31	25.55	-21.97	-22.21	1966	2299	9	2.0	30.05	Sandstone	Gatun	1,026,740.0	618,920.0	3.34
TA2C-04	5-1	29.26	29.45	-25.92	-26.11	2070	726	9	2.0	35.10	Sandstone	Gatun	1,026,740.0	618,920.0	3.34
TA2C-04	6-1	32.86	33.11	-30.80	-31.05	1833	2483	3	2.0	28.10	Sandstone	Gatun	1,026,355.0	618,870.0	2.06
TA2C-04	6-2	32.86	33.11	-30.80	-31.05	1888	1910	4	2.0	26.02	Sandstone	Gatun	1,026,355.0	618,870.0	2.06
TA2C-05	4-1	31.24	34.51	-27.69	-30.96	1794	1552	3	2.0	32.53	Sandstone	Gatun	1,026,170.0	618,715.0	3.55
TA2C-05	4-2	31.24	34.51	-27.69	-30.96	1832	242	5	2.0	28.05	Sandstone	Gatun	1,026,170.0	618,715.0	3.55
TA2C-05	8-1	49.88	50.20	-46.33	-46.65	1679	1452	5	2.0	30.14	Sandstone	Gatun	1,026,170.0	618,715.0	3.55
TA2C-05	8-2	49.88	50.20	-46.33	-46.65	1703	2420	5	2.0	29.06	Sandstone	Gatun	1,026,170.0	618,715.0	3.55
TA2C-06	5-1	35.53	35.76	-30.70	-30.93	1842	372	5	2.1	34.70	Sandstone	Gatun	1,025,950.0	618,820.0	4.83
TA2C-07	3-1	22.54	22.79	-21.29	-21.54	2033	167	3	2.0	34.70	Sandstone	Gatun	1,025,780.0	618,670.0	1.25
TA2C-08	3-1	13.25	13.51	-12.00	-12.26	1924	2793	2	2.0	32.72	Sandstone	Gatun	1,025,780.0	618,670.0	1.25
TA2C-08	3-2	13.25	13.51	-12.00	-12.26	1897	1264	8	2.0	47.69	Sandstone	Gatun	1,025,780.0	618,670.0	1.25
TA2C-08	4-1	15.88	16.18	-14.63	-14.93	1826	1586	9	2.0	45.92	Sandstone	Gatun	1,025,780.0	618,670.0	1.25
TA2C-08	4-2	15.88	16.18	-13.13	-13.43	1807	1506	13	2.0	30.92	Sandstone	Gatun	1,025,590.0	618,595.0	2.75
TA2C-08	6-1	26.80	27.07	-24.05	-24.32	2071	616	7	2.0	38.19	Sandstone	Gatun	1,025,590.0	618,595.0	2.75
TA2C-09	4-1	26.99	27.24	-24.24	-24.49	1864	956	8	2.0	41.25	Sandstone	Gatun	1,025,590.0	618,595.0	2.75
TA2C-10	4-1	22.90	23.14	-20.15	-20.39	1733	552	8	2.0	39.13	Sandstone	Gatun	1,025,590.0	618,595.0	2.75
TA2C-11	2-1	16.66	17.10	-2.60	-3.04	1905	759	5	2.0	30.36	Sandstone	Gatun	1,025,237.0	618,664.0	14.06
TA2C-11	2-2	16.66	37.10	-2.60	-3.04	1915	615	5	2.0	30.36	Sandstone	Gatun	1,025,237.0	618,664.0	14.06
TA2C-11	2-3	16.66	17.10	-2.60	-3.04	1910	575	4	2.0	31.41	Sandstone	Gatun	1,025,237.0	618,664.0	14.06
TA2C-12	9-1	49.09	49.41	-26.73	-27.05	2248	2000	6	2.0	34.78	Sandstone	Gatun	1,024,925.0	618,620.0	22.36
TA2C-12	9-2	49.09	49.41	-26.73	-27.05	2209	1793	5	2.0	37.91	Sandstone	Gatun	1,024,925.0	618,620.0	22.36
TA2C-13	2-1	7.91	8.25	20.87	20.53	1817	586	4	2.0	52.25	Sandstone	Gatun	1,024,600.0	618,400.0	28.78
TA2C-13	2-2	7.91	8.25	20.87	20.53	1839	750	8	2.0	62.95	Sandstone	Gatun	1,024,600.0	618,400.0	28.76
TA2C-15	4-1	31.21	31.58	3.77	3.40	1501	2000	8	2.0	25.51	Sandstone	Gatun	1,024,485.0	618,555.0	34.98
TA2C-15	4-2	31.21	31.58	3.77	3.40	1552	1897	9	2.0	25.51	Sandstone	Gatun	1,024,485.0	618,555.0	34.98
TA2C-16	2-1	20.69	20.95	7.79	7.53	1558	1138	5	2.0	68.68	Sandstone	Gatun	1,024,445.0	618,675.0	28.48
TA2C-16	5-1	29.90	30.17	-1.42	-1.69	1934	1667	4	2.0	31.71	Sandstone	Gatun	1,024,445.0	618,675.0	28.48
TA2C-16	5-2	29.90	30.17	2.16	1.89	1928	1517	13	2.0	52.15	Tuff	Gatun	1,024,840.0	619,355.0	32.06
TA2C-17	2-1	16.65	16.97	15.41	15.09	1873	874	16	2.0	47.23	Tuff	Gatun	1,024,840.0	619,355.0	32.06
TA2C-17	2-2	16.65	16.97	18.33	18.01	1890	690	5	1.9	55.62	Tuff	Gatun	1,024,485.0	618,555.0	34.98

Table C-II-GG-4
ACP 2001 Testing

Rock Type	Elevation Top (m)	Boring	Sample No	Unconf Comp c (Mpa)	Unit Weight Kglm3	Elastic Modulus E (Mpa)	Specific Gravity	Water Content (%)
Conglomerate								
	-21.67	TA1C-08	10-1	7	2043	1164	2.3	13.77
	-21.67	TA1C0-8	10-2	18	2184	4127	2.3	12.93
	-22.94	TA2C-09	4-1	5	1864	956	2.3	29.31
	-24.05	TA2C-08	6-1	3	2071	616	2.3	16.36
	-28.03	TA2C-04	6-2	6	1888	1910	2.3	31.98
	-28.03	TA2C-04	6-1	7	1833	2483	2.3	36.43
	-33.06	TA2C-06	5-1	4	1842	372	2.3	33.95
	-43.01	TA2C-12	9-2	10	2209	1793	2.2	20.4
	-43.01	TA2C-12	9-1	9	2248	2000	2.2	11.25
Microconglomerate								
	-12.92	TA2C-10	4-1	3	1733	552	2.2	42.94
	-17.45	TA2C-07	3-1	2	2033	167	2.2	24.13
	-24.43	TA2C-04	5-1	5	2070	726	2.2	22.72
Sandstone								
	35.51	TA1C-11	7-1	6	1600	852	2	55.51
	14.45	TA2C-13	2-2	5	1839	750	2	37.91
	14.45	TA2C-13	2-1	6	1817	586	2	34.78
	11.83	TA2C-17	2-2	4	1890	690	2	31.71

Rock Type	Elevation Top (m)	Boring	Sample No	Unconf Comp c (Mpa)	Unit Weight Kglm3	Elastic Modulus E (Mpa)	Specific Gravity	Water Content (%)
Sandstone	11.83	TA2C-17	2-1	5	1873	874	2	68.68
	11.79	TA1C-09	3-1	6	1621	907	2	54.13
	5.08	TA2C-16	5-2	9	1928	1517	2	25.51
	5.08	TA2C-16	5-1	8	1934	1667	2	25.51
	1.6	TA1C-10	7-1	5	1668	595	2	54.02
	-0.4	TA1C-01	4-1	5	1890	537	2	30.76
	-0.4	TA1C-01	4-2	10	1903	828	2	29.11
	-1.52	TA1C-01	5-1	4	1899	409	2	32.52
	-1.52	TA1C-01	5-2	5	1874	552	2	32.52
	-2.43	TA2C-15	4-2	8	1552	1897	2	62.95
	-2.43	TA2C-15	4-1	4	1501	2000	2	52.25
	-2.6	TA2C-11	2-2	5	1915	615	2	30.36
	-2.6	TA2C-11	2-3	4	1910	575	2	31.41
	-2.6	TA2C-11	2-1	5	1905	759	2	30.36
	-5.27	TA1C-10	9-1	2	1926	711	2	29.96
	-6.91	TA1C-08	6-2	5	1784	724	2	39.89
	-6.91	TA1C-08	6-1	4	1811	580	2	31.5
	-10.5	TA2C-08	3-1	13	1924	2793	2	30.92
	-10.5	TA2C-08	3-2	7	1897	1264	2	38.19
	-10.75	TA1C-11	17-1	3	2118	567	2	17.34
	-13.13	TA2C-08	4-2	8	1807	1506	2	39.13
	-13.13	TA2C-08	4-1	8	1826	1586	2	41.25
	-15.04	TA1C-07	8-1	3	1905	517	2.1	27.97

Table C-II-GG-4

Rock Type	Elevation Top (m)	Boring	Sample No	Unconf Comp c (Mpa)	Unit Weight Kglm3	Elastic Modulus E (Mpa)	Specific Gravity	Water Content (%)
Sandstone	-15.79	TA1C-04	1-2	5	1978	784	2	26.46
	-15.79	TA1C-04	1-1	5	2049	774	2	24.01
	-18.6	TA1C-02	3-1	25	2396	3241	2.4	9.91
	-20.48	TA2C-04	4-1	5	1966	2299	2.1	34.7
	-20.67	TA1C-07	9-1	5	1745	1011	2.1	34.14
	-20.67	TA1C-07	9-2	5	1746	876	2.1	40.98
	-26.49	TA2C-01	2-2	9	1916	2355	2	35.1
	-26.49	TA2C-01	2-1	9	1932	1705	2	30.05
	-27.65	TA2C-02	5-2	4	1966	1314	2	26.02
	-27.65	TA2C-02	5-1	3	1894	401	2	28.1
	-29.96	TA1C-06	8-1	15	1787	2638	2	34.37
	-29.99	TA2C-05	4-1	3	1794	1552	2	34.7
	-29.99	TA2C-05	4-2	2	1832	242	2	32.72
	-30.69	TA2C-03	5-1	3	1817	544	2	32.53
	-30.69	TA2C-03	5-2	5	1929	743	2	28.05
	-33.5	TA1C-02	6-1	11	1990	1583	2.4	33.17
	-35.42	TA2C-03	6-2	5	1898	1008	2	29.06
	-35.42	TA2C-03	6-1	5	1901	840	2	30.14
	-45.32	TA1C-05	8-1	10	1677	1533	2	44.36
	-45.32	TA1C-05	8-2	11	1678	1724	2	36.36
	-48.63	TA2C-05	8-1	8	1679	1452	2	47.69
	-48.63	TA2C-05	8-2	9	1703	2420	2	45.92

Table C-II-GG-4

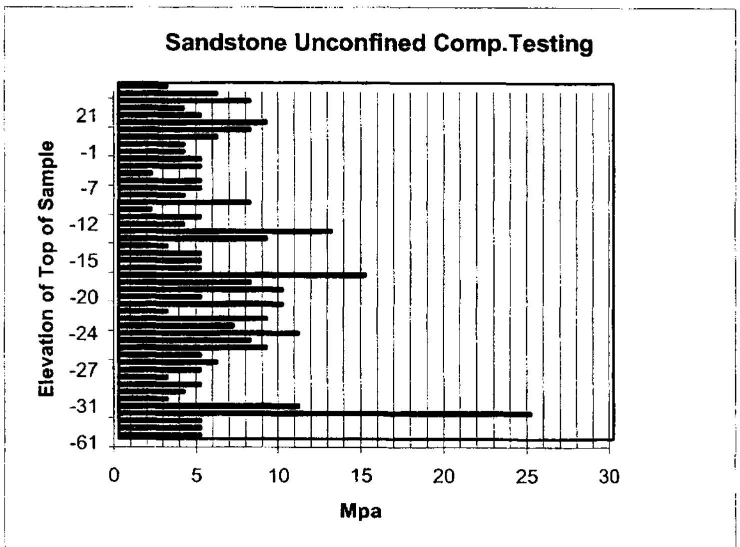
Tuff

Rock Type	*Elevation Top (m)*	*Boring*	*Sample No*	*Unconf Comp c (Mpa)*	*Unit Weight Kglm3*	*Elastic Modulus E (Mpa)*	*Specific Gravity*	*Water Content (%)*
	14.29	TA2C-16	2-1	5	1558	1138	1.9	55.62
	9.04	TA1C-09	4-1	13	1641	2483	2	52.15
	9.04	TA1C-09	4-2	16	1713	3065	2	47.23

Table C-II-GG-4

Table C-II-GG-5
Summary of Sandstone Testing
Average Values

Unit Weight Kg/m3	Elastic Modulus E (Mpa)	Unconf. Comp. (Mpa)	Specific Gravity	Water Cont. (%)
1857	**1291**	**6.51**	**2.02**	**35.40**
115 lb/cf	187250 psi	944 psi		



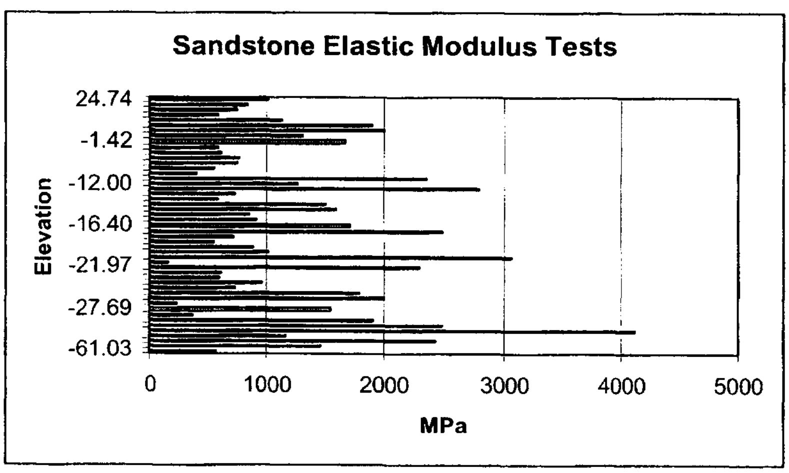


Table C-II-GG-6
Summary of Conglomerate Testing
Average Values

Unit Weight Kg/m3	Elastic Modulus E (Mpa	Unconf. Comp Mpa)	Specific Gravity	Water cont. (%)
1962	**1138**	**7.67**	**2.28**	**22.93**
122.5 lb/cf	165,000 psi	1112 psi		

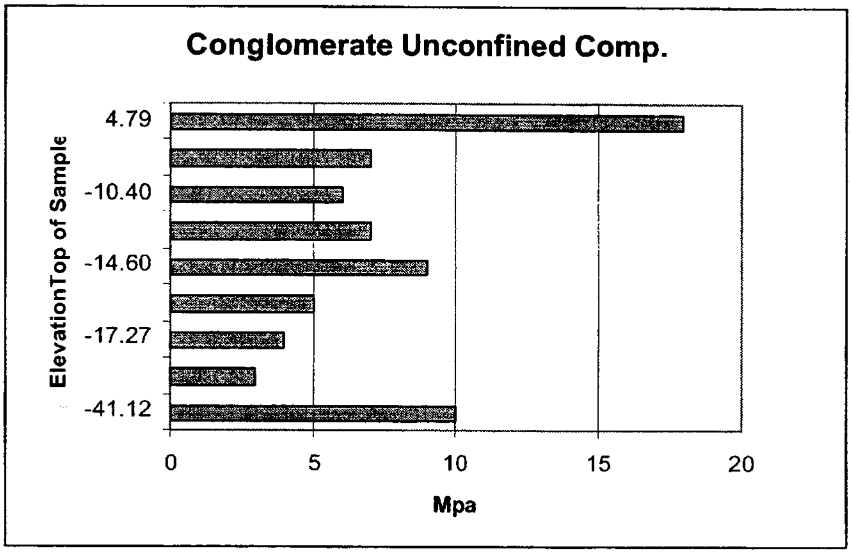


Table C-II-GG-7
Summary of Permeability Testing

Boring Number	Number of Tests	Average K per Boring
TA1C-05	7	1.28E-04
TA1C-07	3	2.24E-05
TA1C-09	2	1.14E-05
TA2C-03	9	4.84E-04
TA2C-05	2	2.95E-05
TA2C-09	2	1.66E-04
TA2C-11	9	5.54E-05
TA2C-13	14	2.14E-04
TA2C-15	1	3.16E-05
TOTAL	49	

Average Coefficient of Permeability (K) for All Testing: 1.88E-04

TABLE C-II-GG-8

Summary of Rock Shear Strength Testing

Description	Phi	Cohesion (psi)	Cohesion (kPa)
Laboratory values used in 1930's design. Triaxial and unconfined-compression tests	15.6	250	1723
Insitu direct shear testing. Concrete on rock-smooth surface	38	60	413
Insitu direct shear testing. Concrete on rock-grooved surface	30	110	758
2001 test - remolded tuffaceous sandstone (1 test)	31	3	21

Table C-II-GG-9
Metadata Used for Surfer Program

Boring No.	N	E	Top of Boring	Top Atlantic Muck	Top of Gatun (ovb)	Top Weath. Rock	Top of Sound Rock	Top of Conglomerate	Bottom of Conglomerate
TA1-01	1,028,285.9	619,036.3	2.91		2.91	2.91	-3.5		
TA1-02	1,026,949.8	619,258.4	4.06	1.5	-9.5	-10.9	-11.3	-29.4	-30.5
TA1-03	1,025,857.4	619,179.6	10.34		10.3	4.3	4.2	-16.1	-16.7
TA1-04	1,024,869.0	619,563.0	29.13		26.1	18.2	14.6	5.7	5.4
TA1-05	1,023,898.8	619,569.5	30.66		30.66	29	28.3	28.3	26.5
TA1C-01	1,027,710.0	618,865.0	13.71		13.4	11.7	10.3		
TA1C-02	1,027,395.0	619,195.0	1.14	-0.9	-6.9	-8.4	-10.8		
TA1C-03	1,026,920.0	619,075.0	3.16	-1					
TA1C-04	1,026,505.0	619,410.0	4.89	-3.1	-9.7	-13.1	-14		
TA1C-05*	1,026,062.0	619,407.0	1.88	0.3	-7.7	-7.7	-8.8		
TA1C-06	1,025,615.0	619,460.0	13.22		6.2	4.2	3.4		
TA1C-07*	1,025,585.0	619,260.0	13.43		10.1		10.1	-8.1	-13.1
TA1C-08	1,025,305.0	619,430.0	18.90		18.9		18.9	2.8	1.1
TA1C-09*	1,024,840.0	619,355.0	32.06		32.06	20.9	19.8	7.3	7.1
TA1C-10	1,024,415.0	619,565.0	27.64	27.6	27.6	26.6	21.6	-20.9	-21
TA1C-11	1,024,145.0	619,750.0	63.71		63.7	62.4	62		
TA2-01	1,026,794.2	618,642.5	1.34	-1.7	-15.2	-18.5	-21.6		
TA2-02	1,026,573.7	618,833.5	2.52	0.8	-17	-23	-25		
TA2-03	1,026,389.5	618,614.3	0.87	-0.6	-17.1	-19.1	-19.6		
TA2-04	1,025,984.5	618,649.9	4.72		4.7	-2.8	-4.6	-28.6	-29.4
TA2-05	1,025,468.0	618,697.3	4.40		-3.1	-4.9	-5.8	-15	-19.1
TA2-06	1,024,953.0	618,436.0	14.90		5.9	2.9	2.6	0.9	-6.6
TA2-07	1,024,680.7	618,487.2	28.57		25.1		25.1	14.9	8.1
TA2-08	1,024,400.0	618,430.0	16.70	16.7	12.5	11.4	9.4		
TA2-09	1,024,190.0	618,450.0	7.60	7.6	-17.7		-20.1		
TA2-10	1,024,050.0	618,385.0	6.50	6.5	-27.3	-29.9			
TA2C-01	1,026,740.0	618,920.0	3.34	-0.2	-13.7	-18.7	-19.4		
TA2C-02	1,026,355.0	618,870.0	2.06	1.1	-2.9	-4.9	-9.2	-27.9	-28.4
TA2C-03*	1,026,170.0	618,715.0	3.55		-1.5	-3.5	-3.8		
TA2C-04	1,025,950.0	618,820.0	4.83	0.8	-3.2	-5.2	-6.1	-26.1	-27.4
TA2C-05*	1,025,780.0	618,670.0	1.25	-0.7	-14.2	-16.1	-18	-26.8	-29.9
TA2C0-6	1,025,795.0	618,550.0	2.47	-1.5	-8.1	-10.1	-13.4	0.9	-6.6
TA2C-07	1,025,595.0	618,794.0	5.09	1.1	-4.4		-9.7	-18	-22.6
TA2C-08	1,025,590.0	618,595.0	2.75		1.8		-0.3	-22.7	-29.3
TA2C-09*	1,025,405.0	618,509.0	4.05		1	-5	-8.7	-17.7	-23.7
TA2C-10	1,025,220.0	618,745.0	9.98		-1		-1.9	-1.9	-4.2
TA2C-11*	1,025,237.0	618,664.0	14.06		9.1	4.1	3.2	-4.4	-7.8
TA2C-12	1,025,135.0	618,525.0	6.08		-2.9	-4	-6.8		
TA2C-13*	1,024,925.0	618,620.0	22.36			19.4	18.5	7.7	3.4
TA2C-14	1,024,665.0	618,686.0	43.05		37		36.1	24.8	20.8
TA2C-15*	1,024,600.0	618,400.0	28.78		18.8	17.3	12.9	17.3	12.9

Boring No.	N	E	Top of Boring	Top Atlantic Muck	Top of Gatun (ovb)	Top Weath. Rock	Top of Sound Rock	Top of Conglomerate	Bottom of Conglomerate
TA2C-16	1,024,485.0	618,555.0	34.98		30	20.2	11.9		
TA2C-17	1,024,445.0	618,675.0	28.48		26.5	20.5	17.2	19.5	
G-40	1,026,865.0	618,895.0	2.62		-13.99	-13.99	-20.09	-22.1	
G-48	1,027,115.0	618,920.0	0.91	3.96		-14.02	-18.85		
G-49	1,027,330.0	618,905.0	3.90			-5.73	-6.37		
G-71	1,029,260.0	618,835.0	0.61	-10.97	-14.02	-16.31			
G-74	1,028,777.0	618,860.0	1.07	-4.72	-6.25	-9.296			
G-76	1,028,575.0	618,800.0	0.92	-1.53	-5.12	-8.14	-9.6		
G-79	1,028,355.0	618,795.0	0.76	-3.2	-4.12	-5.64	-5.94		
G-83	1,027,945.0	618,830.0	1.40	-4.69	-6.61		-8.75		
G-85	1,028,190.0	618,875.0	1.77	-5.55	-7.83	-9.21	-10.73		
G-129	1,027,665.0	618,885.0	9.69			8.9	8.02		
G-420	1,026,109.3	619,444.9	4.72			-3.90	-12.04		
G-421	1,024,104.7	618,730.9	22.16			21.43	19.66		
G-422	1,026,298.2	619,038.2	8.29			7.38			
G-423	1,024,087.3	618,708.5	10.55			9.39	-1.43		
G-424	1,025,995.7	619,511.7	7.53			6.61	-1.77		
G-425	1,024,160.0	618,660.7	13.50			11.98	7.38		
G-426	1,026,323.5	619,031.7	3.47			2.56	-8.78		
G-427	1,024,235.8	618,609.6	14.36			-7.16	-15.18		
G-428	1,025,929.8	619,480.8	5.55			-9.88	-13.96		
G-429	1,026,359.4	619,035.9	3.05			-10.97	-14.57		
G-430	1,025,869.4	619,489.5	4.85			-10.79	-11.13		
G-431	1,024,273.9	618,584.2	12.68			-26.85	-31.39		
G-432	1,026,390.5	619,039.1	3.11			-9.08			
G-433	1,026,256.6	619,290.3	4.66			-16.92	-18.07	-16.90	
G-434	1,026,226.1	619,292.2	5.39			-9.85	-12.74		
G-435	1,024,287.9	618,605.8	23.74			-27.28	-31.33		
G-436	1,026,195.7	619,294.1	2.26			-14.81	-16.12		
G-437	1,025,786.1	619,445.6	7.13			-6.58			
G-438	1,026,210.9	619,292.6	2.96			-6.19	-10.91		
G-439	1,024,211.9	618,656.0	21.43			1.46	-10.82		
G-440	1,025,824.1	619,495.8	7.22			2.96			
G-441	1,026,241.4	619,290.9	4.45			-13.99	-16.28		
G-442	1,024,199.1	618,664.5	21.03			6.49	-5.39		
G-443	1,025,778.1	619,507.8	6.95			-0.82	-2.96		
G-444	1,024,197.8	618,635.0	15.21			1.46	-5.61		
G-445	1,026,271.5	619,285.1	4.60			-17.80	-18.71	-17.80	
G-446	1,024,159.3	618,742.1	21.85			16.86	15.94		
G-447	1,026,331.5	619,166.4	5.79			-8.53	-14.94		
G-448	1,024,197.5	618,716.7	22.74			11.37	9.30		
G-449	1,024,235.2	618,691.3	22.43			1.62	-8.81		
G-450	1,026,260.2	619,183.4	2.38			-2.19	-9.81		
G-451	1,024,308.5	618,635.9	21.37			-22.10	-25.91		
G-452	1,026,227.0	619,188.3	1.89			-15.06	-16.15		

Table C-II-GG-9

Boring No.	N	E	Top of Boring	Top Atlantic Muck	Top of Gatun (ovb)	Top Weath. Rock	Top of Sound Rock	Top of Conglomerate	Bottom of Conglomerate
G-453	1,024,303.2	618,709.6	6.86			2.90	2.10		
G-454	1,026,150.9	619,204.1	2.80			-12.53	-15.09	-12.50	
G-455	1,024,248.3	618,711.0	11.58			10.03	2.53		
G-456	1,024,332.9	618,673.0	16.86			-15.21	-23.13		
G-457	1,026,134.4	619,295.8	2.87			-14.81	-15.21	-14.80	
G-458	1,024,278.1	618,672.8	16.86			-12.71	-18.38		
G-459	1,026,058.3	619,301.9	3.32			-7.41	-8.66		
G-01		619,440.1	63.52				60.695		
G-02		619,433.6	13.2				3.5075		
G-03		619,439.2	31.09				23.79		
G-05		619,203.2	4.88				-0.61		
G-6		619,040.2	3.05				-13.725		
G-07		619,191.5	3.05				-17.995		
G-08		619,273.1	4.57				-16.165		
G-09		619,354.4	5.18				-13.42		
G-10		619,489.8	28.5				24.8575		
G-11		619,343.9	22.25				14.64		
G-12		619,454.6	28.35				19.825		
G-16		619,508.1	27.92				18.1475		
G-17		619,435.6	42.85				40.565		
G-18		619,360.0	37.16				17.8425		
G-19		619,586.3	49.13				46.97		
G-20		619,698.6	74.07				71.675		
G-21		619,137.7	8.29				-16.47		
G-23		619,227.4	5.55				-4.88		
G-38		619,184.6	4.79				-19.825		
G-42		619,272.3	4.63				-19.825		
G-46		619,264.5	6.52				-9.455		
G-98		619,251.4	5.15				-13.42		
G-100		619,098.8	7.04				-13.725		
G-101		619,333.0	4.91				-19.825		
G-103		619,098.7	4.91				-6.71		
G-104		619,274.6	5.12				-22.265		
G-105		619,193.2	5.09				-11.285		
G-107		619,338.4	5.33				-6.2525		
G-108		619,206.8	3.72				-16.9275		
G-109		619,322.7	6.49				-16.3175		
G-110		619,269.8	16.86				10.675		
G-111		619,289.9	4.85				-14.945	-14.6	
G-112		619,255.4	2.38				-14.945		
G-114		619,264.7	15.24				4.88		
G-115		619,370.0	5.36				-9.9125	-11.3	
G-116		619,201.8	3.47				-25.01	-24.7	
G-117		619,307.0	18.41				12.505	-6.4	
G-118		619,383.1	6.28				-3.05	-11.3	

Table C-II-GG-9

Boring No.	N	E	Top of Boring	Top Atlantic Muck	Top of Gatun (ovb)	Top Weath. Rock	Top of Sound Rock	Top of Conglomerate	Bottom of Conglomerate
G-120		619,235.2	4.27				-11.895		
G-121		619,251.6	13.17				7.32		
G-122		619,386.3	6.49				-13.115		
G-123		619,297.0	4.21				-14.335	-14	
G-124		619,284.9	8.96				-2.2875		
G-125		619,235.5	6.77				-3.05	-11	
G-127		619,309.3	4.6				-13.42	-44	
G-128		619,309.0	6.25				-8.9975		
G-130		619,321.7	7.8				-1.9825	-9.1	
G-131		619,236.9	4.05				-9.455	-56	
G-133		619,231.6	5.49				-5.49		
G-134		619,329.9	12.41				5.0325	-8	
G-136		619,254.0	4.66				-12.2	-13.7	
G-138		619,413.1	6.95				-6.405		
G-139		619,362.6	24.14				17.995	-1.2	
G-140		619,418.8	20.67				14.335	-4.2	
G-142		619,411.5	9.3				2.135	-5.8	
G-144		619,443.0	17.1				8.235	-1.8	
G-145		619,287.0	17.92				8.0825	-2.4	
G-146		619,354.8	21.55				16.775	-3	
G-147		619,298.2	11.34				6.405		
G-148		619,476.0	28.86				24.8575		
G-150		619,370.7	12.41				5.795		
G-151		619,527.4	24.17				17.69		
G-152		619,444.3	42.06				18.91		
G-153		619,452.3	21.49				19.3675		
G-155		619,532.6	26.06				20.8925		
G-156		619,459.6	32.16				25.315	7.3	
G-157		619,498.4	33.28				24.4		
G-158		619,371.8	31.09				22.875		
G-159		619,539.8	34.41				28.365		
G-160		619,423.6	24.66				21.655		
G-161		619,423.1	41.97				32.94		
G-162		619,381.8	30.88				26.3825		
G-163		619,381.3	28.62				22.875		
G-164		619,538.3	34.75				35.8375		
G-165		619,391.6	27.46				20.2825		
G-166		619,411.6	30.63				29.28		
G-167		619,284.7	30.18				24.095		
G-168		619,544.9	32.46				27.6025	12.5	
G-169		619,361.6	41.24				35.8375		
G-170		619,533.0	37.09				26.9925		
G-171		619,293.2	25.39				18.3		
G-172		619,523.5	38.01				30.3475		
G-173		619,363.1	37.09				34.0075		

Table C-II-GG-9

Boring No.	N	E	Top of Boring	Top Atlantic Muck	Top of Gatun (ovb)	Top Weath. Rock	Top of Sound Rock	Top of Conglomerate	Bottom of Conglomerate
G-174		619,567.4	31.09				26.23		
G-175		619,324.7	16.31				12.505		
G-176		619,366.7	28.59				17.995		
G-177		619,501.0	31.64				23.485		
G-178		619,463.3	39.62				30.195		
G-179		619,408.1	22.65				17.385		
G-180		619,501.5	30.36				13.115		
G-181		619,380.4	37.67				29.1275		
G-182		619,417.8	41.51				32.635		
G-183		619,323.7	28.35				24.8575		
G-184		619,382.5	41.51				38.43		
G-185		619,427.5	40.33				32.635		
G-186		619,663.4	62.21				58.4075		
G-187		619,327.0	34.75				29.7375		
G-188		619,358.5	49.04				46.97		
G-189		619,579.0	38.44				35.38		
G-190		619,401.4	48.28				46.665		
G-191		619,320.2	43.37				35.2275		
G-192		619,527.6	37.16				31.72	1.2	
G-193		619,346.2	67.15				63.745		
G-194		619,457.5	38.53				38.43	1.9	
G-195		619,699.6	34.96				32.4825		
G-196		619,443.1	58.37				54.595		
G-197		619,540.3	64.56				63.2875		
G-199		619,563.7	68.55				64.965		
G-200		619,615.9	54.44				54.1375		
G-202		619,552.5	77.18				72.285		
G-203		619,664.0	67.09				64.8125		
G-205		619,449.9	25.6				16.9275		
G-206		619,573.4	29.87				20.5875		

Table C-II-GG-9

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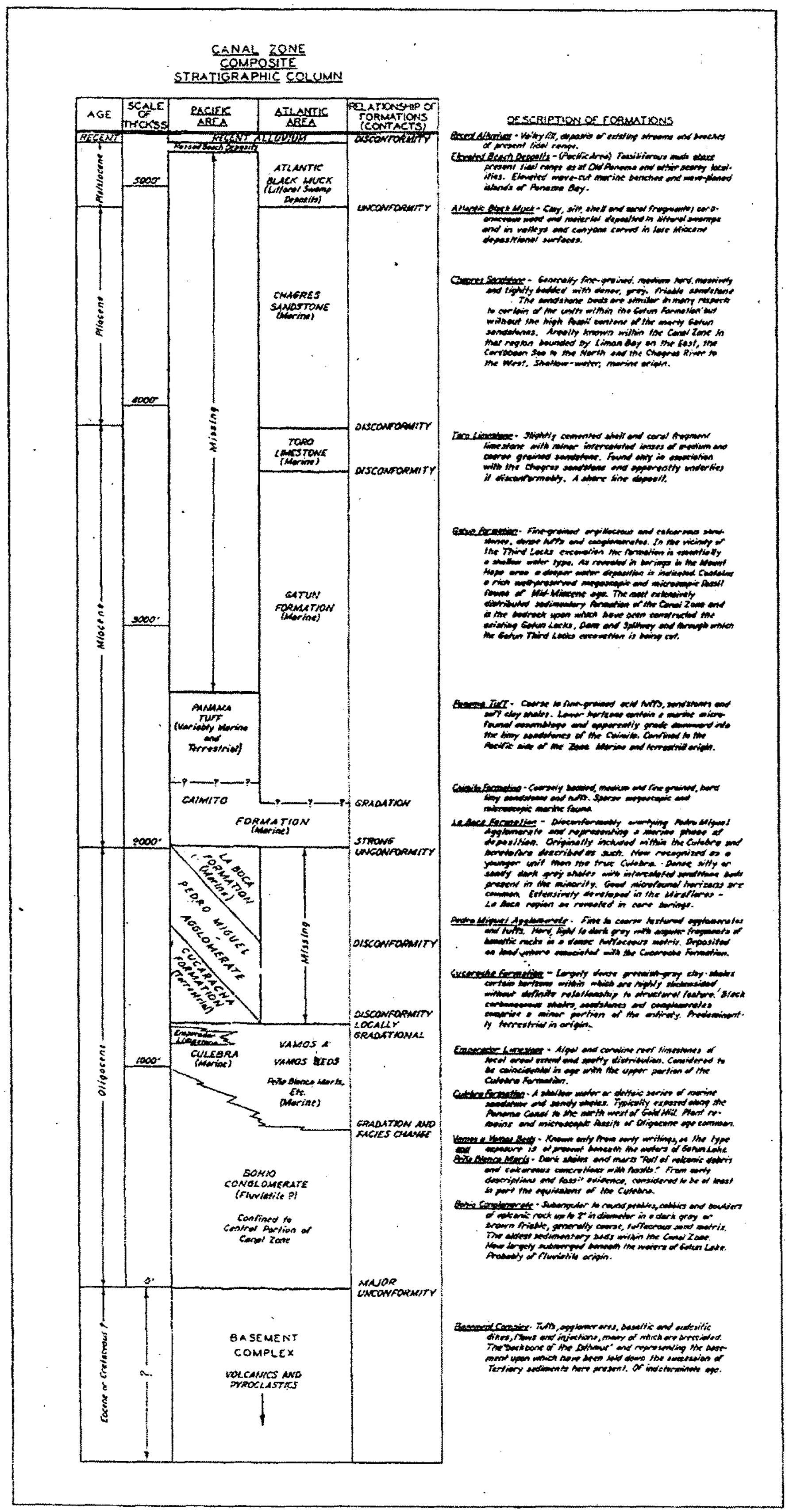
Appendix C – Geotechnical Investigations, Analyses and Designs

Part II – Geology

Figures

Figure C-II-GG-1	Regional Geologic Column
Figure C-II-GG-2	Generalized Geologic Column
Figure C-II-GG-3	Contour Surface on top of Sound Rock
Figure C-II-GG-4	Contour Surface on top of Unit 5 Conglomerate
Figure C-II-GG-5	Contour Surface on Phreatic Surface (based on reading at comp.)
Figure C-II-GG-6	Geologic Profile With Unconfined Compression Tests
Figure C-II-GG-7	Geologic Profile With Water Content Results
Figure C-II-GG-8	Geologic Profile With Units Weights Results
Figure C-II-GG-9	Geologic Profile With Pressure Test Results
Figure C-II-GG-10	2001 Direct Shear Test Result
Figure C-II-GG-11	Typical Rock cut Design within Lock Chamber Excavation
Figure C-II-GG-12	Typical Rock cut Design within Approach Channel Excavation



CANAL ZONE
COMPOSITE
STRATIGRAPHIC COLUMN
AGE
SCALE OF THICK'SS
PACIFIC AREA
ATLANTIC AREA
RELATIONSHIP OF FORMATIONS (CONTACTS)
RECENT
RECENT ALLUVIUM
DISCONFORMITY
Pleistocene
Raised Beach Deposits
ATLANTIC BLACK MUCK (Littoral Swamp Deposits)
5000'
UNCONFORMITY
Pliocene
CHAGRES SANDSTONE (Marine)
Missing
4000'
DISCONFORMITY
TORO LIMESTONE (Marine)
DISCONFORMITY
Miocene
GATUN FORMATION (Marine)
3000'
PANAMA TUFF (Variably Marine and Terrestrial)
CAIMITO FORMATION (Marine)
GRADATION
2000'
STRONG UNCONFORMITY
Oligocene
LA BOCA FORMATION (Marine)
PEDRO MIGUEL AGGLOMERATE
CUCARACHA FORMATION (Terrestrial)
Missing
DISCONFORMITY
DISCONFORMITY LOCALLY GRADATIONAL
Emperador Limestone
CULEBRA (Marine)
VAMOS A VAMOS BEDS
Peña Blanca Marls, Etc. (Marine)
1000'
GRADATION AND FACIES CHANGE
BOHIO CONGLOMERATE (Fluviatile ?)
Confined to Central Portion of Canal Zone
0'
MAJOR UNCONFORMITY
Eocene or Cretaceous ?
?
BASEMENT COMPLEX
VOLCANICS AND PYROCLASTICS
DESCRIPTION OF FORMATIONS
Recent Alluvium - Valley fill, deposits of existing streams and beaches of present tidal range.
Elevated Beach Deposits - (Pacific Area) Fossiliferous muds above present tidal range as at Old Panama and other nearby localities. Elevated wave-cut marine benches and wave-planed islands of Panama Bay.
Atlantic Black Muck - Clay, silt, shell and coral fragments; carbonaceous wood and material deposited in littoral swamps and in valleys and canyons carved in late Miocene depositional surfaces.
Chagres Sandstone - Generally fine-grained, medium hard, massively and tightly bedded with dense, grey, friable sandstone. The sandstone beds are similar in many respects to certain of the units within the Gatun Formation but without the high fossil contents of the marly Gatun sandstones. Areally known within the Canal Zone in that region bounded by Limon Bay on the East, the Caribbean Sea to the North and the Chagres River to the West. Shallow-water, marine origin.
Toro Limestone - Slightly cemented shell and coral fragment limestone with minor intercalated lenses of medium and coarse grained sandstone. Found only in association with the Chagres sandstone and apparently underlies it disconformably. A shore line deposit.
Gatun Formation - Fine-grained argillaceous and calcareous sandstones, dense tuffs and conglomerates. In the vicinity of the Third Locks excavation the formation is essentially a shallow water type. As revealed in borings in the Mount Hope area a deeper water deposition is indicated. Contains a rich well-preserved megascopic and microscopic fossil fauna of Mid-Miocene age. The most extensively distributed sedimentary formation of the Canal Zone and is the bedrock upon which have been constructed the existing Gatun Locks, Dam and Spillway and through which the Gatun Third Locks excavation is being cut.
Panama Tuff - Coarse to fine-grained acid tuffs, sandstones and soft clay shales. Lower horizons contain a marine microfaunal assemblage and apparently grade downward into the limy sandstones of the Caimito. Confined to the Pacific side of the Zone. Marine and terrestrial origin.
Caimito Formation - Coarsely bedded, medium and fine grained, hard limy sandstone and tuffs. Sparse megascopic and microscopic marine fauna.
La Boca Formation - Disconformably overlying Pedro Miguel Agglomerate and representing a marine phase of deposition. Originally included within the Culebra and heretofore described as such. Now recognized as a younger unit than the true Culebra. Dense silty or sandy dark grey shales with intercalated sandstone beds present in the minority. Good microfaunal horizons are common. Extensively developed in the Miraflores - La Boca region as revealed in core borings.
Pedro Miguel Agglomerate - Fine to coarse textured agglomerates and tuffs. Hard, light to dark gray with angular fragments of basaltic rocks in a dense tuffaceous matrix. Deposited on land where associated with the Cucaracha Formation.
Cucaracha Formation - Largely dense greenish-gray clay-shales certain horizons within which are highly slickensided without definite relationship to structural feature. Black carbonaceous shales, sandstones and conglomerates comprise a minor portion of the entirety. Predominantly terrestrial in origin.
Emperador Limestone - Algal and coraline reef limestones of local areal extent and spotty distribution. Considered to be coincidental in age with the upper portion of the Culebra Formation.
Culebra Formation - A shallow water or deltaic series of marine sandstone and sandy shales. Typically exposed along the Panama Canal to the north west of Gold Hill. Plant remains and microscopic fossils of Oligocene age common.
Vamos a Vamos Beds - Known only from early writings, as the type and exposure is at present beneath the waters of Gatun Lake.
Peña Blanca Marls - Dark shales and marls "full of volcanic debris and calcareous concretions with fossils." From early descriptions and fossil evidence, considered to be at least in part the equivalent of the Culebra.
Bohio Conglomerate - Subangular to round pebbles, cobbles and boulders of volcanic rock up to 2' in diameter in a dark gray or brown friable, generally coarse, tuffaceous sand matrix. The oldest sedimentary beds within the Canal Zone. Now largely submerged beneath the waters of Gatun Lake. Probably of fluviatile origin.
Basement Complex - Tuffs, agglomerates, basaltic and andesitic dikes, flows and injections, many of which are brecciated. The "backbone of the Isthmus" and representing the basement upon which have been laid down the succession of Tertiary sediments here present. Of indeterminate age.

Figure GG-1

PANAMA CANAL, CONCEPT DESIGN STUDY, GATUN THIRD LOCK

GENERALIZED GEOLOGIC COLUMN AT STATION 12+000, ALTERNATE ALIGNMENT 2

UNIT	ELEVATION	LEGEND	DESCRIPTION
1	0		RECENT FILL, silty to sandy clay, brown to reddish brown
2			ATLANTIC MUCK, clay, organic highly plastic, high moisture content, weak, black
3	-5		GATUN OVERBURDEN, saprolitic residual soil derived from underlying Gatun Run, silty to sandy clay, low to moderate plasticity, moderate strength, gray to brown
4	-10 -15 -20 -25	4A 4A 4A	SANDSTONE, tuffaceous, medium to fine grained, medium soft to medium hard, massive bedding, moderate jointing, zones of calcareous marine shell detritus, dark gray with mottled black and white zones UNIT 4A horizontally bedded zones, beds 30 - 60 mm thick with 1 - 5 mm clay in fill between beds
5			CONGLOMERATE, conglomeritic sandstone, medium soft, massive, weak, subrounded clasts in fine grained matrix, gray
6	-30		SANDSTONE, pumiceous, soft to moderate hard, moderate strength, massive, light gray
7	-35		TUFF, pumiceous, medium hard, moderate strength, jointed, medium gray

Notes:

1. Contact elevations shown are averages, there is considerable variation over the site.
2. The top of Unit 5 Conglomerate was used as a marker bed for correlation purposes. The elevation of this bed ranges from approximately -30 meters at the north end of the site to approximately 25 meters at the south end.
3. All units shown may not be present over the entire site.

Figure GG-2

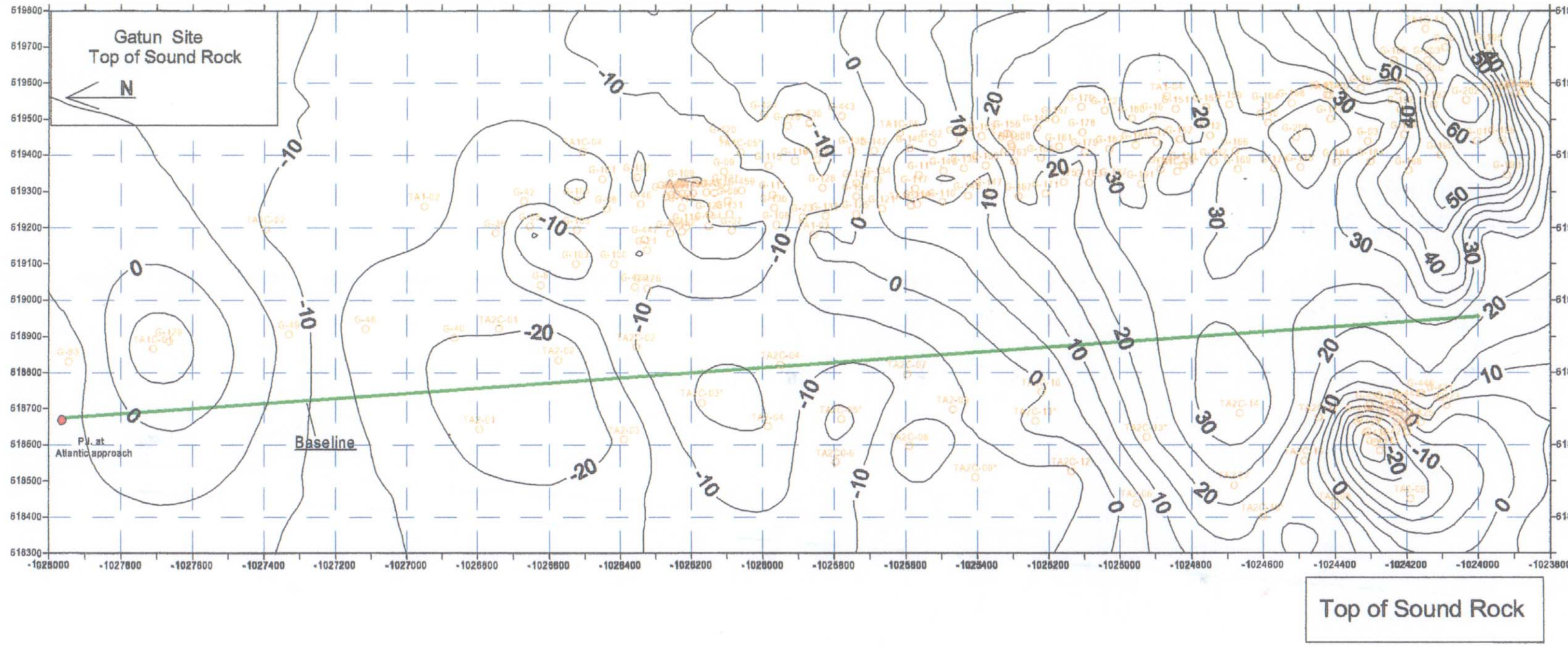
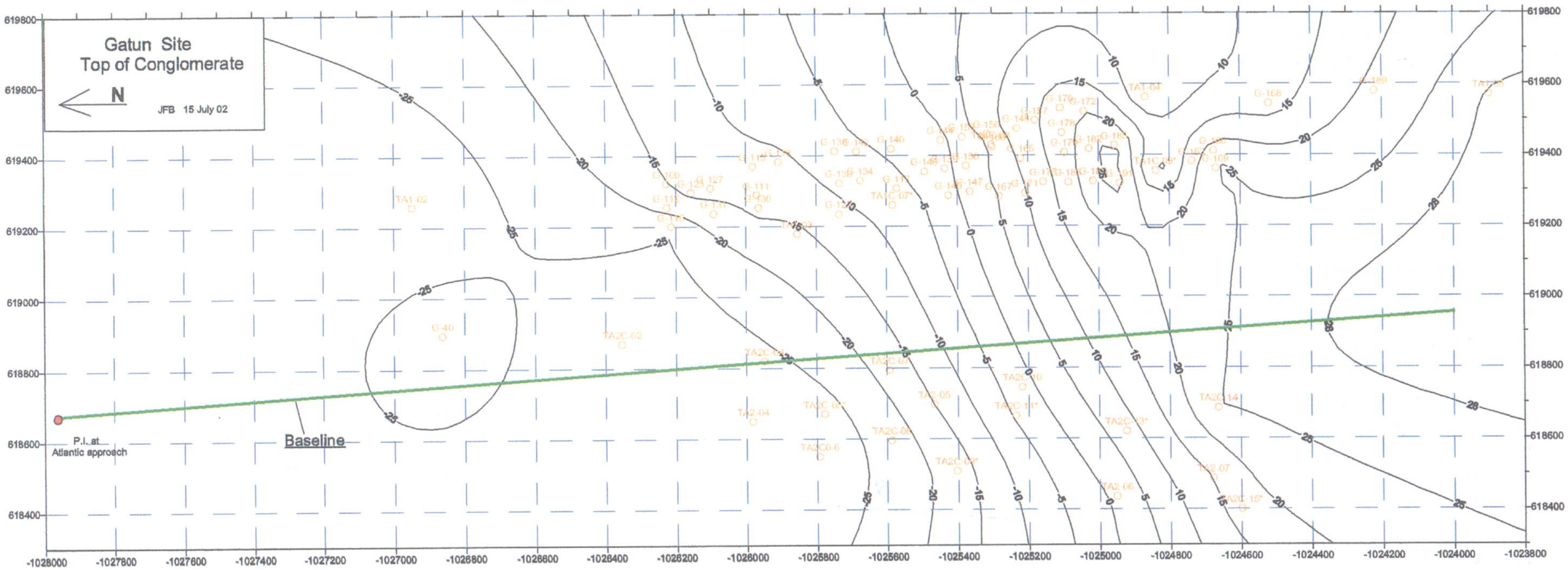
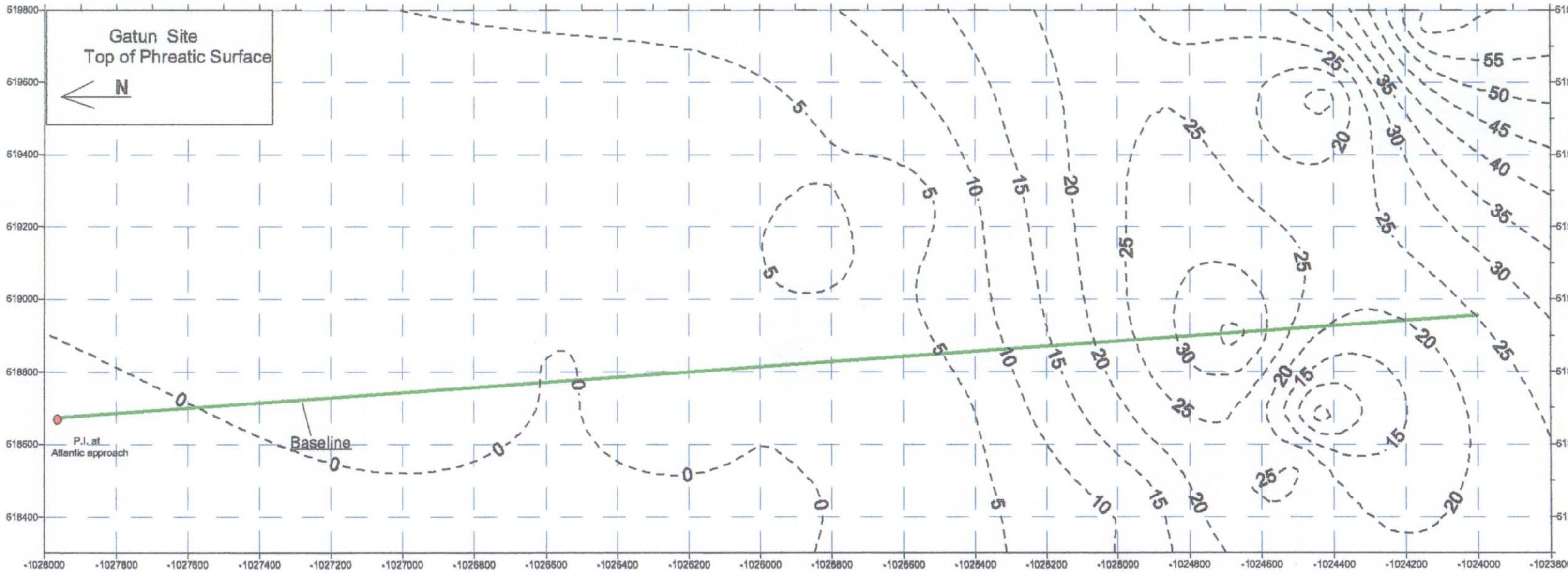

Gatun Site
Top of Sound Rock
N
P.I. at Atlantic approach
Baseline
Top of Sound Rock

Figure GG-3



Top of Unit 5 Conglomerate

Figure GG-4



Top of Phreatic Surface
(based on water readings
at completion of boring)

Figure GG-5

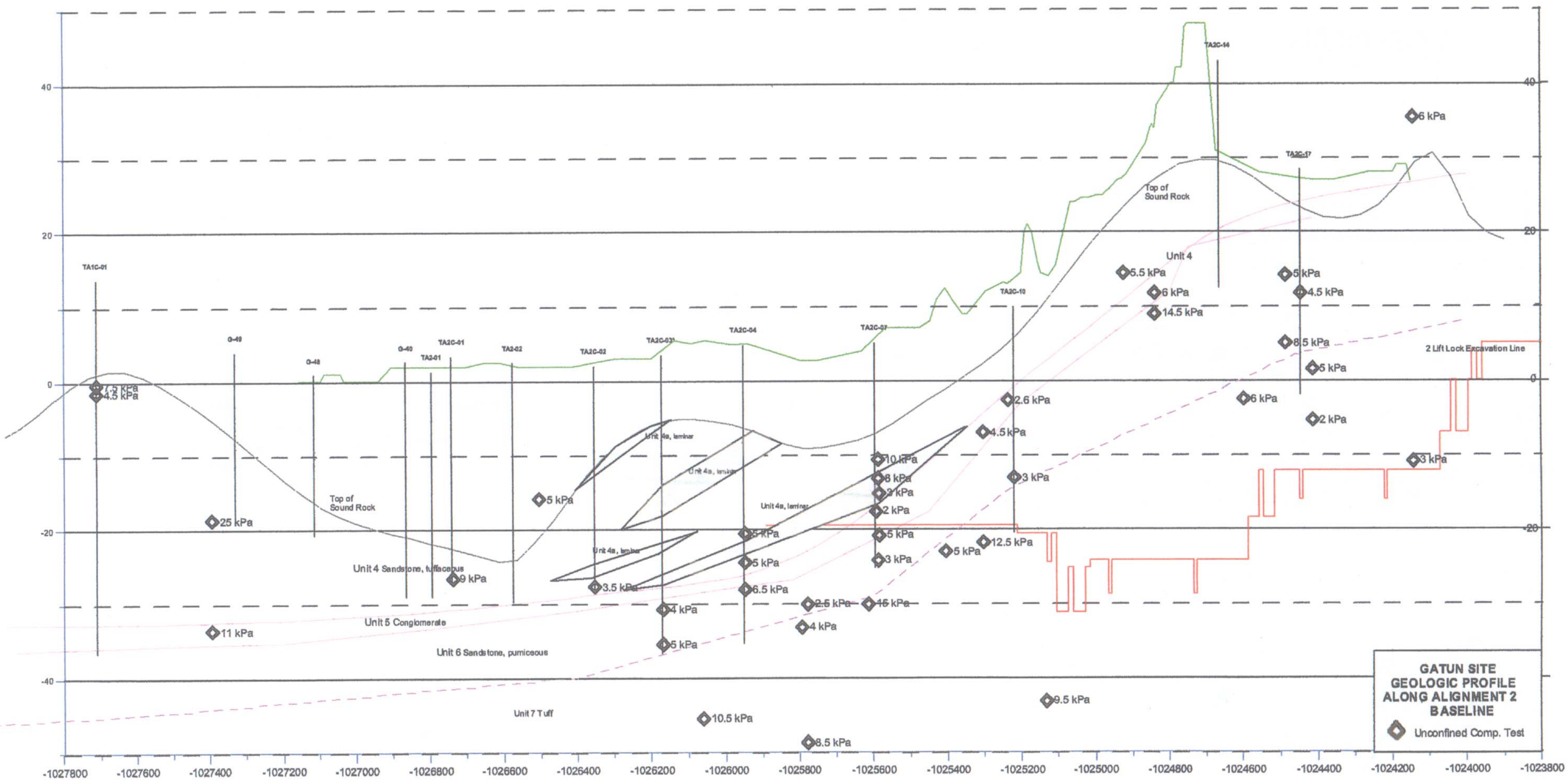

GATUN SITE
GEOLOGIC PROFILE
ALONG ALIGNMENT 2
BASELINE
Unconfined Comp. Test
Top of Sound Rock
Unit 4
Unit 4 Sandstone, tuffaceous
Unit 5 Conglomerate
Unit 6 Sandstone, pumiceous
Unit 7 Tuff
2 Lift Lock Excavation Line
6 kPa
5.5 kPa
6 kPa
14.5 kPa
4.5 kPa
25 kPa
11 kPa
9 kPa
3.5 kPa
10.5 kPa
8.5 kPa
9.5 kPa
12.5 kPa
2.6 kPa
4.5 kPa
3 kPa
10 kPa
2 kPa
16 kPa
2.5 kPa
6.5 kPa
40
20
0
-20
-40
-1027800
-1027600
-1027400
-1027200
-1027000
-1026800
-1026600
-1026400
-1026200
-1026000
-1025800
-1025600
-1025400
-1025200
-1025000
-1024800
-1024600
-1024400
-1024200
-1024000
-1023800

Figure GG-6

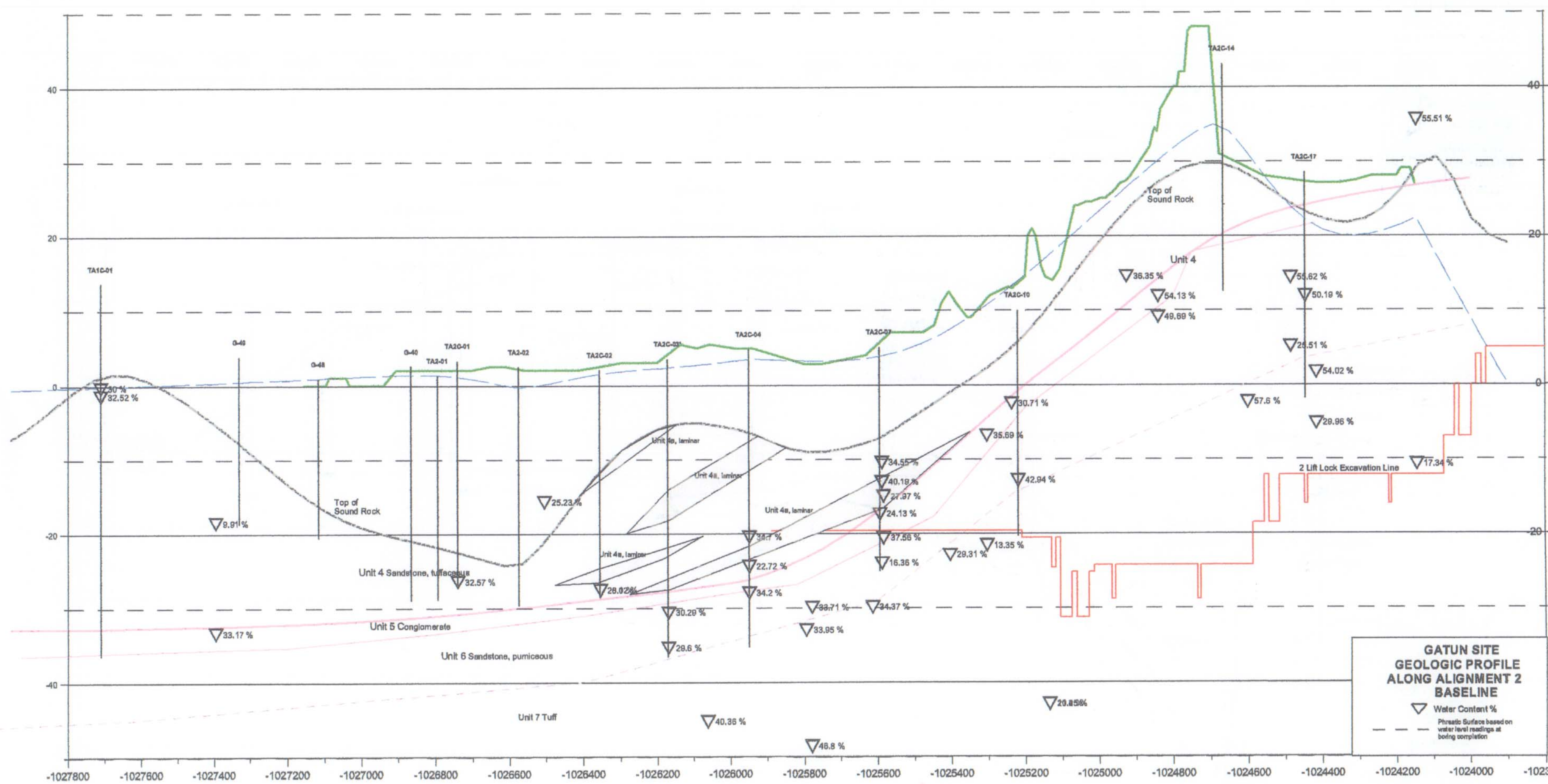
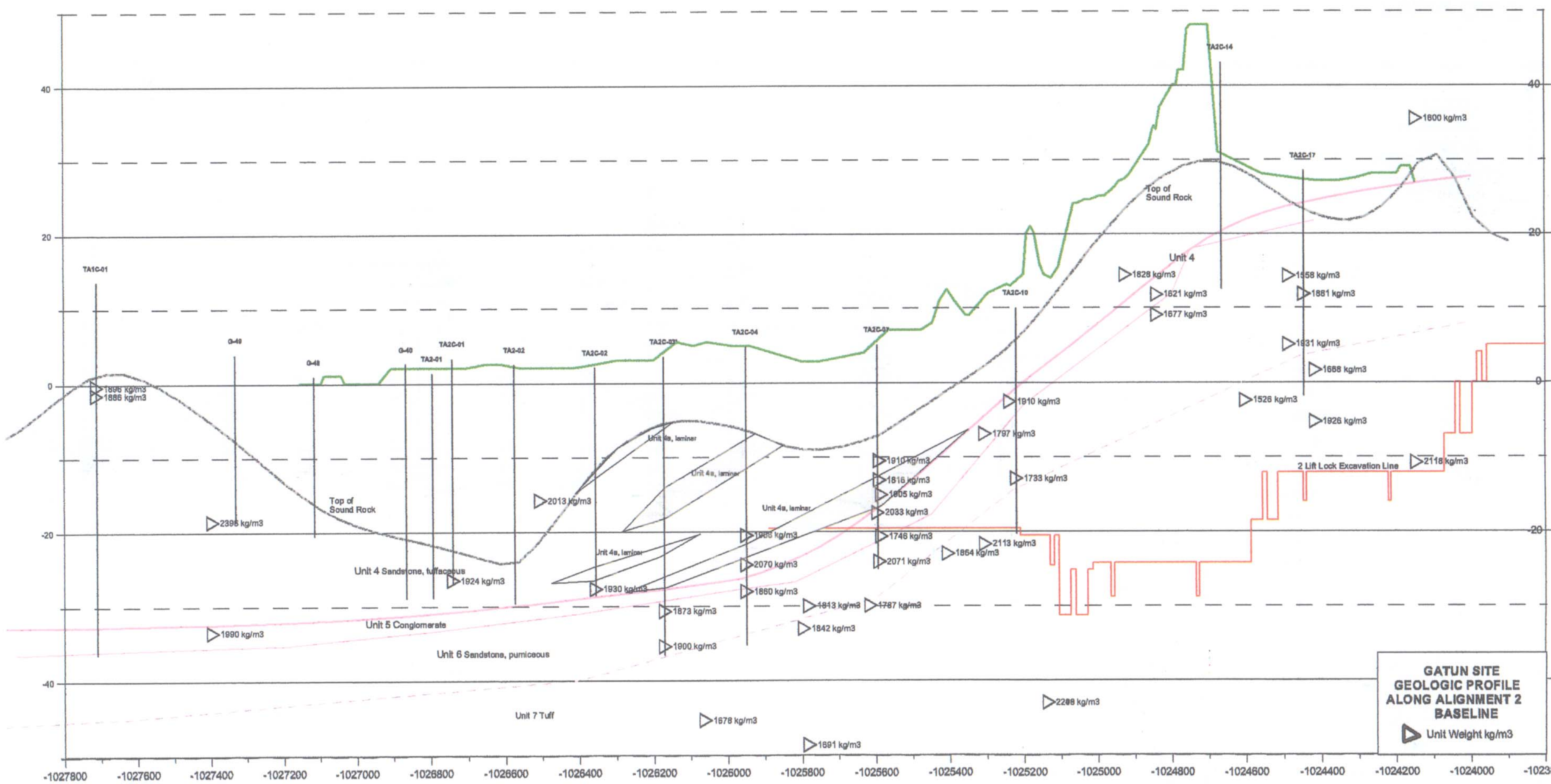

GATUN SITE
GEOLOGIC PROFILE
ALONG ALIGNMENT 2
BASELINE
Water Content %
Top of Sound Rock
Unit 4
Unit 4 Sandstone, tuffaceous
Unit 5 Conglomerate
Unit 6 Sandstone, pumiceous
Unit 7 Tuff
2 Lift Lock Excavation Line

Figure GG-7



Top of Sound Rock
Unit 4
Unit 4 Sandstone, tuffaceous
Unit 5 Conglomerate
Unit 6 Sandstone, pumiceous
Unit 7 Tuff
2 Lift Lock Excavation Line
GATUN SITE
GEOLOGIC PROFILE
ALONG ALIGNMENT 2
BASELINE
Unit Weight kg/m3

Figure GG-8

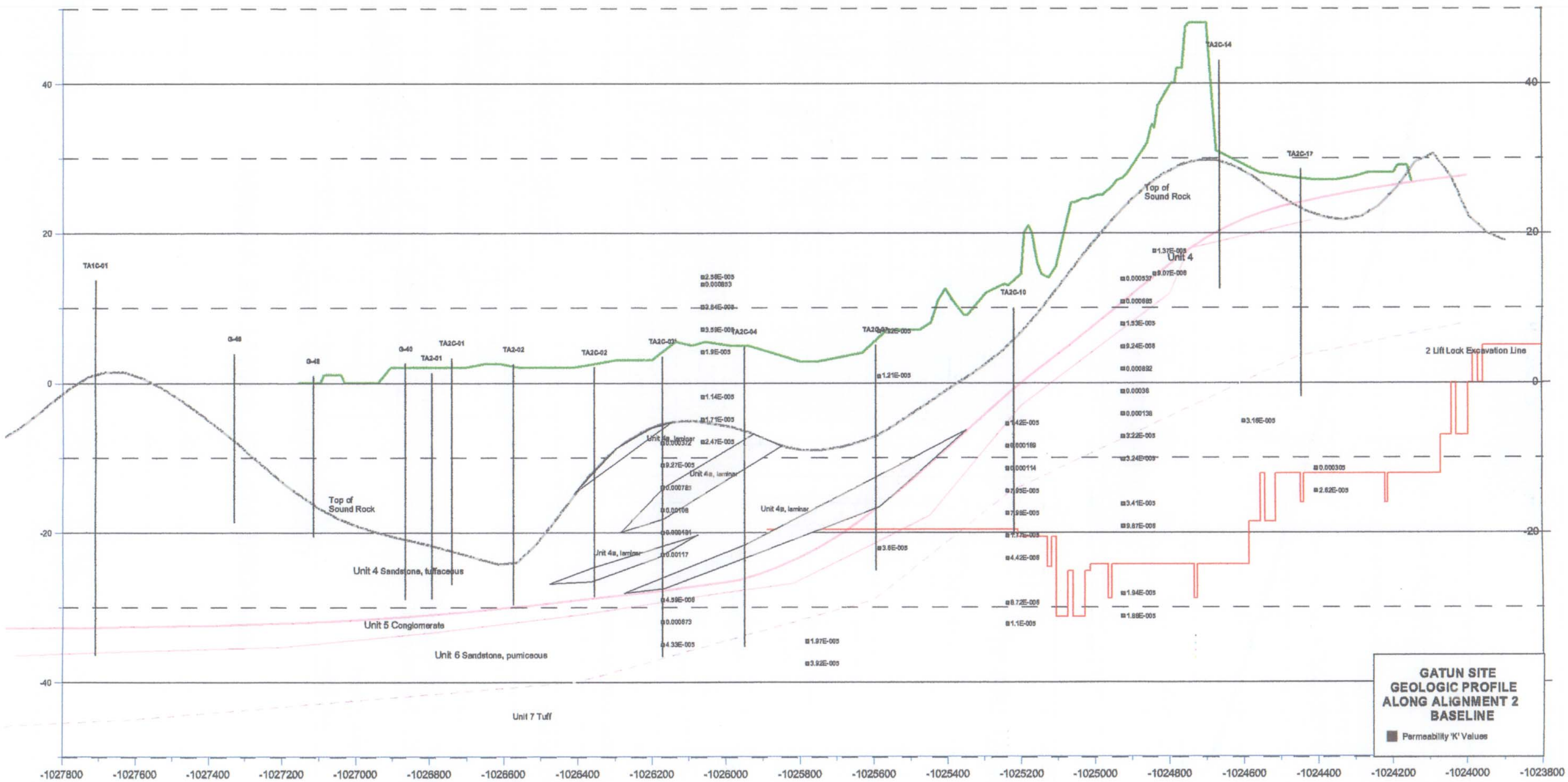
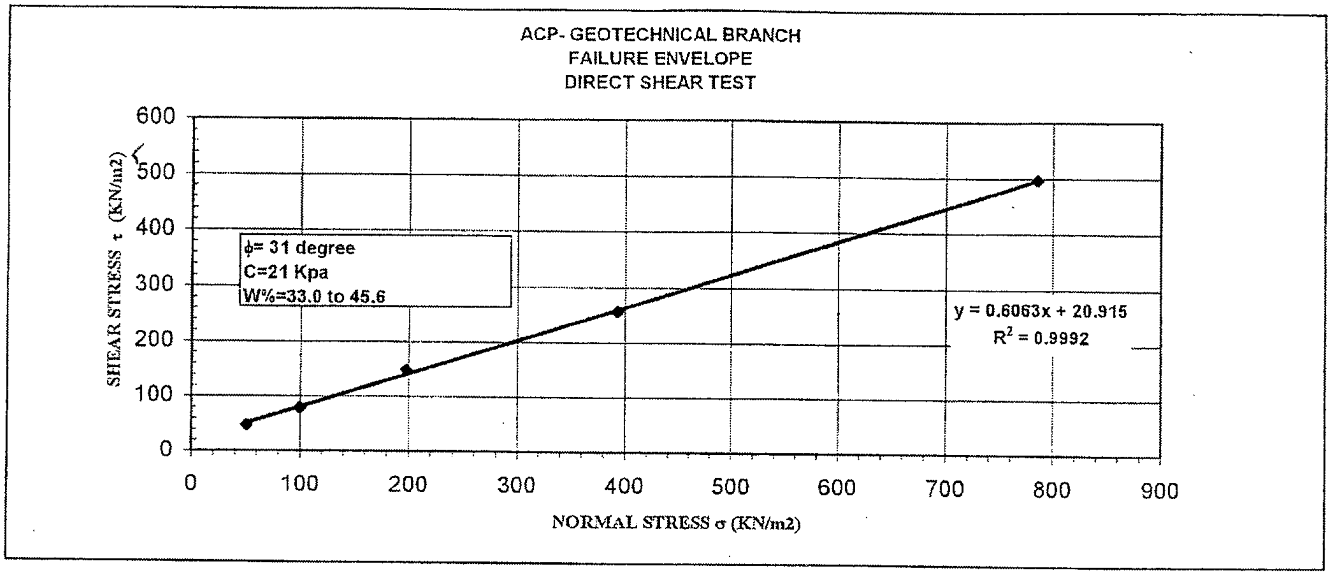
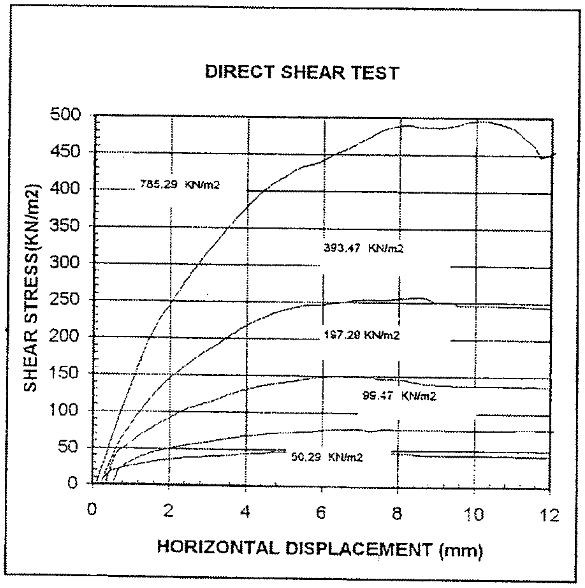

GATUN SITE
GEOLOGIC PROFILE
ALONG ALIGNMENT 2
BASELINE
Permeability 'K' Values
Top of Sound Rock
Unit 4
Unit 4 Sandstone, tuffaceous
Unit 5 Conglomerate
Unit 6 Sandstone, pumiceous
Unit 7 Tuff
2 Lift Lock Excavation Line

Figure GG-9





TEST No	1	2	3	4	5
INITIAL TEST					
Water Content, %	45.6	45.5	44.5	44.3	41
Dry Density, pcf	73.3	77	76.7	78	77
Saturation, %	107	117	112	115	106
Void Ratio	1.1	0.93	0.95	0.9	0.92
AT TEST					
Water Content, %	41.7	40.6	41.7	35.2	33
Dry Density, pcf	81.2	87	81	87	88.8
Saturation, %	121	136	121	117	115
Void Ratio	0.81	0.72	0.81	0.72	0.68
NORMAL STRESS (kPa)	50.29	99.47	197.29	393.47	785.3
SHEAR STRESS (kPa)	47.8	78.9	149.4	256.5	496
STRAIN (mm)	6.5	7.8	6.7	8.6	10.2
SPEED RATE mm/ min	0.003	0.003	0.003	0.003	0.003
DIAMETER (mm)	63.3	63.3	63.3	63.3	63.3

PROJECT: Third Locks
LOCATION: Gatun
BORING: TA2-4
DEPTH: 28.60m @ 30.0m
FORMATION: Gatun (tuffaceous sandstone)
DESCRIPTION (UCS): SM silty sand
COLOR: Gray
SAMPLE TYPE: Remolded
TESTED BY: G. GUERRA 6/7/01

INDEX PROPERTIES

LL= 48 LP= 34 IP= 14

Gs= 2.3 sieve No. 200= 34%

Clay Fraction (passing sieve No. 10)

0.005mm= 14% 0.002mm= 10%

Water content= 33.0 to 45.6%

Shear Strength Parameters

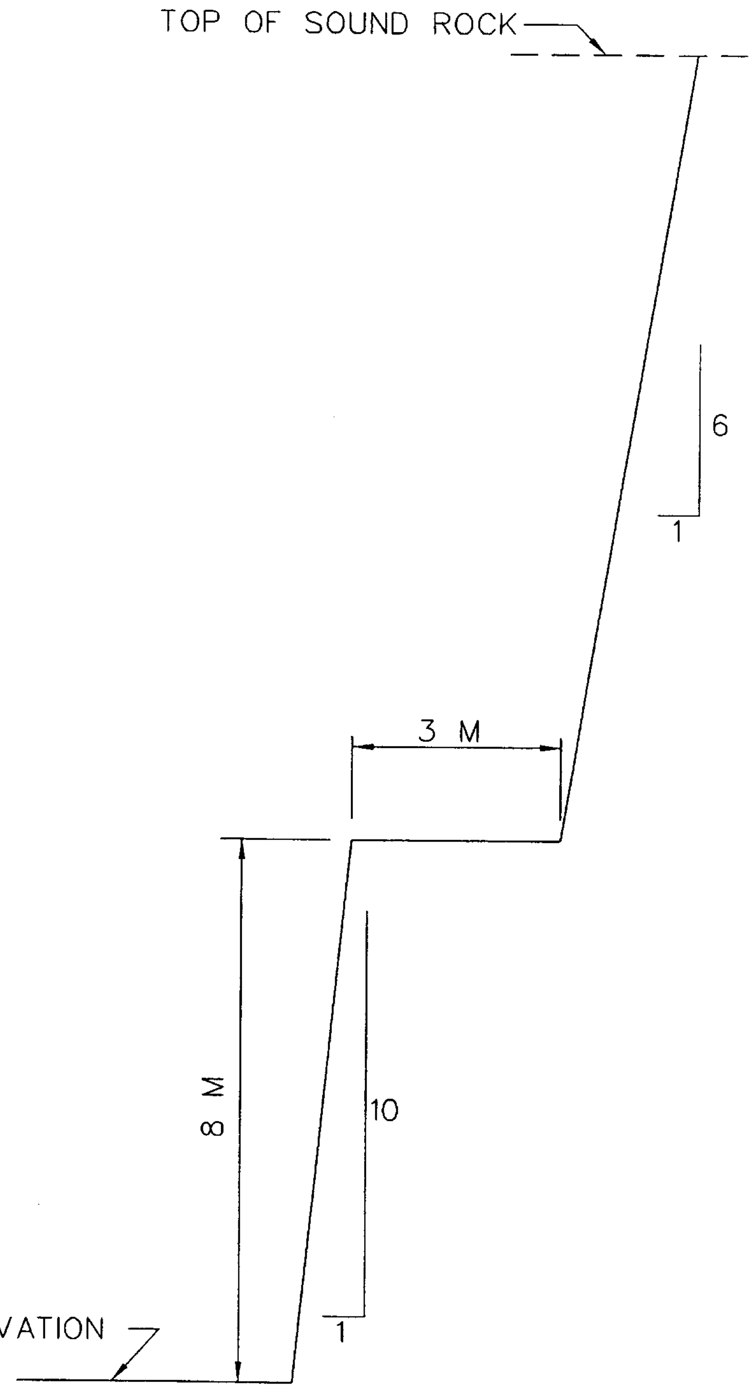
C peak= 21 Kpa

ϕ peak= 31 degree

DIRECT SHEAR TEST REPORT

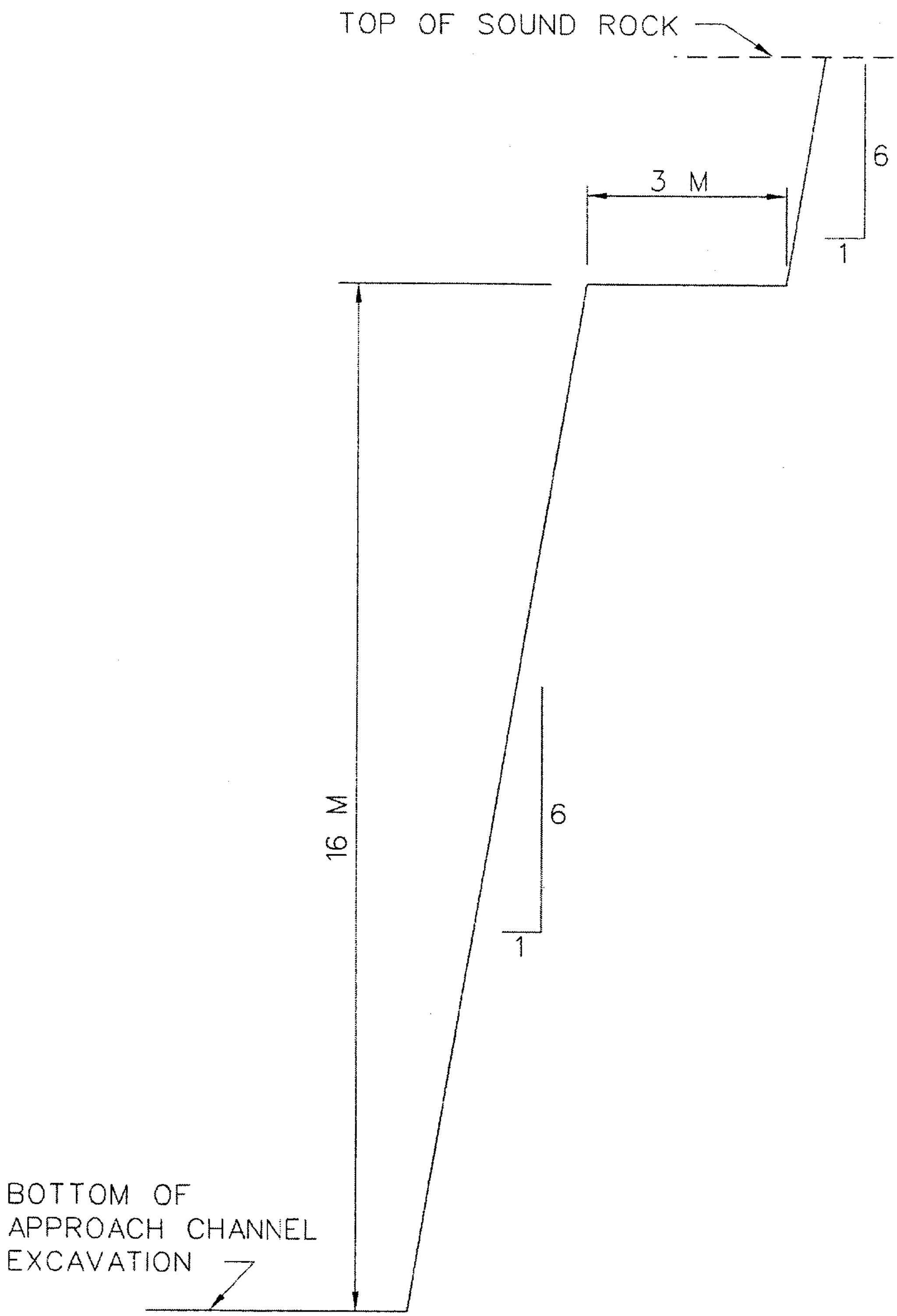
AUTORIDAD DEL CANAL DE PANAMA

Figure GG-10



RECOMMENDED ROCK CUT SLOPE DESIGN
WITHIN LOCK EXCAVATION LIMITS

FIGURE GG-11



RECOMMENDED ROCK CUT SLOPE DESIGN
WITHIN APPROACH EXCAVATION LIMITS

Figure GG-12

Appendix C – Geotechnical Investigations, Analyses and Designs

Part II – Geology

Exhibits

Exhibit C-II-1	Final Report on Modified Third Locks Project, Part II - Design, Chapter 5 – Foundations and Slopes, December 1943

UNITED STATES

THE PANAMA CANAL
THE THIRD LOCKS PROJECT

FINAL REPORT ON
MODIFIED THIRD LOCKS PROJECT

PART II - DESIGN

CHAPTER 5 - FOUNDATIONS AND SLOPES

DECEMBER 1943

DEPARTMENT OF OPERATION AND MAINTENANCE
SPECIAL ENGINEERING DIVISION
BALBOA HEIGHTS, CANAL ZONE

№ . . 77

and facility of excavation. They have been determined to be suitable for foundation purposes under the design maximum unit load of 20 tons per square foot. This fact has been substantiated by studies of the degree of stability of the existing Gatun Locks, which were built with comparable permissible-load assumptions. Recent studies have indicated that no detectable settlement has taken place in the portions of the lock structure that rest directly upon the strata of the Gatun formation.

Foundation Studies

5-63. In 1938 a study was made to correlate the foundation rock at the site of the existing Gatun Locks with the rock at the site of the new locks. Core borings between the existing locks and the location of the new locks permitted the accurate subsurface location of the traces of the beds common to both sites and established that the foundation strata for the new locks would be the same as those that have proved satisfactory under similar conditions at the existing locks.[12] A large amount of field and laboratory testing was done on the foundation rocks in an effort to refine the information available from records of the original construction.

Allowable Bearing Capacity and Modulus of Elasticity

5-64. The allowable bearing capacity of the Gatun-formation rocks was set originally at 20 tons per square foot, and no special testing was considered necessary if bearing pressures were not to exceed this value.[13] However, a test pit was sunk into the rock, and field bearing tests were made on the rock in place to determine the ultimate bearing strength of the rock and also to determine if the material would show any tendency for slow deformation under constant load (plastic flow or consolidation).

5-65. The test pit was sunk at Station 135+30 on the axis of the new locks, and four horizontal drifts were excavated in different strata. Bearing tests were made in each drift, using circular bearing plates 7, 16, and 40 inches in diameter. Where possible, these plates were loaded to failure, after having sustained a constant pressure of 20 tons per square foot for several days. The field tests indicated that the average bearing capacity of the rock varied from about 150 to about 350 tons per square foot. They also showed that the modulus of elasticity of the rocks averaged about 375,000 pounds per square inch and that there was little, if any, tendency for continuous deformation under constant load.

5-66. Rock samples from the test pit were tested in the laboratory, with triaxial- and unconfined-compression tests and a special bearing test developed for this purpose. The special bearing test was made by loading a 3-inch-diameter plate that was placed on the surface of a sample about 10 inches in diameter and 12 inches deep. The sample was confined laterally by being grouted in a cylindrical steel container. The results of the field bearing tests and the triaxial- and unconfined-compression tests are shown in Figures 5-23, 5-24, and 5-25. The laboratory tests showed excellent agreement with the field tests in regard to moduli of elasticity and ultimate strengths. A complete summary was prepared of the tests made in the pit, with the substantiating data from the laboratory triaxial- and unconfined-compression tests.[14] Comparisons were made of the results of the field and laboratory bearing tests.[15]

5-67. Based on field and laboratory tests, values of 20 tons per square foot for the allowable bearing capacity and 375,000 pounds per square inch for the modulus of elasticity were used in the design of the new lock structures at Gatun.[16]

5-68. The tests of the Gatun rocks provided extremely valuable information relative to the general correlation of field and laboratory tests. Very good checks of the modulus of elasticity were obtained by assuming a Poisson's ratio of 0.5, which is equivalent to assuming no volume change during the test. Furthermore, ultimate bearing capacities observed in the field tests checked very well with values calculated from the laboratory test results.

5-69. Clearance studies of the existing locks have indicated that the chamber wall faces are as much as 4 inches inside of the theoretical wall design line. The greatest encroachment occurs near the top of wall, and successively smaller encroachments are encountered toward the bottom. It was considered that this encroachment might be due to tilting as a result of slow movement, or plastic flow, of the rock under the toe of the walls. Therefore in the bearing-test program a constant load of 20 tons per square foot was applied to the 40-inch-diameter plate for a period of approximately five months, but there was no evidence of plastic flow under such conditions. Furthermore, comparative analyses of the vertical profiles showed that the center wall of the existing locks was inside the designed prism line in both east and west lock chambers; therefore the supposed tilt of the walls was undoubtedly caused by yielding of the forms during construction.

Sliding

5-70. In the investigation of the stability of the lock wall sections, it was necessary to estimate the resistance of the rock foundations to the horizontal forces exerted by the backfill. The shearing strength of the rock, as determined in the laboratory by

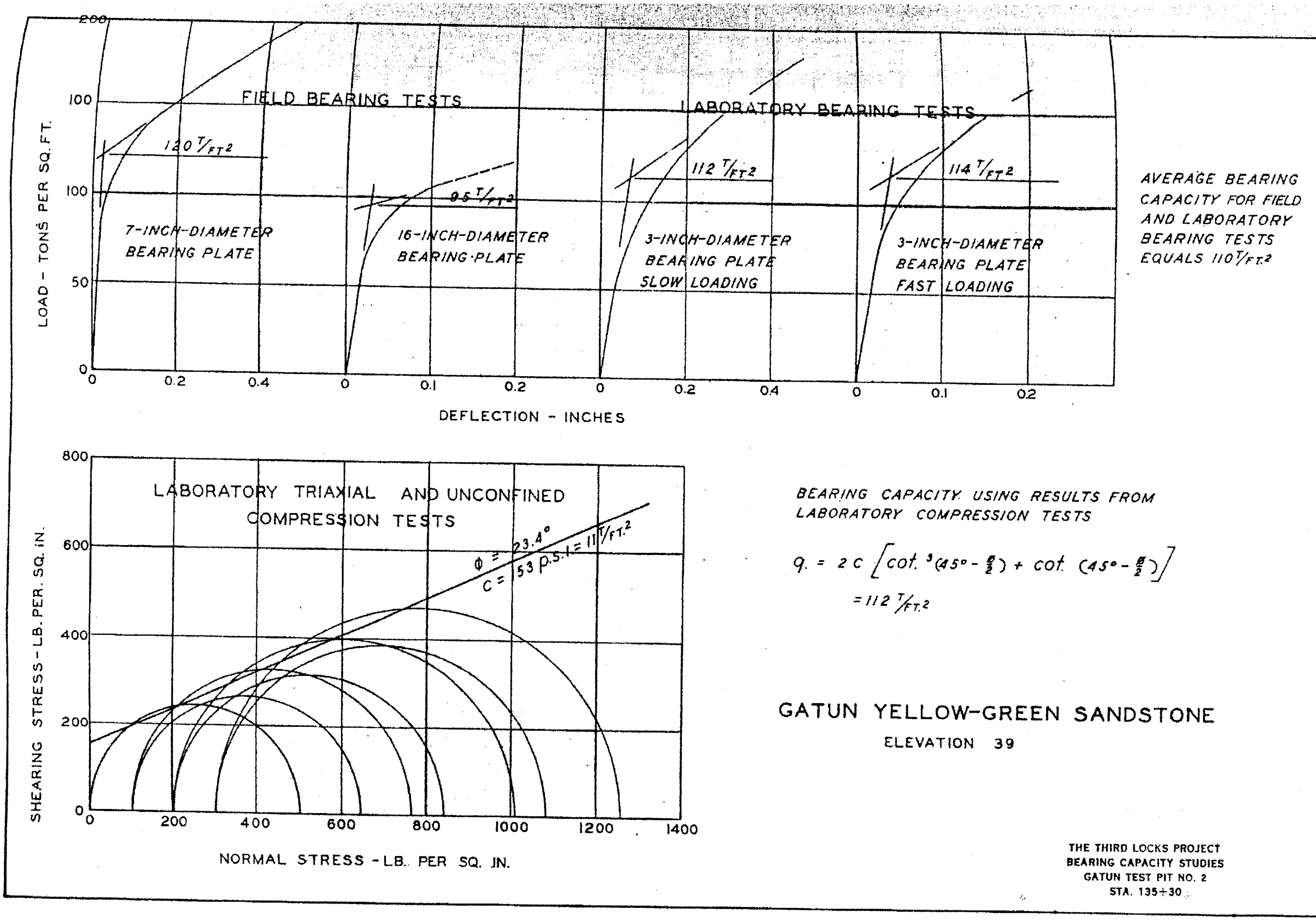
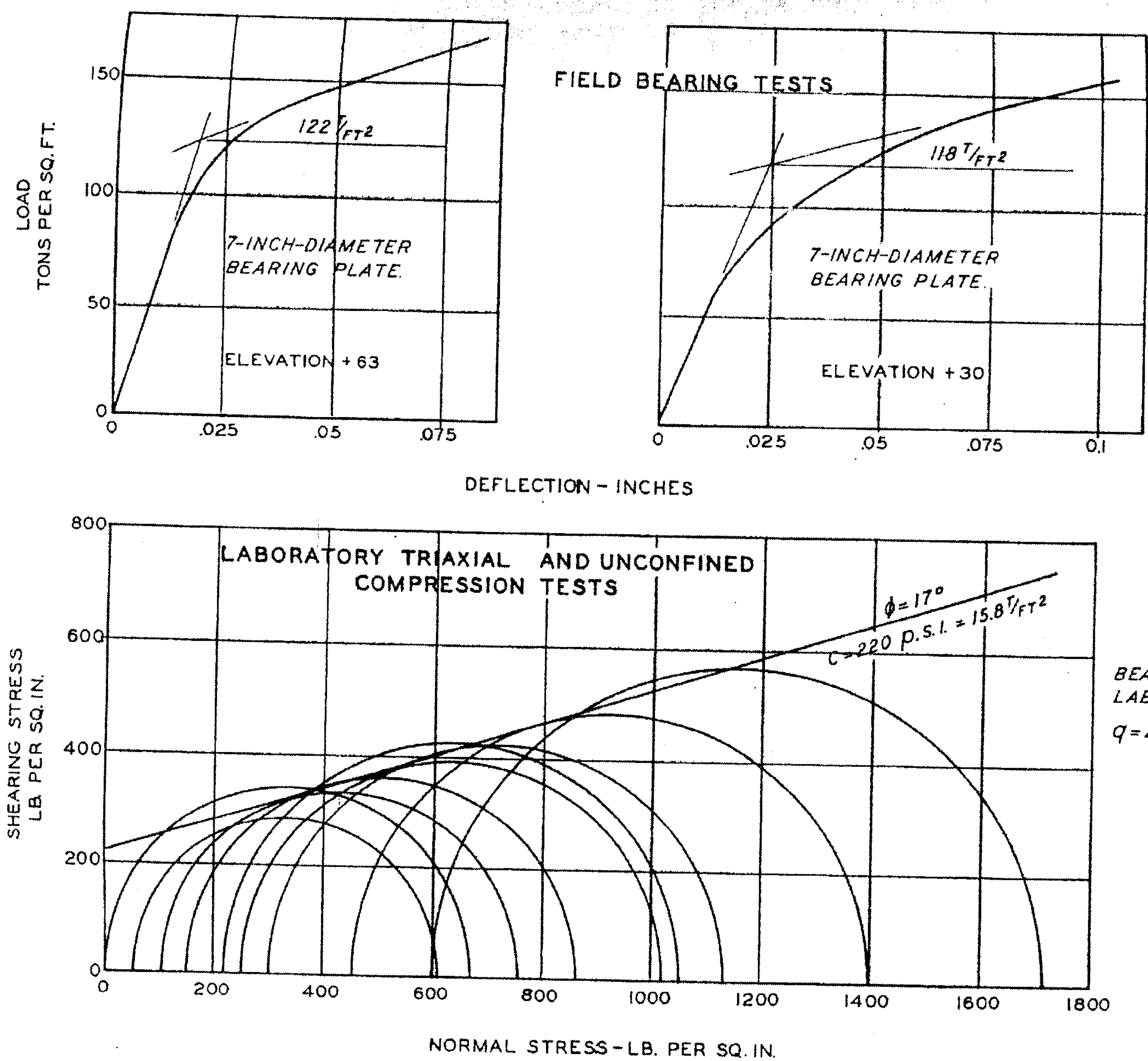

FIELD BEARING TESTS
LABORATORY BEARING TESTS
LOAD - TONS PER SQ. FT.
120 T/FT2
95 T/FT2
112 T/FT2
114 T/FT2
7-INCH-DIAMETER BEARING PLATE
16-INCH-DIAMETER BEARING PLATE
3-INCH-DIAMETER BEARING PLATE SLOW LOADING
3-INCH-DIAMETER BEARING PLATE FAST LOADING
DEFLECTION - INCHES
AVERAGE BEARING CAPACITY FOR FIELD AND LABORATORY BEARING TESTS EQUALS 110T/FT.2
LABORATORY TRIAXIAL AND UNCONFINED COMPRESSION TESTS
SHEARING STRESS - LB. PER. SQ. IN.
Φ = 23.4°
C = 53 P.S.I = 11 T/FT.2
NORMAL STRESS - LB. PER SQ. IN.
BEARING CAPACITY USING RESULTS FROM LABORATORY COMPRESSION TESTS
q. = 2 c [cot.³(45° - Φ/2) + cot (45° - Φ/2)]
= 112 T/FT.2
GATUN YELLOW-GREEN SANDSTONE
ELEVATION 39
THE THIRD LOCKS PROJECT
BEARING CAPACITY STUDIES
GATUN TEST PIT NO. 2
STA. 135+30

FIGURE 5-23



AVERAGE BEARING CAPACITY FOR FIELD BEARING TESTS EQUALS 120 T/FT2

BEARING CAPACITY USING RESULTS FROM LABORATORY COMPRESSION TESTS.

$$q = 2c\left[\cot^3\left(45^\circ - \frac{\phi}{2}\right) + \cot\left(45^\circ - \frac{\phi}{2}\right)\right] = 121\ T/FT^2$$

GATUN GRAY SANDSTONE

THE THIRD LOCKS PROJECT
BEARING CAPACITY STUDIES
GATUN TEST PIT NO. 2
STA. 135+30

FIGURE 5-24

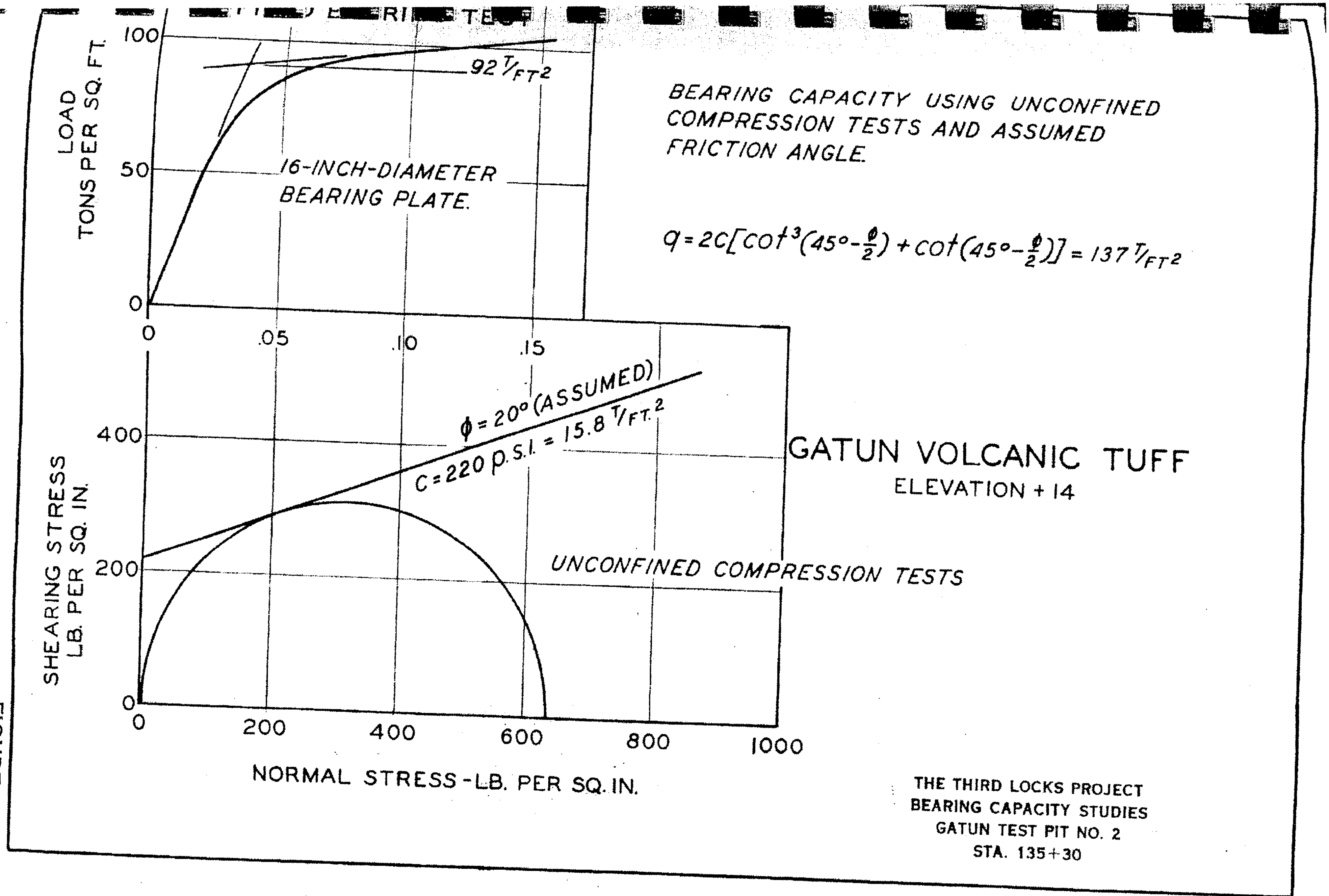

LOAD TONS PER SQ. FT.
100
50
0
0
.05
.10
.15
92 T/FT2
16-INCH-DIAMETER BEARING PLATE.
BEARING CAPACITY USING UNCONFINED COMPRESSION TESTS AND ASSUMED FRICTION ANGLE.
$q = 2c[\cot^3(45° - \frac{\phi}{2}) + \cot(45° - \frac{\phi}{2})] = 137$ T/FT2
SHEARING STRESS LB. PER SQ. IN.
400
200
0
0
200
400
600
800
1000
NORMAL STRESS - LB. PER SQ. IN.
φ = 20° (ASSUMED)
C = 220 p.s.i. = 15.8 T/FT.2
GATUN VOLCANIC TUFF
ELEVATION + 14
UNCONFINED COMPRESSION TESTS
THE THIRD LOCKS PROJECT
BEARING CAPACITY STUDIES
GATUN TEST PIT NO. 2
STA. 135+30

FIGURE 5-25

triaxial- and unconfined-compression tests, was used as a basis for determining the constants for use in the following formula[17] to find the "shear-friction" factor, Q:

$$Q = \frac{KA + FV}{H}$$

where

A = area of base of wall

H = resultant horizontal load on the wall

V = resultant vertical load on the wall

K = cohesion of the rock

F = friction coefficient of the rock

The values for K and F for the Gatun sandstone, as determined from laboratory tests and used in design, were 250 pounds per square inch and 0.28, respectively.[18]

5-71. To provide a check for these values, direct-shear tests were made at the site of the new Gatun Locks on an area of undisturbed rock. For these tests 12-inch concrete cubes were cast on prepared surfaces of the rock. The cubes were sheared at the joint between the concrete and the rock after the application of vertical loads ranging from 5 to 20 tons per square foot. Two types of joint surfaces were prepared: A smooth surface and a grooved surface. For the smooth surfaces, the cohesion value, K, was 60 pounds per square inch, and the friction coefficient, F, was 0.78. For the grooved surfaces, K was 110 pounds per square inch and F was 0.58. Since the field tests on grooved surfaces represent more closely the conditions in the foundation of the lock walls, the values of K = 110 and F = 0.58, as found from the field tests, were recommended for use in determining the shear-friction factors for the lock-wall design.[19] This recommendation was made subsequent to the completion of the lock-wall design, but the revised factors were found to affect the stability criteria only at the lock-chamber walls between gate bays. The results of the direct-shear tests were very consistent, and in view of the fact that very few lock features vital to the operation of the locks occur in the wall sections involved, it was considered satisfactory to reduce the safety factor of Q to 3.0 from the standard 4.0 used elsewhere.

5-72. The field shear tests and most of the laboratory tests were made on the dark-gray sandstone of the Gatun formation. The tests in Gatun Test Pit No. 2 on other rocks of the Gatun formation showed that they varied in strength only slightly from the dark-gray sandstone;[18] consequently the tests for the determination of the shearing strength of the dark-gray sandstone are considered to be applicable to all the foundation rocks of the new Gatun Locks.

Backfill

5-73. The first design of the lock walls was based on the assumption that the backfill behind the walls would be free-draining, permitting the assumption of a low water table behind the wall. The soft and friable character of the rock found in the core drilling led to some apprehension as to the assumption of a free-draining backfill. After excavation for the lock structure had begun, it was apparent that the rock broke down considerably upon handling, and it was decided to investigate the problem more thoroughly.

5-74. Experiments were made in the cut to determine the relative resistance to breakdown and the permeability of the fines resulting from breakdown. Considerable breakdown of the different materials took place in abrasion tests. It was found that the fines would be suitable for pervious backfill if handled properly.[20]

5-75. Using various methods of compaction, field tests were made on the rock from the excavation.[21] These tests showed that the method of compaction provided by the backfill specifications, which requires layers 1 foot thick, compacted by careful routing of loading equipment over the layers, will result in a dense, strong backfill. The dry density of the compacted material will be about 85 pounds per cubic foot with a moisture content of 30 per cent of the dry weight. However, the backfill will probably be quite impervious and may have a high water table. Accordingly, the lock walls were designed for a water table varying in a straight line from Gatun Lake level (elevation 87) at the upstream end to sea level at the lower end.

5-76. The angle of internal friction for the backfill was assumed as 32 degrees for use in obtaining the earth pressures. Tests on the fine material resulting from breakdown of the Gatun rocks indicated angles of approximately 32 to 33 degrees, and since the fines will be the weakest material in the mass of the backfill, the design assumption was verified as being satisfactory.[21]

Foundation Drainage

5-77. The thickness of the floor of the upper flight of the existing Gatun Locks was increased, and the floor was anchored by means of steel rails grouted into the foundation rock, so that it would resist full hydrostatic head. This precaution was taken since part of the floor covers a pervious stratum of tuffaceous sandstone, which might erode gradually and clog any weep holes used to relieve pressure in this stratum. However, weep holes were provided in the other chambers of the existing locks, where the rock was less pervious. The design for the new Gatun Locks provides weep holes for all chamber floors. These holes are to be from 5 to 15 feet deep, 2-1/2 inches in diameter, and filled with gravel. In order to check the adequacy of this design, two steps were taken: (1) Eight holes were drilled through the floor of the upper east chamber of the existing locks to determine the type of rock immediately below the floor and to check the pressures existing there, and (2) laboratory tests were made on the most pervious and loosely cemented rocks found in the excavation for the new locks. The drilling and pressure testing in the existing locks showed that a maximum pressure of about 10 pounds per square inch exists.[22] It was also found that the foundation of the upper part of the existing lock chamber consists of a yellow-green sandstone, which is a weathered phase of a blue-gray sandstone. This yellow-green sandstone, although found in the walls of the excavation for the new locks, is present in the floor only in a small area, and this small amount is of the less weathered, less pervious, lower part of the stratum. Since the maximum pressure and the maximum flow occurred in this sandstone in the existing locks, the new locks have a more favorable condition.

5-78. Samples of the blue-gray sandstone (Bed 9) and of a porous, coarse-grained sandstone (Bed 6) were tested for permeability and resistance to erosion by sealing pieces about 6 inches long in a 6-inch pipe and forcing water through the rock under a pressure of 30 pounds per square inch (70-foot head). After several months, the effects of erosion or leaching were observed in these samples. The coefficients of permeability of the blue-gray sandstone and of the coarse-grained sandstone were about 1.0×10^{-5} feet per minute. The coefficient of permeability of the yellow-green sandstone was about [illegible] times greater.[10] (For description of Beds 6 and 9 refer to Paragraphs 5-47 and 5-54, respectively.)

5-79. Since the strata that caused apprehension in the floors of the existing locks are present in the floors for the new locks only in very small amounts, and since tests on rock from the floors of the new locks show that it is not subject to raveling by water flowing through it, the use of weep holes in the design for the new locks should cause no complications.