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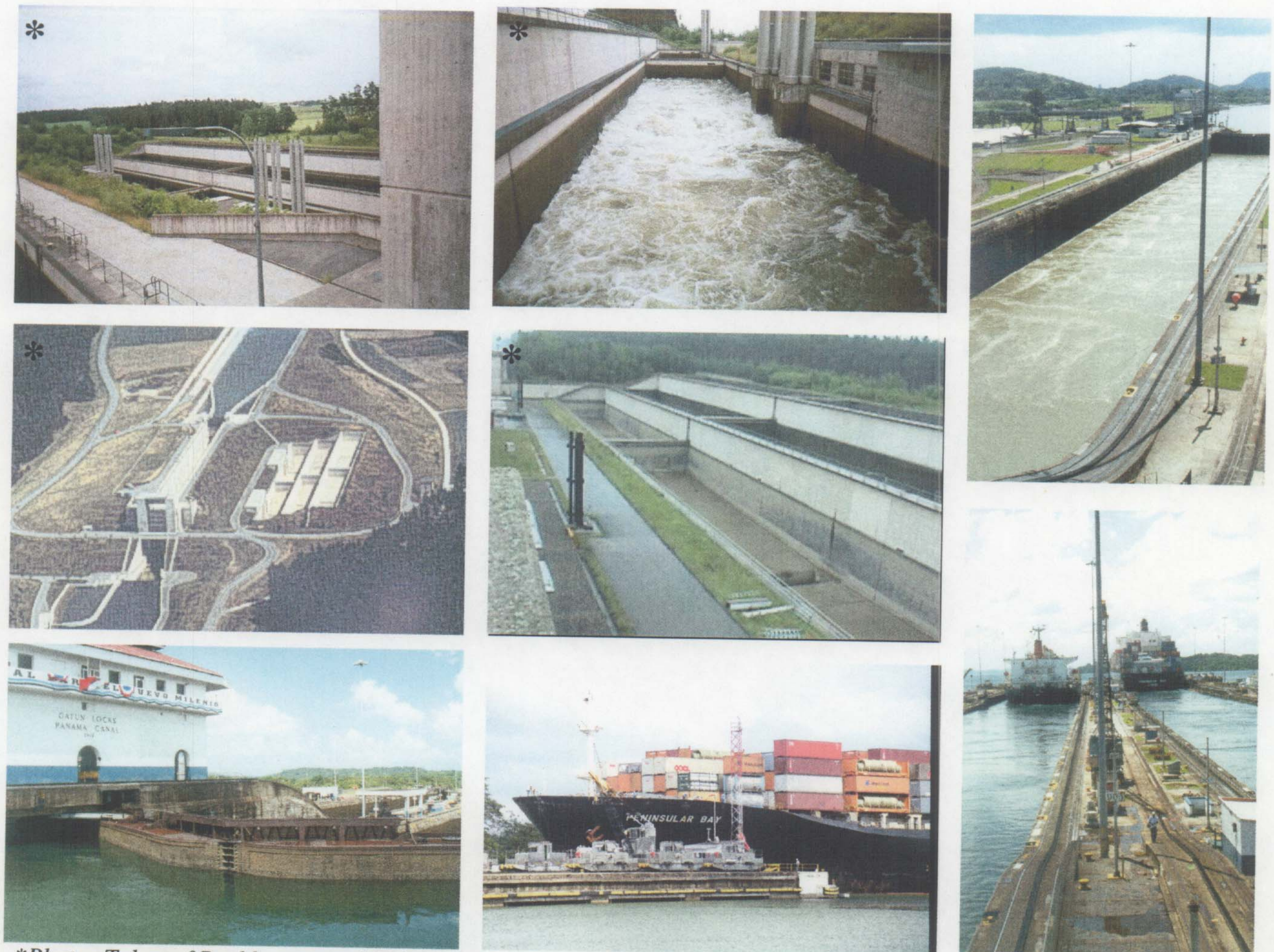
ACP
AUTORIDAD DEL CANAL DE PANAMÁ

CONCEPTUAL DESIGN STUDY OF LOCKS WATER SAVING BASINS FOR PROPOSED POST-PANAMAX LOCKS AT THE PANAMA CANAL

FINAL REPORT



*Photos Taken of Bachhausen and Hilpolstein Locks near Nurnberg, Germany (by INCA)

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EXECUTIVE SUMMARY

The Panama Canal Authority (ACP) is conducting a study of the Panama Canal to evaluate the feasibility of constructing facilities and features to augment the Canal's capacity and capability to transit vessels. The proposed locks (~61m x 457m x 18.3m– 200' x 1500' x 60') will be significantly larger than the existing locks (33.5m x 305m x 13m – 110' x 1000' x 43'). Therefore, the new larger locks will dramatically increase the water demands from Gatun Lake. The current lock facilities in addition to the municipal water consumption, hydropower generation, occasional spillage, and evaporation have caused water level changes of up to 2.5 m (9') in Gatun Lake. Therefore, a conceptual study for the design of water saving basin systems for the new locks was warranted to determine the feasibility of various water saving basin system options and the water saving gains which might be realized. The study options for the project were:

- OPTION 1 – Three-lift lock structure – side by side water savings basins to one side of lock – 50% water savings,
- OPTION 2 – Two-lift lock structure – side-by-side water savings basins to one side of lock – 60% water savings,
- OPTION 3 – Three-lift lock structure – side-by-side water savings basins on both sides of lock – 60% water savings, and
- OPTION 4 – Two-lift lock structure – stacked water savings basins on one side of lock – 50% water savings (a side-by-side basin arrangement was also studied).

As part of the study, a comprehensive data collection was completed along with a formulation of detailed design criteria. These criteria and design procedures were applied to determine basin and conduit layouts and associated sizes. These features were then conceptually designed hydraulically, structurally and geotechnically. Detailed breakdowns of opinions of probable costs were also completed.

An in-house spreadsheet model was created to complete the hydraulic analyses. This model was checked against the USACOE's LOCKSIM model and calibrated/verified to the existing locks with satisfactory results. A preliminary design of the lock Filling and Emptying (F/E) culverts was also completed to determine reasonable head loss estimates at the interface of the two systems and more importantly to determine the upper threshold of WSB conduit size (i.e., the WSB conduit should not be larger than the lock F/E culvert).

Hundreds of individual model runs were completed to create parametric curves which provided an opportunity to investigate "what-if" scenarios with a range of culvert sizes and arrangements. These curves were also plotted against the two most important design criteria which were equalization time and instantaneous maximum F/E rate. The explicit criteria for the lock F/E culverts were:

- the instantaneous maximum F/E rate should not exceed 2.28 m/min (7.5 ft/min) (the maximum for the existing locks with two culvert operations), and

- F/E times for a 3-lift system should be 8 – 9 min per lift (based on the existing system) and for a two-lift system, (3 lift x 8 – 9 = 24-27 min total)/2 lift = 12 – 13.5 min/lift. A factor of 3/2 was used to compute the F/E time for a two-lift system, assuming equal total operational times for the three-lift and two-lift systems. This factor was used only as a target in designing the preliminary F/E system.

In applying these criteria, the finalized lock F/E culvert sizes were found to be:

- Options 1 & 3 – Atlantic Side (8.84 m - 29'),
- Options 1 & 3 – Pacific Side (8.53 m - 28'),
- Options 2 & 4 – Atlantic Side (7.92 m - 26'), and
- Options 2 & 4 – Pacific Side (7.62 m - 25').

A comparative study, described in detail on pages 122-124 verified that vertical lift valves should be used for the WSB conduits due to faster equalization times and symmetrical behavior with bi-directional flow. Parametric curves were also created for the design of the water saving basin conduits. The design criteria for the WSB conduits were:

- the WSB conduits should not be larger than the preliminary F/E culvert sizes,
- no conduit solution should exceed an instantaneous maximum F/E rate of 2.28 m/min (7.5 ft/min) for basin to lock operations, and
- no conduit solution should have a single basin operation time of less than 2 minutes (which is the assumed shortest time needed to open and immediately close the valves).

Using these criteria, a myriad of solutions were available so methodologies were formulated to combine the results from the lock F/E culvert and the WSB conduit analyses to compute more meaningful statistics including total operation time, allowable transits/day, etc.

These statistics were submitted to ACP for review. The finalized WSB conduit arrangement and sizes chosen (for square conduits) by ACP were:

- Option 1 – 4 conduits/basin (6.10 m - 20'),
- Option 2 – 4 conduits/basin (7.32 m - 24'),
- Option 3 – 2 conduits/basin (8.53 m - 28'), and
- Option 4 (side-by-side basins) – 4 conduits/basin (6.71 m - 22')

ACP's selection of the number and sizes of conduits for the above options is based upon the desire to obtain a range of price scales for the different options. Therefore, the conduit selections are not necessarily the optimum for each option, but will provide a range of price options from most to least costly.

At this point, work was halted by ACP before the WSB conduit size could be finalized for the stacked basin arrangement for Option 4. The main reason for the work stoppage was that ACP now had new, revised alternative arrangements to be studied. ACP desired that as much of the remaining fees be applied to the new work order as possible so ACP directed the project team to stop all work on the current contract. The contract amount was reduced in accordance with this directive. A preliminary analysis using the same conduit size as that selected for the side-by-side basins indicated that the performance characteristics of the stacked WSB would be similar. However, these preliminary reviews also indicated that further refinement would be needed to size the conduits for the stacked basin arrangement since the equalization times were approximately 15% longer when compared with the side-by-side basin arrangement.

Based on the finalized conduit sizes selected by ACP, it is expected that the conduits will need to be bifurcated in order to accommodate more manageable, reliable, and likely less costly valves. Therefore, the valve recess to each conduit will house two main control valves and four closure bulkhead recesses (recesses will be located both upstream and downstream of control valves).

Throughout the conceptual design process, it became quickly apparent that two of the most important influences on the size of water saving basins and conduits required was the range of water levels (both lake and ocean) and lockage lengths (426.7 m - 1400', 457.2 m - 1500', and 487.7 m - 1600') for which the systems should be designed. This is especially important on the Pacific Ocean side where the tide range can exceed 7 meters. These variations had significant impacts, especially on the basin wall heights required for the theoretical water savings percentage to be achieved under all conditions. If the results of this study show that ***water saving basin systems*** that would accommodate the full range of water level and lockage length variations are not economically justifiable, the systems could be re-designed under a narrower range of hydrologic (see percent exceedance data in **Appendix C**) and hydraulic conditions (426.7 m - 1400', 457.2 m - 1500', and 487.7 m - 1600' lockage lengths) which may significantly reduce the conduit sizes as well as the basin wall heights.

During the hydraulic design, it was also discovered that the design of a stacked basin arrangement with basins only on one side of the lock is problematic because the range of water levels and lockage lengths necessitate an "overlap" between basins if the theoretical water saving percentage is always to be realized (for a more detailed explanation on "overlap", see pg. 79). Nonetheless, the problem can be overcome by increasing the width of the basins to a value greater than that of the locks (m > 1.0). However, this entails additional excavation for the upper locks and higher costs. Therefore, in future studies, a stacked arrangement with basins on both sides of the lock would be a superior configuration based on hydraulic consideration, although it would undoubtedly be more costly. Having basins on both sides of the lock will allow for the basins on one side to be offset from those on the other side (which will better accommodate the necessary "overlap").

The results of the hydraulic analyses are summarized and compared for all options (in both metric and English units) in the following tables.

System Layout and Theoretical Water Savings Percentages for all Options

OPTION	# Lifts	# WSBs per Lift	# Conduits per WSB	Conduit Diameter	Theroretical Water Savings Percentage
Option 1	3	2	4	6.10 m (20')	50%
Option 2	2	3	4	7.32 m (24')	60%
Option 3	3	6 (half size)	2	8.53 m (28')	60%
Option 4	2	2	4	6.71 m (22')	50%

Overall Water Usage and F/E Times per Lockage for Atlantic Side Options

Metric Units

OPTION	Water Intake Height Without Basins (m)			Water Intake Height With Basins (m)			Water Intake Volume With Basins (Avg. Lockage = 61 m x 457 m) ($10 m^3$)			Average Total F/E Time per Lockage (min)
	min	mean	max	min	mean	max	min	mean	max	
Option 1	7.01	8.49	10.04	3.51	4.24	5.02	97.71	118.27	139.85	31.46
Option 2	10.48	12.73	15.00	4.19	5.10	6.00	116.74	142.06	167.38	26.52
Option 3	7.01	8.49	10.04	2.80	3.40	4.02	78.17	94.65	111.98	36.43
Option 4*	10.48	12.73	15.00	5.24	6.37	7.50	145.97	177.58	209.18	26.35

* side-by-side basins

English Units

OPTION	Water Intake Height Without Basins (ft)			Water Intake Height With Basins (ft)			Water Intake Volume With Basins (Avg. Lockage = 200' x 1500') (million gal)			Average Total F/E Time per Lockage (min)
	min	mean	max	min	mean	max	min	mean	max	
Option 1	23.00	27.85	32.93	11.50	13.92	16.46	25.81	31.24	36.94	31.46
Option 2	34.37	41.78	49.23	13.74	16.72	19.70	30.83	37.52	44.21	26.52
Option 3	23.00	27.85	32.93	9.20	11.14	13.18	20.65	25.00	29.58	36.43
Option 4*	34.37	41.78	49.23	17.18	20.90	24.62	38.55	46.90	55.25	26.35

* side-by-side basins

Overall Water Usage and F/E Times per Lockage for Pacific Side Options

Metric Units

OPTION	Water Intake Height Without Basins (m)			Water Intake Height With Basins (m)			Water Intake Volume With Basins (Avg. Lockage = 61 m x 457 m) ($10 m^3$)			Average Total F/E Time per Lockage (min)
	min	mean	max	min	mean	max	min	mean	max	
Option 1	6.10	8.46	11.17	3.05	4.23	5.58	84.96	117.93	155.66	34.72
Option 2	9.12	12.70	16.69	3.65	5.08	6.67	101.62	141.55	186.07	27.58
Option 3	6.10	8.46	11.17	2.44	3.38	4.47	67.97	94.31	124.56	37.28
Option 4*	9.12	12.70	16.69	4.56	6.35	8.35	127.11	177.07	232.63	27.55

* side-by-side basins

English Units

OPTION	Water Intake Height Without Basins (ft)			Water Intake Height With Basins (ft)			Water Intake Volume With Basins (Avg. Lockage = 200' x 1500') (million gal)			Average Total F/E Time per Lockage (min)
	min	mean	max	min	mean	max	min	mean	max	
Option 1	20.01	27.77	36.64	10.00	13.92	16.46	22.44	31.15	41.11	34.72
Option 2	29.91	41.67	54.77	11.96	16.72	19.70	26.84	37.39	49.15	27.58
Option 3	20.01	27.77	36.64	8.00	11.14	13.18	17.95	24.91	32.90	37.28
Option 4*	29.91	41.67	54.77	14.96	20.90	24.62	33.57	46.77	61.44	27.55

* side-by-side basins

The opinions of probable costs estimates for three of the four options can be seen in the following tables.

Opinions of Probable Costs (Conceptual Level) for Atlantic Side Options

OPTION	Civil/Structural Construction Costs (U.S. Dollars, in Millions)	Mechanical Item Costs (U.S. Dollars, in Millions)	Electrical Item Costs (U.S. Dollars, in Millions)	**Total Costs (U.S. Dollars, in Millions)**
Option 1	$325	$30.0	$1.6	**$357**
Option 2	$360	$30.0	$1.6	**$392**
Option 3	$418	$23.7	$1.6	**$444**

Opinions of Probable Costs (Conceptual Level) for Pacific Side Options

OPTION	Civil/Structural Construction Costs (U.S. Dollars, in Millions)	Mechanical Item Costs (U.S. Dollars, in Millions)	Electrical Item Costs (U.S. Dollars, in Millions)	**Total Costs (U.S. Dollars, in Millions)**
Option 1	$380	$30.0	$1.6	**$412**
Option 2	$466	$30.0	$1.6	**$498**
Option 3	$547	$23.7	$1.6	**$573**

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I. PROJECT PURPOSE AND SCOPE

A. Introduction

The Autoridad Del Canal De Panama (ACP) is conducting a study of the Panama Canal to evaluate the feasibility of constructing facilities and features to augment the Canal's capacity and capability to transit vessels. As part of this work, ACP will soon begin a study for the conceptual design of new locks capable of transmitting Post-Panamax ships through the Panama Canal Area (see **Figure I.1**). The proposed locks (~61m x 457m x 18.3m– 200' x 1500' x 60') will be significantly larger than the existing locks (33.5m x 305m x 13m – 110' x 1000' x 43'), and with the addition of these new larger locks, water demands from Gatun Lake are expected to increase dramatically. In fact, ACP estimates that water volumes required by the new locks could approach 2.6 – 7.7 times the current lock volumes based on preferred new lock configurations. Therefore, the volumes of water moving through the system from Gatun Lake will be substantially more than with the current locks - even with water saving basins providing a reduction in the total water volumes required.

Figure I.1 - Panama Canal Area Location Map

B. Study Purpose

In light of the above, and coupled with the fact that operation of the current locks, municipal water consumption, hydropower generation, occasional spillage, and evaporation cause seasonal water level changes of up to 2.5 m (9') in Gatun Lake, a conceptual study for the design of new locks equipped with water saving basins was warranted. Based on other studies being completed by ACP, the expected range of water level changes will be reduced from 2.5 m to 1.83 m. For this conceptual study of the water saving basin systems, ACP commissioned Moffatt & Nichol Engineers (M&N) in association with INCA Engineers and Golder Associates.

C. Canal System Description

As stated above, the evaluation of the proposed Post-Panamax locks must address important issues such as water availability. Operating the new locks will create a future demand for water from Gatun, Madden, and Miraflores Lakes that must compete with future municipal, industrial and other demands from Panama's growing economy and population. With numerous studies already being carried out by ACP to identify new water supplies to supplement those already in use, it becomes crucial to make as wise use of these water resources as possible. Therefore, it is imperative that ACP has a detailed understanding of the water savings versus cost issues for various water saving basin systems alternatives so that informed decisions can be made about the best options available for further study and possible implementation. However, in order to begin the process, an understanding of the existing system is vital.

On the Atlantic side, Gatun Locks raise and lower ships between the Atlantic Ocean and Gatun Lake in three consecutive lifts. Lock operations are supplied using freshwater from Gatun Lake. The average tide range on the Atlantic side is approximately 0.2 m (0.7') while the maximum tide range is ~1 m (3.3'). The average water level in Gatun Lake is 25.9 m (85.0') above PLD (Precise Level Datum), but the lake level can vary between 23.9 m (78.5') to 26.7 m (87.5').

On the Pacific side, Miraflores Locks raise and lower ships between the Pacific Ocean and Miraflores Lake in two lifts using freshwater from Miraflores Lake (and ultimately, Gatun Lake). The average tide range on the Pacific side is approximately 3.8 m (12.6') while the maximum tide range is ~7.0 m (23.1'). The average water level in Miraflores Lake is 16.5 m (54.0').

The Pedro Miguel locks are used to raise or lower ships between Miraflores and Gatun Lakes using only freshwater from Gatun Lake. For plan and profile views of the existing system, see **Figures I.2** and **I.3**.

Operationally, the ACP runs uplockages (ocean to lake – raising the ship) and downlockages (lake to ocean – lowering the ship) in varying time increments as a function of transit scheduling. Therefore, the new locks and water saving basin system must be designed to work under a varying range conditions that can change from performing uplockages to downlockages very quickly.

Figure I.2 - Satellite Image of Panama Canal Area (Taken from CZ Brats Website)

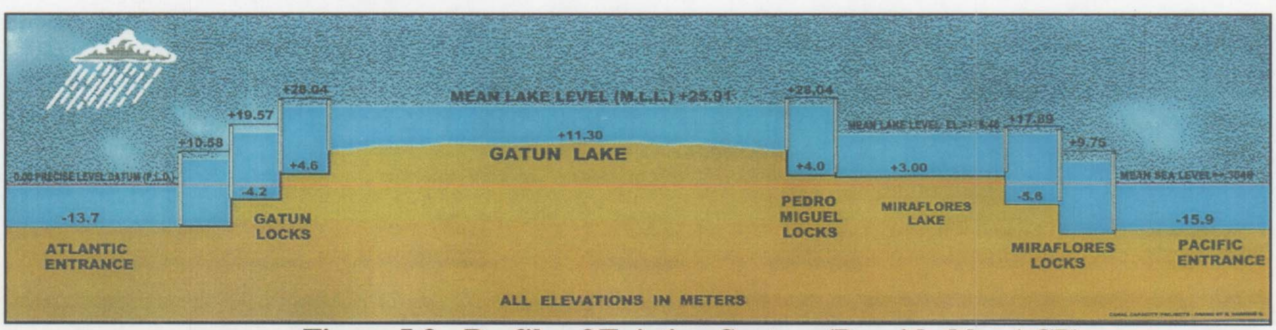


Figure I.3 - Profile of Existing System (Provided by ACP)

As one can see from the above range of lake and tide levels, the range of equalization levels within the locks and water saving basins will be related to hydrologic conditions in Gatun Lake and to tide elevations in the ocean. Consequently, the conceptual design of the locks water saving basins requires a detailed analysis that accounts for these differing water levels and their effect on equalization levels. Of particular concern will be the larger tide range on the Pacific side, which requires the new locks and water saving basins to operate and equalize under a much wider range of elevations than on the Atlantic side.

D. Water Saving Basin Operation – Conceptual Description

Consider the simplest case of a single-lift lock (see **Figure I.4**). For a ship to pass from the high level to the exit level, the lock is sequentially filled and emptied using source water from the high-level water body. Each locking operation consumes a volume of water equal to the lift times the surface area of the lock or in this case (100 – 0) * A = 100A.

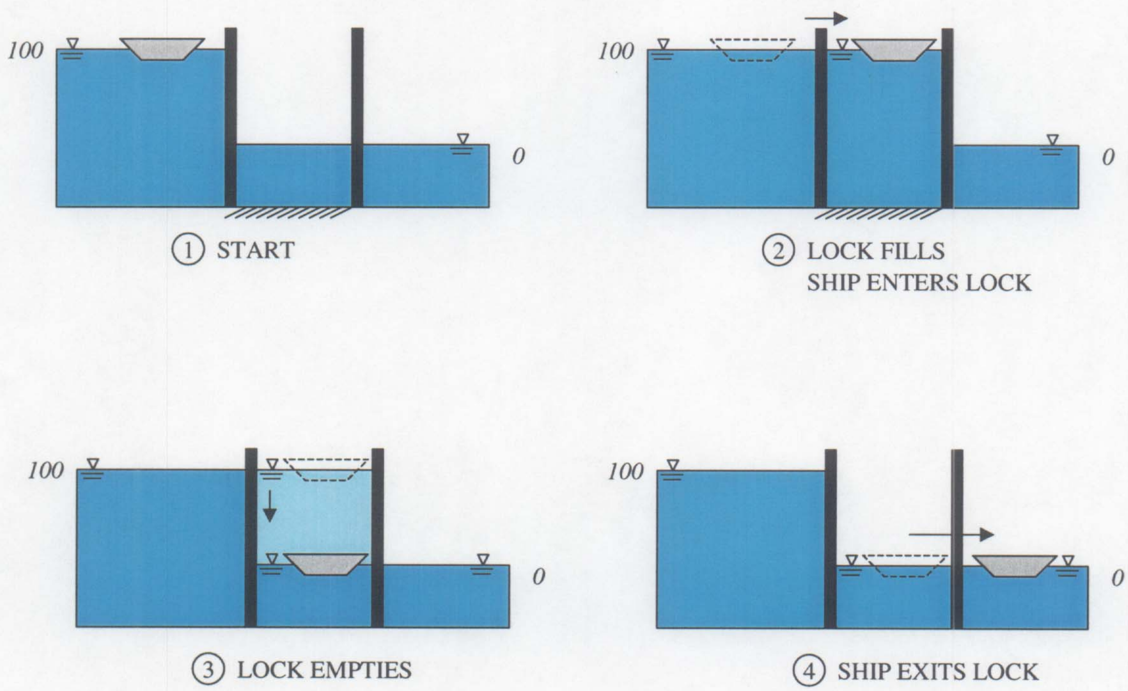


Figure I.4 – Typical Downlockage Operation for a Single Lift Lock

If conserving water becomes an important goal, then it is feasible to equip the lock with holding basins that are placed to either or both sides of the lock. The basins can be connected to the main lock filling/emptying system through a standard culvert and valve system such as is used for lock-to-lock operations. For simplicity, assume that the basins have the same surface area as the lock. Consider again the single lift lock already described above, now fitted with two water saving basins as shown in **Figure I.5**. For the system to function entirely under gravity flow, the geometry must be set so that the lift of the lock is vertically segmented into n+2 parts, where n is the number of basins. Therefore, in the example, the lock lift comprises 4 –25 unit slices.

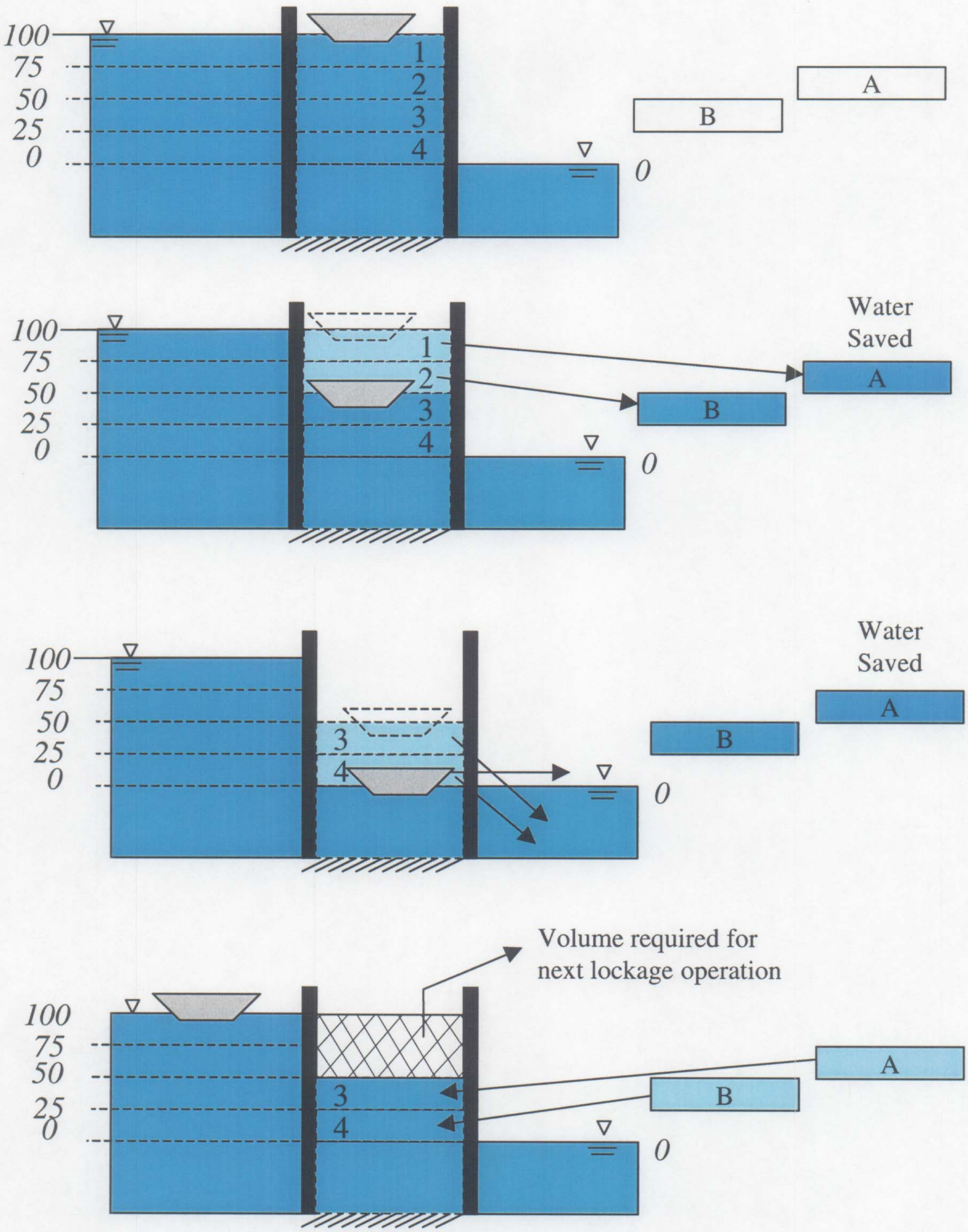


Figure I.5 – Typical Water Saving Basin Operation

In an emptying operation, the lock is drained sequentially into Basin A, followed by Basin B. This saves the lock water from Segments 1 and 2. Finally the water in Segments 3 and 4 is drained to the receiving water body to complete the emptying operation. When refilling the lock, the water in Basin B is drained to Segment 4, and then Basin A is drained to Segment 3. Segments 1 and 2 are filled using makeup water from the higher source water body. In this way, 50% of the volume of water is conserved. In general terms, it can be shown that the theoretical water savings is $n/(n+2)$ – (n is the number of basins) when the basin surface area is equal to the lock surface area. It follows then, that three basins would yield a potential savings of 60% (3/5), while four would save 67% (4/6).

E. Project Scope

The Project Scope considers several different lock and water saving basin configurations aimed at determining an optimized solution which maximizes water savings while minimizing costs. Specific tasks to be completed include:

- Project Work Plan and QA/QC Plan,
- Design Criteria,
- Features Layout and Design of Study Alternatives,
- Quantity Take-offs and Cost Estimates for Study Alternatives, and
- Associated Reports and Meetings Necessary to Complete the Work.

The alternative layouts to be included within the study were the following:

- Three-lift lock structure – side by side water savings basins to one side of lock – 50% water savings,
- Two-lift lock structure – side-by-side water savings basins to one side of lock – 60% water savings,
- Three-lift lock structure – side-by-side water savings basins on both sides of lock – 60% water savings, and
- Two-lift lock structure – stacked water savings basins on one side of lock – 50% water savings.

M&N was responsible for the overall project management, the conceptual hydraulic design of the new water saving basin systems, features layout and design including hydraulic analyses, and the determination of preliminary mechanical and electrical issues related to valving operations. INCA was responsible for the conceptual structural design of the new water saving basin systems, features layout and design (as related to structural issues), and the development of quantity take-offs and associated cost estimates for all alternatives. Golder Associates were responsible for reviewing existing geotechnical reports and studies to make preliminary, conceptual recommendations for foundation designs for the water saving basin systems and resolution of features layout and design related to geotechnical issues.

II. DATA COLLECTION

A. Reconnaissance Trip To Panama

As a first step in the study, a Project Kickoff Meeting was held at the Canal Capacity Projects Office (CCPO) of ACP during November 13-15, 2000. This meeting was informative and helpful as ACP gave a brief history of the Canal and a general overview of the project. ACP then led the Project Team on an extensive tour of the existing Gatun and Miraflores Locks. The filling/emptying (F/E) system of the locks was discussed as well as the lock miter gates and the electrical/mechanical operators which currently operate the lock gates and valves (see **Figures II.1-5**). Current limitations on vessel size were also discussed and observed as a Panamax cruise vessel went through a downlockage while the Project Team was present (see **Figure II.6**). Generalized geology of the Panama Canal Area was also presented and observed during a boat trip through Gaillard Cut (see **Figure II.7**).

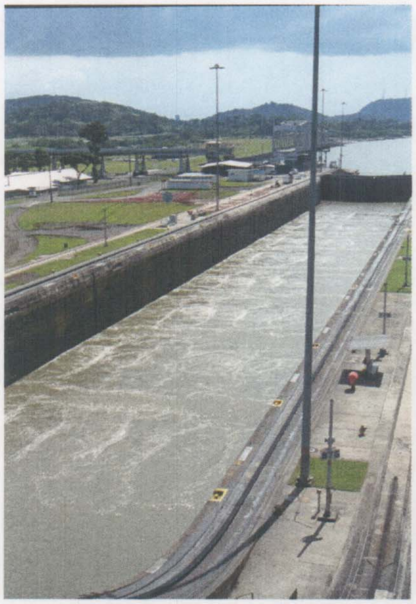


Figure II.1 – Lock F/E System

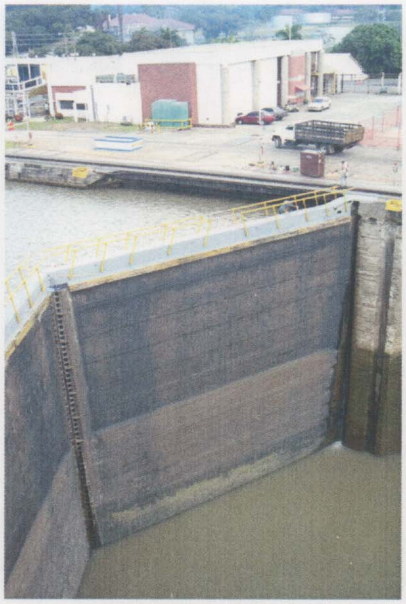


Figure II.2 – Lock Miter Gate

Figure II.3 – Control Room

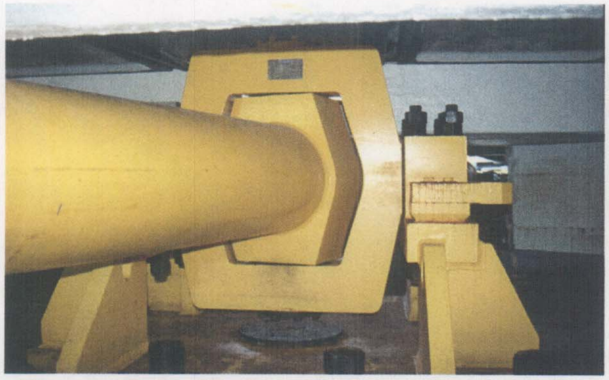


Figure II.4 – Hydraulic Operator for Miter Gate

Figure II.5 – Mechanical Operator for F/E Culvert Valve

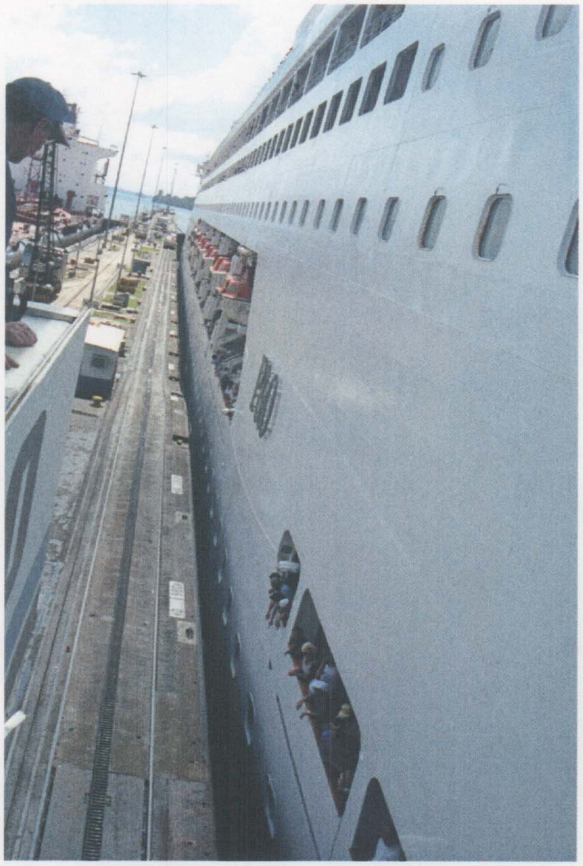


Figure II.6 – Panamax Cruise Vessel Transiting Gatun Locks

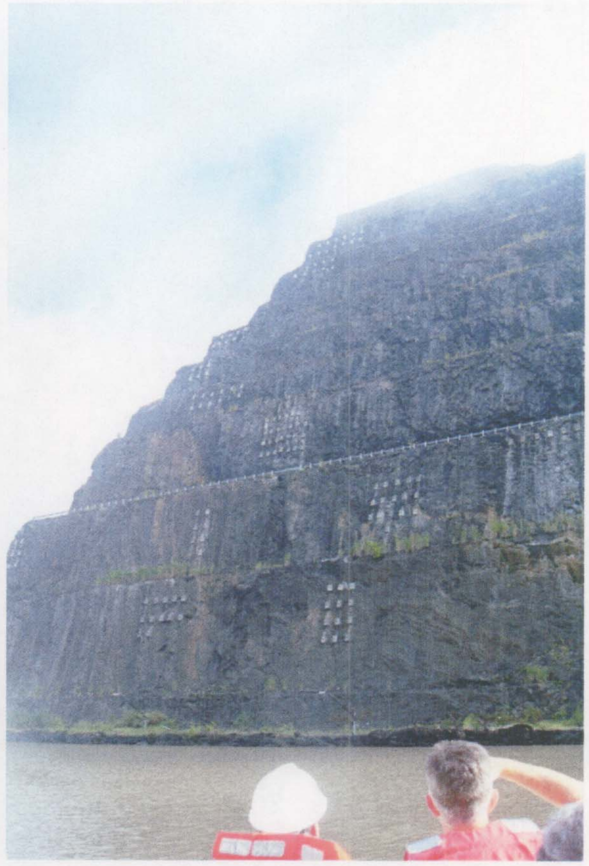


Figure II.7 – Geologic Formations at Gaillard Cut

After completion of the field trip, the Project Team presented the work plan and QA/QC plan for the project to ACP. ACP then provided technical and contractual points of contact for the project and discussed existing guidance in PIANC literature concerning the design of water saving basins. The four preliminary alternative lock and basin layouts were then discussed along with the preliminary alignments for the new locks. ACP then explained that the new locks on the Pacific side would tie directly into Gatun Lake rather than Miraflores Lake so variations in Miraflores Lake would not have to be considered in the design process. Existing datasets were provided to the Project Team by ACP. These datasets included:

- Existing Surveys and Geotechnical Data,
- Pertinent Sections of the Harza Report on Canal Future Alignments (see **Appendix B** for the Alignment Drawings),
- Historic Water Level Data for Gatun Lake, Atlantic Ocean, and Pacific Ocean,
- 1940's Hydraulic Model Study Report,
- Preliminary Profile of a Three-Lift Option Post-Panamax Lock Configuration, and
- ACP CADD Standards and Existing Drawings.

- Powerpoint Presentation by J. Wong Providing an Overview of How the Water Saving Basins Operate in Germany and How ACP Forsees They Will Operate in the New System.

B. Hydraulic Data Collection

1. Water Level Data

As stated above, one of the most important considerations in the conceptual design of the new locks water saving basin systems is the range of water levels (both lake and ocean) under which the systems will have to operate. In order to quantify this, water level data was analyzed for Gatun Lake as well as the Atlantic and Pacific Oceans.

a) Gatun Lake Water Levels

For Gatun Lake water level data, ACP provided daily measurements collected at midnight from 01/01/1966 – 10/18/2000 (see **Figure II.8**). From **Figure II.8**, the water level in Gatun Lake does have a seasonal pattern which corresponds directly to the rainfall wet and dry seasons. In most years the water levels in Gatun Lake range from 25.3 m (83.0') to 26.7 m (87.5'), but it is interesting to note that on occasion there have been some rather dramatic droughts which have lowered lake levels significantly. The range of the entire dataset varies from a high of 26.8 m (87.97') to a low of 23.9 m (78.55').

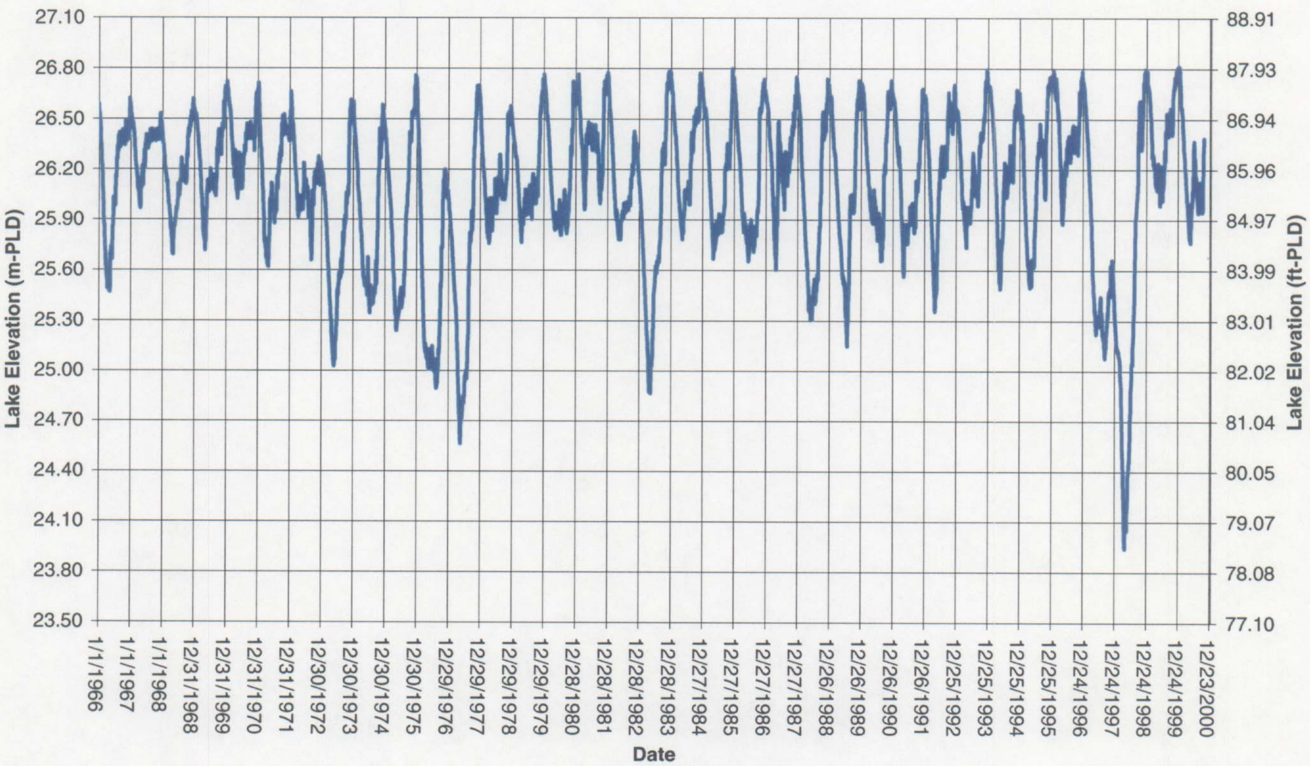


Figure II.8 – Gatun Lake Daily Water Level Measurements (Taken @ Midnight – 01/01/1966 thru 10/18/2000)

The exceedance distribution of lake levels is shown in **Figure II.9**. Tabulated exceedance statistics are in **Appendix C**.

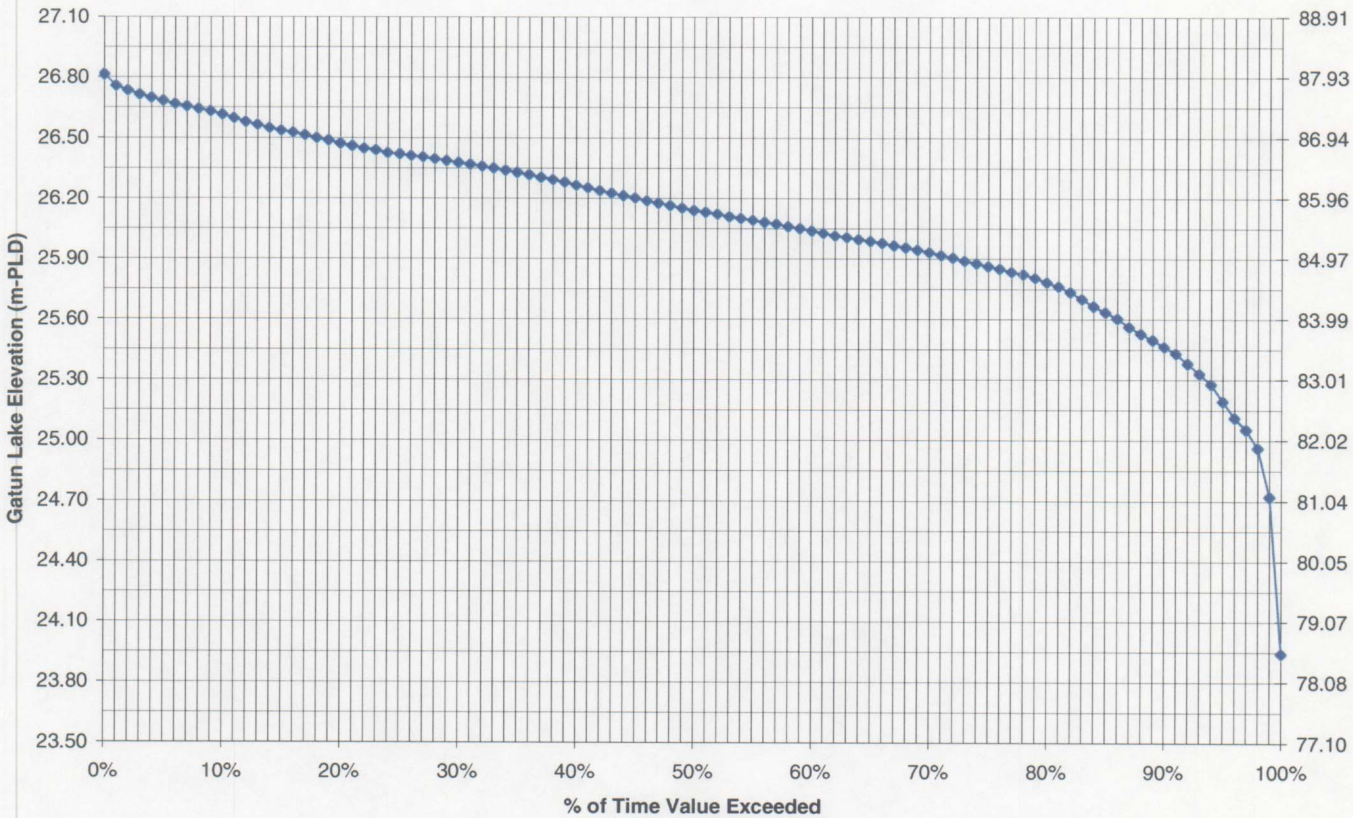


Figure II.9 – Finalized Percent Exceedance Distribution For Gatun Lake Levels

b) Atlantic Ocean Tide Levels

For the Atlantic Ocean tide data, ACP provided 15-minute measurements collected at Coco Solo within Limon Bay (see **Figure II.10**). Altogether, the measured data series includes tide levels for a period of 10 years (01/01/89 through 12/31/98). Since a long-term data series would be needed to capture the full range of expected tide elevations, predicted tide data for the nearby station at Cristobal (see **Figure II.10**) was computed for the 1978 tidal epoch (1960-1978). This tidal epoch was chosen since the current tidal datums near the site are based on it. If the predicted and measured tide data matched well, the predicted data for the tidal epoch could be used to calculate our percent exceedance distribution. For computation efficiency and accuracy, comparisons between the datasets were made on hourly measurements.

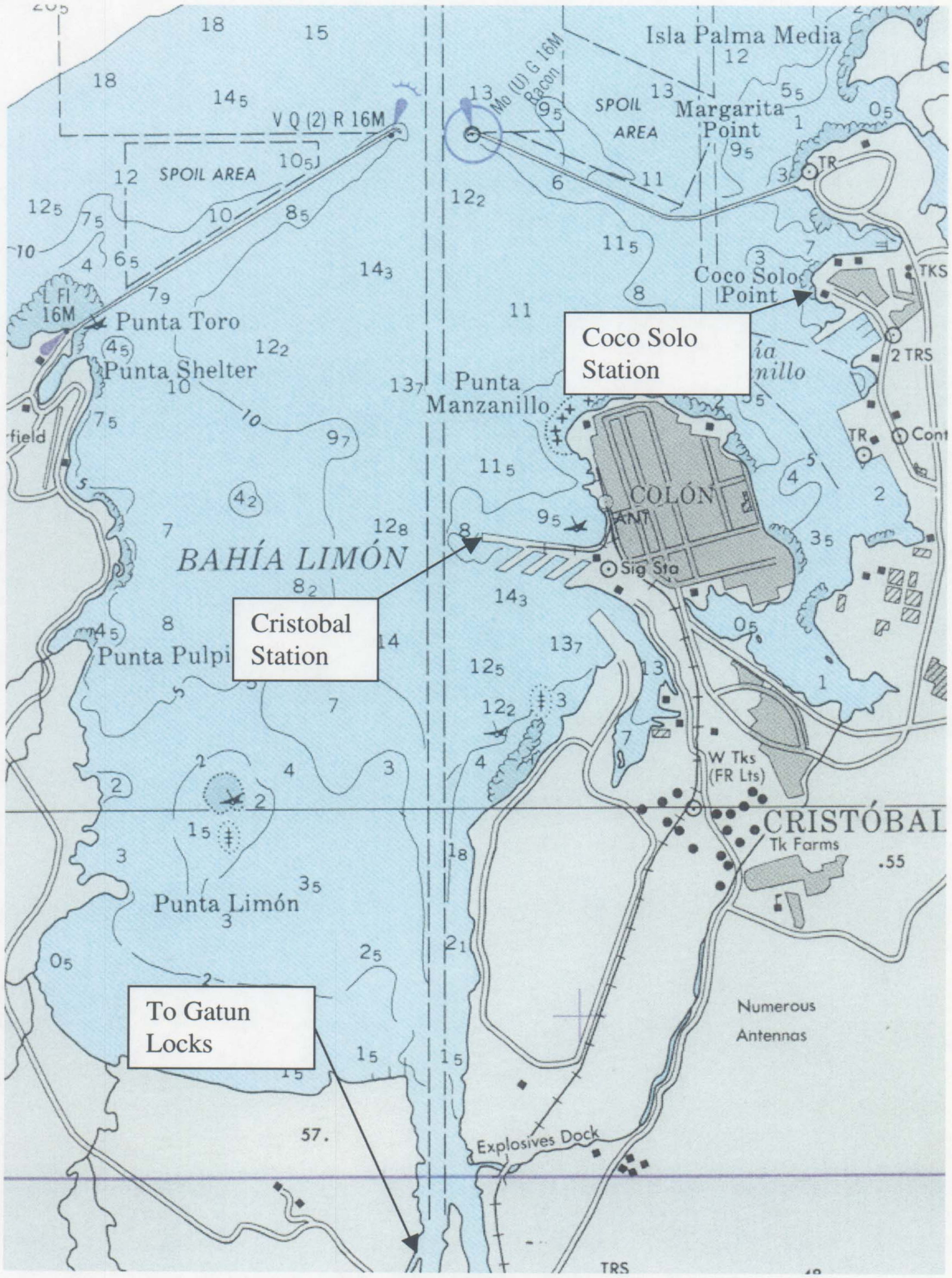


Figure II.10 – Atlantic Ocean Tide Station Location Map

As one can see from **Figure II.11** below, the ranges of the measured and predicted tide data do match very well. This is not surprising as the average tide range on the Atlantic side is very small (0.2 m – 0.7') and tide ranges should be very similar throughout the region. However, there does seem to be a datum problem since the mean tide elevations do not match (~0.20m). Nonetheless, when looking at the location of Coco Solo in relation to Cristobal (see **Figure II.10**), one can readily see why the mean tide elevation would be higher at Coco Solo since Coco Solo is located off the main bay in a narrow reach. The area near Coco Solo is also shallower and rougher which would also tend to lead to a higher mean tide elevation. Even with this datum problem, the tide ranges matched very well, and since Cristobal is located on the main portion of Limon Bay which then leads directly to Gatun Locks, using the Cristobal station dataset for the percent exceedance distribution was justified.

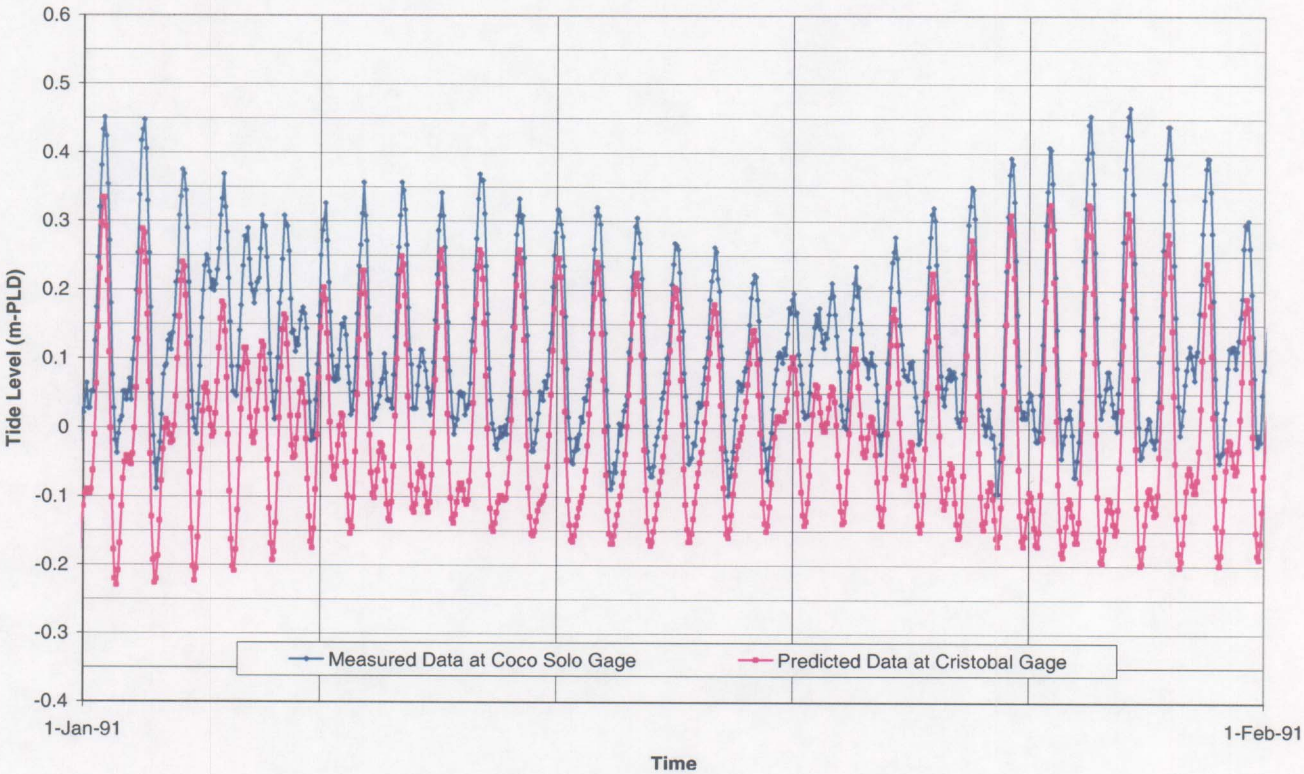


Figure II.11 – One Month Comparison of Measured Versus Predicted Tide Levels - Atlantic Side

As a secondary check of using the predicted Cristobal tide dataset, a comparison of the percent exceedance distributions for both the measured and predicted tide data was completed for the period of time that the measured data was collected. **Figure II.12** shows the comparison of the two distributions. As one can see from the **Figure II.12**, the graphs are very similar except the same datum shift problem is still present. However, since the range and distribution of relative values were very similar, this analysis reinforced the idea that the predicted tide data at Cristobal could be used to describe tide ranges near the new Atlantic locks.

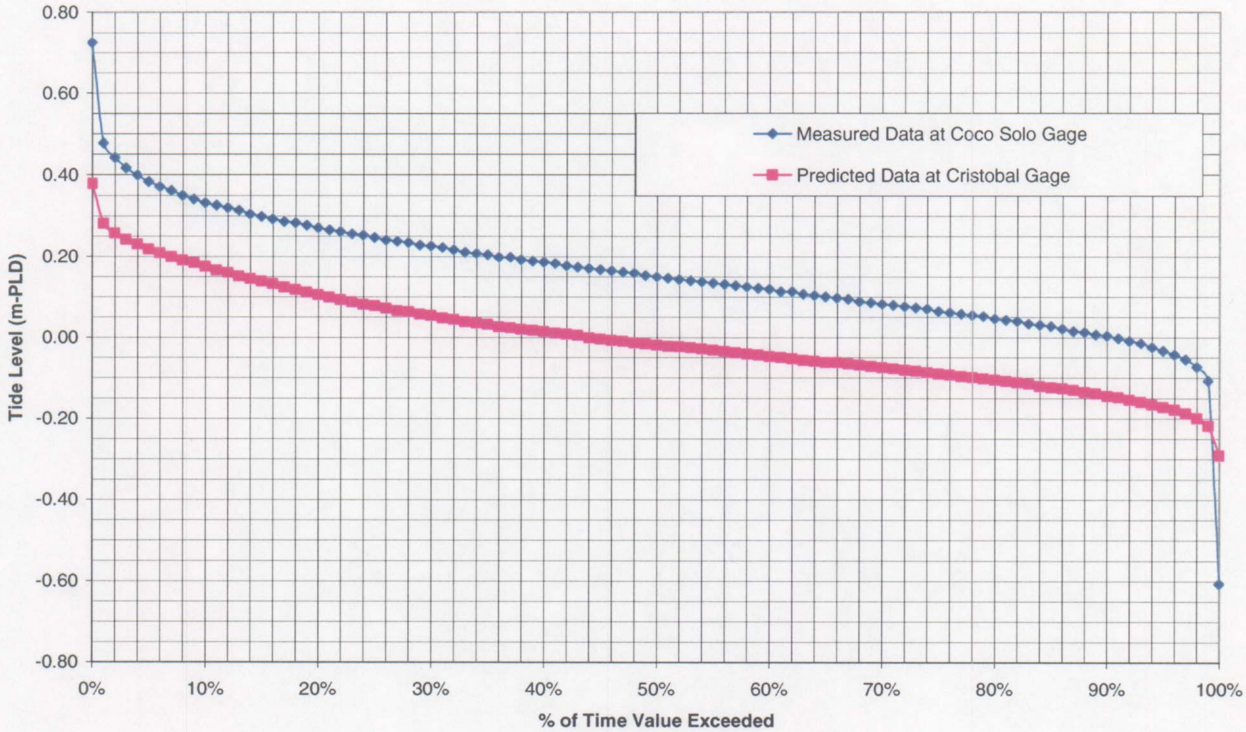


Figure II.12 – Comparison of Measured Versus Predicted Tide Level Percent Exceedance Distributions - Atlantic Side

Therefore, the next step of the process was to create a predicted tide series at Cristobal for the 1978 tidal epoch (1960-1978). This was done with hourly data, and a percent exceedance distribution was created. The last comparison to be made was then to compare the maximum and minimum predicted tide levels over the epoch to the extreme high and low tide elevations provided by ACP. The extreme high water level reported by ACP was 0.56 m (+1.85') while the extreme low water level was reported to be –0.38 m (-1.25'). In comparing these values to those predicted, it was found that the tidal extremes reported by ACP were on average 1.37 times greater than the predicted extremes.

There are several possible explanations for this behavior. For instance, these extreme highs and lows might have been missed by using a one-hour time step in the tidal prediction program. A more likely explanation perhaps, is that the tide station at Cristobal is seaward of the Gatun Locks, and one would expect to see some tidal amplification as one moves inland from the wider Limon Bay to the narrow channel near the locks. However, since the factors were relatively symmetric (on both the extreme high and low tide elevations), M&N decided to modify the predicted percent exceedance by multiplying each value times a linearly interpolated factor between the calculated high and low water level factors. The finalized percent exceedance distribution can be seen below in **Figure II.13**, while the data values can be seen in **Appendix C**.

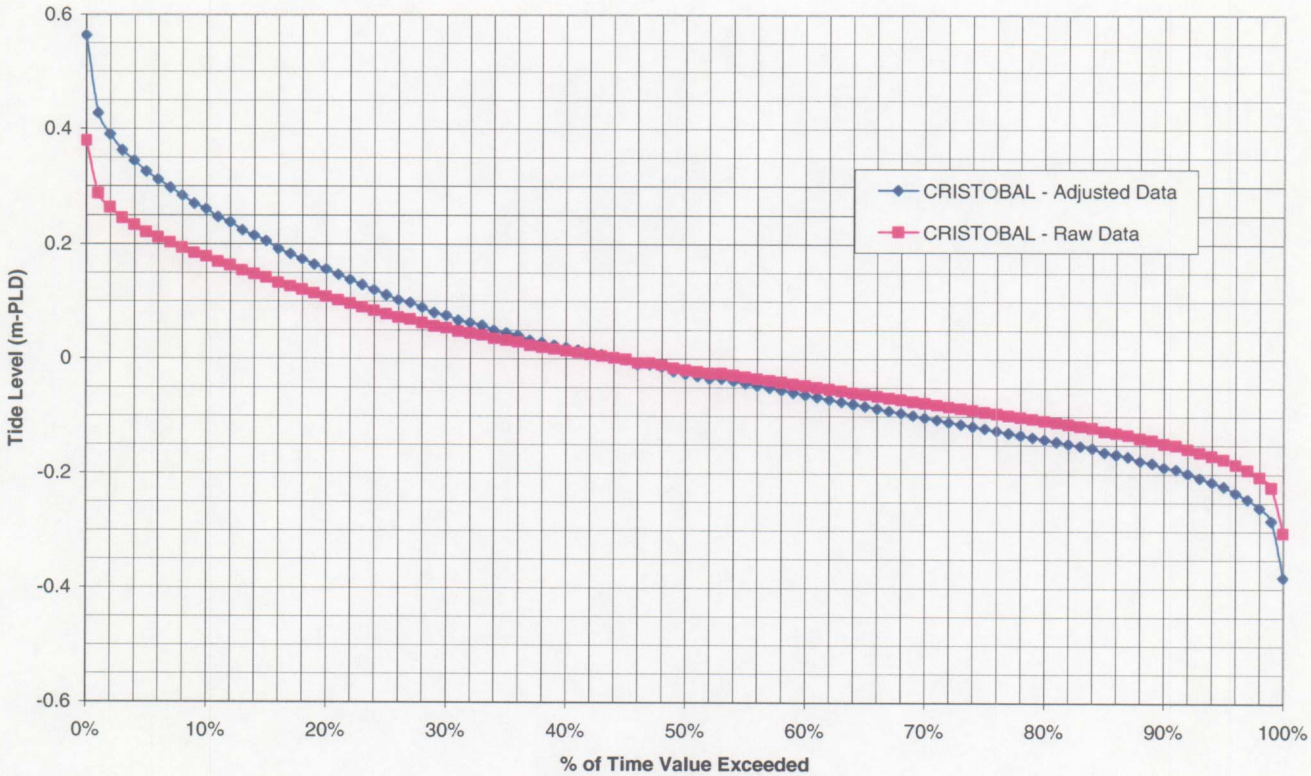


Figure II.13 – Finalized Percent Exceedance Distribution for Atlantic Ocean Tide Levels

c) Pacific Ocean Tide Levels

For the Pacific Ocean tide data, ACP provided 15-minute measurements collected at Diablo Heights (see **Figure II.14**). Altogether, the measured data series includes tide levels for a period of 8 years (01/01/91 through 12/31/98). For a long-term data series, the closest predicted tide station was located at Balboa (see **Figure II.14**). The same comparison and analysis procedures followed for the Atlantic Ocean tides were followed for the Pacific Ocean tides. As one can see from **Figure II.15**, the measured and predicted tide data (both ranges and elevations) match very well. This was somewhat surprising as the average tide range on the Pacific side is very large (3.8m – 12.6') and tide ranges can vary significantly throughout the region. Nonetheless, considering the fact that both stations are located near the main shipping channel where amplification and roughness variations should be minimal, and are fairly close together, it is not unexpected that the tide ranges and elevations should be similar.

As before, after the raw data comparison had been made, comparisons between the predicted and measured tide level percent exceedance distributions were made for the period of record. **Figure II.16** shows the comparison of the two distributions, and it is evident that the percent exceedance distributions are almost identical and that the predicted tide data distribution at Balboa could be used to describe tide ranges near the new Pacific locks.

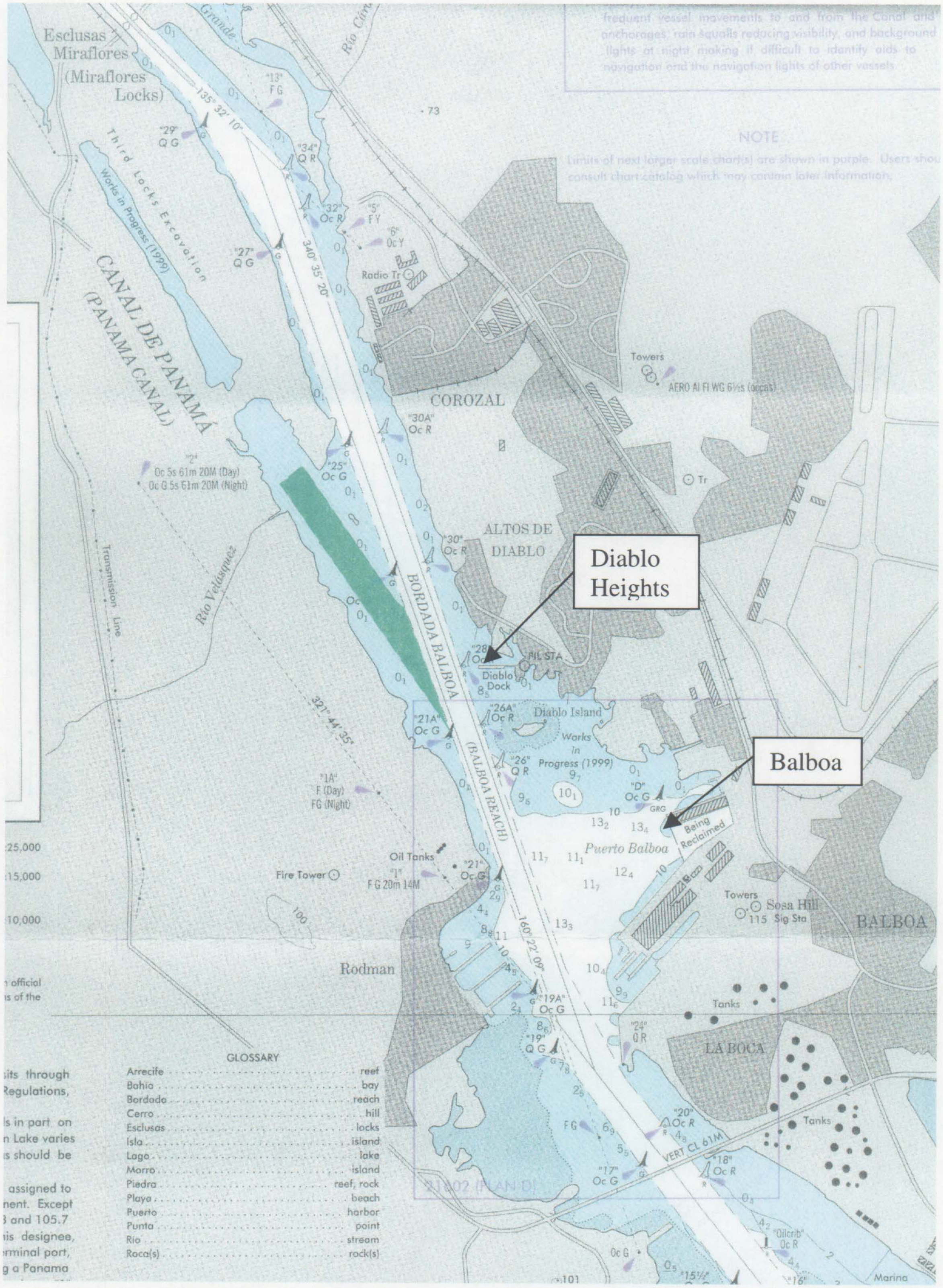


Figure II.14 – Pacific Ocean Tide Station Location Map

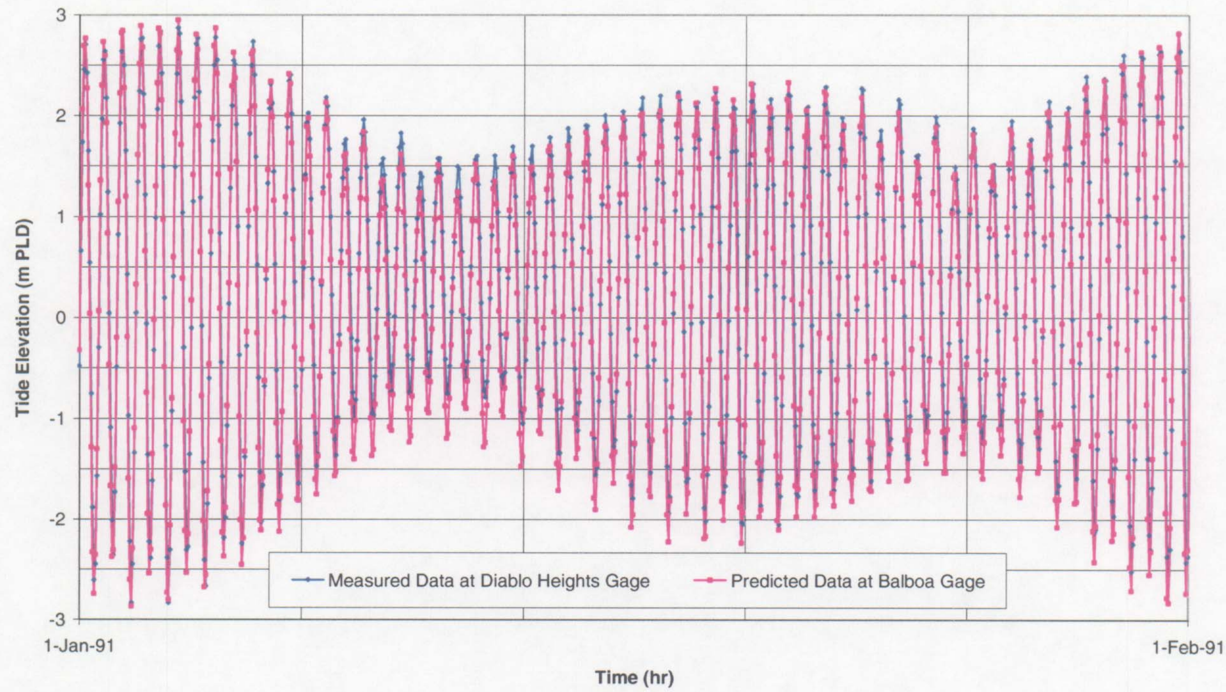


Figure II.15 – One Month Comparison of Measured Versus Predicted Tide Levels - Pacific Side

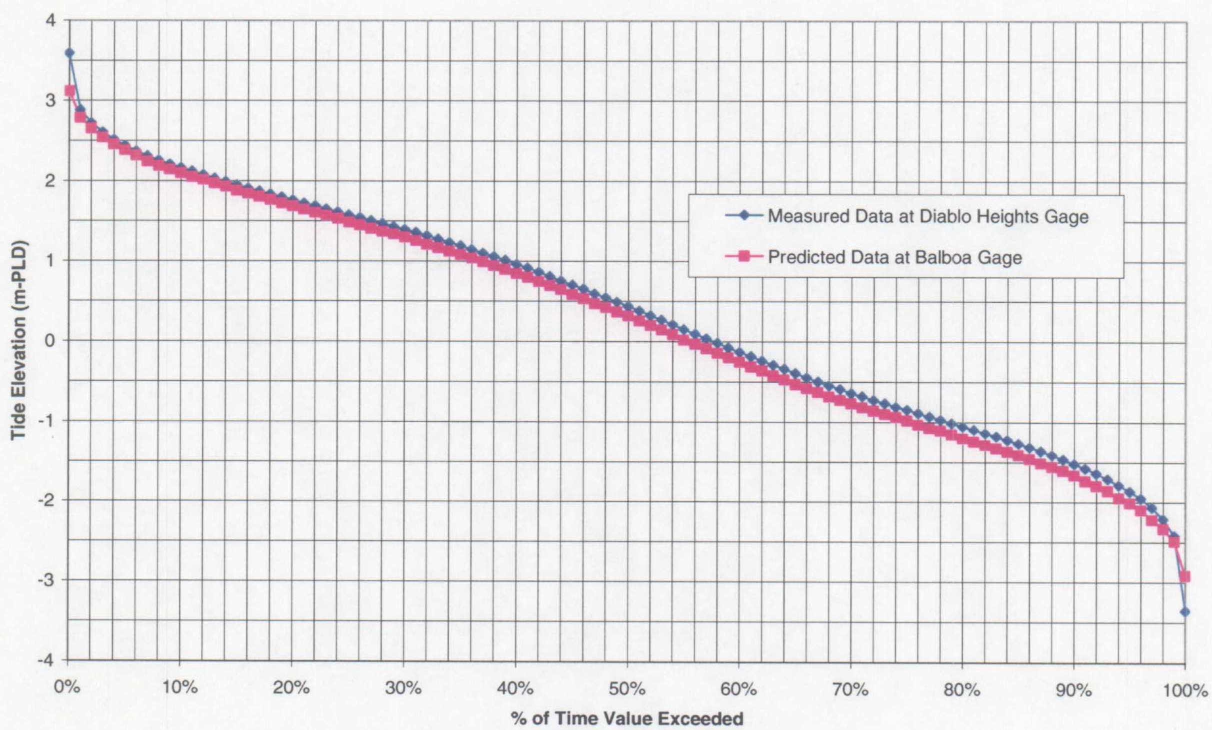


Figure II.16 – Comparison of Measured Versus Predicted Tide Level Percent Exceedance Distributions - Pacific Side

Again, the final step to the process would be to create a predicted tide series for the 1978 tidal epoch (1960-1978) and compare the extreme high and low elevations to those reported by ACP. The tidal predictions were roughly a factor of 1.06 below those extremes reported by ACP. Again, since the error was symmetric, M&N decided to modify the predicted percent exceedance distribution values by multiplying times a linearly interpolated factor between the calculated high and low water level factors. The finalized percent exceedance distribution can be seen below in **Figure II.17**, while the data table can be seen in **Appendix C**.

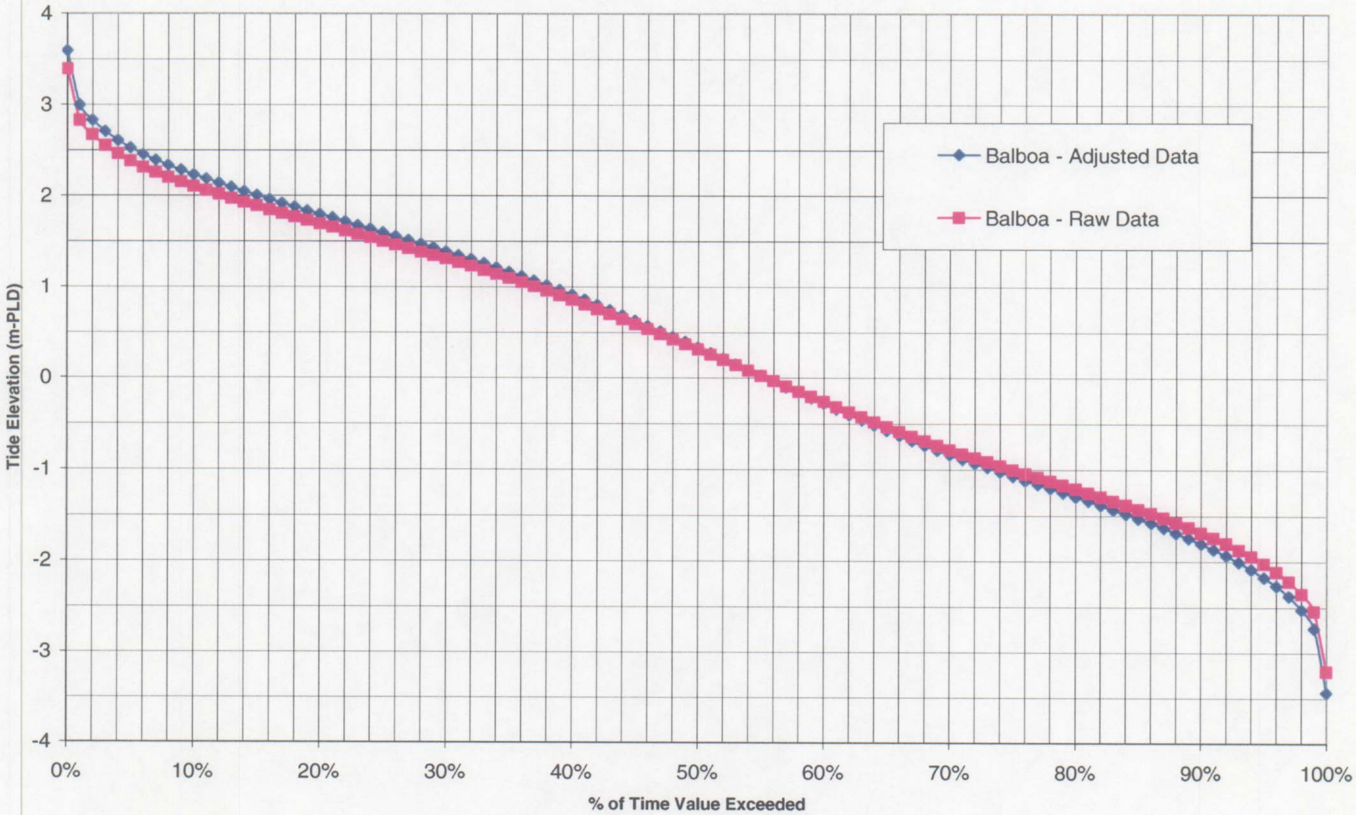


Figure II.17 – Finalized Percent Exceedance Distribution for Pacific Ocean Tide Levels

2. Range of Possible Lockage Lengths and Combinations

Another important factor in the hydraulic design of the new facilities to consider was the importance of incorporating the variation in lockage lengths that are possible with the new locks. ACP determined that the lockage lengths could vary from 426.7 m (1400') to 457.2 m (1500') to a maximum of 487.7 m (1600'), but that the water saving basin should be designed for an average 457.2 m (1500') lockage length. Nonetheless, the basins themselves should be sized so that all of the volume variations associated with the other lockage lengths could be managed by the basins without water spillage.

Along with the variability of the lockage lengths themselves, the possible combinations of lockages lengths also needed to be considered since these variations would also affect water volumes within the locks and basins and hence equalization levels. ACP stated that the overall lock configuration for the two-lift and three-lift options would be to have double gates for redundancy, a 30.5 m (100') spacing between the double gates and 426.7 m (1400') of clear space between the inner gates for the ship to be contained within. Based on these guidelines and considering all the various combinations of gates being out for service, etc., the four study alternatives (2 two-lift options and 2 three-lift options) were investigated and each possible combination of lockage length was drawn. **Figure II.18** shows the eight possible combinations for a two-lift option, while **Figure II.19** outlines the sixteen three-lift option possibilities.

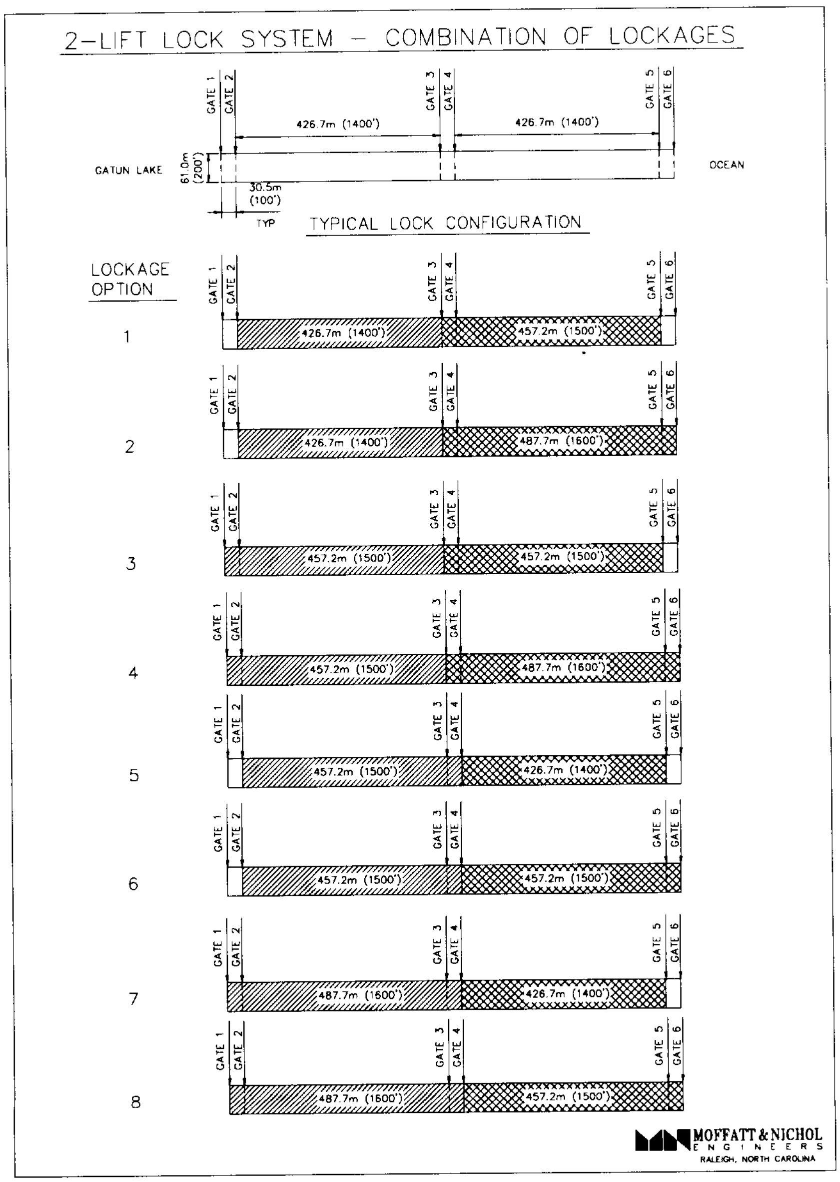


Figure II.18 – Possible Lockage Combinations – Two-Lift Options

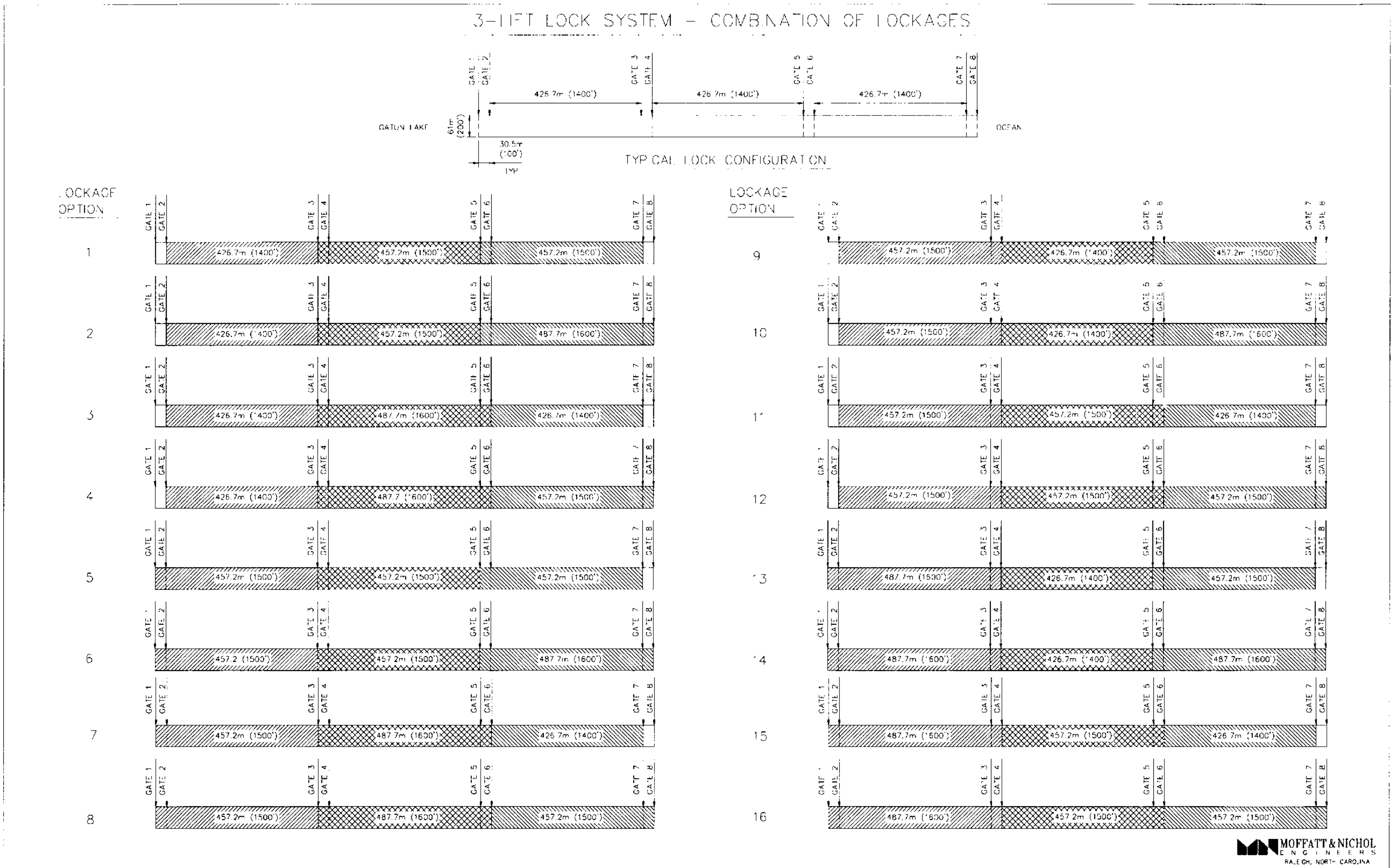


Figure II.19 – Possible Lockage Combinations – Three-Lift Options

3. Preliminary Post-Panamax Profile Elevations for a Three-Lift Option

ACP also provided the Project Team preliminary drawings of profile elevations for a three-lift option. (see **Figure II.20**). Maximum, mean, and minimum tide and lake level elevations are shown along with preliminary structural elevations for the lock floors, gate copings, and gate sills. The maximum and minimum equalization levels within the locks were also shown. The maximum and minimum equalization levels were calculated using the procedure presented in the 1910 Annual Report of the Isthmian Canal Commission by the Assistant Chief Engineer, H.F. Hodges. This provided a simple formula to calculate the lifts in a multiple lift lock based on available head differential and the plan area of the lock chambers.

For a 3-Lift system the equations were as follows:

$$\text{LOCK 1 LIFT} = \frac{(LAKE\ LEVEL - SEA\ LEVEL)}{\left[\left(\dfrac{LOCK\ 1\ AREA}{LOCK\ 3\ AREA}\right) + \left(\dfrac{LOCK\ 1\ AREA}{LOCK\ 2\ AREA}\right) + 1\right]}$$

$$\text{LOCK 2 LIFT} = \frac{(LAKE\ LEVEL - SEA\ LEVEL)}{\left[\left(\dfrac{LOCK\ 2\ AREA}{LOCK\ 3\ AREA}\right) + \left(\dfrac{LOCK\ 2\ AREA}{LOCK\ 1\ AREA}\right) + 1\right]}$$

$$\text{LOCK 3 LIFT} = \frac{(LAKE\ LEVEL - SEA\ LEVEL)}{\left[\left(\dfrac{LOCK\ 3\ AREA}{LOCK\ 1\ AREA}\right) + \left(\dfrac{LOCK\ 3\ AREA}{LOCK\ 2\ AREA}\right) + 1\right]}.$$

The plan areas of the lock chambers would have numerous combinations based on the lockage lengths and gate recesses. The gate recesses are included in the area calculations since they also fill with water during lockage operations. ACP estimated gate recess dimensions of 16 m (52.49’) wide x 80 m (262.47’) deep which were based upon a pre-dimensioning utilizing the maximum head of water, grade 50 structural steel, and double skin gates using LRFD and SAP2000. To estimate how many gate recesses would be associated with the 1400’, 1500’, 1600’ lockages, ACP used the following convention in their analyses: 1400’ Lockage – 1 Recess, 1500’ Lockage – 2 Recesses, and for the 1600’ Lockage – 3 Recesses.

Therefore, using these equations, the maximum, mean, and minimum water levels for the oceans and lake, and the various plan areas for the locks (based on the lockage lengths and gate recesses), the associated maximum and minimum equalization water levels in the locks could be calculated based on the lifts calculated for all these various combinations of water levels and lockage lengths.

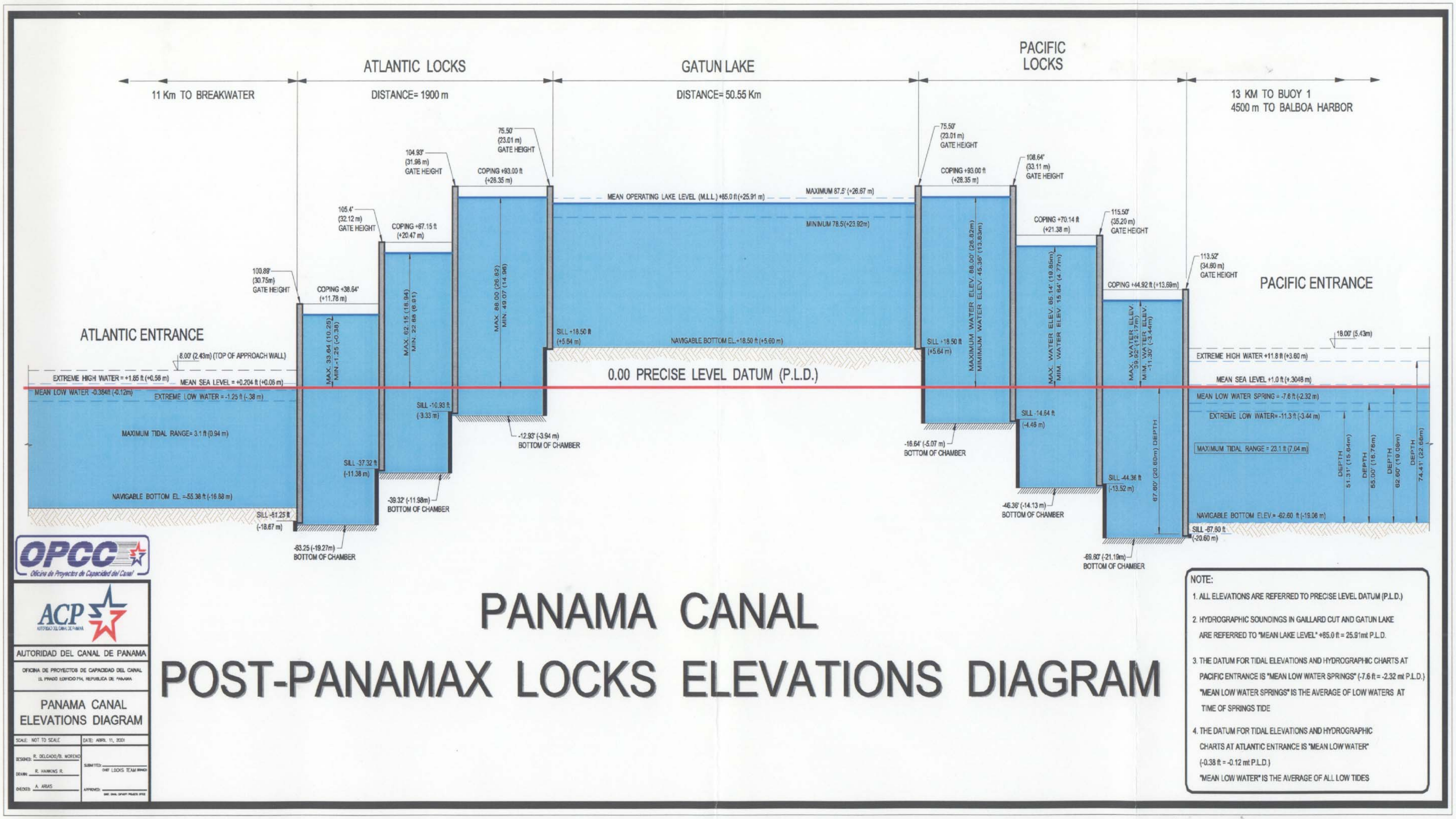


Figure II.20 - Preliminary Profile Elevations for a Three-Lift Option (provided by ACP)

The gate sill elevations on **Figure II.20** were calculated by subtracting 18.3 m (60') from the minimum lock equalization level to provide adequate draft for the Post-Panamax ships transiting the canal. A 0.6 m (2') space was also provided between the gate sill and the lock chamber floor for debris accumulation between lock maintenance. The gate coping elevations were set 1.5 m (5') above the maximum lock equalization elevation to provide adequate freeboard. This freeboard depth is the same as is currently provided on the existing lock.

4. Literature Review

M&N conducted an exhaustive literature review to support the design effort. A complete listing of hydraulic references can be found in **Appendix A**, while the most useful references are listed below.

General Hydraulic

- Hydraulic Design Criteria – Volumes 1 & 2, United States Corps of Engineers, 1980.
- Miller, D. S., Internal Flow Systems, BHR Group, 1990.
- Hwang, N. and C. Hita, Fundamentals of Hydraulic Engineering Systems, Prentice Hall, 1987.
- Munson, B., etal., Fundamentals of Fluid Dynamics, John Wiley & Sons, 1990.
- Brater, E.F. etal., Handbook of Hydraulics, Seventh Edition. 1996.

Lock Design

- EM 1110-2-1604, "Hydraulic Design of Navigation Locks", USCOE, 1995.
- Davis, John P., "Hydraulic Design of Navigation Locks", USCOE, 1989.
- Schohl, Gerald A., "User's Manual for LOCKSIM: Hydraulic Simulation of Navigation Lock Filling and Emptying Systems", 1999.
- AIPCN-PIANC Supplement to Bulletin No. 55, "Final Report of the International Commission for the Study of Locks", Permanent International Association of Navigation Congresses, 1986.
- Transactions of the International Engineering Congress – The Panama Canal, ASCE, 1915.
- EM 1110-2-2602, "Planning and Design of Navigation Locks", USCOE, 1995.
- American Society of Civil Engineers' Manuals and Reports on Engineering Practice No. 94, "Inland Navigation: Locks, Dams, and Channels, ASCE, 1998.
- EM 1110-2-1602. "Hydraulic Design of Reservoir Outlet Works", USCOE, 1980.
- EM 1110-2-1610, "Hydraulic Design of Lock Culvert Valves", USCOE, 1989.

Water Saving Basin Design

- AIPCN-PIANC Supplement to Bulletin No. 55, "Final Report of the International Commission for the Study of Locks", Permanent International Association of Navigation Congresses, 1986.

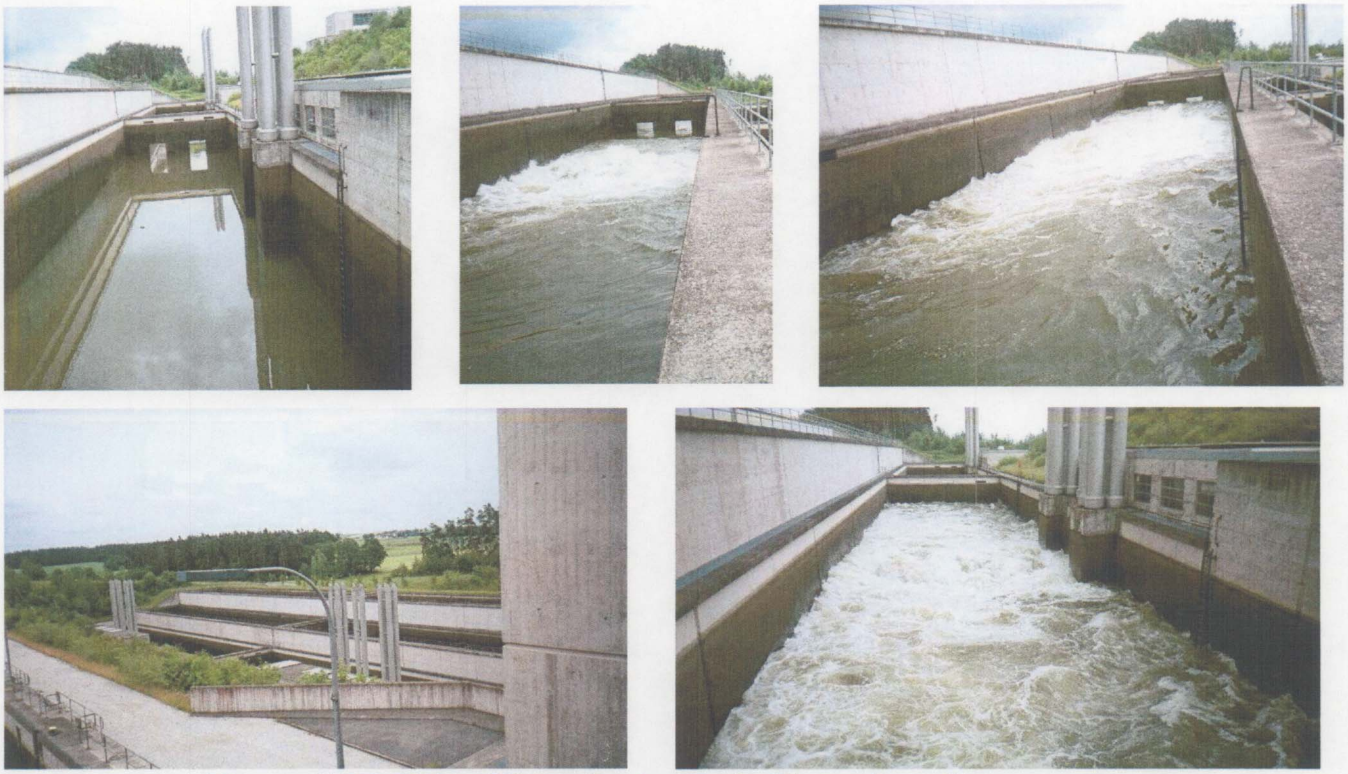
C. Structural Data Collection

For the civil and structural engineering effort, the following items were collected and reviewed:

- Atlantic site A-2 and Pacific site P-1 alignment drawings (see **Appendix B**),
- PIANC Final Report of the International Commission for the Study of Locks, Chapter 9, Water Saving Systems (Bull. 55, 1986),
- "Construction of Gatun Locks, Dam and Spillway." Transactions of the International Engineering Congress, 1915,
- "Development in the Construction of Water-Saving Locks on the Main-Danube Canal." Chara, Gerhard, RMD. Planning and Construction of the Main-Danube Canal,
- Geotechnical Reports for the Atlantic and Pacific sites, and
- Soil and rock contour files for the Atlantic and Pacific sites.

In addition, a site visit trip was conducted by INCA to Bachhausen Lock and Hilpolstein Lock, near Nurnberg, Germany on June 10, 2001. Photos of the Bachhausen Lock and Hilpolstein Lock water saving basins are shown in **Figures II.21-II.25**. These locks are approximately 12 meters (40 feet) wide, 183 meters (600 feet) long, and 24 meters (80 feet) deep, and they each have three levels of side-by-side water saving basins. Meetings were held with the lock operators and one of the lock designers. Lock and basin operation was observed and documented. Flow turbulence and wall freeboard requirements were noted. The approximate fill/empty time was measured and found to be roughly 13 minutes. The German company RMD was responsible for designing these locks and water saving basins, located on the Main-Danube Canal. Based on a series of articles related to the planning and construction associated with this work, the following information was provided for the typical water saving lock:

- Approximate filling/emptying time = 15-16 minutes (based on physical model result)
- Volume of operating water = 60,000 m^3
- Volume of lost water = 20,000 m^3
- Average rising rate = 1.54 m/min (based on physical model results)
- Average lowering rate = 1.62 m/min (based on physical model results)
- Maximum velocity in transverse culverts (conduits) = 8 m/sec
- Maximum water inlet flowrate between the lock and the basins = 140 m^3/sec



Figures II.21-II.25 – Water Saving Basin Photos from Germany

D. Geotechnical Data Collection

Data on subsurface conditions were taken from ACP's files and from published sources.

1. Atlantic Side

At the A-2 site, information from ACP's files included subsurface profiles prepared in 1908 for Gatun Locks, logs of boreholes drilled in 1938, 1940, 1966, 1987, 1990 and 1998, and excerpts from a 1943 report on field-bearing tests in the Gatun Formation. Published sources for the A-2 site included the United States Geologic Survey's Geologic Map of the Panama Canal and technical papers published in 1915 on subsurface conditions encountered during canal construction.

The USGS Geologic Map of the Panama Canal and Vicinity dated 1980 shows the Gatun Locks are located in the Gatun Formation and Holocene sediments. The Gatun Formation is middle Miocene and is described as locally calcareous and fossiliferous sandstone, siltstone, tuff, and conglomerate. Bedding measurements show dips of about 6 degrees toward the north-northwest in the vicinity of the Gatun Locks. No faulting is identified in the immediate vicinity of the locks.

Profiles prepared in 1908 (reproduced in **Appendix D**) showing foundation conditions for the Gatun Locks indicate that the original ground surface sloped upward from the ocean to an elevation of about 27 meters (89 ft.) above sea level at the south end of the lock. The profile shows the bedrock under the lock consists of an argillaceous sandstone underlain by a variably 1.5 to 7.5 meters (5- to 25-foot) thick conglomerate bed. The conglomerate is underlain by a soft sandstone and a lower argillaceous sandstone that is variably interlayered with tuffaceous beds. The strata dip gently toward the north and are consistent in dip magnitude and direction with that shown on the USGS Geologic Map. Locally, the profile shows zones up to 20 meters (66 ft.) deep where the upper sandstone and conglomerate were weathered to soil and decomposed rock. On the north end of the lock, the profile shows a stratum of clay at ground surface (currently referred to as Atlantic muck) extending up to about 20 meters (66 ft.)deep. No groundwater levels are indicated on the profile.

Information on the subsurface conditions encountered during construction of the Canal, presented in the Transactions of the International Engineering Congress (IEC), 1915, is useful in helping understand the conditions that may be encountered during construction of new locks and water saving basins. Specifically, Paper No. 3 in the IEC describes the geologic conditions encountered all along the canal during construction, and Paper No. 11 describes in detail the soil and rock conditions encountered during construction of the Gatun Locks, Dam and Spillway.

Paper No. 11 describes the results of borings advanced 15 meters (50 feet) below the lock foundation level during construction. Based on this drilling, the argillaceous sandstone units identified in the 1908 profile were reclassified as indurated clay. Additionally, information on the soft sandstone identified on the 1908 profiles was reported by drillers as black sand that had greater permeability than the overlying and underlying rock units. A test pit was excavated in the soft sandstone to gain a better understanding on the permeability characteristics of this unit. Groundwater in the test pit was removed by pumping and water levels in surrounding boreholes were monitored. Water levels in the boreholes were noted to "answer quickly to the pumping" and the permeability was considered to be controlled in this unit by interconnected fractures in the rock. Based on this observation, a >2 meter (6-foot) wide by 4 to 5.5 meter (12 to 18-foot) deep cut off trench was excavated through the soft sandstone where it occurred in the lock foundation and the trench was filled with concrete. Four-inch diameter observation wells were installed in the lock walls to monitor groundwater levels beneath the lock floors while the water level in the lake rose after the locks were finished. Despite the cut-off wall, lake pressure was transmitted under the lock floor as indicated by a corresponding rise in the lake water levels and groundwater levels measured in the wells.

Paper No. 11 describes extensive problems with supporting construction equipment and landsliding that developed in the Atlantic Muck on the north end of the lock excavation. The landsliding was described as particularly troublesome on the east side of the lower lock where ground instability in the muck was observed as far as 150 meters (500 ft.) from the lock centerline. **This zone of instability extended into the area proposed for construction of the new locks and water saving basins and presumably, the effected area was backfilled after completion of lock construction.**

Logs of some of the boreholes drilled since canal construction near the Gatun Locks were provided by the ACP and are included in **Appendix D**. These investigations were completed during different time periods, including 1938, 1940, 1966, 1987, 1990, and 1998. Locations of the boreholes are shown on **Figure 1 in Appendix D**, and a subsurface profile along the west wall of the proposed new lock is provided in **Figure 2 in Appendix D**.

The ACP also supplied an excerpt on Allowable Bearing Capacity and Modulus of Elasticity from Chapter 5 – Foundations and Slopes of the Final Report on Modified Third Locks Project, Part II – Design, dated December 1943. The excerpt describes field bearing tests in the Gatun Formation using circular plates with diameters in the range of 180 mm to 1 meter (7 to 40 inches). The tests were performed in four horizontal drifts excavated off a test pit sunk at Station 135+30 on the axis of the proposed new lock. Each drift was in a different bearing strata. Test results were provided for the Gatun gray sandstone, the Gatun volcanic tuff, and the Gatun yellow-green sandstone. Where possible, the plates were loaded to failure, after having sustained a constant pressure of 1.9 MPa (20 tons per square foot) for several days. The report concluded that the average bearing capacity of the rock varied from about 15 to 34 MPa (150 to 350 tons per square foot), the modulus of elasticity averaged 2,600 MPa (375,000 psi), and there was little, if any, tendency for continuous deformation under a constant load of 1.9 MPa. Based on these results and laboratory testing, values of 1.9MPa (20 tons per square foot) for the allowable bearing capacity and 2,600 MPa (375,000 psi) for modulus of elasticity were used in the design of the new lock structures at Gatun.

A copy of a table entitled Engineering Properties – Canal Zone Rock Units that was part of a document entitled *Isthmian Canal Studies – 1947* is included in **Appendix D**. In it, the Gatun Formation is described as fine-grained argillaceous and calcareous sandstones with interbedded dense tuffs and conglomerates. It is indicated to be a medium hard rock which is defined as one that can be picked with moderate blows of a geologic hammer and can be cut with a knife. The compressive strength of the Gatun Formation is reported to be in the range of 3.2 to 6.5 MPa (470 to 940 psi), and the unit weight is reported to be 18.8 kN/m^3 (120 pcf).

The boreholes show a sequence of strata generally consistent with that shown on the 1908 profile (**Appendix D**). As shown on **Figure 2 in Appendix D**, there is fill material at ground surface in most of the boreholes plotted on the profile. The fill material generally consists of up to 17 meters (56 ft.) of soft clay, silt and sand mixed with pebbles, cobbles and boulders of sandstone, tuff and agglomerate. Pieces of concrete, coral and wood fragments and canal excavation debris are also described in the logs. Residual and locally, alluvial soils, are recorded below the fill material. The north end of the profile (right on the profile in **Appendix D**) depicts clay at the surface (now referred to as Atlantic muck). This material is described as gray soupy mud.

Rock (including both weathered rock and sound rock) occurs variably between about 1 and 20 meters below ground surface within the vicinity of the Gatun Locks. Generally three laterally consistent rock horizons are observed in the boreholes presented on Figure 2: upper sandstone; conglomerate; and lower sandstone. The upper sandstone is described as soft and weak when weathered and medium hard and moderately strong when fresh to slightly weathered (refer to the table in **Appendix D** for guidance on the probable meaning of these terms). The unit is generally fine-grained, moderately bedded, and variably well jointed to locally massive. Bedding dip recorded on the logs (6°) is consistent with that observed on the 1908 profile and depicted on the

USGS geologic map. The unit is also described as locally silty, very fossiliferous and calcareous.

The conglomerate is described as medium soft to medium hard and weak to moderately strong. The top of this unit is suspected to be an old erosional surface, and therefore, might show increased weathering locally. The unit is reported to be finer-grained near the top and coarser-grained, sandy and loosely cemented at the base. The unit is predominately comprised of basaltic and andesitic pebbles in a calcareous, tuffaceous matrix.

The lower sandstone is described as variably soft to medium hard and weak to moderately strong. The unit generally consists of fine- to very coarse-grained volcanic debris (primarily soft pumice) that is thinly bedded. Tuffaceous and volcanic ash beds and lenses and lava flows were also encountered in some of the boreholes. Additionally, local beds of black sandstone were also observed. Although joints within this unit are described as near vertical, discontinuous and tight or closed, the unit is described as "permeable" on several logs reviewed in this area.

Most of the borehole information provided does not include data on groundwater levels; the few logs that present groundwater information show water levels occurring around 5 to 10 meters (16 to 32 ft.) below ground surface.

Further geotechnical information and discussion for the Atlantic side can be found in **Appendix D**.

2. Pacific Side

At the P-1 site, information from ACP's files included logs of boreholes drilled in 1939-1942 and 1956 (with some re-classification done in 1966), an undated, unreferenced plan map which depicts the layout of the Miraflores Locks and the location of borings advanced during investigations related to the Locks and geologic contacts drawn by others, and a profile constructed through the centerline of the eastern lock proposed for the P-1 alignment prepared by Harza and Tams for the Report on Evaluation of Lock Channel Alignments(March 2000) that shows existing ground surface, bottom elevation of proposed excavation, and an estimated top of rock surface along the alignment.

The USGS Geologic Map of the Panama Canal and Vicinity dated 1980 shows that the Panama Canal Area near the Miraflores and Pedro Miguel Locks is underlain by sedimentary and igneous rocks in a complex juxtaposition created by faulting and igneous activity. Rocks of the Panama Formation, which consist of Tertiary-age tuffaceous sandstone and siltstone, and limestone, underlie the Pedro Miguel Locks. Fine- to coarse-grained agglomerate of the Pedro Miguel Formation occurs south and west of the Pedro Miguel Locks. Two local small-scale faults and one regional fault are shown to transect across and/or near the location of the Lock. The Miraflores Locks are underlain by the Panama and La Boca Formations. The La Boca Formation consists variably of mudstone, siltstone, sandstone, limestone and tuff. The USGS map shows that basalt primarily occurs south and west of the Miraflores Locks and the Miraflores Fault traverses near the northwest end of the lock.

Logs of some of the boreholes drilled since canal construction near the Miaflores Locks were provided by the ACP and are included as **Appendix D**. These boreholes were completed during 1939-1942, and 1956; some of the geologic formations presented on the logs were reclassified in 1966. Locations of the boreholes are shown on **Figure 1 in Appendix D**. An undated, unreferenced plan map was also provided by the ACP, which depicts the layout of the Miraflores Locks and the location of borings advanced during investigations related to the Locks. Geologic contacts drawn by others are shown on this plan. Other information about the project provided by the ACP includes a profile constructed through the centerline of the eastern lock proposed for the P-1 alignment. This profile, prepared by Harza and Tams for the Report on Evaluation of Lock Channel Alignments, March 2000 (Exhibit 34), extends from Station 1+000 to 10+500; the proposed locks and guide walls for the P-1 alignment are shown to occur between about Station 6+300 and 8+550 on the profile. The profile shows existing ground surface, bottom elevation of proposed excavation, and an estimated top of rock surface along the alignment. The location of several of the boreholes drilled within the vicinity of the Miraflores Locks, geologic formations and fault projections are sketched by hand on the profile.

We were also supplied a copy of a table entitled Engineering Properties Canal Zone Rock Units, Isthmian Canal Studies – 1947 which is included in **Appendix D**.

The following description of interpreted subsurface conditions near the proposed P-1 alignment is based on information presented in the boring logs, geologic contacts sketched on the boring plan provided by the ACP, and the USGS Geologic Map. Based on the information presented on the plans, profiles, and boring logs, Golder constructed a subsurface profile along the centerline of the proposed eastern lock on the P-1 alignment (**Figure 2 in Appendix D**) and a geologic map of the area (Figure 3). The locations of the borings shown on **Figures 1, 2, and 3 in Appendix D** are approximate. Groundwater levels are not shown on the profile because the boring logs did not contain groundwater level data. Further geotechnical information and discussion for the Pacific side can be found in **Appendix D**.

E. Mechanical Data Collection

A review of available applicable design information was conducted. Drawings from the locks with water saving basins on the Elbe River near Magdeburg, Germany and project information were also reviewed. The United States Army Corps of Engineers (COE) Engineer Manuals and gate and actuator manufacturer's information provided the most useful and appropriate information.

The COE references are:

- Hydraulic Design of Lock Culvert Valves, Engineer Manual 1110-2-1610
- Vertical Lift Gates, Engineer Manual 1110-2-2701
- Design of Spillway Tainter Gates, Engineer Manual 1110-2-2702
- Lock Gates and Operating Equipment, Engineer Manual 1110-2-2703

The manufacturers that provided information are noted below.

A survey of existing installations was also made to determine current designs used for large valves and valves with fast operating times. The design review and current installation survey would determine the most practical type of design of a valve and actuator for this application.

Once the general size and speed of the valves was determined, more in-depth discussions were held with the manufacturers, as the combination of the size and travel speed of the control valves is not common.

Valve and Actuator Manufacturers

Specific Assistance

- GE Hydro – Doncaster, UK
- Hydro Gate - Denver, Colorado, USA
- Bosch-Rexroth – Bethlehem, Pennsylvania, USA
- Waterman – Exeter, California, USA

General Information and Assistance

- Armtec – Guelph, Ontario, Canada
- Ingersol-Rand – Bryan, Ohio, USA
- Rodney Hunt – Orange, Massachusetts, USA
- Steel-Fab – Fitchburg, Massachusetts, USA
- Tungabhadra Steel Products –District Bellary, Karnataka, India

F. Electrical Data Collection

A review of the existing facilities and interviews were carried out with ACP personnel to determine the specifications for the incoming feeder lines. For the Atlantic side, 6900V incoming feeders are reduced to 480V, 60Hz, 3-phase for the existing locks machinery. For the Pacific side, 2400V incoming feeders are reduced to 480V, 60Hz, 3-phase for the existing locks machinery.

III. DEVELOPMENT OF DESIGN CRITERIA

A. Need for Design Criteria

As stated before, this project is entirely unique in size and scope. Therefore, the design criteria need to be defined with care. The finalized criteria for each discipline (hydraulic, structural, geotechnical, mechanical, and electrical) follow. Due to the nature and scope of the project, the criteria are general in nature. However, the basic design items are detailed enough to provide proper guidance and direction in the preliminary design of the water saving basins to determine the feasibility of construction and their general operation.

B. Hydraulic Design Criteria

1. Properties of Water

Following is a list of the properties assumed:

Temperature:	24°C / 75.2°F

Based on information provided by ACP, fresh, brackish and fully saline water is found near the current locks. Based on recent measurements, the average salinities could be classified as follows:

Class	Salinity	Locations
Fresh	0 ppt	Gatun Lake, Upper Chamber at Gatun, Pedro Miguel Lock
Lower Brackish	1 ppt	Miraflores Lake, Middle Chamber at Gatun
Low Brackish	4.5 ppt	Lower Chamber at Gatun
Brackish	10 ppt	Downstream (D/S) of Gatun Locks During Emptying
High Brackish	20 ppt	Upper Chamber at Miraflores
Higher Brackish	26 ppt	Lower Chamber at Miraflores (D/S of Miraflores) During Emptying
Salt	33 ppt	Saline D/S at Gatun and Miraflores

These classes then have the following water properties:

Density (fresh):	997.3 kg/m^3 / 1.93 slugs/ft^3
Density (lower brackish):	998.1 kg/m^3 / 1.94 slugs/ft^3
Density (low brackish):	1000.7 kg/m^3 / 1.94 slugs/ft^3
Density (brackish):	1004.9 kg/m^3 / 1.95 slugs/ft^3
Density (high brackish):	1012.7 kg/m^3 / 1.96 slugs/ft^3
Density (higher brackish):	1017.4 kg/m^3 / 1.97 slugs/ft^3
Density (salt):	1022.9 kg/m^3 / 1.98 slugs/ft^3

Specific Weight (fresh):	9783.5 N/m^3 / 62.3 lb/ft^3
Specific Weight (lower brackish):	9790.9 N/m^3 / 62.3 lb/ft^3
Specific Weight (low brackish):	9817.1 N/m^3 / 62.5 lb/ft^3
Specific Weight (brackish):	9858.4 N/m^3 / 62.7 lb/ft^3

Specific Weight (high brackish):	9980.4 N/m^3 / 63.5 lb/ft^3
Specific Weight (salt):	10034.5 N/m^3 / 63.9 lb/ft^3
Dynamic Viscosity (fresh):	9.16 x 10-4 N*sec/m^2 / 1.91 x 10-5 lb*sec/ft^2
Dynamic Viscosity (lower brackish):	9.18 x 10-4 N*sec/m^2 / 1.92 x 10-5 lb*sec/ft^2
Dynamic Viscosity (low brackish):	9.26 x 10-4 N*sec/m^2 / 1.93 x 10-5 lb*sec/ft^2
Dynamic Viscosity (brackish):	9.39 x 10-4 N*sec/m^2 / 1.96 x 10-5 lb*sec/ft^2
Dynamic Viscosity (high brackish):	9.62 x 10-4 N*sec/m^2 / 2.01 x 10-5 lb*sec/ft^2
Dynamic Viscosity (higher brackish):	9.76 x 10-4 N*sec/m^2 / 2.04 x 10-5 lb*sec/ft^2
Dynamic Viscosity (salt):	9.92 x 10-4 N*sec/m^2 / 2.07 x 10-5 lb*sec/ft^2
Kinematic Viscosity (fresh):	9.19 x 10-7 m^2/sec / 9.89 x 10-6 ft^2/sec
Kinematic Viscosity (lower brackish):	9.20 x 10-7 m^2/sec / 9.90 x 10-6 ft^2/sec
Kinematic Viscosity (low brackish):	9.26 x 10-7 m^2/sec / 9.96 x 10-6 ft^2/sec
Kinematic Viscosity (brackish):	9.34 x 10-7 m^2/sec / 10.06 x 10-6 ft^2/sec
Kinematic Viscosity (high brackish):	9.50 x 10-7 m^2/sec / 10.23 x 10-6 ft^2/sec
Kinematic Viscosity (higher brackish):	9.59 x 10-7 m^2/sec / 10.33 x 10-6 ft^2/sec
Kinematic Viscosity (salt):	9.70 x 10-7 m^2/sec / 10.45 x 10-5 ft^2/sec

2. Hydraulic Analyses

a) Geometry

Specific details, dimensions, and layouts of the lock filling/emptying systems are unknown at this stage. The analysis will assume that the water saving basin conduits will tap into the lock filling/emptying system at the lock wall. The water saving basin conduit soffit elevation shall be assumed to be 4.57m (15 feet) below the proposed lock floor elevations based on direction provided by ACP. This assumption is based on direction given to the Project Team by ACP during the kickoff meeting. The details of this connection will not be determined at this stage.

Turbulence of the exiting flows into the water saving basins is not a critical design issue. The design will provide a reasonably smooth geometry that could be expected to perform satisfactorily for both filling and emptying operations. Final performance will have to be verified in hydraulic model tests at a later design phase.

The entrance losses (when filling the basins) and exit losses (when emptying the basins) will depend on flow characteristics through the lock emptying / filling system, which will be undefined for this analysis. Therefore, the "typical" loss coefficients will be selected based upon data developed in hydraulic model studies. These loss coefficients will govern flows between the ports in the lock floor and the connection to the water savings basins in the lock wall. Using "typical" loss coefficients at this conceptual stage is justified since most authorities agree that the short distances between loss-generating elements in a lock filling/ emptying system causes the flow, hence the losses, to never fully develop between each transition element. Therefore, the calculated losses using the sum of individual loss coefficients should be greater than the actual losses in the prototype system and thereby, using "typical" values will be conservative.

b) Frictional Properties of Conduits

The roughness height for the conduit will be allowed to vary to determine the hydraulic behavior of the system when just completed as well as when the system has aged considerably. Therefore, the required conduit size shall be determined using 3.0 mm (most conservative), and the expected conduit velocity shall be estimated using 0.01 mm (most conservative). While the value of 3.0 mm may seem high, the pitting that is evident on the existing conduit wall warrants a high value.

Roughness Height for Concrete Conduits: 0.01 mm – 3.0 mm

c) Water Saving Basin Performance

The maximum lock filling/emptying rate for the new locks and water saving basins shall be no greater than 2.28 m/min (7.5 ft/min) based on safe ship handling procedures in the current locks.

Along with the design constraint of a maximum F/E rate of 2.28 m/min (7.5 ft/min), a target total F/E operation time equal to the current locks will be used as a guideline (3 operations*(8-9 min/operation) = 24-27 min total operation time – including valve operations).

Valve opening/closing times shall be approximately one minute and shall be included in the basin and lock filling/emptying times listed above, i.e., the total time for filling/emptying is nine minutes, including valve operations.

The water saving basins will be sized to accommodate volumes from all possible combinations of lockage lengths – 426.7 m (1400'), 457.2 m (1500'), and 487.7 m (1600'). The dimensions of the water saving basin conduits, however, will be sized based on a 457.2 m (1500') lockage length based on ACP direction. Therefore, the filling/emptying rates and times for a 487.7 m (1600') lockage will likely be lower and longer than for the design condition of 457.2 m (1500').

d) Valves

Control shall be provided by either vertical lift, reverse tainter, or tainter valves depending on the specific application. Each conduit may have a service valve and an identical emergency valve. The possibility of using manifolds will be investigated to reduce the overall size of the valves. Bulkhead slots will be provided on both sides of the valve chamber to effect closure for maintenance.

Streamlined contractions/expansions on both sides of the valve chamber were not investigated in detail at this level of study. Ultimately, this should be investigated to determine if valves smaller than the overall conduit dimensions are feasible. Since flow through the system will be in both directions, transition rates will be no more than 1:6. This is the fastest recommended rate of transition as defined in the Corp of Engineer's Report EM 1110-2-1604, *"Hydraulic Design of Navigation Locks"*.

Valve operating times shall be investigated and optimized to the extent practicable for this conceptual level study. However, a target operating time of approximately one minute (opening and closing) will be a goal as the existing timings of one minute for the existing rising stem valves are a valuable benchmark in that they have provided acceptable performance for many years in a system that is closer to the future prototype than other available examples. Furthermore, these are among the shortest valve times in use at any major lock. Faster valve timings will raise safety concerns and will increase uncertainty about whether it will be possible to maintain tranquil conditions within the lock during each WSB-to-lock filling operation.

Excessive negative pressures shall be avoided below the valve for both partial and full opening conditions.

e) Water Levels and Datums

Gatun Lake water levels shall be described by statistics obtained through daily measurements for the period of 1966 to 2000. Future lake operations and expected water level variations shall also be included in the analyses.

Historic extreme water level excursions in Gatun Lake beyond the range of available statistics shall be handled as singular cases (outliers) to verify satisfactory performance of the system.

Water level variations on the Pacific side shall be governed by measured tide data at Diablo Heights, statistics of predicted tides at Balboa, as well as other tidal datums provided by ACP.

Water level variations on the Atlantic side shall be governed by measured tide data, statistics of predicted tides at Cristobal, as well as other tidal datums provided by ACP.

All elevations reported and used for this project shall be referenced to P.L.D. (Precise Level Datum). The conversions of tidal datums to P.L.D. shall be as follows:

Table III.1 – Project Tidal Datums

Tidal Datum	Elevation (P.L.D.)	
	Pacific side	Atlantic side
Extreme High Water	+3.60 m (+11.8 ft)	+0.56 m (+1.85 ft)
Mean Sea Level	+0.304 m (+1.0 ft)	+0.06 m (+0.204 ft)
Mean Low Water	-2.32 m (-7.6 ft)	-0.12 m (-0.384 ft)
Extreme Low Water	-3.44 m (-11.3 ft)	-0.38 m (-1.25 ft)

f) Analysis Procedures

Conduits shall be analyzed using standard closed conduit hydraulic procedures as outlined in EM 1110-2-1604, "Hydraulic Design of Navigation Locks."

The analysis accounts for inertia head. The inertia head is defined as:

$$H_{inertia} = \frac{L}{g}\int \frac{dv}{dt}$$

For purposes of calculation, the differential is approximated by the change in velocity between consecutive time steps (normally one second). By including the inertia head term, the flow between the lock and basins will cause the water level in the receiving basin to "overtravel" or rise beyond the nominal equilibrium level represented by the arithmetic average of the initial water levels.

g) Loss Coefficients

Loss coefficients were assigned to each feature of the conduit system using published values from standard texts, Corps EM's, USCOE's Hydraulic Design Criteria, and other published hydraulic texts and model test reports. Typical losses include:

- Entrance
- Gates/Valves
- Transitions
- Expansions/Contractions
- Bends
- Exit

Past experience has shown that the cumulative loss coefficients overstate the losses for the entire system taken as a whole. Therefore, the individual loss coefficients were adjusted, as needed, to yield overall loss coefficients that are in line with measured "norms" for similar systems obtained from hydraulic model tests.

Variable loss and discharge coefficients were included in the analysis as a function of valve opening/closing percentages based on guidance within USCOE's Hydraulic Design Criteria.

Separate values for loss coefficients were assigned at each location according to flow direction to account for preferred geometries that must necessarily exist for flow in either direction.

h) Cavitation and Air Demand

Cavitation and air demand shall not be examined in detail at the concept study level. Ultimately, pressures within the system should be analyzed to determine where air vents may be required. Pressure drops that occur immediately downstream of the valve slots should also be included in the analysis. Air will be provided if expected negative pressures below the valves may exceed –3.048 meters (-10 feet).

The need for steel liners just below the valves should be investigated based on exit velocities and pressures below the valves.

Air demand shall be provided with vents, if necessary, to offset negative pressures in the conduits that cannot be controlled by varying the conduit geometry.

i) Other Items

Conduit velocities will be checked to verify that scouring of the lining will not occur.

3. References

Fluid properties, design procedures, loss coefficients, etc. from the hydraulic publications listed in **Appendix A** will be used in the design of the water saving basins systems.

C. STRUCTURAL DESIGN CRITERIA

1. General

This section presents the basic material properties, loads, load combinations, and references that shall be used in the structural design of the water saving basin walls, floor/roof slabs, and connecting culverts at the Panama Canal Atlantic and Pacific sites.

2. Material Properties

Following is a list of the preliminary material properties assumed:

Cast-in-place Concrete	f'c = 28 Mpa (4000 psi) at 28 days
Non-Shrink Grout	f'c = 28 Mpa (4000 psi) at 28 days
Reinforcing Steel	ASTM A 615M, fy = 420 MPa (60 ksi)
Embedded Structural Steel	ASTM A 36M, fy = 250 MPa (36 ksi)
Sheet Piling	ASTM A 328M, fy = 270 MPa (39 ksi)
Rock/Soil Anchor Strands	ASTM A 416M, fy=1860 MPa (270 ksi), Low Relaxation

3. Loads

Wall loads are discussed in the Corps of Engineers manual EM 1110-2-2502, *"Retaining and Flood Walls"*. These loads were reviewed and adapted to apply to water saving basins. Following is a list of design loads with pertinent design notes:

a) Dead Loads

The following unit weights of materials shall be assumed:

Item	Unit Weight
Freshwater	9.8 kN/m^3 (62.4 pcf)
Saltwater	10.0 kN/m^3 (63.9 pcf)
Concrete	23.6 kN/m^3 (150 pcf)
Steel	77.1 kN/m^3 (490 pcf)

Although there may be some salinity in the water, especially in the chambers closest to the oceans, the groundwater salinity is anticipated to be low. Therefore, freshwater density is recommended for groundwater pressures. For interior water densities, the applicable conservative value shall be selected for design purposes.

b) Hydrostatic (Including Uplift) Loads

Maximum Gatun Lake Level:	El. 26.82 m (88.0 ft.)
Minimum Gatun Lake Level:	El. 23.92 m (78.5 ft.)
Maximum Atlantic Ocean Level:	El. 0.56 m (1.8 ft.)
Minimum Atlantic Ocean Level:	El. -0.38 m (-1.2 ft.)
Maximum Pacific Ocean Level:	El. 3.60 m (11.8 ft.)
Minimum Pacific Ocean Level:	El. -3.44 m (-11.3 ft.)

The elevations listed above were provided by the ACP. All elevations are relative to the Precise Level Datum (PLD).

The PIANC publication, "Final Report of the International Commission for the Study of Locks" specifies avoiding uplift pressure on the bottom of empty water saving basins by providing a drainage system. Installation of a drainage system will be assumed. However, both clear and blocked drainage conditions will be investigated. Calculation of uplift forces shall be in accordance with EM 1110-2-2502, "*Retaining and Flood Walls*", and flotation stability shall be calculated in accordance with TL 1110-2-307, "*Flotation Stability Criteria for Concrete Hydraulic Structures*".

Provision of pressure relief valves will be considered (as a backup for a blocked drain condition). If these valves are used, then the saturation level shall be assumed to be level with the bottom of the valves for the blocked drain condition. The combined probability of blocked drains and blocked pressure relief valves is anticipated to be too low for consideration.

c) Earth Loads

Per geotechnical report completed by Golder Associates (see **Appendix D**)

d) Seismic Loads

During future design phases, seismic loads shall be considered. Seismic loads will not be addressed during conceptual design.

e) Wind Loads

Wind loads usually are not included in final basin analysis (except where major portions are not backfilled). However, wind loads may need to be considered for the construction condition, prior to backfill placement. Where included, a pressure of 1450 N/m^2 (30 psf) may be used for feasibility-level design. If this loading condition is found to be critical, then this wind load assumption should be investigated further. See Section IV of EM 1110-2-2502 and TM 5-809-1 for additional guidance. Construction load conditions will not be considered for conceptual design.

f) Temperature and Shrinkage

During future design phases, temperature and shrinkage loads shall be considered, specifically with respect to expansion joint design and minimum reinforcing requirements. Temperature and shrinkage loads will not be addressed during conceptual design.

g) Construction

During construction, wind loads, as described above, may be applicable. It is anticipated that the basin area will be dewatered. Therefore, uplift pressure is assumed to be zero during construction. Construction load conditions will not be considered for conceptual design. For feasibility-level design, the temporary construction load conditions should be checked as unusual load cases.

4. Load Conditions

EM 1110-2-2602, "*Planning and Design of Navigation Locks*", Appendix B provides guidance on design load conditions for lock walls, gate bays, approach walls, and sills. Water saving basins are not specifically addressed. However, based on this manual, in conjunction with EM 1110-2-2502, "*Retaining and Flood Walls*" and descriptions provided in PIANC literature, some comparable load conditions were derived. The following loads shall be considered:

a) General Design Load Conditions

1. Usual
2. Unusual
3. Extreme

b) General Design Load Locations

1. Exterior walls and bottom floors
2. Interior walls
3. Middle floors (for stacked basins only)
4. Roof slab (for stacked basins only)
5. Culverts

Specific load cases below follow this numbering system (e.g., Load Case 2B refers to interior walls, etc.).

c) Specific Load Conditions

<u>Exterior Walls and Bottom Floors</u> **(Cases 1A Through 1D)**	
Case 1A – Normal Operating Condition **(Usual)**	• Backfill assumed to match existing topography. Earth pressure to top of backfill. • Surcharge load from uphill basin, if applicable, assuming full water level in uphill basin. • Clear drains (no hydrostatic pressure). • No water inside basin (neglect any "residual" water).
Case 1B – Earthquake Conditions **(Unusual/Extreme)**	• Same as Case 1A Plus OBE Earthquake (Unusual). • Same as Case 1A Plus MDE Earthquake (Extreme). (This condition will not be considered for conceptual design.)
Case 1C – Blocked Drains Operating Condition **(Unusual)**	• Backfill assumed to match existing topography. Earth pressure to top of backfill. • Surcharge load from uphill basin, if applicable, assuming full water level in uphill basin. • Blocked drains (hydrostatic pressure, including uplift). • No water inside basin (neglect any "residual" water).
Case 1D – Construction Condition **(Unusual)**	• Earth pressure up to top of backfill. • Construction surcharge. • No water in basin. • No uplift. • No earthquake. • Wind loading, if applicable. (This condition will not be considered for conceptual design.)

Interior Walls **(Cases 2A Through 2C)**	
Case 2A – Normal Operating Condition **(Usual)**	• No water in one basin (neglect any "residual" water). • Water level to top of interior wall in adjacent basin (conservative for normal condition). • Clear drains.
Case 2B – Earthquake Conditions **(Unusual/Extreme)**	• Same as Case 2A Plus OBE Earthquake (Unusual). • Same as Case 2A Plus MDE Earthquake (Extreme). (This condition will not be considered for conceptual design.)
Case 2C – Blocked Drains Operating Condition (Unusual)	• No water in one basin (neglect any "residual" water). • Water level to top of interior wall in adjacent basin (conservative for normal condition). • Blocked drains.

Middle Floors (Stacked Basins) **(Cases 3A Through 3C)**	
Case 3A – Normal Operating Condition **(Usual)**	• Backfill assumed to match existing topography. Earth pressure to top of backfill. • Clear drains (no hydrostatic pressure). • Full water level in basin above middle floor.
Case 3B – Earthquake Conditions **(Unusual/Extreme)**	• Same as Case 3A Plus OBE Earthquake (Unusual). • Same as Case 3A Plus MDE Earthquake (Extreme). (This condition will not be considered for conceptual design.)
Case 3C – Blocked Drains Operating Condition (Unusual)	• Backfill assumed to match existing topography. Earth pressure to top of backfill. • Blocked drains (hydrostatic pressure, including uplift). • No water inside basins (neglect any "residual" water).

Roof Slab (Stacked Basins) **(Cases 4A Through 4D)**	
Case 4A – Normal Operating Condition **(Usual)**	• Backfill assumed to match existing topography. Earth pressure to top of backfill. • Clear drains (no hydrostatic pressure). • No water inside basin (neglect any "residual" water). • Normal design roof live load = 4800 N/m^2 (100 psf), Minimum roof live load = 0 N/m^2.
Case 4B – Earthquake Conditions **(Unusual/Extreme)**	• Same as Case 4A Plus OBE Earthquake (Unusual). • Same as Case 4A Plus MDE Earthquake (Extreme). (This condition will not be considered for conceptual design.)
Case 4C – Blocked Drains Operating Condition **(Unusual)**	• Backfill assumed to match existing topography. Earth pressure to top of backfill. • Blocked drains (hydrostatic pressure, including uplift). • No water inside basin (neglect any "residual" water). • Normal design roof live load = 4800 N/ m^2 (100 psf), Minimum roof live load = 0 N/m^2.
Case 4D – Construction Condition **(Unusual)**	• Earth pressure up to top of backfill. • Construction surcharge or live load. • No water in basin. • No uplift. • No earthquake. (This condition will not be considered for conceptual design.)

Culverts **(Cases 5A Through 5C)**	
Case 5A – Normal Operating Condition (Usual)	• Backfill assumed to match existing topography near basins. Backfill to match top of new lock wall near locks. Earth pressure to top of backfill. • Hydrostatic pressure. (Saturation level assumed to vary as a straight line from the upper pool to the lower pool.) • Culvert full of water.
Case 5B – Earthquake Conditions **(Unusual/Extreme)**	• Same as Case 5A Plus OBE Earthquake (Unusual). • Same as Case 5A Plus MDE Earthquake (Extreme). (This condition will not be considered for conceptual design.)
Case 5C – Maintenance Condition (Unusual)	• Backfill assumed to match existing topography near basins. Backfill to match top of new lock wall near locks. Earth pressure to top of backfill. • Hydrostatic pressure. (Saturation level assumed to vary as a straight line from the upper pool to the lower pool.) • No water inside culvert.

5. Loading Combinations

The following is a list of the abbreviations and the descriptions of the individual loads required for the analyses. These notations are used in the load combinations.

DL	= Dead Load
LL1	= Live Load (medium to long duration and probability)
LL2	= Live Load (short duration and/or with low probability of occurrence)
OBE	= Operating Basis Earthquake Load
MDE	= Maximum Design Earthquake Load

Live loads refer to all applicable loads other than dead loads. Therefore, live loads include loads such as earth pressure and hydrostatic pressure.

The recommended load combinations, based on EM 1110-2-2104, "*Strength Design of Reinforced Concrete Hydraulic Structures*", are shown below. (H_f = Hydraulic Factor = 1.3.)

Load Combinations:
For the conceptual design, the following load combinations shall be considered:

1. $1.7\ H_f\ [DL + LL1]$
2. $1.3\ H_f\ [DL + LL2]$

In later design phases, the following load combinations shall be considered if only nonsite-specific ground motions have been determined:

3. 1.7 [DL + LL1] +1.9 OBE
4. 1.1 [DL + LL1] +1.25 MDE

If site-specific ground motions have been determined, then the following load combinations shall be substituted for load combinations 3 and 4 above, during later design phases:

5. 1.4 [DL + LL1] +1.5 OBE
6. 1.0 [DL + LL1 + MDE]

6. Stability Criteria

The flotation stability shall be evaluated for the condition in which the drains are blocked. This evaluation shall be completed in accordance with TL 1110-2-307, "*Flotation Stability Criteria for Concrete Hydraulic Structures*". As the blocked drain condition is considered to be an unusual load case, the minimum acceptable flotation safety factor is 1.3.

7. Serviceability

In order to minimize crack widths and improve the durability of the water saving basins, the basin designs shall follow ACI 350R-89, "Environmental Engineering Concrete Structures", where applicable. ACI 350R-89 limits the parameter, z, to 20 kN/mm (115 kips/in). The parameter z is defined in ACI 318-95. In ACI 318-99, the z parameter was replaced by maximum reinforcement spacing limitations. However, ACI 350R-89 has not yet been updated to address this revision. For basin design purposes, both the z parameter and the maximum spacing limitations shall be calculated, and the governing approach shall be used.

8. References

The structures shall be designed in accordance with the criteria and guidance furnished in Corps of Engineers manuals for engineering and design, industry standards, and other technical references as shown in **Appendix A**.

D. Geotechnical Design Criteria

1. Evaluation of Subsurface Conditions

Data from pre-1914 borings, 1940's borings, and any other subsurface information supplied by the Authority will be considered. Information from technical publications describing conditions that were encountered during construction of the current locks will also be considered.

2. Foundations

Foundation designs for the basins and culverts will be developed using the following reference documents as guidelines:

- NAVFAC DM-7, May 1982
- EM 1110-2-2906 Design of Pile Foundations, 15 Jan. 1991
- EM 1110-1-1904 Settlement Analysis, 30 Sept. 1990
- EM 1110-1-2908 Rock Foundations, 30 Nov. 1994
- FHWA-IF-99-025 Drilled Shafts: Construction Procedures and Design Methods, Aug. 1999

Foundation options will be selected based on an allowable total settlement of less than 1 inch for basins with full water load and allowable differential settlement of less than ¾ inch over a distance of 30 feet. The U.S. Army Corps of Engineers' EM 1110-1-1904, *"Settlement Analysis"* states: "A safe limit for no cracking in properly designed and constructed steel or reinforced concrete frame structures is angular distortion of 1/500 (or ¾ inch in 30 feet)." Angular distortion or differential settlement will rarely exceed 3/4 inch if total settlement is 1 inch or less (Peck, Hanson & Thornburn's Foundation Engineering, 2nd edition, 1973).

Of course, settlement estimates will be made during a later stage of design. At this stage, the criteria are only used as a screening tool in selecting foundation options.

3. Temporary Excavation Support

Excavation support will be evaluated in terms of the type best suited to the conditions at this stage of the design. Unusual subsurface conditions, if any, that may affect construction costs, were also identified. The evaluation will be based on the premise that mechanical excavation or controlled blasting will be used in rock at final excavation faces. Guidelines that apply to the work are as follows:

- EM 1110-2-3800 Systematic Drilling and Blasting for Surface Excavations, 1 March 1972
- EM 1110-1-2907 Rock Reinforcement, 15 Feb. 1980
- EM 1110-2-2005 Standard Practice for Shotcrete, 31 Jan. 1993
- FHWA-SA-96-069 Manual for Design and Construction Monitoring of Soil Nailed Walls, Nov. 1996
- FHWA-IF-99-015 Geotechnical Engineering Circular No. 4, Ground Anchors and Anchored Systems, June 1999
- FHWA-HI-99-007 Rock Slopes Reference Manual, Oct. 1998
- NAVFAC DM-7, May 1982

4. Tunneling for Culverts

The feasibility of excavating the culverts connecting the locks and basins using tunnels in rock were evaluated using analysis and design guidelines in the following:

- Hoek, E. and Brown, E.T., Underground Excavations in Rock, 1980
- EM 1110-1-2907 Rock Reinforcement, 15 Feb. 1980
- EM 1110-2-2901 Tunnels and Shafts in Rock, 30 May 1997

5. Earth Pressures

Recommendations for static earth pressures required for basin wall design were based on EM 1110-2-2502 *"Retaining and Flood Walls"*, 29 Sept. 1989. Seismic earth pressures were not evaluated at this stage of design.

6. Dewatering

Requirements for temporary and permanent dewatering were evaluated using TM 5-818-5 *"Dewatering and Groundwater Control for Deep Excavations"*. Dewatering will be required during construction to lower the groundwater level to 3 feet or more below the excavation level in soil or degradable rock and to at least the excavation grade in competent rock. Permanent drainage will be required to remove excess hydrostatic pressures beneath floors and behind walls. Grouting as a groundwater control measure was evaluated using EM 1110-2-3506 *"Grouting Technology"*.

7. Slopes

Stability of temporary and permanent slopes in soil were evaluated using methods described in NAVFAC DM-7, May 1982. Stability of temporary and permanent slopes in rock were evaluated using methods described in FHWA-HI-99-007 Rock Slopes Reference Manual, Oct. 1998. The 1993 CAS study recommendations was also used.

E. MECHANICAL DESIGN CRITERIA

1. General

This section presents the basic design criteria and philosophy for the application, selection, and design of the water saving basin gate valves, actuators, instrumentation, and controls.

2. Application

Valves are provided in the saving basin culverts as an integral part of the lock fill and drain system. Each application includes redundant valves to ensure system availability. Each valve includes bulkhead recesses (emergency valves) to facilitate service and repair. A reasonable arrangement of valves is presented at this concept level. Later stages of design development should optimize the number and size of valves to minimize the overall size, complexity, and cost of the water saving basin system.

3. Selection of Valves, Actuators, Instrumentation, and Control Systems

Valve selection was based on valve type, performance, design, availability, and cost.

Valve types considered included:

- Vertical Lift
- Tainter (radial) – Lock to Basin
- Reverse Tainter – Basin to Lock
- Others

Performance factors were:

- Opening/Closing Time-minimize basin drain and fill time
- Hydraulic Losses
- Sealing

Valve and system design factors included:

- Basin/culvert design interface
- Hoist loads
- Elevation of lift, peak head, surge pressure
- Cavitation potential & controlled entrained air
- Construction/Fabrication
- Materials of construction
- Corrosion resistance
- Cathodic protection
- Maintainability

Actuation, instrumentation, and control systems included:

- Mechanical
- Electric
- Hydraulic
- Hybrid
- System operating time and complexity shall be minimized.

Availability:

- Standard model and design – no experimental prototype if possible
- Standard construction & materials
- Readily available components and spare parts

Cost factors included:

- Initial valve, activation system, and control costs
- Maintenance and repair costs
- Spare parts cost
- Impact on overall water saving basin cost
- Shipping and Installation

4. Design Philosophy

Valve, valve actuation, and valve control system design philosophy shall be based on redundancy and simplicity. Systems shall be constructed of standard materials and products to the fullest degree possible, and shall be easily maintained and serviced.

5. References

Valves and actuating systems shall be designed in accordance with the Corps of Engineers manuals and industry standards, as applicable, listed in **Appendix A**.

F. ELECTRICAL DESIGN CRITERIA

1. General

This section presents basic information regarding the electrical criteria to be utilized in reviewing appropriate power and control systems associated with the water saving basin. All of the recommended system will be items that are readily available if possible to reduce costs. The maintenance requirements and service life for these items will also be known. Items that will be unique to this project will be detailed sufficiently for accurate costs to be obtained from various manufacturers.

2. National Electrical Code

All systems will be in accordance with the 1999 edition of the National Electrical Code (NFPA 70) with personnel safety and long-term operation of equipment of primary concern.

3. Power Distribution

All equipment requiring power including pumps, motors, motorized valves, lighting, etc. will use local distribution voltage levels. For the Atlantic side, 6900V incoming feeders are reduced to 480V, 60Hz, 3-phase for the existing locks machinery. For the Pacific side, 2400V incoming feeders are reduced to 480V, 60Hz, 3-phase for the existing locks machinery. Direct current where appropriate and applicable. Direct current would only be used for control (not power distribution) and would be for reliability in a specific application of a piece of equipment, not a general rule of our design.

4. Control systems

All control systems will be systems that are controllable locally and remotely, from one or more locations as required. Water level indicators, valve status, and other required monitoring will utilize solid-state sensors to maximize accuracy and life expectancy. The solid-state sensor design and parts must be reliable.

5. Redundancy

All electrical systems will be reviewed with the importance of redundancy in mind. The main purposes for system redundancy are for system reliability and safety. Where components requiring frequent maintenance cannot be avoided, redundant systems will be designed to assure operations are not interrupted.

6. Ease of Maintenance

All systems, including power distribution, controls, and lighting will be reviewed for ease of maintenance. High quality, long lasting equipment will be recommended to reduce failures, service and maintenance requirements.

IV. FEATURES LAYOUT

A. Study Options

The study options for the project were:

- OPTION 1 – Three-lift lock structure – side by side water savings basins to one side of lock – 50% water savings,
- OPTION 2 – Two-lift lock structure – side-by-side water savings basins to one side of lock – 60% water savings,
- OPTION 3 – Three-lift lock structure – side-by-side water savings basins on both sides of lock – 60% water savings, and
- OPTION 4 – Two-lift lock structure – stacked water savings basins on one side of lock – 50% water savings.

The features layout portion of the study would include (for each option and ocean side):

- determination of preliminary lock and basin layout plan considering general, hydraulic, structural , and geotechnical considerations,
- determination of preliminary lock and water saving basin dimensions and elevations, and
- development of layout and section views as needed to show relationships of locks to basins.

B. Features Layout Considerations

1. General Layout Considerations

a) Maintenance and Geometric Considerations

The layout of the Water Savings Basins (WSB's) and their relationship to the locks was developed to allow the most convenience for maintenance operations and to minimize geometric conflicts. The WSBs were set behind the roller gate recesses for the locks to reduce interference with the gates and to provide room for operations. In addition, room was needed for the conduit valve recesses and to allow access to the conduit valves during maintenance. This offset also reduced the slope of the conduits between the locks and basins to approximately 10%. Since the conduits will have to be inspected and maintained on a regular basis, moving the WSB closer to the locks would steepen the slope and make inspection of the conduits more difficult and on a steeper slope. Nonetheless, if cost concerns are paramount, steeper scopes could be used with significant cost savings realized due to a smaller overall footprint.

The basins were separated to allow access around the perimeter of each. The roads parallel to the WSB's and locks will be level across the length of the WSB and will provide a substantial staging area for maintenance operations. This will assist during the routine cleaning and maintenance operations since small truck mounted cranes will be able to access the perimeter of each basin to lift maintenance equipment and supplies into the basin. This will also allow each basin to settle and react independently from the other basins. This will reduce the probability of a foundation problem on one WSB causing problems to an adjacent basin. This separation will initially cost more due to the increased footprint and the loss of a common wall between basins. However, this will allow ACP to do maintenance work on a basin without affecting operations in the other basins.

b) Constructability Considerations

For ease of construction, the WSB conduits should be constructed on a constant slope from the junction with the F/E culvert to the furthest basin away. The connection point at the furthest basin will be ~ 1 diameter below the basin floor. The inner basins will then have longer vertical risers to meet the constant slope. In other words, the conduits would be constructed side-by-side on a constant slope to allow one conduit to act as formwork for the adjacent conduit.

2. Hydraulic Layout Considerations

a) Tie-In to Lock Filling/Empting (F/E) System

Numerous hydraulic considerations were important to the preliminary features layout. One such consideration was the geometry of the possible connection to the lock filling/emptyting (f/e) system since this system was outside the scope of work. The conduit was assumed to meet the lock wall and tie into the lock f/e system at a soffit elevation of 4.57m (15') below the lock chamber floor based on direction received from ACP. For symmetry of flow in both directions, and for ease of construction, it was also assumed that the conduits would perpendicularly enter the lock f/e system.

b) Conduit Spacing

The conduits will be equally spaced along the nominal 1500' lock length (gates closed). 4 conduits – 300' 2 Conduits – 500'. The conduit layout will be offset from the center of the lock, with the middle of the conduit group (depending on the # of basins) entering the locks at the equidistant points.

c) Conduit Alignment

At this preliminary concept study phase, conduits perpendicular to the locks and basins would be sufficient. This was mainly done for ease of construction and cost-effectiveness (due to shorter conduit lengths, lower form losses, etc.). This arrangement also provides symmetric flow geometries when considering water transfers in either direction. Providing advantageous geometries for flow in one direction would be offset by greater losses when the flow reverses. Although some of the German systems do have conduits with varying alignments, these alignments are mainly a function of the lock f/e system chosen. For example, one German plan provided by ACP had conduits that entered the lock f/e system perpendicularly but the conduits then turned at an angle so that each conduit entered the water saving basin in the center. But again, the main reason that this was necessary was the design required that the conduits enter the lock at the manifold of the lock f/e system and not the f/e culverts themselves. Other systems in Germany in which the basin conduits tie into the lock f/e culverts have perpendicular conduits, so the alignment chosen has a precedent. Ultimately, the alignment may have to be altered, depending on the type of lock f/e system chosen, but as this was not part of the scope of this study, a simplified perpendicular alignment was selected.

d) Conduit Shape

Numerous shapes were considered for the water saving basin conduits. Circular conduits are hydraulically efficient, but more expensive to build. They also require more expensive transition sections when entering the valve recesses, which are typically rectangular. Based on the project team's experience and judgement, pre-cast concrete conduits with diameters up to 5.2 m (17') would be economical. Larger sizes would require cast-in-place construction. This would favor the use of square sections. Based on preliminary hydraulic calculations, the conduit sizes would be 6.7 m (22') to 8.5 m (28'). Therefore, the square conduits were chosen for this conceptual level study for constructability and cost-effectiveness concerns.

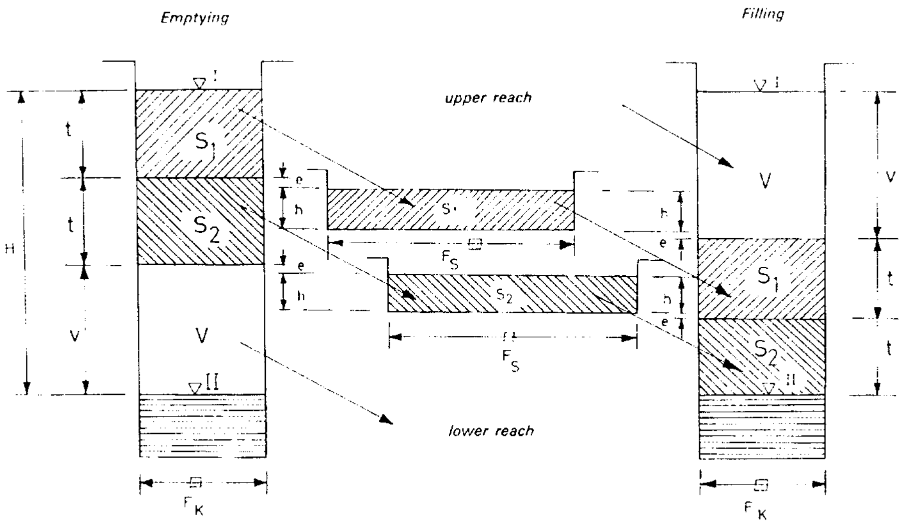
e) Outlet Configuration

The conduits are currently envisioned to have vertical morning glory outlets at the basins. The conduit will have a vertical exit with the inside wall (closest to the lock) being vertical for one diameter before transitioning to a sloped conduit until reaching the lock F/E culvert near the valve/lock wall. Again, this outlet type was chosen for ease of construction and cost concerns. However, sloped basin floors and wide slots could be investigated in later design phases. The small differences in loss coefficients for the present and any possible configurations would likely have no significant impacts on the geometry of performance of the future system as presently conceived.

f) PIANC Design Guidelines Considerations

In addition to the hydraulic considerations listed above, PIANC literature was found to provide valuable design guidelines in helping to determine the size and elevations required for the water saving basins. The PIANC Supplemental Bulletin No. 55, "Final Report of the International Commission For The Study of Locks", provided guidance in the determination of the required water saving basin area to meet the target water saving percentages. As one can see from the upper graphic in **Figure IV.1**, the principle of water savings is built upon the segmentation of total lift height as explained in **Chapter I, Figure I.5**. However, the PIANC guidelines introduce a few new factors into the analysis. One such factor is "e" which is used to describe a residual depth left in the basin or lock to help optimize operation time (since toward the end of the operation equalization is occurring more slowly due to reducing head differences). The PIANC report did list an example "e" of 15 cm (~6 in.), but this was not used in our conceptual study for reasons that will be explained more fully later. Suffice it here to say that the 15 cm example is based on systems in Germany which are much smaller in scale than the Post-Panamax locks and that the much larger volumes of water necessitate a study of an optimal "e" for this system. However, determination of an optimal "e" factor will be highly dependent on how quickly the valves can be operated in the final system, how much time can be allowed for filling and emptying to meet the ACP's operational constraints, and other design issues that have not been determined yet. Based on these considerations, a detailed study to determine the optimal "e" should be done at a later design stage once some of these issues have reached conclusions.

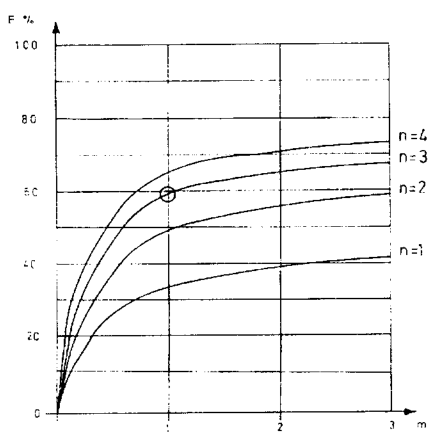
However, two factors that were immediately applicable were that of the basin area/lock area ratio-"m" and number of basins –"n". As one can see in the lower left graph within **Figure IV.1**, the optimal "m" for each "n" corresponds to approximately one (m=1). As basin area (hence, "m") increases, the water saving percentage also increases, but at a declining rate. Also, as "m" increases along with water saving percentages, the amount of time needed to drain the basin back to the lock would also increase since the initial static head will be lower. As with the "e" factor, selecting an optimal "m" values involves a complicated mix of hydraulic design, operational, foundational, and construction cost considerations and is better left to a later design stage. Therefore, m = 1 was used for this concept study, and it is unlikely that the ultimate design would depart significantly from this value. In conclusion, given the scale of this project in relation to others and the importance of the "inertial head" term at this scale, unique studies to determine optimum "e" and "m" will be necessary, once the lock design and foundation conditions have been established.



Legend

I. Upper reach level
II. Lower reach level
S. Water saved
V. Water used
F_k. Surface area of lock chamber
F_s. Surface area of water saving basin

Fig. 5 - Principle of water saving basins



Legend

$t.F_k = h.F_s \quad t = \frac{F_s}{F_k} h \qquad t = m.h$

$H - nt = v = t + 2e + h$

$H = h + t(1+n) + 2e$

$= h + mh(1+n) + 2e$

$= h[1 + m(1+n)] + 2e$

$h = \frac{H - 2e}{1+m(1+n)}$

$E = \frac{n.t}{H} = \frac{n.m.h}{H}$

$E = n.m.\frac{h}{H} = \frac{n.m.(H-2e)}{H[1+m(1+n)]}$

if H >> 2e :

$h = \frac{H}{1+m(1+n)}$

$E = \frac{n.m}{1+m(1+n)}$

Figure IV.1 – Figures Taken From PIANC Literature Describing WSB's

g) Operational Considerations

The operational conditions to consider for the features layout design were:

- Gatun Lake Water Levels,
- Atlantic Ocean Water Levels,
- Pacific Ocean Water Levels, and
- Lockage Length Combinations.

Based on guidance in the RFP and preliminary calculations completed by ACP personnel, it was determined that the new features layout for each option should be designed to incorporate all variability in lake levels, ocean levels, and lockage lengths with no water spillage. If the systems were designed to account for all of this variability the theoretical water savings percentages would be acheived. Otherwise, the water savings would be less than optimum. The combinations of water levels (given in Precise Level Datum - PLD) and lockage lengths to be included in the features layout design follow.

(1) Water Levels

Based on statistics from the raw data set (see **Chapter II**) and preliminary calculations completed by ACP personnel, the following elevations were used to describe water levels in the features layout design:

Table IV.1 Design Water Levels Above PLD

		Minimum	Mean	Maximum
Gatun Lake	(m)	+ 23.93	+ 25.91	+ 26.82
	(ft)	+ 78.5	+ 85.0	+ 88.0
Atlantic Ocean	(m)	- 0.38	+ 0.06	+ 0.56
	(ft)	- 1.25	+ 0.20	+ 1.85
Pacific Ocean	(m)	- 3.44	+ 0.30	+ 3.60
	(ft)	- 11.3	+ 1.0	+ 11.80

(2) Lockage Length Combinations

Based on the discussion (see **Chapter II**) and guidance provided by ACP, the range of lockage lengths experienced would be 426.7 m (1400'), 457.2 m (1500'), and 487.7 m (1600'). The eight possible combinations of lockage lengths for the two-lift alternatives (Options 2 & 4) can be seen in **Figure IV.2**. The sixteen possible combinations of lockage lengths for the three-lift alternatives (Options 1 & 3) can be seen in **Figure IV.3**.

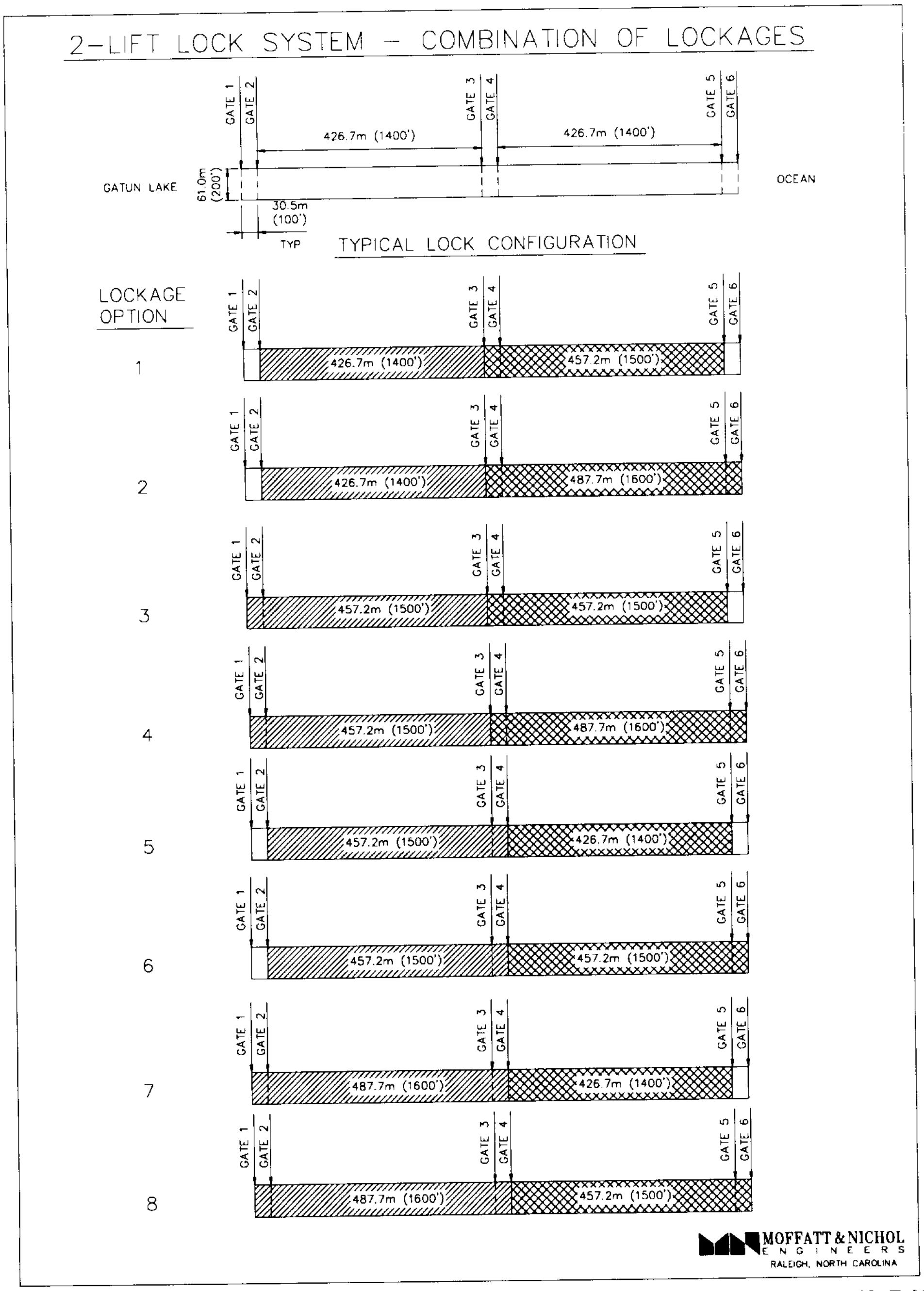


Figure IV.2 – Possible Lockage Combinations – Options 2 & 4 (2-Lift)

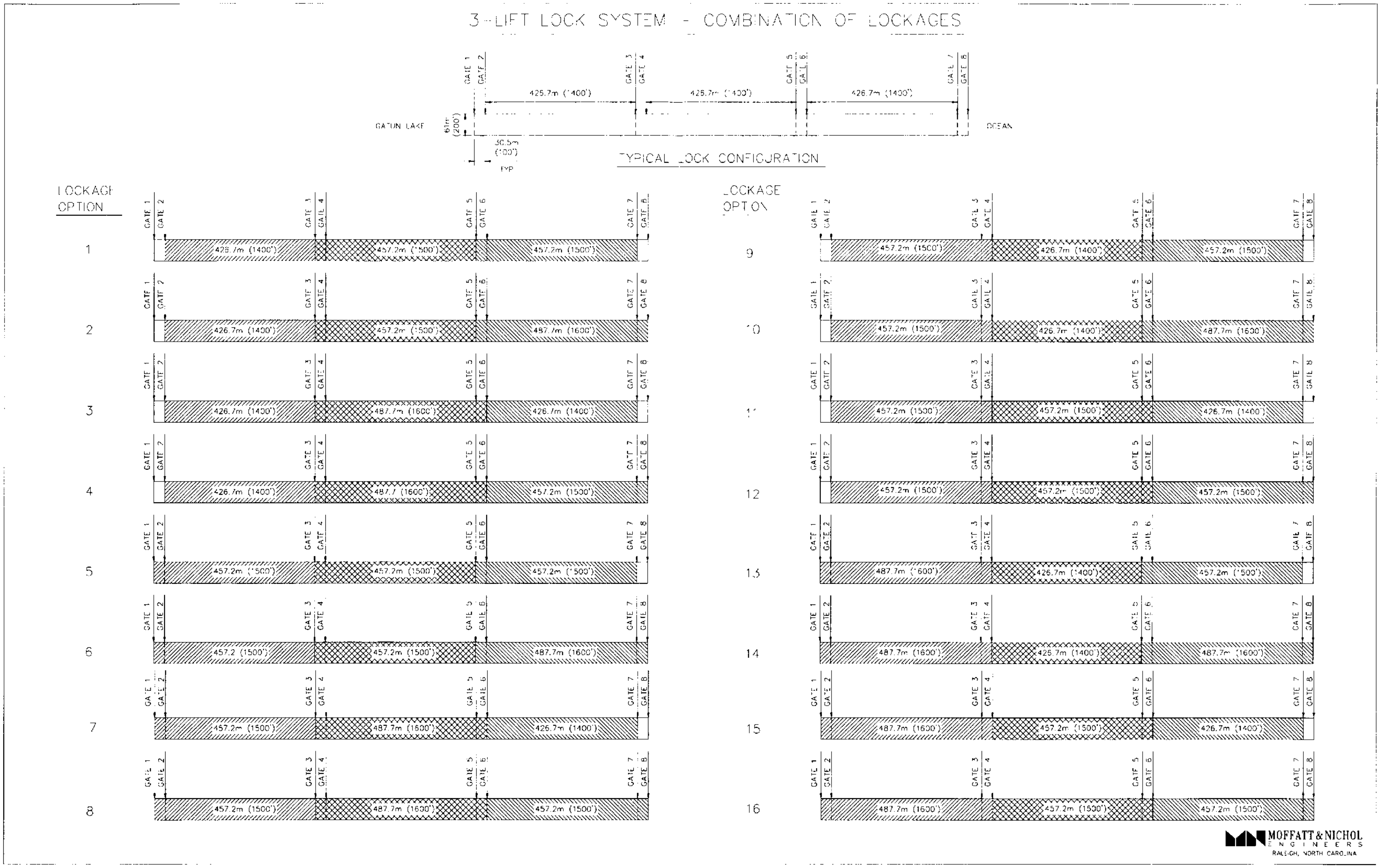


Figure IV.3 – Possible Lockage Combinations – Options 1 & 3 (3-Lift)

3. Structural Layout Considerations

a) Atlantic Side

On the Atlantic site, various arrangements and numbers of water saving basins were investigated for use in conjunction with new locks constructed along Alignment A-2. Three configurations of water saving basins were considered, with features common to all configurations. For example, all basins included transverse interior walls. The interior wall for Options 1 and 2 divided each basin into two compartments. This interior wall provides the ability to continue operation with half the basin while maintenance is performed on the other half. To reduce wave heights, openings (fitted with removable maintenance bulkhead gates) are recommended in the interior walls. In order to provide improved access for operations and maintenance personnel, the basins were also separated by a space of 16 m (52 ft.). In order to provide room for the lock roller gate monoliths, the closest basin wall was placed 125 m (410 ft.) from the centerline of the closest new lock lane. Square conduits connect the basins to the new lock chambers. The conduit connections to each basin are located symmetrically with respect to the basin centerline. The conduits are controlled by valves in the conduit valve monoliths that are adjacent to the new lock. The valves are located close together and near the lock so that they can be grouped in the monoliths. These monoliths are centrally located and well situated for maintenance access.

b) Pacific Side

The structural arrangement and layout is identical on the Pacific side, except that Alignment P-1 was used.

4. Geotechnical Layout Considerations

a) Atlantic Side

(1) West-Side Basins

The available information described above and summarized in **Figure 2 in Appendix D – Atlantic Report** provides an adequate basis for interpreting subsurface conditions for water saving basins on the west side of the proposed locks. The existing ground level rises from near sea level at the north end of the proposed locks to about El. 30 meters (98 ft.) at the south end adjacent to Gatun Lake. Variable consistency soil and rock fill and natural alluvial, marine and/or residual soils are present from the existing ground level to depths of 5 to 18 meters (16 to 59 ft.). The fill soils generally consist of clay and silt with minor amounts of gravel-, cobble-, and boulder-sized rock fragments. Fill materials typically range in consistency from very soft to stiff, and their consistency varies spatially in unpredictable ways because of extensive grading and re-grading of the area. Soils derived from weathering of the underlying rocks (residual soils) are typically firm to very stiff.

Rock (including weathered rock and sound rock) underlies the soils. The level of the rock is expected to range from about El. –7 meters (-23 ft.) on the north end of the proposed lower lock to about El. 12 meters (39 ft.) on the south end.

(2) East-Side Basins

Little specific information is available on subsurface conditions for water saving basins on the east side of the proposed locks. The conditions must be inferred from general geologic and topographic conditions and the information available from nearby boreholes. The types and consistency of soil and rock are interpreted to be the same as described above for the west side of the proposed locks. The thickness of soil (above weathered rock or rock) is expected to be in the range of 1 to 15 meters (3 to 50 ft.), with a typical thickness of 7 meters (23 ft.). Fill is expected to be present in some locations, but not as thick and extensive as on the west side of the proposed locks. With the topography rising toward the east of the proposed locks, the elevation of the top of weathered rock or rock is also expected to gently rise toward the east. This inference is supported by a top of rock contour map presented as Plate V, Paper No. 11 (IEC. 1915), which shows the top of rock surface on the north end of the east lock to rise from about -27 meters (-90 ft.) MSL at the centerline to about –9 meters (-30 ft.) MSL east of the centerline.

(3) Groundwater

Based on observations described in Paper No. 11 of the conference cited above, groundwater levels on the west side of the proposed locks are interpreted to range from sea level on the north end of the lower lock to El. 26 meters (85 ft.) (the level of Gatun Lake) on the south end of the upper lock. On the east side of the proposed locks, groundwater levels adjacent to the lock walls are expected to be the same as on the west side of the proposed locks, but are expected to rise to the east, as the general topographic level rises.

b) Pacific Side

(1) General

As shown on **Figure 1 in Appendix D – Pacific Report**, the northwestern end of the proposed P-1 alignment starts on the flank of a small, isolated hill with about 50m (157 ft.) to 60m (197 ft.) of relief and ends in lowlands flooded by Miraflores Lake. Topographic relief along the alignment is about 30m (98 ft.) to 35m (115 ft.), ranging from about elevation 35m (115 ft.) on the flank of the hill to less than elevation 4m (13 ft.) in the lowlands.

The proposed alignment appears to be underlain by basalt flows, rocks of the La Boca Formation and overburden derived as the weathering product of these rock types. At least one fault zone also appears to intersect the proposed alignment. **Figure 3 in Appendix D – Pacific Report** is a geologic plan showing structural and lithologic contacts modified from those shown on the ACP boring plan. As shown on this figure, the basalt is projected to occur from the northwest end of the alignment near Station 6+300 to about Station 6+885 and between Station 7+135 to about Station 8+295. The La Boca Formation is projected to occur along the alignment between Station 6+885 and about Station 7+135 and between Station 8+295 to the end of the proposed P-1 alignment (near Station 8+550). The approximate fault zone is projected to occur along the alignment between Station 7+000 and Station 7+135 in the La Boca Formation.

In areas underlain by basalt, the overburden generally consists of dark brownish-red to brown, soft to stiff, plastic, locally saprolitic silty and sandy clay with minor amounts of organic material. The clay locally contains weathered and unweathered gravel to boulder-size fragments of basalt. As indicated in the borings, weathered rock and sound rock generally occur variably between less than 0.5m (1.6 ft.) and 15m (49 ft.) below ground surface within the vicinity of the P-1 alignment. The term "sound rock" is used in the descriptions in the borehole logs. We have interpreted "sound rock" to mean fresh to moderately weathered rock.

The depth of residual soil over the basalt is typically 0.5m (1.6 ft.) to over 8m (26 ft.) thick. The weathered basalt is described as being soft, weak, locally altered and spheroidally weathered. This unit is typically sampled as either very fractured rock or unconsolidated sand- to gravel-size fragments with relatively poor sample recovery from the boreholes. The sound basalt is fine- to medium grained, hard, strong, and moderately to very fractured by steep to vertical joints. Sample recovery was generally good. Unconfined compressive strengths were reported for the basalt to range from 19 MPa to 180 MPa (2740 psi to 26,150 psi) in the 1947 data included in Appendix B. The joint surfaces are typically infilled or coated with calcite and chlorite, and are locally slickensided. Flow fronts were crossed in one borehole, which refers to crossing different basaltic "formations", but no comment on differential weathering, presence of ash layers or other compositional change was noted in the log. Geologic logs for four borings advanced within the basalt (M-10, M-28, M-35, and M-36) around Station 7+500 to about 7+600 indicate the presence of highly weathered basalt underlying fresh basalt. The weathered basalt is described as soft, weak, highly fractured and slickensided, and is interpreted to represent a shear zone or fault zone.

Golder was provided boring logs for three holes (M-242, M-9, and M-247) that were advanced in the La Boca Formation where it is projected to occur near or within the projected fault zone between station 7+000 and station 7+135. These borings were drilled from a barge on a body of water that filled a drainage leading to Miraflores Lake. The depth of water where these borings were drilled ranged from 8m (26 ft.) to 18m (59 ft.). The overburden encountered below the lake water is described as brown to black, soft, plastic clay with local concentrations of sand and gravel-size rock fragments. Assuming the body of water was created by excavation related to lock construction, the original depth of the overburden in this area could have locally been over 20m thick. As shown on **Figure 2 in Appendix D – Pacific Report**, a thin layer of muck (referred to as lacustrine mud on **Figure 2 in Appendix D – Pacific Report**) was encountered at the water/overburden interface. Weathered rock and sound rock is significantly deeper in areas underlain by the La Boca Formation than that observed for the basalt. Depths to weathered rock in the La Boca generally occur between about 10m (33 ft.) to 30m (98 ft.) below ground surface. Sound rock was not encountered in most of the borings advanced in the La Boca Formation. This differential weathering is likely a result of the highly contrasting rock types between the basalt and the sedimentary strata of the La Boca. The logs indicate that shale is the dominant rock type in this area and is described as dark green to gray, sandy, locally carbonaceous, soft, and moderately to very fractured. The shale is locally interbedded with fine-grained, soft, highly fractured "conglomerate". The original field geologist classified the shale as Cucaracha Formation; during 1966, the shale was reclassified as La Boca Formation.

One boring advanced within the La Boca Fault zone (M-9) shows a depth to the top of weathered rock of 17m (56 ft.). The hole was advanced to about 63m (207 ft.) and sound rock was not encountered. This boring encountered andesite and basalt boulders and fragments overlying highly weathered "agglomerate". The volcanic agglomerate was sampled as gray-black, hard, and highly fractured clayey silt and rock. Between about 42.6m (140 ft.) below ground surface to the bottom of the hole, only fine cuttings and slickensided rock fragments were recovered during drilling. This material is interpreted to represent a fault zone and the rock fragments are described as being "not unlike Cucaracha". The western-most two borings, M-347 and M-348, shown on **Figures 1 and 3 in Appendix D – Pacific Report** indicate that agglomerate similar to that encountered in M-9 was observed. Although these borings were only advanced to a depth of 12m (39 ft.) to 14m (46 ft.), this material might represent the same geologic conditions encountered in M-9 at depth.

In areas underlain by the La Boca Formation between Station 8+295 and the end of the proposed P-1 alignment, the overburden is described as reddish brown to dark gray, soft, plastic clay with local concentrations of sand and gravel-size rock fragments. As indicated in the borings, weathered rock and sound rock generally occur between about 2m (7 ft.) to 18m (59 ft.) below ground surface. Two of the borings in this area (M-161 and M-29) appear to have been through the contact between the basalt and the La Boca Formation. These borings show an interfingered contact relationship of the basalt and the La Boca, as schematically shown on **Figure 2 in Appendix D – Pacific Report**. The La Boca Formation in this area is characterized by shale and sandstone sequences. The shale is described as gray-black, soft to moderately hard, variably sandy, carbonaceous, and calcareous. The shale is moderately fractured to brecciated with locally slickensided surfaces. The sandstone is described as gray to black, fine-grained and shaly, hard, locally carbonaceous to "coaly" and calcareous, and highly fractured to brecciated. The rock is described as having a slightly "baked" or hardened appearance. Where the rock is not brecciated, bedding planes inclined at about 15 degrees were noted. Pyrite is present in the shale and sandstone horizons.

The description for both lithologies indicates that the sedimentary strata have likely experienced contact metamorphism at the basalt flow front. Brecciation might also be related to locally intrusive effects of the basalt flows. The original field geologist classified this material as Culebra Formation; during 1966, this material was reclassified as La Boca Formation.

Fill material was described in the two borings advanced in 1956, CT-8 and CT-9. This material is described as loosely consolidated fill comprised of red and black, low to moderate plasticity and dry strength, and high water content clay and boulder-size basalt fragments. Golder did not have access to information at the time of this report that would allow an evaluation of the extent of fill material in this area.

The borehole information provided does not include data on groundwater levels.

(2) Northeast-Side Basins

The subsurface conditions anticipated for the water saving basins proposed northeast of the P-1 alignment are based on our inference of general geologic and topographic conditions and the information available from nearby boreholes. Because the structural and lithologic conditions are variable in this area, the subsurface soil and rock conditions also will vary. The basins in the area around the highest lock are interpreted to occur primarily in rocks of the La Boca Formation and the La Boca Fault zone. Overburden soil (above weathered rock or sound rock) in this area could be over 20m (66 ft.) thick and is expected to generally consist of soft clay. A thin layer of lacustrine mud might also be present in submerged areas. Fill is expected to be present in some locations. Sound rock was not encountered in borings advanced in this area and is expected to occur significantly below the proposed range in floor elevations for the water saving basins. The weathered rock that occurs below the overburden consists of highly fractured, soft, weak shale, agglomerate and fault gauge. Groundwater flow in this unit might be significantly greater than the surrounding basalt due to extreme fracturing relative to adjacent rock units.

The basins around the middle and lowest locks are interpreted to occur in basalt. Overburden soil in this area is expected to be about 0.5m (1.6 ft.) to 11m (36 ft.) thick, with a typical thickness of about 2.5m (8.2 ft.) and is typically soft to stiff clay. The top of sound rock occurs between about elevation 0m up to greater than 21m (69 ft.), with the top of sound rock surface projected to generally slope upward from the southwest to the northeast side of the proposed basins. The top of sound rock is generally expected to occur above the proposed range in floor elevations for the water saving basins. The basalt in which the basins will occur is generally fresh, hard, strong and moderately fractured by steep to vertical joints. Local shearing/faulting in the basalt might create spatially restricted zones of highly fractured, softer, weaker rock such as around Station 7+500.

(3) Southwest-Side Basins

Little specific information is available on subsurface conditions for water saving basins on the southwest side of the proposed locks. Because the geology in this area consists of a complex relationship of sedimentary and igneous sequences, shearing/faulting, and resultant differential weathering, inference of subsurface conditions in areas with limited to no existing data cannot be made with confidence. As shown on **Figure 1 in Appendix D – Pacific Report**, the ground surface becomes more irregular with greater relief southwest of the P-1 alignment. Residual soils are expected to occur at ground surface, with weathered and sound rock beneath the soils. Fill material is generally not expected to occur in this area, unless construction subsequent to the Miraflores Locks was performed of which we are unaware. The depth to weathered or sound rock might vary significantly over short horizontal distances should different geologic formations and/or fault zones occur within the footprint of the water saving basins. Based on existing borehole data in the area, depth to rock could be as shallow as ground surface or could be as deep as 30m (98 ft.) or greater.

Due to the lack of subsurface data on the southwest side of the locks, we recommend assuming for preliminary design that the subsurface conditions on the southwest side of the P-1 alignment are similar to those described above for basins on the northeast side of the P-1 alignment.

(4) Groundwater

No information on groundwater conditions was provided in the boring logs. Although information on the depth to groundwater in this area was not available, based on our experience working in similar geologic environments, groundwater in the fractured rock will likely have preferential flow paths. Due to extreme fracturing associated with the La Boca Fault zone and significant rheologic contrast between the shale of the La Boca and the basalt, we consider that preferential weathering will occur within the fault zone and along lithologic contacts, creating preferential flow paths for groundwater.

C. Features Layout Design

1. General

With the layout considerations now known, the features layout design for the four (4) options could now commence.

2. Hydraulic Features Layout Design

a) Design Procedures

As stated before, in order to ensure that the water saving basins will function under all anticipated variation in water levels (Pacific and Atlantic Oceans, Gatun Lake) and lockage lengths, an analysis was needed to determine the range of lock lifts expected during all combinations of these water levels and lockage lengths.

To match the ranges calculated by ACP staff reported on the "Panama Canal Post-Panamax Lock Elevations Diagram" shown in **Figure II.20**, we used the same procedure outlined in the "1910 Annual Report of the Isthmian Canal Commission" by the Assistant Chief Engineer, H.F. Hodges who derived a simple formula to calculate the lifts in a multiple lift lock based on available head differential and the plan area of the chambers.

For a 3-Lock System the equations were as follows:

$$\text{LOCK 1 LIFT} = \frac{(LAKE\ LEVEL - SEA\ LEVEL)}{\left[\left(\dfrac{LOCK\ 1\ AREA}{LOCK\ 3\ AREA}\right) + \left(\dfrac{LOCK\ 1\ AREA}{LOCK\ 2\ AREA}\right) + 1\right]},$$

$$\text{LOCK 2 LIFT} = \frac{(LAKE\ LEVEL - SEA\ LEVEL)}{\left[\left(\dfrac{LOCK\ 2\ AREA}{LOCK\ 3\ AREA}\right) + \left(\dfrac{LOCK\ 2\ AREA}{LOCK\ 1\ AREA}\right) + 1\right]}, \text{ and}$$

$$\text{LOCK 3 LIFT} = \frac{(LAKE\ LEVEL - SEA\ LEVEL)}{\left[\left(\dfrac{LOCK\ 3\ AREA}{LOCK\ 1\ AREA}\right) + \left(\dfrac{LOCK\ 3\ AREA}{LOCK\ 2\ AREA}\right) + 1\right]}.$$

Likewise, for a 2-Lock System:

$$\text{LOCK 1 LIFT} = \frac{(LAKE\ LEVEL - SEA\ LEVEL)}{\left[\left(\dfrac{LOCK\ 1\ AREA}{LOCK\ 2\ AREA}\right) + 1\right]} \text{ and}$$

$$\text{LOCK 2 LIFT} = \frac{(LAKE\ LEVEL - SEA\ LEVEL)}{\left[\left(\dfrac{LOCK\ 2\ AREA}{LOCK\ 1\ AREA}\right) + 1\right]}.$$

The plan areas of the lock chambers would have numerous combinations based on the lockage lengths and gate recesses. The gate recesses were included in the area calculations since they also fill with water during lockage operations. ACP estimated gate recess dimensions of 16 m (52.49') wide x 80 m (262.47') deep which were based upon a pre-dimensioning utilizing the maximum head of water, grade 50 structural steel, and double skin gates using LRFD and SAP2000. To estimate how many gate recesses would be associated with the 426.70 m (1400'), 457.18 m (1500'), 487.66 m (1600') lockages, ACP used the following convention in their analyses: 426.70 m (1400') Lockage – 1 Recess, 457.18 m (1500') Lockage – 2 Recesses, and for the 487.66 m (1600') Lockage – 3 Recesses. This was based on the fact that for a 426.70 m (1400') lockage, one-half of two gate recesses (at the ends of the 426.70 m (1400')) would be filled – hence one (1) total gate recess. Likewise, for a 457.18 m (1500') lockage, two (2) one-half gate recesses would be filled (again, at the ends) in addition to one (1) full gate recess in the middle for a total of two (2) total gate recesses. Similarly, for the 487.66 m (1600') lockage, an additional full gate recess in the middle would be required for a total of three (3) total gate recesses. Please see **Figures IV.2** and **IV.3**, when envisioning these scenarios, as they will help to show the gate recesses involved for each lockage length.

Therefore, using these equations, the maximum, mean, and minimum water levels for the oceans and lake, and the various plan areas for the locks (based on the lockage lengths and gate recesses), the associated lift heights (for each lock) for all combinations of water levels and lockage lengths could be calculated. Adding or subtracting these lift heights from the tide or lake elevations then yields the maximum and minimum equalization water levels in the locks for all of the combinations.

Once the lift heights for each lock were calculated, the starting and equalizing water elevation in each water saving basin for an option could also be determined based on the following logic. First, the optimal "m" factor for each option was assumed to be 1 based on the PIANC guidance and reasoning outlined in **Chapter IV, Section B.2.f**. Based on this fact and the discussions in **Chapter I, Section D**, the number of basins required for each option could be calculated by the equation:

$$\text{Water Savings Percentage} = \frac{n}{(n+2)}$$

For example, Option 2 – Two-lift lock structure – side-by-side water savings basins to one side of lock – 60% water savings;

$$60\% = \frac{n}{(n+2)}$$

$$n = 3.$$

Following the same procedure, the number of water saving basins for each option can be calculated:

- OPTION 1 – Three-lift lock structure – side by side water savings basins to one side of lock – 50% water savings **– 2 full size water saving basins on one side of lock,**
- OPTION 2 – Two-lift lock structure – side-by-side water savings basins to one side of lock – 60% water savings **– 3 full size water saving basins on one side of lock,**
- OPTION 3 – Three-lift lock structure – side-by-side water savings basins on both sides of lock – 60% water savings **– 3 half size water saving basins on each side of lock**, and
- OPTION 4 – Two-lift lock structure – stacked water savings basins on one side of lock – 50% water savings**– 2 full size water saving basins on one side of lock.**

Once the number of basins was known, the "step height" or "t" as shown in **Figure IV.1** could be calculated. Knowing the number of basins for each option and that m=1, the total lift height would have to be divided into n+2 (n = number of basins) equal "step heights" for the entire system to work by gravity flow alone. Given the step heights, the starting and equalizing elevation for each operation could then be calculated by subtracting the step height from the lake elevation as the first equalization elevation. Subtracting the step height again would give the starting water surface elevation in the first basin as well as the equalization level of the second water saving basin. The process could continue for each basin and for each lock to determine the operating elevations for a given combination of lake and ocean levels as well as lockage lengths.

For example, for a 2-lock system with 3 water saving basins (Option 2) the cross-section would appear as seen in **Figure IV.4**.

Again, the step height can be calculated by $= \dfrac{LIFT}{(n+2)}$ $\quad n = \#$ of WSB,

then for WSB1, Equalizing ELEV = Lake Elev – Step Height
Starting ELEV = Equalizing Elev – Step Height,

for WSB2, Equalizing ELEV = Starting ELEV for WSB1
Starting ELEV = Equalizing ELEV – Step Height, and

for WSB3, Equalizing ELEV = Starting ELEV for WSB2
Starting ELEV = Equalizing ELEV – Step Height.

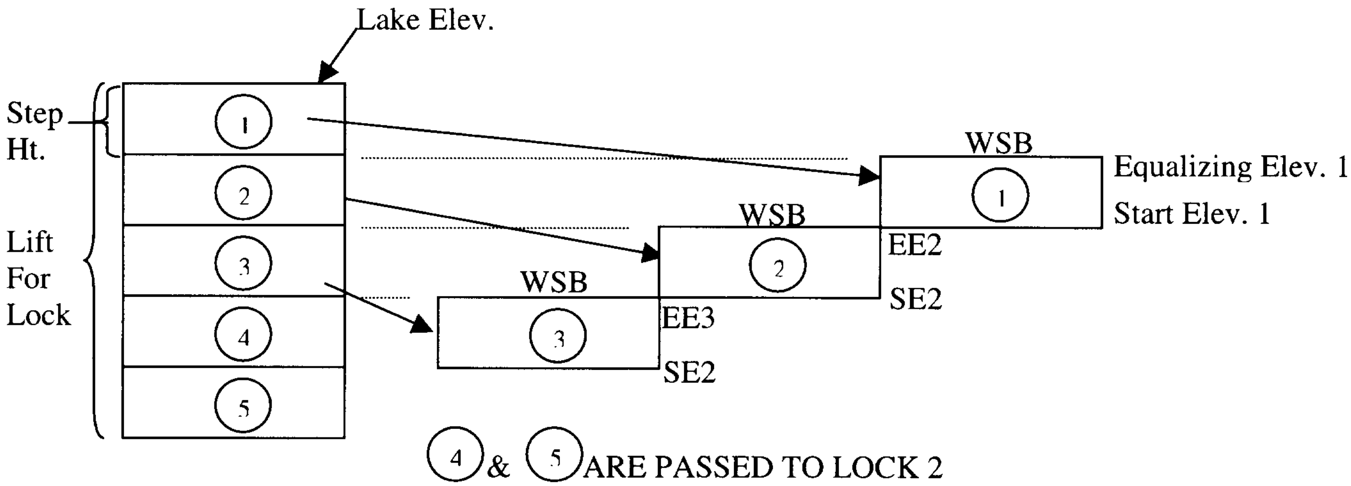


Figure IV.4 – Conceptual Section Showing Calculation of WSB Operating Elevations

In using this procedure, the calculated lift heights provide the operational water levels in the locks, while the calculated step heights provide the operational water levels in the water saving basins. When completed for all water level and lockage length combinations this procedure yields the minimum and maximum operating levels in both the locks and water saving basins. The minimum starting elevation for each basin provides the floor elevation required so that the theoretical water saving percentage is achieved. Likewise, the maximum equalizing elevation provides the basin elevation.

For the locks, adding or subtracting the calculated lift heights from the tide or lake elevations provides the maximum and minimum equalization water levels in the locks for all of the combinations. Given these maximum and minimum operating water levels, the elevations for the lock structures themselves can be calculated given:

- There must be 18.3m (60') of clearance over the sill, Sill Elev = Min Equalization Level – 18.3m (60'), except on the Pacific side, where dredging costs and ability to schedule ships to avoid low tide, allows relaxation of this requirement to 18.30m (60') below the MLWS elevation. (Verified by ACP),
- Lock chamber floor elevation will be 0.61m (2') below sill elevation (Provided by ACP - similar to existing locks), and
- Top of Gate Coping = 1.52m (5') + maximum water level (Provided by ACP - similar to existing locks).

The following spreadsheet shows all of the above calculations completed for Option 2 on the Atlantic side. All spreadsheets for all options and ocean sides can be found in **Appendix E.**

OPTION 2 - TWO LOCKS WITH THREE SIDE-BY-SIDE WSB ON ONE SIDE OF LOCK - ATLANTIC OCEAN SIDE

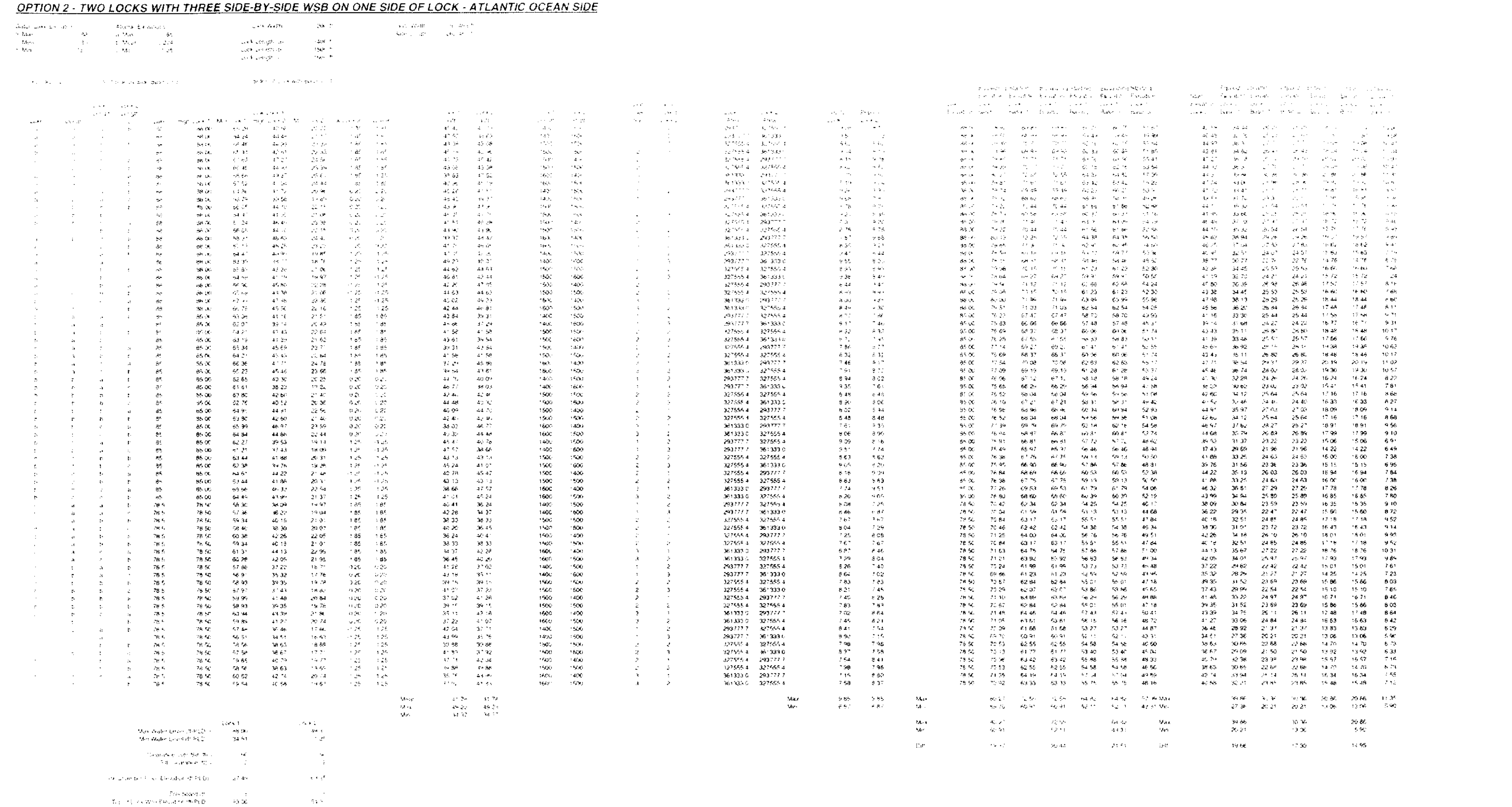


Figure IV.5 – Example Spreadsheet Calculations – Options 2 (Atlantic Side)

b) Check of Layout Spreadsheet and Lock Profile Creation for Three-Lift (Options 1 &3) & Two-Lift Options (Options 2 &4)

The lock elevations provided by ACP for a three-lift lock configuration (see **Figure IV.6**) and those calculated for Options 1 & 3 (three-lift alternatives) matched perfectly. For comparative purposes, Figure IV.6 (provided by ACP) was modified to show the lock elevations for the two-lift options (Options 2 & 4) as well. **Figure IV.7** shows the lock profile and elevations for the two-lift alternatives.

c) Option 1 Hydraulic Layout

Figures IV. 8 – 10 show plan and section views of Option 1 for the Atlantic and Pacific sides. The conduit arrangements and sizes shown in the drawings will be explained in the next chapter, **Chapter V. – Features Design**.

d) Option 2 Hydraulic Layout

Figures IV. 11 – 13 show plan and section views of Option 2 for the Atlantic and Pacific sides. The conduit arrangements and sizes shown in the drawings will be explained in the next chapter, **Chapter V. – Features Design**.

e) Option 3 Hydraulic Layout

Figures IV. 14 – 16 show plan and section views of Option 3 for the Atlantic and Pacific sides. The conduit arrangements and sizes shown in the drawings will be explained in the next chapter, **Chapter V. – Features Design**.

f) Option 4 (Side-by-Side Basins) Hydraulic Layout

Option 4 is complicated because the theoretical equalization levels create an overlap in elevation between the stacked basin levels. This is caused by the widely varying water levels in Gatun Lake and the ocean, particularly on the Pacific side. Since the additional effort to size a side-by-side arrangement is minimal, M&N decided to provide both a side-by-side basin arrangement as well as a stacked basin arrangement. **Figures IV. 17 – 19** show plan and section views of Option 4 (side-by-side basins) for the Atlantic and Pacific sides. The conduit arrangements and sizes shown in the drawings will be explained in the next chapter, **Chapter V. – Features Design**.

g) Option 4 (Stacked Basins) Hydraulic Layout

As stated above, Option 4 did have a problem of how the necessary overlap (necessary to save full % of water at all times with varying water levels – see pg. 79 for explanation) could be incorporated into a stacked basin arrangement. **Figures IV. 33 – 35** show plan and section views of Option 4 (stacked basins) for the Atlantic and Pacific sides following a detailed discussion of the stacked water saving basin development. The conduit arrangements and sizes shown in the drawings will be explained in the next chapter, **Chapter V. – Features Design**.

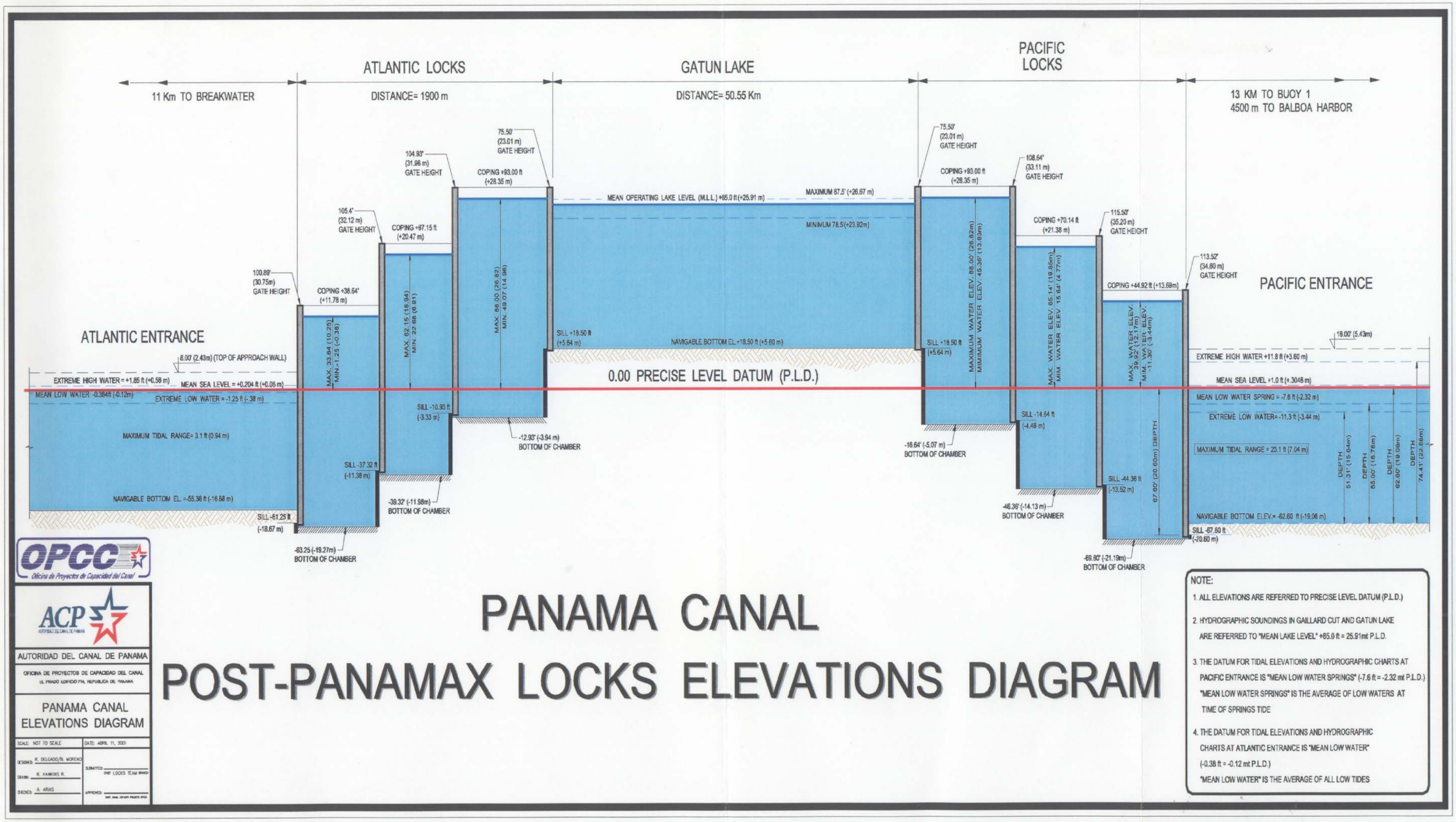


Figure IV.6 - Lock Profile Elevations for Three-Lift Options – Options 1 & 3 (provided by ACP)

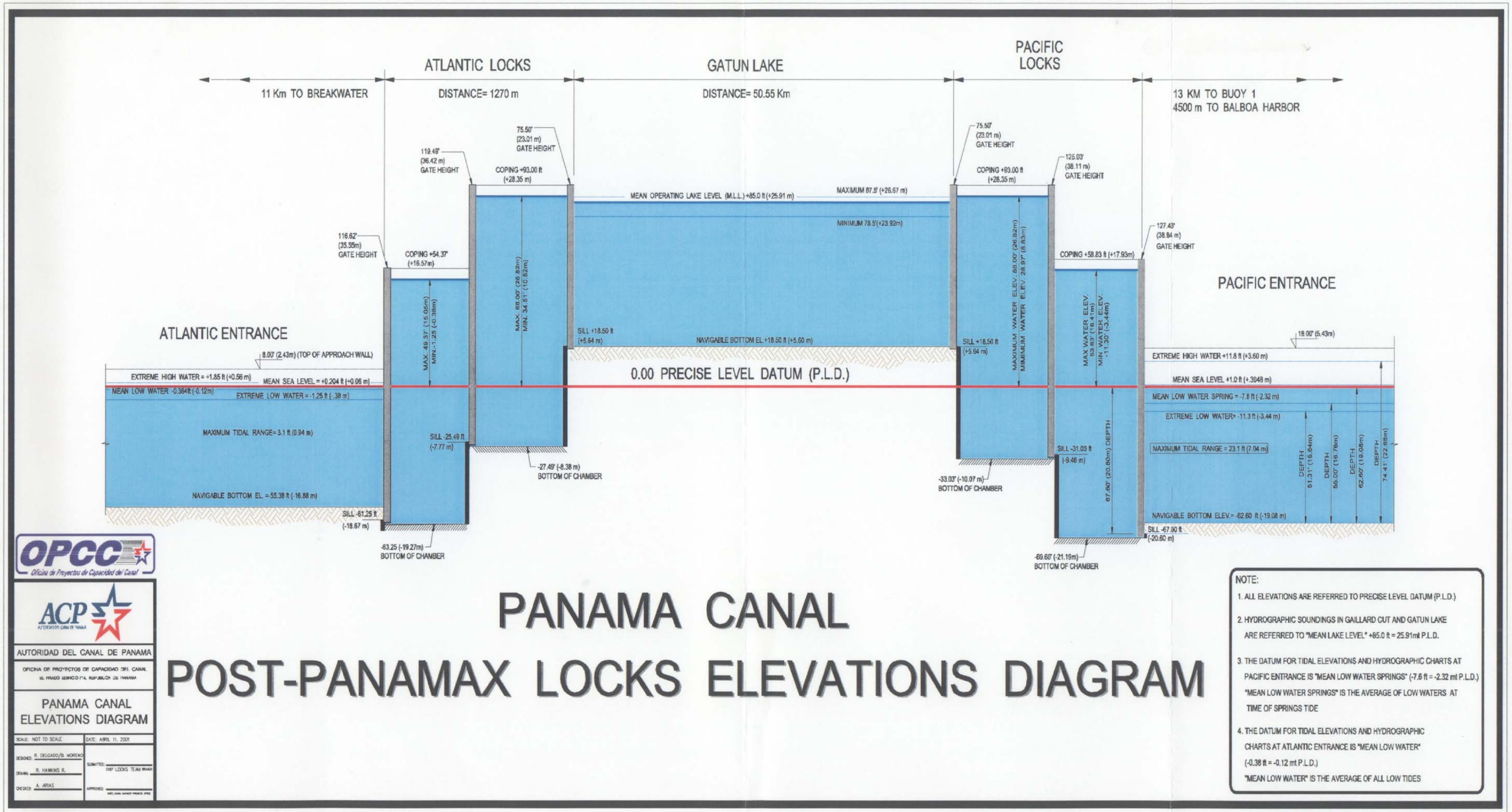


Figure IV.7 - Lock Profile Elevations for a Two-Lift Option – Options 2 & 4 (modified from drawing provided by ACP)

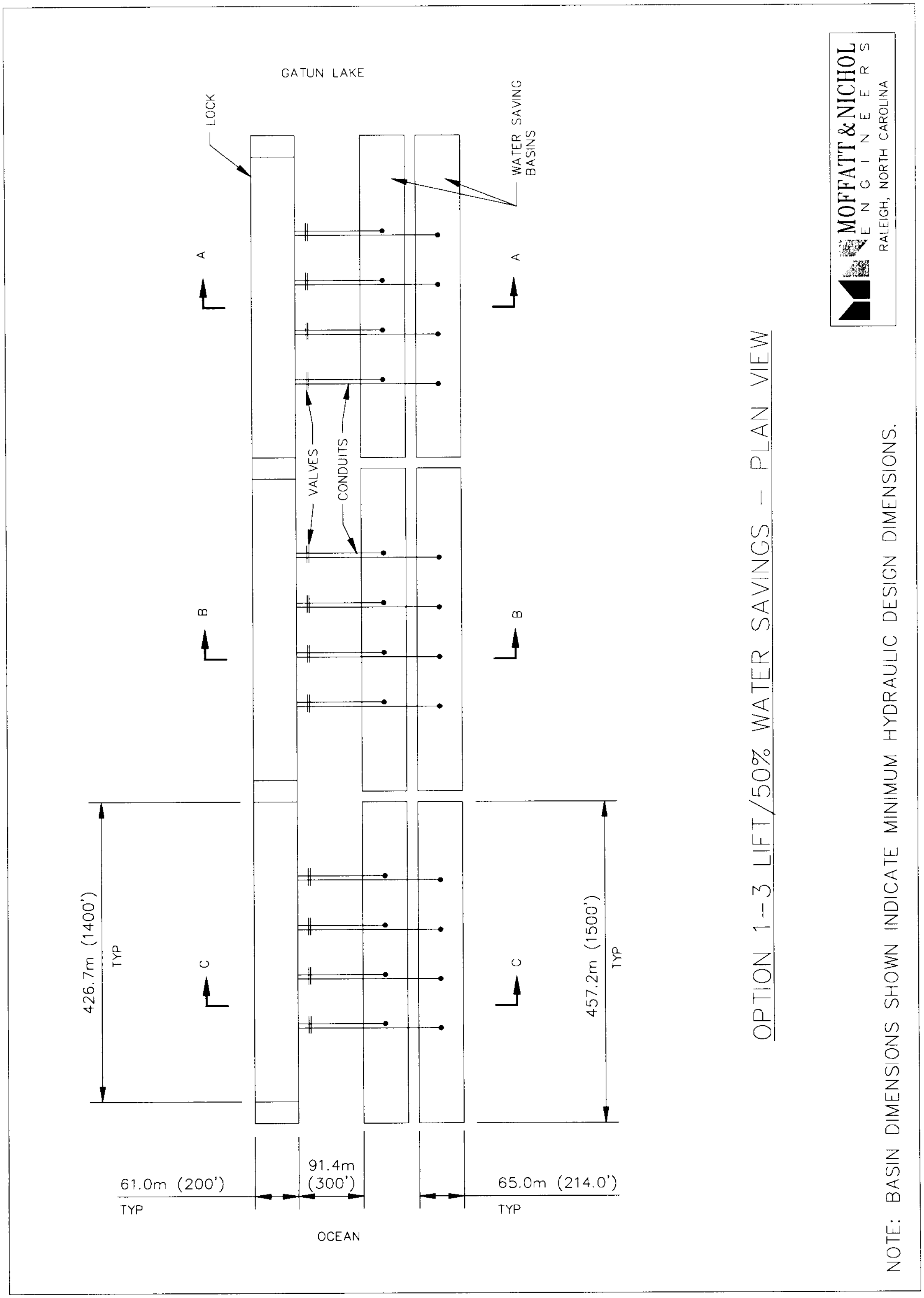


Figure IV.8 – Option 1 – Plan View

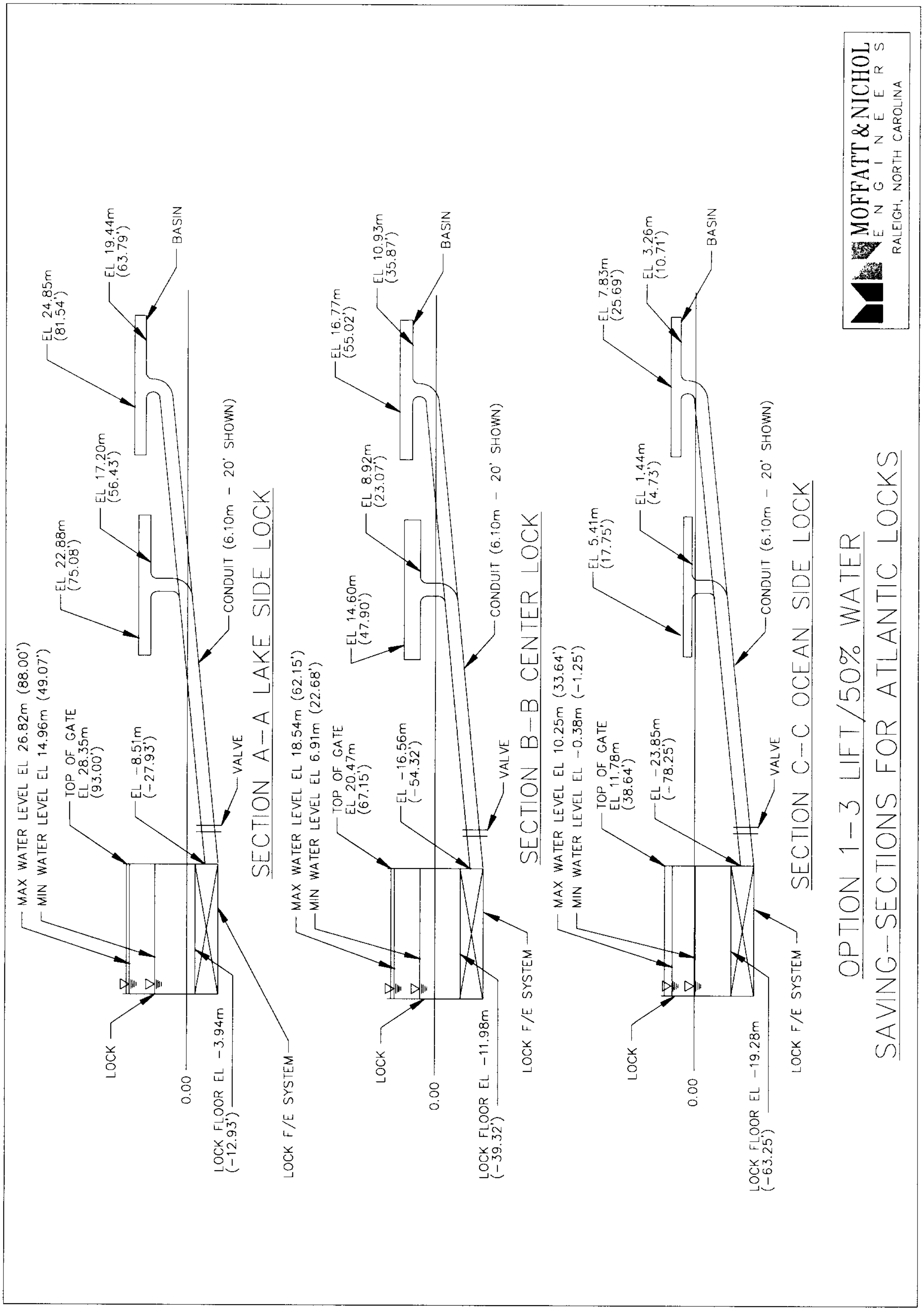


Figure IV.9 – Option 1 – Section View (Atlantic Side)

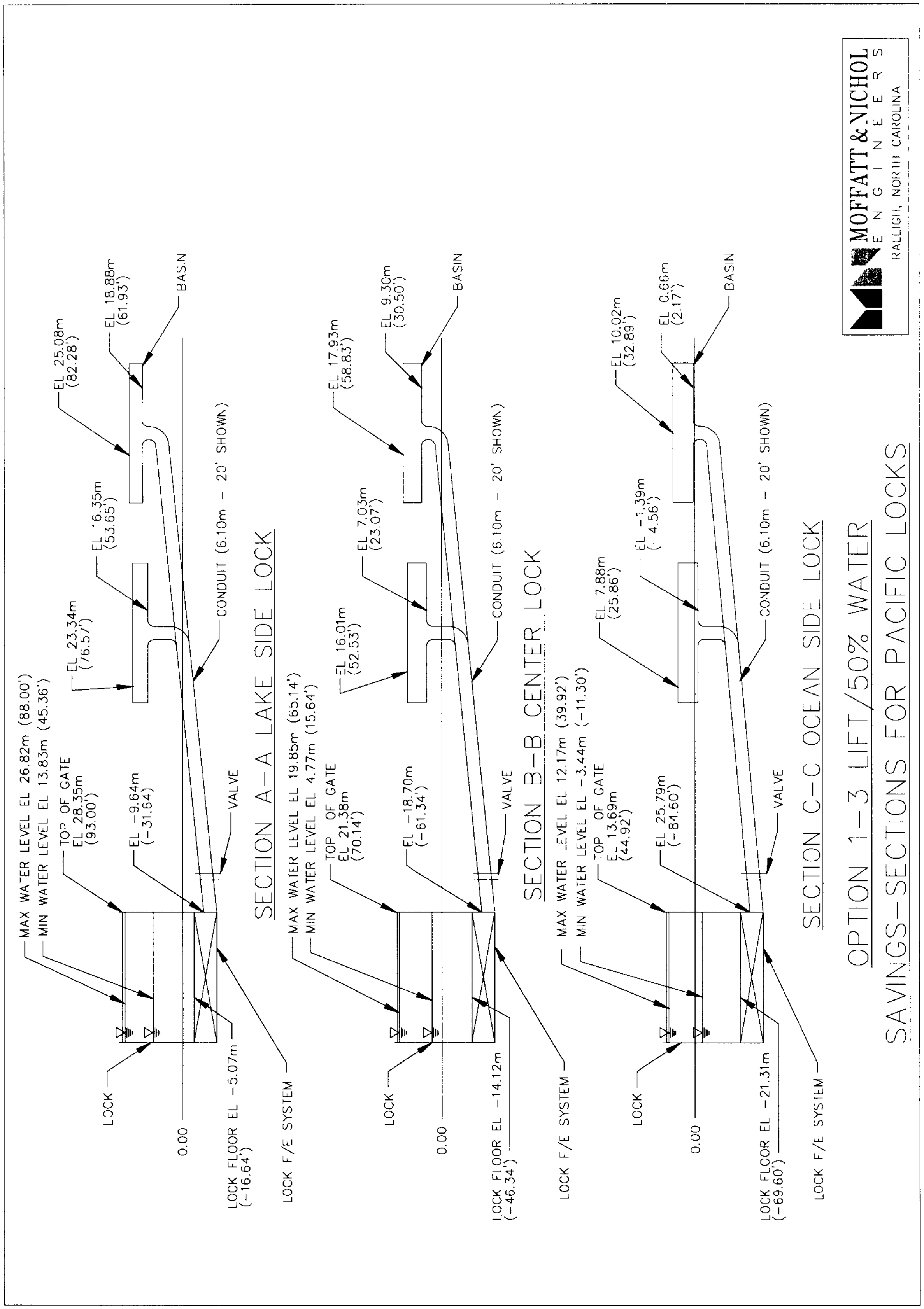


Figure IV.10 – Option 1 – Section View (Pacific Side)

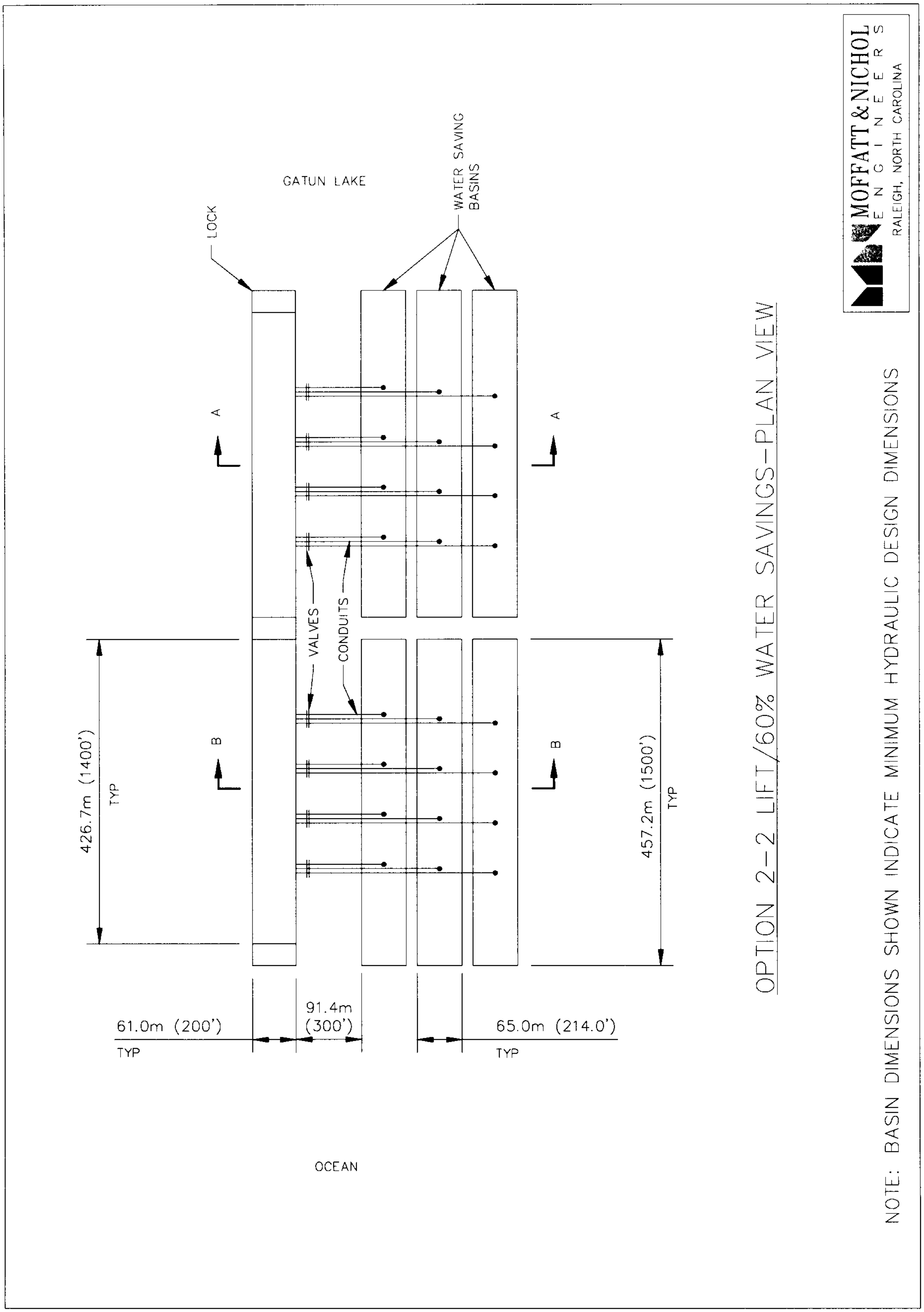


Figure IV.11 – Option 2 – Plan View

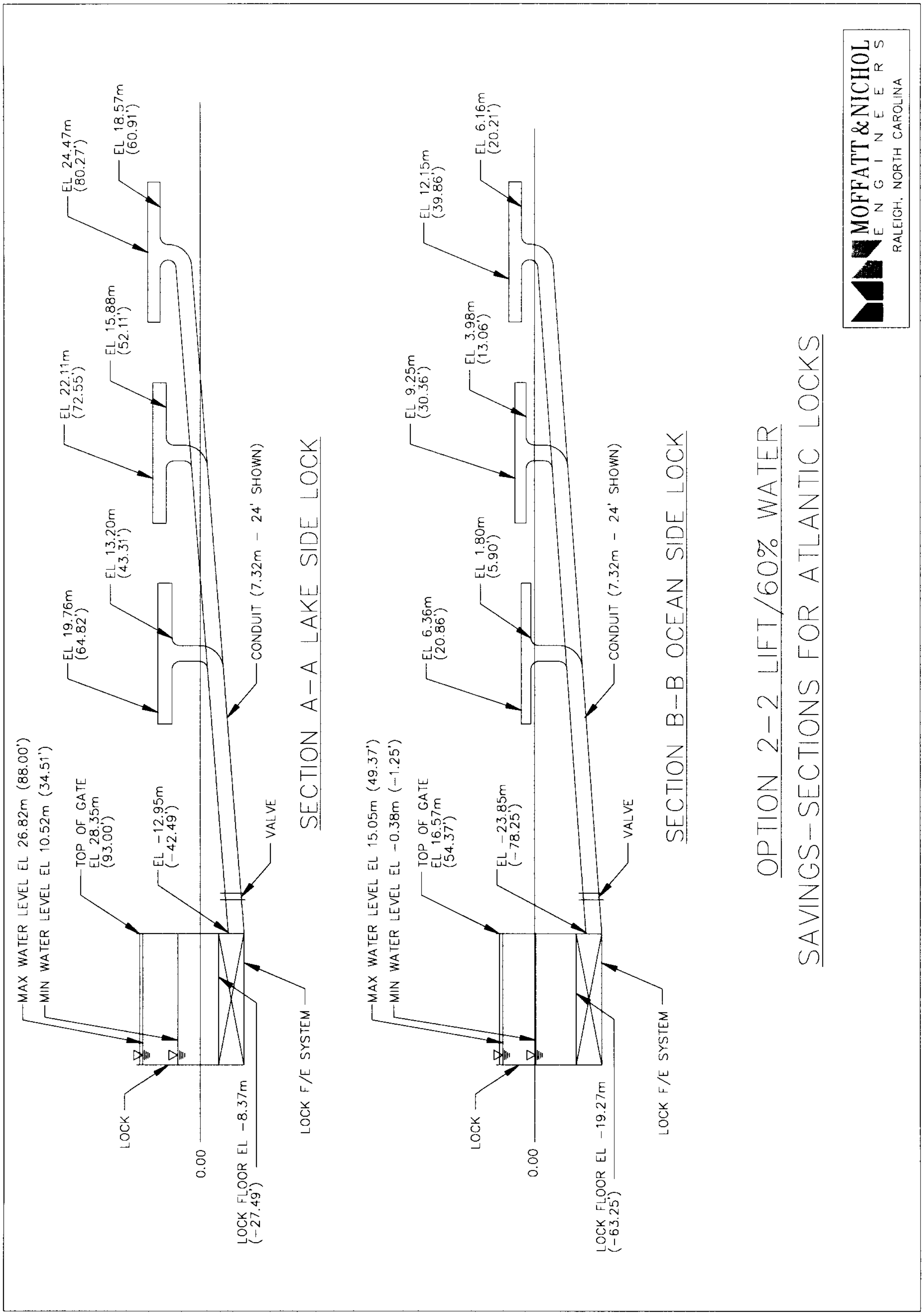


Figure IV.12 – Option 2 – Section View (Atlantic Side)

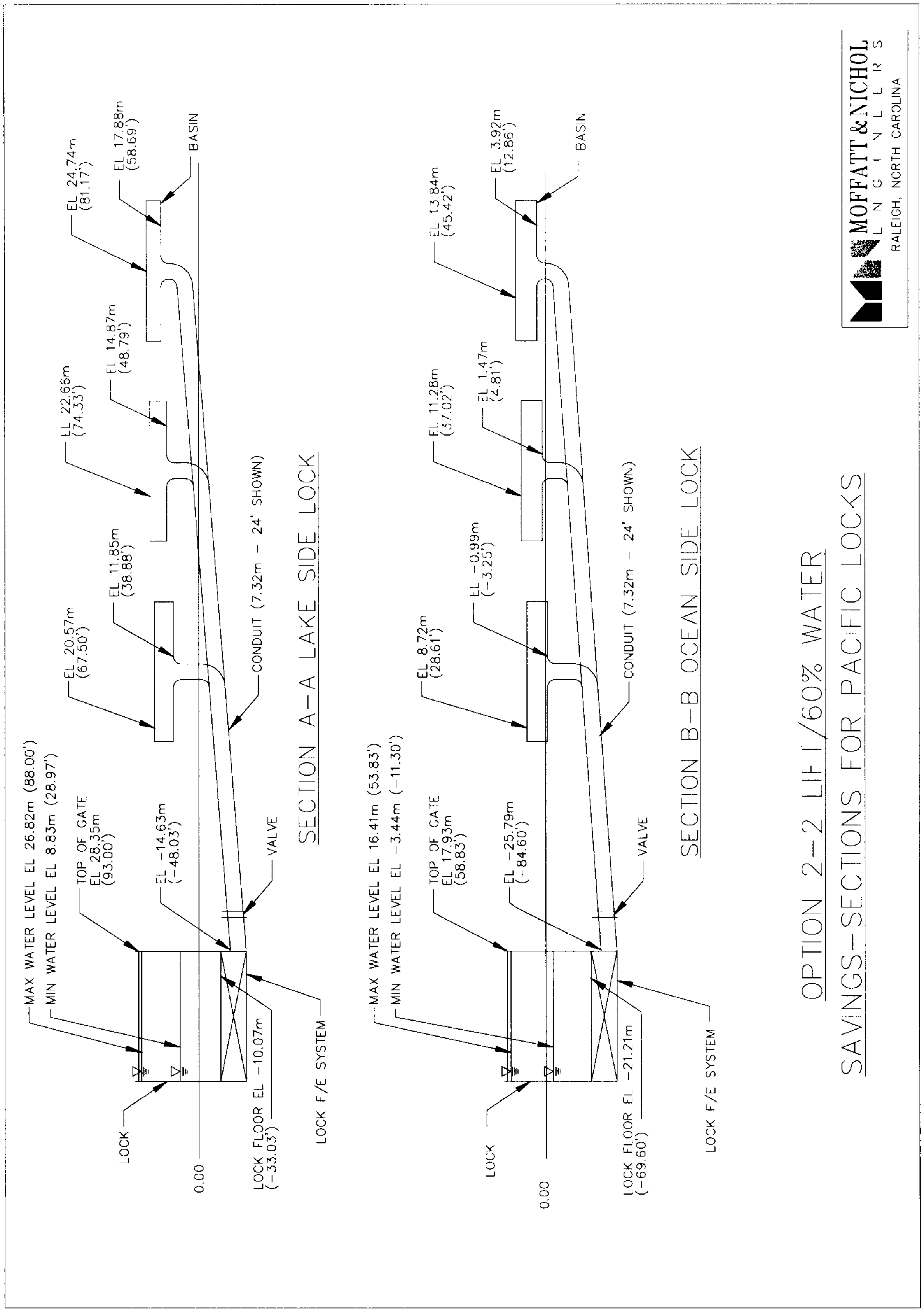


Figure IV.13 – Option 2 – Section View (Pacific Side)

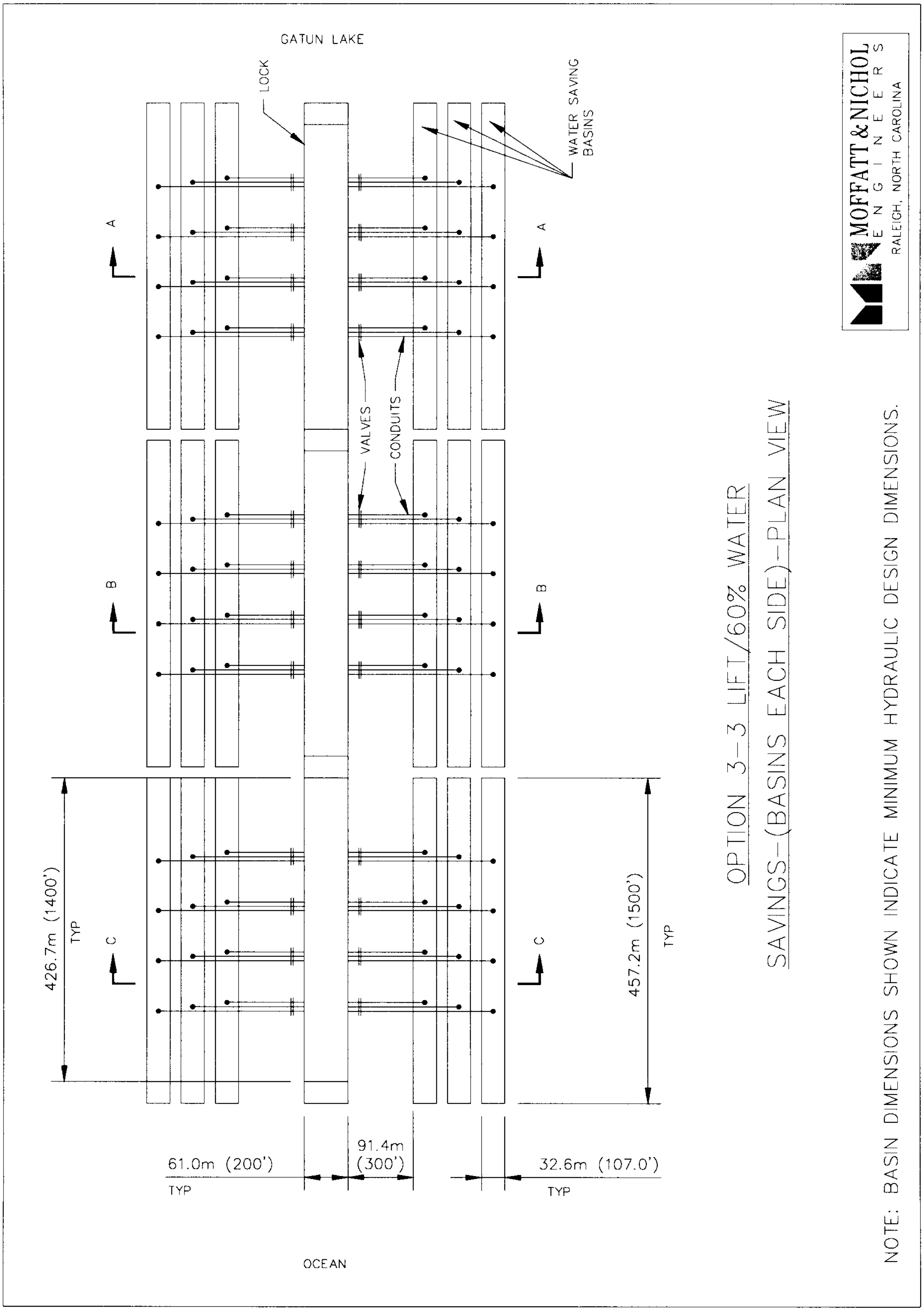


Figure IV.14 – Option 3 – Plan View

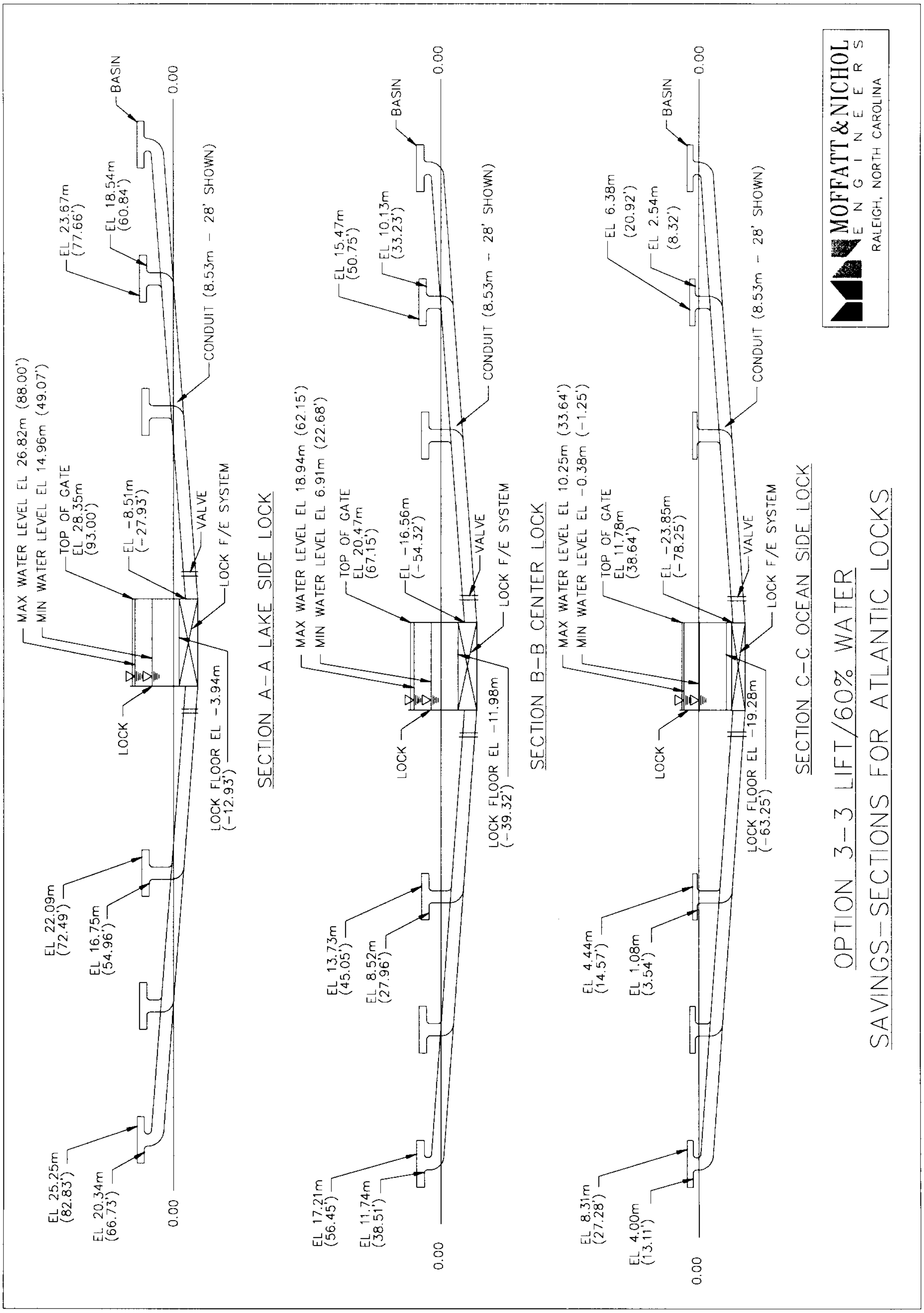


Figure IV.15 – Option 3 – Section View (Atlantic Side)

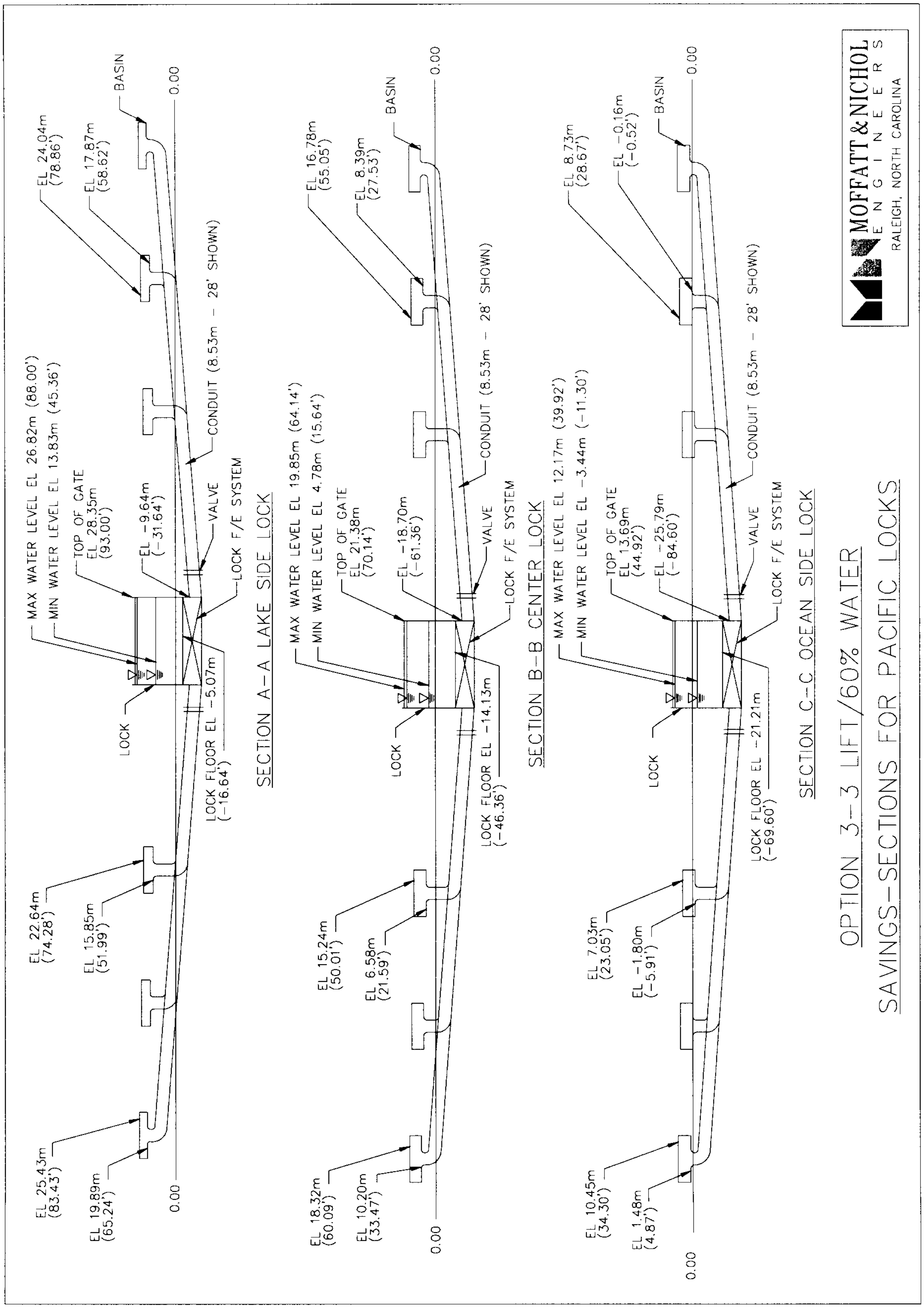


Figure IV.16 – Option 3 – Section View (Pacific Side)

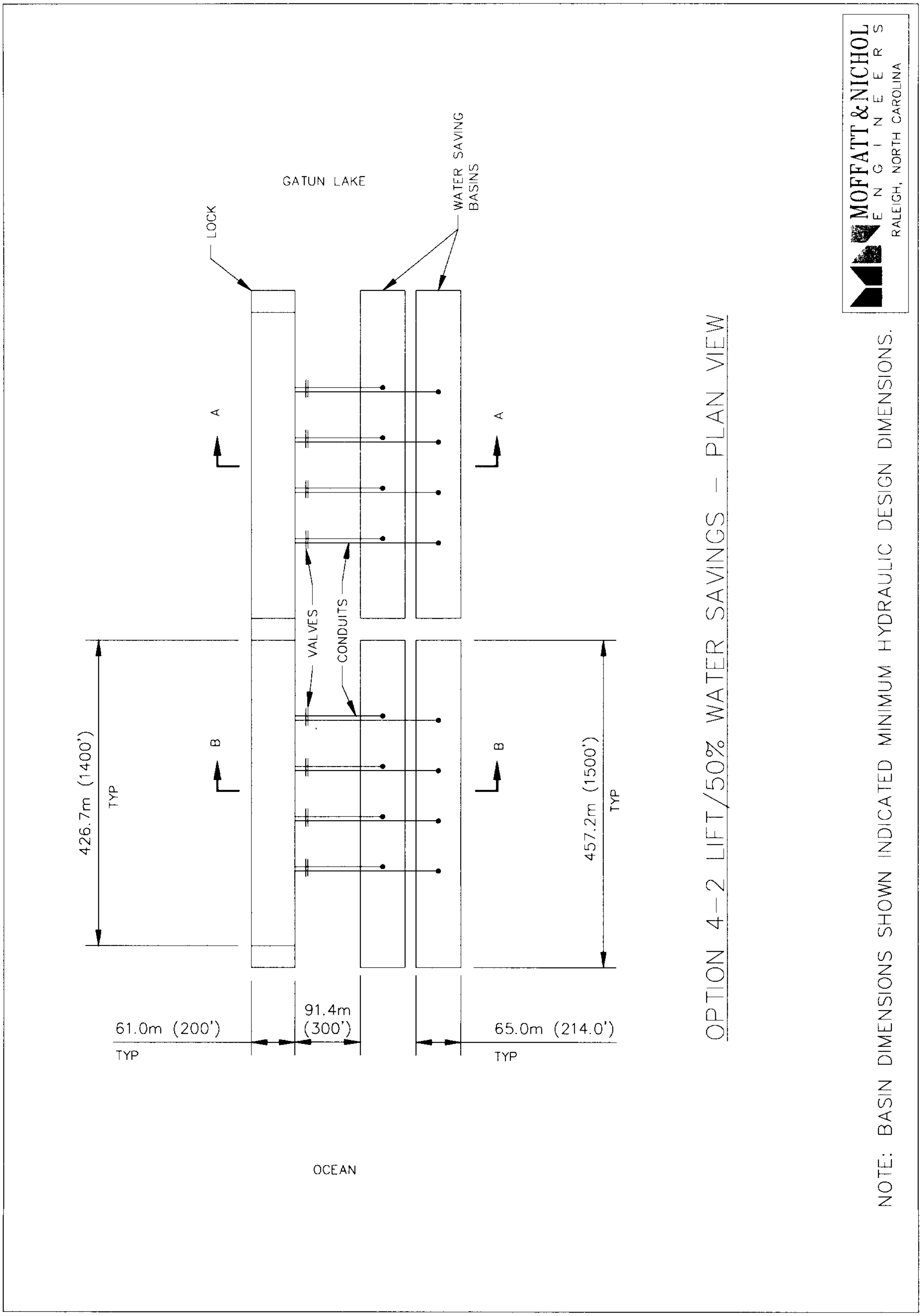


Figure IV.17 – Option 4 (Side-by-Side Basins) – Plan View

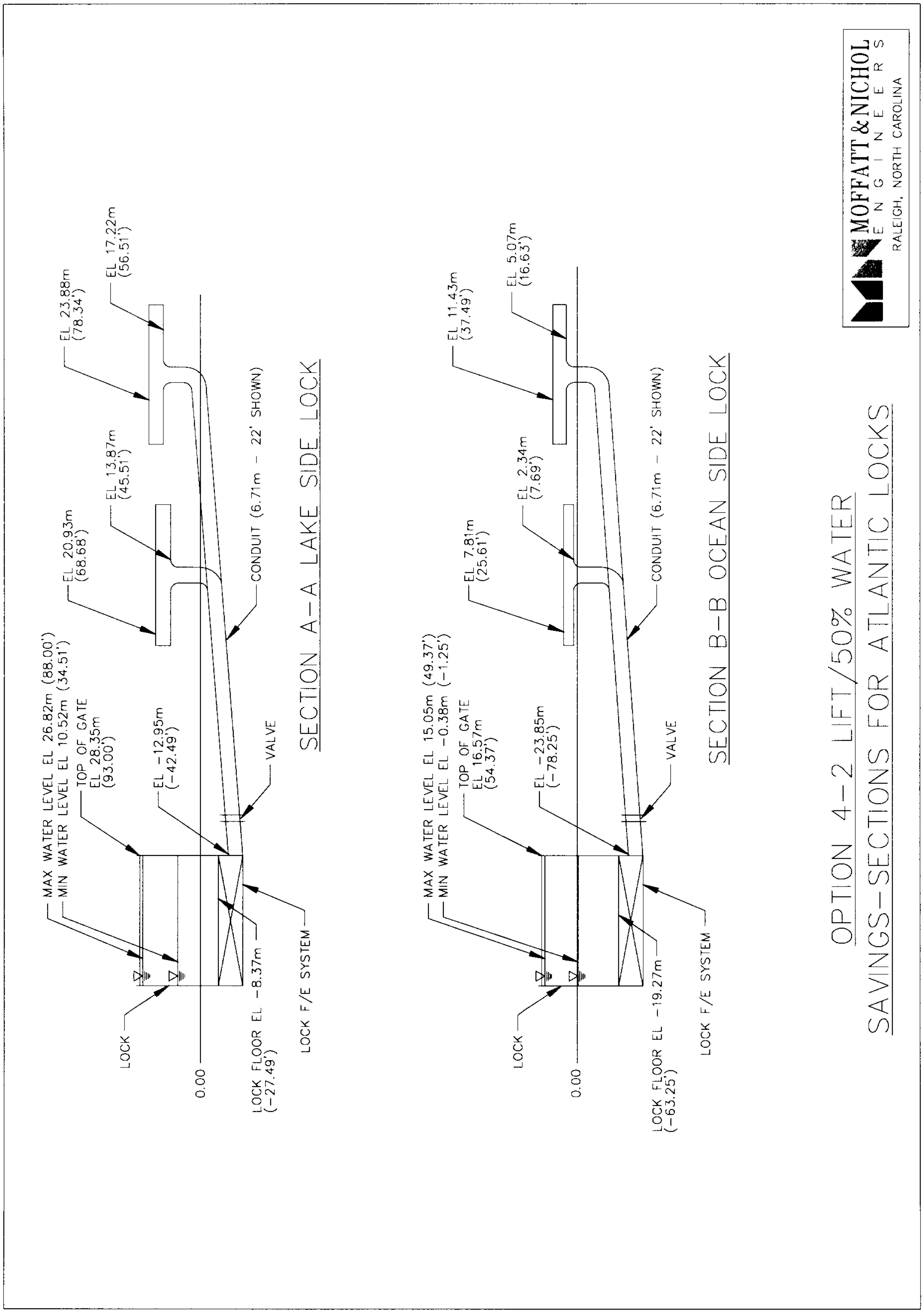


Figure IV.18 – Option 4 (Side-by-Side Basins) – Section View (Atlantic Side)

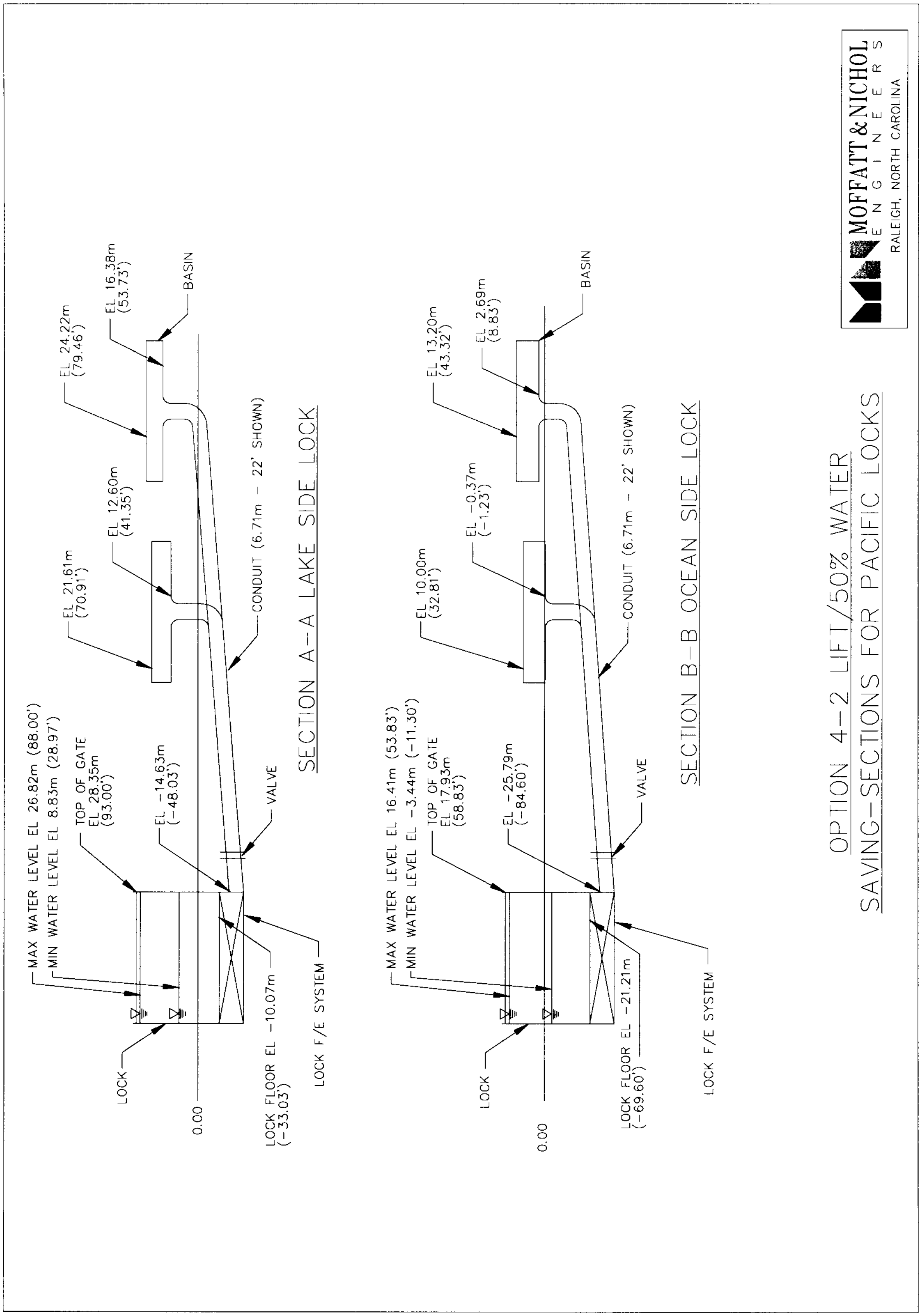


Figure IV.19 – Option 4 (Side-by-Side Basins) – Section View (Pacific Side)

To better understand the concept of the "overlap", it would help to look at an example. For instance, for the upper lock of the two-lift lock system, the basin elevations required to provide the full theoretical water savings percentage for all combinations of water levels and lockage lengths are as shown in **Figure IV.20**. The reason that this occurs can be traced back to the design procedures outlined on pages 59-62. With the wide variation in water levels and locakge lengths, there will also be variations in the lift heights calculated. Since these lift heights set the operational levels in the lock and its basins (see **Figure IV.4**), there is a resulting variation in the operational elevations for the lock and basins. Completing these calculations for the entire range of design water levels and lockage lengths then gives us the range of operational elevations for each basin and the resulting overlap between adjacent basins as is illustrated below. This also illustrates why the considerable overlap required cannot be easily incorporated into a stacked basin arrangement.

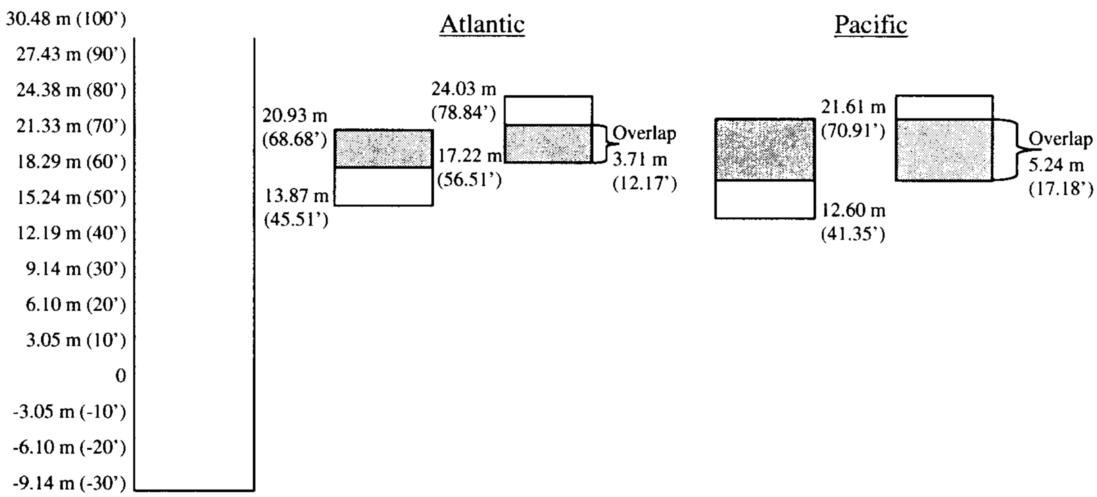


Figure IV.20 – Option 4 – Overlap Requirements for Atlantic and Pacific Lakeside Locks (Metric and English Units Shown)

Analysis procedures and option configurations to overcome these overlaps were needed. Some of the options investigated included:

- Put stacked basins on both sides of lock as this would help allow for overlap. (examples in PIANC show stacked basins on both sides with offsets to help incorporate the necessary overlap) This option is not really feasible for Option 4 since two stacked basins reduce to two side-by-side basins in this case.

- Alter basin width to limit overlap. However, this option will alter equalization levels so that a revised analysis is necessary.

From the PIANC Literature (see **Figure IV.1**),

$H - nt = V = t + 2e + h$, where

H = Lift
t = Step Height (m=1)
$t = m*h$,

it can be shown that

$$h = \frac{H - 2e}{(1 + m(1 + n))}$$

In our case, assume **H** >> 2e ("e" not set yet – should be optimized at a later design stage), then

$$h \cong \frac{H}{1 + m(1 + n)}, \quad \text{therefore, when } m = 1 \quad t = \frac{H}{1 + 1\,(1 + n)} = \frac{H}{n + 2}, \text{ or } t = h$$

With m ≠ 1, the spreadsheet model had to be changed. Columns were inserted within the spreadsheet to calculate **t** and **h**. Since **t** represents the step heights/equalization levels needed to save the theoretical % of water draining from the lock, the **t** theoretical levels should be used to set the top of the highest basin and the floor of the lowest basin.

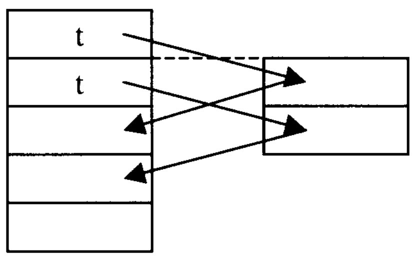


Figure IV.21 – Conceptual Section of t Depths Draining from Locks

For example, see **Figure IV.21**. In order to achieve theoretical water saving percentage, the water column height, **t** must be drained from the lock to the WSB. Therefore, the upper basin ceiling elevation must be set at the maximum **t** equalization elevation regardless of the basin width. Likewise, the minimum **t** value sets the lower basin floor elevation so that water can always be saved within the lower basin and then returned to the lock.

Using the PIANC nomenclature, **h** was calculated as the basin height required to accommodate **t** based on the basin area ratio – **m**. Therefore, if the lower basin floor elevation and the upper basin ceiling is set by **t**, varying **h**'s (based on **m**) would then be added to the lower basin floor to calculate a new lower basin ceiling elevation. Likewise, **h** would be subtracted from the upper basin ceiling to calculate a new upper basin floor elevation (see **Figure IV.22**).

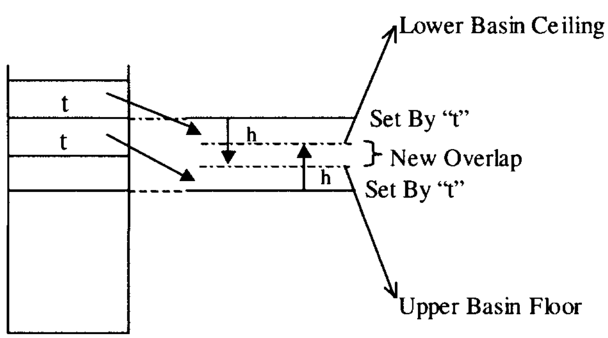


Figure IV.22 – Schematization of Stacked Basin Elevations Based on h

The new required overlap could then be calculated based on the calculations outlined above for all combinations of water levels and lockage lengths. These calculations were completed for various **m** values, in hopes of minimizing the required overlap. For a schematization of the minimizing overlap versus **m**, see **Figure IV.23**.

For example,

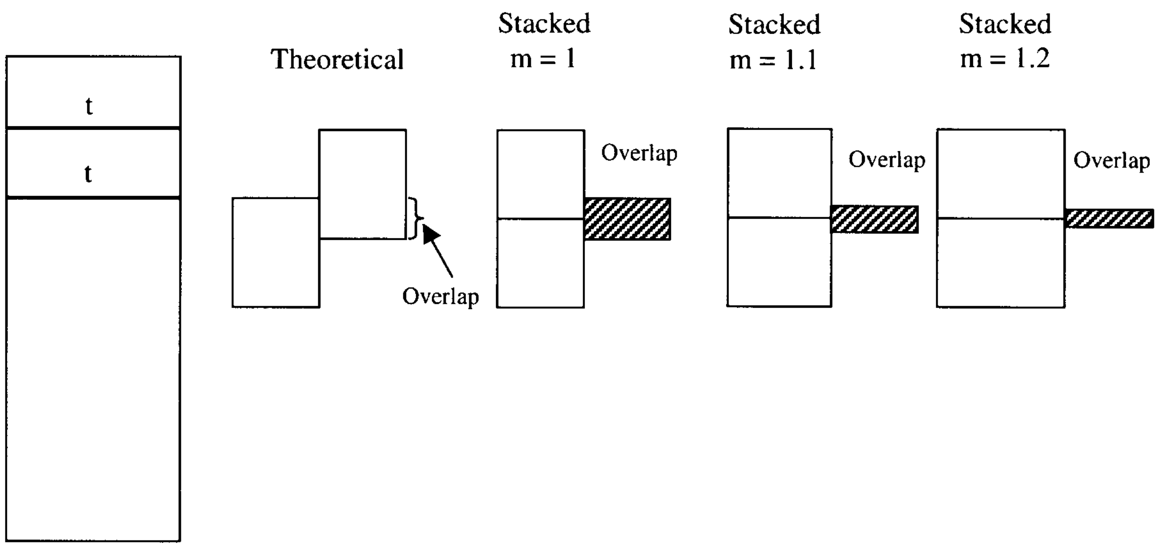


Figure IV.23 – Schematization of Minimizing Overlap versus m

Some envisioned benefits of using this modified design procedure were:

- Since **t** is being used, the lock equalization levels and lifts would be similar to the side-by-side arrangements and

- Since the maximum upper basin ceiling elevation and the minimum lower basin floor elevation would be equivalent to those of the side-by-side arrangement, the basin dimensions would be relatively equal so that operations would not vary much from lock to basin and lock to lock.

However, some disadvantages that would be experienced would be that as **m** increases, **h** would become less and less (if **m** increases, the required **h** to drain the fixed **t** is reduced), and by default the equalization times for basin to lock operations would become somewhat more than lock to basin operations since driving heads will be reduced with **h** (portion of driving head for basin to lock operations) now being considerably less than **t** (portion of driving head for lock to basin operations). Also, since it is not possible to eliminate all of the overlap, without impractically large **m** ratios, the theoretical water saving percentages for this arrangement of stacked basins will be less than for a comparable number of side-by-side basins.

To calculate the new water savings percentages for this design procedure, the upper basin floor elevation and the lower basin ceiling elevation had to be set. In the interest of symmetry of operations the upper basin floor was initially set directly at this midpoint. Based on information provided by INCA (INCA estimated that a 0.3-meter thick floor would be sufficient for the anticipated column size and spacing), the floor thickness was assumed to be approximately 0.3 meters (1 foot).

With these elevations set, the water savings percentages were calculated. This was done for all water levels and lockage length combinations by estimating the actual volume of water saved versus the theoretical ideal.

The theoretical volume would be equal to **t** times the lock area, while the actual volume saved would be either **h** or **h'** times the basin area. The use of **h** or **h'** is described below and in **Figure IV.24**.

h was used for cases where the calculated value fits within the set upper floor and lower ceiling elevations. However, **h'** was used when the calculated **h** volume was outside the established range.

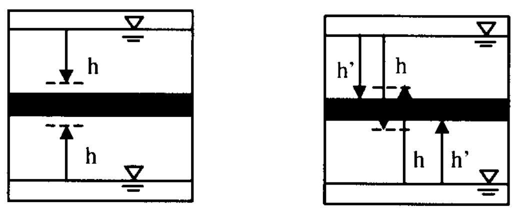


Figure IV.24 – Schematic Showing Use of h or h' in Water Saving Percentage Calculations

Hence, the actual % water saved would be:

$$\left(\frac{(\text{h or h'})*\text{Basin Area}}{\text{t}*\text{Lock Area}}\right)*\text{Theor. Water Saving \%}.$$

This calculation could be done for all water level and lockage combinations for a given **m**. Since **t** is being used, the water savings should be equivalent for down lockages as well as up lockages. This follows because there should always be room to drain the basins even after lock to lock equalization (see **Figure IV.25**).

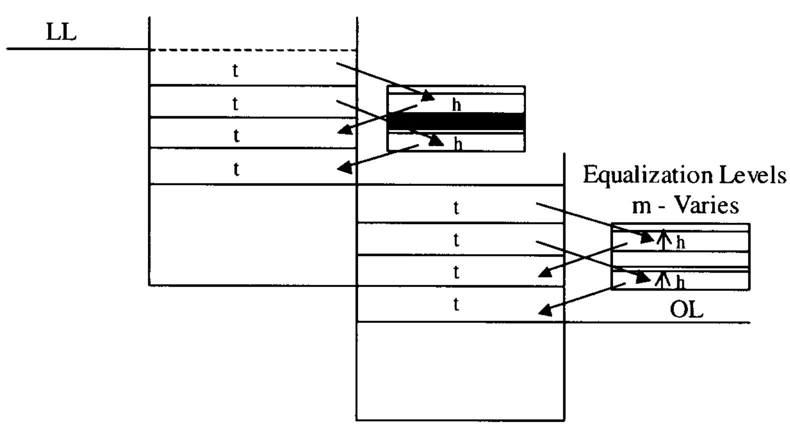


Figure IV.25 – Schematic Section

These computations were completed and the maximum, minimum, and average % water savings for all combinations of water levels and lockage lengths were recorded for the upper lock, the lower lock, as well as an overall average between the two locks. The resulting water saving percentages versus basin/lock surface area ratio (**m**) can be seen on **Figures IV.26 and 27** for both ocean sides. (Please note that the average maximum results were identical so that only one line is visible.

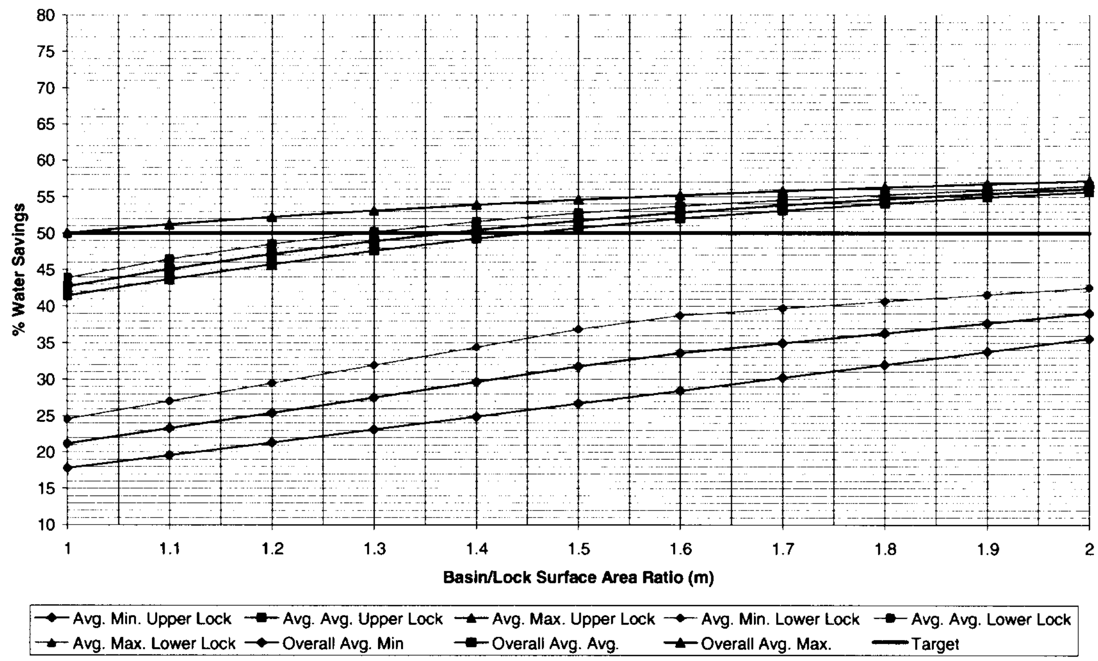


Figure IV.26 - % Water Savings vs. Basin/Lock Surface Area Ratio (m) – Atlantic Side

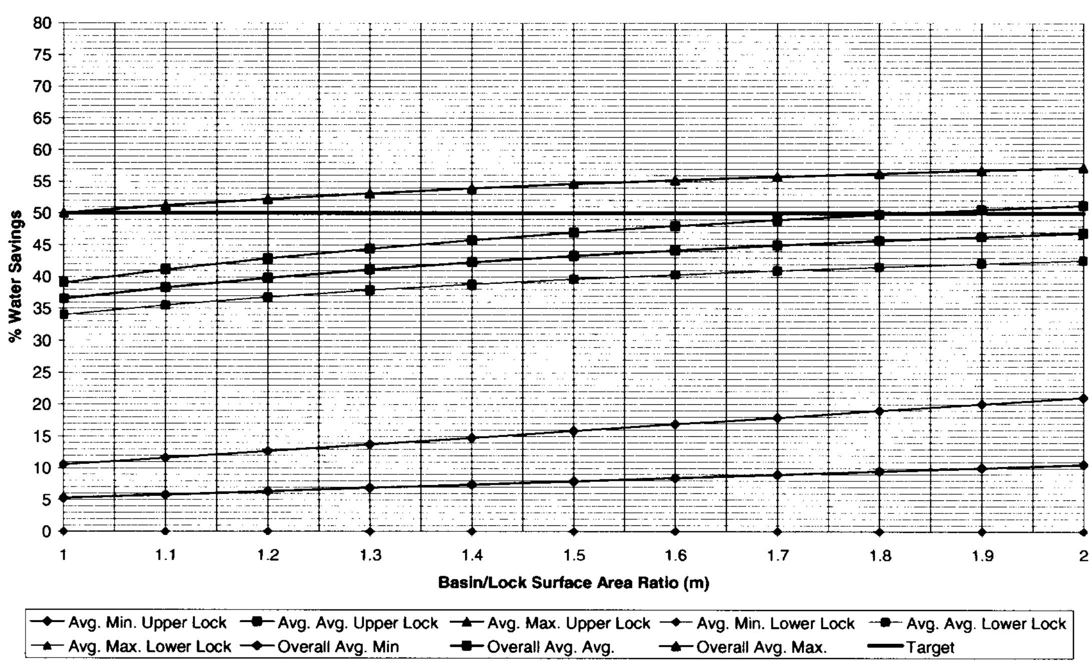


Figure IV.27 - % Water Savings vs. Basin/Lock Surface Area Ratio (m) – Pacific Side

From **Figure IV.26**, a basin/lock ratio **m** of 1.4 is needed on the Atlantic side to achieve an overall 50% water savings percentage. The Pacific side, however, is much more problematic. The **m** factor has to be set to approximately 3.0 before overall average savings reach 50%. This extremely high **m** factor is not practical as it would be cheaper to build side-by-side basins since the **m** in that case would be 2. Nonetheless, some interesting patterns emerged from these graphs which helped determine another design procedure.

First, the average maximum water savings are equal for the upper and lower locks. This is not surprising since there are always cases in which it is possible to save the theoretical maximum.

Second, the variability of the water saving percentages is much higher on the Pacific than on the Atlantic side. This is not surprising given the large tide range on the Pacific side which necessitates more overlap. Therefore, once basin elevations are fixed, it is much more difficult to save the theoretical percentage of water over most of the operational conditions on the Pacific side.

Third, on the Atlantic side, the lower lock saves more water, whereas on the Pacific side, the upper basin saves more. On the Atlantic side, the higher water level variability is in the lake, whereas on the Pacific, the higher water level variability is on the ocean side. Therefore, allowing variability in both basin sizes and depths may need to be considered to save more water in a fixed, stacked basin arrangement.

Therefore, to allow more variability in basin sizes and depths, a revised analysis methodology was needed. As part of the new design methodology, the concept of using **t** to set preliminary basin ceiling and floor elevations was discarded. However, it was important to consider that too much change could severely alter the operational symmetry of the entire system, which was one of the benefits of using the **t** dimension. Therefore, to minimize this problem, it was decided that the lower basin floor elevations would be held to those estimated by using **t**. It was hoped that this would minimize the changes in lock-to-lock equalization levels, which in turn would help minimize the asymmetry that this alternate approach might create. Minimizing the changes to the lock-to-lock equalization levels would also help to limit changes to the upper lock chamber floor elevation which would help to not incur significant additional dredging.

For the new analysis procedure, the lower basin floor elevations were set using **t** as in the prior analysis. For a given **m**, a trial upper basin floor elevation was set and by using 0.30 m (1') floor depth (provided by INCA), the lower basin ceiling elevation was calculated. Using the area relationships between the lock and the basin, the equalization elevations between the lock and upper basin were then calculated. Next, the equalization elevation between the lock and the lower basin was calculated. (see **Figure IV.28**) This was done in the spreadsheet for all water level and lockage length combinations. In some of the combinations, EL. E would be higher than EL C. Therefore, the elevations set in ② (EL. B & C) were raised and lowered until EL. E was equal to or less than EL C in all cases.

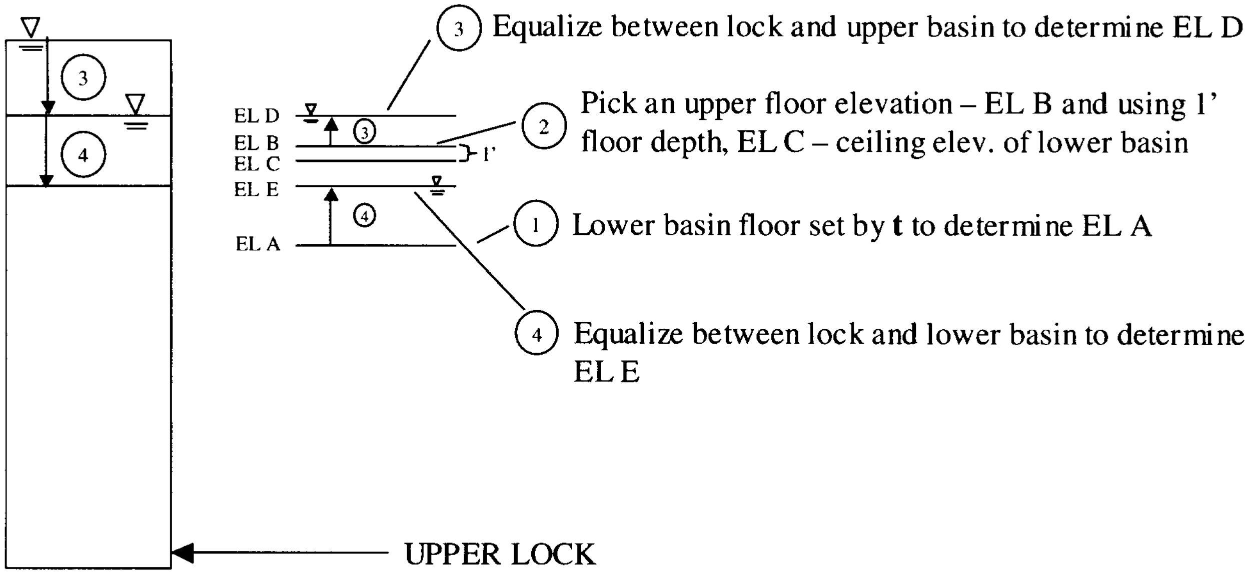


Figure IV.28 – New Design Methodology Schematic – Upper Lock/Basin Equalization

After the upper basin floor had been finalized, the elevation of the upper basin ceiling was then set by the maximum EL. D calculated for all combinations. The water level that was then left in the upper lock by ④ (EL. E) was then used with the tide level in the lower lock to equalize ⑤ and determine EL. F (with the lock to lock area ratio included). (see **Figure IV.29**).

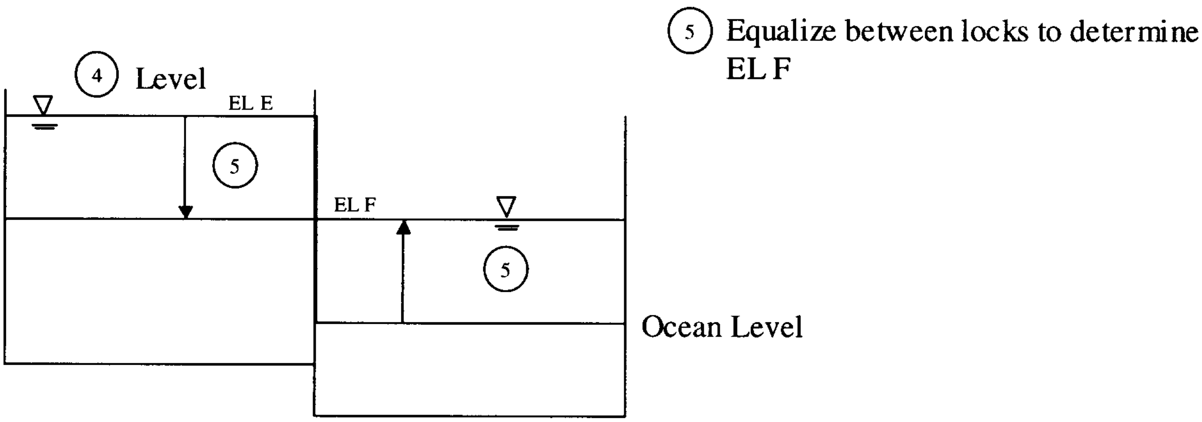


Figure IV.29 – New Design Methodology Schematic – Lock to Lock Equalization

Once the new equalization level between the locks was calculated, the process outlined above was repeated with the lower lock and basins with the following exception: When equalizing between the locks and the basin, the higher of the tide level or the basin floor elevation was selected to equalize against the lock level elevation. (see **Figure IV.30**)

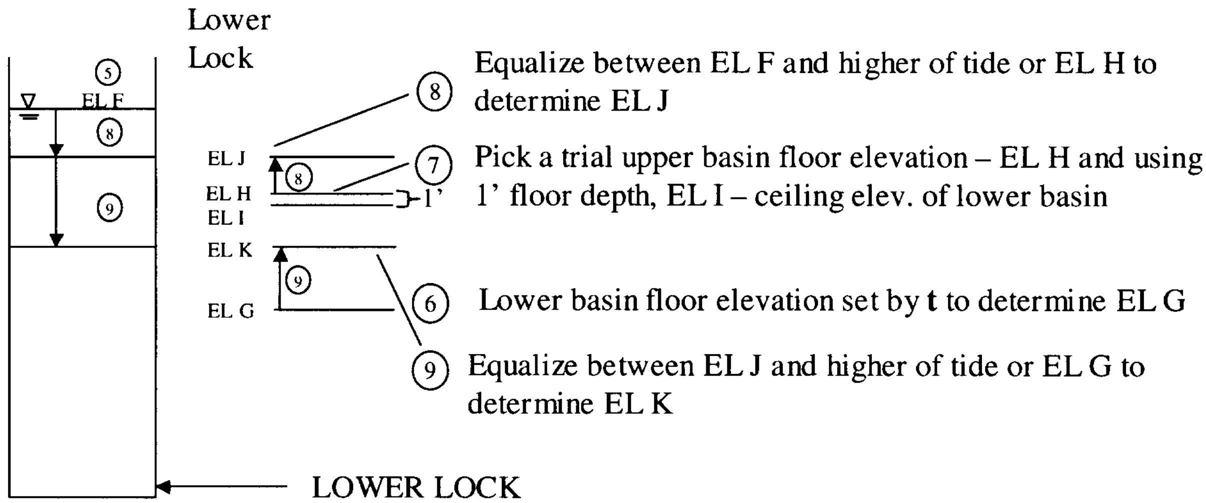


Figure IV.30 – New Design Methodology Schematic – Lower Lock/Basin Equalization

As with the upper lock, EL. H and EL. I would be raised or lowered until EL. K was equal to or less than EL. I for all combinations of water levels and lockage lengths. The upper basin ceiling elevation was set by the maximum elevation calculated for EL. J.

These calculations were done for a given **m** and additional calculations were made to determine water saving percentages and possible impacts to upper lock chamber floor elevation (since **t** was no longer being used to set the equalization level between the locks). The additional dredging required was calculated as the difference between the new and old minimum equalization levels which were used to set the lock chamber floor (lock chamber floor = minimum equalization elevation – 18.3m (60') – 0.61m (2')).

To calculate the expected water saving percentages, separate computations were required for downlockages and uplockages since **t** was no longer being used to keep operations and equalization levels equivalent. The calculations would have to be detailed volumetric calculations that compare the saved water (depth of water in basin * basin area) to the water used (**t** * lock area). These same computations were done for the upper and lower locks and the maximum, minimum, and average water saving percentages for all combinations of water levels and lockage lengths were recorded. These results were then averaged for both of the locks for downlockage and uplockage conditions. A final averaging of the results for the up- and downlockages was completed for an overall average. The following graphs show the results from the analyses.

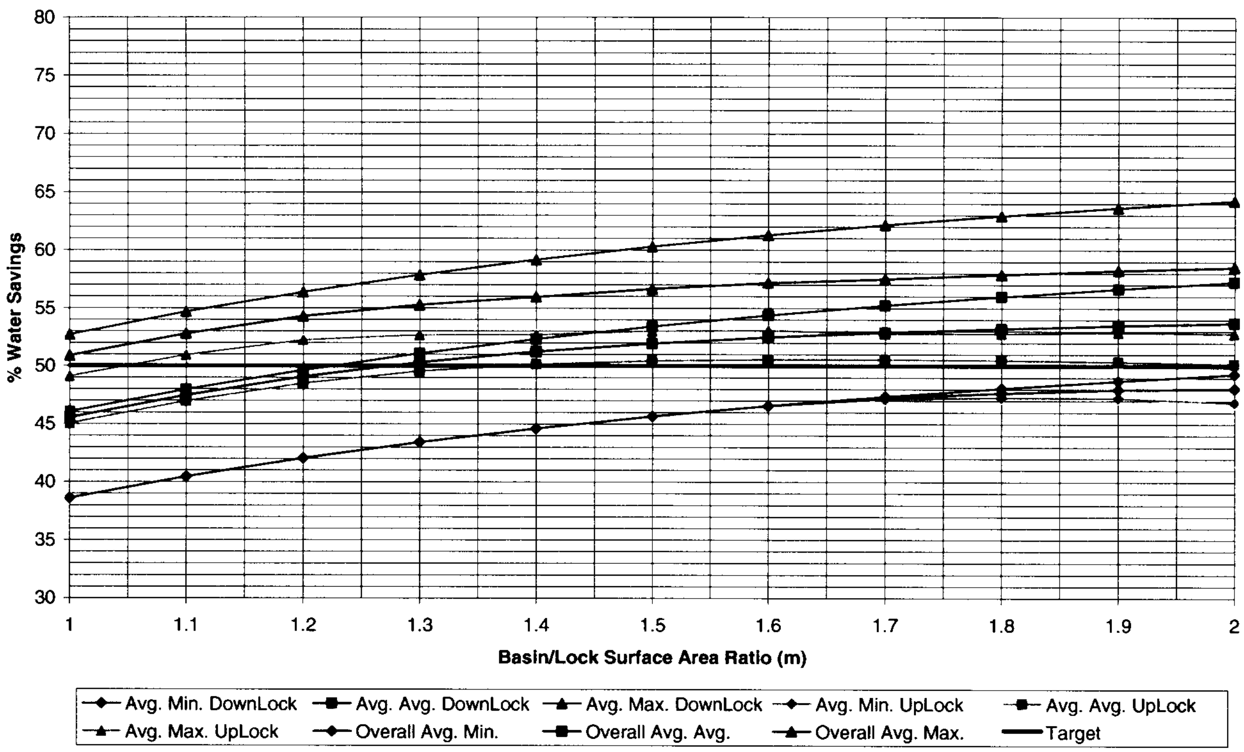


Figure IV.31 - % Water Savings vs. Basin/Lock Surface Area Ratio (m) – Atlantic Side (Revised Analysis)

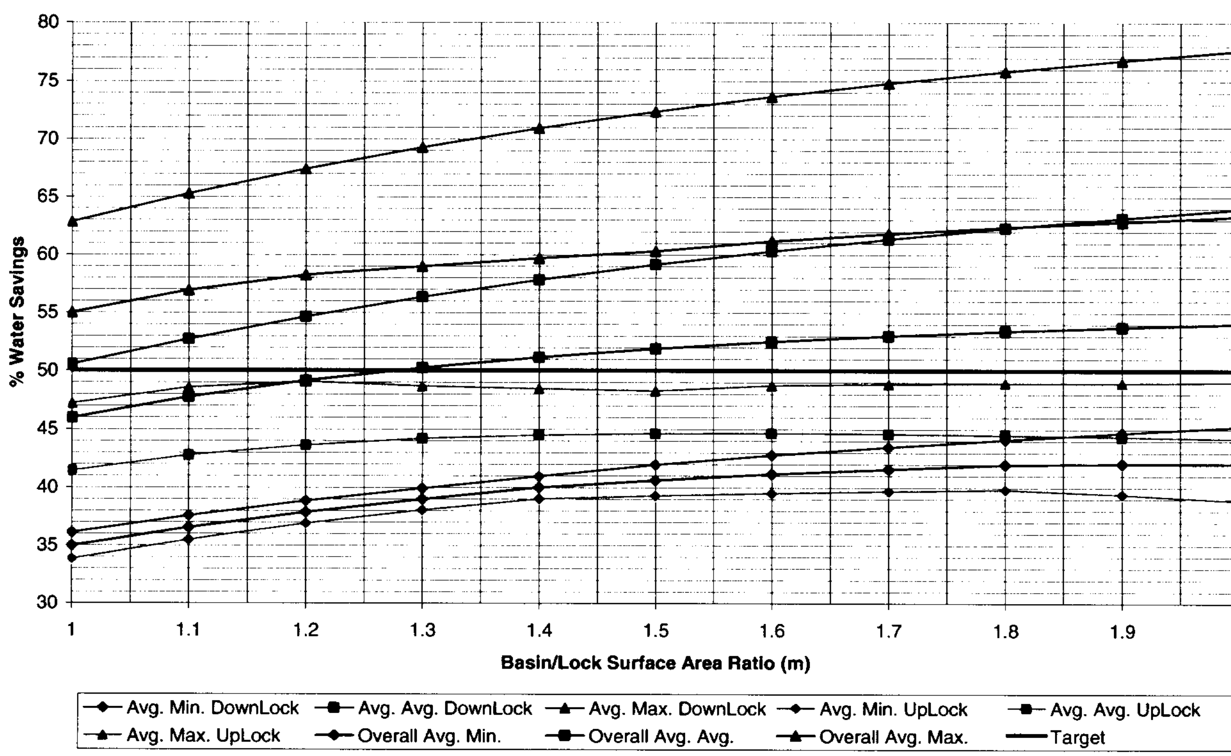


Figure IV.32 - % Water Savings vs. Basin/Lock Surface Area Ratio (m) – Pacific Side (Revised Analysis)

As can be seen from the previous graphs, it is not possible to achieve the target theoretical water saving percentage under all conditions, with the stacked basin arrangement, with basins only on one side of the lock, and with all of the variability that this project has in relation to water levels and lockage lengths. However, with a "m" factor of 1.3, the ***overall average*** water saving percentage is approximately the desired target of 50%. At some times the water savings percentage can approach 65% while at other times it will be less than 40%. The spreadsheet calculations can be seen in **Appendix E**. Nonetheless, for this conceptual level study, it was felt that an overall savings of 50% would achieve the goal of the concept study. The design could be modified at a later stage if additional water savings over a wider range of conditions are desired.

Figures IV.33-35 show the preliminary hydraulic layout and section views for the stacked basin arrangement for Option 4.

Figures IV. 36-37 show a schematic section view of the locks and the relative sizes and elevations of the side-by-side and stacked basin arrangements for comparative purposes. It is also interesting to note that in order to reach the theoretical water savings percentage even on an average basis requires 3.29m (10.8') of additional excavation in the upper lock on the Atlantic Side while an additional 3.72m (12.2') is required on the Pacific Side. This additional dredging will likely be in rock so the additional costs to reach this goal with a stacked basin arrangement (with basins only on one side) will need to be accounted for.

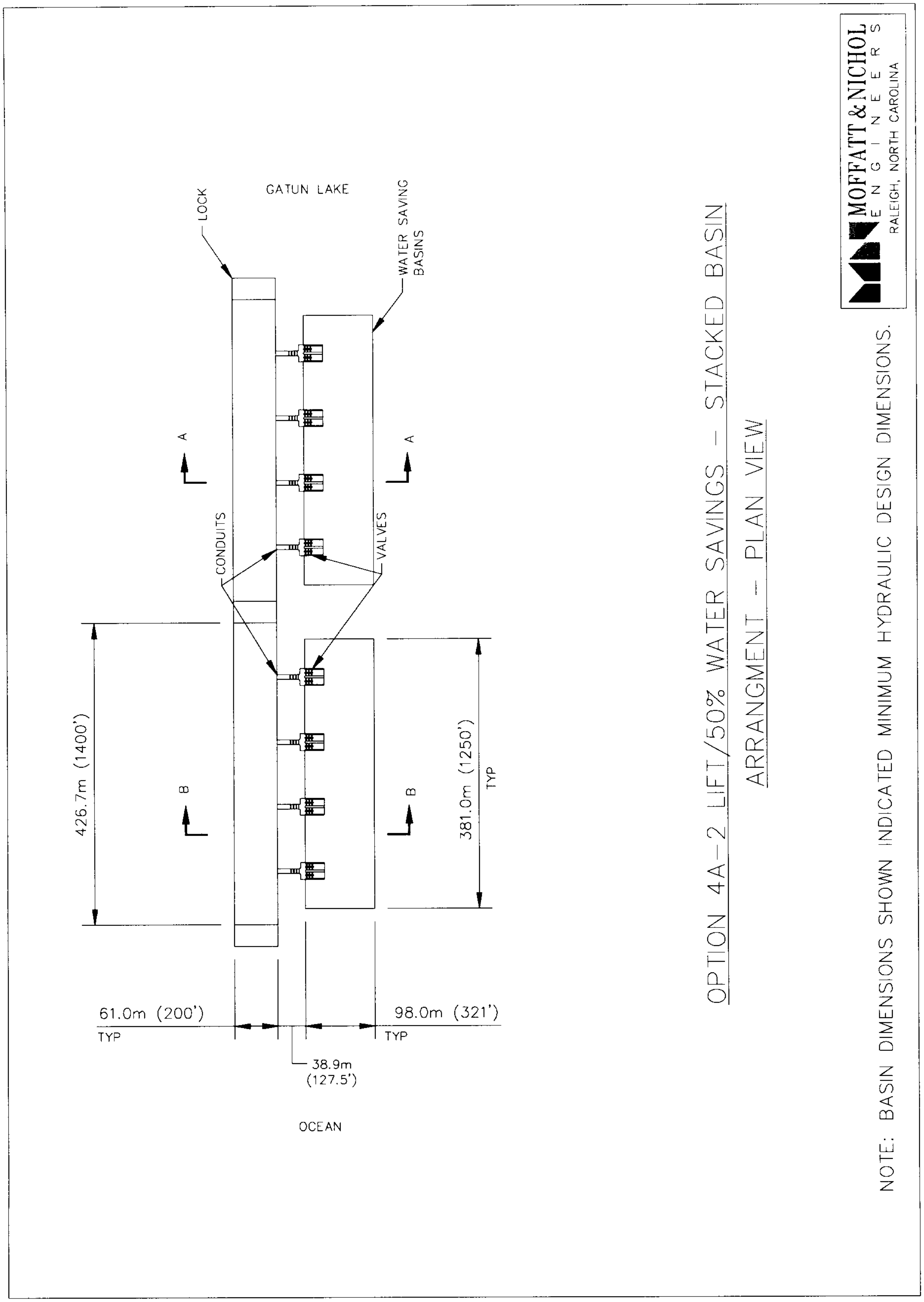


Figure IV.33 - Option 4A (Stacked Basins) – Plan View

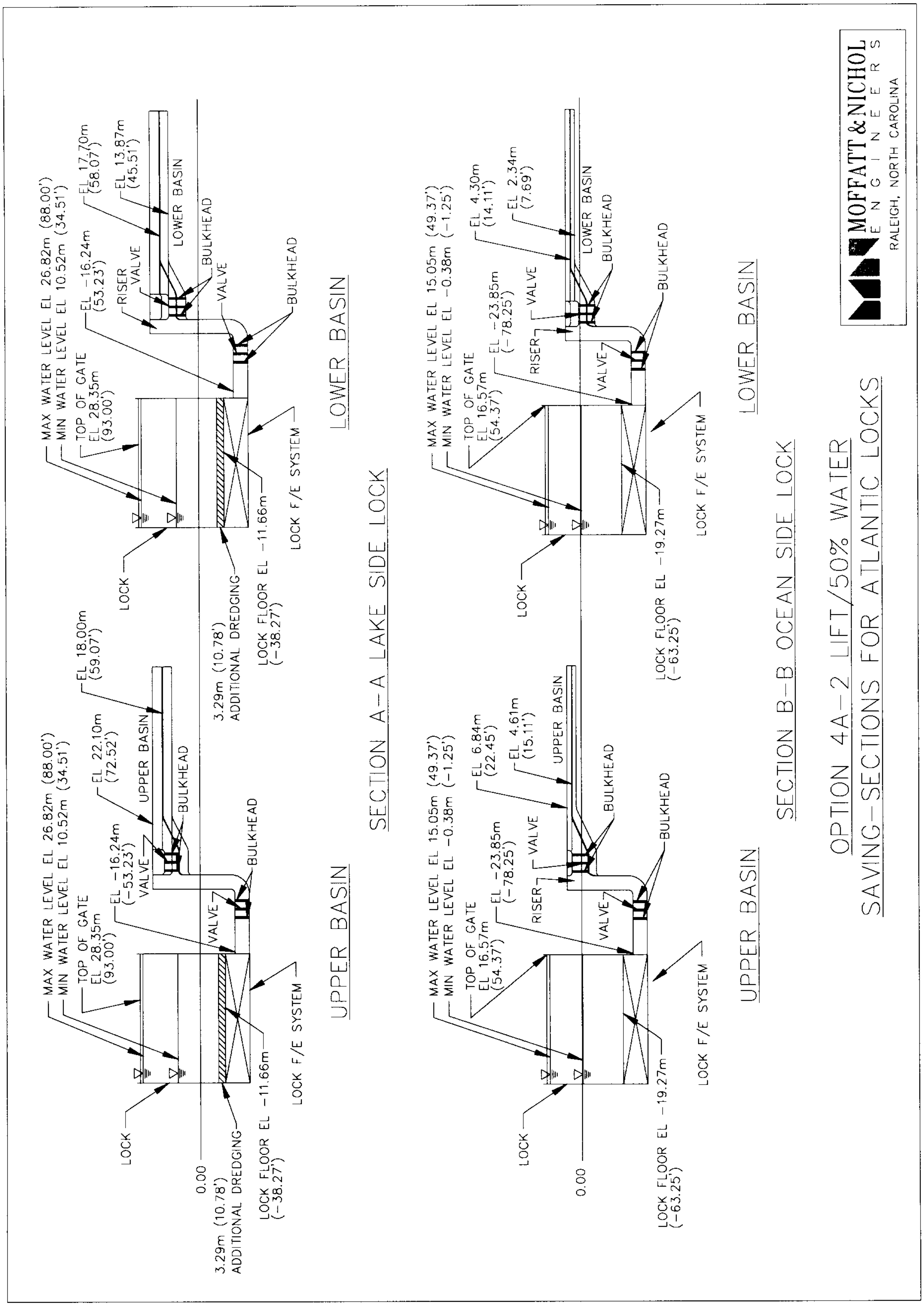


Figure IV.34 - Option 4A (Stacked Basins) – Section View (Atlantic Side)

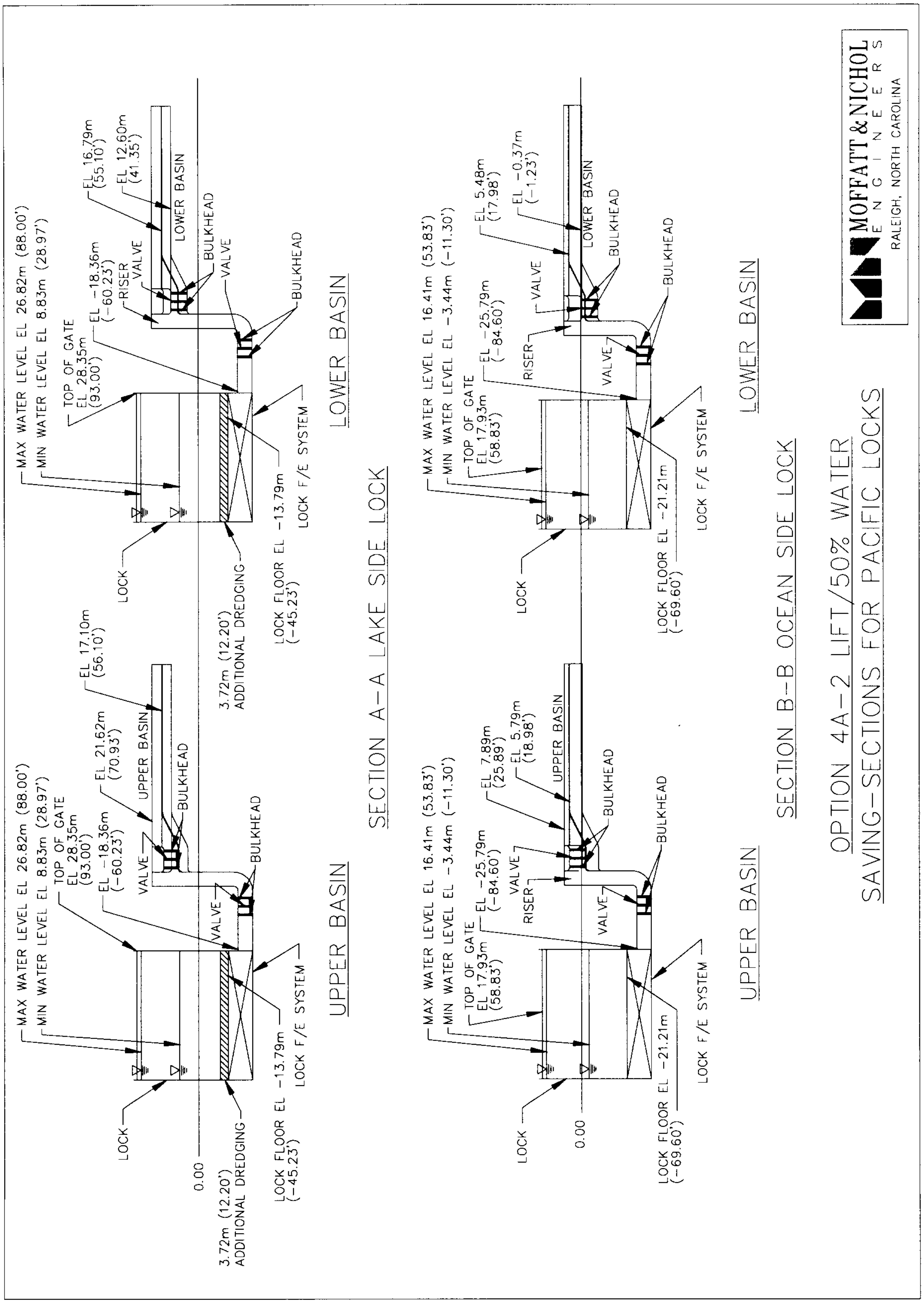


Figure IV.35 - Option 4A (Stacked Basins) – Section View (Pacific Side)

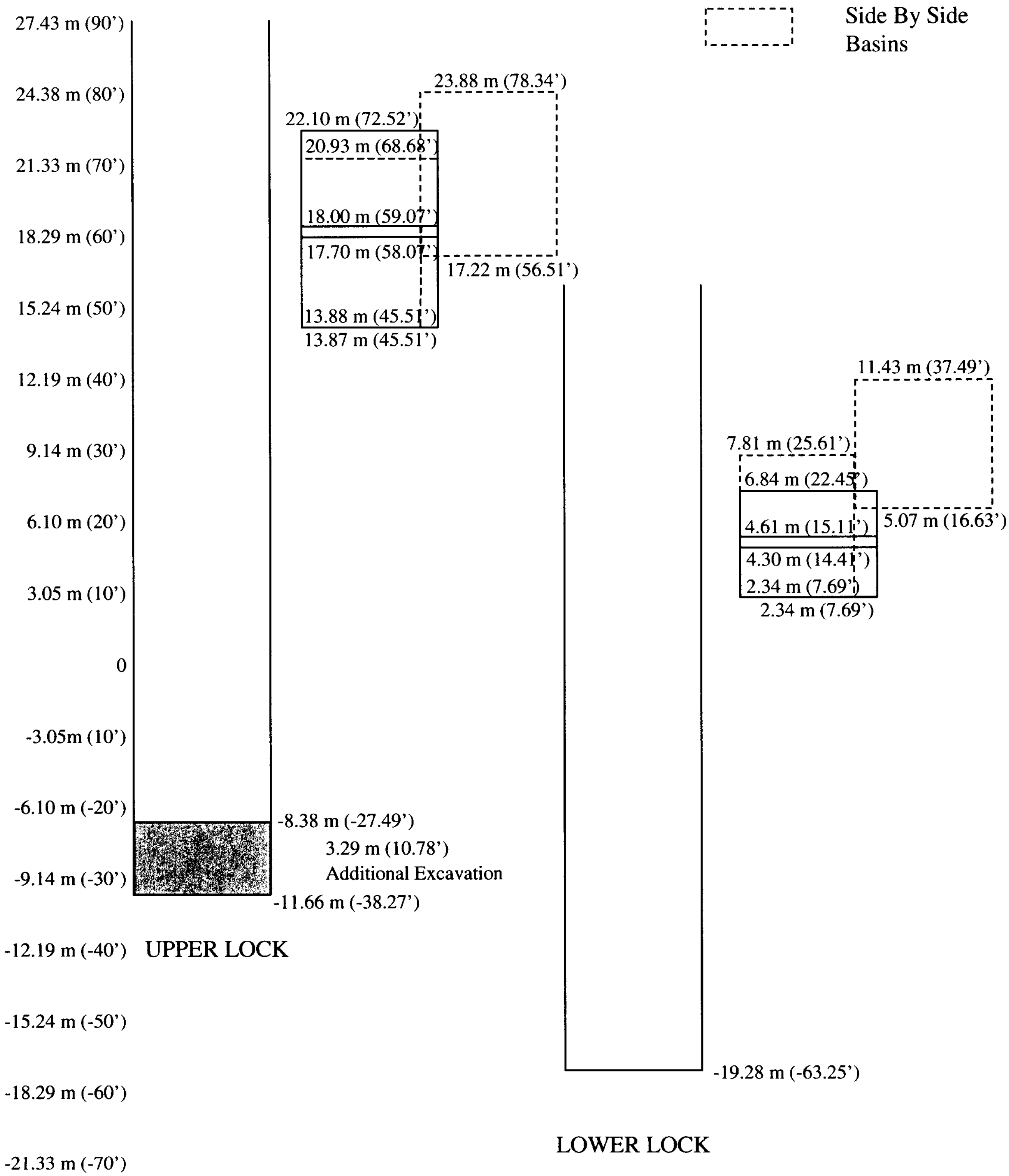


Figure IV.36 – Schematic Section Showing Comparison of Side-by-Side and Stacked Basin Arrangements -Atlantic Side

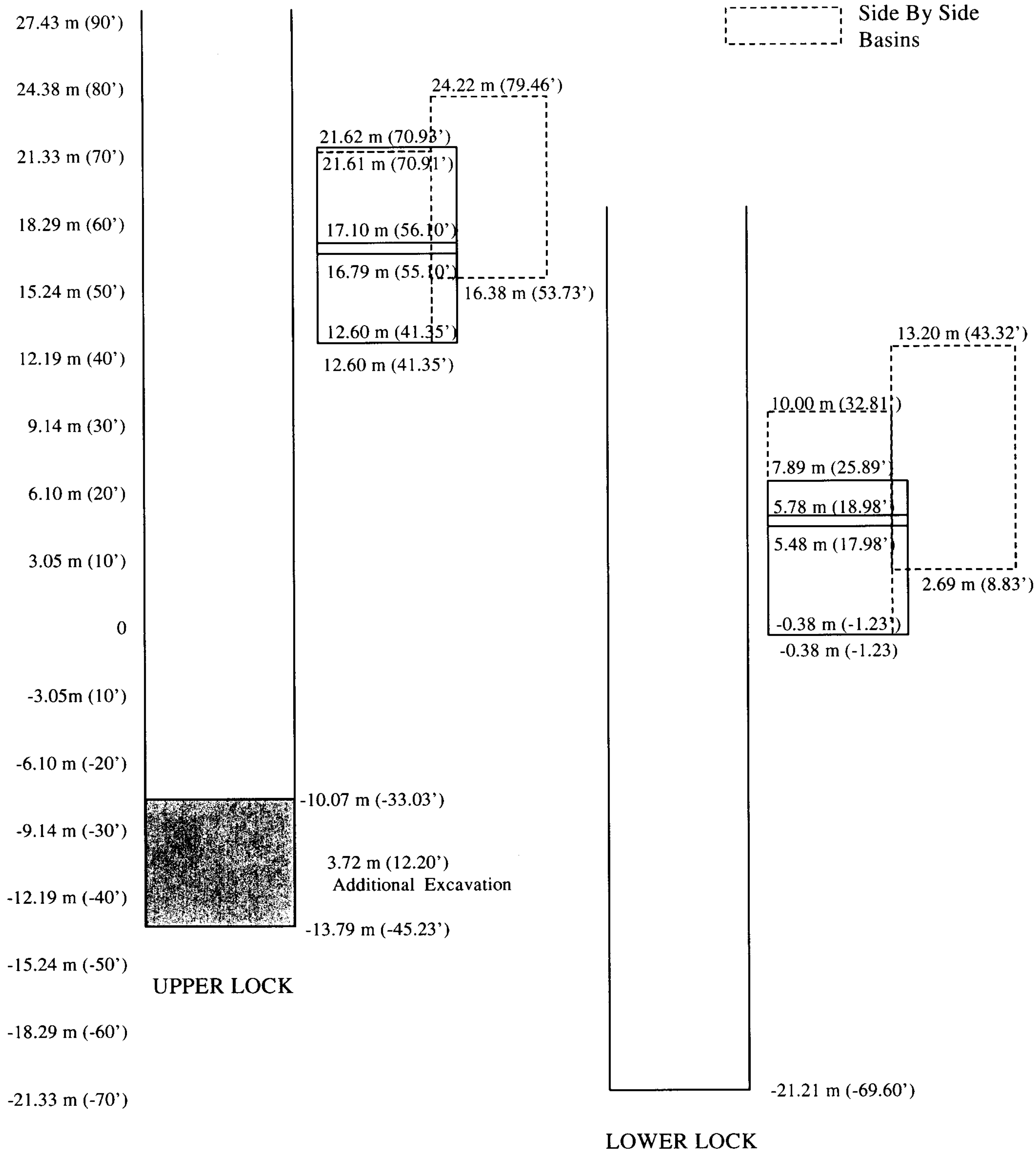


Figure IV.37 – Schematic Section Showing Comparison of Side-by-Side and Stacked Basin Arrangements - Pacific Side

h) Salinity Considerations

The impact of the salinity differential between the ocean and the lock water on equalization levels is another complicating factor that was not included at this conceptual stage but should be included in later design. This differential equalization level is due to the fact that the lock and ocean will not equalize to the same elevation since saline water is more dense than freshwater. Consequently, at the end of equalization, the water level in the lock may be higher than that in the ocean if there is a large salinity difference. After some preliminary calculations using the salinity data presented in **Chapter III**, and discussions with ACP staff, it was agreed that these effects could be incorporated at a later design phase since this phenomenon represents second or third order behavior when compared with tide differentials.

3. Structural/Civil Features Layout Design

a) Atlantic Side

(1) General

For centralized operations, it is desirable to locate the new lanes as close as possible to the existing lanes. As a stacked basin configuration has a significantly narrower footprint than a side-by-side basin configuration, ACP directed use of a stacked basin configuration with the new third lane (located between the existing lanes and the new third lane) and a side-by-side basin configuration with the new fourth lane (located east of the new fourth lane). Therefore, the side-by-side basin configurations, described below, are connected to the new fourth lane and are situated to the east of the A-2 alignment.

Based on hydraulic studies and resulting ACP direction, the original stacked basin configuration, intended to serve the new third lane locks, was abandoned as unfeasible. Therefore, in future conceptual designs, it will be necessary to shift Alignment A-2 to the east to accommodate side-by-side basins between the existing and new lanes, to serve the new third lane locks.

(a) Option 1 Configuration

Option 1 configuration of water saving basins is shown in **Figure IV.38**. Option 1 consists of six side-by-side basins for a three-lift lock, with two basins per lift. Each basin is 473 m (1551 ft.) long by 64 m (210 ft.) wide. Four 6.1 m x 6.1 m (20 ft. x 20 ft.) conduits per basin connect each basin to the new lock chamber.

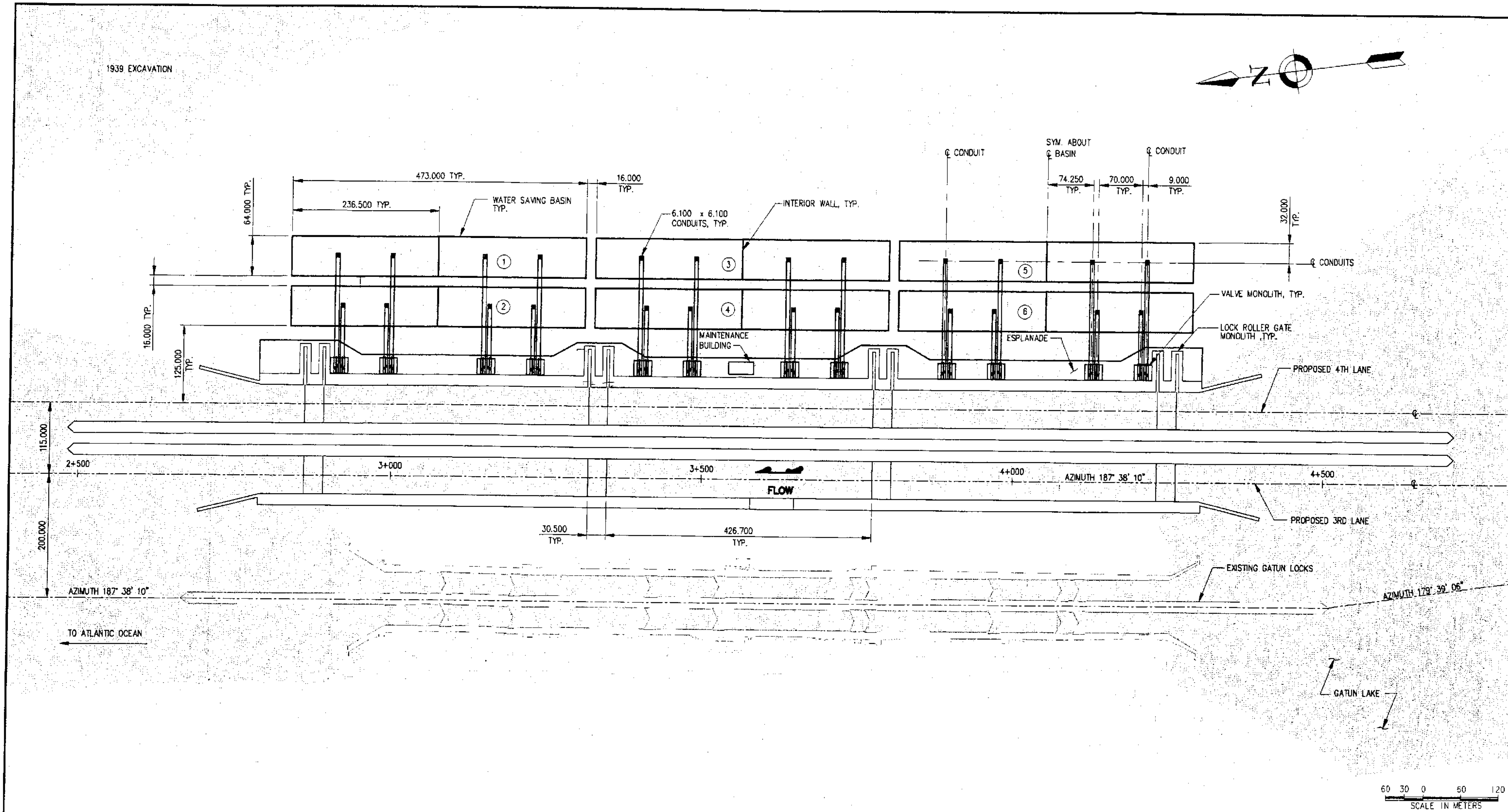
(b) Option 2 Configuration

Option 2 configuration of water saving basins is shown in **Figure IV.39**. Option 2 consists of six side-by-side basins for a two-lift lock, with three basins per lift. Each basin is 473 m (1551 ft.) long by 64 m (210 ft.) wide. Four 7.3 m x 7.3 m (24 ft. x 24 ft.) conduits per basin connect each basin to the new lock chamber.

(c) Option 3 Configuration

Option 3 configuration of water saving basins is shown in **Figure IV.40**. Option 3 consists of a total of eighteen side-by-side basins, with nine basins on each side of the new lock. This configuration is for a three-lift lock, with six basins per lift (three on each side). Each basin is

473 m (1551 ft.) long by 32 m (105 ft.) wide. One 8.5 m x 8.5 m (28 ft. x 28 ft.) conduit connects each basin to the lock chamber. Option 3 includes two transverse interior walls (placed at the basin quarter points) that divide each basin into three compartments, for the purpose of wave reduction. To allow flow to the conduits, openings must be included in the Option 3 interior walls. For maintenance purposes, it is recommended that these openings be fitted with removable maintenance bulkhead gates. The distance between the proposed new third lane locks and the proposed new fourth lane locks, along alignment A-2, is not adequate for basin placement in that area. In order to provide enough room for the proposed fourth lane basins, the fourth lane centerline must be shifted at least 204 m (669 ft.) to the east. Due to the large space requirements and lock spacings associated with the Option 3 configuration, centralized lock operations would be difficult in this case.



BASIN	TOP OF FLOOR SLAB ELEV.
1	3.260
2	1.440
3	10.930
4	8.920
5	19.440
6	17.200

NOTE:
ALL DIMENSIONS ARE IN METERS,
UNLESS OTHERWISE NOTED.

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

STUDY OF LOCKS WATER SAVING BASINS
OPTION 1 AT ALIGNMENT A-2

NCA ENGINEERS, INC. | FEBRUARY 2002 | FIGURE IV.38

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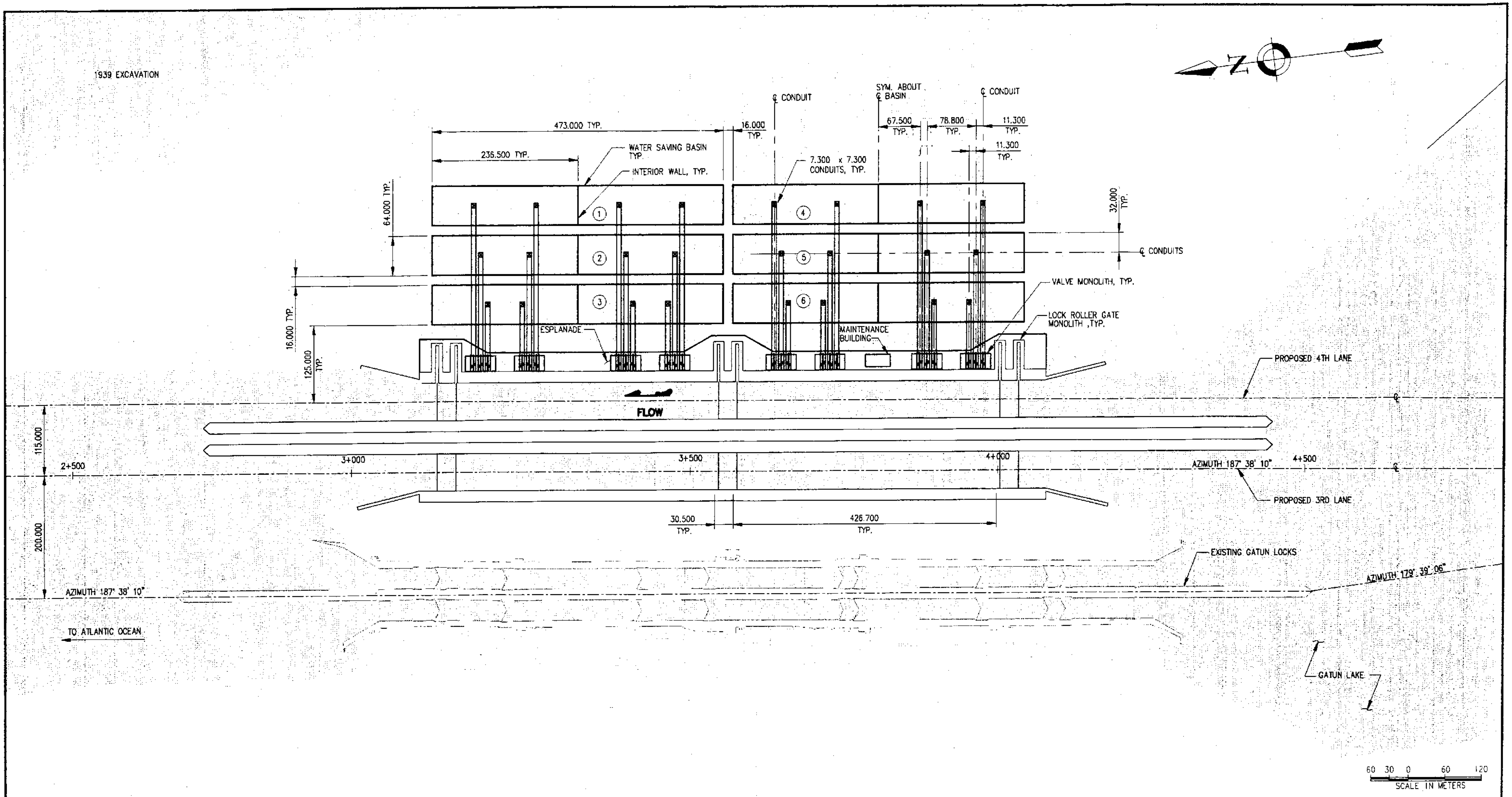

BASIN	TOP OF FLOOR SLAB ELEV.
1	6.160
2	3.980
3	1.800
4	18.570
5	15.880
6	13.200

NOTE:
ALL DIMENSIONS ARE IN METERS, UNLESS OTHERWISE NOTED.

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Oficina de Proyectos de Capacidad del Canal

STUDY OF LOCKS WATER SAVING BASINS
OPTION 2 AT ALIGNMENT A-2

NCA ENGINEERS INC. | FEBRUARY 2002 | FIGURE IV.39

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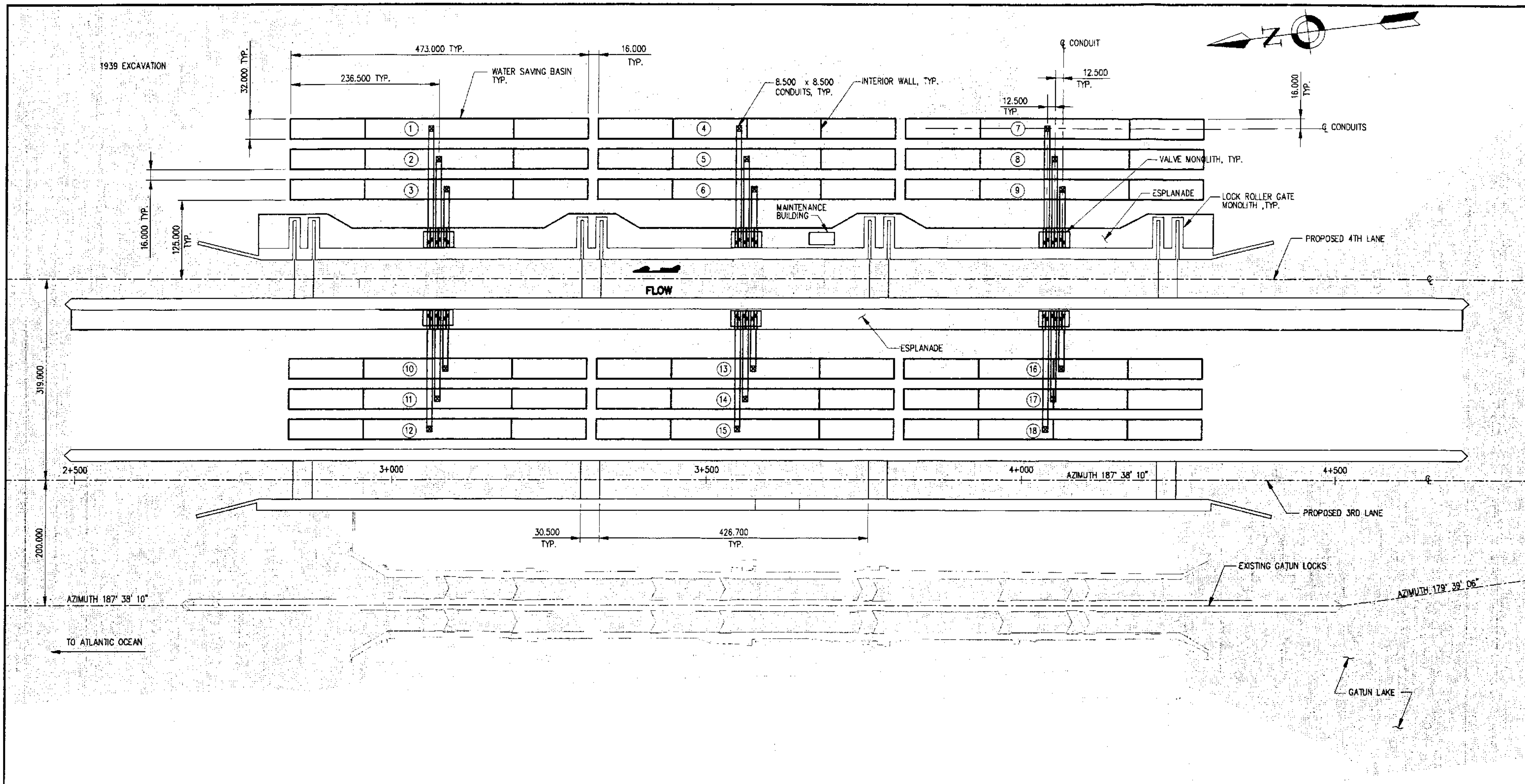

BASIN	TOP OF FLOOR SLAB ELEV.
1,12	4.000
2,11	2.540
3,10	1.080
4,15	11.740
5,14	10.130
6,13	8.520
7,18	20.340
8,17	18.540
9,16	16.750

NOTE:
ALL DIMENSIONS ARE IN METERS, UNLESS OTHERWISE NOTED.

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

STUDY OF LOCKS WATER SAVING BASINS
OPTION 3 AT ALIGNMENT A-2

NCA ENGINEERS INC. | FEBRUARY 2002 | FIGURE IV.40

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At the proposed new lock locations along Alignment A-2, there are a number of existing structures and roads. The impact of the conduit and water saving basin construction on these existing areas will be a function of the footprint of the selected basin configuration. Basin configuration Options 1 and 2 have similar effects on the existing areas adjacent to the locks, as they have similar footprint areas (approximately 330 hectares and 295 hectares for Options 1 and 2, respectively). The footprint area for Option 3, however, is approximately twice the size of the footprint areas for Options 1 and 2. Therefore, from the standpoint of landside impacts, Option 3 is the least desirable configuration.

(2) Soil and Rock Excavation

The existing topography varies significantly along the length of the proposed alignment. In order to minimize basin wall heights, it was assumed that the existing ground surface would be excavated to slope down to the top of basin walls at locations where the basins will be below the existing grade. Where a permanent slope is required immediately adjacent to the water saving basins, a minimum bench width of 5 meters (16 feet) was provided at the base of the excavation (top of the basin) for maintenance vehicle access.

In general, open cut excavation was assumed for the basin construction and braced or anchored vertical trench excavation was assumed for the conduit installation. Given the relatively short lengths of the conduits, tunneling methods are not anticipated to be cost effective for this application. Soil and rock excavation quantities, presented in the following sections, are bank volumes and do not include swell factors to account for the increased void ratios in the material as it is hauled and spoiled. If used to calculate haul quantities or uncompacted spoil volumes, the following excavation values should be increased on the order of 20 to 30 percent, to account for swell.

Depending on the basin configuration considered, approximately 50 percent of the basin floors are founded on rock. Rock will provide a competent foundation for the basins. However, based on the geotechnical recommendations, the existing overburden is unsuitable (it may produce unacceptable differential settlement) as a foundation for the basin floors, unless it has been preloaded with 150 percent of the design load. Even with preload, the existing overburden is not an acceptable foundation material if it is found to include decaying organic matter. As a result, a foundation of either select backfill or drilled shafts was considered.

In order to evaluate these two foundation types, Option 2 was selected for the purpose of cost comparison. It was found that, for Option 2 configuration, approximately 690,000 m^3 (900,000 cy) of additional soil excavation, 410,000 m^3 (540,000 cy) of select backfill, and 20,000 m^3 (26,000 cy) of common backfill (replacement of existing overburden) would be required for a select backfill foundation. For the same configuration, approximately 18,500 m^3 (24,000 cy) of additional floor slab concrete, 4,100 m^3 (5,400 cy) of shaft soil excavation, 4,100 m^3 (5,400 cy) of shaft rock excavation, and 8,200 m^3 (11,000 cy) of additional shaft concrete would be required for a drilled shaft foundation.

Based on these quantities and preliminary unit cost estimates, the select fill and drilled shaft foundations are expected to have very similar construction costs. The preliminary costs for these foundation systems are well within the margin of error for this conceptual study. The geotechnical report indicates that the rock excavated for the new locks may meet the criteria required for the select fill material. If this rock can be crushed and reused as select fill, it is likely that the select fill foundation would be more cost effective for the basins. Therefore, this foundation was assumed for all Atlantic site configurations, where the basins were not founded on rock. However, this foundation assumption should be reevaluated during future design phases, as the rock contours are better defined and the foundation designs are developed further.

At the Atlantic site, the top of rock elevations were provided in 3-D Autocad files, provided by ACP. These rock surface files were then used to generate cross sections so that soil and rock excavation quantities could be determined. This surface was sufficient for conceptual design purposes, but additional rock contour data may be required in future design phases.

(a) Option 1 Configuration

For the Atlantic Option 1 configuration, approximately 7,400,000 m^3 (9,700,000 cy) of soil excavation and 700,000 m^3 (900,000 cy) of rock excavation is required for the water saving basins. In addition, it is estimated that approximately 1,700,000 m^3 (2,200,000 cy) and 800,000 m^3 (1,000,000 cy) of select fill and native backfill, respectively, are required. For the conduits, approximately 73,000 m^2 (790,000 sf) of braced or anchored trench wall is required. Avoiding duplication of basin and lock excavation estimates, approximately 55,000 m^3 (72,000 cy) of additional soil excavation and 520,000 m^3 (680,000 cy) of rock excavation are required for these vertical trenches.

(b) Option 2 Configuration

For the Atlantic Option 2 configuration, approximately 5,300,000 m^3 (7,000,000 cy) of soil excavation and 1,100,000 m^3 (1,400,000 cy) of rock excavation is required for the water saving basins. In addition, it is estimated that approximately 410,000 m^3 (540,000 cy) and 560,000 m^3 (730,000 cy) of select fill and native backfill, respectively, are required. For the conduits, approximately 79,000 m^2 (850,000 sf) of braced or anchored trench wall is required. Avoiding duplication of basin and lock excavation estimates, approximately 88,000 m^3 (115,000 cy) of additional soil excavation and 930,000 m^3 (1,200,000 cy) of rock excavation are required for these vertical trenches.

(c) Option 3 Configuration

For the Atlantic Option 3 configuration, approximately 12,000,000 m^3 (15,600,000 cy) of soil excavation and 1,700,000 m^3 (2,200,000 cy) of rock excavation is required for the water saving basins. In addition, it is estimated that approximately 2,500,000 m^3 (3,300,000 cy) and 1,600,000 m^3 (2,100,000 cy) of select fill and native backfill, respectively, are required. For the conduits, approximately 34,000 m^2 (365,000 sf) of braced or anchored trench wall is required. Avoiding duplication of basin and lock excavation estimates, approximately 70,000 m^3 (92,000 cy) of additional soil excavation and 460,000 m^3 (600,000 cy) of rock excavation are required for these vertical trenches.

(d) Summary

For the Atlantic site, the following tables summarize the total excavation and fill requirements for the basin options that were studied:

Table IV.2 – Excavation Quantities for Options 1 – 3 (Atlantic Side)

	Basin and Conduit Excavation	
	Soil Excavation	Rock Excavation
Option 1	$7.5x10^6$ m^3 ($9.8x10^6$ cy)	$1.2x10^6$ m^3 ($1.6x10^6$ cy)
Option 2	$5.4x10^6$ m^3 ($7.1x10^6$ cy)	$2.0x10^6$ m^3 ($2.6x10^6$ cy)
Option 3	$12.0x10^6$ m^3 ($15.7x10^6$ cy)	$2.1x10^6$ m^3 ($2.7x10^6$ cy)

Table IV.3 – Fill Quantities for Options 1 – 3 (Atlantic Side)

	Basin Fill	
	Select Fill	Native Fill
Option 1	$1.7x10^6$ m^3 ($2.2x10^6$ cy)	$0.8x10^6$ m^3 ($1.0x10^6$ cy)
Option 2	$0.4x10^6$ m^3 ($0.5x10^6$ cy)	$0.6x10^6$ m^3 ($0.8x10^6$ cy)
Option 3	$2.5x10^6$ m^3 ($3.3x10^6$ cy)	$1.6x10^6$ m^3 ($2.1x10^6$ cy)

b) Pacific Side

(1) General

(a) Option 1 Configuration

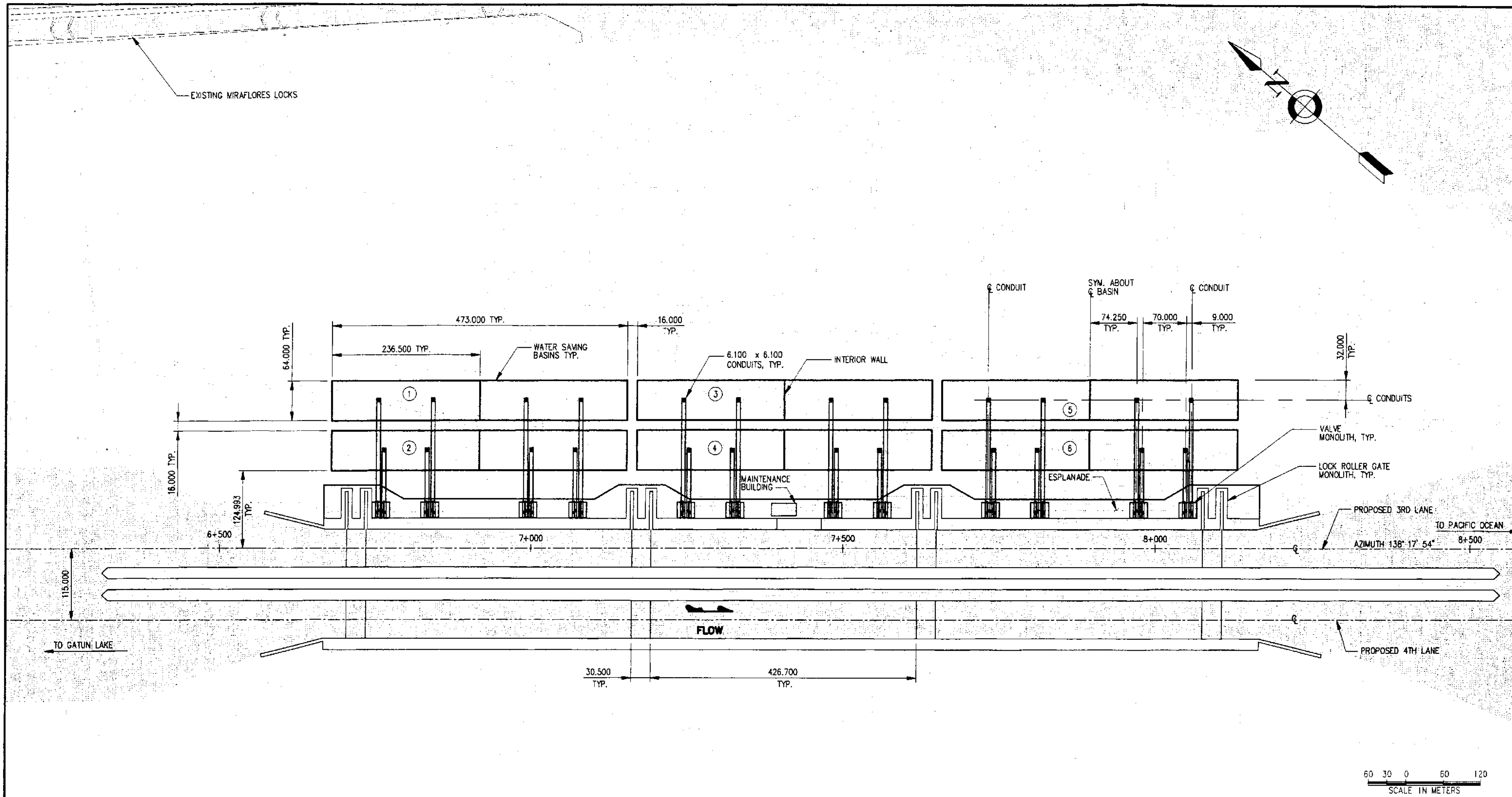
Option 1 configuration of water saving basins is shown in **Figure IV.41**. As can be seen from this drawing, Option 1 consists of six side-by-side basins for a three-lift lock, with two basins per lift. Each basin is 473 m (1551 ft.) long by 64 m (210 ft.) wide. Four 6.1 m x 6.1 m (20 ft. x 20 ft.) conduits per basin connect each basin to the new lock chamber.

(b) Option 2 Configuration

Option 2 configuration of water saving basins is shown in **Figure IV.42**. As can be seen from this drawing, Option 2 consists of six side-by-side basins for a two-lift lock, with three basins per lift. Each basin is 473 m (1551 ft.) long by 64 m (210 ft.) wide. Four 7.3 m x 7.3 m (24 ft. x 24 ft.) conduits per basin connect each basin to the new lock chamber.

(c) Option 3 Configuration

Option 3 configuration of water saving basins is shown in **Figure IV.43**. As can be seen from this drawing, Option 3 consists of a total of eighteen side-by-side basins, with nine basins on each side of the new lock. This configuration is for a three-lift lock, with six basins per lift. Each basin is 473 m (1551 ft.) long by 32 m (105 ft.) wide. One 8.5 m x 8.5 m (28 ft. x 28 ft.) conduit per basin connects each basin to the new lock chamber. Option 3 includes two transverse interior walls (placed at the basin quarter points) that divide each basin into three compartments, for the purpose of wave reduction. To allow flow to the conduits, openings must be included in the Option 3 interior walls. The distance between the proposed new third and fourth lanes, along alignment P-1, is not adequate for basin placement in that area. In order to provide enough room for the proposed third lane basins, the proposed fourth lane centerline must be shifted at least 204 m (669 ft.) to the east. Due to the large space requirements and lock spacing associated with the Option 3 configuration, centralized lock operations would be difficult in this case.



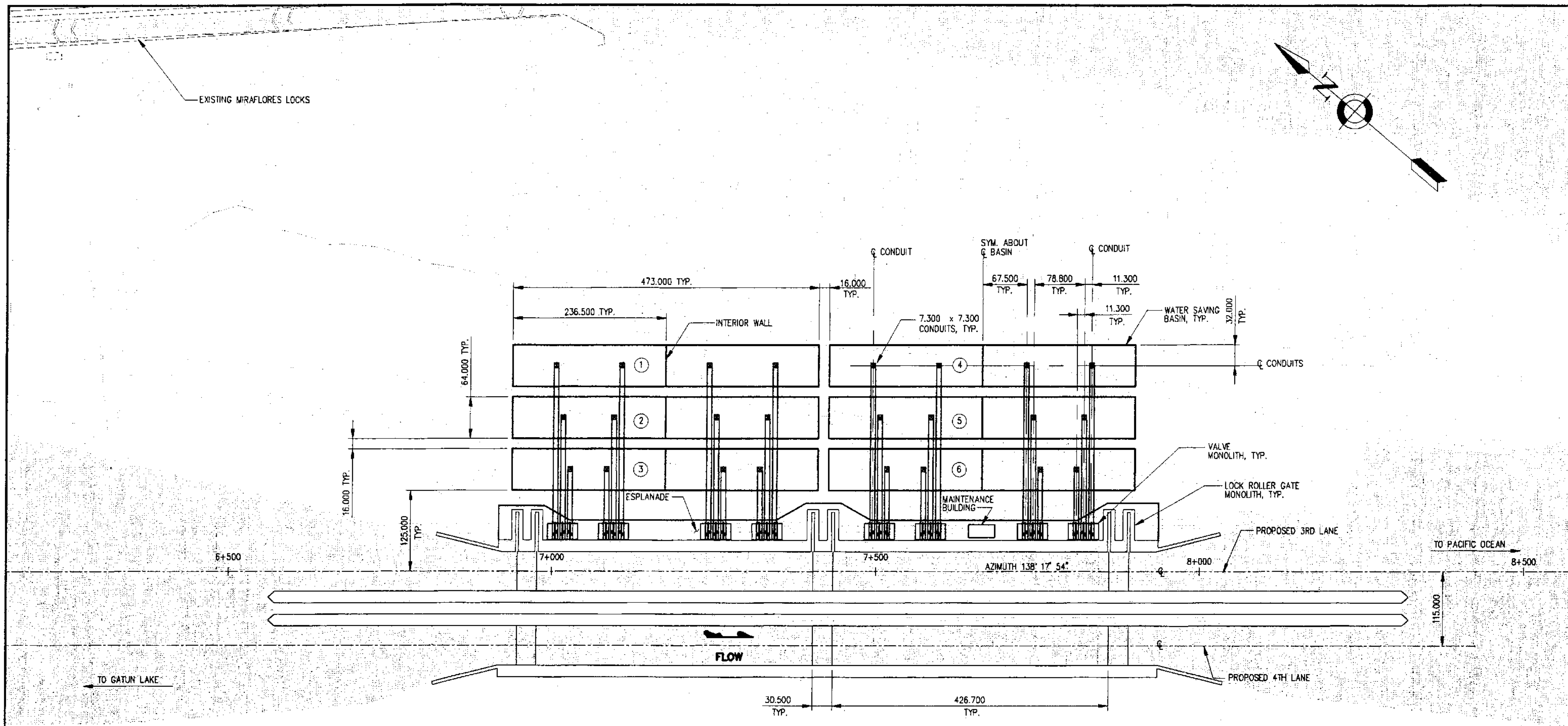
BASIN	TOP OF FLOOR SLAB ELEV.
1	18.880
2	16.350
3	9.300
4	7.030
5	0.660
6	-1.390

NOTE:
ALL DIMENSIONS ARE IN METERS, UNLESS OTHERWISE NOTED.

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

STUDY OF LOCKS WATER SAVING BASINS
OPTION 1 AT ALIGNMENT P-1

INCA ENGINEERS INC. | FEBRUARY 2002 | FIGURE IV.41



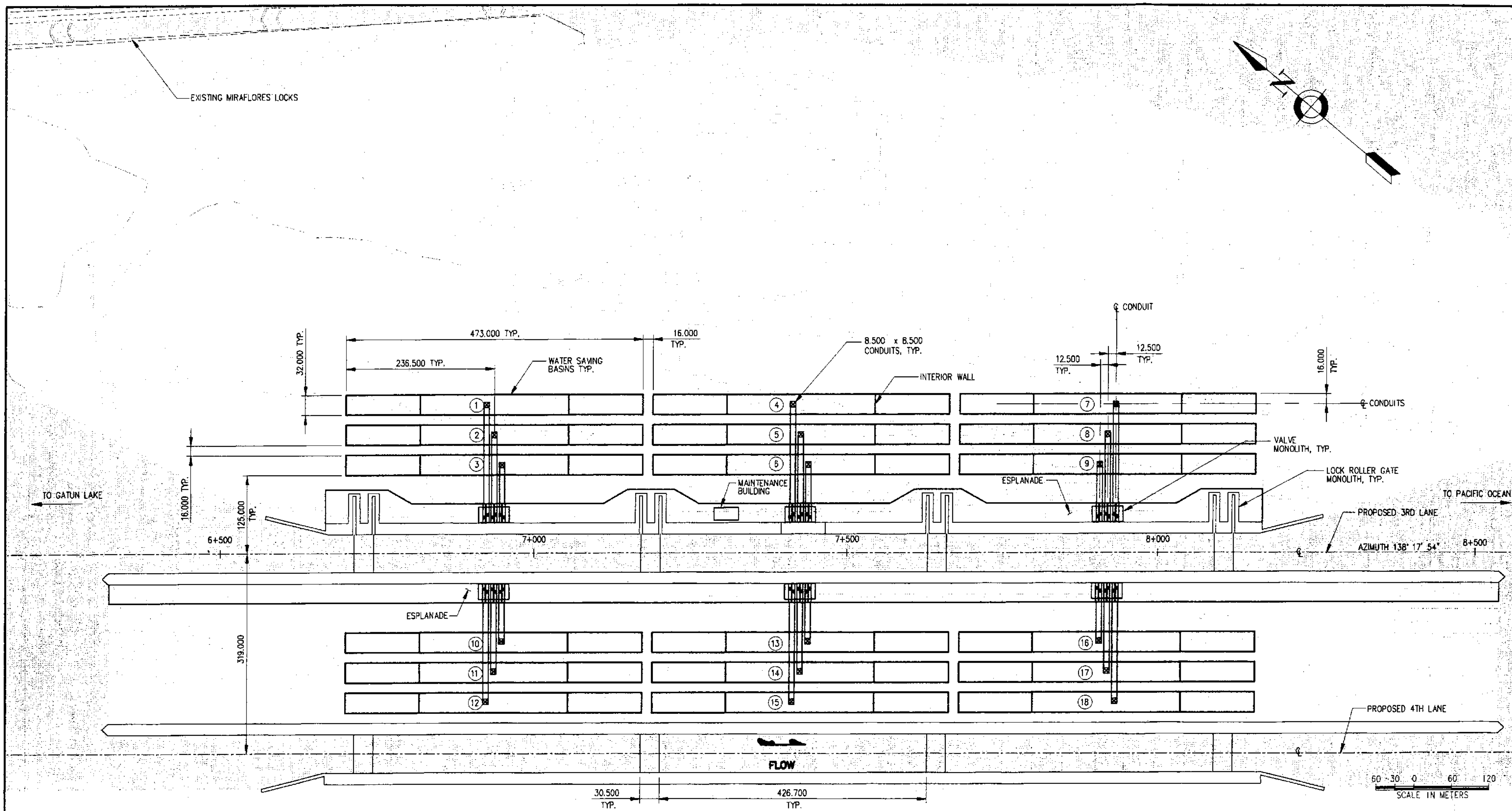
BASIN	TOP OF FLOOR SLAB ELEV.
1	17.890
2	14.870
3	11.850
4	3.920
5	1.470
6	-0.990

NOTE:
ALL DIMENSIONS ARE IN METERS, UNLESS OTHERWISE NOTED.

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

STUDY OF LOCKS WATER SAVING BASINS
OPTION 2 AT ALIGNMENT P-1

INCA ENGINEERS INC. | FEBRUARY 2002 | FIGURE IV.42



BASIN	TOP OF FLOOR SLAB ELEV.
1,12	19.890
2,11	17.870
3,10	15.850
4,15	10.200
5,14	8.390

BASIN	TOP OF FLOOR SLAB ELEV.
6,13	6.580
7,18	1.480
8,17	-0.160
9,16	-1.800

NOTE:
ALL DIMENSIONS ARE IN METERS, UNLESS OTHERWISE NOTED.

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

STUDY OF LOCKS WATER SAVING BASINS
OPTION 3 AT ALIGNMENT P-1

NCA ENGINEERS INC. | FEBRUARY 2002 | FIGURE IV.43

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(2) Soil and Rock Excavation

The existing topography varies significantly along the length of the proposed alignment. In order to minimize basin wall heights, it was assumed that the existing ground surface would be excavated to slope down to the top of basin walls at locations where the basins will be below the existing grade. In order to provide foundation support in areas where the basins will be above the existing grade, select fill material will be placed under and around the basins. Where permanent cuts or fills are required immediately adjacent to the water saving basins, a minimum bench width of 5meters (16 feet) was provided for maintenance vehicle access.

In general, open cut excavation was assumed for the basin construction and braced or anchored vertical trench excavation was assumed for the conduit installation. Given the relatively short lengths of the conduits, tunneling methods are not anticipated to be cost effective for this application. Also, the geotechnical review indicated that the La Boca rock was unsuitable for tunneling.

Many of the Pacific site basins will be founded on rock. However, some of the basins may extend into the 1939 channel excavation, and these basins as well as other basins may be founded above the rock. Based on the geotechnical recommendations, either the soft unsuitable overburden shall be removed and replaced with select backfill or a drilled shaft foundation shall be utilized. The select fill foundation is anticipated to be more cost effective for the majority of the affected basins. Therefore, this foundation was assumed for all Pacific site basins that were not founded on rock.

Soil and rock excavation quantities, presented in the following sections, are bank volumes and do not include swell factors to account for the increased void ratios in the material as it is hauled and spoiled. If used to calculate haul quantities or uncompacted spoil volumes, the following excavation values should be increased on the order of 20 to 30 percent, to account for swell.

At the Pacific site, ACP provided In-Roads files for the rock surface data. However, due to software incompatibilities, it was not possible to import these files. Therefore, the rock surface was recreated from top of rock data points that were obtained from boring logs, supplied by ACP. This rock surface was then used to generate cross sections so that soil and rock excavation quantities could be determined. Rock surface data was limited, especially west of the proposed new third lane alignment. While this data was sufficient for conceptual design purposes, the rock contour data will need to be expanded significantly for future design phases, particularly for new fourth lane designs.

(a) Option 1 Configuration

For the Pacific Option 1 configuration, approximately 2,200,000 m^3 (2,900,000 cy) of soil excavation and 1,800,000 m^3 (2,300,000 cy) of rock excavation is required for the water saving basins. In addition, it is estimated that approximately 1,400,000 m^3 (1,800,000 cy) and 1,000,000 m^3 (1,300,000 cy) of select fill and native backfill, respectively, are required. For the conduits, approximately 80,000 m^2 (860,000 sf) of braced or anchored trench wall is required.

Avoiding duplication of basin and lock excavation estimates, approximately 30,000 m^3 (40,000 cy) of additional soil excavation and 600,000 m^3 (790,000 cy) of rock excavation are required for these vertical trenches.

(b) Option 2 Configuration

For the Pacific Option 2 configuration, approximately 2,500,000 m^3 (3,300,000 cy) of soil excavation and 1,800,000 m^3 (2,400,000 cy) of rock excavation is required for the water saving basins. In addition, it is estimated that approximately 1,100,000 m^3 (1,400,000 cy) and 1,000,000 m^3 (1,300,000 cy) of select fill and native backfill, respectively, are required. For the conduits, approximately 76,000 m^2 (820,000 sf) of braced or anchored trench wall is required. Avoiding duplication of basin and lock excavation estimates, approximately 26,000 m^3 (34,000 cy) of additional soil excavation and 1,000,000 m^3 (1,300,000 cy) of rock excavation are required for these vertical trenches.

(c) Option 3 Configuration

For the Pacific Option 3 configuration, approximately 4,800,000 m^3 (6,300,000 cy) of soil excavation and 3,300,000 m^3 (4,400,000 cy) of rock excavation is required for the water saving basins. In addition, it is estimated that approximately 1,800,000 m^3 (2,400,000 cy) and 2,100,000 m^3 (2,700,000 cy) of select fill and native backfill, respectively, are required. For the conduits, approximately 38,000 m^2 (405,000 sf) of braced or anchored trench wall is required. Avoiding duplication of basin and lock excavation estimates, approximately 27,000 m^3 (36,000 cy) of additional soil excavation and 550,000 m^3 (700,000 cy) of rock excavation are required for these vertical trenches.

(d) Summary

For the Pacific site, the following tables summarize the total excavation and fill requirements for the basin options that were studied:

Table IV.4 – Excavation Quantities for Options 1 – 3 (Pacific Side)

	Basin and Conduit Excavation	
	Soil Excavation	Rock Excavation
Option 1	$2.2 x 10^6$ m^3 ($2.9 x 10^6$ cy)	$2.4 x 10^6$ m^3 ($3.1 x 10^6$ cy)
Option 2	$2.5 x 10^6$ m^3 ($3.3 x 10^6$ cy)	$2.8 x 10^6$ m^3 ($3.7 x 10^6$ cy)
Option 3	$4.8 x 10^6$ m^3 ($6.3 x 10^6$ cy)	$3.8 x 10^6$ m^3 ($5.0 x 10^6$ cy)

Table IV.5 – Fill Quantities for Options 1 – 3 (Pacific Side)

	Basin Fill	
	Select Fill	Native Fill
Option 1	$1.4 x 10^6$ m^3 ($1.8 x 10^6$ cy)	$1.0 x 10^6$ m^3 ($1.3 x 10^6$ cy)
Option 2	$1.1 x 10^6$ m^3 ($1.4 x 10^6$ cy)	$1.0 x 10^6$ m^3 ($1.3 x 10^6$ cy)
Option 3	$1.8 x 10^6$ m^3 ($2.4 x 10^6$ cy)	$2.1 x 10^6$ m^3 ($2.7 x 10^6$ cy)

V. FEATURES DESIGN

A. Hydraulic Features Design

1. General Assumptions

With the features layout design complete, the next step was to complete a conceptual features design of the water saving basin conduits (both hydraulic and structural designs), basin walls, operators, valves, etc. Please note that in the following discussion and for the remainder of the report the use of the term "conduit" is used for the water saving basins water conveyances while the term "culverts" is used for lock to lock hydraulic conveyances.

The assumptions included:

- the geometry of sections and layouts used in the features design would be those determined in **Chapter IV – Features Layout** and
- the valve opening/closing times would be one minute (based on the existing locks – this would later be confirmed or refined based on ongoing discussions with valve manufacturers).

2. Design Procedures

a) Hydraulic Theory and Design Approach

(1) Design of the Existing Locks

In any conceptual design study, it is first helpful to look at projects of the same size and scale in order to gain insights into possible behaviors and into procedures that were used to design those facilities. In this case, there are very few projects of this size and scale where design procedures and outcomes are readily available. Nonetheless, considerable design information was available for the existing locks in the 1915 International Engineering Congress which dealt entirely with the design of the existing Panama Canal.

First, the engineers reported that for the existing locks, a maximum F/E rate of 7.5 ft/min (rate of change of water levels) was considered safe for ships transiting the locks. This was subsequently demonstrated based on operational experience. Secondly, detailed explanations of the system geometry and head loss coefficients were discussed for the F/E system.

(2) General Design Analysis Approach

After an exhaustive search for available models that could be applied to closed conduit systems with equalizing heads, the selected tools were a spreadsheet developed in-house and the LOCKSIM model developed by the Tennessee Valley Authority (TVA).

A printout of the spreadsheet model can be seen in **Appendix F**. It is largely based on procedures and equations found within WES MPHL-89-5, "Hydraulic Design of Navigation

Locks by John P. Davis and the USACOE's EM 1110-2—1604, "Hydraulic Design of Navigation Locks".

The LOCKSIM (LOCK SIMulation) program was developed by Dr. Jerry Schohl of the TVA and has been used extensively by the U.S. Army Corps of Engineers for various lock designs around the United States. LOCKSIM is a one-dimensional, finite-difference, unsteady flow model specifically written for lock F/E systems with special functions included for reverse tainter valves, valve wells, manifolds, t-sections, etc. It also allows for accurate modeling with true elevations so that expected hydraulic grade lines (HGL'S) and pressures can be calculated to avoid cavitation concerns. LOCKSIM can also estimate hawser stresses based on the differential water levels in the lock. Unfortunately, the model has only a limited user interface, so that input files are created by hand in text editors and output files must be manipulated and imported into spreadsheets to produce plots. Examples input and output files can be seen in **Appendix F**.

With the large number of runs expected and the time required to pre- and post-process the LOCKSIM input files and output files, it was decided that the spreadsheet model would be the primary analysis tool. Before processing, however, a test case was devised to compare with LOCKSIM to ensure its validity.

Both models require descriptions of geometry and head loss coefficients for all loss producing features of the system.

The following head losses were included (see **Figure V.1**):

(1) Entrance/Manifold
(2) Bulkhead Slots
(3) Valve Well
(4) Valves
(5) Frictional Resistance
(6) Bends and Transitions and Junctions
(7) Exit/Manifold Losses

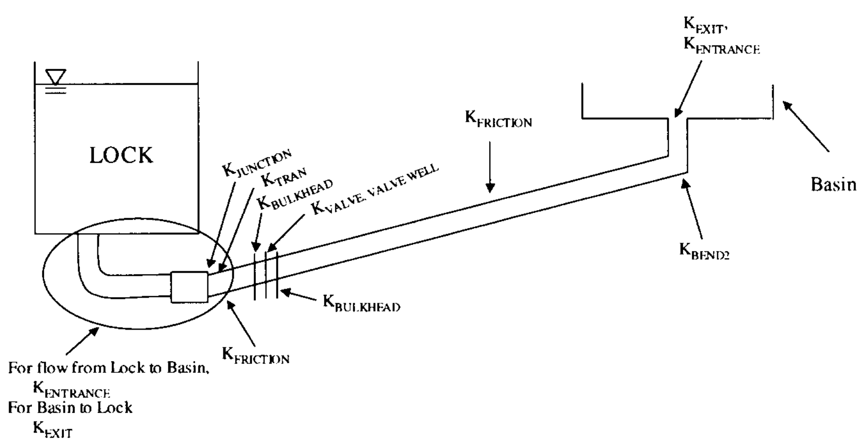


Figure V.1 – Schematic Section Showing Locations of Head Loss Coefficients Used in Hydraulic Analyses

The head loss coefficients specified for the models were devised from four main sources:

- Davis, John P., "Hydraulic Design of Navigation Locks", USCOE, 1989.,
- Miller, D. S., Internal Flow Systems, BHR Group, 1990.,
- Hydraulic Design Criteria – Volumes 1 & 2, United States Corps of Engineers, 1980., and
- Schohl, Gerald A., "User's Manual for LOCKSIM: Hydraulic Simulation of Navigation Lock Filling and Emptying Systems", 1999.

For the comparative run, models with identical geometries, head losses, and initial water surface elevations were set up in the spreadsheet and LOCKSIM. The resulting time history of water surface elevations in the lock for both models can be seen in **Figure V.2.** The predicted behavior is essentially identical.

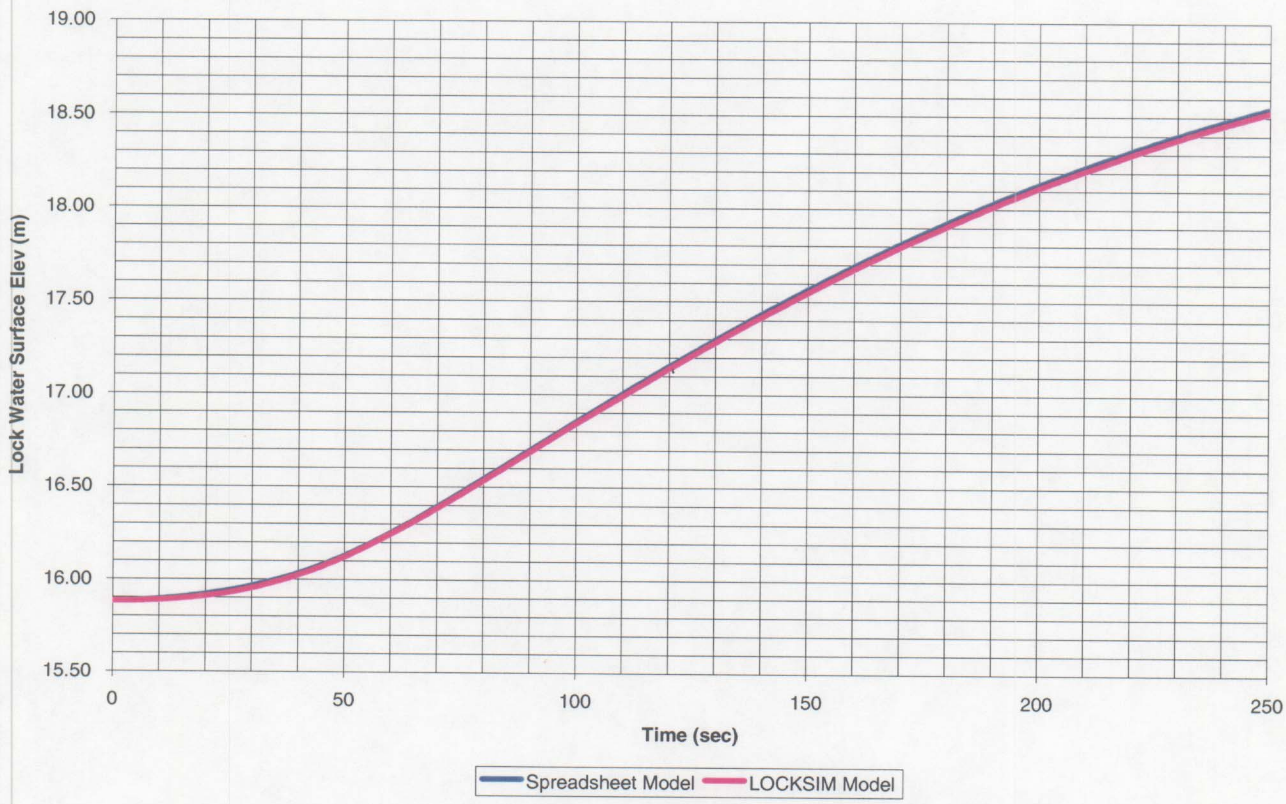


Figure V.2 – Comparison of Spreadsheet Model to LOCKSIM Model Results

Having been verified through this sample test, the spreadsheet model was then used to determine preliminary sizes for the WSB conduits.

b) Design Guidelines

Design criteria were needed to guide selection of optimal conduits arrangements and sizes. Given the excellent performance record of the existing locks, it was felt that some if its operational properties would be appropriate starting points for the new locks and water saving basins. These included:

- the maximum instantaneous F/E rate shall not exceed 2.28m/min (7.5 ft/min),
- the average F/E rate shall be checked for consistency – 0.9-1.22m/min (3-4 ft/min) based on existing lock behavior (8.4m (27.7') lift for 3-lift locks, 12.7m (41.7') for two lift locks)
- total overall F/E time shall be consistent with the existing locks.

3. Detailed Hydraulic Features Design

Although the computational accuracy of the spreadsheet model was verified against LOCKSIM, further checking against historical prototype measurements was considered desirable. This was done for data collected at the existing lock.

a) Calibration/Verification of Spreadsheet Model

The model was tested against observed F/E times reported in the 1915 Congress for various operational and water level conditions. The equalization times reported for the conditions were (for "normal" water levels):

- Gatun Lake to Lock: 7.5 – 8 min (2 cases – one with a measured F/E curve),
- Pedro Miguel Lock: 7.5 – 8 min (2 Culverts)
  14.0 min to fill with 1 Culvert – Lift from 50.9' to 84.4',
- Intermediate Locks: 6.5 – 7.5 min, and
- Lock to Ocean: 7.5 – 8.5 min.

For these runs, the lifts had to be determined and the proper geometry and loss coefficients had to be fed to the model. First, the elevations spreadsheet used in the features layout design (see **Appendix E**) was used to estimate the lifts for the spreadsheet runs. The normal, average water levels for Gatun Lake and the Atlantic/Pacific Oceans were input into the elevations spreadsheet as well as the lock dimensions to determine the expected lifts.

The measured friction factor, f = 0.0128 from 1915 Congress was used. The opening/closing time was set to 1 minute based on the existing valves. Head loss coefficients (K) were tabulated based on the different operations and associated geometries (see **Table V.1**).

Table V.1 – Total Loss Coefficients for Calibration/Verification Model Runs

Operation	K
Gatun Lake to Lock	3.42
Pedro Miguel Lock – 1 Culvert	1.98
Pedro Miguel Lock – 2 Culvert	3.42
Intermediate Locks	5.88
Lock to Ocean	3.62

Using these loss coefficients and the calculated lifts, the spreadsheet model was run with equalization times recorded and compared to the measured data. Percent differences were also computed and the results can be seen in **Table V.2**.

Table V.2 – Measured vs. Modeled Equalization Times

Operation	Measured Equalization Time (min)	Modeled Equalization Time (min)	% Diff
Gatun Lake to Lock	7.5-8.0 avg=7.75	7.7	0.6%
Pedro Miguel Lock – 1 Culvert	14.0	13.9	0.7%
Pedro Miguel Lock – 2 Culvert	7.5-8.0 avg=7.75	8.0	3.2%
Intermediate Locks	6.5-7.5 avg=7.0	7.1	1.4%
Lock to Ocean	7.5-8.5 avg=8.0	7.8	2.5%

The results in the previous table, with an overall percent difference of 1.7% from measured to modeled data, demonstrates the adequacy of the spreadsheet model and assigned loss coefficients. As an additional check, a measured equalization curve provided in the 1915 Congress was compared with model solutions (see **Figure V.3**).

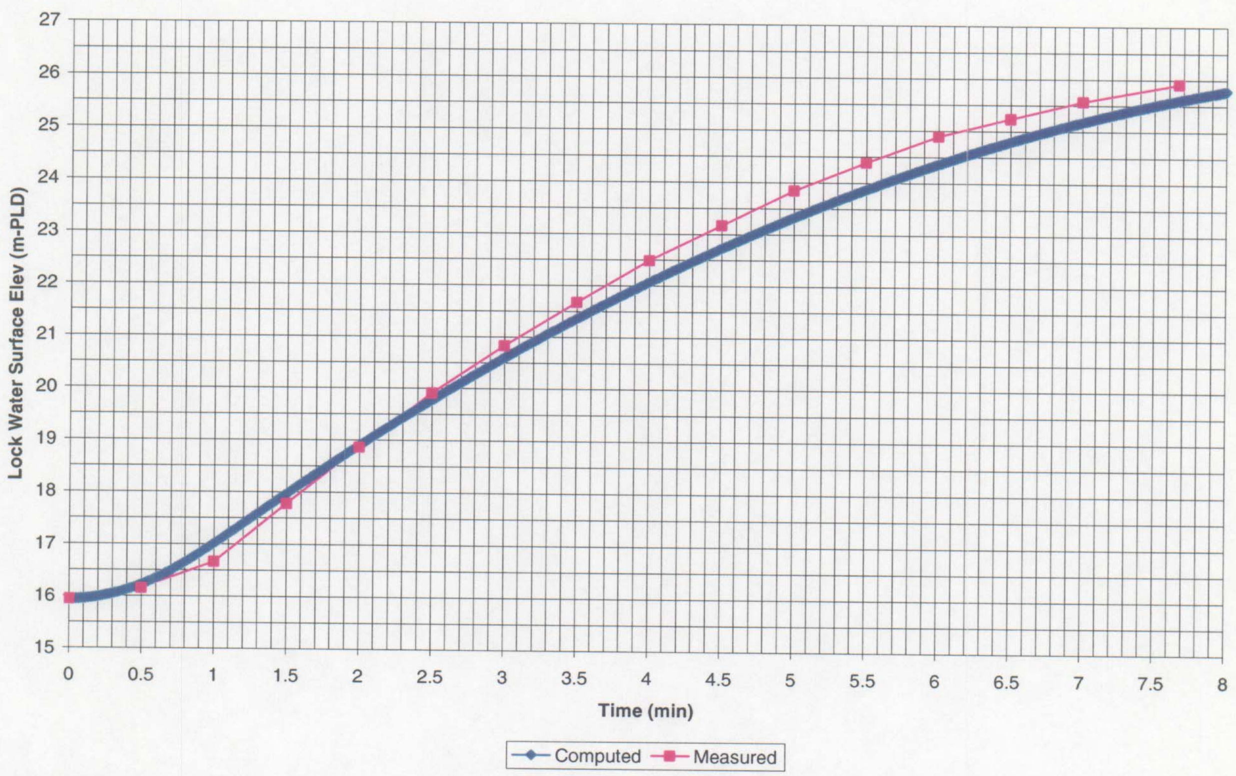


Figure V.3 – Comparison of Measured vs. Modeled Filling Curve for Gatun Lock

The predicted and measured values agree to within 1.4%, an acceptable outcome.

b) Preliminary Design of Lock F/E System

The verified spreadsheet model was used to determine preliminary sizes of lock filling/emptying culverts. Although not part of the original scope of work, this was deemed necessary for two reasons:

- the water saving basin (WSB) conduits should be no larger than lock F/E culverts (the sizes of the lock F/E culverts were needed), and
- to model more accurately the portion of the lock F/E system between the lock chamber and the entrance to the WSB conduits.

To complete a preliminary design of the lock F/E system, an assumption was needed about the type of F/E system that might be used (side-port, bottom lateral, bottom longitudinal, etc.). Based on information in the literature, the required lifts, and recent practice runs on larger locks, a bottom longitudinal manifold system was indicated at this stage.

To determine the range of expected lifts, the elevation spreadsheets used in the features layout design (see **Appendix E**) was revisited and the maximum, mean, and minimum lift heights *for the lock chambers* were recorded. The *lock* lift heights would be the same for both Option 4 arrangements, so no distinction is made in the results given in **Table V.3**.

Table V.3 – Range of Lock Lifts for Each Alternative

OPTION	LIFT HEIGHTS (m)			OVERALL RESULTS
	MAX	MEAN	MIN	
Opt. 1&3 - ATL	10.36 m (33.99')	8.49 m (27.86')	6.75 m (22.15')	3 Lift Option
Opt. 1&3 - PAC	11.52 m (37.81')	8.47 m (27.78')	5.88 m (19.28')	Overall Range = **(5.88m (19.28')-11.52m (37.81'))**
Opt. 2&4 - ATL	15.00 m (49.23')	12.73 m (41.78')	10.48 m (34.37')	2 Lift Option
Opt. 2&4 - PAC	16.69 m (54.77')	12.70 m (41.67')	9.12 m (29.91')	Overall Range = **(9.12m (29.91')-16.69m (54.77'))**

Based on standard design references, these values fall within the intermediate to high lift lock range. Given the size of the proposed locks and the compelling need for good distribution flow and energy dissipation a <u>bottom longitudinal manifold F/E system</u> was assumed. However, even if this type of system is not ultimately chosen, the loss coefficient, hydraulic behavior, and overall performance should be similar to what has been envisioned in this study.

An assumption was also needed about the kind of valve (reverse tainter, normal tainter, vertical lift) that would be used for the lock F/E system. A reverse tainter valve was assumed for this system due to its predominance in recent decades on large, high lift locks. Reverse tainter valves have fewer problems with downstream cavitation and generate neither excessive air demands or large slugs or impulses of entrained air that can create turbulence in the lock chamber. It was also assumed that the valves would have an opening/closing time equivalent to the existing valves (1 minute). This opening/closing time is simply the target time used at the conceptual level and was selected in consideration of maximizing the system efficiency. This 1 minute operating time can likely be achieved if the valves are bifurcated. Even if the culvert sizes were too large to be served by a single valve that could be operated in 1 minute, it would be preferable to employ a manifold with multiple smaller valves that could be operated quickly. This would also provide system redundancy.

To determine preliminary sizes for the lock F/E culverts, multiple model runs (for varying initial heads) were needed to create parametric curves of initial head vs. equalization time and initial head vs. instantaneous maximum F/E rate. The explicit constraints were:

- the instantaneous maximum F/E rate should not exceed 2.28m/min (7.5 ft/min) (the maximum for the existing locks with two culvert operations), and
- F/E times for a 3-lift system should be 8 – 9 min per lift (based on the existing system) and for a two-lift system, (3 lift x 8 – 9 = 24-27 min total)/2 lift = 12 – 13.5 min/lift.

The estimated F/E time for a 2-lift system is based on the assumption that the overall F/E time for a 3-lift and 2-lift system should be equivalent. Therefore, the target F/E time for a 2-lift system was determined by multiplying the F/E time per lift for a 3lift system by a factor of 3/2. Again, this factor is used to calculate a target F/E time to be used for preliminary sizing of F/E culverts and serving only to set the boundaries for acceptable and unacceptable solutions determined using the spreadsheet model.

Separate sets of model runs would be required for each of the three types of lock operations (with varying loss coefficients – see **Table V.4**)

Table V.4 – Head Loss Coefficients for Lock F/E System Model Runs

Operation	K
Gatun Lake to Lock	2.56
Lock to Lock	5.17
Lock to Ocean	4.17

The elevations spreadsheets in **Appendix E** were used to derive the range of initial heads for each option given varying water levels and lockage lengths:

The range of initial heads for the various options are shown in **Table V.5**.

Table V.5 – Range of Lock Initial Heads for Each Option and Operation Type

Metric Units

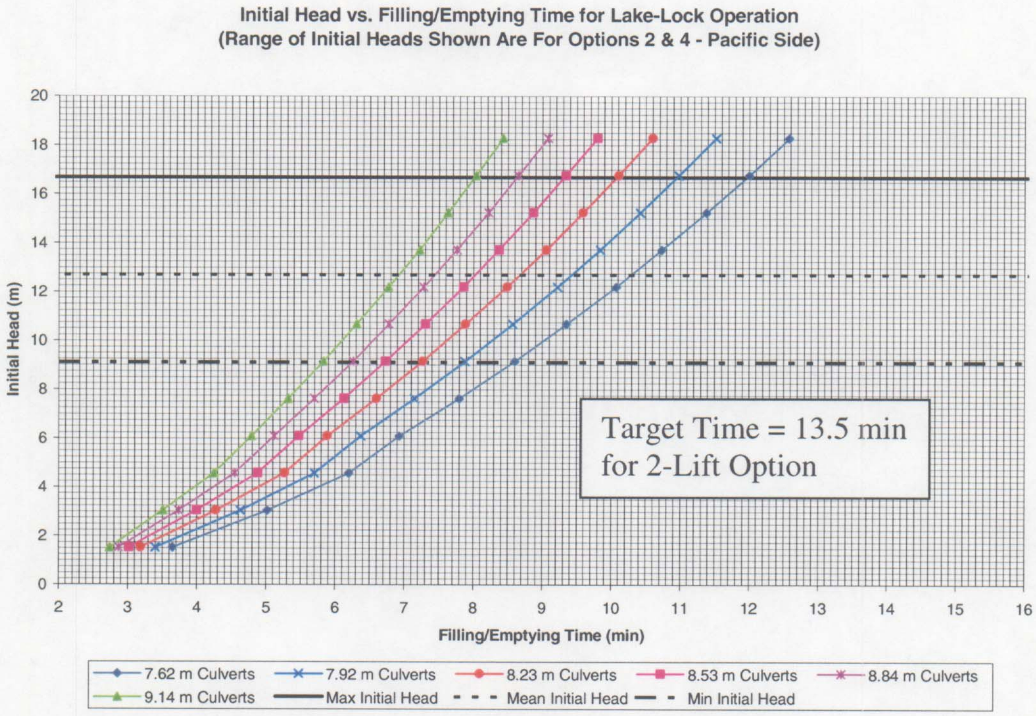
OPTION	INITIAL HEAD BY OPERATION TYPE (M)								
	LAKE-LOCK			LOCK-LOCK			LOCK-OCEAN		
	MAX	MIN	MEAN	MAX	MIN	MEAN	MAX	MIN	MEAN
Options 1 &3 ATL	10.04	7.01	8.49	19.04	14.74	16.98	10.04	7.01	8.49
Options 1 & 3 PAC	11.17	6.10	8.46	21.19	12.83	16.93	11.17	6.10	8.46
Options 2 & 4 ATL	15.00	10.48	12.73	27.20	23.36	25.47	15.00	10.48	12.73
Options 2 & 4 PAC	16.69	9.12	12.70	30.27	20.33	25.40	16.69	9.12	12.70
OVERALL RANGES	**6.10 – 16.69**			**12.83-30.27**			**6.10 – 16.69**		

English Units

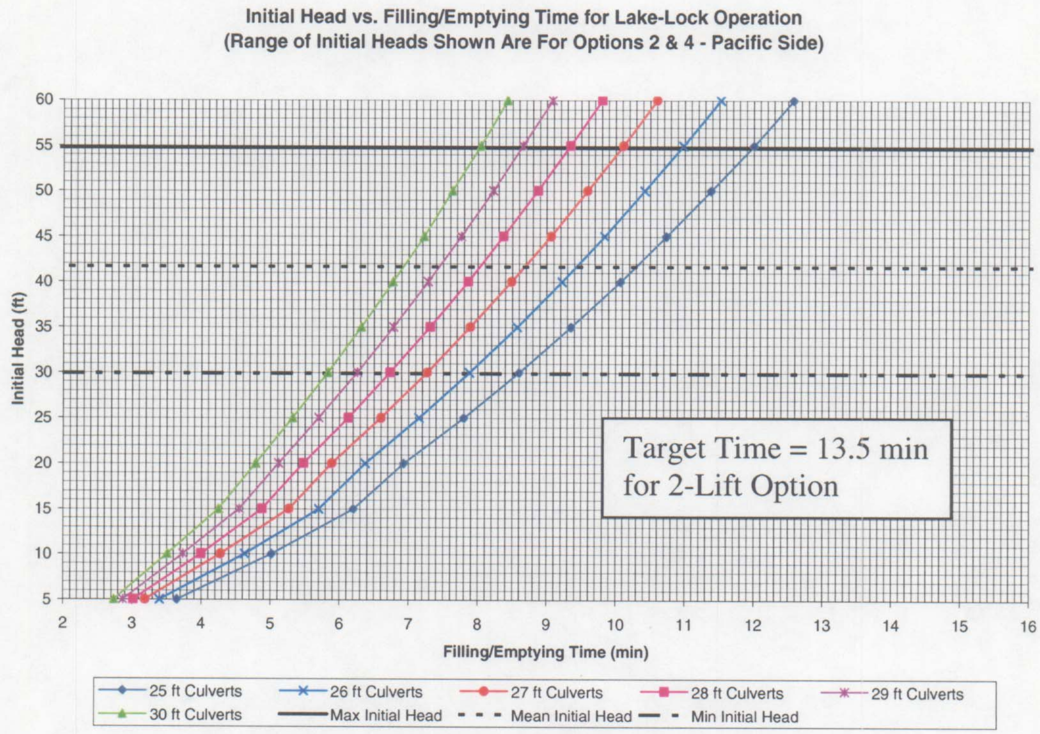
OPTION	INITIAL HEAD BY OPERATION TYPE (FT)								
	LAKE-LOCK			LOCK-LOCK			LOCK-OCEAN		
	MAX	MIN	MEAN	MAX	MIN	MEAN	MAX	MIN	MEAN
Options 1 &3 ATL	32.93	23.00	27.85	62.47	48.37	55.72	32.93	23.00	27.85
Options 1 & 3 PAC	36.64	20.01	27.77	69.51	42.09	55.56	36.64	20.01	27.77
Options 2 & 4 ATL	49.23	34.39	41.78	89.25	76.65	83.57	49.23	34.37	41.78
Options 2 & 4 PAC	54.77	29.91	41.67	99.30	66.70	83.33	54.77	29.91	41.67
OVERALL RANGES	**20.01 – 54.77**			**42.09-99.30**			**20.01 – 54.77**		

For the Lake to Lock operation, 12 model runs were completed for each test culvert diameter (initial heads from 1.52 m (5') to 18.29 m (60') @ 1.52 m (5') intervals). The Lock to Lock operation required 18 model runs for each diameter (initial heads from 6.10 m (20') to 32.00 m (105') @ 1.52 m (5') intervals). Finally, the Lock to Ocean operation required the same model runs (with respect to initial head differences) as the Lake to Lock operation.

Parametric curves were constructed from the spreadsheet model results for F/E equalization time and the instantaneous maximum F/E rate for each culvert size. The expected initial head ranges for each option were also plotted on these graphs. The following graphs show the parametric curves created for the 2-Lift options (Options 2 and 4) for the Pacific side. The complete set of curves for all options and ocean sides can be found in **Appendix G**.

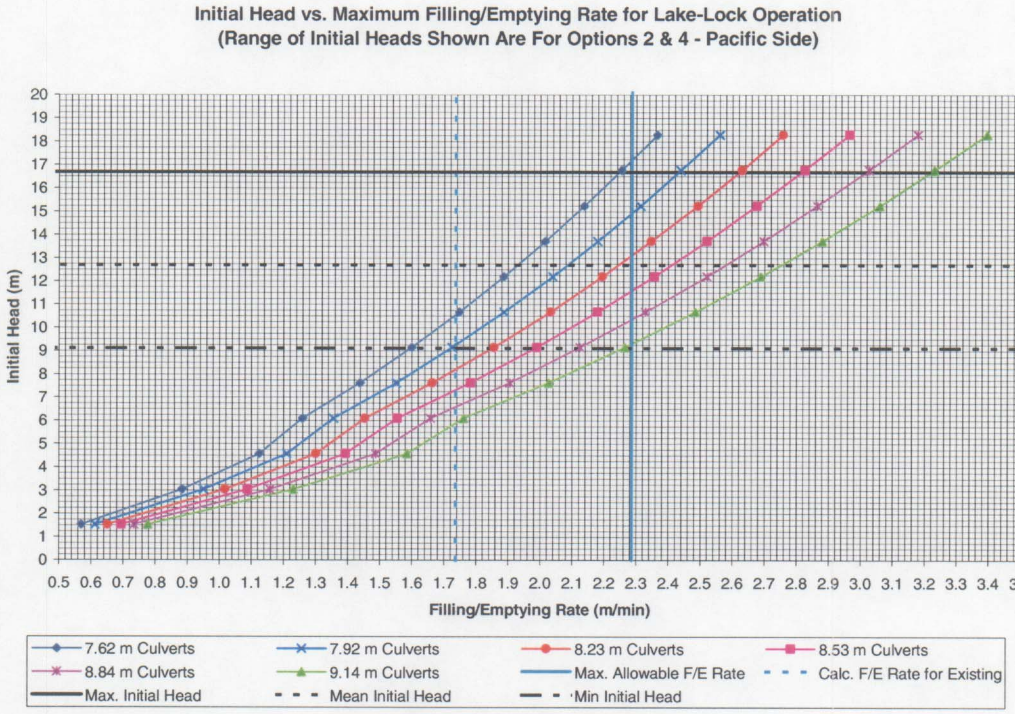


Metric Units

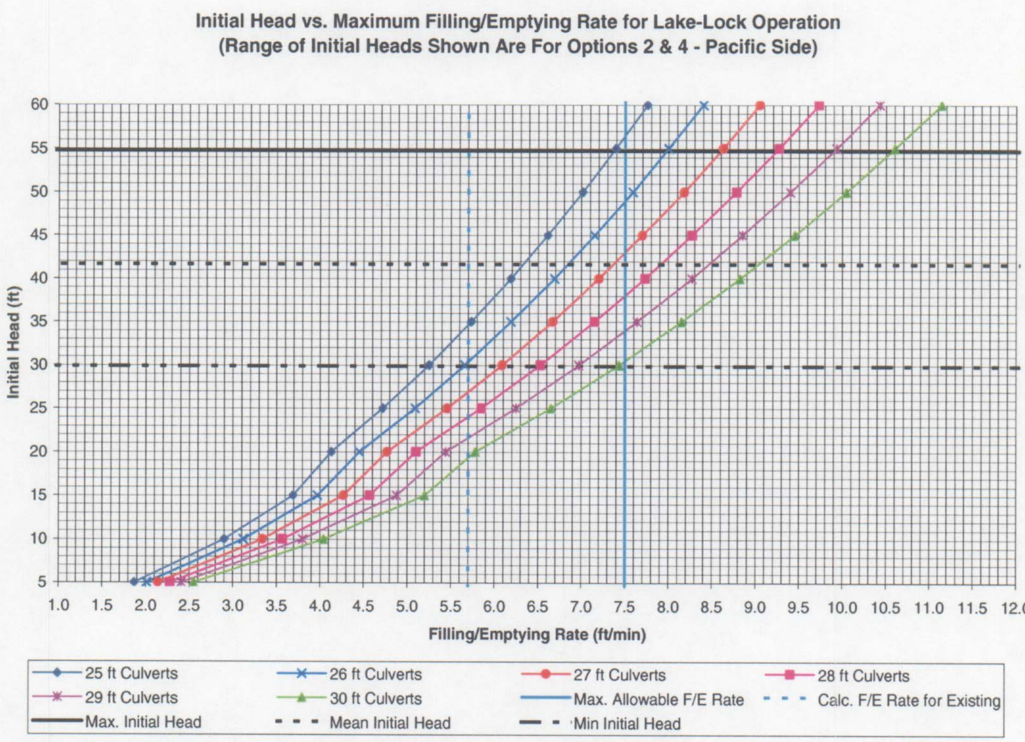


English UNITS

Figure V.4 – Initial Head vs. F/E Equalization Time for Lake to Lock Operation

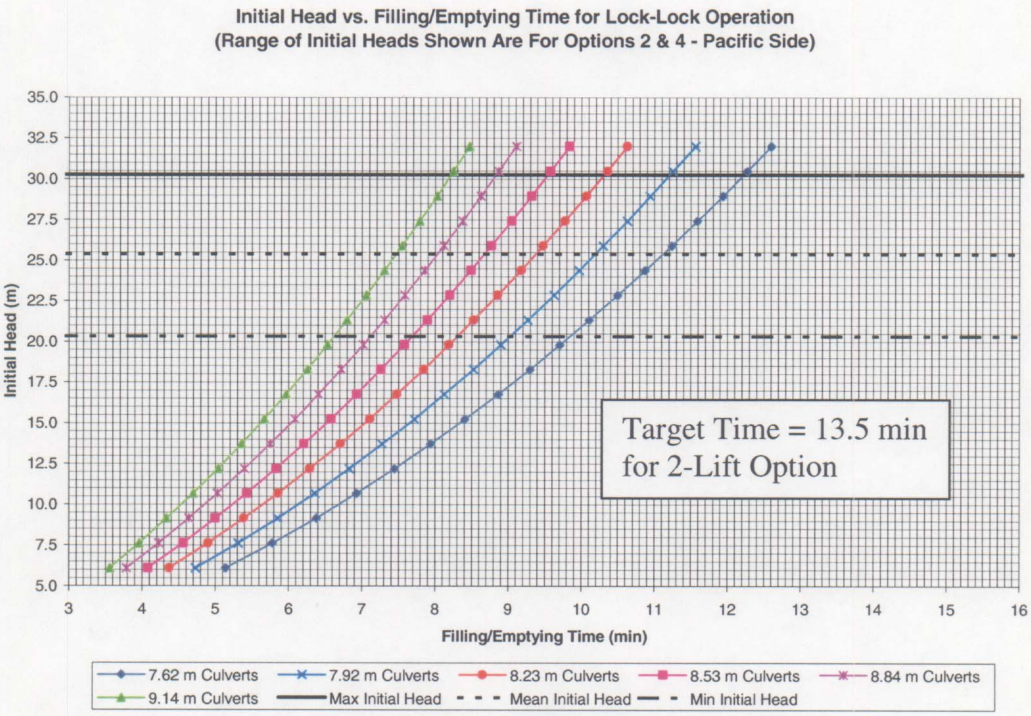


Metric Units

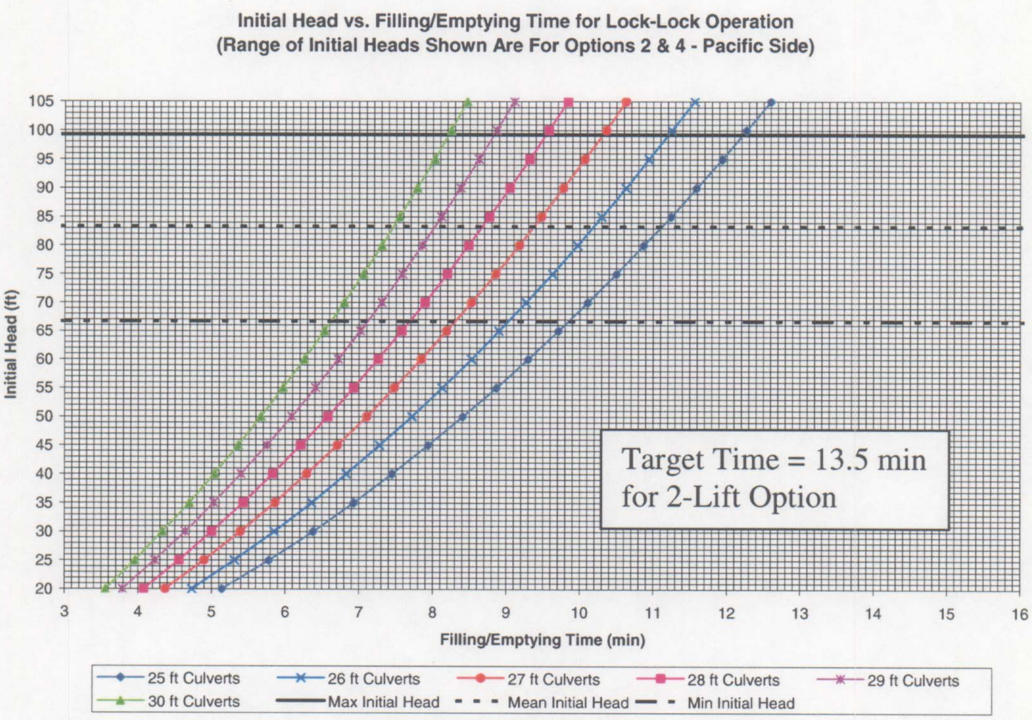


English Units

Figure V.5 – Initial Head vs. Max. Instantaneous F/E Rate for Lake to Lock Operation

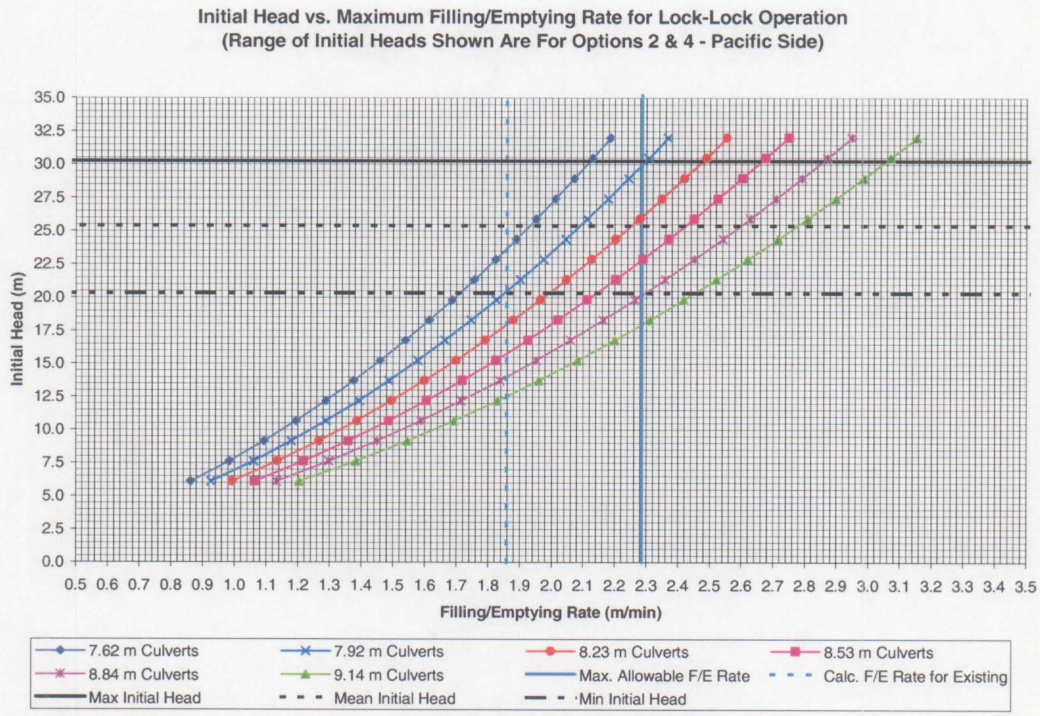


Metric Units

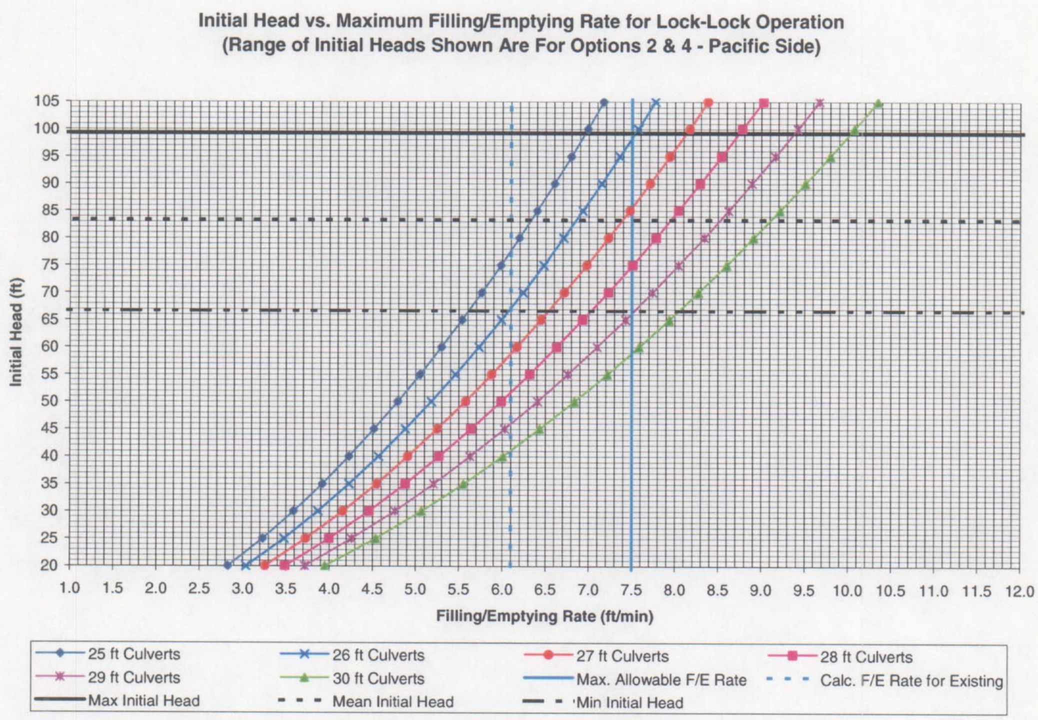


English Units

Figure V.6 – Initial Head vs. F/E Equalization Time for Lock to Lock Operation

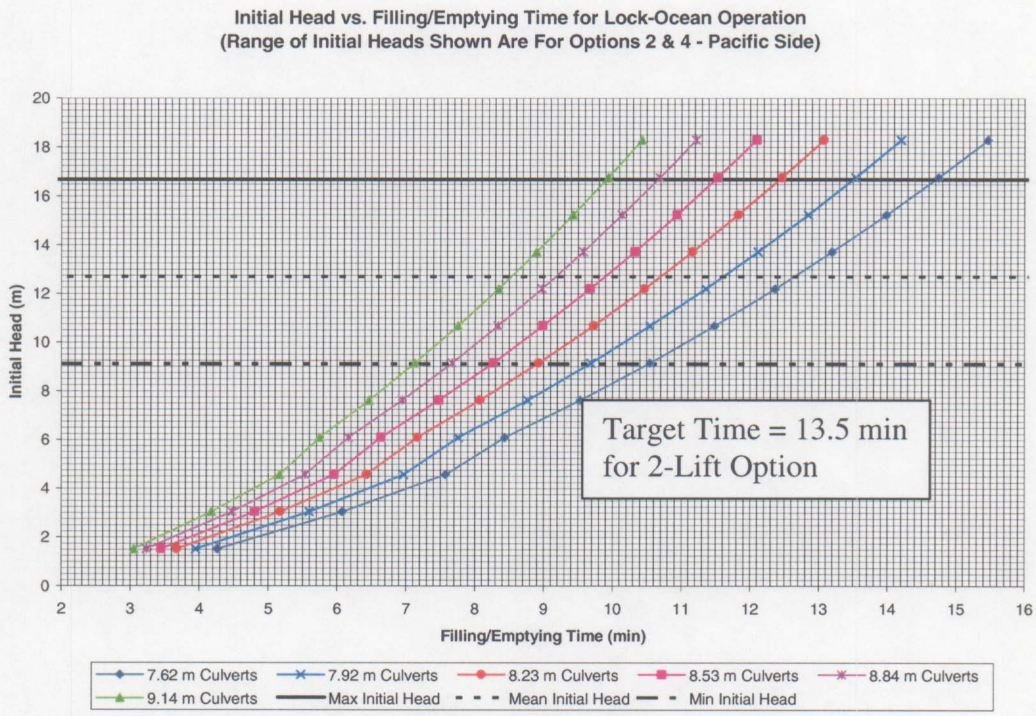


Metric Units

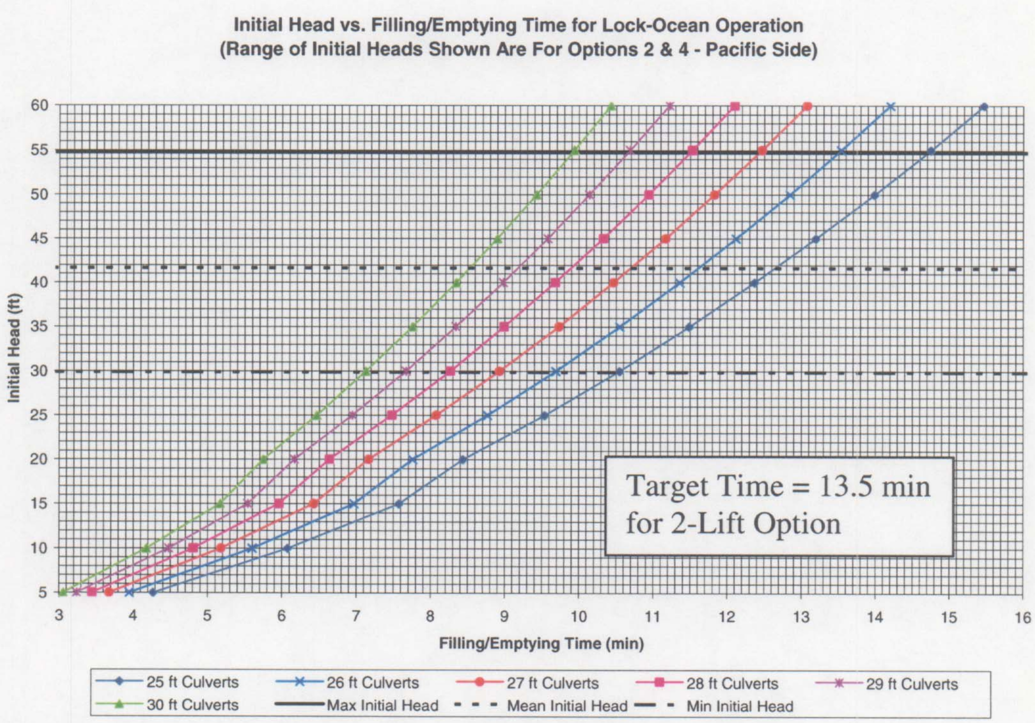


English Units

Figure V.7 – Initial Head vs. Max. Instantaneous F/E Rate for Lock to Lock Operation

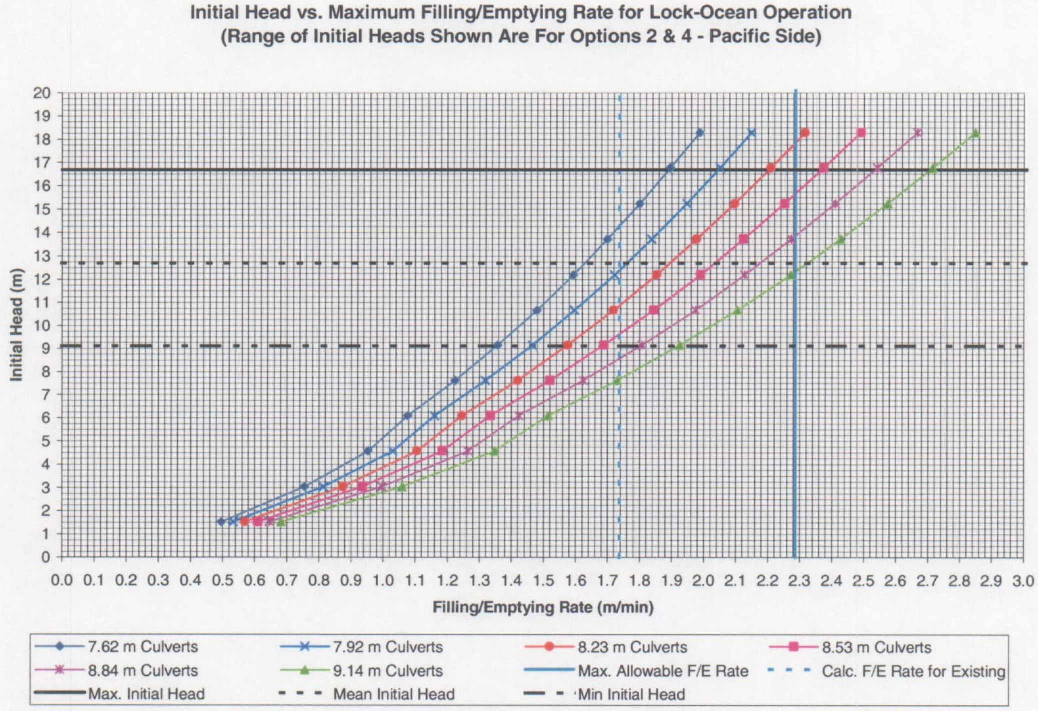


Metric Units

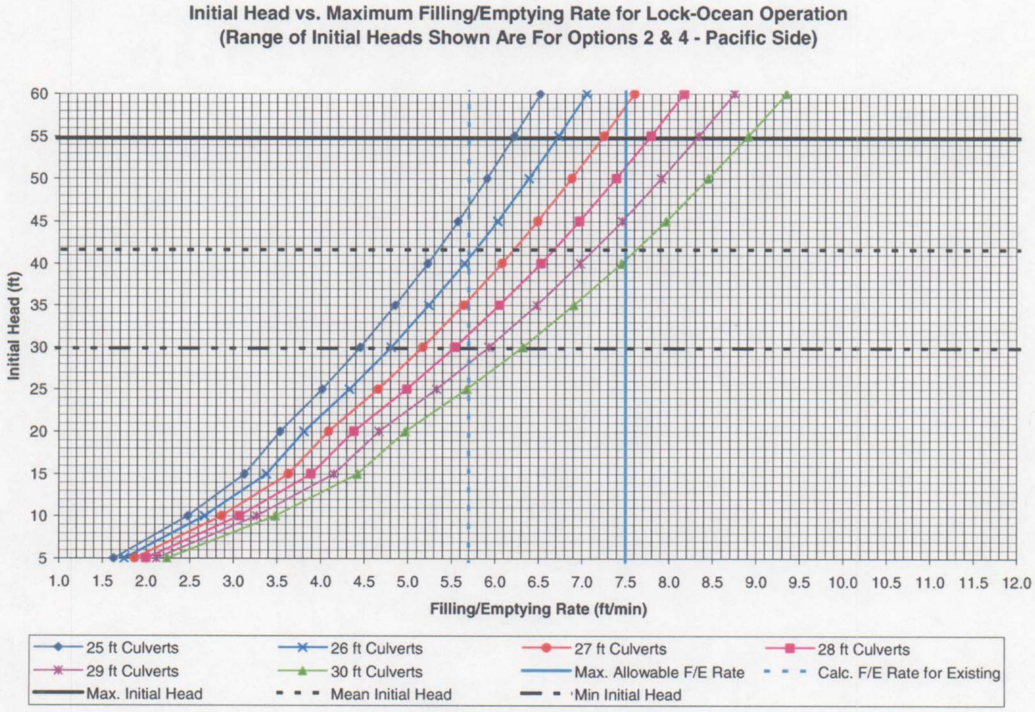


English Units

Figure V.8 – Initial Head vs. F/E Equalization Time for Lock to Ocean Operation



Metric Units



English Units

Figure V.9 – Initial Head vs. Max. Instantaneous F/E Rate for Lock to Ocean Operation

Preliminary sizes for the lock F/E culverts were determined from these curves and the operational criteria already described. This was done for each option by first selecting the largest culvert diameter that would not violate the instantaneous maximum of 2.28m/min (7.5 ft/min) for the highest initial head experienced. For the time criterion, the smallest culvert diameter (under the highest initial head conditions) with F/E equalization times exceeding 9 min. (3-Lift) or 13.5 min (2-Lift) was chosen. This was done for each type of operation.

For example, from the preceding parametric curve plots, for Options 2 & 4 on the Pacific side, with a lake to lock operation, and for a F/E time constraint of 13.5 minutes against the maximum initial head, a minimum culvert diameter of about 7.31 m (24' – extrapolated) is needed. However, when the max F/E rate criterion is checked for the largest initial head experienced, the culvert diameter should be no greater than about 7.62 m (25'). With a lock to lock operation, the culvert should be at least 7.31 m (24' –extrapolated) to meet the F/E time requirement, but no greater than 7.62 m (25') to limit the maximum rate of rise to 7.5 ft/min. Finally, for a lock to ocean operation, the culvert should be at least 7.92 m (26') for F/E time, but no more than 8.23 m (27') for maximum rate of rise.

To determine the final culvert sizes for each option, results would be compared for each operation type. For the maximum instantaneous F/E rate, one would want the smallest D to control so that this F/E rate would never be greater than 7.5 ft/min in any operation (Lake-Lock, Lock-Lock, Lock-Ocean). Also, when looking at F/E times, one would choose the largest D so that in any operation the time would never be greater than the 9 (3-lock) or 13.5 (2-lock) min. limit. Therefore, for the example above, a culvert diameter of 7.92 m (26') provides an acceptable F/E time while a culvert diameter no greater than 7.62 m (25') is necessary not to exceed the maximum allowable F/E rate in the lock chamber. **Table V.6** shows the required sizes for each criteria check.

Table V.6 – Comparative Matrix for Preliminary Sizing of Lock F/E Culverts

Metric Units

OPTION	PRELIMINARY CULVERT SIZE (M)						RESULTS	
	OPERATION TYPE							
	LAKE – LOCK		LOCK – LOCK		LOCK – OCEAN			
	RATE	TIME	RATE	TIME	RATE	TIME	**RATE**	**TIME**
Options 1 & 3 - ATL	8.84	7.62	8.84	7.92	>9.14	8.84	***8.84***	**8.84**
Options 1 & 3 - PAC	8.53	7.92	8.53	8.23	>9.14	8.84	***8.53***	**8.84**
Options 2 & 4 - ATL	7.92	7.31	7.92	7.31	8.53	7.92	***7.92***	**7.92**
Options 2 & 4 - PAC	7.62	7.31	7.62	7.31	8.23	7.92	***7.62***	**7.92**

English Units

OPTION	PRELIMINARY CULVERT SIZE (FT)						RESULTS	
	OPERATION TYPE							
	LAKE – LOCK		LOCK – LOCK		LOCK – OCEAN			
	RATE	TIME	RATE	TIME	RATE	TIME	**RATE**	**TIME**
Options 1 & 3 - ATL	29’	25’	29’	26’	>30’	29’	***29’***	**29’**
Options 1 & 3 - PAC	28’	26’	28’	27’	>30’	29’	***28’***	**29’**
Options 2 & 4 - ATL	26’	24’	26’	24’	28’	26’	***26’***	**26’**
Options 2 & 4 - PAC	25’	24’	25’	24’	27’	26’	***25’***	**26’**

Once the results had been tabulated for rate and time, it was decided to let rate control since safety considerations are paramount and ACP personnel had expressed desires that ship handling should be as close to the existing system as possible. Therefore, in our example for the 2-Lift options on the Pacific side (Options 2 and 4), the finalized culvert diameter would be 7.62 m (25’).

The finalized lock F/E culvert diameters were:

- Options 1 & 3 – Atlantic Side (8.84m - 29’),
- Options 1 & 3 – Pacific Side (8.53m - 28’),
- Options 2 & 4 – Atlantic Side (7.92m - 26’), and
- Options 2 & 4 – Pacific Side (7.62m - 25’).

c) Preliminary Design of Water Saving Basin Conduits

To complete a preliminary design of the WSB conduits, an evaluation was needed of the type of valve to be used (reverse tainter, normal tainter, vertical lift).

(1) Selection of Valve Type and Operation Times

Since the flows would be bi-directional (basin to lock, lock to basin), vertical lift valves offer certain advantages. In situations where flows are bi-directional, guidance from Davis and the Corps of Engineers recommended that vertical lift valves would have the least problems with unequal flow and cavitation patterns (since flow conditions through the valves are similar regardless of the direction). Vertical lift valves would also require smaller (thinner) gate recesses that would allow the basins to be located closer to the locks, potentially saving money on excavation costs. Being smaller and lighter, the valves would be easier to remove and maintain. The Canal Authority already has decades of experience servicing similar valves on the existing locks. Finally, vertical lift valves also have the advantage of a more linear opening/closing pattern which would likely lead to quicker equalization times between the lock and basin and vice-versa (see **Figure V.10**). Tainter valve opening/closing patterns often exhibit a substantial "sag" which results in a concave upward opening pattern as shown on the figure.

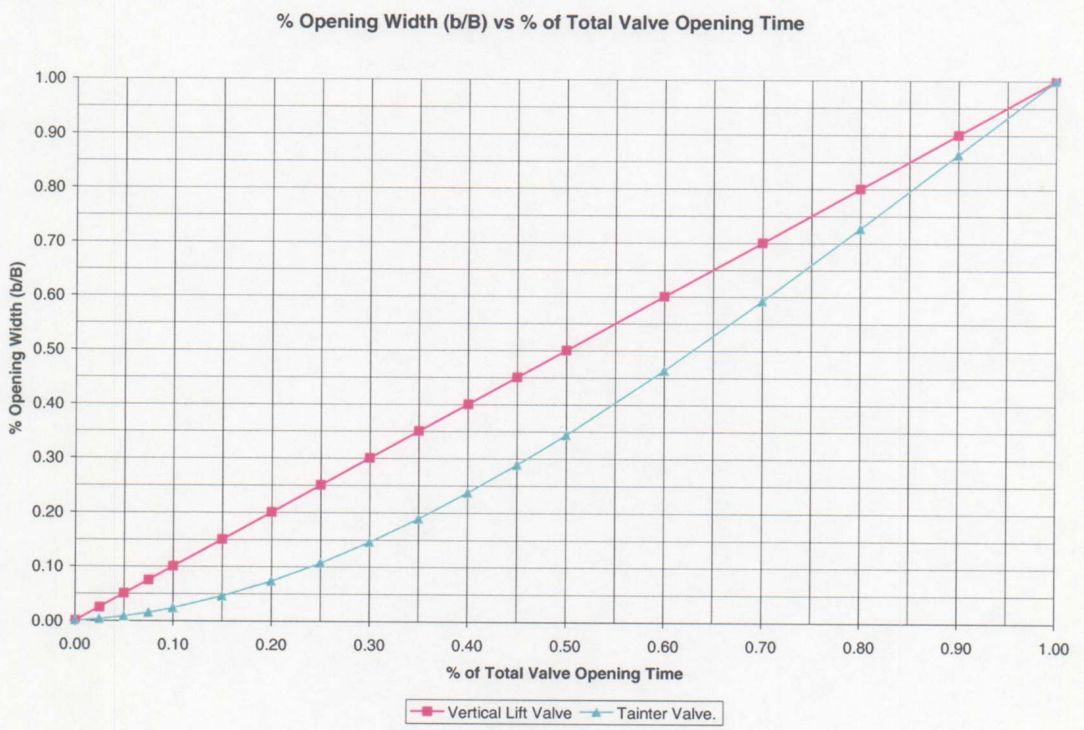


Figure V.10 – Comparison of Valve Opening Width vs. Time Curves

Despite these apparent advantages, it was necessary to complete a comparative study of the operational characteristics of vertical lift and tainter valves. For this analysis, spreadsheet models with identical inputs were run (initial head = 28', D = 24', Opening/Closing Time = 60 sec). These same parameters were used in comparative model runs for 2-, 4- ,6- , and 8 conduits/basin. The results for these runs can be seen in **Table V.7**.

Table V.7 – Comparison of Valve Type Effects on Model Results

Condition	Max Fill Rate (m/min)	Max Fill Rate (ft/min)	Avg. Fill Rate (m/min)	Avg. Fill Rate (ft/min)	Equalization Time (min)
2-conduit-vertical valve	1.62	5.32	0.93	3.05	3.98
2-conduit-reverse tainter valve	1.62	5.31	0.91	2.99	4.08
2-conduit-normal tainter valve	1.61	5.28	0.91	2.99	4.08
4-conduit-vertical valve	2.11	6.94	1.20	3.94	2.95
4-conduit-reverse tainter valve	2.11	6.92	1.17	3.84	3.06
4-conduit-normal tainter valve	2.10	6.88	1.17	3.84	3.05
6-conduit-vertical valve	2.17	7.13	1.34	4.41	2.94
6-conduit-reverse tainter valve	2.17	7.11	1.30	4.27	2.98
6-conduit-normal tainter valve	2.15	7.07	1.31	4.29	2.98
8-conduit-vertical valve	2.09	6.85	1.29	4.22	3.06
8-conduit-reverse tainter valve	2.09	6.84	1.25	4.09	3.12
8-conduit-normal tainter valve	2.07	6.80	1.25	4.11	3.11

From the table, vertical lift valves do have slightly faster equalization times as was expected, but this difference is not significant enough to warrant selection of lift valves for this reason alone. However, this reason coupled with the other advantages already outlined favored the selection of vertical lift valves for the WSB conduits at the conceptual study phase.

(2) Determination of Loss Coefficients to Use for Model Runs

Head loss coefficients for the water saving basin conduits also had to be determined. Again, **Figure V.1**, served as the basis for assigning loss coefficients. Using the hydraulic design references cited earlier, loss coefficients for the different operations (lock to basin, basin to lock) can be seen in **Table V.8**.

Table V.8 – Head Loss Coefficients for Different WSB Operations

Operation	K
Lock to Basin	4.51
Basin to Lock	3.91

Since the use of parametric curves was going to be employed for this preliminary design and many more model runs were going to be required due to the study of various arrangements (2-, 4-, 6-, and 8 conduits/basin), it was decided that the WSB conduits should be sized using model runs with K = 4.51. Using the higher head loss would be conservative, as it would provide us a larger diameter required to meet the design constraints. However, by using the higher total loss coefficient, the maximum F/E rate would be slightly underpredicted. This was judged to be acceptable for this concept-level study in which basic geometries and costs are the items most in question. Additional requirements can be revisited at later phases when the particular form and geometry of the lock F/E system has been identified.

Nonetheless, a comparative study was completed to determine the differences in equalization time and maximum instantaneous F/E rate between the two K factors. The results can be seen in **Table V.9**. It should be noted that the comparative model runs were made for the highest initial head case and large diameter conduits. As can be seen from the table, the equalization times and F/E/rates are nearly identical. In two of the test cases, the maximum F/E rate did slightly exceed the criteria of 7.5 ft/min, for 6- and 8. However, it is unlikely that these configurations would be economically feasible. Therefore, the parametric curves using model runs using K=4.51 were judged to be acceptable.

Table V.9 – Comparison of Head Loss Coefficients Effects on Model Results

CONDITION	Init. Head (ft)	Diam (ft)	Max Fill Rate (m/min)	Max Fill Rate (ft/min)	Avg. Fill Rate (m/min)	Avg. Fill Rate (ft/min)	Equalization Time (min)
2-conduit existing - K = 4.51	28	29	1.62	5.32	0.93	3.05	3.98
2-conduit-smooth - K = 3.91	28	29	1.81	5.93	0.97	3.18	3.88
4-conduit existing - K = 4.51	28	22	1.81	5.95	1.02	3.35	3.51
4-conduit smooth - K = 3.91	28	22	2.03	6.67	1.06	3.49	3.40
6-conduit existing - K = 4.51	28	20	2.17	7.13	1.34	4.41	2.94
6-conduit smooth - K = 3.91	28	20	2.44	7.99	1.36	4.46	2.88
8-conduit existing - K = 4.51	28	17	2.09	6.85	1.29	4.22	3.06
8-conduit smooth - K = 3.91	28	17	2.35	7.71	1.32	4.34	3.06

(3) Determination of Range of Initial Heads to Use for Model Runs

The next step was to determine the range of initial heads for which the spreadsheet models should be run to create the parametric curves. To do this, the elevations spreadsheets used in the features layout (see **Appendix E**) were revisited. As part of the spreadsheet calculations, a "step height" was calculated by the following equation:

$$\mathrm{STEP} = \frac{\mathrm{LIFT}}{n+2}, \text{ where n = number of basins.}$$

Figure IV.4 is repeated here for illustration as **Figure V.11**.

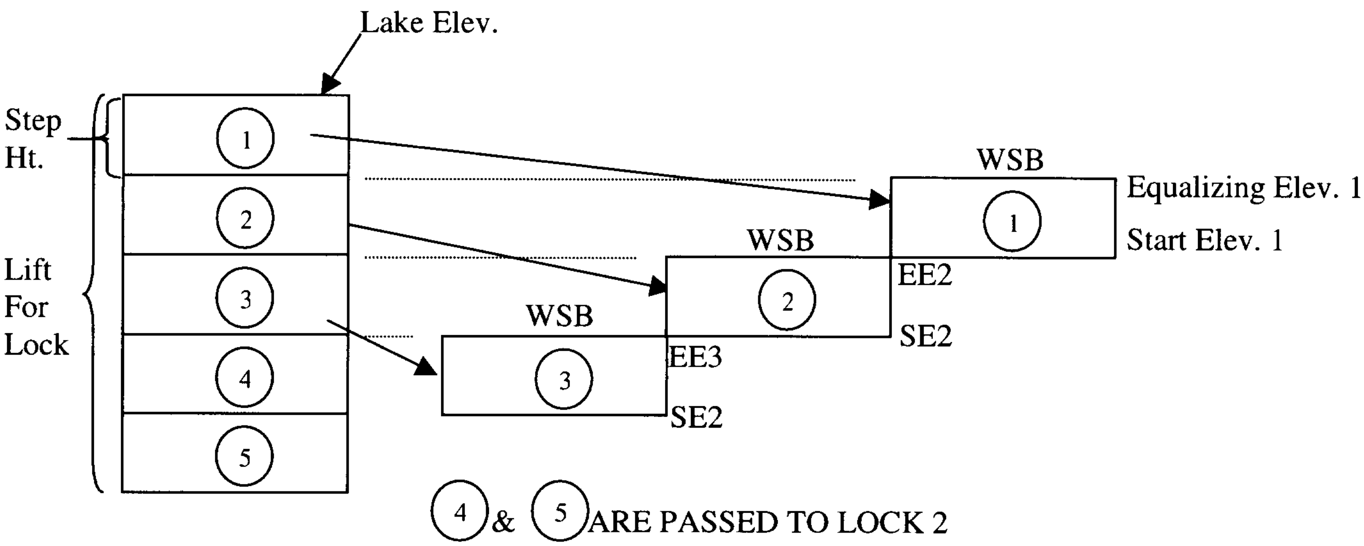


Figure V.11 – Conceptual Section Showing Calculation of WSB Operating Elevations

From **Figure V.11**, the initial head for the water saving basins would be 2 times the "step height". For example, in draining the first block in a lock emptying procedure, the lock is at the lake level while the first basin is completely empty. This initial head difference is equivalent to 2 times the step height. Using the elevation spreadsheet, the range of step heights and initial heads could be calculated. The results can be seen in **Table V.10**.

Table V.10 – Range of Lock Initial Heads for Each Option and Operation Type

Metric Units

OPTION	STEP HEIGHT (m)			STEP HEIGHT *2 = INITIAL HEAD	INITIAL HEADS (m)		
	MAX	MIN	MEAN		MAX	MIN	MEAN
Option 1 - ATL	2.59	1.69	2.12		5.18	3.38	4.25
Option 1 - PAC	2.88	1.47	2.12		5.76	2.94	4.24
Option 2 - ATL	3.00	2.09	2.55		6.00	4.19	5.10
Option 2 - PAC	3.44	1.82	2.54		6.68	3.65	5.08
Option 3 - ATL	2.07	1.35	1.70		4.15	2.70	3.40
Option 3 - PAC	2.30	1.18	1.69		4.61	2.35	3.39
Option 4 - ATL	3.75	2.62	3.19		7.50	5.24	6.37
Option 4 - PAC	4.17	2.28	3.18		8.35	4.56	6.35
OVERALL RANGES	**1.18 – 4.17**				**2.35 – 8.35**		

English Units

OPTION	STEP HEIGHT (FT)			STEP HEIGHT *2 = INITIAL HEAD	INITIAL HEADS (FT)		
	MAX	MIN	MEAN		MAX	MIN	MEAN
Option 1 - ATL	8.50	5.54	6.97		17.00	11.08	13.94
Option 1 - PAC	9.45	4.82	6.95		18.90	9.64	13.90
Option 2 - ATL	9.85	6.87	8.36		19.70	13.74	16.72
Option 2 - PAC	10.95	5.98	8.33		21.90	11.96	16.66
Option 3 - ATL	6.80	4.43	5.57		13.60	8.86	11.14
Option 3 - PAC	7.56	3.86	5.56		15.12	7.72	11.12
Option 4 - ATL	12.31	8.59	10.45		24.62	17.18	20.90
Option 4 - PAC	13.69	7.48	10.42		27.38	14.96	20.84
OVERALL RANGES	**3.86 – 13.69**				**7.72 - 27.38**		

With the results from the previous table, it was decided to make the runs with initial heads ranging from 6' –28' @ 2' intervals for each conduit diameter tested. These model runs would also be made for the conduit arrangements of 2 conduits/basin, 4 conduits/basin, 6 conduits/basin, and 8 conduits/basin.

(4) Formulation and Application of Design Criteria

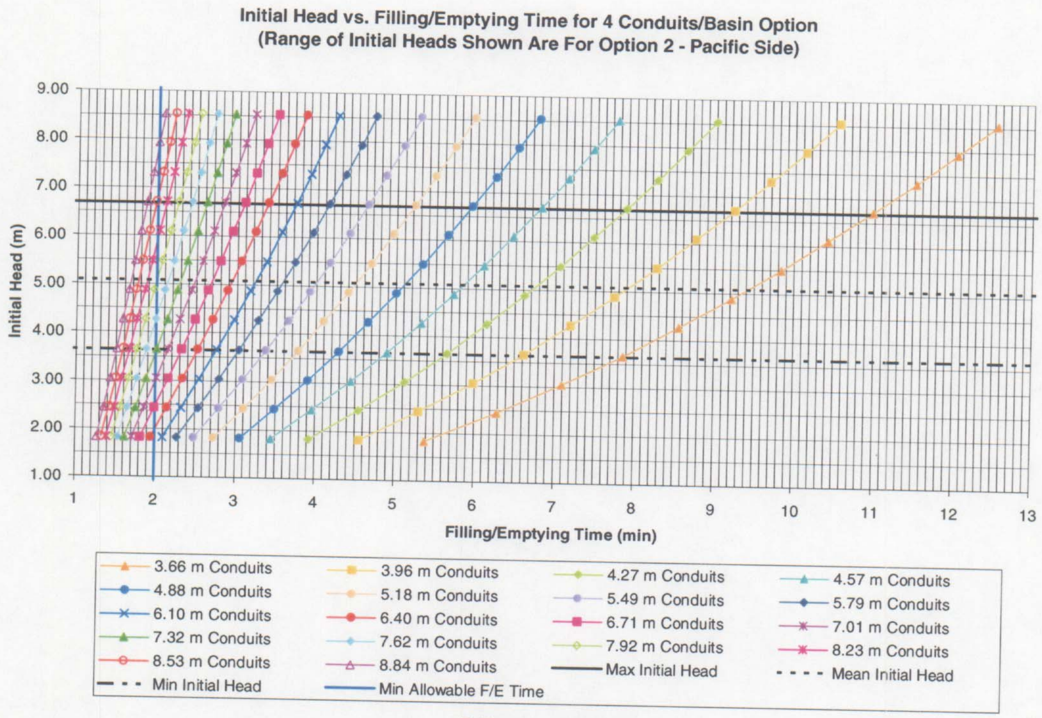
Overall, the analysis procedure for the WSB conduits was the same as for the lock F/E culverts, but with different design criteria. The new criteria were:

- The WSB conduits should not be larger than the preliminary F/E culvert sizes,
- No conduit solution should exceed an instantaneous maximum F/E rate of 7.5 ft/min, and
- No conduit solution should have a single basin operation time of less than 2 minutes (which is the assumed shortest time needed to open and immediately close the valves).

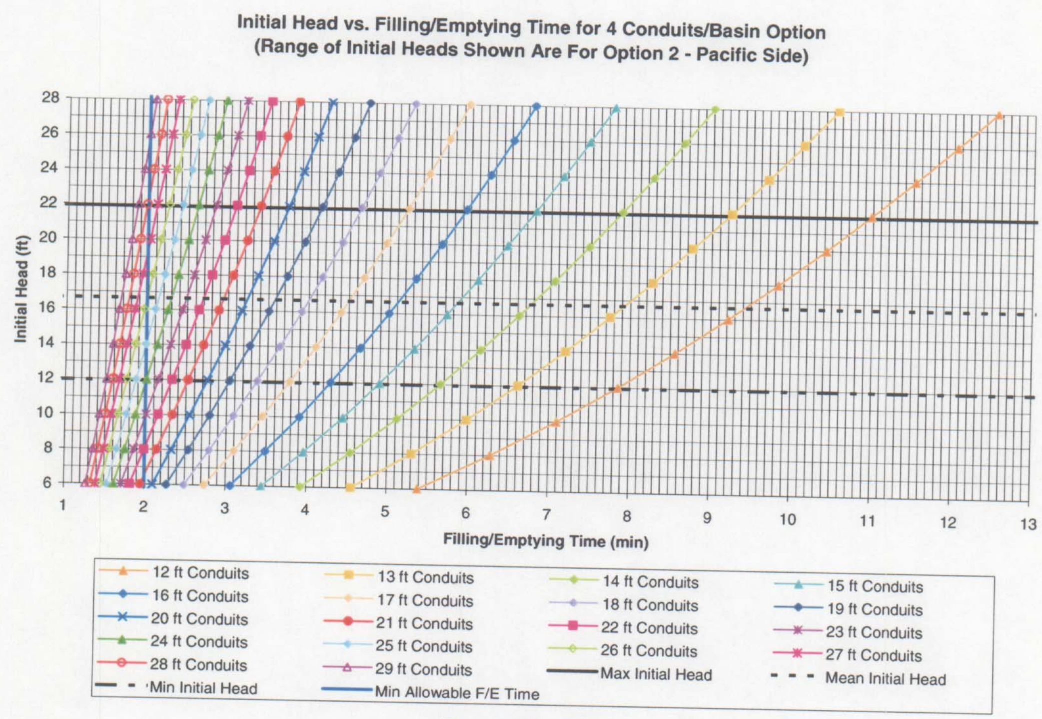
Example parametric curves for the WSB conduits for the 4 conduits/basin arrangement can be seen in **Figures V. 12-13** (see **Appendix H** for parametric curves for all options and conduit arrangements). Based on the above criteria, for the 4 conduits/basin arrangement for Option 2 on the Pacific side, the recommended conduit diameters are:

Criterion 1) no larger than 7.62 m (25'),
Criterion 2) be no larger than 8.23 m (27'), and
Criterion 3) be no larger than 7.32 m (24').

Therefore, the controlling diameter would be 7.32m or 24' for Option 2 on the Pacific Ocean side with 4 conduits/basin.

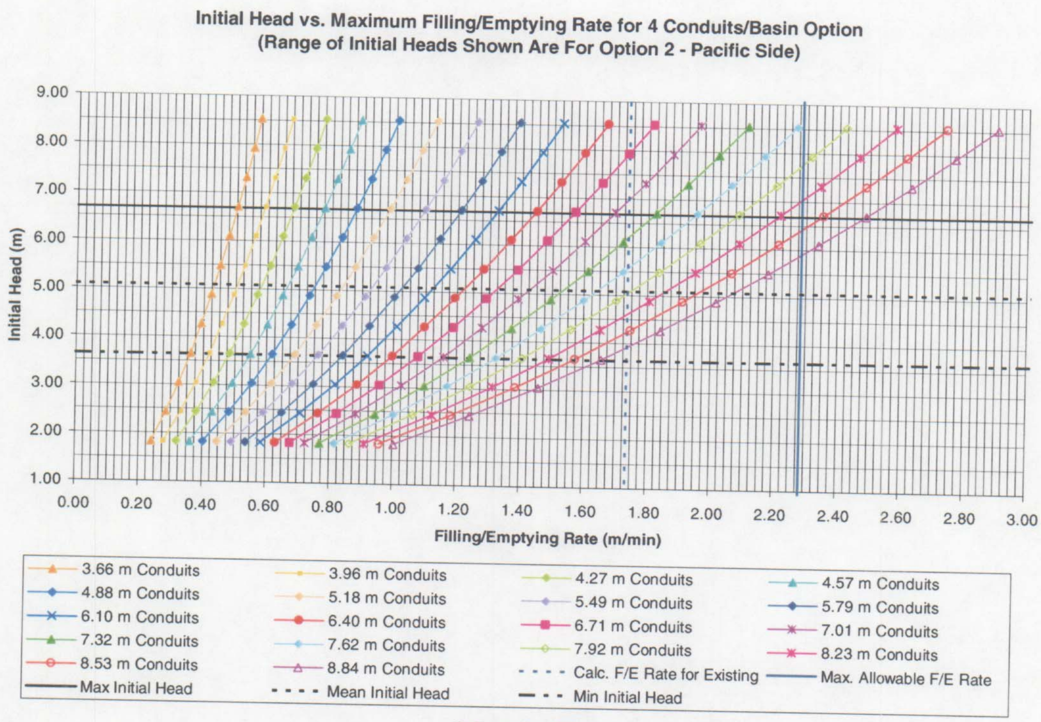


Metric Units

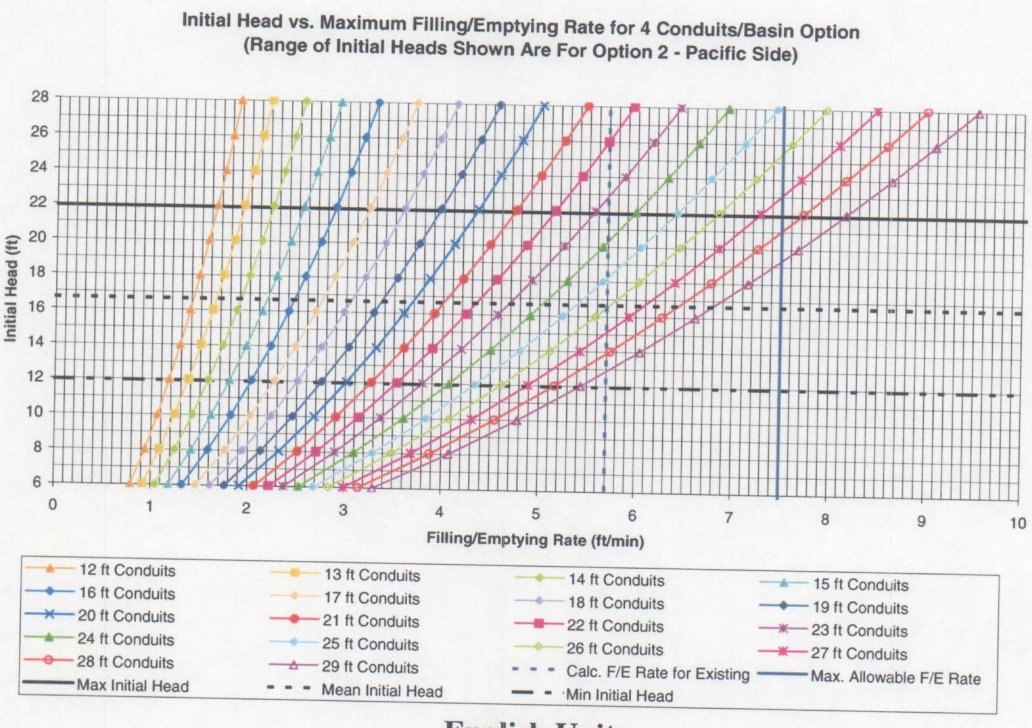


English Units

Figure V.12 – Parametric Curves for Equalization Time (Opt. 2, Pacific, 4 conduits/basin)



Metric Units



English Units

Figure V.13 – Parametric Curves for Max F/E Rate (Opt. 2, Pacific, 4 conduits/basin)

As a final step, it is possible to combine the results from the lock F/E culvert and the WSB conduit analyses to compute more meaningful statistics including total operation time, allowable transits/day, etc. by the following procedure.

Once the viable conduit arrangements had been identified (by applying the three design criteria), best fit curves relating initial heads to F/E time, maximum instantaneous F/E rates, and average F/E rates were created. These equations were then plugged into the elevation spreadsheets used for the features layout design (see **Appendix E**) to calculate how F/E time and the F/E rates would vary under all water level and lockage length combinations. The total equalization operation time was then estimated by adding the sum of the WSB equalization operations to the estimates of time for the lock F/E system to drain the residual water left in the lock. All of the resulting distributions were then averaged (except for the max F/E rate) to present the results.

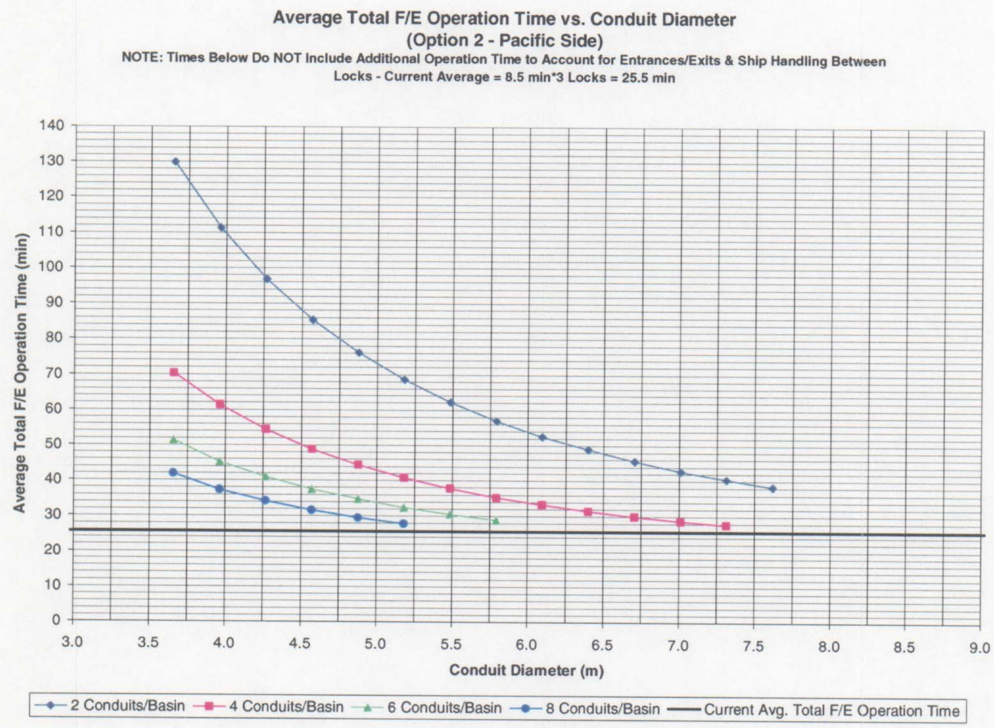
This analysis yields graphs that show (for each option, ocean side and # of conduits/basin):

<u>Average Total F/E Operation Time vs. Conduit Diameter</u>
This is simply the time needed for filling/emptying. - No allowance is made for ship entrances/exits and ship handling between locks. The average is taken for the sum of the WSB operations plus the time to drain the residual water in the lock chamber for each possible lock operation (lake-lock, lock-lock, lock-ocean) for both up and down lockages.

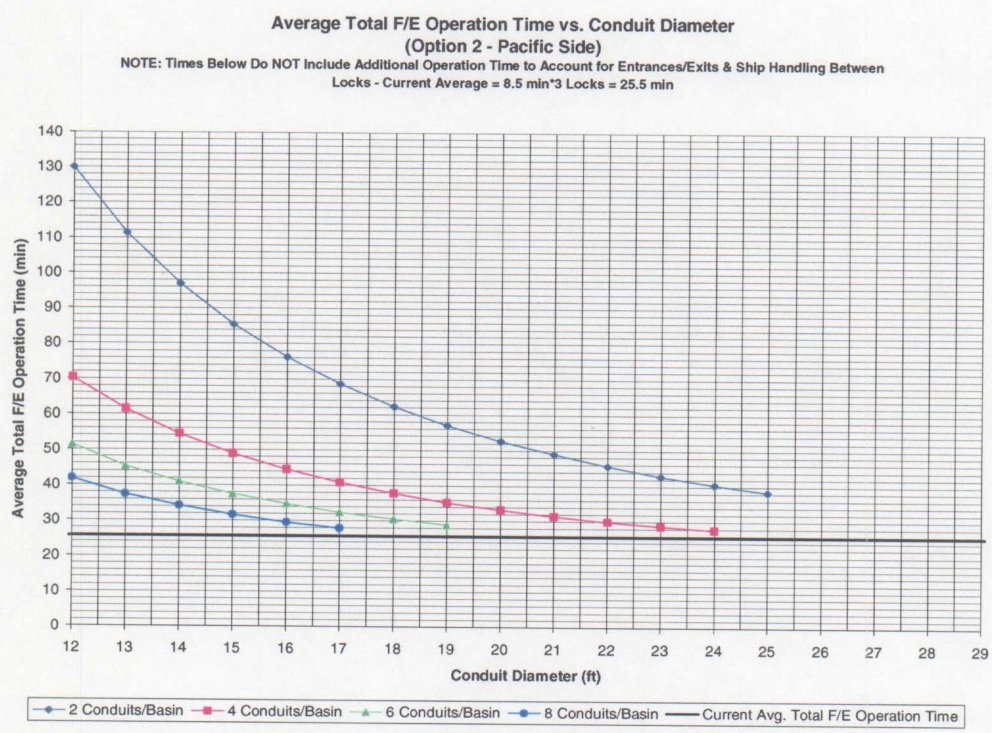
<u>Instantaneous Maximum F/E Rate vs. Conduit Diameter</u>
This is the maximum F/E rate that theoretically should be experienced for each alternative. The maximum F/E rate for the residual height equalized between locks is also included so that comparisons between the type of operation can also be considered and made more equivalent if ACP deems this important.

<u>Average F/E Rate vs. Conduit Diameter</u>
This is the average F/E rate for all water level and lockage lengths for each alternative. An average F/E rate is also plotted for the lock-lock operation as well. This number provides a reasonable way to compare the overall behavior of the alternatives to the current locks).

Example graphs from the results spreadsheet follow for Option 2 on the Pacific Side (see **Appendix I** for graphs for all options).

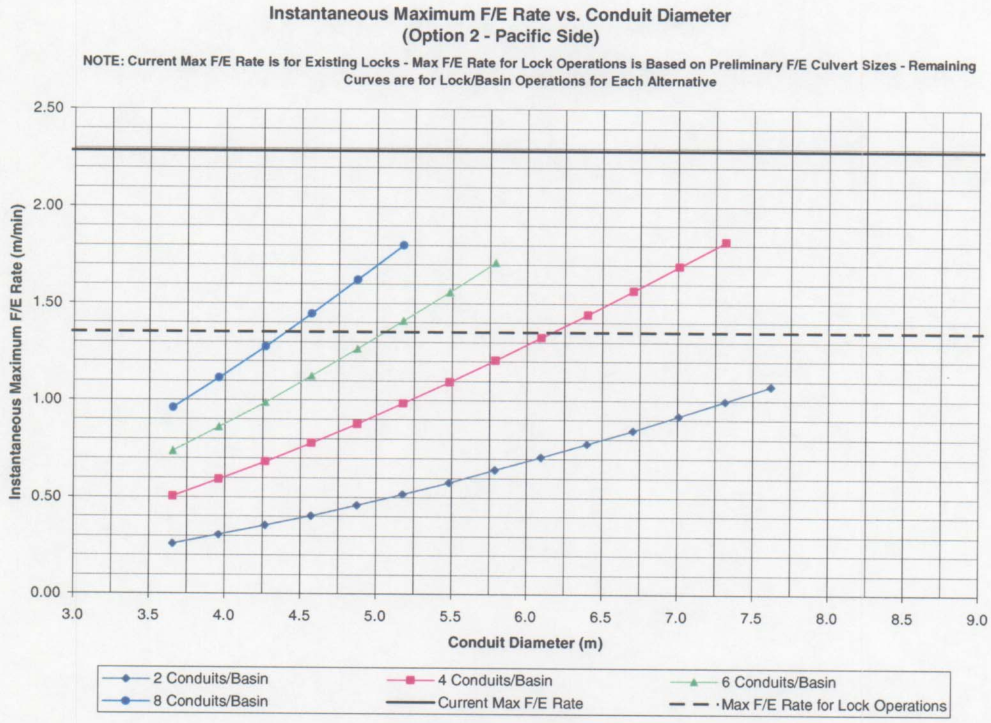


Metric Units

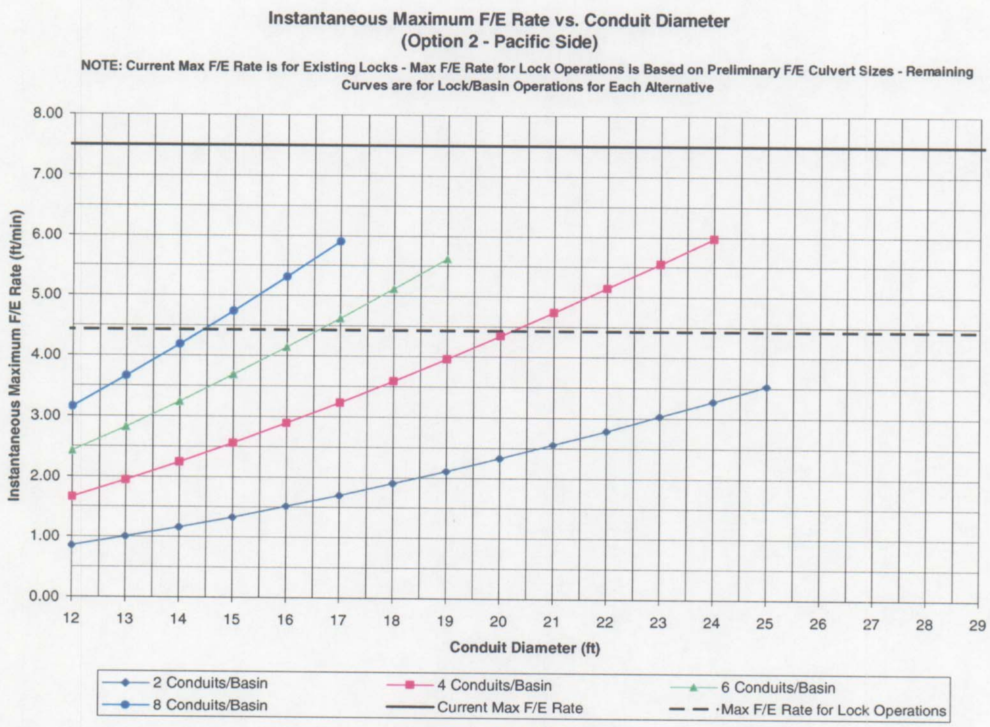


English Units

Figure V.14 – Parametric Curves for Total F/E Operation Time (Opt. 2, Pacific)

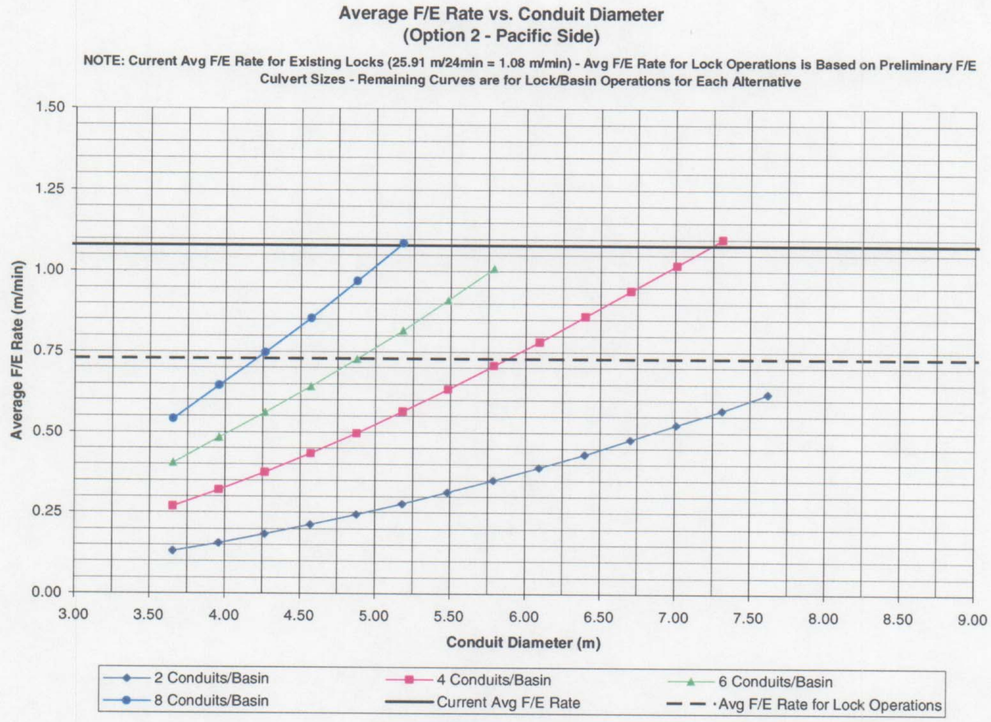


Metric Units

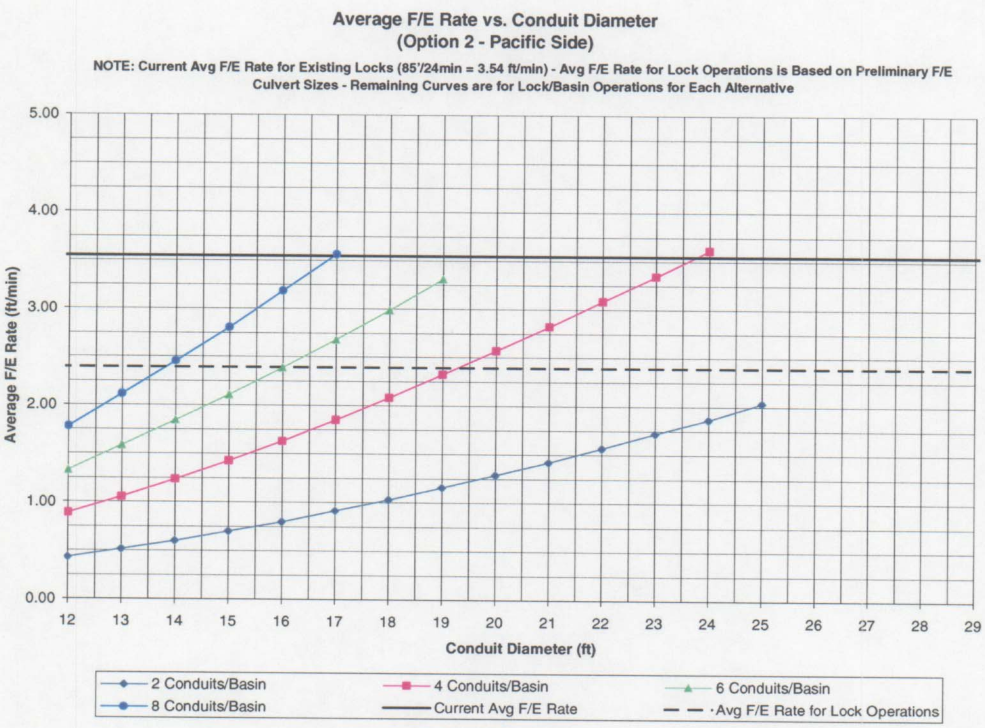


English Units

Figure V.15 – Parametric Curves for Maximum F/E Rate (Opt. 2, Pacific)



Metric Units



English Units

Figure V.16 – Parametric Curves for Average F/E Rate (Opt. 2, Pacific)

These results were then submitted to ACP for review. The finalized WSB conduit arrangement and sizes chosen for further study were:

- Option 1 – 4 conduits/basin (6.10m - 20'),
- Option 2 – 4 conduits/basin (7.32m - 24'),
- Option 3 – 2 conduits/basin (8.53m - 28'), and
- Option 4 (side-by-side basins) – 4 conduits/basin (6.71m - 22'). (see **Figures IV. 8-19**)

At this point, work was halted by ACP before the WSB conduit size could be finalized for the stacked basin arrangement for Option 4. A preliminary analysis using the same conduit size as that selected for the side-by-side basins (see **Figures IV. 33-34**) indicated that the performance characteristics of the stacked WSB would be similar. However, these preliminary reviews also indicated that further refinement was needed to size the conduits for the stacked basin arrangement since the equalization times were approximately 15% longer when compared to the side-by-side basin arrangement.

B. Geotechnical Features Design

1. Atlantic Side

a) Basin Foundations

The top of rock is typically 1 to 10 meters (3 to 33 ft.) below the subgrade levels of the basins on the west side of the proposed locks and is expected to be higher on the east side. Figure 3 illustrates the typical position of the basins and culverts relative to the top of rock. Where the subgrade level of the basins is at or below the top of rock, the rock will provide a competent foundation. Where the subgrade level is above the top of rock, we anticipate that excessive differential settlements are likely to occur if the basins are supported completely or partially on the existing soils, and the foundation options we recommend considering for preliminary design are:

Removing the soils above rock and backfilling to grade with select material that is compacted to achieve a low compressibility. Typically, to achieve low compressibility, the material will be broadly graded across the sand and gravel sizes and will have less than about 15 percent non-plastic fines (material passing the No. 200 sieve). The rock excavated for the new locks may meet these criteria.

If suitable select backfill is not available locally, it may be more cost effective to support the basins on drilled shafts that develop their capacity in end bearing in the sandstone. Preliminary estimates indicate that a 1 meter (3 ft.) diameter shaft extending to relatively unweathered sandstone will achieve an allowable capacity of 2000 kN (220 tons).

As further subsurface information is obtained for the final design of the basins, it will be necessary to re-visit these recommendations. For example, there may be areas identified in the investigation where undercutting is needed only part of the way down to the top of rock.

b) Basin Wall Design

Granular backfill and a longitudinal drain at foundation level are recommended behind basin walls. Recommended earth pressure coefficients are as follows in **Table V.11**:

Table V.11 – Recommended Earth Pressure Coefficients

Backfill Material	Active Earth Pressure Coefficient		At-Rest Earth Pressure Coefficient		Allowable Passive Earth Pressure Coefficient (FS=2)
Well-graded sand-gravel mixture with less than 15% non-plastic fines. Estimated effective friction angle of 38 degree, a saturated unit weight of 22 kN/m^3 (140 pcf) , and a moist unit weight of 21 kN/m^3 (135 pcf).	β 0 10 20 25	K_A 0.24 0.27 0.30 0.33	β 0 10 20 25	K_0 0.38 0.45 0.52 0.55	2.1
Silty sand with less than 35% non-plastic fines. Estimated effective friction angle of 32 degrees,a saturated unit weight of 20 kN/m^3 (130 pcf), and a moist unit weight of 19 kN/m^3 (120 pcf).	β 0 10 20 25	K_A 0.31 0.34 0.41 0.45	β 0 10 20 25	K_0 0.47 0.55 0.63 0.67	1.6
β = slope of the ground surface behind the wall measured in degrees from the horizontal. K_A = active earth pressure coefficient K_0 = at-rest earth pressure coefficient					

Earth pressure coefficients for in-place soils are provided in **Table V.12** below. They are appropriate for estimating long-term loads on walls constructed by top-down methods.

Table V.12 – Recommended Earth Pressure Coefficients For In-Place Soils

In-Place Materials	Active Earth Pressure Coefficient		At-Rest Earth Pressure Coefficient		Allowable Passive Earth Pressure Coefficient (FS=2)
Existing fill, alluvial and residual soils. Estimated effective friction angle of 20 degrees, a saturated unit weight of 19.5 kN/m^3 (125 pcf) , and a moist unit weight of 19 kN/m^3 (120 pcf).	β 0 10 20 25	K_A 0.49 0.57 N/R N/R	β 0 10 20 25	K_0 0.66 0.77 N/R N/R	1.0
Weathered sandstone. Estimated effective friction angle of 37 degrees (for normal stresses in the range of 150 to 350 kPa), a saturated unit weight of 19.5 kN/m^3 (125 pcf), and a moist unit weight of 19 kN/m^3 (120 pcf).	β 0 10 20 25	K_A 0.25 0.28 0.31 0.34	β 0 10 20 25	K_0 0.40 0.47 0.53 0.57	2.0
β = slope of the ground surface behind the wall measured in degrees from the horizontal. K_A = active earth pressure coefficient K_0 = at-rest earth pressure coefficient N/R = not recommended					

Groundwater pressures, as well as the effects of surcharge loads, need to be added to the earth pressures calculated from the coefficients provided above. We anticipate that with the stiffness of the foundation soils and the walls, active earth pressures may not be able to develop and recommend designing at this stage for at-rest earth pressures.

Where the basins are supported on select granular backfill, we anticipate that the foundations for the basin walls will be integral with the basin floor but will apply a larger pressure to the foundation soils. For this situation, an allowable bearing pressure of 0.5 MPa (10 ksf) is recommended at this stage of design for foundations supported on select backfill or weathered rock. An allowable bearing pressure of 1 MPa (20 ksf) is recommended for foundations supported on relatively unweathered rock. The allowable bearing pressure may be increased by 50 percent for transient loading conditions.

The interface friction angle between concrete and select backfill, weathered rock, and rock is estimated to be at least 38 degrees. So, for a factor of safety of 1.5, the allowable coefficient of friction against sliding for cast-in-place concrete foundations bearing on select backfill, weathered rock, and rock is 0.52.

With the limited information available at this stage of the project, we recommend using a cohesion of 100 kPa (2 ksf) and a friction angle of 30 degrees for evaluating global stability in the rock mass (slightly to moderately weathered) at this site.

c) Conduit Alignment and Foundation Support

We anticipate that the conduits for filling and emptying the water saving basins will be constructed with cast-in-place, reinforced concrete. For the typical subsurface conditions illustrated in **Figure 3 in Appendix D – Atlantic Report**, we anticipate that the most cost effective vertical alignment for the conduits will be the one shown in **Figure 3 in Appendix D – Atlantic Report**. To minimize the quantity of excavation in rock, as well as the total quantity of excavation, the alignment brings the conduit up from the base of the lock into the soil as quickly as hydraulic constraints will allow. It then turns horizontally and is incorporated into the construction of the basin floor. An alignment that involves tunneling is discussed in a separate section below.

Rock, weathered rock or the select backfill described above will provide suitable foundation support for the culverts. The existing soils may not provide adequate support and are not recommended at this stage. The options for supporting the culverts above the top of rock are the same as those recommended above for the basins.

d) Permanent Dewatering

Although very limited data are available presently on groundwater levels at the site, we expect that long-term groundwater levels will vary from El. 26 meters (85 ft.) (Gatun Lake level) on the south end of the new locks to about sea level on the north end. Additionally, substantially higher transient levels are likely to occur during and after extended periods of heavy rainfall. Consequently, we recommend providing a drainage blanket under the basin floors and a longitudinal drain at foundation level behind the basin walls. Where practical, these drains should work by gravity to minimize water pressures on the basin walls and floors. Where gravity drainage is not practical, the basin floors and walls should be designed to resist sustained water levels that increase linearly from sea level at the north end of the north basins to El. 26 meters at the south end of the south basins. In addition, transient level should be considered in design. Based on our experience and the limited information available, we recommend designing for a transient level that adds 5 meters of head at all locations if gravity drainage is not practical. If site-specific data becomes available, we recommend adjusting the transient level used in design to be consistent with the site-specific data. We anticipate that the cut-off measures employed during construction (described below) will be effective in the long term and will substantially reduce the quantity of water that has to be handled by the permanent drainage system.

e) Excavation

The soils encountered in excavations for the basins and culverts should be excavateable with conventional earthmoving equipment such as scrapers, loaders, and hydraulic excavators. Fill materials may contain boulders that need to be broken down before they can be hauled away. Also, borehole data indicate very soft organic soils that will not support the weight of excavation and hauling equipment may be encountered on the north end of the construction. So, grading activities may need to be modified locally to accommodate these very soft soils.

Experience during excavation of the Gatun Locks (IEC, 1915 Paper No.11), as well as the low compressive strengths reported from testing in 1947 (Appendix C) indicate that rock at the site can be excavated with hydraulic excavators and track loaders. Some of the material may require ripping before excavation. We do not expect that blasting will be needed for excavation at this site.

The limited available data suggests that soils excavated for the construction of the water saving basins may be difficult to re-use as structural fill because of its high moisture content, the presence of cobbles and boulders larger than 6 inches, and the presence of concrete, coral and wood fragments and canal excavation debris.

f) Construction Dewatering

An evaluation of dewatering requirements for new locks is beyond the scope of this study, but the limited information available suggests that dewatering will be a major challenge. Gatun Lake is adjacent to the new lock excavations, and the head difference between Gatun Lake and the base of the lock excavations range from 30 to 45m (100 to 150 ft.). Regional geologic trends suggest that the sandstone units that occur beneath the lake extend into and beneath the lock excavations, so there may be a pervious hydraulic connection through the sandstone between the lake and the excavations. We believe that a major exploration and evaluation effort is warranted to address this dewatering challenge.

We understand that lock excavations in 1939-1942 were performed without cut-off walls. Nevertheless, at this stage, we believe it is prudent to assume that dewatering wells and a groundwater cut-off wall or grout curtain that surrounds the excavation for the new locks and water saving basins (like that illustrated in **Figure 4 in Appendix D – Atlantic Report**) will be needed. The objective of the cut-off wall or grout curtain would be to reduce the quantity of water that has to be pumped from wells and sumps to a manageable level. A drain system that provides gravity drainage into the ocean for the groundwater that is above sea level may also be beneficial.

The excavations required for the water saving basins are shallow relative to the lock excavations, except the excavation for the culverts adjacent to the lock. There, we expect that the lock dewatering system will also be effective in dewatering the excavation for the culvert. Elsewhere, we anticipate that groundwater can be controlled by pumping from sumps. Therefore, the incremental cost of dewatering associated with building the water saving basins is in the extra length of the cut-off wall or grout curtain needed to encompass the basins.

g) Temporary Slopes

We recommend designing for temporary slopes of 1.5H:1V in soils for slopes up to 6 meters high. For slopes higher than 6 m, we recommend adding a 3-meter wide (10 ft) bench for every 6 meters of slope height.

Excavations in rock are expected only for the installation of the culverts connecting the locks and basins. Stability of these cuts will be affected if discontinuities (joints, bedding planes or faults) that slope toward the excavation are present in the rock mass. At present, the only data available to assess this is very general. Based on the USGS Geologic Map, the 1908 profile and **Figure 2 in Appendix D – Atlantic Report**, bedding planes appear to be relatively flat lying, with dips ranging from about 5 to 7° to the north-northwest. The information on joint orientation in the borehole logs is limited to general comments that the joints are typically near-vertical, tight or infilled with calcite, and discontinuous. Sliding is most likely on discontinuities that slope steeper than about 45 degrees. So, we recommend designing for temporary slopes of 1H:1V in rock, thus eliminating the risk of sliding on discontinuities steeper than 45 degrees.

h) Excavation Support

On a site with such variable soil conditions and the potential for high groundwater, a support system that can accommodate a variety of conditions is desirable. A system of soldier piles, lagging, and tie-back anchors is flexible enough to deal with the variety expected and is recommended for excavation support where constraints do not allow excavation at the temporary slopes in the manner recommended above. Pre-drilling may be needed to extend the soldier piles through debris in the old fill and into the rock below the base of the excavation. Casing may also be needed to stabilize the drill holes for tie-backs and soldier piles where old fill material is present. Two alternatives to the soldier-pile-lagging system for the facing on a tie-back wall are (1) overlapping drilled shafts, and (2) a slurry wall excavated in panels and backfilled with concrete. These alternatives may offer the advantage of serving both as a temporary and permanent wall for the basins. Soil nailing is not recommended. Because of groundwater and variable soil conditions, the stand-up time of the excavation may not be adequate for successful soil nailing.

In general, we recommend designing tie-back walls in accordance with guidelines provided in *FHWA-IF-99-015 Geotechnical Engineering Circular No. 4, Ground Anchors and Anchored Systems,* June 1999.

At this stage of design, we recommend assuming the tie-backs will be anchored in weathered rock or rock. The limited information available on the overburden soils suggests it is highly variable and soft and may not provide suitable anchorage for tie-backs. Ultimate bond strengths of 0.2 MPa (30 psi) and 0.5 MPa (70 psi) are recommended for weathered, and fresh to moderately weathered rock, respectively. A factor of safety of two is recommended on these ultimate bond strengths. With anchors in rock, the minimum unbonded lengths of 4.5 m for strand tendons and 3 m for bar tendons will be easily achieved. Installation angles of 10 to 45 degrees measured from the horizontal are recommended. The hole diameter depends on the anchor capacity and the level of corrosion protection required. In general, larger diameter holes are needed in permanent applications to provide adequate corrosion protection. Anchor capacities in the range of 130 to 400 kN (30 to 90 kips) may be achieved with Grade 60 bar tendons. For temporary conditions, hole diameters in the range of 100 to 125 mm are considered reasonable. Approximately 25 to 50 mm of additional diameter is needed to accommodate corrosion protection for permanent installations. Capacities in the range of 470 to 1200 kN (100 to 270 kips) may be achieved with strand tendons in similar diameter holes, although larger diameter holes may be desirable to reduce the required anchor lengths.

Once the layouts are further advanced and the type, locations, and height of any walls can be identified, we will provide recommendations on pressures to use in preliminary design. If walls higher than about 6 m (20 feet) are needed, recommendations provided below on geotechnical investigations will need to be modified to include investigations for the walls. If permanent walls are planned, investigations will need to include evaluation of corrosivity of the soils.

i) Tunneling for Conduit Installation

A conceptual vertical alignment for the conduits (used for emptying and filling the basins) if they are installed by tunneling is provided in **Figure 5 in Appendix D – Atlantic Report**. As described above, the upper rock at the site consists of a weak to moderately strong, jointed to locally massive sandstone that is sometimes highly weathered and weakly cemented. The tunnels would be located below the groundwater table and some of the rock units are likely to be highly pervious.

Tunnel and shaft excavation in the rock at this site could be done with hydraulic excavators or roadheaders. Blasting should not be needed.

Support and dewatering, however, are expected to be significant challenges. Rock behavior that can be expected in this rock includes loosening and raveling along pre-existing discontinuities in the stronger units, slaking and raveling in argillaceous (or clay-rich) units, raveling and running of sand in the weaker units, and flowing of sand in the weaker units if dewatering is ineffective.

Dewatering to lower the groundwater levels below the tunnel invert would be needed for safe excavation and to prevent running or flowing conditions in the weakly cemented rock units. Methods like those illustrated in **Figure 4 in Appendix D – Atlantic Report** for the lock excavation would need to encompass the tunnels as well.

The available information indicates the rock at the site will not provide reliable anchorage for rock reinforcement. Thus, for construction safety, as well as tunnel support, full arch support will be needed. The excavation will need to proceed in increments of about 1.5 meters (5 ft.) with arch support installed after each excavation increment. The recommended arch support involves lattice girders on 1.5 meter (5 ft.) spacings; the girders are embedded in shotcrete and a minimum of 200 mm (8 in.) of shotcrete is applied between girders to form a structural arch (refer to **Figure 6 in Appendix D – Atlantic Report**). Alternatively, steel ribs and timber lagging could be used for initial support.

For the shafts, we recommend sloping the excavation back as described above in the soils. In rock, we recommend using the same level of support as shown for the tunnels at this stage of the design. Where the overburden soils are too deep or other constraints hinder the slope layback, we recommend assuming steel ribs and timber lagging will be used for initial support at this stage of design. Effective dewatering is critical to the success of these shaft construction techniques.

As the tunnels and shafts will need to withstand internal water pressure during operation, we anticipate that a final lining of reinforced concrete and an internal circular cross-section will be needed. The concrete lining would be installed after tunnel excavation is completed. The excavated cross-section is an inverted horseshoe, as shown in **Figure 6 in Appendix D – Atlantic Report**, because the horseshoe shape is simpler to excavate and a wide, level invert is desirable during excavation for equipment access. Keep in mind that the hydraulic analysis assumed a rectangular or square cross section so that additional analysis would be needed to size the conduits if tunneling were adopted as the preferred construction alternative.

All things considered, the site is not well-suited to tunneling, and tunneling is expected to be substantially more costly than cut-and-cover installation for the conduit. Cut-and-cover construction is relatively inexpensive because the rock can likely be excavated without blasting. The advantages of cut-and-cover construction are further increased because of the high costs of the dewatering and support needed for tunnel construction.

j) Comments on Basin Locations

From a geotechnical perspective, there is a strong preference for locating the water saving basins on the east side of the proposed locks. The elevation of the top of rock is expected to be higher on the east side, so less excavation and replacement of unsuitable material would be needed. On the east side, the excavation for basin construction would be well removed from the existing locks and thus less likely to interfere with lock operations. On the west side, we estimate that the excavation for basin construction would extend to within about 20 meters (65 ft.) of the existing locks in plan dimension and would extend to a level about 16 meters (52 ft.) below the maximum water level in the existing lock. Temporary excavation support would be needed, but more critically, the stability of the existing lock walls would have to be evaluated under these conditions and special reinforcement or other support measures may be needed.

2. Pacific Side

a) Basin Foundations

For the middle and lower locks, the top of sound rock is typically 15m (49 ft.) to 20m (66 ft.) above the subgrade levels of the basins on the southwest side of the proposed locks and near the subgrade elevation of the basins on the northeast side. The rock is anticipated to be basalt. Figure 4 illustrates the interpreted position of the basins and culverts relative to the top of sound rock for the middle and lower locks. Weathered rock and sound rock will provide a suitable foundation for the basins. As shown on **Figure 1 in Appendix D – Pacific Report**, if the water saving basins are located on the northeast side of the proposed P-1 alignment, a portion of the water saving basins could extend into a channel that was partially excavated in 1939. We do not have information regarding the material in this channel. However, this area may be underlain by soft materials or fill that are not likely suitable as foundation material. Foundation options for this area are described below.

A rock floor may be feasible for the basins that are founded in slightly weathered to fresh basalt. However, at this time there is no data on the rock permeability and not enough data to reliably evaluate the rock quality for the basin floors. Consequently, we recommend that a concrete floor be assumed for the basins at this stage in the project.

For the upper lock, weathered rock of the La Boca Formation may be at the subgrade elevations or up to 7m (23 ft.) below the subgrade elevations of the basins. **Figure 5 in Appendix D – Pacific Report** shows the positions of the basins and culverts relative to the top of weathered rock. Weathered rock should be a suitable foundation material for the basins. However, based upon the data available, the residual soil above the weathered La Boca Formation is not considered a suitable foundation material. Excessive differential settlements are likely to occur if the basins are supported completely or partially on the existing soils.

Pre-loading the residual soils may be an option to reduce excessive differential settlements for shallow foundations. For preliminary estimates, a pre-load of approximately 1.5 times the load applied by the structure can be assumed. Shallow foundations may also be feasible in areas where the effective vertical stress from the excavation is 1.5 times the average pressure exerted by the structure. If pre-loading is considered further, site-specific analysis based upon consolidation test data may be necessary.

We recommend considering the following foundation options for preliminary design for the basins for the upper locks and the part of the basins for the middle and lower locks that extend into the 1939 channel:

Remove the soft, unsuitable overburden above the weathered rock and backfill to grade with select material that is compacted to achieve a low compressibility. Typically, to achieve low compressibility, the material will be broadly graded across the sand and gravel sizes and will have less than about 15 percent non-plastic fines (material passing the No. 200 sieve). The basalt excavated for the new locks may meet these criteria.

If suitable select backfill is not available locally, it may be more cost effective to support the basins on drilled shafts socketed into rock or weathered rock that develop their capacity in end bearing and side resistance. Preliminary estimates indicate that a 1m (3.3 ft.) diameter shaft with a 3m (10 ft.) to 4m (13 ft.) long socket in weathered rock will achieve an allowable capacity of 2000 kN (220 tons).

Where fill is required to reach the desired foundation subgrade elevation, we recommend for preliminary design that the crest of the fill is maintained at least 1.5m (5 ft.) but preferably 3m (10 ft.) from the outside face of the foundations for the basins. For preliminary design, a slope of 2H:1V up to 6m (20 ft.) high can be assumed. For slopes higher than 20m, a bench 3m (10 ft.) wide should be added at mid-slope or every 6m of height.

As further subsurface information is obtained for the final design of the basins, it will be necessary to re-visit these recommendations. For example, there may be areas identified in the investigation where undercutting is needed only part of the way down to the top of rock.

b) Basin Wall Design

Granular backfill and a longitudinal drain at foundation level are recommended behind basin walls. Earth pressure coefficients recommended for preliminary design are provided in **Table V.13** for backfill materials and in **Table V.14** for in-place soils.

For preliminary design, the resultant earth pressure load (P) on top down constructed, yielding walls for the permanent condition can be calculated using:

$P = 0.65K_A\gamma H^2$ where; K_A = Coefficient of active earth pressure
γ = Unit weight of soil
H = Height of wall

The resultant load (P) is located at the mid-height of the wall. For preliminary design, the resultant earth load (P) on top down constructed, unyielding walls for the permanent condition can be calculated using:

$P = 0.5K_0\gamma H^2$ where; K_0 = Coefficient of at-rest earth pressure
γ = Unit weight of soil
H = Height of wall

The resultant load (P) is located at the mid-height of the wall.

Table V.13 - Recommended Earth Pressure Coefficients for Backfill Material

Backfill Material	**Active Earth Pressure Coefficient**		**At-Rest Earth Pressure Coefficient**		**Allowable Passive Earth Pressure Coefficient (FS=2)**
Well-graded sand-gravel mixture with less than 15% non-plastic fines. Estimated effective friction angle of 38 degrees, a saturated unit weight of 22 kN/m³ (140 pcf), and a moist unit weight of 21 kN/m³ (135 pcf).	<u>β</u> 0 10 20 25	<u>**K_A**</u> 0.24 0.27 0.30 0.33	<u>β</u> 0 10 20 25	<u>**K_0**</u> 0.38 0.45 0.52 0.55	2.1
Silty sand with less than 35% non-plastic fines. Estimated effective friction angle of 32 degrees, a saturated unit weight of 20 kN/m³ (130 pcf), and a moist unit weight of 19 kN/m³ (120 pcf).	<u>β</u> 0 10 20 25	<u>**K_A**</u> 0.31 0.34 0.41 0.45	<u>β</u> 0 10 20 25	<u>**K_0**</u> 0.47 0.55 0.63 0.67	1.6
Notes: β = slope of the ground surface behind the wall measured in degrees from the horizontal.	K_A = active earth pressure coefficient		K_0 = at-rest earth pressure coefficient		

Table V.14 - Recommended Earth Pressure Coefficients for In-Place Materials

In-Place Materials	**Active Earth Pressure Coefficient**		**At-Rest Earth Pressure Coefficient**		**Allowable Passive Earth Pressure Coefficient (FS=2)**
Existing fill, alluvial and residual soils. Estimated effective friction angle of 20 degrees, a saturated unit weight of 19.5 kN/m^3 (125 pcf), and a moist unit weight of 19 kN/m^3 (120 pcf).	β 0 10 20 25	**K_A** 0.49 0.57 N/R N/R	β 0 10 20 25	**K_0** 0.66 0.77 N/R N/R	1.0
Weathered shale (La Boca Formation). Estimated effective friction angle of 25 degrees (for normal stresses in the range of 150 to 350 kPa), a saturated unit weight of 19.5 kN/m3 (125 pcf) and a moist unit weight of 19 kN/m3 (120 pcf)	β 0 10 20 25	**K_A** 0.41 0.46 0.57 N/R	β 0 10 20 25	**K_0** 0.58 0.68 0.77 N/R	1.2
Weathered basalt. Estimated effective friction angle of 37 degrees (for normal stresses in the range of 150 to 350 kPa), a saturated unit weight of 22.8 kN/m^3 (145 pcf), and a moist unit weight of 22.0 kN/m^3 (140 pcf).	β 0 10 20 25	**K_A** 0.25 0.28 0.31 0.34	β 0 10 20 25	**K_0** 0.40 0.47 0.53 0.57	2.0
Notes: β = slope of the ground surface behind the wall measured in degrees from the horizontal.	K_A = active earth pressure coefficient		K_0 = at-rest earth pressure coefficient N/R = not recommended		

Groundwater pressures, as well as the effects of surcharge loads need to be added to the earth pressures calculated from the coefficients provided above.

For conventional, concrete retaining walls, we anticipate that with the stiffness of the foundation soils and the walls, active earth pressures may not be able to develop and recommend designing for at-rest earth pressures at this stage of the project.

Where the basins are supported on select granular backfill, we anticipate that the foundations for the basin walls will be integral with the basin floor but will apply a larger pressure to the foundation soils. For this situation, an allowable bearing pressure of 0.5 MPa (10 ksf) is recommended at this stage of design for foundations supported on select backfill or weathered rock of the La Boca Formation. An allowable bearing pressure of 2.4 MPa (50 ksf) is recommended for foundations supported on the sound (fresh to moderately weathered) basalt expected to be present at the subgrade elevations of the middle and lower locks. The allowable bearing pressure may be increased by 50 percent for transient loading conditions. If ACP obtains additional information on the allowable bearing capacity of these materials, we recommend re-evaluating the values provided above in light of this information.

The interface friction angle between concrete and select backfill, weathered basalt, and sound basalt is estimated to be approximately 38 degrees. So, for a factor of safety of 1.5, the allowable coefficient of friction against sliding for cast-in-place concrete foundations bearing on select backfill, weathered basalt and sound basalt is 0.52. The interface friction angle between concrete and weathered La Boca Formation is estimated to be 25 degrees. For a factor of safety of 1.5, the allowable coefficient of friction against sliding for cast-in-place concrete foundations bearing on weathered La Boca Formation is 0.31.

With the limited information available at this stage of the project, we recommend using the following values for evaluating global stability in the rock mass at this site:

- Slightly to moderately weathered basalt - cohesion of 100 kPa (2 ksf) and a friction angle of 35 degrees, and
- Weathered La Boca Formation (shale) - friction angle of 25 degrees.

These values should be used with care because the sliding resistance of the rock mass may also be controlled by discontinuities such as joints, shear zones, and bedding planes in the rock mass. At this stage of the project, we do not have data to evaluate the effect of these features on slope stability.

c) Conduit Alignment and Foundation Support

We anticipate that the conduits for filling and emptying the water saving basins associated with the upper locks will be constructed with cast-in-place, reinforced concrete. For the typical subsurface conditions at the upper locks illustrated in **Figure 5 in Appendix D – Pacific Report**, we anticipate that the most cost-effective vertical alignment for the conduits will be the one shown in **Figure 5 in Appendix D – Pacific Report**. To minimize the quantity of potential excavation in rock, as well as the total quantity of excavation, the alignment brings the conduit up from the base of the lock, as quickly as hydraulic constraints will allow. It then turns horizontally and is incorporated into the construction of the basin floor. An alignment that involves tunneling for the basins associated with the middle and lower locks is discussed in a later section.

Sound rock, weathered rock or the select backfill described above will provide suitable foundation support for the conduits. However, soils above the weathered rock are not anticipated to provide adequate support and are not recommended at this stage. The options for supporting the conduits above the top of weathered rock are the same as those recommended above for the basins.

d) Permanent Dewatering

Although no data are available presently on groundwater levels at the site, we expect that long-term groundwater levels will vary from El. 26m (85 ft.) (Gatun Lake level) on the northwest end of the new locks to about sea level on the southeast end. Additionally, substantially higher transient levels are likely to occur during and after extended periods of heavy rainfall. Consequently, we recommend providing a drainage blanket under the basin floors and a longitudinal drain at foundation level behind the basin walls. Where practical, these drains should work by gravity to minimize water pressures on the basin walls and floors. Where gravity drainage is not practical, the basin floors and walls should be designed to resist sustained water levels that increase linearly from sea level at the southeast end of the southern basins to El. 26m (85 ft.) at the northwest end of the northern basins. In addition, a transient level should be considered that adds 5m (16 ft.) of head at all locations. We anticipate that the cut-off measures employed during construction (described below) will be effective in the long term also and will substantially reduce the quantity of water that has to be handled by the permanent drainage system.

e) Excavation

We anticipate the soils that will be encountered in excavations for the basins and culverts can be excavated with conventional earthmoving equipment such as scrapers, loaders, and hydraulic excavators. Fill materials may contain boulders that need to be broken down before they can be hauled away. Also, site observations and descriptions on the borehole logs indicate soft soils, that may not support the weight of excavation and hauling equipment, may be encountered near the drainage leading to Miraflores Lake at the north end of the proposed locks. So, grading activities may need to be modified locally to accommodate these soft soils. This drainage leading to Miraflores Lake and the area of the 1939 excavations for the supplemental locks are under water. In order to excavate these areas and construct the basins, cofferdams will be required to dewater the location of the proposed basins.

The relative low compressive strengths reported from testing in 1947 (**Appendix D – Pacific Report**) and the description on the boring logs indicate that the weathered rock of the La Boca Formation can be excavated with hydraulic excavators and track loaders. Some of the material may require ripping before excavation.

The relatively high compressive strengths reported from the 1947 testing (**Appendix D – Pacific Report**) and the descriptions on the boring logs for the basalt indicate that blasting will be required to excavate the basalt. The blast rock from the basalt is likely to be hard, and durable and could be used on site for rockfill. The blast rock could also be used for structural fill after processing to remove particles greater than 200mm (8 inches) in diameter.

The limited available data suggests that soils excavated for the construction of the water saving basins may be difficult to re-use as structural fill because of its high moisture content and high plasticity.

f) Construction Dewatering

Dewatering for construction of the new locks is expected to be a major challenge. Miraflores Lake is adjacent to the new lock excavations, and the head difference between the lake and the base of the lock excavations ranges from 30m (98 ft.) to 45m (148 ft.).

In the basalt and the La Boca Formations, groundwater will generally flow into excavations through discontinuities in the rock mass as well as fault and fracture zones. It is possible that these pervious zones may be hydraulically connected to Miraflores Lake. We believe that a major exploration and evaluation effort is warranted to address this dewatering challenge. However, based on the limited available information, we envision that a groundwater cut-off wall or grout curtain that surrounds the excavation for the new locks and water saving basins as shown in **Figure 6 in Appendix D – Pacific Report** may be needed. The objective of the cut-off wall or grout curtain would be to reduce the quantity of water that has to be pumped from wells and sumps to a manageable level. A combination of grout curtain and cutoff trench may be required. We anticipate the cutoff trench would be used in overburden or weathered rock (including much of the La Boca Formation) that can be excavated with hydraulic excavators or clamshell buckets. The grout curtain would be used in rock to restrict flow through discontinuities in the rock mass such as fractured and faulted zones. A drain system that provides gravity drainage into the ocean for the groundwater that is above sea level may also be beneficial.

The excavations required for the water saving basins are shallow relative to the lock excavations, except the excavation for the conduits adjacent to the lock. There, we expect that the lock dewatering system will also be effective in dewatering the excavation for the culvert. Elsewhere, we anticipate that groundwater can be controlled by pumping from sumps. Therefore, the incremental cost of dewatering associated with building the water saving basins is expected to be negligible.

g) Temporary Slopes

We recommend designing for temporary slopes of 1.5H:1V in soil and weathered shale of the La Boca Formation for slopes up to 6m (20 ft.) high. For slopes higher than 6m, we recommend adding a 3m (10 ft.) wide bench for every 6m of slope height. For slopes in weathered basalt, up to 6m high, we recommend designing for temporary slopes of 1H:1V. For slopes higher than 6m, we recommend adding a 3m wide bench for every 6m of slope height.

Excavations in sound basalt are expected for basins associated with the middle and lower locks. Stability of these cuts will be affected if discontinuities (joints, bedding planes or faults) that slope toward the excavation are present in the rock mass. At present, the only data available to assess this is very general. The information on joint orientation in the borehole logs is limited to general comments that the joints are typically near vertical, tight or infilled with calcite or chlorite and may be slickensided. At this stage of the design, based upon experience with good quality rock masses like the basalt, we recommend designing for temporary slopes at 0.5H:1V. Wire mesh should be draped (attached only at the top) over the rock face and rock bolts may be required locally to protect workers from falling rock. Steeper slopes may achievable depending upon discontinuities in the rock mass and would require detailed investigations.

h) Excavation Support

(1) Overburden and Weathered Rock

On a site with such variable soil conditions and the potential for high groundwater, a support system that can accommodate a variety of conditions is desirable. A system of soldier piles, lagging, and tie-back anchors is flexible enough to deal with the variety expected and is recommended for excavation support where constraints do not allow excavation at the temporary slopes recommended above. Pre-drilling will be needed to socket the soldier piles into the basalt and provide support at the base of the wall. Pre- drilling may also be required to extend the piles into the La Boca Formation. Casing may also be needed to stabilize the drill holes for tie-backs and soldier piles where soft or loose soils are present. Two alternatives to the soldier-pile-lagging system for the facing on a tie-back wall are (1) overlapping drilled shafts, and (2) a slurry wall excavated in panels and backfilled with concrete. These alternatives may offer the advantage of serving both as temporary and permanent walls for the basins. Soil nailing is not recommended. Because of groundwater and poor soil conditions, the stand-up time of the excavation may not be adequate for successful soil nailing.

In general, we recommend designing tie-back walls in accordance with guidelines provided in *FHWA-IF-99-015 Geotechnical Engineering Circular No. 4, Ground Anchors and Anchored Systems,* June 1999. At this stage of design, we recommend assuming the tie-backs will be anchored in weathered rock or rock. The limited information available on the overburden soils suggests it is highly variable, soft, and highly plastic and so may not provide suitable anchorage for tie-backs. Values of ultimate bond strengths recommended for preliminary design are:

- 0.2 MPa (30 psi) for weathered La Boca Formation ,
- 0.5 MPa (70 psi) for weathered basalt, and
- 1.5 MPa (215 psi) for fresh to moderately weathered basalt.

A factor of safety of two is recommended on these ultimate bond strengths. With anchors in rock, the minimum unbonded lengths of 4.5m (15 ft.) for strand tendons and 3m (10 ft.) for bar tendons will be easily achieved. Installation angles of 10 to 45 degrees down as measured from the horizontal are recommended. The hole diameter depends on the anchor capacity and the level of corrosion protection required. In general, larger diameter holes are needed in permanent applications to provide adequate corrosion protection. Anchor capacities in the range of 130 kN

(30 kips) to 400 kN (90 kips) may be achieved with Grade 420 MPa (60 ksi) bar tendons. For temporary conditions, hole diameters in the range of 100mm (4 in.) to 125mm (5 in.) are considered reasonable. Approximately 25mm (1 in.) to 50mm (2 in.) of additional diameter is needed to accommodate corrosion protection for permanent installations. Capacities in the range of 470 kN (100 kips) to 1200 kN (270 kips) may be achieved with strand tendons in similar diameter holes, although larger diameter holes may be desirable to reduce the required anchor lengths. Because of the pyrite described in the boring logs, we recommend assuming that the anchors installed will be subject to aggressive corrosion conditions.

Once the layouts are further advanced and the type, locations, and height of any walls can be identified, we will provide recommendations on pressures to use in preliminary design. If walls higher than about 6m (20 ft.) are needed, recommendations provided below on geotechnical investigations will need to be modified to include investigations for the walls.

(2) Rock

As noted previously, the stability of excavations in rock are controlled by discontinuities such as joints and faults in the rock mass. If the slopes in rock can not be laid back as recommended previously, steeper slopes, approximately 0.25H:1V to vertical, could be designed with rock support. We anticipate that the rock support will consist of untensioned rockbolts and shotcrete installed at the excavation proceeds. The lengths and number of rock bolts necessary for support of the cuts depend upon the geometry of the cuts and the characteristics of the discontinuities (i.e. strength, orientation). At this stage of the project, the data available on the rock cuts and discontinuities is not detailed enough to allow for design of the support elements. However, for preliminary design of vertical cuts in basalt, we recommend assuming a pattern of rockbolts on a 2m (6.6 ft.) by 2m grid with a length of 5m (16 ft.) and shotcrete 75mm (3 in.) to 100mm (4 in.) thick. This type of support is anticipated to be adequate to stabilize small blocks of rock on the face of the cuts, but not blocks formed by major through-going discontinuities. Rock support for the large blocks, if present, will have to be specifically designed based upon discontinuity orientation and strength.

i) Tunneling for Conduit Installation

A conceptual alignment for the conduits (used for emptying and filling the basins) underlying the lower and middle water saving basins if they are installed by tunneling is provided in **Figure 6 in Appendix D – Pacific Report**. We have assumed for this preliminary geotechnical report that the conduits will be 5m (16 ft.) to 10m (33 ft.) in diameter. As described above, the rock underlying the basins for the middle and lower locks is interpreted to consist of generally hard, highly to moderately fractured basalt. The basalt may locally contain zones of poor quality rock that are highly fractured to brecciated, slickensided, and/or highly weathered.

Based upon the existing data, the water saving basins for the upper locks are interpreted to be underlain by the La Boca Formation and the La Boca Fault Zone. We do not recommend tunneling for conduit installation in areas underlain by the La Boca Formation due to the poor quality rock encountered in the existing borings and the anticipated ease of trench excavation in the La Boca.

For areas underlain by the basalt, we recommend, at this stage, that a minimum rock cover equal to 1.5 times the span of the tunnel diameter be maintained over the crown of the tunnels. Less cover may be feasible but will require more rock support to ensure development of a stable rock arch. The excavated cross-section for the tunnels was assumed to be an inverted horseshoe as shown in **Figure 7 in Appendix D – Pacific Report**. The horseshoe shape is simple to excavate and a wide, level invert is desirable during excavation for equipment access. Construction access and portals for the tunnels could be from within the lock excavation or through the shafts. Access through portals in the locks is likely the easier option for construction. Keep in mind that the hydraulic analysis assumed a rectangular or square cross section so that additional analysis would be needed to size the conduits if tunneling were adopted as the preferred construction alternative.

We anticipate that the tunnel and shaft excavations will require blasting due to the high strength of the rock and the variability of conditions that may be encountered. Rock behavior in the tunnels and shafts will be controlled by discontinuities (i.e., joints, faults, bedding planes) in the rock mass. Slaking or squeezing conditions are not expected to be a concern in the basalt. Rock support will be required to stabilize blocks or wedges of rock bounded by discontinuities.

Rock support for the tunnels and shafts will be required to prevent blocks and wedges of rock from sliding or falling out of the crown or sidewalls of the tunnels. A conceptual sketch of typical rock support for the tunnels is shown on **Figure 7 in Appendix D – Pacific Report**. Category I rock support consists of spot positioned rock bolts for specific block or wedges identified in the tunnel excavations. This type of support would be installed in massive rock with few joints. Category II rock support consists of pattern rock bolts and shotcrete. This type of support would be installed in more fractured ground to support rock blocks and wedges and prevent loosening of the rock mass. Shotcrete will prevent small rock blocks from falling from the tunnel crown or sidewalls. Category I and II rock excavation may proceed in 1.5m (5 ft.) to 4m (13 ft.) long rounds and rock support would be installed as the excavation proceeds and typically prior to excavating the next round. Category III ground support would be used in areas of highly fractured rock or fault zones where reliable anchorage for rock bolts is not available. Excavation in Category III ground will need to proceed in increments of about 1m (3.3 ft.) with arch support installed after each excavation increment. The recommended arch support involves lattice girders on 1m (3.3 ft.) spacings; the girders are embedded in shotcrete and a minimum of 200mm (8 in.) of shotcrete is applied between girders to form a structural arch (refer to **Figure 7 in Appendix D – Pacific Report**). Alternatively, steel ribs and timber lagging could be used for initial support.

For the shafts, we recommend sloping the overburden excavation back as described above in the soils and avoid completing the shafts through the overburden. Where the overburden soils are too thick or other constraints hinder the slope layback, we recommend assuming steel ribs and timber lagging will be used for initial support at this stage of design. Alternatively, temporary support measures described previously may be used. In rock, we recommend using the same level of support as shown for the tunnels at this stage of the design.

The tunnels and portions of the shafts will be excavated below the groundwater table. Dewatering to lower the groundwater table is not necessarily required for the tunnel excavations in the basalt. At this stage we do not have any data on the permeability of the rock mass or on quantities of inflow into the tunnels. Inflows into the tunnel will occur preferentially along discontinuities in the rock mass and should not result in significant stability problems such as expected on the Atlantic Locks. However, we recommend considering dewatering measures since this would significantly help during construction of the tunnels. Grouting of the tunnels from the surface prior to excavating or dewatering the areas, as shown in Figure 6 for the lock excavation, may be beneficial for the tunnels as well.

As the tunnels and shafts will need to withstand internal and external water pressure during operation and maintenance, we anticipate that a final lining of reinforced concrete and an internal circular cross-section will be needed. The concrete lining would be installed after tunnel excavation is completed.

j) Comments on Basin Locations

At this stage in the project, we do not have enough data from a geotechnical perspective to prefer locating the water saving basins on one side of the locks or the other. As noted previously, there is little data on the southwest side of the proposed locks. However, the areas excavated in 1939 and the drainage connected to Miraflores Lake are located on the northeast side of the P-1 alignment. These areas will likely require special treatment potentially including cofferdams, additional excavation and special foundations. If the geotechnical conditions on the southwest side are comparable to the northeast, then the southwest side would be preferable from a geotechnical perspective.

C. Structural/Civil Features Features Design

1. Atlantic Side

a) Drainage Considerations

The water saving basins must be designed as watertight containers. However, the water saving basins are frequently empty, as their contents are used to fill the lock chambers. If proper drainage is not provided outside the basins, the basin floors and walls must be designed to withstand the high external hydrostatic pressure as a normal load condition. In order to minimize the basin construction cost, a permanent basin drainage system is required. The geotechnical report provided the following permanent drainage recommendations:

- Provide a drainage blanket under the basin floors
- Provide a longitudinal drain at the foundation level behind the basin walls
- Leave the construction cut-off walls permanently in place, in order to significantly reduce the permanent drainage requirements

The geotechnical engineers anticipate that gravity drainage will be practical, but the drainage system will need to be further developed during future design phases, including a full evaluation of the feasibility of gravity drainage.

Drain cleanouts should be provided in order to allow removal of debris buildup. However, due to maintenance concerns and the consequences of blocked drains, a back up measure is recommended. To provide this redundancy, one-way pressure relief valves (flap or tide-flex valves) are included in the basin floors and walls. In this way, if the underdrains are plugged and the basins are emptied, the external water will drain into the basin if the external hydrostatic pressure exceeds the flap valve threshold value. This threshold value is typically on the order of 0.007 MPa (1 psi).

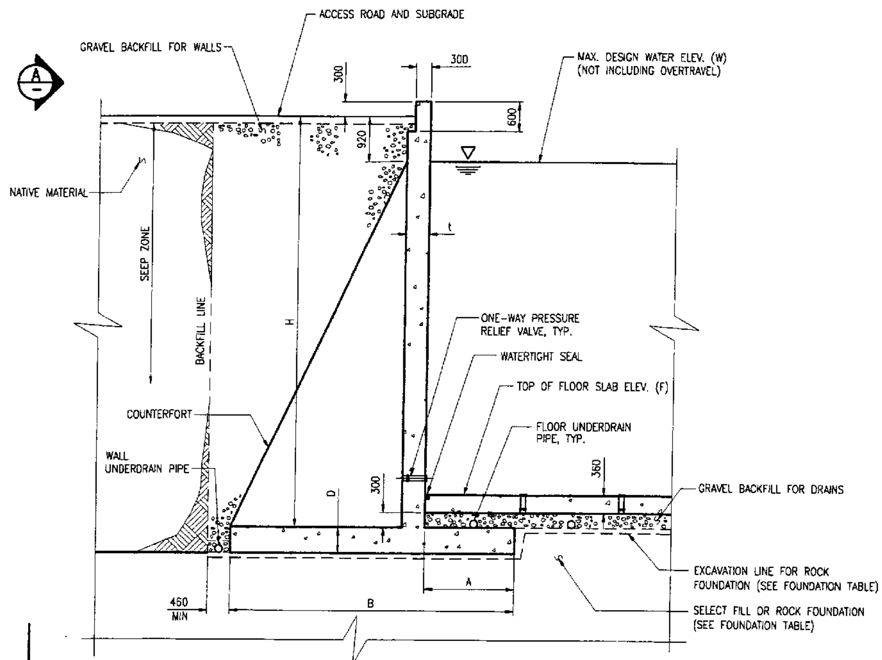
b) Basin Design Concept

The general concept for the water saving basin design is a structure with independent floor and wall systems. The basins were assumed to have vertical reinforced concrete retaining walls and a separate reinforced concrete floor, with a watertight seal at the interface. A typical section of the wall and the interface between the wall and the floor is shown on **Figures V.17-19**. By keeping the wall and floor separate, the effects of settlement and thermal expansion are minimized. This design should also simplify reinforcing details and construction.

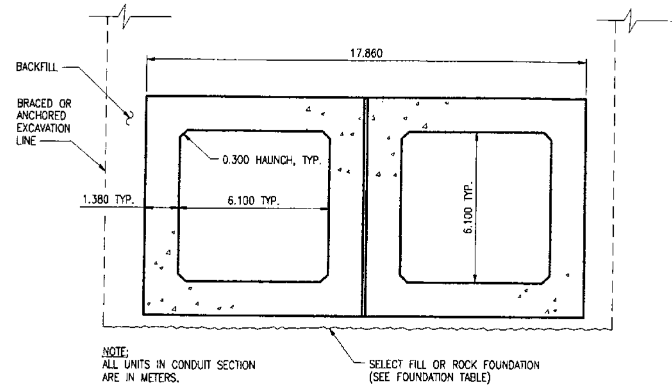
Basins with sloped sides were also briefly evaluated. See **Section C.1.e** for a discussion of this alternate design concept. At this stage, the geotechnical/seismic information is not sufficient to ensure that this alternate design concept is feasible (will provide the required design life and serviceability characteristics). However, it would be advisable to consider this concrete-lined, slope-sided reservoir design in future design phases in order to potentially realize some cost savings.

Seismic ground accelerations and loads were not determined for this conceptual design stage. Seismic analysis and design will need to be addressed in future design phases.

NOTES:
1. ALL DIMENSIONS IN CALLOUTS/NOTES ARE IN MILLIMETERS, UNLESS OTHERWISE NOTED.
2. INTERIOR WALLS REQUIRE OPENINGS WITH REMOVABLE GATES FOR WATER BALANCE AND SURGES.

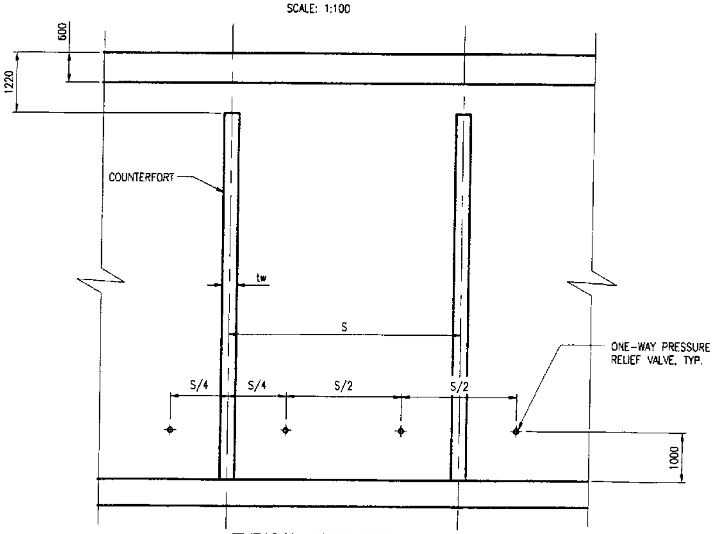


TYPICAL WALL SECTION
SCALE: 1:100

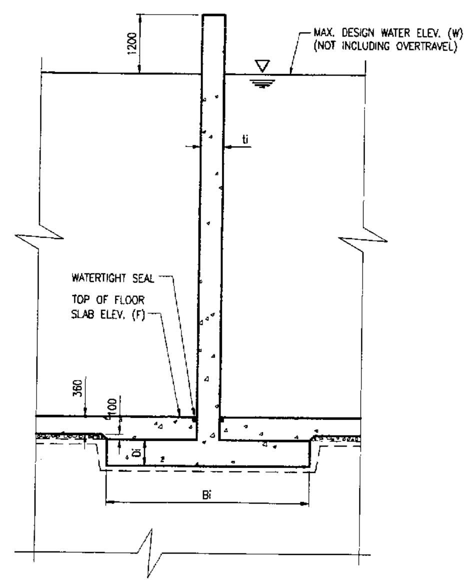


NOTE:
ALL UNITS IN CONDUIT SECTION ARE IN METERS.

TYPICAL CONDUIT SECTION
SCALE: 1:200



TYPICAL WALL VIEW (A)
SCALE: 1:100



TYPICAL INTERIOR WALL SECTION
SCALE: 1:100

WALL ELEVATIONS AND DIMENSIONS

BASIN	W	F	H	A	B	D	t	S	tw	Bi	Di	ti
1	7.830	3.260	6.140	1.370	4.270	0.380	0.300	3.660	0.300	3.350	0.610	0.380
2	5.410	1.440	5.540	1.370	4.270	0.380	0.300	3.660	0.300	3.050	0.610	0.380
3	16.770	10.930	7.410	1.830	5.790	0.530	0.460	4.570	0.300	4.270	0.760	0.610
4	14.600	8.920	7.250	1.680	5.180	0.460	0.380	4.270	0.300	4.270	0.760	0.610
5	24.850	19.440	6.990	1.680	5.180	0.460	0.380	4.270	0.300	3.960	0.610	0.530
6	22.880	17.200	7.260	1.680	5.180	0.460	0.380	4.270	0.300	4.270	0.760	0.610

NOTE: ALL UNITS IN TABLE ARE IN METERS.

FOUNDATION TABLE

BASIN	FOUNDATION TYPE
1	SELECT FILL
2	ROCK
3	ROCK
4	ROCK
5	SELECT FILL
6	SELECT FILL

20 10 0 20 40
SCALE IN METERS
10 5 0 10 20
SCALE IN METERS

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

STUDY OF LOCKS WATER SAVING BASINS
OPTION 1 TYPICAL SECTIONS AT ALIGNMENT A-2

MWH ENGINEERS INC. | FEBRUARY 2002 | FIGURE V. 17

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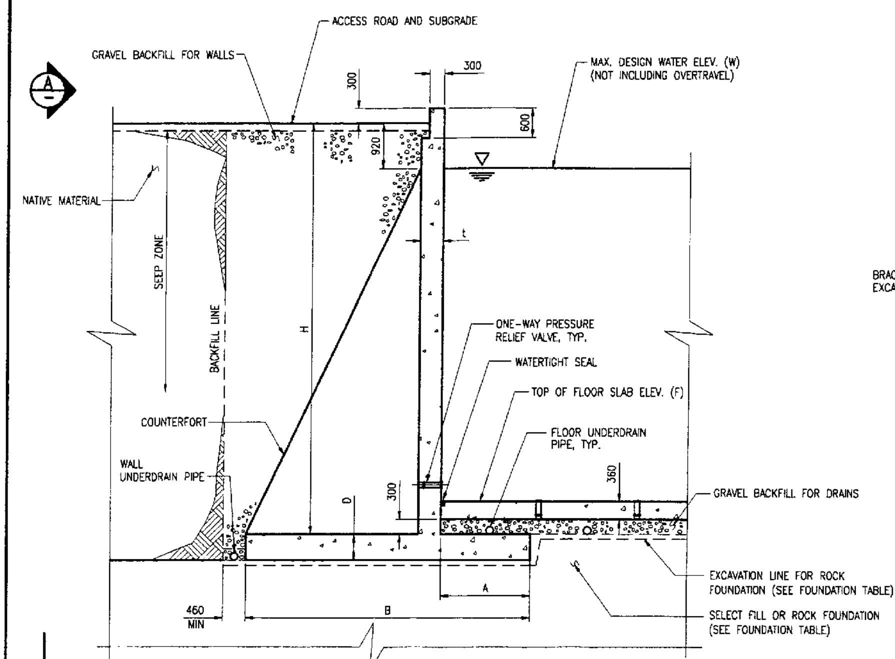
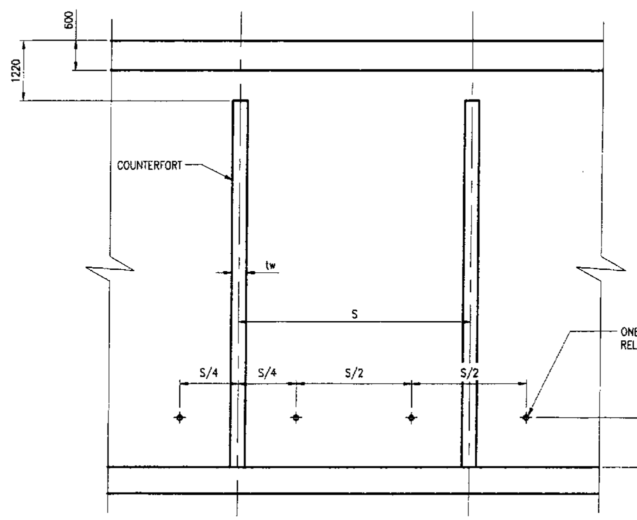


TYPICAL WALL SECTION

SCALE: 1:100

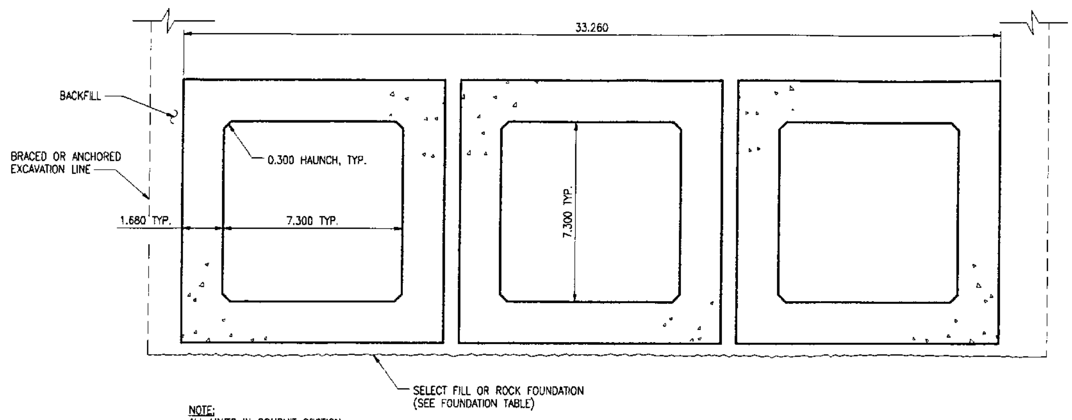


TYPICAL WALL VIEW (A)

SCALE: 1:100

NOTES:

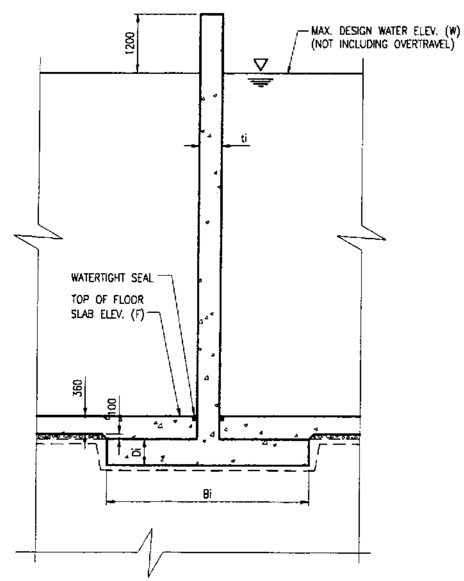
1. ALL DIMENSIONS IN CALLOUTS/NOTES ARE IN MILLIMETERS, UNLESS OTHERWISE NOTED.
2. INTERIOR WALLS REQUIRE OPENINGS WITH REMOVABLE GATES FOR WATER BALANCE AND SURGES.



NOTE: ALL UNITS IN CONDUIT SECTION ARE IN METERS.

TYPICAL CONDUIT SECTION

SCALE: 1:200



TYPICAL INTERIOR WALL SECTION

SCALE: 1:100

WALL ELEVATIONS AND DIMENSIONS

BASIN	W	F	H	A	B	D	t	S	tw	Bi	Di	ti
1	12.150	6.160	7.560	1.830	5.790	0.530	0.460	4.570	0.300	4.570	0.760	0.610
2	9.250	3.980	6.850	1.680	5.180	0.460	0.380	4.270	0.300	3.960	0.610	0.460
3	6.360	1.800	6.130	1.370	4.270	0.380	0.300	3.660	0.300	3.530	0.610	0.380
4	24.470	18.570	7.480	1.830	5.790	0.530	0.460	4.570	0.300	4.570	0.760	0.610
5	22.110	15.880	7.800	1.830	5.790	0.530	0.460	4.570	0.300	4.570	0.760	0.690
6	19.760	13.200	8.130	1.980	6.100	0.610	0.530	4.880	0.300	4.880	0.760	0.760

NOTE: ALL UNITS IN TABLE ARE IN METERS.

FOUNDATION TABLE

BASIN	FOUNDATION TYPE
1	ROCK
2	ROCK
3	ROCK
4	SELECT FILL
5	SELECT FILL
6	SELECT FILL

20 10 0 20 40
SCALE IN METERS

10 5 0 10 20
SCALE IN METERS

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

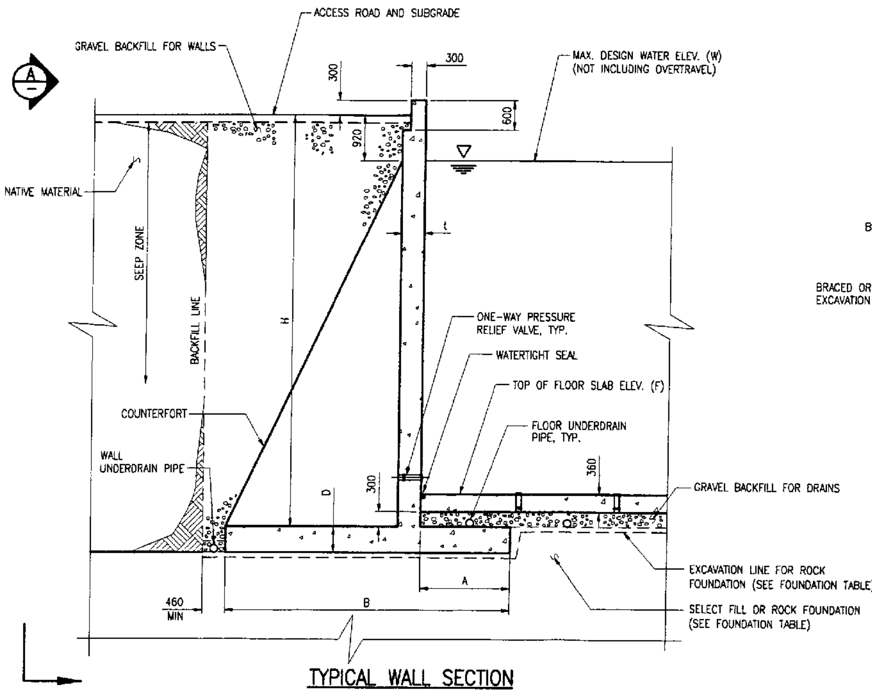
STUDY OF LOCKS WATER SAVING BASINS
OPTION 2 TYPICAL SECTIONS AT ALIGNMENT A-2

INCA ENGINEERS INC. | FEBRUARY 2002 | FIGURE V.18

2/13/2002 7:18 P.M. L:\2000\00-072e\CADD\STRUCT\lon-rev\a_alt2_s001.DWG 01F 12

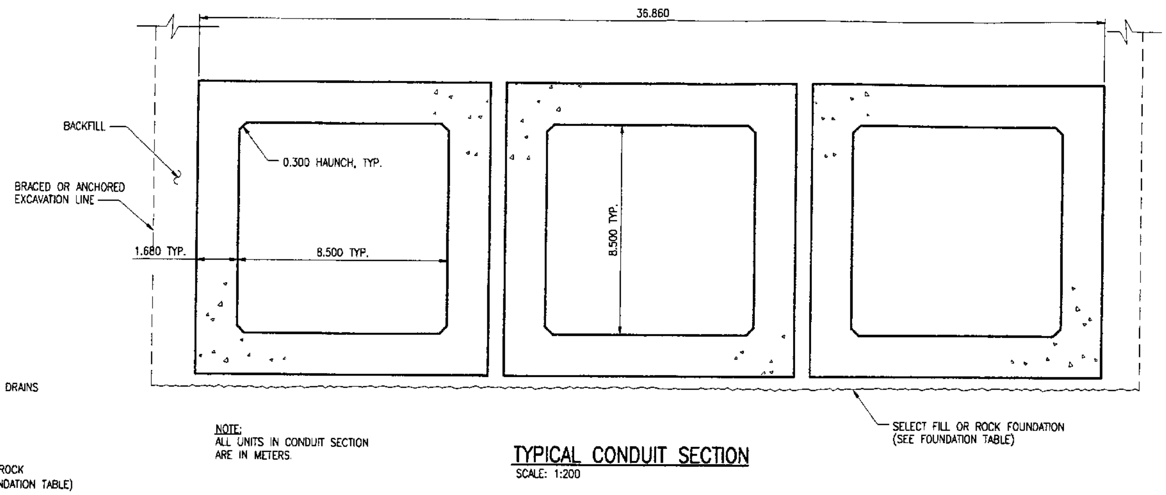
NOTES:

1. ALL DIMENSIONS IN CALLOUTS/NOTES ARE IN MILLIMETERS, UNLESS OTHERWISE NOTED.
2. INTERIOR WALLS REQUIRE OPENINGS WITH REMOVABLE GATES FOR WATER BALANCE AND SURGES.



TYPICAL WALL SECTION

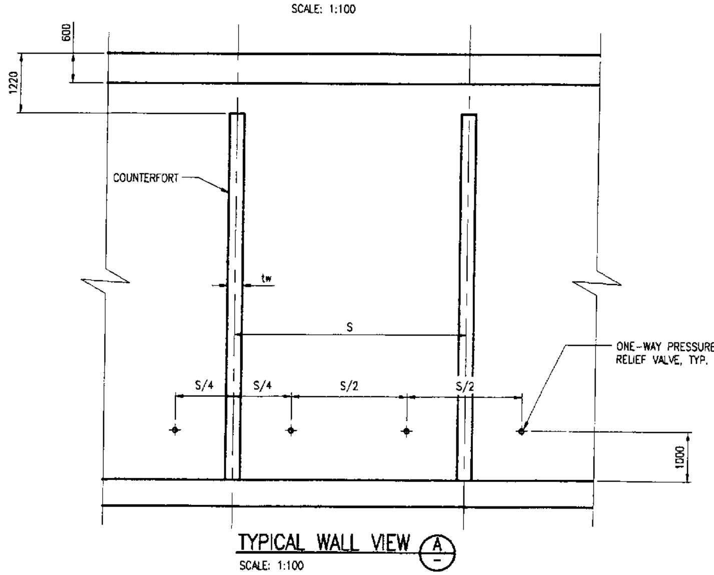
SCALE: 1:100



NOTE:
ALL UNITS IN CONDUIT SECTION ARE IN METERS.

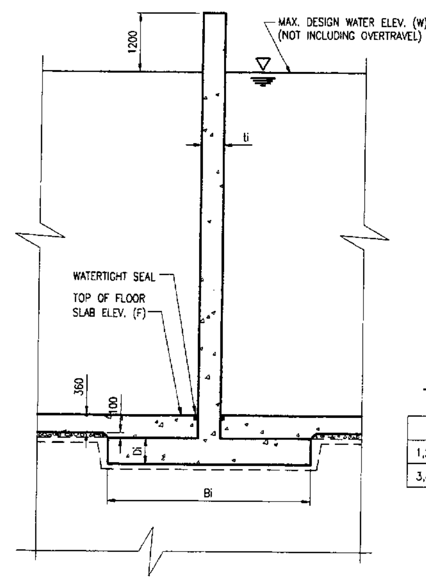
TYPICAL CONDUIT SECTION

SCALE: 1:200



TYPICAL WALL VIEW (A)

SCALE: 1:100



TYPICAL INTERIOR WALL SECTION

SCALE: 1:100

WALL ELEVATIONS AND DIMENSIONS

BASIN	W	F	H	A	B	D	t	S	tw	Bi	Di	ti
1,12	8.310	4.000	5.890	1.370	4.270	0.380	0.300	3.660	0.300	3.350	0.610	0.380
2,11	6.380	2.540	5.420	1.370	4.270	0.380	0.300	3.660	0.300	3.050	0.610	0.380
3,10	4.440	1.080	4.940	1.370	4.270	0.380	0.300	3.660	0.300	3.050	0.610	0.380
4,15	17.210	11.740	7.040	1.680	5.180	0.460	0.380	4.270	0.300	3.960	0.610	0.533
5,14	15.470	10.130	6.910	1.680	5.180	0.460	0.380	4.270	0.300	3.960	0.610	0.533
6,13	13.730	8.520	6.780	1.680	5.180	0.460	0.380	4.270	0.300	3.960	0.610	0.460
7,18	25.250	20.340	6.480	1.520	4.880	0.460	0.380	3.960	0.300	3.660	0.610	0.460
8,17	23.670	18.540	6.700	1.520	4.880	0.460	0.380	3.960	0.300	3.960	0.610	0.460
9,16	22.090	16.750	6.920	1.680	5.180	0.460	0.380	4.270	0.300	3.960	0.610	0.533

NOTE: ALL UNITS IN TABLE ARE IN METERS.

20 10 0 20 40
SCALE IN METERS

10 5 0 10 20
SCALE IN METERS

FOUNDATION TABLE

BASINS	FOUNDATION TYPE
1,2,7,8,9,11,12,14,15,16,17,18	SELECT FILL
3,4,5,6,10,13	ROCK

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

STUDY OF LOCKS WATER SAVING BASINS
OPTION 3 TYPICAL SECTIONS AT ALIGNMENT A-2

MWH / NCA ENGINEERS INC. | FEBRUARY 2002 | FIGURE V.19

c) Floor Design

The following water saving basin floor designs were considered:

- A concrete floor, with a drainage blanket, supported by a select fill or rock foundation
- A concrete floor, with a drainage blanket, supported by a drilled shaft foundation
- An unlined rock floor, with no floor drainage required

An unlined rock floor may be feasible for the basins that are founded in slightly weathered or sound rock. (It is not unusual for locks that are founded in rock to have unlined rock floors.) However, as described in the geotechnical report, there is insufficient data on the rock permeability and rock quality, at this time, to ensure that the unlined rock floor performance will meet the project requirements and that the cost of any required rock drilling and grouting will be less than the cost of a cast-in-place concrete floor. Therefore, as recommended by the geotechnical engineers, a concrete floor was assumed for this conceptual design. Evaluation of an unlined rock floor, for applicable basins, is recommended for future design phases.

For a drilled shaft foundation, the concrete floor must be thick enough to resist the full vertical load, spanning between shafts. Based on the preliminary shaft size and spacing, it was determined that a 760 mm (30-inch) thickness was required for a concrete floor on a drilled shaft foundation.

For the select fill or rock foundation, the water saving basin concrete floor thickness was sized to resist an uplift pressure of 0.006895 MPa (1 psi), which is the anticipated threshold level of the pressure relief valve. It was determined that a 360 mm (14-inch) thickness was required for a concrete floor on a select fill or rock foundation. As described in the Feature Layouts chapter, the select fill or rock foundation was selected for conceptual design. However, this foundation assumption should be reevaluated during future design phases, as the rock contours are better defined and the foundation designs are developed further.

d) Wall Design

For all basin walls, a 1.22 meter (4-foot) freeboard was provided above the design water elevation. This freeboard should be adequate to contain the water, accommodating the turbulence and wave heights anticipated in the water saving basins. The following wall types were considered for the water saving basin walls:

- Standard Cantilever Wall
- Counterfort Wall
- Braced Counterfort Wall (Interior Columns required to support the bracing)
- Drilled Shaft Cantilever Wall
- Drilled Shaft Tieback Wall
- Braced Drilled Shaft Wall (Interior Columns required to support the bracing)

For the wall heights required at the Atlantic site, the Counterfort Wall design was found to be the most cost effective of the wall types listed above.

Basins with sloped sides were also briefly evaluated. See Section C.1.e for a discussion of this alternate design concept.

(1) Option 1 Design

For the Atlantic Option 1 basin wall designs, typical wall details and tables are provided on **Figure V.17**. The basin exposed wall heights (including freeboard) range from 5.2 m to 7.1 m (17 to 23 feet), with wall thicknesses of 300 mm to 460 mm (12 to 18 inches). The footing widths vary from 4.3 m to 5.8 m (14 to 19 feet), with footing thicknesses of 380 mm to 530 mm (15 to 21 inches).

(2) Option 2 Design

For the Atlantic Option 2 basin wall designs, typical wall details and tables are provided on **Figure V.18**. The basin exposed wall heights (including freeboard) range from 5.8 m to 7.8 m (19 to 26 feet), with wall thicknesses of 300 mm to 530 mm (12 to 21 inches). The footing widths vary from 4.3 m to 6.1 m (14 to 20 feet), with footing thicknesses of 380 mm to 610 mm (15 to 24 inches).

(3) Option 3 Design

For the Atlantic Option 3 basin wall designs, typical wall details and tables are provided on **Figure V.19**. The basin exposed wall heights (including freeboard) range from 4.6 m to 6.7 m (15 to 22 feet), with wall thicknesses of 300 mm to 380 mm (12 to 15 inches). The footing widths vary from 4.3 m to 5.2 m (14 to 17 feet), with footing thicknesses of 380 mm to 460 mm (15 to 18 inches).

e) Alternate Basin Design

Basins with sloped sides were briefly evaluated. In this case, instead of bounding the basins with vertical retaining walls, the basins would have sloped sides with a concrete lining on top. As this slope would be permanent, it was assumed to be 2H:1V or more gentle than that. The minimum basin width was assumed to be between 50 meters (164 feet) and 61 meters (200 feet). The exact slope and the minimum basin width would need to be determined by the geotechnical engineers and the hydraulic engineers, if this design was selected for further study. The feasibility and economic benefits of this alternate design are fairly sensitive to these assumptions.

In order to prevent high external hydrostatic pressures, a drainage blanket would be required under the concrete lining on the sloped basin sides as well as under the floor. Again, for redundancy, one-way pressure relief valves would be provided in the concrete lining on the basin floor and sides. To resist an external hydrostatic pressure equal to the anticipated threshold level of the pressure relief valve, the concrete side lining would need to be at least as thick as the concrete floor.

In general, the advantages of the concrete-lined sloped basin design are lower cost and ease of construction. The disadvantages are a bigger footprint and potential settlement or undermining problems. For the water saving basins associated with a new third lane at the Atlantic site Alignment A-2, a bigger footprint would require a very significant change in the alignment, shifting the new lanes much further away from the existing lanes. This revised alignment would make centralized control of lock operations very difficult. For the water saving basins associated with a new fourth lane, a bigger footprint would be feasible but would affect a larger number of existing roads and structures. Finally, the sloped-sided basins would introduce complexity into the hydraulic analysis in that the planform area of the basin would change with water depth. This complication was not considered in this concept-level analysis.

The vertical retaining wall designs, presented in this report, provide a feasible and conservative design. It is anticipated that this design will yield a good conceptual cost estimate and will capture the relative costs of the considered basin layout options. However, depending on the associated geotechnical and hydraulic requirements for basins with permanently sloped sides, a basin design with concrete-lined slopes could potentially reduce total basin construction costs anywhere between 0 and 10 percent. Further evaluation of this alternate design is recommended for future design phases.

f) Conduit Design

Rectangular conduits connect the water saving basins to the lock fill/empty system. The conduits slope downward, towards the lock chamber. At the Atlantic site, the conduit grades are approximately 13 percent, 10 percent, and 12 percent for Options 1, 2, and 3, respectively.

The conduits were conceptually designed as rectangular cast-in-place concrete box structures. Typical conduit sections are shown on **Figures V.17-19**. A rectangular shape is required at the conduit valve and bulkhead locations. A circular conduit would be acceptable elsewhere if smooth transitions were provided between the rectangular and circular sections. However, circular conduits would only be cost effective if a precast construction method was specified, and precast construction is not anticipated to be practical for the required conduit sizes.

The governing design load for the rectangular cast-in-place concrete conduits is the overburden dead load. It is assumed that the conduits will be constructed in vertical trenches, formed by braced or anchored temporary walls. Due to this construction method, it is assumed that soil arching will be negligible for the dead load condition.

2. Pacific Side

a) Drainage Considerations

The water saving basins must be designed as watertight containers. However, the water saving basins are frequently empty, as their contents are used to fill the lock chambers. If proper drainage is not provided outside the basins, the basin floors and walls must be designed to withstand the high external hydrostatic pressure as a normal load condition. In order to minimize the basin construction cost, a permanent basin drainage system is required. The geotechnical report provided the following permanent drainage recommendations:

- Provide a drainage blanket under the basin floors
- Provide a longitudinal drain at the foundation level behind the basin walls
- Leave the construction cut-off walls permanently in place, in order to significantly reduce the permanent drainage requirements

The geotechnical engineers anticipate that gravity drainage will be practical, but the drainage system will need to be further developed during future design phases, including a full evaluation of the feasibility of gravity drainage.

Drain cleanouts should be provided in order to allow removal of debris buildup. However, due to maintenance concerns and the consequences of blocked drains, a back up measure is recommended. To provide this redundancy, one-way pressure relief valves (flap or tide-flex valves) are included in the basin floors and walls. In this way, if the underdrains are plugged and the basins are emptied, the external water will drain into the basin if the external hydrostatic pressure exceeds the flap valve threshold value. This threshold value is typically on the order of 0.007 MPa (1 psi).

b) Basin Design Concept

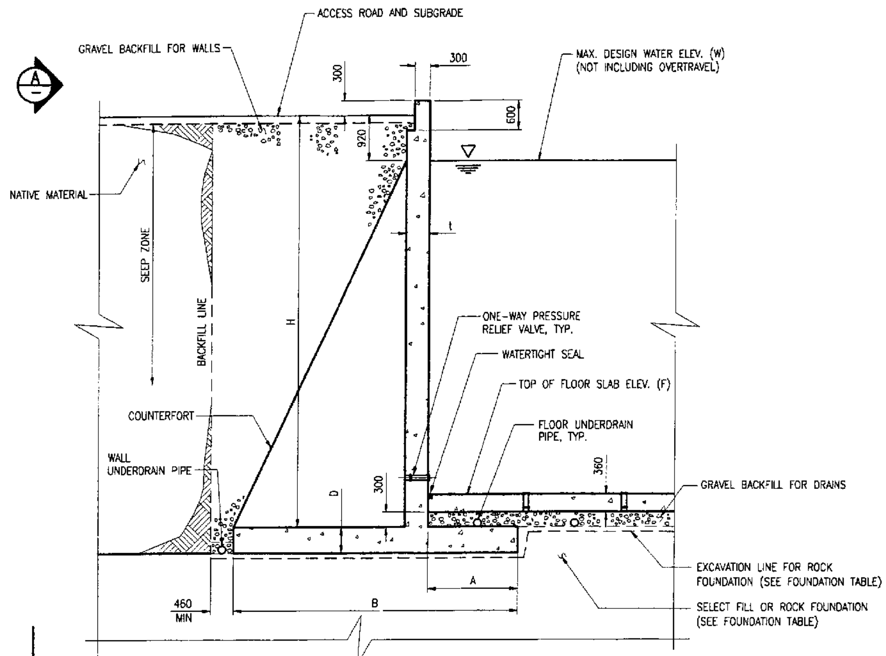
The general concept for the water saving basin design is a structure with independent floor and wall systems. The basins were assumed to have vertical reinforced concrete retaining walls and a separate reinforced concrete floor, with a watertight seal at the interface. A typical section of the wall and the interface between the wall and the floor is shown on **Figures V.20-22**. By keeping the wall and floor separate, the effects of settlement and thermal expansion are minimized. This design should also simplify reinforcing details and construction.

Basins with sloped sides were also briefly evaluated. See **Section C.2.e** for a discussion of this alternate design concept.

Seismic ground accelerations and loads were not determined for this conceptual design stage. Seismic analysis and design will need to be addressed in future design phases.

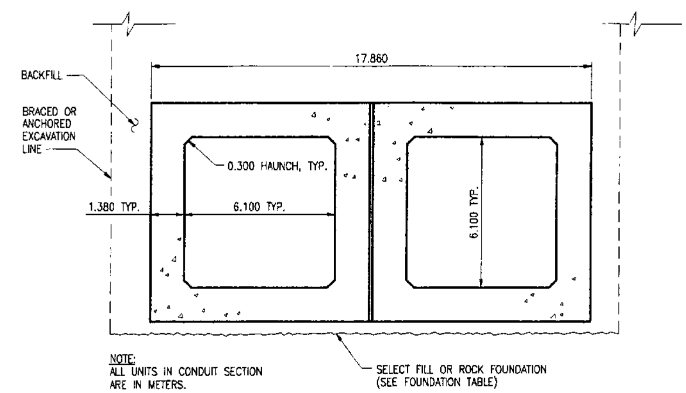
NOTES:

1. ALL DIMENSIONS IN CALLOUTS/NOTES ARE IN MILLIMETERS, UNLESS OTHERWISE NOTED.
2. INTERIOR WALLS REQUIRE OPENINGS WITH REMOVABLE GATES FOR WATER BALANCE AND SURGES.



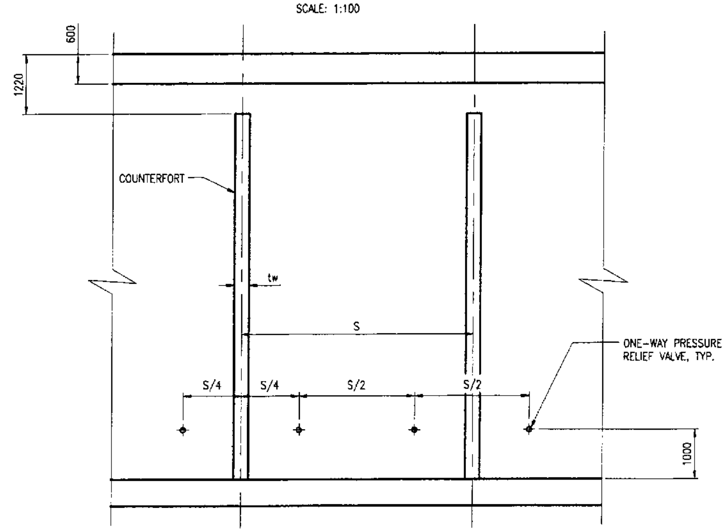
TYPICAL WALL SECTION

SCALE: 1:100



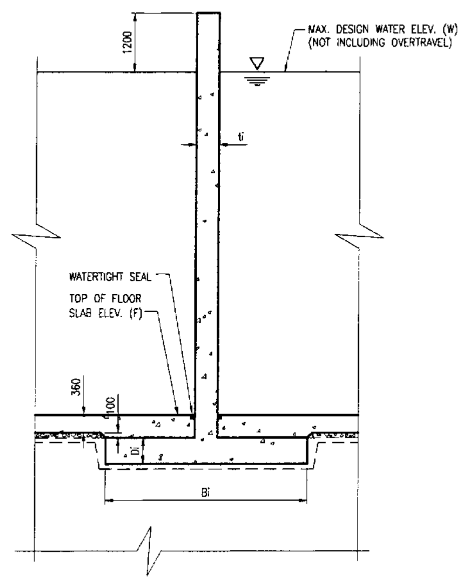
TYPICAL CONDUIT SECTION

SCALE: 1:200



TYPICAL WALL VIEW (A)

SCALE: 1:100



TYPICAL INTERIOR WALL SECTION

SCALE: 1:100

WALL ELEVATIONS AND DIMENSIONS

BASIN	W	F	H	A	B	D	t	S	tw	Bi	Di	ti
1	25.080	18.880	7.780	1.830	5.790	0.530	0.460	4.570	0.300	4.570	0.760	0.690
2	23.340	16.350	8.560	1.980	6.100	0.610	0.530	4.880	0.300	5.182	0.760	0.840
3	17.930	9.300	10.210	2.440	7.620	0.840	0.760	6.100	0.530	5.790	1.070	1.140
4	16.010	7.030	10.550	2.590	7.920	0.910	0.840	6.400	0.610	6.400	1.070	1.130
5	10.020	0.560	10.940	2.590	7.920	0.910	0.840	6.400	0.610	6.400	1.070	1.370
6	7.880	-1.390	10.850	2.590	7.920	0.910	0.840	6.400	0.610	6.400	1.070	1.370

NOTE: ALL UNITS IN TABLE ARE IN METERS.

20 10 0 20 40
SCALE IN METERS

10 5 0 10 20
SCALE IN METERS

FOUNDATION TABLE

BASIN	FOUNDATION TYPE
1	SELECT FILL
2	SELECT FILL
3	SELECT FILL
4	ROCK
5	ROCK
6	ROCK

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

STUDY OF LOCKS WATER SAVING BASINS
OPTION 1 TYPICAL SECTIONS AT ALIGNMENT P-1

MWH ENGINEERS INC. | FEBRUARY 2002 | FIGURE V.20

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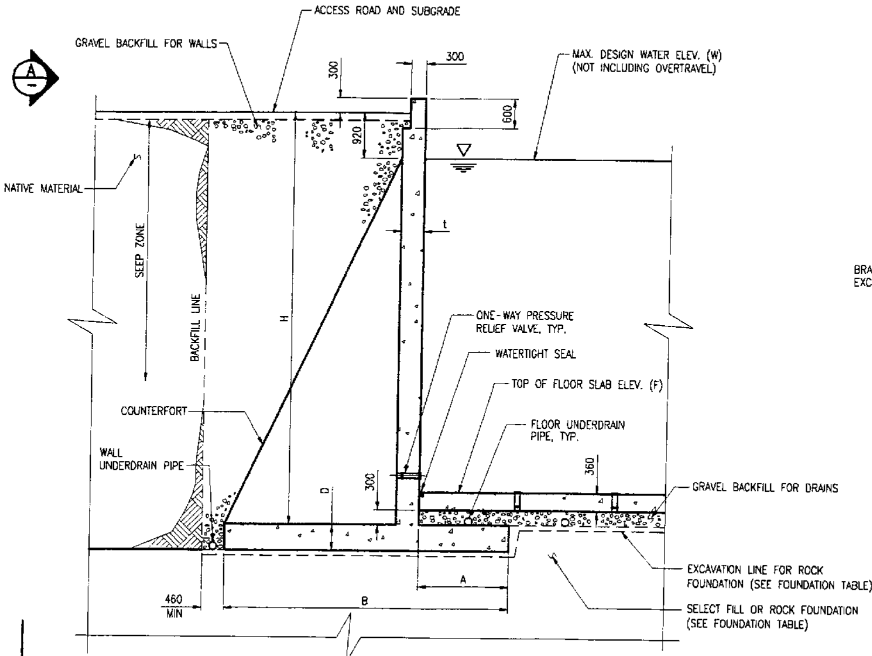
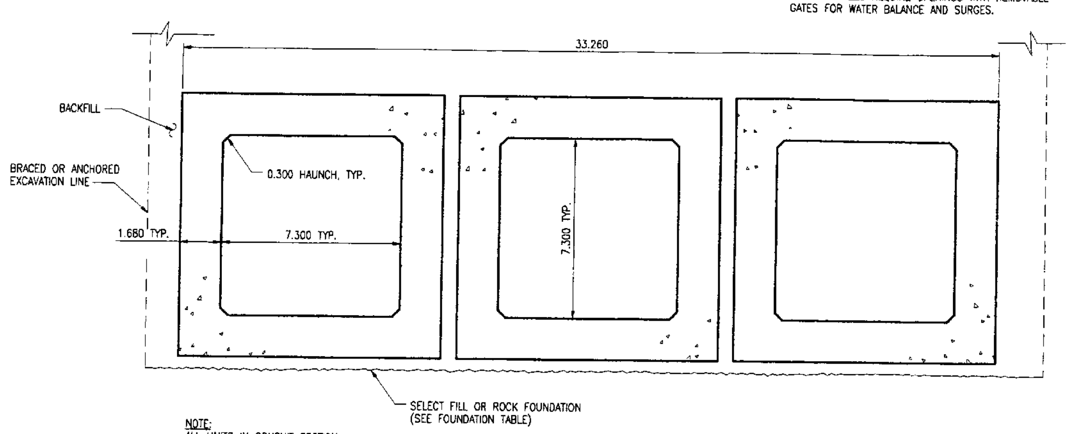
NOTES:

1. ALL DIMENSIONS IN CALLOUTS/NOTES ARE IN MILLIMETERS, UNLESS OTHERWISE NOTED.
2. INTERIOR WALLS REQUIRE OPENINGS WITH REMOVABLE GATES FOR WATER BALANCE AND SURGES.



TYPICAL WALL SECTION

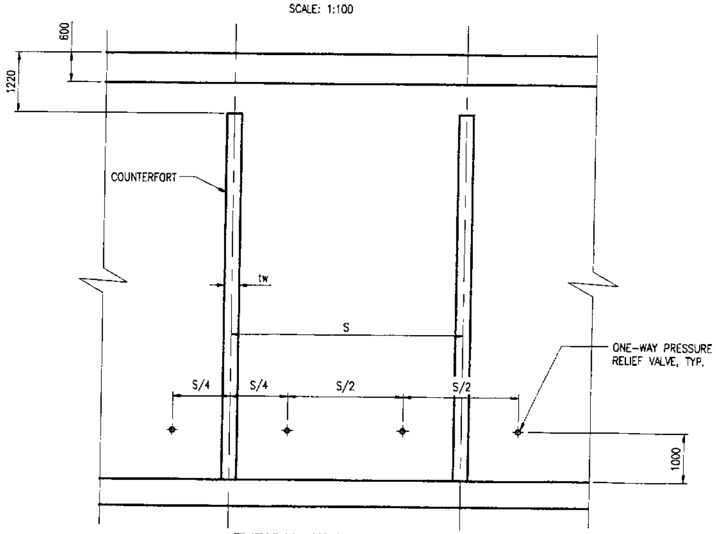
SCALE: 1:100



NOTE: ALL UNITS IN CONDUIT SECTION ARE IN METERS.

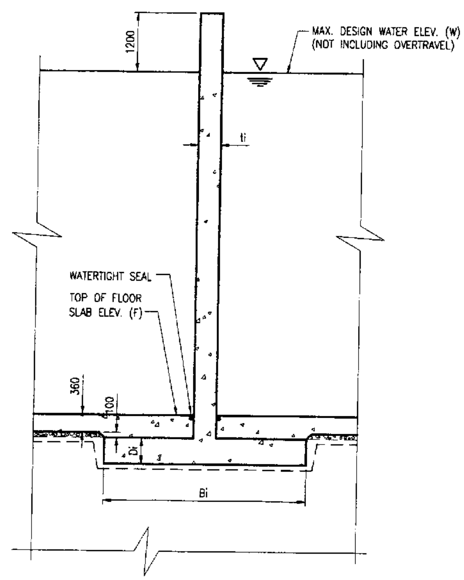
TYPICAL CONDUIT SECTION

SCALE: 1:200



TYPICAL WALL VIEW (A)

SCALE: 1:100



TYPICAL INTERIOR WALL SECTION

SCALE: 1:100

WALL ELEVATIONS AND DIMENSIONS

BASIN	W	F	H	A	B	D	t	S	tw	Bi	Di	ti
1	24.740	17.890	8.430	1.980	6.100	0.610	0.530	4.880	0.300	5.180	0.760	0.840
2	22.660	14.870	9.360	2.290	7.010	0.760	0.690	5.790	0.460	5.490	0.910	0.990
3	20.570	11.680	10.470	2.590	7.920	0.910	0.840	6.400	0.610	6.400	1.070	1.300
4	13.840	3.920	11.500	2.740	8.530	0.990	0.910	6.710	0.610	6.710	1.070	1.600
5	11.280	1.470	11.390	2.740	8.530	0.990	0.910	6.710	0.610	6.710	1.070	1.520
6	8.720	-0.990	11.290	2.740	8.530	0.990	0.910	6.710	0.610	6.710	1.070	1.520

NOTE: ALL UNITS IN TABLE ARE IN METERS.

FOUNDATION TABLE

BASIN	FOUNDATION TYPE
1	SELECT FILL
2	SELECT FILL
3	SELECT FILL
4	ROCK
5	ROCK
6	ROCK

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

STUDY OF LOCKS WATER SAVING BASINS
OPTION 2 TYPICAL SECTIONS AT ALIGNMENT P-1

NCA ENGINEERS INC. | FEBRUARY 2002 | FIGURE V.21

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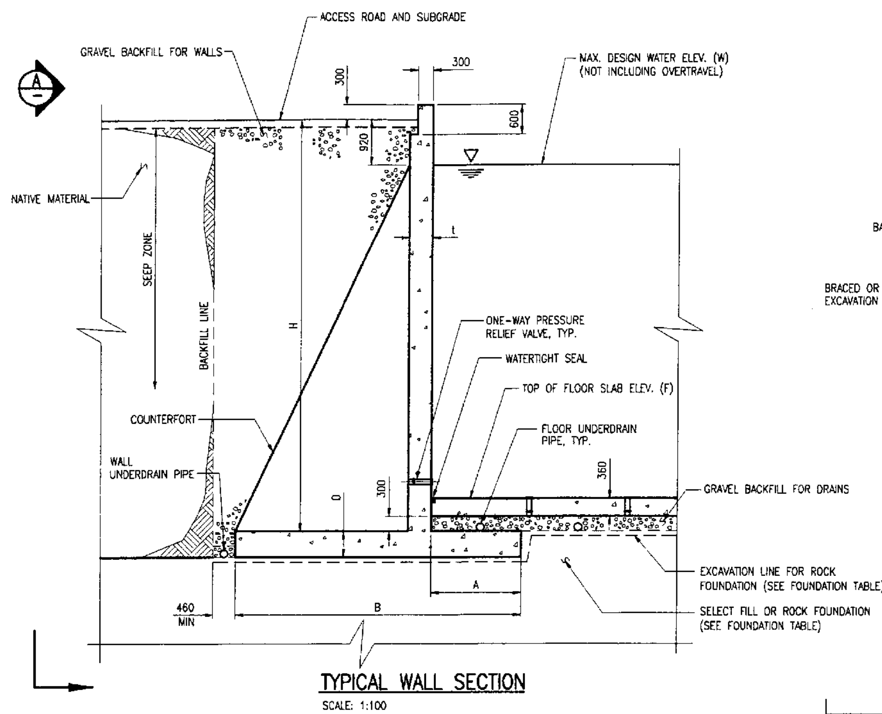
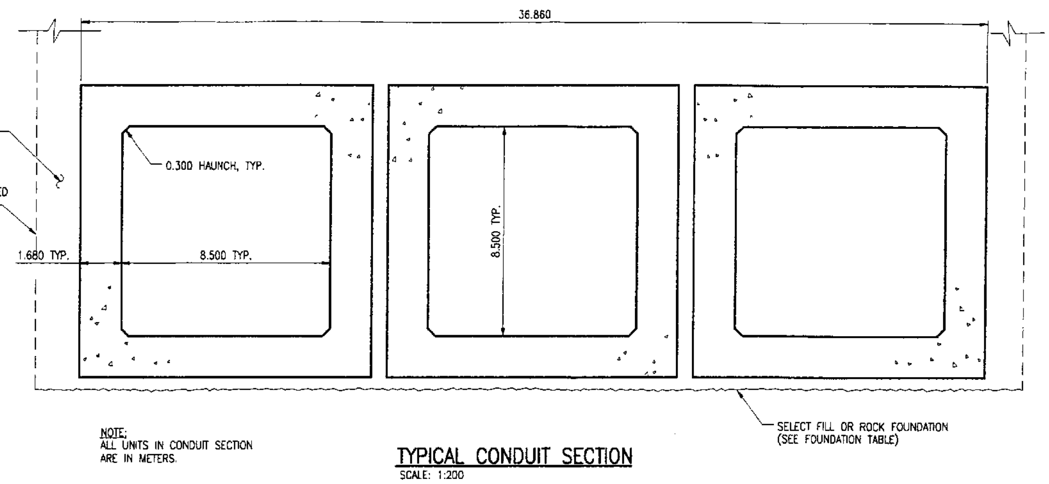


TYPICAL WALL SECTION
SCALE: 1:100

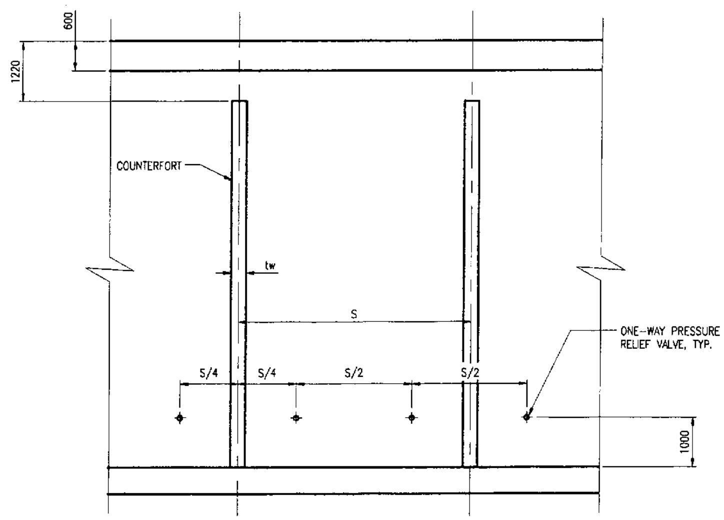


NOTE:
ALL UNITS IN CONDUIT SECTION ARE IN METERS.

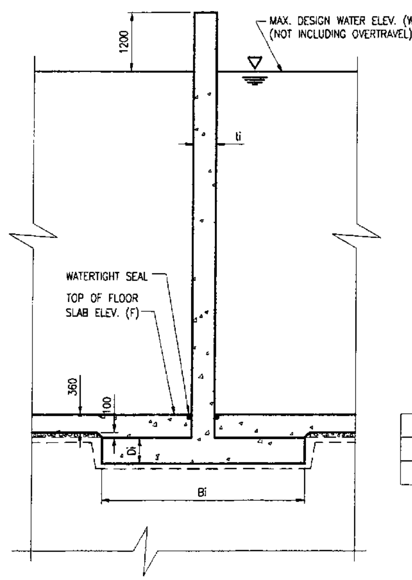
TYPICAL CONDUIT SECTION
SCALE: 1:200

NOTES:

1. ALL DIMENSIONS IN CALLOUTS/NOTES ARE IN MILLIMETERS, UNLESS OTHERWISE NOTED.
2. INTERIOR WALLS REQUIRE OPENINGS WITH REMOVABLE GATES FOR WATER BALANCE AND SURGES.



TYPICAL WALL VIEW (A)
SCALE: 1:100



TYPICAL INTERIOR WALL SECTION
SCALE: 1:100

WALL ELEVATIONS AND DIMENSIONS

BASIN	W	F	H	A	B	D	t	S	tw	Bi	Di	ti
1,12	25.430	19.890	7.120	1.680	5.180	0.460	0.380	4.270	0.300	3.960	0.610	0.530
2,11	24.040	17.870	7.740	1.830	5.790	0.530	0.460	4.570	0.300	4.570	0.760	0.610
3,10	22.640	15.850	8.370	1.980	6.100	0.610	0.530	4.880	0.300	4.880	0.760	0.760
4,15	18.320	10.200	9.690	2.290	7.010	0.760	0.690	5.790	0.460	5.790	0.910	1.070
5,14	16.780	8.390	9.960	2.440	7.620	0.840	0.760	6.100	0.530	5.790	1.070	1.140
6,13	15.240	6.580	10.240	2.440	7.620	0.840	0.760	6.100	0.530	6.100	1.070	1.220
7,18	10.450	1.480	10.550	2.590	7.920	0.910	0.840	6.400	0.610	6.400	1.070	1.300
8,17	8.740	−0.160	10.470	2.590	7.920	0.910	0.840	6.400	0.610	6.400	1.070	1.300
9,16	7.030	−1.800	10.400	2.440	7.620	0.840	0.760	6.100	0.530	6.100	1.070	1.220

NOTE: ALL UNITS IN TABLE ARE IN METERS.

20 10 0 20 40
SCALE IN METERS

10 5 0 10 20
SCALE IN METERS

FOUNDATION TABLE

BASINS	FOUNDATION TYPE
1,2,3,7,8,9,10,11,12,15	SELECT FILL
4,5,6,13,14,16,17,18	ROCK

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

STUDY OF LOCKS WATER SAVING BASINS
OPTION 3 TYPICAL SECTIONS AT ALIGNMENT P-1

NCA ENGINEERS INC. | FEBRUARY 2002 | FIGURE V.22

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c) Floor Design

The following water saving basin floor designs were considered:

- A concrete floor, with a drainage blanket, supported by a select fill or rock foundation
- A concrete floor, with a drainage blanket, supported by a drilled shaft foundation
- An unlined rock floor, with no floor drainage required

An unlined rock floor may be feasible for the basins that are founded in slightly weathered or sound rock. (It is not unusual for locks that are founded in rock to have unlined rock floors.) However, as described in the geotechnical report, there is insufficient data on the rock permeability and rock quality, at this time, to ensure that the unlined rock floor performance will meet the project requirements and that the cost of any required rock drilling and grouting will be less than the cost of a cast-in-place concrete floor. Therefore, as recommended by the geotechnical engineers, a concrete floor was assumed for this conceptual design. Evaluation of an unlined rock floor, for applicable basins, is recommended for future design phases.

For a drilled shaft foundation, the concrete floor must be thick enough to resist the full vertical load, spanning between shafts. Based on the preliminary shaft size and spacing, it was determined that a 760 mm thickness was required for a concrete floor on a drilled shaft foundation.

For the select fill or rock foundation, the water saving basin concrete floor thickness was sized to resist an uplift pressure of 0.006895 MPa (1 psi), which is the anticipated threshold level of the pressure relief valve. It was determined that a 360 mm thickness was required for a concrete floor on a select fill or rock foundation. As described in the Feature Layouts Report, the select fill or rock foundation was selected for conceptual design. However, this foundation assumption should be reevaluated during future design phases, as the rock contours are better defined and the foundation designs are developed further.

d) Wall Design

For all basin walls, a 1.22 meter (4-foot) freeboard was provided above the design water elevation. This freeboard should be adequate to contain the water, accommodating the turbulence and wave heights anticipated in the water saving basins. The following wall types were considered for the water saving basin walls:

- Standard Cantilever Wall
- Counterfort Wall
- Braced Counterfort Wall (Interior Columns required to support the bracing)
- Drilled Shaft Cantilever Wall
- Drilled Shaft Tieback Wall
- Braced Drilled Shaft Wall (Interior Columns required to support the bracing)

The cantilever and counterfort walls are practical for the recommended assumptions of open cut excavation and select fill or rock foundations. For the wall heights required at the Pacific site, the Counterfort Wall design was found to be the most cost effective of the the three wall types that are consistent with the excavation and foundation assumptions.

A very small percentage of concrete may be saved with the use of the Braced Counterfort Wall, but additional forming costs would offset any potential savings. Additionally, slightly larger basins would be required to compensate for the water volume displaced by the columns that are needed to support the bracing.

In future design phases, a drilled shaft foundation may be found to be more cost effective than a select fill foundation. In this case, a braced drilled shaft wall should also be considered at the Pacific site. A braced drilled shaft wall design could potentially reduce concrete quantities by 5 to 10 percent and excavation quantities would be reduced as well, as open cut excavation would only be required to the top of the wall elevation. Within the basin, top-down construction methods could be utilized to minimize excavation requirements.

Basins with sloped sides were also briefly evaluated. See **Section C.2.e** for a discussion of this alternate design concept.

(1) Option 1 Design

For the Pacific Option 1 basin wall designs, typical wall details and tables are provided on **Figure V.20**. The basin exposed wall heights (including freeboard) range from 7.4 m to 10.6 m (24 to 35 feet), with wall thicknesses of 460 mm to 840 mm (18 to 33 inches). The footing widths vary from 5.8 m to 8.0 m (19 to 26 feet), with footing thicknesses of 530 mm to 910 mm (21 to 36 inches).

(2) Option 2 Design

For the Pacific Option 2 basin wall designs, typical wall details and tables are provided on **Figure V.21**. The basin exposed wall heights (including freeboard) range from 8.1 m to 11.1 m (27 to 36 feet), with wall thicknesses of 530 mm to 910 mm (21 to 36 inches). The footing widths vary from 6.1 m to 8.6 m (20 to 28 feet), with footing thicknesses of 610 mm to 990 mm (24 to 39 inches).

(3) Option 3 Design

For the Pacific Option 3 basin wall designs, typical wall details and tables are provided on **Figure V.22**. The basin exposed wall heights (including freeboard) range from 6.8 m to 10.2 m (22 to 33 feet), with wall thicknesses of 380 mm to 840 mm (15 to 33 inches). The footing widths vary from 5.2 m to 8.0 m (17 to 26 feet), with footing thicknesses of 460 mm to 910 mm (18 to 36 inches).

Where the basin walls are founded on the La Boca Formation, the sliding friction is significantly reduced, according to the geotechnical reports. Due to the reduced friction force, a shear key or inclined rock anchors may be required in these regions in order to provide an adequate factor of safety against sliding.

e) Alternate Basin Design

Basins with sloped sides were briefly evaluated. In this case, instead of bounding the basins with vertical retaining walls, the basins would have sloped sides with a concrete lining on top. As this slope would be permanent, it was assumed to be 2H:1V or more gentle than that. The minimum basin width was assumed to be between 50 meters (164 feet) and 61 meters (200 feet). The exact slope and the minimum basin width would need to be determined by the geotechnical engineers and the hydraulic engineers, if this design was selected for further study. The feasibility and economic benefits of this alternate design are fairly sensitive to these assumptions.

In order to prevent high external hydrostatic pressures, a drainage blanket would be required under the concrete lining on the sloped basin sides as well as under the floor. Again, for redundancy, one-way pressure relief valves would be provided in the concrete lining on the basin floor and sides. To resist an external hydrostatic pressure equal to the anticipated threshold level of the pressure relief valve, the concrete side lining would need to be at least as thick as the concrete floor.

In general, the advantages of the concrete-lined sloped basin design are lower cost and ease of construction. The disadvantages are a bigger footprint and potential settlement or undermining problems. Finally, the sloped-sided basins would introduce complexity into the hydraulic analysis in that the planform area of the basin would change with water depth. This complication was not considered in this concept-level analysis. Other than environmental impact, there are no significant problems associated with a bigger footprint at the Pacific site.

The vertical retaining wall designs, presented in this report, provide a feasible and conservative design. It is anticipated that this design will yield a good conceptual cost estimate and will capture the relative costs of the considered basin layout options.

However, depending on the associated geotechnical and hydraulic requirements for basins with permanently sloped sides, a basin design with concrete-lined slopes could potentially reduce total basin construction costs anywhere between 5 and 20 percent. While these savings could be very significant at the Pacific site for the designs described in this report, at the time of this writing new direction is being formulated for basin designs that are anticipated to have significantly shorter walls. For designs with shorter walls, comparable to the existing designs for the Atlantic site, the potential construction cost savings are likely to be reduced to the values anticipated for the Atlantic site (i.e., 0 to 10 percent). Further evaluation of this alternate design is recommended for future design phases.

f) Conduit Design

Rectangular conduits connect the water saving basins to the lock fill/empty system. The conduits slope downward, towards the lock chamber. At the Pacific site, the conduit grades range from 12.3 percent to 13.5 percent for the Option 1 design. For Option 2, the conduit grade is approximately 10.8 percent at the upper lift chamber and is approximately 9.7 percent at the lower lift chamber. For Option 3, the conduit grades range from approximately 11.8 percent to approximately 12.9 percent.

The conduits were conceptually designed as rectangular cast-in-place concrete box structures. Typical conduit sections are shown on **Figures V.20-22**. A rectangular shape is required at the conduit valve and bulkhead locations. A circular conduit would be acceptable elsewhere if smooth transitions were provided between the rectangular and circular sections. However, circular conduits would only be cost effective if a precast construction method was specified, and precast construction is not anticipated to be practical for the required conduit sizes.

The governing design load for the rectangular cast-in-place concrete conduits is the overburden dead load. It is assumed that the conduits will be constructed in vertical trenches, formed by braced or anchored temporary walls. Due to this construction method, it is assumed that soil arching will be negligible for the dead load condition.

D. Mechanical Features Design

1. General Description

Valves control the flow of water though the lock and water saving basins F/E systems. Based on the finalized conduit sizes selected by ACP, it is expected that the conduits will need to be bifurcated in order to accommodate smaller, more manageable valves, to save costs, and to achieve greater reliability through redundancy. Therefore, the valve recess to each conduit will house two main control valves and four auxiliary valves. The control valves must have fast opening/closing times to meet ACP's operational requirements for overall cycle times. (Refer to **Figure V.23** and **Figure V.24**).

The auxiliary valve slots are located upstream and downstream of each control valve. Each slot may be permanently equipped with operating valves, or may be served with non-operating closure bulkheads that are used only in emergencies or when maintenance is needed. For the purposes of this study, closure bulkheads will be assumed rather than auxiliary valves. Since closure bulkheads would be used infrequently, the opening/closing times can be much longer than 1 minute, hence requiring a much less complicated (and less expensive) operator. The closure bulkheads would be the same type and size as the control valves.

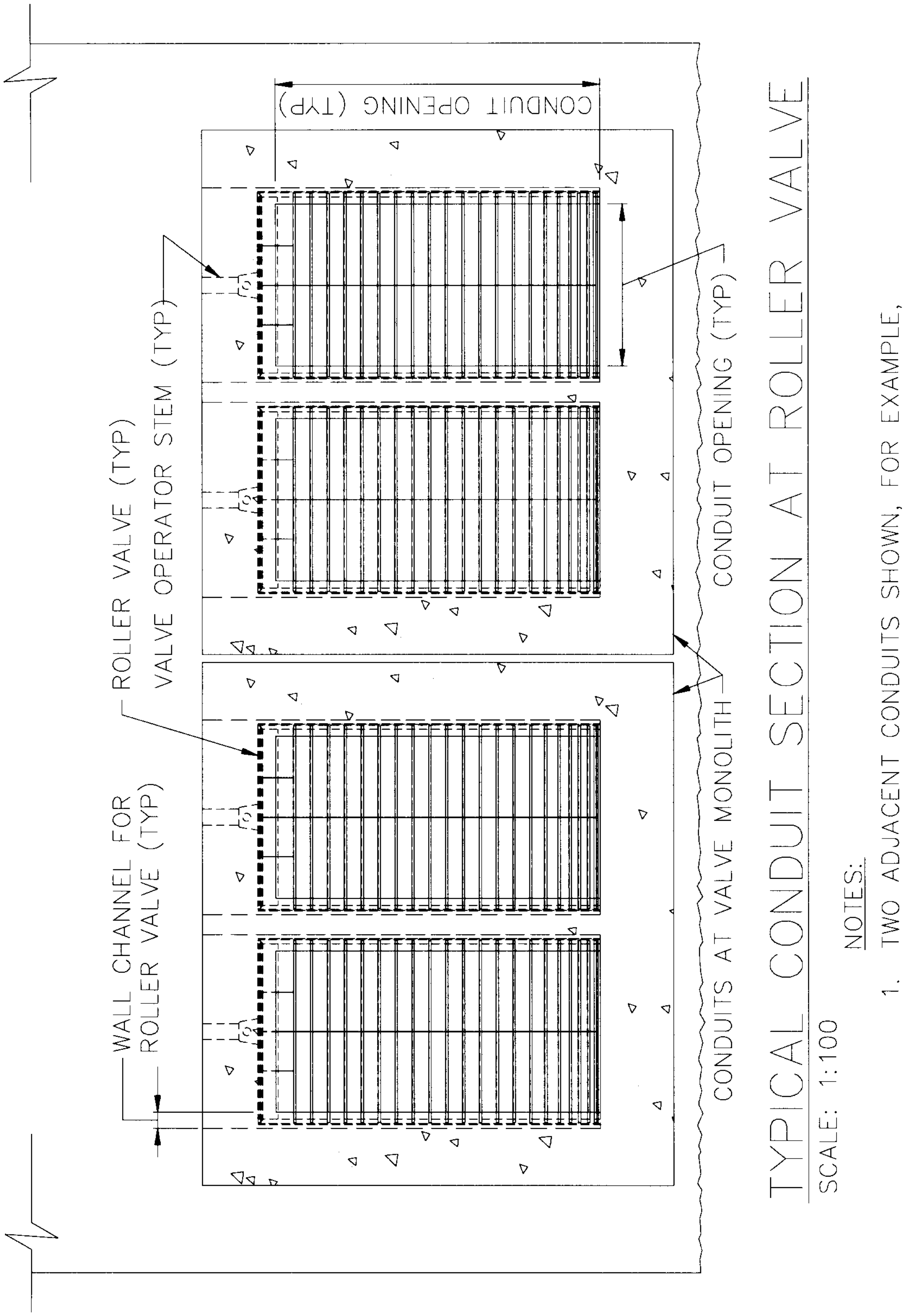

Figure V.23 - Typical Conduit Section

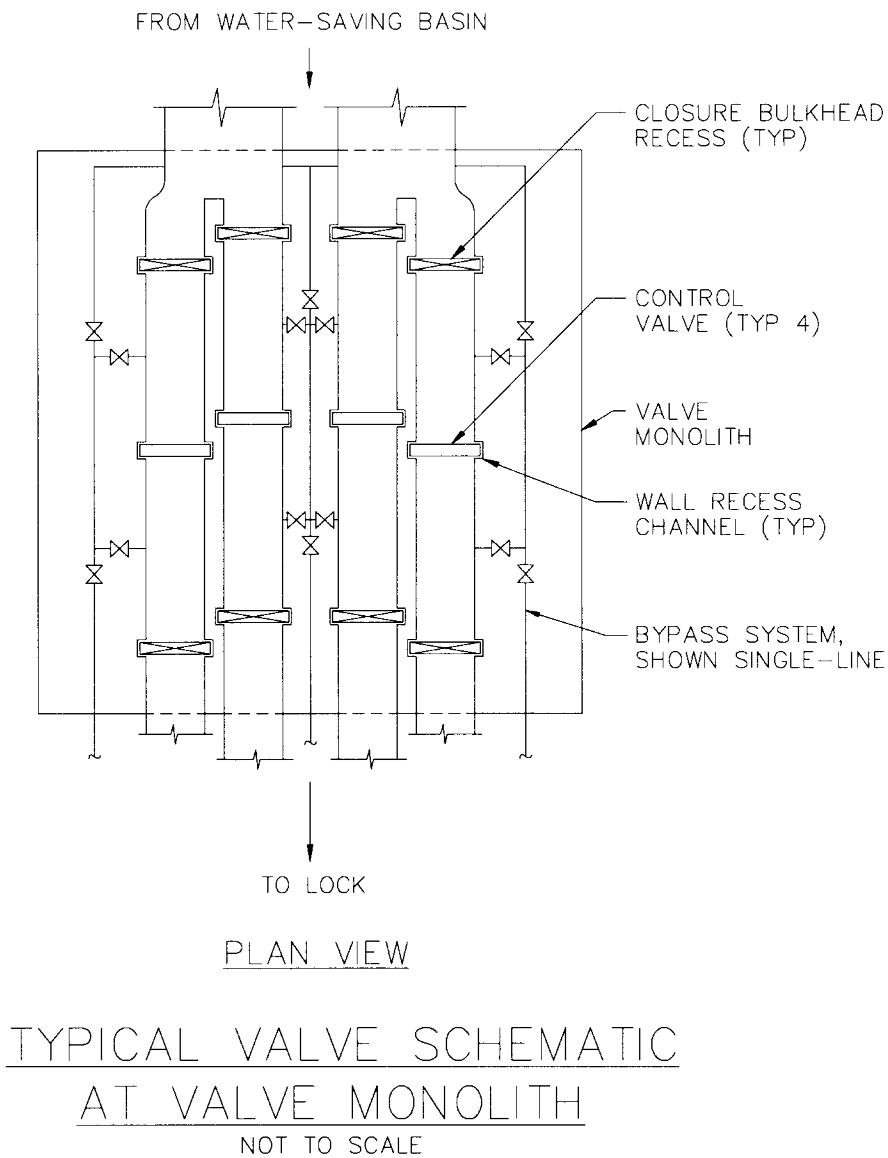

Figure V.24 – Control and Closure Bulkhead Recess Layout

The actuators and valve access shaft openings shall be housed within a building. The valves will be grouped together as much as practical to reduce the number of buildings. Refer to **Figure IV.41** for an example of locations and again to sketch **Figure V.24** for a basic arrangement of the valves.

2. Valve General Criteria

The main conduit is split vertically, becoming bifurcated at the valve monolith. The valves are located in each of the split passageways. The outer walls flare out to compensate for the vertical separation wall. This reduces the inside dimensions of the split conduit at the valve so that the width is half of the main conduit, but the height is the same. Reducing the width provides greater rigidity and serves to make repeated, rapid operations more feasible. The height of the conduits at the valve range from 6.1 meters (20 feet) to 8.5 meters (28 feet) with a corresponding width from 3.0 meters (10 feet) to 4.3 meters (14 feet) at the split valve section. The valve support and sealing structures are not to protrude into the conduit pathway to minimize local turbulence.

The travel time of the control valves is relatively fast for both opening and closing: approximately one minute. Travel time is the time the valve takes to go in one direction: from open to closed or closed to open. The closure bulkheads will be used primarily for maintenance and emergency backup control, in case the primary control valve fails. The operation time of the closure bulkheads is longer, as they could be operated using a winch system or a crane, via cable. The valves could be ballasted so the valve weight will allow the valve to travel down to the closed position.

Operating hydraulic head differential of the conduit valves is as much as about 20 meters (65 feet). The head differential for culvert valves is about 30 meters (100 feet). Both control valves and closure bulkheads would be designed for a maximum static differential head that varies with the layout options: from 33.5 meters (110 feet) to 43 meters (140 feet). This head differential would only occur during maintenance operations. Bypass valves would be installed between upstream and downstream sides of the gates to raise the water level on the low head side of the gate (refer to **Figure V.24**). These bypass valves can be manually or motor operated and could be conventional valves for water service. The control valve operators could then be activated once the water level on each side of the valve is equalized.

The maximum velocity of water at the valve occurs shortly after the valve begins to open against maximum static head. As the valve opens, the water transfer starts and continues to completion and the head differential minimizes. The valve closes when the water differential is equalizing. The maximum velocity of the water through the main and split conduits and culverts is expected to be about 4.5 meters/sec (15 feet/sec).

The valves are to be designed for bi-direction flow in the conduit, as the water is transferred from the lock to the water saving basin and vice versa.

A control valve is expected to have about thirty (30) operation cycles per day (~15 lockages*2 cycles (1-fill, 1-empty), with a cycle being an up and down travel of the valve.

3. Valve Type and Design

Many designs were investigated for this application. The best design for the control gates and closure bulkheads in this application appears to be a roller vertical-lift type. A roller valve is a vertical lift type that contacts the support structure with wheels or bearings. The support structure is at both sides of the gate and provides guided vertical slots. Rollers or bearings are attached to the valve at the sides. These rollers or bearings contact a running surface in the conduit structure slot. Refer to **Figures V.25 and V.26** for possible options. The final design will be dependent upon valve manufacturers and project requirements.

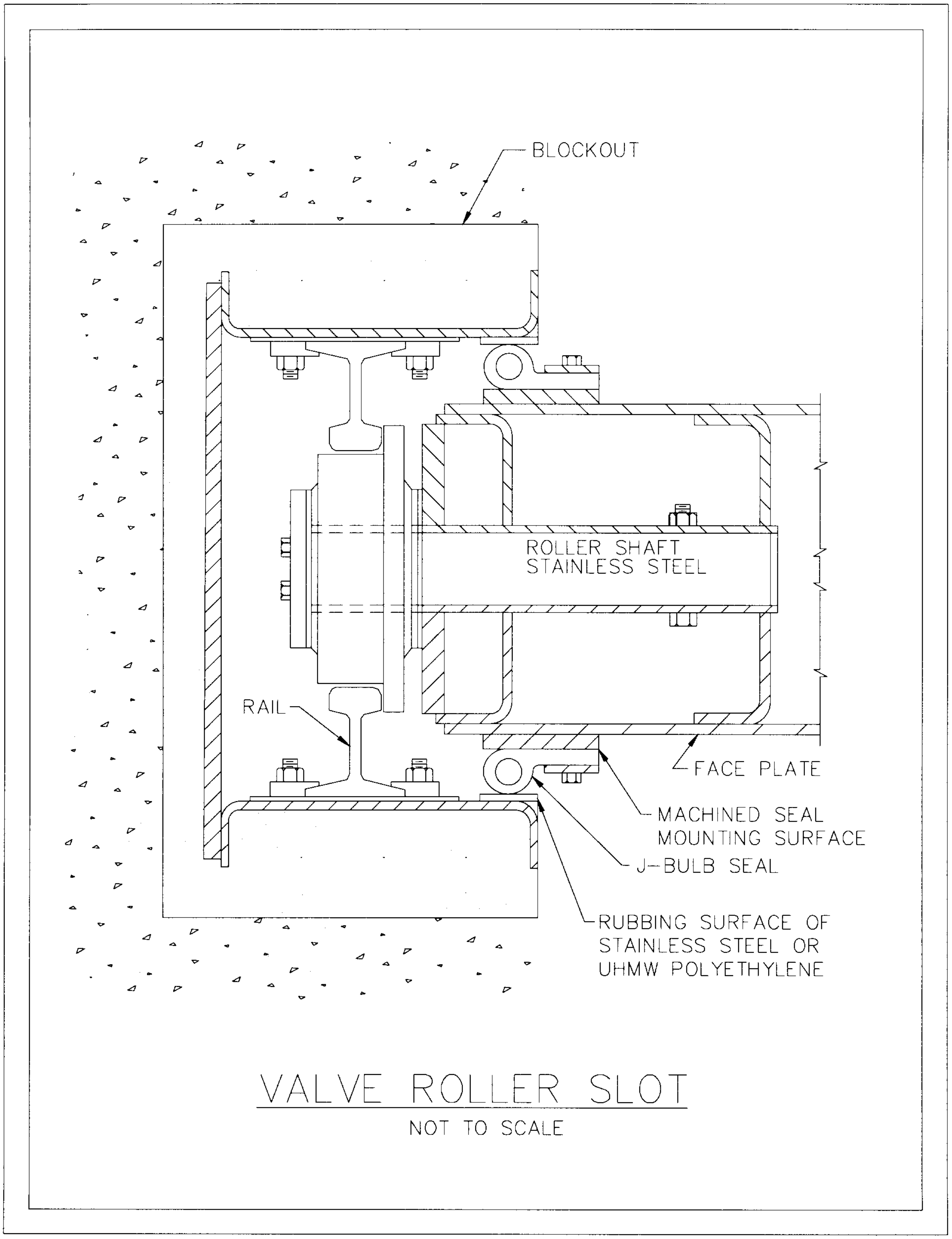

Figure V.25 - Fabricated Roller Slot with Double Rollers

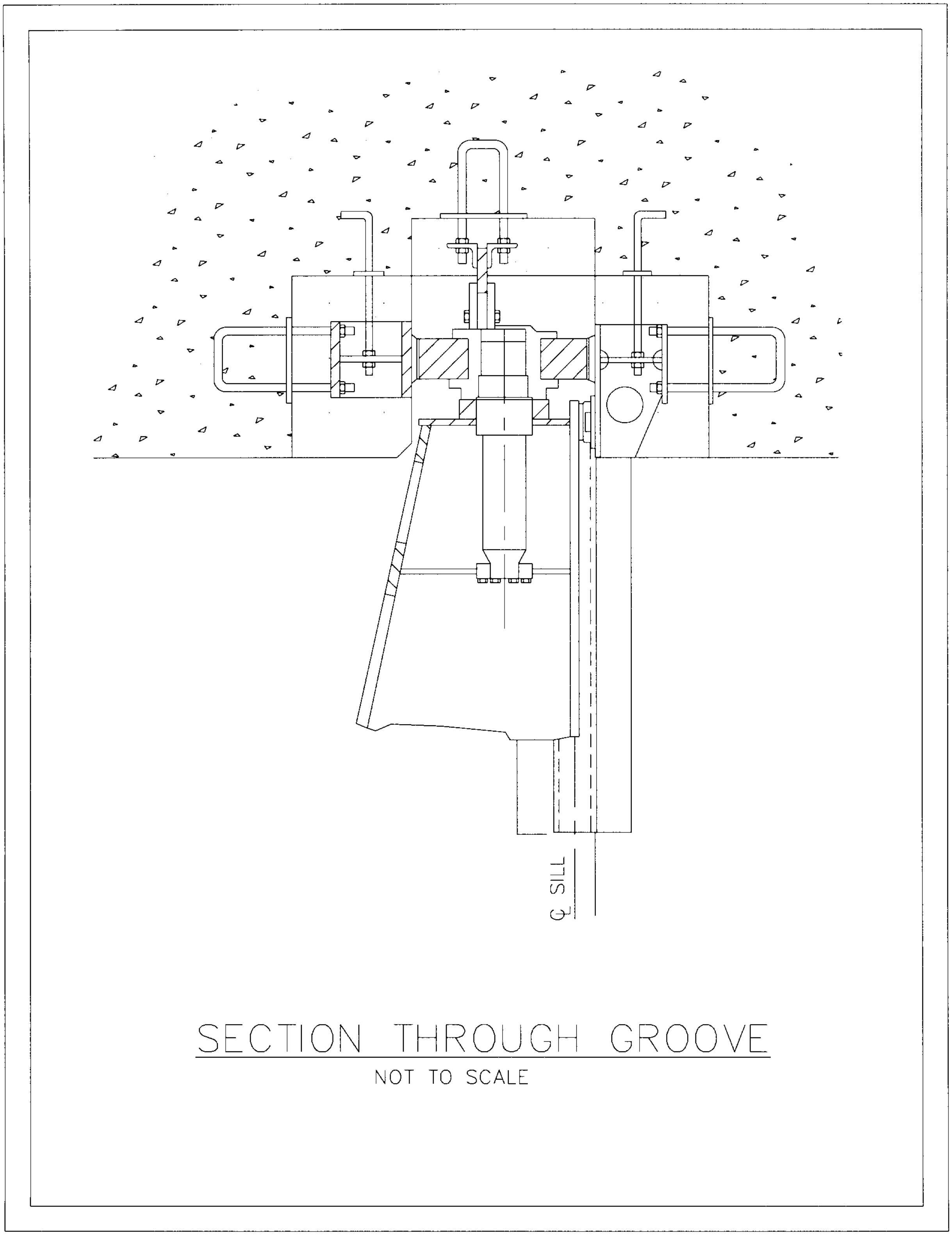

Figure V.26 - Section Through Groove

As the valves are to be designed for bi-direction flow in the conduit, the roller type valve offers similar flow and pressure characteristics in both directions. A normal or reverse radial, or tainter valve has greater pressure gradient disruptions than a roller lift type. When operating in the "normal configuration" (lock to basin flows), large volumes of air may be entrained with pronounced surging, slug flows, and the turbulence in the basin occurring for particular valve opening settings. The vertical lift type valve also reduces the required access shaft size as compared with a tainter valve. The roller valve wheels hold the valve in position and since the contact area is limited, the frictional loads are reduced, especially compared to a vertical slide valve. The vertical type valve can also be completely removed from the operating shaft without having to access the shaft. The valve is simply pulled directly up for inspection and maintenance.

The flow velocity and/or the water head difference on each side of the valve produces axial loads on the valve. The side ends of the valve slide in a wall recess channels. This recess in the concrete has a metal frame to transfer the axial loads. Also the frame can be used in forming the wall recess. The rollers run vertically within the recessed frame.

4. Side Seal

Side seals prevent leakage along the sides of the valve. The seals are generally attached to the valve at a location within the wall channel. The seals contact a finished contact surface. Refer again to **Figures V.23 and 24**. The higher head or upstream flow side of the valve will push the valve toward the frame slot on the low head side. The side seal is generally on the low head side of the valve, so the valve is pushed onto the seal. As the flow is bi-directional, this side seal is preloaded, and the axial travel and tilt of the valve controlled. In this manner, if the high head is on the side of the valve with the seal, the seal will still be effective. The high head side of the valve is generally on the lock side of the valve. Another option is to provide a side seal on each side of the valve.

Typical seal material is ethylene propylene (EPDM), Buna N or neoprene. The seal could be coated with Teflon to reduce friction, however seal servicing would be more frequent. Sunlight exposure of the seal will not be an issue as the seals will normally be down in the access shaft or within the actuator/valve building. The seals are to be protected from damage and sunlight during shipment and storage. The material for the other seals noted below will be similar. The mating hard surface of the seals, located on the wall chamber and referred to as the "rubbing surface" in **Figure V.25**, would be stainless steel or ultra high molecular weight polyethylene (UHMWPE) to reduce corrosion and thereby wear on the seals. The selection of the seal material should be based on seal loading, predicted wear, and experience. The stainless steel surfaces could be prefabricated with a carbon steel-backing piece with fasteners to connect to the wall frame and expedite maintenance.

5. Bottom Seal

A bottom sill plate would be a stainless steel structural member embedded in the floor with the top surface flush with the conduit invert. Refer to **Figures V.27 and 28**.

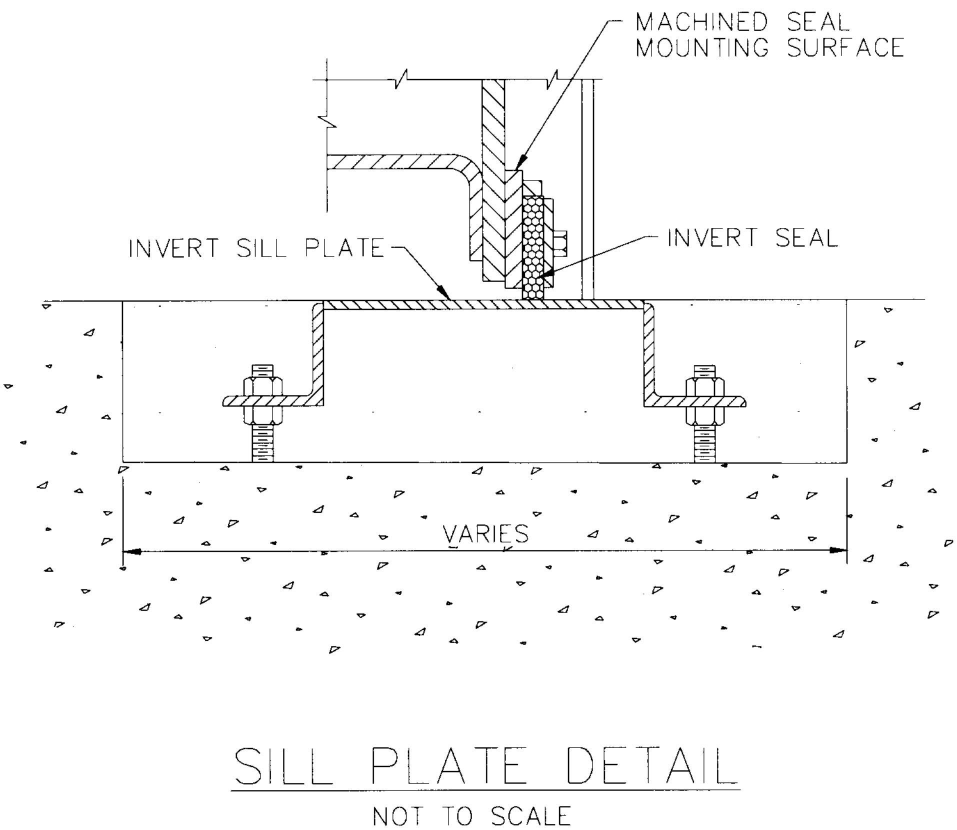

Figure V.27 – Sill Plate Detail

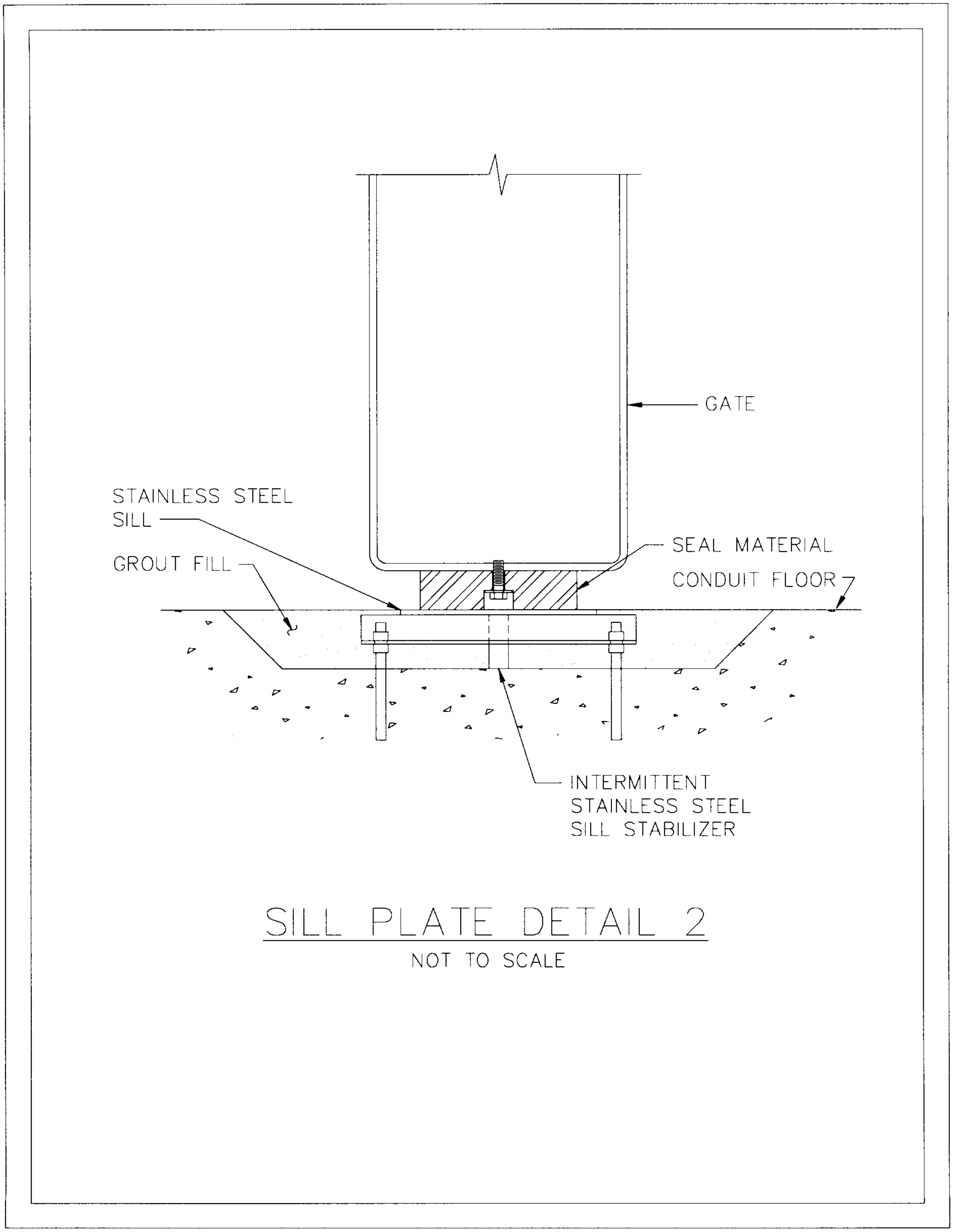

Figure V.28 – Sill Plate Detail 2

A resilient elastomeric or plastic seal, such as UHMWPE, would be attached to the bottom edge or side of the valve. This seal material is softer than the stainless steel and would conform around particulate debris on the sill plate, without damaging the sill plate. Again, the sill plate is embedded in the conduit floor. It can be inspected or replaced if this section of the conduit is isolated and the water removed. The seal at the bottom of the valve is replaced by pulling up the valve and replacing the seal at grade, and then sending the valve back down the operating shaft. The seal profile should be low because of the flow velocities and load. A high profile seal is more likely to warp, chip and vibrate. This low seal requires the sill plate to be carefully installed true and straight.

6. Top Seal

The height of the valve is taller than the height of the conduit. Therefore, the top of the valve would extend into the roof of the conduit. The top seal would be attached to the top section of the valve face. When the valve is closed, the seal would contact a mating surface. Multiple rows of seals could be used as there is more sealing surface available at the top of the valve. The seals could be a compound or J-type seal design. The seal projection from the valve should be as short as reasonable to reduce vibration and unbalanced head loads.

7. Valve Construction

The roller valve is expected to be metal plate on the outer skin on both sides of the valve with metal framing inside. The framing determines the thickness of the valve, as it is to resist the hydraulic forces and maintain dimensional tolerances. The body cavity can be filled with inert material for buoyancy if the valve manufacturer or operator manufacturer determines that it is required. Any ballast required could be modular and removable to accommodate maintenance activities.

Materials of valve construction and fabrication shall be in compliance to internationally recognized standards. Internationally recognized standards include standards that are recognized by material manufacturers and fabricators such as the International Organization for Standardization (ISO), as well as country derived standards such as American Society for Testing and Materials (ASTM), British Standards (BS), DIN (Deutsches Institut fur Normung), Japanese Institute for Standardization (JIS) and European Standards (EN).

The primary portions of the valve and actuator can be fabricated in the manufacturer's shop. Some site assembly may be required due to the shipping and handling issues involving the size and weight of the valve and actuator.

8. Actuator

The actuator produces and controls the vertical motion of the valve. As noted previously there are two different valve services: control valve and closure bulkhead. The control valve requires the quick-acting actuator, as the travel time is approximately one minute. To achieve the fast time with a valve of this size and service criteria, a hydraulic cylinder is the best selection. Pressurizing a cylinder section of the actuator is required to lift the valve. Gravity may close the valve, so fluid would be released from the cylinder to allow the valve to lower. In other words, the control valves would actually travel downward, by the force of gravity alone, as hydraulic fluid is allowed to exit the cylinder. Control valves and orifices in the hydraulic system would control the fluid flowrate out of the cylinder, thereby reducing the closing speeds as the gate approaches the sill and seals. The flow and pressure of the hydraulic fluid and equipment sizes determines the speed and load capacity of the actuator.

Accumulators can be used to store the energy (pressurized hydraulic fluid) required to operate the valve and reduce the momentary power consumption. The required motor horsepower for the hydraulic fluid pumps when using an accumulator is about 15 kW (for a recharge time of 10 minutes), based on a preliminary analysis of this project by GE Hydro. Motor horsepower without an accumulator is about 100 kW. If an accumulator is used, it will require a certain amount of time, between operating cycles, to recharge the accumulator. An initial estimate is 10 minutes, based on the assumption that cycle time between travels would be longer. However, this is not seen as a limiting factor that reduces vessel throughput. This recharge time could be made shorter by increasing the capacity of the pumps and the rest of the hydraulic system. In conclusion, decisions on the hydraulic system and equipment are best made once the specific gate arrangement and information have been selected. This work would be performed in a later phase of the project when hydraulic requirements, gate size, operating cycle times, etc. have been defined and approved.

The peak load at operating differential head and moving the valve from static seal contact at the same time is expected to be a minimum of around 535 kN (120,000 pounds).

9. Serviceability

The valves and installation must be maintainable. Ease of maintenance should be a strong consideration due to the size and location of the valves relative to grade.

The parts that may wear, primarily seal material and rollers, should be attached to the valve itself, as described above. In this manner, the valve could be raised out of the recess for maintenance or replacement and then dropped back down into operating position. The seals and wheels are softer than their contacting surfaces. These softer materials will wear as opposed to the contacting surfaces. The harder materials remain in the conduit structures. The parts remaining in the conduit can also be inspected and maintained. The bulkheads are lowered into position and the water in the sections are transferred and pumped out. Submersible pumps can be lowered to complete the pumpout and remain in the dry section to remove any leakage. Permanent pumps are not required. After the maintenance is performed, water can be reintroduced into the dry chamber with the bypass valves described above.

10. Model Testing

It is recommended that hydraulic model study of the lock, water saving basin, conduit and culvert system be conducted as well as prototype testing of the conduit and culvert valve design and installation.

E. Electrical Features Design

The electrical requirements for the project include power to the hydraulic motors which operate the valves, outdoor lighting for the esplanade and basins, and a reliable control system for the lock and WSB operations. Power is available locally, and will be distributed primarily underground to serve both the motor loads and the lighting components. Each of the hydraulic motors will be approximately 15 kilowatts and will have a dedicated circuit. The motors will be protected from overloads, and each motor circuit will be individually protected from short circuit. Control of the motors will be available both locally at each of the valves, and also remotely at a central location.

Lighting for the site will be designed to provide 75 lux of illumination along the esplanade and associated areas where personnel will travel. High pressure sodium fixtures will be used for the outside lighting, mounted on poles of approximately 9m (30'). Also, the water saving basins will be illuminated for monitoring purposes, with an average lighting level of 50 lux. The entire lighting system will be automatic, with on and off override options for both maintenance and special conditions. Electrical distribution to each of the motors, and the light poles will be installed underground or otherwise concealed, and will consist of conduits and other wiring materials designed to be resistant to the corrosive nature of the ocean environment. AC power is available at the site, so new power panels and transformers will be provided to serve the new motor and lighting electrical loads. All miscellaneous power requirements for the site will be provided with power from the new power panels installed.

One of the more important electrical requirements will be a reliable control system to automate the operations of the combined lock F/E systems and water saving basins. Due to the complexity of the system, caused by multiple equalizations involving locks and basins, the system will be near impossible for a human to operate without computer assistance and control. Therefore, this control system should include water sensors and logical programming for managing the complex equalization operations. In addition, the system should provide for fail-safes implemented with specific programming and/or mechanical provisions to prevent the operations from being done out of sequence.

In general, the electrical requirements for the project will be standard and typical, with no unusual or special procedures, methods or materials required. Providing lighting and electrical power to all motors and equipment that require it should present no significant difficulties. However, some specialized and complex electrical controls and equipment will be needed to provide a very reliable control system for the lock and basin operations.

VI. QUANTITY TAKE-OFFS AND OPINIONS OF PROBABLE COST

A. General Assumptions and Cost Information

1. General

This section presents the preliminary cost estimates for the water saving basin configurations considered at the Panama Canal Atlantic and Pacific sites. These cost estimates are in 2001 Dollars, based on the conceptual designs (which do not include seismic considerations) developed for these features.

It should be emphasized that the construction costs are based on limited field data with respect to excavation, temporary/permanent drainage, and foundation design. Contingencies have been included for each line item cost, and the magnitude of each contingency reflects the level of knowledge of the site data, design, and local costs for the specific line item. The average contingency rate for the total estimated costs was less than 33%, which is below the 35% to 40% contingency range specified in the scope of work for this study.

In order to increase confidence levels and further reduce these contingencies, additional data and design refinement is needed. Refinements, in future design phases, that would better define the excavation and disposal cost items include improved soil contours and rock contours, additional hard and soft rock strength and quality data (with documentation of rock excavation methods), and identification of location and capacities of spoil disposal sites. Examples of other refinements that would better define cost items include verification of crushed existing rock suitability for the select fill foundation, additional geotechnical information pertaining to potential soil consolidation and settlement, and seismic analysis/design of the reinforced concrete basin structures. Future design phase refinements that would better define construction dewatering costs include determination of groundwater levels, soil and rock permeabilities, and preliminary design details for cut-off walls and grout curtains.

The preliminary, conceptual-level cost estimates do not include costs for the following:

- Mobilization, demobilization, and preparatory work (assumed to be included in the overall lock contract)
- Demolition of existing structures and roads (only applicable to Atlantic Side)
- Hazardous, Toxic, and Radioactive Waste (HTRW) disposal
- Cultural resources (if items of archeological significance are discovered during construction)
- Local taxes on labor, materials, and equipment
- Engineering and design
- Construction management

These items should be considered as the designs are developed further.

2. Assumptions

This section describes the assumptions that have been made in developing the conceptual cost estimates. The following is a list of the assumptions:

a) All work within each estimate is advertised in one contract package.

b) A suitable nearby dock facility will be available to the contractor for barge delivery of materials.

c) Mass earthwork is accomplished with scrapers, hydraulic excavators, and large off-road dump trucks. It is assumed that excavation methods will be similar to those used for the recent Gaillard Cut widening and that excess soil and rock will be hauled to the original third lane excavation site for disposal. The remaining excess soil and rock will be hauled to a nearby construction dock facility and dumped onto barges. The barges will then deliver the material to another disposal site. At the Atlantic side, this secondary disposal site is assumed to be nearby. At the Pacific side, it is assumed that the other disposal site is near the mouth of the canal, where an artificial island will be constructed with the material. See **Appendix J** for a detailed breakdown of the assumed excavation and disposal requirements, productivities, and costs.

d) Atlantic Side
Based on the geotechnical reports, it is anticipated that the rock at the Atlantic site will be excavated using similar equipment and techniques as used for the soil excavation. However, the rock excavation productivities are assumed to be lower.

Pacific Side
Based on the geotechnical reports, approximately 20 percent of the Pacific site rock is composed of the La Boca Formation, and the geotechnical report indicates that this rock may be excavated using similar equipment and techniques as used for the soil excavation. However, the rock excavation productivities are assumed to be lower. The remaining 80 percent of the rock at the Pacific site is sound basalt. It is assumed that drilling and shooting will be required to excavate the basalt. While Gaillard Cut widening bids were reviewed in the development of the unit costs for basalt excavation, it is assumed that relatively inexpensive explosives (such as ANFO) were used at the Gaillard Cut. In the damp, below grade holes at the water saving basins, it is assumed that more expensive explosives will be required.

e) To the extent required, excavated rock (from the basin and lock sites) will be processed by the contractor for use as select backfill material. It is not assumed that this material will be suitable for concrete aggregate.

f) The vertical trench walls for the conduit construction are anchored sheet pile walls. This is a conservative assumption, as it precludes sheet pile salvage and reuse.

g) The construction schedule will be long enough to allow the contractor to purchase large custom steel forms, which will be reused numerous times for the concrete structures.

h) In the determination of dewatering costs, it is assumed that temporary sump installation and pumping will be required for approximately three years, and the water saving basin construction will require operation and maintenance of the dewatering wells for an additional year, beyond that required for the lock construction.

i) Temporary and permanent drainage design has not been explicitly determined for this conceptual design stage. Cut-off wall, grout curtain, and underdrain quantities have been roughly estimated in order to determine the order of magnitude increase in dewatering cost attributable to the basin construction and the relative cost differences between the basin configurations. See **Appendix J** for the basis of dewatering design and unit cost assumptions. Additional geotechnical information is required to adequately address the drainage and dewatering systems in future design phases.

j) The allowance for rock grouting below the cut-off walls will effectively limit the groundwater intrusion into the work site. If this is not the case, either the rock grouting or the dewatering (pumping) line item costs will need to be increased.

k) Concrete reinforcing has not been explicitly determined for this conceptual design stage. Reinforcing steel quantities were calculated on a kg/m^3 of concrete basis, using engineering judgment and previous relevant experience.

3. Local Unit Costs

In an effort to evaluate the local unit costs for this construction project, several international contractors were contacted. See **Appendix J** for local cost documentation, including telephone conversation records with these contractors and Gaillard Cut widening bids. The contractors provided general information relating to local labor rates, productivity rates, material and labor costs, and contractor mark ups (including field overhead). They indicated the following:

- Panamanian labor rates are approximately 20 to 30 percent less than U.S. West Coast labor rates.

- Local labor is available and is skilled or trainable. However, due to the scale of this project and other simultaneous large-scale projects in Panama, some expatriate labor requirements are anticipated, after the local crane/equipment operator pool is exhausted. The need for expatriate labor will increase the labor costs, possibly offsetting the savings due to lower Panamanian labor rates.

- Local productivity rates are less than U.S. West Coast productivity rates (indicated percent reductions ranged from 10 to 40 percent, per contractor discussions documented in **Appendix J**).

- Local labor benefits are higher than U.S. labor benefits, including additional paid holidays.

- The net effect of the above items may increase the total labor costs by approximately 10 percent. (Labor is assumed to be 30 percent of the total cost; 0.1 x 0.3 = 0.03, yielding a composite local cost adjustment factor of 3 percent.)

- Local material, equipment, and fuel costs are higher than in the U.S. West Coast.

- Contractor mark up, including field overhead, is approximately 15 to 20 percent. Per ACP direction, a mark up of 15 percent was assumed.

- Contractor profit margin is typically higher for international work. However, profit margins are also highly variable, being closely related to the global economy. Contractors may include as much as 20 percent to 25 percent profits during very good market conditions. During recessions, contractor profits may be reduced to as little as 5 percent. Market conditions at the time of construction, can not be accurately predicted. Per ACP direction, a contractor profit of 12 percent was assumed.

The above factors were considered and incorporated into the unit and mark up costs in the preparation of the conceptual-level cost estimates.

As stated in **Section A.1**, local taxes have not been included in the construction cost estimates. However, unless these taxes are waived for this project by the Panamanian government, it is anticipated that these taxes may increase the costs significantly. Therefore, further evaluation of the implication of local taxes on the estimated construction costs is recommended for future design stages.

B. Preliminary Mechanical Costs

In an effort to determine costs for the water saving basin valves and actuators, numerous manufacturers were contacted. Based on the size and number of the conduits selected by ACP, GE Hydro provided a general estimate of the following costs (cost backup provided in **Appendix J**):

- Control Valve = $250,000, Control Gate Actuator = $180,000
  Control valve cost includes sealfaces, tracks, guides, lift links, cylinder support, and anchors. The control gate actuator cost includes cylinder, power unit, accumulator, control panel, piping, and oil.

- Closure Bulkhead = $200,000, Actuator = $180,000
  The closure bulkhead cost includes sealfaces, tracks, and guides. The closure bulkhead actuator cost includes bed frame, electric motor, drum gearing, brake, etc.
  For purposes of this study, it is assumed that actuators will not be required for the closure bulkheads. ACP envisions use of a movable crane for installation/retrieval of closure bulkheads.

- Shipping/Contingency = 20%
  Shipping cost is estimated at 7% of total cost. Contingency cost is estimated at 13% of total cost.

So for each conduit,

Two (2) Control Valves and Actuators:	US $ 860,000
Shipping/Contingency (20%)	US $ 172,000
Total for Control Valves and Actuators:	US $1,032,000
	SAY ***US $1,050,000/conduit***

And for each option on each ocean side,

Twenty (20) Closure Bulkheads (Enough for 10 Conduits)	US $4,000,000
Shipping/Contingency (20%)	US $ 800,000
Total for Closure Bulkheads:	US $4,800,000
	SAY *US $4,800,000*

These valve costs were based upon the following parameters:

- Opening size of 4.267m wide by 8.534m high,
- Maximum static head in both directions = 30.48m,
- Maximum operating head in both directions = 12.19m,
- Operating time for control gates, and
  - Open or close – 1 minute
  - 10 min to recharge the accumulators after opening
  - closing by gravity
- All power units to be within 50m of cylinder

The manufacturers stated that these were very rough estimates and given the similar size of the conduits and the scope of this study, these estimates would be applicable to all of the options. ***The estimates above include the design, manufacture, and preparation for shipment of the listed items and do not include installation, supervision of installation, import duties and taxes, progress payments, nor a full scale wet test. The estimates are good for the current cost levels for labor and materials, standard delivery times, and current exchange rates, under standard GE terms and conditions with no sourcing restrictions.*** Additional cost backup can be found in **Appendix J**.

C. Preliminary Electrical Costs

The electrical requirements for the project include power to the motors which operate the hydraulic valves, outdoor lighting for the esplanade and basins, etc. Power is available locally, and will be distributed primarily underground to serve both the motor loads and the lighting components. Therefore, the main electrical components will be lighting, motors, and distribution (conduit, wires, panels). These costs are very preliminary and are applicable for each option at this level of study. Additional cost backup can be found in **Appendix J**.

Lighting:	US $ 434.000
Motors:	US $ 180,000
Distribution (panels, conduits, etc.):	US $ 642,000
Subtotal Electrical:	US $1,256,000
Contingency (20%)	US $ 251,000
Total Electrical:	US $1,507,000
	SAY ***US $1,600,000***

D. Opinions of Probable Costs for Atlantic Side Options

For the Atlantic site, **Table VI.1** summarizes the total construction cost estimates for the basin conceptual design options that were studied:

Table VI.1 – Opinions of Probable Costs (Conceptual Level) for Atlantic Side Options

OPTION	Civil/Structural Construction Costs (U.S. Dollars, in Millions)	Mechanical Item Costs (U.S. Dollars, in Millions)	Electrical Item Costs (U.S. Dollars, in Millions)	**Total Costs (U.S. Dollars, in Millions)**
Option 1	$325	$30.0	$1.6	**$357**
Option 2	$360	$30.0	$1.6	**$392**
Option 3	$418	$23.7	$1.6	**$444**

Notes: Effective date of pricing is 2001. Again, this conceptual-level cost estimate does not include costs for the following:

1. Demolition of existing structures and roads
2. HTRW disposal
3. Cultural resources (if items of archeological significance are discovered during construction)
4. Local taxes on labor, materials, and equipment
5. Engineering and design
6. General and administrative expenses, including project and construction management
7. Interest on borrowed money during construction

For a detailed breakdown of costs for these options as well as quantity take-offs and local cost documentation, see **Appendix J**.

E. Opinions of Probable Costs for Pacific Side Options

For the Pacific site, **Table VI.2** summarizes the total construction cost estimates for the basin conceptual design options that were studied:

Table VI.2 – Opinions of Probable Costs (Conceptual Level) for Pacific Side Options

OPTION	Civil/Structural Construction Costs (U.S. Dollars, in Millions)	Mechanical Item Costs (U.S. Dollars, in Millions)	Electrical Item Costs (U.S. Dollars, in Millions)	**Total Costs (U.S. Dollars, in Millions)**
Option 1	$380	$30.0	$1.6	**$412**
Option 2	$466	$30.0	$1.6	**$498**
Option 3	$547	$23.7	$1.6	**$573**

Notes: Effective date of pricing is 2001. Again, this conceptual-level cost estimate does not include costs for the following:

1. HTRW disposal
2. Cultural resources (if items of archeological significance are discovered during construction)
3. Local taxes on labor, materials, and equipment
4. Engineering and design
5. General and administrative expenses, including project and construction management
6. Interest on borrowed money during construction

For a detailed breakdown of costs for these options as well as quantity take-offs and local cost documentation, see **Appendix J**.

VII. STUDY FINDINGS AND RECOMMENDATIONS

A. Hydraulic

It is apparent that two of the most important influences on the required size of water saving basins and conduits are the range of water levels (both lake and ocean) and lockage lengths (426.7 m - 1400', 457.2 m - 1500', and 487.7 m - 1600') for which the systems are designed. This is especially important on the Pacific Ocean side where the tide range can exceed 7 meters. These variations had significant impacts, especially on the basin wall heights required for the theoretical water savings percentage to be achieved under all conditions. If the results of this study show that the ***water saving basin systems*** are not economically justifiable when designed for this full range of variation, the systems could be re-designed by considering a narrower range of hydrologic (see percent exceedance data in **Appendix C**) and hydraulic conditions (426.7 m - 1400', 457.2 m - 1500', and 487.7 m - 1600' lockage lengths) which may significantly reduce the conduit sizes, the basin wall heights, and costs.

The stacked arrangement with basins on only one side of the lock is problematic. The range of water levels and lockage lengths necessitate an "overlap" between basins if the theoretical water saving percentage is always to be realized. Nonetheless, the problem can be overcome by increasing the width of the basins to a value greater than that of the locks ($m > 1.0$). However, this entails additional excavation for the upper locks and higher costs. Therefore, in future studies, a stacked arrangement with basins on both sides of the lock would be a superior configuration based on hydraulic consideration, although it would undoubtedly be more costly. Having basins on both sides of the lock will allow for the basins on one side to be offset from those on the other side (which will better accommodate the necessary "overlap").

An in-house spreadsheet model was created that was checked against the USACOE's LOCKSIM model and calibrated/verified to the existing locks with satisfactory results. The preliminary design of the lock F/E culverts was also completed to determine reasonable head loss estimates at the interface of the two systems and more importantly to determine the upper threshold of WSB conduit size (i.e., the WSB conduit should not be larger than the lock F/E culvert).

Parametric curves were created based on the results of hundreds of individual model runs. These allowed "what-if" scenarios to be evaluated with a range of culvert sizes and arrangements. These curves could also be plotted against the two most important design criteria -- equalization time and instantaneous maximum F/E rate. The explicit criteria for the lock F/E culverts were:

- the instantaneous maximum F/E rate should not exceed 2.28 m/min (7.5 ft/min) (the maximum for the existing locks with two culvert operations), and
- F/E times for a 3-lift system should be 8 – 9 min per lift (based on the existing system) and for a two-lift system, (3 lift x 8 – 9 = 24-27 min total)/2 lift = 12 – 13.5 min/lift.

In applying these criteria, the finalized lock F/E culvert sizes were found to be:

- Options 1 & 3 – Atlantic Side (8.84 m - 29'),
- Options 1 & 3 – Pacific Side (8.53 m - 28'),
- Options 2 & 4 – Atlantic Side (7.92 m - 26'), and
- Options 2 & 4 – Pacific Side (7.62 m - 25').

A comparative study verified that vertical lift valves should be used for the WSB conduits due to faster equalization times and symmetrical behavior with bi-directional flow. Parametric curves were also created for the design of the water saving basin conduits. The design criteria for the WSB conduits were:

- the WSB conduits should not be larger than the preliminary F/E culvert sizes,
- no conduit solution should exceed an instantaneous maximum F/E rate of 2.28 m/min (7.5 ft/min) for basin to lock operations, and
- no conduit solution should have a single basin operation time of less than 2 minutes (which is the assumed shortest time needed to open and immediately close the valves).

Using these criteria, a myriad of solutions were available so methodologies were formulated to combine the results from the lock F/E culvert and the WSB conduit analyses to compute more meaningful statistics including total operation time, allowable transits/day, etc.

These statistics were submitted to ACP for review. The finalized WSB conduit arrangement and sizes chosen by ACP were:

- Option 1 – 4 conduits/basin (6.10 m - 20'),
- Option 2 – 4 conduits/basin (7.32 m - 24'),
- Option 3 – 2 conduits/basin (8.53 m - 28'), and
- Option 4 (side-by-side basins) – 4 conduits/basin (6.71 m - 22')

ACP's selection of the number and sizes of conduits for the above options is based upon the desire to obtain a range of price scales for the different options. Therefore, the conduit selections are not necessarily the optimum for each option, but will provide a range of price options from most to least costly.

At this point, work was halted by ACP before the WSB conduit size could be finalized for the stacked basin arrangement for Option 4. A preliminary analysis using the same conduit size as that selected for the side-by-side basins indicated that the performance characteristics of the stacked WSB would be similar. However, these preliminary reviews also indicated that further refinement would be needed to size the conduits for the stacked basin arrangement since the equalization times were approximately 15% longer when compared with the side-by-side basin arrangement.

The results of the hydraulic analyses are summarized and compared for all options in **Tables VII.1-.3**.

Table VII.1 System Layout and Theoretical Water Savings Percentages for all Options

OPTION	# Lifts	# WSBs per Lift	# Conduits per WSB	Conduit Diameter	Theroretical Water Savings Percentage
Option 1	3	2	4	6.10 m (20')	50%
Option 2	2	3	4	7.32 m (24')	60%
Option 3	3	6 (half size)	2	8.53 m (28')	60%
Option 4	2	2	4	6.71 m (22')	50%

Table VII.2 Overall Water Usage and F/E Times per Lockage for Atlantic Side Options

Metric Units

OPTION	Water Intake Height Without Basins (m)			Water Intake Height With Basins (m)			Water Intake Volume With Basins (Avg. Lockage = 61 m x 457 m) ($10 m^3$)			Average Total F/E Time per Lockage (min)
	min	mean	max	min	mean	max	min	Mean	max	
Option 1	7.01	8.49	10.04	3.51	4.24	5.02	97.71	118.27	139.85	31.46
Option 2	10.48	12.73	15.00	4.19	5.10	6.00	116.74	142.06	167.38	26.52
Option 3	7.01	8.49	10.04	2.80	3.40	4.02	78.17	94.65	111.98	36.43
Option 4*	10.48	12.73	15.00	5.24	6.37	7.50	145.97	177.58	209.18	26.35

* side-by-side basins

English Units

OPTION	Water Intake Height Without Basins (ft)			Water Intake Height With Basins (ft)			Water Intake Volume With Basins (Avg. Lockage = 200' x 1500') (million gal)			Average Total F/E Time per Lockage (min)
	min	mean	max	min	mean	max	min	mean	max	
Option 1	23.00	27.85	32.93	11.50	13.92	16.46	25.81	31.24	36.94	31.46
Option 2	34.37	41.78	49.23	13.74	16.72	19.70	30.83	37.52	44.21	26.52
Option 3	23.00	27.85	32.93	9.20	11.14	13.18	20.65	25.00	29.58	36.43
Option 4*	34.37	41.78	49.23	17.18	20.90	24.62	38.55	46.90	55.25	26.35

* side-by-side basins

Table VII.3 Overall Water Usage and F/E Times per Lockage for Pacific Side Options

Metric Units

OPTION	Water Intake Height Without Basins (m)			Water Intake Height With Basins (m)			Water Intake Volume With Basins (Avg. Lockage = 61 m x 457 m) ($10 m^3$)			Average Total F/E Time per Lockage (min)
	min	mean	max	min	mean	max	min	mean	max	
Option 1	6.10	8.46	11.17	3.05	4.23	5.58	84.96	117.93	155.66	34.72
Option 2	9.12	12.70	16.69	3.65	5.08	6.67	101.62	141.55	186.07	27.58
Option 3	6.10	8.46	11.17	2.44	3.38	4.47	67.97	94.31	124.56	37.28
Option 4*	9.12	12.70	16.69	4.56	6.35	8.35	127.11	177.07	232.63	27.55

* side-by-side basins

English Units

OPTION	Water Intake Height Without Basins (ft)			Water Intake Height With Basins (ft)			Water Intake Volume With Basins (Avg. Lockage = 200' x 1500') (million gal)			Average Total F/E Time per Lockage (min)
	min	mean	max	min	mean	max	min	mean	max	
Option 1	20.01	27.77	36.64	10.00	13.92	16.46	22.44	31.15	41.11	34.72
Option 2	29.91	41.67	54.77	11.96	16.72	19.70	26.84	37.39	49.15	27.58
Option 3	20.01	27.77	36.64	8.00	11.14	13.18	17.95	24.91	32.90	37.28
Option 4*	29.91	41.67	54.77	14.96	20.90	24.62	33.57	46.77	61.44	27.55

* side-by-side basins

The importance of additional future hydraulic numeric and physical modeling for this project cannot be overstated. The planned concept study of the lock F/E system will provide the opportunity to look at the entire system as a whole. This will narrow the range of alternatives and should be followed by physical model tests to verify the concepts carried forward.

B. Geotechnical

1. Atlantic Side

From a geotechnical perspective, water saving basins located on the east side of the proposed locks are expected to be much less costly than basins located on the west side. The elevation of the top of rock is expected to be higher on the east side, so less excavation and replacement of unsuitable material would be needed. On the east side, the excavation for basin construction would be well removed from the existing locks and thus less likely to interfere with lock operations. On the west side, we estimate that the excavation for basin construction would extend to within about 20 meters (65 ft.) of the existing locks in plan dimension and would extend to a level about 16 meters (52 ft.) below the maximum water level in the existing lock. Temporary excavation support would be needed, but more critically, the stability of the existing lock walls would have to be evaluated under these conditions and special reinforcement or other support measures may be needed. All things considered, the site is not well-suited to tunneling, and tunneling is expected to be substantially more costly than cut-and-cover installation for the culvert. Cut-and-cover construction is relatively inexpensive because the rock can likely be excavated without blasting. The advantages of cut-and-cover construction are further increased because of the high costs of the dewatering and support needed for tunnel construction. Detailed recommendations for the future exploration and testing program recommended for the final design phase for water saving basins adjacent to the A-2 alignment are provided in **Appendix D**.

2. Pacific Side

At this stage in the project, we do not have enough data to identify differences between geotechnical conditions on the southwest side of the proposed locks and geotechnical conditions on the northeast side. As noted previously, there is little data on the southwest side of the proposed locks. However, the areas excavated in 1939 and the drainage connected to Miraflores Lake are located on the northeast side of the P-1 alignment. These areas will likely require special treatment potentially including cofferdams, additional excavation and special foundations. If the geotechnical conditions on the southwest side are comparable to the northeast, then the southwest side would be preferable from a geotechnical perspective. The exploration and testing program recommended for the final design phase for water saving basins adjacent to the P-1 alignment are provided in **Appendix D**.

C. Structural/Civil

1. Atlantic Side

At the Atlantic site, various arrangements and numbers of water saving basins were investigated for use in conjunction with new locks constructed along Alignment A-2. Three configurations of water saving basins were considered at the Atlantic site, with features common to all configurations. For example, all basins included transverse interior walls. The interior wall for Options 1 and 2 divided each basin into two compartments. This interior wall, for these options, provides the ability to continue operation of half the basin while maintenance is performed on the other half. To reduce wave heights, openings (fitted with removable maintenance bulkheads) are recommended in the interior walls. In order to provide improved access for operations and maintenance personnel, the basins were separated by a space of 16 m (52 ft.). In order to provide room for the lock roller gate monoliths, the closest basin wall was placed 125 m (410 ft.) from the centerline of the closest new lock lane.

Option 3 consists of a total of eighteen side-by-side basins, with nine basins on each side of the new lock, and one conduit connects each basin to the lock chamber. The Option 3 configuration includes two transverse interior walls (placed at the basin quarter points) that divide each basin into three compartments, for the purpose of wave reduction. To allow flow to the conduits, openings must be included in the Option 3 interior walls. For maintenance purposes, it is recommended that these openings be fitted with removable maintenance bulkhead gates. The distance between the proposed new third lane locks and the proposed new fourth lane locks, along alignment A-2, is not adequate for basin placement in that area. In order to provide enough room for the proposed fourth lane basins, the fourth lane centerline must be shifted at least 204 m

(669 ft.) to the east. Due to the large space requirements and lock spacings associated with the Option 3 configuration, centralized lock operations would be difficult in this case.

For the Atlantic site, the **Tables IV.1 and 2** summarize the total excavation and fill requirements for the basin options that were studied. Option 3 required the most cut/fill while Option 2 required the least (in most cases).

The following water saving basin floor designs were considered:

- A concrete floor, with a drainage blanket, supported by a select fill or rock foundation
- A concrete floor, with a drainage blanket, supported by a drilled shaft foundation
- An unlined rock floor, with no floor drainage required

An unlined rock floor may be feasible for the basins that are founded in slightly weathered or sound rock. (It is not unusual for locks that are founded in rock to have unlined rock floors.) However, as described in the geotechnical report, there is insufficient data on the rock permeability and rock quality, at this time, to ensure that the unlined rock floor performance will meet the project requirements and that the cost of any required rock drilling and grouting will be less than the cost of a cast-in-place concrete floor. Therefore, as recommended by the geotechnical engineers, a concrete floor was assumed for this conceptual design. Evaluation of an unlined rock floor, for applicable basins, is recommended for future design phases.

For all basin walls, a 1.22 meter (4-foot) freeboard was provided above the design water elevation. This freeboard should be adequate to contain the water, accommodating the turbulence and wave heights anticipated in the water saving basins. The following wall types were considered for the water saving basin walls:

- Standard Cantilever Wall
- Counterfort Wall
- Braced Counterfort Wall (Interior Columns required to support the bracing)
- Drilled Shaft Cantilever Wall
- Drilled Shaft Tieback Wall
- Braced Drilled Shaft Wall (Interior Columns required to support the bracing)

For the wall heights required at the Atlantic site, the Counterfort Wall design was found to be the most cost effective of the wall types listed above.

Rectangular conduits connect the water saving basins to the lock fill/empty system. The conduits slope downward, towards the lock chamber. At the Atlantic site, the conduit grades are approximately 13 percent, 10 percent, and 12 percent for Options 1, 2, and 3, respectively.

The conduits were conceptually designed as rectangular cast-in-place concrete box structures. A rectangular shape is required at the conduit valve and bulkhead locations. A circular conduit would be acceptable elsewhere if smooth transitions were provided between the rectangular and circular sections. However, circular conduits would only be cost effective if a precast construction method was specified, and precast construction is not anticipated to be practical for the required conduit sizes.

2. Pacific Side

At the Pacific site, various arrangements and numbers of water saving basins were investigated for use in conjunction with new locks constructed along Alignment P-1. Three configurations of water saving basins were considered at the Pacific site, with features common to all configurations. For example, all basins included transverse interior walls. The interior wall for Options 1 and 2 divided each basin into two compartments. This interior wall, for these options, provides the ability to continue operation of half the basin while maintenance is performed on the other half. To reduce wave heights, openings (fitted with removable maintenance bulkhead gates) are recommended in the interior walls. In order to provide improved access for operations and maintenance personnel, the basins were separated by a space of 16 m (52 ft.). In order to provide room for the lock roller gate monoliths, the closest basin wall was placed 125 m (410 ft.) from the centerline of the closest new lock lane.

Option 3 consists of a total of eighteen side-by-side basins, with nine basins on each side of the new lock, and one conduit connects each basin to the lock chamber. The Option 3 configuration includes two transverse interior walls (placed at the basin quarter points) that divide each basin into three compartments, for the purpose of wave reduction. To allow flow to the conduits, openings must be included in the Option 3 interior walls. For maintenance purposes, it is recommended that these openings be fitted with removable maintenance bulkheads. The distance between the proposed new third lane locks and the proposed new fourth lane locks, along alignment A-2, is not adequate for basin placement in that area. In order to provide enough room for the proposed third lane basins, the proposed fourth lane centerline must be shifted at least 204 m (669 ft.) to the southwest. Space is available for this basin arrangement at the Pacific site. However, the Option 3 configuration results in a large spacing between the third and fourth lanes, making centralized lock operations more difficult and precludes the possibility of a common middle wall between the lanes.

For the Pacific site, **Tables IV.3 and 4** summarize the total excavation and fill requirements for the basin options that were studied. Option 3 required the most cut/fill while Option 1 required the least (in most cases).

The following water saving basin floor designs were considered:

- A concrete floor, with a drainage blanket, supported by a select fill or rock foundation
- A concrete floor, with a drainage blanket, supported by a drilled shaft foundation
- An unlined rock floor, with no floor drainage required

An unlined rock floor may be feasible for the basins that are founded in slightly weathered or sound rock. (It is not unusual for locks that are founded in rock to have unlined rock floors.) However, as described in the geotechnical report, there is insufficient data on the rock permeability and rock quality, at this time, to ensure that the unlined rock floor performance will meet the project requirements and that the cost of any required rock drilling and grouting will be less than the cost of a cast-in-place concrete floor. Therefore, as recommended by the geotechnical engineers, a concrete floor was assumed for this conceptual design. Evaluation of an unlined rock floor, for applicable basins, is recommended for future design phases.

For all basin walls, a 1.22 meter (4-foot) freeboard was provided above the design water elevation. This freeboard should be adequate to contain the water, accommodating the turbulence and wave heights anticipated in the water saving basins. The following wall types were considered for the water saving basin walls:

- Standard Cantilever Wall
- Counterfort Wall
- Braced Counterfort Wall (Interior Columns required to support the bracing)
- Drilled Shaft Cantilever Wall
- Drilled Shaft Tieback Wall
- Braced Drilled Shaft Wall (Interior Columns required to support the bracing)

The cantilever and counterfort walls are practical for the recommended assumptions of open cut excavation and select fill or rock foundations. For the wall heights required at the Pacific site, the Counterfort Wall design was found to be the most cost effective of the the three wall types that are consistent with the excavation and foundation assumptions.

Rectangular conduits connect the water saving basins to the lock fill/empty system. The conduits slope downward, towards the lock chamber. At the Pacific site, the conduit grades range from 12.3 percent to 13.5 percent for the Option 1 design. For Option 2, the conduit grade is approximately 10.8 percent at the upper lift chamber and is approximately 9.7 percent at the lower lift chamber. For Option 3, the conduit grades range from approximately 11.8 percent to approximately 12.9 percent.

The conduits were conceptually designed as rectangular cast-in-place concrete box structures. A rectangular shape is required at the conduit valve and bulkhead locations. A circular conduit would be acceptable elsewhere if smooth transitions were provided between the rectangular and circular sections. However, circular conduits would only be cost effective if a precast construction method was specified, and precast construction is not anticipated to be practical for the required conduit sizes.

3. Structural/Civil Recommendations

The following items are recommended for future design phases:

- Refine the geotechnical parameters based on currently scheduled additional geotechnical explorations. The conceptual designs may then be updated using these refined parameters.

- Determine the seismicity in the region near the proposed new Panama Canal lanes. If the seismicity is determined to be low, then a coefficient method analysis for the design earthquake may be sufficient. If the seismicity is moderate to high, then an Operating Basis Earthquake (OBE) and Maximum Design Earthquake (MDE) should be determined and consideration of the dynamic and hydrodynamic effects is recommended.

- Investigation of permanent drainage requirements (potentially including permanent cut-off walls), anticipated water levels adjacent to the basins, and hydrostatic/uplift pressures resulting from blocked drains should be further evaluated.

- Investigation of temporary construction dewatering requirements should be further evaluated.

- Consider use of shallow foundation design to substantially reduce excavation and fill costs. Additional geotechnical information is required (including consolidation and settlement considerations) to determine whether a shallow foundation design is feasible.

- Investigation of alternate basin design concept (concrete-lined reservoirs with sloped sides) should be further evaluated, potentially reducing total basin construction costs.

D. Mechanical

Based on the finalized conduit sizes selected by ACP, it is expected that the conduits will need to be bifurcated in order to accommodate smaller, more manageable valves, to save costs, and to achieve greater reliability through redundancy. Therefore, the valve recess to each conduit will house two main control valves and recesses for four closure bulkheads. A comparative study, described in detail on pages 122-124, verified that vertical lift valves should be used for the WSB conduits due to faster equalization times and symmetrical behavior with bi-directional flow. A control valve is expected to have about thirty (30) operation cycles per day (~15 lockages *2 operations (1-fill, 1-empty)), with a cycle being an up and down travel of the valve.

The best design for the control valves and closure bulkheads in this application appears to be a roller vertical-lift type. A roller valve is a vertical lift type that contacts the support structure with wheels or bearings. The support structure is at both sides of the gate and provides guided vertical slots. Rollers or bearings are attached to the valve at the sides. These rollers or bearings contact a running surface in the conduit structure slot.

The following items are recommended for future design phases:

- Model flow testing of selected valves and controls to verify energy gradients, losses, and loading.

- Modeling of complete water saving basin, culvert, valve, and actuator control system to simulate system dynamic operating parameters, and verify lock fill/drain cycle times.

E. Electrical

In general, the electrical requirements for the project will be standard and typical, with no unusual or special procedures, methods or materials required. Providing lighting and electrical power to all motors and equipment that require it should present no significant difficulties. However, some specialized and complex electrical controls and equipment will be needed to provide a very reliable control system for the lock and basin operations.

F. Overall Findings and Recommendations

The opinions of probable costs estimates for three of the four options can be seen in **Tables VII.4 and 5**. ***ACP's selection of different conduits and valves among the different options was intended to obtain a range of prices and so that the costs could be interchanged between options.***

Table VII.4 – Opinions of Probable Costs (Conceptual Level) for Atlantic Side Options

OPTION	Civil/Structural Construction Costs (U.S. Dollars, in Millions)	Mechanical Item Costs (U.S. Dollars, in Millions)	Electrical Item Costs (U.S. Dollars, in Millions)	**Total Costs (U.S. Dollars, in Millions)**
Option 1	$325	$30.0	$1.6	**$357**
Option 2	$360	$30.0	$1.6	**$392**
Option 3	$418	$23.7	$1.6	**$444**

Table VII.5 – Opinions of Probable Costs (Conceptual Level) for Pacific Side Options

OPTION	Civil/Structural Construction Costs (U.S. Dollars, in Millions)	Mechanical Item Costs (U.S. Dollars, in Millions)	Electrical Item Costs (U.S. Dollars, in Millions)	**Total Costs (U.S. Dollars, in Millions)**
Option 1	$380	$30.0	$1.6	**$412**
Option 2	$466	$30.0	$1.6	**$498**
Option 3	$547	$23.7	$1.6	**$573**

In conclusion, Option 3 is the most costly option, but it also consumes the least water. Option 1 would seem to be preferable to Option 2 as it is less expensive and consumes less water. However, the total average F/E time per lockage for Option 2 is considerably less than either Option 1 or 3. The Option 4 (Side-by-Side Basins) arrangement also deserves further investigation as it would likely provide the most vessel throughput (with the smallest total average F/E time) and could prove to be the most cost effective due to its having the smallest footprint and the fewest number of valves of any of the options studied to date. Nonetheless, Option 2 would also likely consume the most water of any option as well. Therefore, weightings of costs, overall water usage, lock operating characteristics, and vessel throughput goals should be considered when deciding which options warrant further investigation.