



Maintenance and Expansion Evaluation of the Panama Canal

A Major Qualifying Project Report:

Submitted to the faculty of the

WORCESTER POLYTECHNIC INSTITUTE

in partial fulfillment of the requirements for the

Degree of Bachelor of Science

in corporation with the

Autoridad del Canal de Panama

Submitted on December 19th, 2014

Submitted By:

Denzel Amevor

Sarah Antolick

Alex Manternach

Kevin Lynch

Nicolette Yee

Project Advisors:

Dr. Tahar El Korchi PhD

Dr. Aaron Sakulich PhD

Authorship

Abstract- Nicolette Yee

Executive Summary-Denzel Amevor, Sarah Antolick, Kevin Lynch, Alex Manternach, and Nicolette Yee

Table of Contents- Denzel Amevor

Table of Figures- Denzel Amevor

Table of Tables- Denzel Amevor

Table of Equations- Denzel Amevor

Acknowledgements- Nicolette Yee

Introduction-Nicolette Yee

Literature Review- Denzel Amevor, Sarah Antolick, Kevin Lynch, Alex Manternach, and Nicolette Yee

Pacific Entrance Maintenance Dredging-Denzel Amevor and Kevin Lynch

Development of As-Built Stitch Grouting Drawings for Borinquen Dam 1E- Nicolette Yee

Embankment Construction: Compaction of Zone 1 Materials- Alex Manternach

Borinquen Dam 1E Construction Management Process Analysis- Sarah Antolick

Conclusion- Kevin Lynch

References- Denzel Amevor, Sarah Antolick, Kevin Lynch, Alex Manternach, and Nicolette Yee

Appendices - Denzel Amevor, Sarah Antolick, Kevin Lynch, Alex Manternach, and Nicolette Yee

Abstract

The Panama Canal Authority (ACP) is currently undergoing maintenance and expansion projects on the Panama Canal. This project includes progress analyses and design elements related to the Borinquen Dam 1E and the Pacific canal entrance. The goals of this project is to achieve four design oriented objectives including: 1) recommendations for the improvement of the dredging operations at the Pacific canal entrance; 2) the design process for producing as-built grout profile drawings along Dam 1E; 3) the compaction and testing specifications for the clay core of Dam 1E and; 4) a progress and cost analysis including recommendations for Dam 1E. Results from this paper provided the ACP with information and recommendations to improve the functionality of several operations.

Executive Summary

Pacific Entrance Maintenance Dredging

Background

The Autoridad del Canal de Panama (ACP) is currently performing maintenance on the Pacific entrance of the Canal. This entails an extensive dredging project where the ACP plans to reduce the slopes on the Canal banks and deepen the center by removing loose sediment. We have been tasked with determining the progress being made by the Canal Authority sub-contractor, Dredging International, and design more efficient ways for the project to be done. We will discuss and analyze the work done by Dredging International and develop a comprehensive procedure to improve the current dredging operation as well as future maintenance plans.

For the Panama Canal, the maintenance dredging taking place will be for navigation. High mobility is required to function properly in areas where high traffic and currents are present. The Panama Canal maintenance-dredging project will use a trailing suction hopper dredger. Also known as a 'trailer', this dredger has the ability to load a hopper contained within its structure by means of pumps while the vessel is moving ahead (Page 157, Bray, 1978). The ability to move while dredging is helpful in the Canal as the vessel must navigate one of the busiest waterways in the world. In order to unload the vessel will either have a pump discharge or a bottom-discharge to unload sediment below its location. This ability to bottom-discharge is especially useful in the maintenance dredging because the sediment is transported out to sea where it is released.

Methodology

The Panama Canal Authority has asked us to determine the progress of the maintenance dredging project. The dredging operation began on November 5th, 2014 and will end on January 7th, 2015.

The goal of our project is to aid the Panama Canal Authority in the completion of their maintenance dredging project. To accomplish this goal our team will complete the following two objectives:

- Determine if the dredging project is on schedule
- Research and design possible improvements to operations

Results and Discussion

Based on the 1,234,340 m³ the contract requires to be dredged, we were able to determine that Dredging International must dredge at least 32,908m³ per day in order to finish the job by the January 8th, 2015 completion date. The sub-contractor consistently manages to dredge the above the required minimum amount, meaning they will most likely be able to complete the project on schedule.

The maintenance dredging project cost is based on volume of sediment dredged. Since the draghead will pick up slurry that includes water along with the sediment, an accurate way to determine the volume of the sediment was needed. To ensure the volume measurements are accurate, the Breughel uses multiple methods in its calculations including, on board equipment to measure density of slurry and volume it collects, estimates from surveys of how much sediment must be removed, and onboard equipment to measure discharge volume and density. These three methods are compared and fall within 10% of each other.

The Breughel does have many tools to lessen the effects of traffic on the dredging operation. The most helpful is offered by the Canal Authority, which gives a schedule of all vessel activity through the Canal for the current day and one day in advance. Additionally, there are vessel tracking systems that will show the

location of every vessel in and around the canal. The vessels name, dimensions, and velocity and bearing all are available.

Dredging International must allow a minimum clearance of 122m in the navigation channel during “one-way” periods. This means that the prism lines outside of the navigation channel will still be available to work in. However the navigation channel will not be accessible for work during these times. This prevents sectors II and III from being dredged during one-way traffic hours. During times when “two-way” traffic is passing through the Pacific Entrance Channel, no work can be done, severely limiting production hours for Dredging International. Areas that are not easily accessible are dredged when there is no traffic on the canal, giving Dredging International the necessary time to maneuver through the access channel. As the project wore on and the canal widened, the dredge was able to work on the edges of the channel while traffic was passing through.

Daily traffic schedules are only posted one day in advanced, giving the contractor a very limited timeframe to plan their daily operations. Coordination from traffic control, the contractor, and the ACP dredging division is paramount for the timely completion of this project. There is always an ACP pilot aboard the dredge to make decisions on behalf of the ACP and to ensure that the dredge is not disrupting traffic.

Tide also plays a major role in the maintenance dredging operation. The water level in the entrance can fluctuate by about 5 meters due to the tide. This means that some shallower areas of the channel are inaccessible during certain points of the day, forcing the contractor to work on other areas. The tide also effects how the contractor dumps material at the disposal site. In order to ensure that the sediment is evenly distributed throughout the site, tide and currents must be considered. Strong currents could carry sediment into different parts of the dumping site or even out of the site completely.

Dredging International is only contracted to remove sedimentary materials. While the trailing suction hopper dredge excels at excavating sand and other loose materials, it struggles to handle rock. As the contractor digs deeper, they face harder materials. In some cases in order to reach the design depth it may be necessary to excavate some rocks and larger aggregates. Because the dredger was not designed to handle these materials the process is slow and it production.

Delays also significantly impacted production values. Pacific entrance Channel must remain open and operational while the project is underway. This means that the dredging must not at all interfere with the daily operations of the channel. Our team was able track not only the delays to the project, but the cause of the delays as well.

Most of the delays are under 20 minutes, which in a 24 hour operation is only a minor setback. However, there are some outliers. On November 15th operation was halted for about 15 hours due to maintenance on the engines. On the 21st and 22nd more minor maintenance was done, thus delaying production.

Overall about 57 hours in production time is lost due to delays. While a vast majority of the delays are due to traffic issues some are also caused by maintenance, pilot changes, and collisions.

The Breughel dredger is equipped with an overflow valve that can be used to increase the slurry density. Since the sediment has a higher density then water, as the hopper becomes full with slurry the top levels that have low concentrations of sediment are released back into the channel. This technique is used to increase the productivity of a dredger as the hopper will hold slurry with a higher content of sediment, the volume of which is used to measure productivity. With the maintenance dredging of the Canal however the use of an overflow is nearly impossible. Since the sediment being collected by the trailing suction draghead is of a relatively low density, it takes a long time to settle. If the overflow were to be

used, the density of the slurry exiting the hopper would have a water content that is not much higher than the original slurry. Additionally, environmental concerns of dumping slurry with higher sediment contents means the Breughel must first seek approval before using the overflow valve. These two problems compound each other and make the use of an overflow valve impractical for the maintenance dredging project.

The second aspect of the operation is sailing. We combined the sailing from the site to discharge area and sailing from discharge area back to the site as one component due to their similarities. Though the options in this process are very limited, it was still necessary to explore any way for the sailing aspect to become more efficient. To improve the time it takes to move from work site to discharge area, we focused on ways to improve sailing speed and minimizing sailing distance.

The sailing speed discussed is not dependent on the distance as it is the velocity of the vessel and not the time it takes to travel. It is also dependent on many external factors including Canal restrictions, vessel traffic, and weather conditions. The sailing speed of the Breughel is a fixed speed and any change in efficiency from going faster would be negligible.

The current distance between the dredging site and discharge area is a fixed length. For the current maintenance project, discharging at sea is the most feasible option. The only way to reduce the sailing distance would be to have a different method of discharge.

Another area that has potential for improvement is the discharge of sediment from the dredger. The operation currently uses a dump site in the Pacific Ocean, meaning the vessel must sail to and from this site to discharge sediment. To improve sailing time between these two sites, alternative means of discharge must be considered. In addition to its bottom door, the Breughel also has the ability to pump its sediment to other barges for transportation or use a pump to shoot the slurry back to land for reclamation.

An additional way to transport sediment from the project site is by use of a barge. By pumping slurry into the barge instead of the dredger's hopper, there will be no waste of time in sailing to the discharge area. If the Breughel used a barge, they would save over 45 minutes that is required to sail and unload the hopper.

When considering the implications of using a barge, the benefits begin to diminish. Since the Breughel must dredge while navigating a busy channel, having an additional vessel would only complicate matters. The cost of having a barge would also offset any savings from the increase dredging production. Finally, given the size of the project and the time allotted for completion, there is no pressure to increase dredging production, especially at the cost of adding vessels and their crew to the project.

Development of As-Built Stitch Grouting Drawings for Borinquen Dam 1E

Background

The stability of the foundation of Borinquen Dam 1E is extremely important since the dam obliquely crosses the Pedro Miguel Fault, which is one of the largest faults in the world. For this reason, it is

important to ensure that the Dam 1E does not fail. There are several reasons why dams may fail. These include:

1. Poor Site Selection
2. Poor Design/Construction
3. Seismic Activity
4. Impact/Collision

The Figure 1 below displays the placement of the Borinquen Dam 1E over both the Pedro Miguel and Limon Faults.

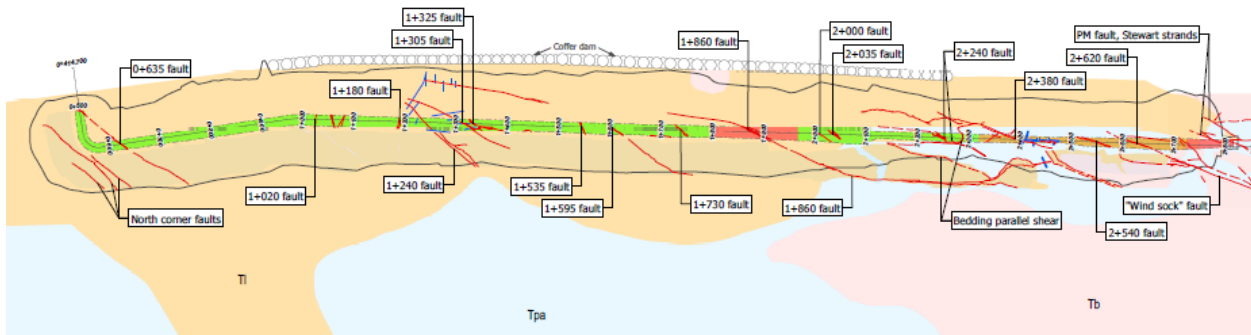


Figure 1: Faults and Shear Zones Located within the Footprint of Borinquen Dam 1E (URS Holding, Inc., 2009)

Boring logs along the entire length of the dam indicate the presence of crushed rock, sheared and altered as a result of rock movement. Therefore, the design of the Dam 1E posed challenges including (United States Society on Dams, 2011):

1. Variable foundation conditions with occasional weak features;
2. A high seismic hazard, including possible surface fault rupture across the dam foundations; and
3. Potential for grounding of Post-Panamax-size ships against the inboard face of the dams.

For these reasons, it is important that the dam foundation has sufficient strength for static and seismic stability (URS Holdings, Inc., 2009). The Table 1 below details the stability criteria for Dam 1E. Seismic dam deformation must not compromise the ability of the structure to retain the Gatun Lake, lead to overtopping, or require emergency response that impedes the operation of the canal.

Table 1: Embankment Stability Criteria (URS Holdings, Inc., 2009)

Load Condition	Slopes	Water Surface Elevation		Minimum Acceptable Factor of Safety
		Inboard Side	Outboard Side	
End of and During Construction	Inboard and Outboard	Empty	Maximum Operating Level	1.3
Long Terms, Steady Seepage	Outboard	Maximum Operating Level	Maximum Operating Level	1.5

Maximum Flood Surchage Pool	Outboard	New Locks Design Flood Level	Maximum Operating Level	1.4
Rapid Drawdown	Inboard	Maximum Operating Level	Maximum Operating Level	1.3

In order to fulfill the criteria previous stated for the foundation of Dam 1E, several steps need to be taken. Firstly, the area needs to be excavated until sound rock is found and that surface needs to be treated. Next, the area needs to be properly dewatered, and cutoff walls installed in the necessary areas. Lastly, grout curtains need to be created to control seepage (URS Holdings, Inc., 2009).

Seepage will be controlled using foundation grouting throughout the entire dam. Cement slurry or chemicals are forced into grout holes under pressure into the rock defects including joints, fractures, bedding partings and faults. Grouting aims to accomplish the following (Fell, 2005):

1. Reduce leakage through the dam foundation;
2. Reduce seepage erosion potential;
3. Reduce uplift pressures; and
4. Reduce settlements in the foundation.

Foundation grouting takes two forms: Curtain Grouting and Consolidation (Fell, 2005). Curtain grouting, specifically permeation grouting is used in the Borinquen Dam 1E. Permeation grouting functions by creating a narrow barrier or curtain in highly permeable rock. This grouting method usually consists of a single row of grout holes which are drilled and grouted to the base of the permeable rock.

In areas where shear zones are present in the foundation, additional measures need to be taken to ensure that the foundation meets the stability criteria set by the ACP. These efforts are referred to as stitch grouting. Stitch grouting uses angled fans of grout holes that are crisscrossed at various depths and locations (Weaver, 2007)

The grout mixture used consists of Type III Portland cement, superplasticizer, bentonite and water. Table 2 below demonstrates the combinations of different materials for the grout mixtures.

Table 2: Grout Mixture Components and Quantities (URS Holdings, Inc., 2009)

	Mix 1	Mix 2	Mix3	Mix 4	Mix 5
Water (Lts.)	55	30	21	13	16
Cement (kg)	85	85	85	85	85
6% Bentonite Soln. (Lts.)	48	48	48	48	45
R-1000 Additive (kg)	3.55	2.12	2.12	1.918	1.692
Density (g/cm³)	1.44 – 1.46	1.55 – 1.57	1.60 – 1.63	1.66 - 1.68	1.66 - 1.68
Viscosity *s for 100 cm³ sample)	≤35	40 - 50	60 - 70	>95	≤118

2-Hour Settlement	<2%	<2%	<2%	<2%	<2%
--------------------------	-----	-----	-----	-----	-----

Methodology

The goal for this project was to design a simple set of instructions for producing As-Built drawings for the stitch grout holes drilled and grouted in the construction of the foundation for Dam 1E. To do this, four objectives were developed. These include:

Objective 1: Become well versed on the terminologies and calculations

Objective 2: Identify internal resources available

Objective 3: Become familiarized on the databases provided

Objective 4: Determine the layout of the final drawings

Several interviews and Document Analyses were conducted to gather information to successfully accomplish these objectives.

Findings and Recommendations

During this internship, several facts have been discovered regarding grouting and the production of the As-Built drawings. These findings include:

1. Grouting is performed in stages no longer than 6 meters to more effectively target fractures and other defects in the rock
2. Mix 1 is the most common mix being used
3. Results from Lugeon tests and grout take values in verification holes are used to prove the effectiveness of foundation grouting on the rock masses
4. Remediation holes may also be drilled and grouted if verification testing shows that the initial production grouting was ineffective
5. All drawings should be at a scale of 1:100 on a 22"x34" sheet to make the drawings readable at half size (11" x 17")
6. The Deere Classification is used to color code the As-Built drawings based on grout take
7. The extensive grouting data from thousands of holes can be more easily interpreted when sorted by Row and then by Station
8. Approach used to create As-Built drawings from an Excel database using an AutoCAD Visual Basic macro
9. The instruction manual used to produce the As-Built drawings for stitch grout areas worked very well and proved to be more efficient than manually preparing the drawings directly with AutoCAD

The only recommendation from the findings previously stated, is to use the Instruction Manual created from information gathered from this internship. This Instruction Manual can be found in Appendix A.

Conclusion

This instruction manual, entitled "Process Manual for Generating As-Built Drawings for Stitch and Production Grouting" aims to be extremely simple and reduce production time dramatically. In order to improve the efficiency of the production of these drawings, it was advised to follow this process since it provides insight on how to manipulate the massive database of grouting information.

Embankment Construction: Compaction of Zone 1 Materials

Introduction

The Panama Canal Authority (ACP) is proposing to construct four embankment dams, Borinquen Dams 1E, 2E, 1W, and 2W, as part of the Pacific Access Channel (PAC) that will connect and allow navigation from the Gaillard Cut section of the Panama Canal to the new Pacific Post-Panamax Locks. The function of Dam 1E is to retain the Gatun Lake.

The main purposes of this project is to: (1) select the most adequate type of structure to be used for Borinquen Dam 1E, (2) develop compaction requirements and testing specifications for Zone 1 clay core based on the results of Zone 1 test fill number 7, and (3) evaluate the actual compaction achieved in the field, based on analysis of test results, against project specifications and against personally designed specifications.

Background

Both earth embankment dams and concrete dams were taken into consideration for the construction of Borinquen Dam 1E. The main types of dams taken in consideration were: (1) zoned earthfill embankment dam, (2) central core earth and rockfill embankment dam, (3) roller compacted concrete dam (RCC), and (4) mass concrete gravity dam. The report addresses the main features of each dam along with their advantages and disadvantages.

The main design requirements specific to the construction of Borinquen Dam 1E that constrain the selection of alternative designs are: (1) foundation conditions, (2) availability of construction material, (3) static and seismic stability, (4) ship grounding, (5) seepage analysis considerations and (6) construction practicality in present environmental conditions.

The type of embankment dam used Borinquen Dam 1E is a central core earth and rockfill dam. The total embankment volume is estimated to be 4,920,000 m³, of which the earthfill clay core would be 460,000 m³. To prevent piping of the clayey residual soil core materials and to transfer seepage away from the dam embankment, an arrangement of filters and drains has been included in the design. The report outlines the description of the embankment zones, their functionalities, the materials available for construction at the site area and the quantities and types of materials needed to build the embankment zones.

Specific project constraints regarding Zone 1 material and its placement and compaction were discussed. The types of materials found, during burrow area investigations, for construction of the core were only residual soils formed through the weathering of the underlying bedrock. Not all of the excavated material will be suitable to be used in the embankment. Material will be lost during clearing and stripping operations. Oversized material might be encountered. Due to the to the high precipitation conditions some materials might have high water content values. Hence it will take time to process the materials to achieve adequate moisture content values. It will be challenging and it is critical to place materials in Zone 1 at adequate moisture contents. When compacting cohesive soils, their shear strength increases, compressibility and permeability decrease, but if compacting too wet of optimum these soil characteristics will not be achieved.

The central vertical earth core, Zone 1, is designed to be (1) impermeable to prevent seepage through the dam, (2) have sufficient strength to resist static, seismic and construction loads, (3) be sufficiently ductile and flexible to have the ability to accommodate for fault displacements, and (4) have an adequately low

compressibility to avoid potential damages due to future settlement. The report goes in to detail regarding what engineering soil properties need to be specified for a soil to be impermeable, strong, ductile and flexible and have low compressibility. The research found that what needs to be specified is: (1) the source and Soil Classification, (2) the maximum particle size and particle size distribution, (3) the Atterberg limits, (4) The shear strength, (5) the water content placement range and (6) the density ratio. The zone 1 key specification requirements are: (1) Material to be residual soil and not contain any organic material, (2) $PI > 10$, (3) 100% passing 6" (150mm) sieve, $\geq 70\%$ passing $\frac{3}{4}$ " (19mm) sieve, and $> 35\%$ passing No.200 (0.075 mm) sieve, (4) minimum undrained shear strength = 75 kPa, (5) Compaction water content range between +2% and +12% of OMC. The report further describes the procedures for material burrow excavations, placement, compaction and the ASTM testing methods and frequencies.

Methodology

To select the best possible type of dam structure for construction of Borinquen Dam 1E research on the proposed options was performed. Evaluation of each option against the design criteria, with discussion of advantages and disadvantages of each type, was performed. The best suitable option was chosen for construction of Borinquen Dam 1E.

New specifications for the compaction and testing requirements for Zone 1 earthfill were produced based on: research regarding the compaction of soils and testing methods, regarding the design of central core earth and rockfill embankment dams, regarding testing specifications and frequency criteria for clay cores, on knowledge of the constraints specific to the project site location, on the criteria requirements for Zone 1 and on analysis of results of Zone 1 test fill number 7. The objectives of Zone 1 test fill number 7 were to gain information on the engineering properties of the materials of the burrow areas

The compaction procedures and laboratory tests were supervised to determine if specifications were being followed correctly. The laboratory test results were analyzed to determine whether the compaction of zone 1 was meeting specifications and the personally developed specifications. Recommendations based on the findings were produced.

Finding and recommendations

Objective 1

After the comparison between the two embankment dams, it was decided that the central core and rockfill dam was the superior option. The central core earth and rockfill dam has more advantages compared to the zoned earthfill embankment and it is more compatible with the design criteria requirements.

A similar comparison between the two concrete dams was done as well. The RCC dam was the deemed to be the better option because of its adherence to the design criteria requirements.

After comparison with between the two remaining options the central core earth and rockfill embankment dam design was determined to be the ideal option. The decision was mainly based on the materials available at the site, the foundation's strength and the seismic displacement consideration. This decision coincides with the type of structure that is being built at present.

Objective 2

The developed specifications for Zone 1 are: (1) Material to be residual soil (MH and CH preferred, GM and SC alternatives), (2) 100% passing 3" (75mm) sieve, $\geq 70\%$ passing $\frac{3}{4}$ " (19mm) sieve and $> 25\%$ passing No.200 (0.075 mm) sieve, (3) $PI > 10$, (4) Minimum undrained shear strength = 75 kPa, and (5) Compaction water content range between +2% and +8% of OMC. It was determined that the density ratio is best to not be specified since compaction is going to be done at moisture content values high above optimum.

Specifications on testing frequencies were produced for quality control purposes; based on the research regarding common practices for testing frequencies and on the knowledge of the project's circumstances. The vane shear strength test was specified to be done once per lift per 100m horizontally and once per lift per 200m horizontally, thereafter. The moisture content test every shear strength test location. The testing frequency for the particle size, Atterberg limits and the optimum moisture content was chosen to be done every 1500 m³. The sand cone test for dry density was specified to be done to test the validity of the vane shear test result, whenever the field inspector feels it's needed and every 10,000 m³. All test were specified to be done following ASTM standards.

Objective 3

During the supervision of the Zone 1 compaction process it was found that occasionally the dozer was used for compaction instead of the tamping-foot compactor. The specifications recommend the latter, during evaluation of the test fills, since it was found to lead toward a better bonding between lifts.

The supervision of the tests being done in the laboratory concluded that the ASTM procedures were being followed correctly except for the procedure for determining the bulk density of the sand for the sand cone test. "Alternative method B" and not the "preferred method A" in the ASTM standard D 1556 Annex was being used. The "Preferred method A" was then performed and the results were determined to be more accurate. Hence, "preferred method A" was used from then on.

During field and laboratory supervisions of Zone 1 testing procedures, the vane shear test was acknowledged to be a more efficient quality control testing method compared to the sand cone test. Thus, testing for shear strength was determined to be a better choice for the primary quality control method.

All of the analyzed test results adequately met the project's specifications and the personally designed specifications. Except for the moisture content of the personally developed specifications, which was 0.8% above specification. Since all of the other test results complied with the specification, it was decided that the lift was adequately compacted and removal was not necessary.

88.9% compaction of Zone 1 was achieved. A percent compaction value was not specified. The value was recognized as a low value but still acceptable, since the percent compaction the test fill number 7 all were between 87 % & 98% and since most of the samples had achieved satisfactory shear strengths.

Conclusions

It was determined that the central core earth and rockfill embankment dam structure, which is being used to construct Borinquen dam 1E, is the best possible option for the projects circumstances. It was determined that the compaction requirements and testing specifications for quality control are accurate for the construction of Borinquen dam 1E. They are more suitable than the more restrictive personally developed specifications. It was determined that the compaction being achieved In Zone 1, meets the project's specifications and is satisfactory for the outcome of the core's functionalities.

It was recommended that the Zone 1 field compaction processes and the laboratory test procedures be carefully supervised. It was recommended that lab test results be precisely and critically evaluated before their approval. It was recommended to do the sand cone density test more often than what is currently being done, in order to test the validity of the vane shear strength result, to confirm that adequate compaction is being achieved and to know for certain that the outcome of the core's functionalities will be attained.

Borinquen Dam 1E Construction Management Process Analysis

Background

Construction management is a balance of time, quality, and cost. Thorough and early planning is the most effective technique in controlling the balance of cost and schedule of a project while ensuring a quality product. In order to develop an adequate plan, significant testing and surveys must be completed to optimize predictability. With adequate planning, variations may be reduced which can lead to significant cost savings (Cooper, 2008).

Several aspects of construction were investigated that may impact the quality and efficiency of the operation including but not limited to the following: equipment, hauling roads, construction schedule, and wage rates. The key aspect relevant to the cost analysis was the cost of rental equipment.

Rental Equipment

Construction equipment is expensive equity for a contracting business and rentals can be a better option. Purchasing and owning equipment is expensive upfront and may incur expensive maintenance. The contractor needs to have a large company with regular work in order to afford the expense without losing capital while the equipment is not being used. Additionally, because equipment is highly specialized for specific types of jobs in terms of size, weight, capacity, or specialty sensors or extensions the contractor may need a diverse fleet in order to ensure regular use of the equipment. Renting equipment gives the contractor more versatility with low up front cost. Renting equipment also give the contractor more flexibility to increase or decrease the fleet at various points in construction. A set rental rate also helps contractors prepare bids and make cost projections. The contractor also avoids storage and transportation costs and can rent the newest, most effective models.

The contractor at the Borinquen Dam is renting excavation, hauling, dozing, compaction, water, and several other pieces of equipment. The primary equipment used for each zone and at each stage of construction is summarized below with the associated rental rates.

Zone	Activity	# Fronts	Equipment	#	Rental Fee	
					\$/Day	\$/Month
1/ 1A	Source	2	CAT Excavator 336	2	\$60.00	\$480.00
	Stock Pile	2	Water Truck	2	\$66.73	\$533.80
			CAT Dozer D6	2	\$40.00	\$320.00
			Vibrating Compactor Hamm 3520	1	\$32.00	\$256.00
	Hauling	2	CAT 740	10	\$75.00	\$600.00
	Embankment	3	Water Truck	3	\$66.73	\$533.80
			Vibrating Compactor Hamm 3520	3	\$32.00	\$256.00
			CAT Dozer D6	6	\$40.00	\$320.00
Manual Compactor			12	\$4.00	\$32.00	
					\$1,374.65	\$10,997.00
3	Hauling	4	CAT 773	16	\$85.95	\$687.60
			CAT 740	3	\$75.00	\$600.00
	Embankment	4	CAT Dozer D8	6	\$75.00	\$600.00
			Vibrating Compactor Hamm 3520	6	\$32.00	\$256.00

			CAT Backhoe 416	3	\$27.00	\$216.00
			Water Truck	3	\$66.73	\$533.80
					\$2,328.39	\$163,147.09
4	Hauling	4	CAT 773	16	\$85.95	\$687.60
			CAT 740	3	\$75.00	\$600.00
	Embankment	1	CAT Excavator 336	1	\$60.00	\$480.00
			CAT Dozer D8	1	\$75.00	\$600.00
					\$1,735.20	\$177,899.22
3B, 3A, 5,6	Hauling	4	Small Trucks	27	\$10.00	\$80.00
	Embankment	3	Vibrating Compactor Hamm 3520	3	\$32.00	\$256.00
			Water Truck	3	\$66.73	\$533.80
			Manual Compactor	15	\$4.00	\$32.00
						\$431.19
Total					\$5,869.43	\$361,187.19

The following table compares the contractors submitted Base Line Schedule (BL7-3) for each material as compared to the most recent survey of placed material. The difference is noted in the right column.

Material	Placed	BL7-3	Diff.
Zona 1/1A	233,371.12	301,362.65	- 67,991.53
Zona 5	208,067.10	225,225.64	- 17,158.54
Zona 6	174,234.43	173,013.83	1,220.60
Zona 3A	11,309.29	17,318.77	- 6,009.48
Zona 3B	154,994.70	134,177.49	20,817.21
Zona 3	1,456,370.95	1,594,980.29	- 138,609.34
Zona 4	82,235.62	82,988.06	- 752.44
Backfill Total	335,787.00	442,827.75	- 107,040.75
Total	2,656,370.21	2,971,894.48	- 315,524.27

Methodology

The goal of this section of the project was to develop a process analysis of the construction of the Borinquen Dam 1E. This process analysis investigated key points in the operation including the excavation, processing, hauling, and construction of each zone of the Dam. From this analysis, potential points for improving production and cost efficiency were identified. In order to accomplish this goal, several objectives were developed including the following:

1. Gain an understanding of the construction process and identify central crutches to productivity;
2. Gain metrics on the current construction process at the identified crutches;
3. Analyze the efficiency of the current process and develop recommendations for the contractor at several key points including:
 - a. Excavation,
 - b. Stock Piling,
 - c. Construction,
 - d. Hauling; and
4. Project the time of completion and the associated cost at the current rate of construction and after recommendations.

In order to accomplish these objects, equipment, scheduling practices, and other construction management tools were researched. The researched information in addition to supportive information provided by ACP employees were used to develop a frame for the process analysis. Using that frame, field work and interviews could be effectively used to maximize resources and achieve the following objectives.

Findings and Recommendations

The following findings were identified through the course of this internship:

- Finding 1: The contractor is placing less material on the embankment than scheduled
- Finding 2: The contractor has not been placing Zone 1 and Zone 3 materials as scheduled over the past 5 months
- Finding 3: The contractor is not producing enough material for the filter layers to place as scheduled
- Finding 4: The stock piled zone 1 residual soil is being drawn from faster than excavation can restock it
- Finding 5: The two main zones that will most likely slow construction are zone 1 and zone 3
- Finding 6: Projected time of completion and most cost effective completion time

From these findings, recommendations for the optimization of the operation were developed. The recommendations are based around the observation of several key limitations for productivity including the Excavation of Zone 1 material, and placement of Zone 3 material on the embankment.

Zone 1:

- Recommendation 1: Improve Length and Quality of Hauling Route
- Recommendation 2: Increase Excavation Fronts
- Recommendation 3: Utilize Bull Dozers to Assist the Excavators

Zone 3:

- Recommendation 1: Widen placement planes at embankment to allow for increased maneuverability of hauling trucks and bull dozer.
- Recommendation 2: Use multiple bull dozers to expedite spreading of material
- Recommendation 3: Use Compactor to improve temporary construction road surface conditions
- Recommendation 4: Optimize number of hauling trucks and route distance to construction sites
- Recommendation 5: Increase Embankment construction sites to three fronts continuously: North, South, and Center

The below table summarizes the projected dates of completion based on several recommendations. The three dates are highlighted in red represent the three key projections. If the contractor continues to follow the current rate of production, construction may be completed May 4th, 2015, two months after the contractor's scheduled completion March 2015. The January 15th 2015 completion date is the optimal projected completion date assuming the contractor follows all the Zone 3 recommendations. The third date is the suggested balance of the two with a February 15th completion date.

Day of Completion (Start 12/1/2014)					
Fronts		2 Front		3 Front	
Shifts	Work Week	1 Dozer	2 Dozer	1 Dozer	2 Dozer
Day	6 Day Week	13-Feb-15	4-May-15	29-Jan-15	5-Mar-15
	7 Day Week	24-Aug-15	13-Apr-15	25-Jan-15	26-Feb-15
Day & Night	6 Day Week	4-May-15	16-Feb-15	5-Mar-15	21-Jan-15
	7 Day Week	13-Apr-15	4-Feb-15	26-Feb-15	13-Jan-15

Conclusion

It was projected that at the current rate of construction as observed over the past month, the date of completion may be May 4th 2015, two months after the contractor's submitted schedule. It was concluded that the superior combination of recommendations for best time and cost efficiency was for the contractor to increase the width of the placement planes to accommodate 2 dozers each at the two current fronts and increase the associated equipment fleet likewise for a February 15th completion date assuming a 6 day work week. This updated schedule is projected to cost the contractor \$1,179,833.68 starting from December 1st 2014.

Table of Contents

Contents

Authorship	2
Abstract.....	3
Executive Summary.....	4
Background	4
Methodology.....	4
Results and Discussion	4
Background	6
Methodology.....	9
Findings and Recommendations.....	9
Conclusion.....	9
Introduction	10
Background	10
Methodology.....	11
Finding and recommendations	11
Conclusions	12
Background	13
Methodology.....	14
Findings and Recommendations.....	15
Conclusion.....	16
Table of Contents.....	17
Table of Figures.....	22
Table of Tables	24
Table of Equations	26
Acknowledgements.....	27
Introduction	28
Literature Review	29
Background	29
Natural Conditions	30
Environment.....	31
Panama Canal Construction.....	31

The French Attempt	32
American Construction	33
Gradual Upgrading and Expansion Projects.....	34
Panama Canal Authority	36
The Fourth Treaty	36
The Panama Canal Commission	36
Organic Law.....	37
Panama Canal Authority Organizational Structure.....	38
Vision, Mission and Values.....	39
Expansion Program	39
Post-Panamax Locks.....	39
Dredging.....	40
Pacific Access Channel and the Boriquen Dam.....	41
Pacific Entrance Maintenance Dredging.....	42
Introduction	42
Background	44
Navigational Improvement Dredging.....	44
Bends.....	46
Basins and Maneuvering Areas.....	46
Trailing Suction Hopper Dredger	47
Site Characteristics and Conditions.....	48
Dredging Production Cycle.....	48
Dredging Production	51
Dredge Efficiency	52
Dredging Methodology	53
Objective 1: Determine Completion Time of Dredging Project.....	53
Objective 2: Identify Operation Improvements.....	55
Findings, Discussions, and Recommendations	57
Dredging Production Analysis	57
Investigation of Dredging Efficiency	62
Recommendation 1: The ACP should purchase a trailing suction hopper dredger to use on future maintenance dredging operations.....	77
Conclusion.....	79

Development of As-Built Stitch Grouting Drawings for Borinquen Dam 1E.....	80
Introduction	80
Background	81
Dam Failure	81
Foundation Design	83
Methodology.....	92
Objective 1: Become well versed on the terminologies and calculations	92
Objective 2: Identify internal resources available	93
Objective 3: Become familiarized on the databases provided	94
Objective 4: Determine the layout of the final drawings	94
Objective 5: Determine mode of process presentation	94
Project Restraints.....	94
Findings and Recommendations.....	96
Finding 1: Grouting is performed in stages no longer than 6 meters to more effectively target fractures and other defects in the rock	96
Finding 2: Mix 1 is the most common mix being used.....	98
Finding 3: Results from Lugeon tests and grout take values in verification holes are used to prove the effectiveness of foundation grouting on the rock masses	98
Finding 4: Remediation holes may also be drilled and grouted if verification testing shows that the initial production grouting was ineffective.....	101
Finding 5: All drawings should be at a scale of 1:100 on a 22"x34" sheet to make the drawings readable at half size (11" x 17").....	102
Finding 6: The Deere Classification is used to color code the As-Built drawings based on grout take	103
Finding 7: The extensive grouting data from thousands of holes can be more easily interpreted when sorted by Row and then by Station.....	103
Finding 8: Approach used to create As-Built drawings from an Excel database using an AutoCAD Visual Basic macro.....	104
Finding 9: A instruction manual would be the most effective mode of presentation for the process design.....	105
Finding 10: The instruction manual used to produce the As-Built drawings for stitch grout areas worked very well and proved to be more efficient than manually preparing the drawings directly with AutoCAD.....	105
Recommendations	105
Conclusion.....	106
Embankment Construction: Compaction of Zone 1 Materials	108

Introduction	108
Dam Structures	109
Design Requirements for the Embankment of Borinquen Dam 1E	115
Embankment Design	116
General design criteria.....	116
Design of Clay Core	121
Specific project constrains regarding the clay core	122
Criteria requirements for the Zone 1 Core	123
Specifications Required for Central Core Earth and Rockfill Dam Construction	127
Design specifications of clay core	128
Literature review on the compaction of soils and testing methods.....	128
Specifications for Zone 1 for Borinquen Dam 1E.....	130
Processing of Burrow Materials.....	131
Zone 1 lift placement and compaction procedures.....	131
Quality control	132
Methodology.....	134
Objectives.....	134
Objective 1: Select most adequate type of structure to be used for construction of Borinquen Dam 1E.....	134
Objective 2: Development of compaction requirements and testing specifications for Zone 1 clay core, based on the results of Zone 1 test fill number 7.....	136
Objective 3: Evaluation of actual compaction achieved in the field and comparaisn to project specifications and personally developed specifications	139
Findings and Recommendations.....	141
Objective 1: Selection of best type of dam structure to use for construction of Borinquen Dam 1E..	141
Finding 1: Selection of best type of embankment dam option	141
Finding 2: Selection of best type of concrete dam option.....	142
Finding 3: Selection of best type of dam structure option	142
Recommendations	143
Objective 2: Developed compaction requirements and testing specifications for Zone 1 clay core, based on the results of Zone 1 test fill number 7.....	143
Finding 1: Specified requirement for the source and Soil Classification of the core material	143
Finding 2: Specified requirement for the maximum particle size and particle size distribution.....	144
Finding 3: Specified requirement for the Atterberg limits	147

Finding 4: Specification requirement for the strength	149
Finding 5: specified requirements for water content and density ratio.....	150
Finding 6: Specification requirements for quality control	159
Summary of Zone 1 personally generated specifications	161
Recommendations	161
Objective 3: Evaluated Field and Laboratory Test Results.....	163
Finding 1: outcome of field compaction supervision.....	163
Finding 2: outcome of laboratory test supervision.....	163
Finding 3: Comparaision between vane shear test for strength and sand cone test for density.....	163
Finding 4: evaluation of whether the tested lift of Zone 1 met project specifications	164
Finding 5: evaluation of whether the tested lift of Zone 1 met personally developed project specifications.....	165
Finding 6: degree of compaction achieved in tested Zone 1 lift material	167
Recommendations	171
Conclusion.....	173
Borinquen Dam 1E Construction Management Process Analysis.....	174
Introduction	174
Background	174
Methodology.....	189
Results.....	193
Recommendations	202
Conclusions	209
Conclusion.....	209
References	211
Appendix A: Summary of the main critical design criteria requirements specific to the site and construction of Borinquen Dam 1E.....	216
Table comparing advantages and disadvantages of types of embankment dams available for the construction of Borinquen Dam 1E.....	217
Main features of concrete dam structures	218
Appendix B: Summary of the Contractor’s Lab Test Results	220
Test Fill Number 7 Results.....	220
Summary of the Contractor’s Lab Test Results.....	221
Summary of the ACP’s Lab Test Results.....	222
Summary of Zone 1 personally generated specifications	222

Appendix C: Summary of Zone 1 specifications.....	223
Tests performed.....	223
Test Results	223
Summary of Zone 1 specifications	224
Appendix D: Delay Summary	225
Appendix E: Dredging Sectors.....	226
Appendix F: Dredging Operations Summary.....	227
Appendix G: Delay Classification.....	228
Appendix H: Questions for Dredging International	229
Appendix I: Contractor’s Schedule.....	231
Appendix J: Surveyed Embankment Construction.....	233
Appendix K: Instruction Manual for Generating As-Built Drawings for Stitch and Production Grouting.....	235
Step 1: Sort Injection Database by Row and then by Station	235
Step 2: Create the Master Grout file.....	235
Step 3: Record Ground Surface Elevation for each hole.....	236
Step 4: Create smaller spreadsheets with drilling and grouting information for each line.....	238
Step5: Run Visual BASIC Macro.....	239
Step 6: Populate the Master Grout file.....	241
Step 7: Layout drawings on a 22”x34” sheet	242
Appendix L: As-Built Drawings Produced.....	244

Table of Figures

Figure 1: Faults and Shear Zones Located within the Footprint of Borinquen Dam 1E (URS Holding, Inc., 2009)	7
Figure 2: Geographic Map of Panama (452 Encyclopedia of world Geography).....	29
Figure 3: Panama Canal Trade Routes (Panama Canal Authority, 2014).....	30
Figure 4: Rendered Drawing of Post-Panamax Locks	40
Figure 5: Connection between Lock Chambers and Water Saving Basins.....	40
Figure 6: Flattening of the Paraiso Hill.....	41
Figure 7: Relocation of the new Boriquen Road	41
Figure 8: Brueghel Trailer Suction Hopper Dredge (Dredging International, 2014)	43
Figure 9: Daily Report Example from Dredging International	54
Figure 10: Sample Schedule for Dredging Operations.....	54

Figure 11: Average Volume per Trip	61
Figure 12: Total Volume per Day	62
Figure 13: Dredging Sectors of the Pacific Entrance of the Panama Canal	63
Figure 14: Tortolita South Dumping Site and Access Channel.....	64
Figure 15: Vessel Information Traveling the Panama Canal (Shiptracking AIS, 2014).....	67
Figure 16: Average Delay per Day.....	71
Figure 17: Total Delay per Day.....	72
Figure 18: Average Sail Time	74
Figure 19: Faults and Shear Zones Located within the Footprint of Borinquen Dam 1E (URS Holding, Inc., 2009)	84
Figure 20: Upstage Grouting Procedure (Fell, 2005)	97
Figure 21: Grouting Downstage with Packer Procedure (Fell, 2005).....	98
Figure 22: Configuration of Lugeon Test (United States Society on Dams, 2010).....	99
Figure 23: Location of Production, Verification and, Remediation Holes.....	102
Figure 24: Examples of Earthfill Dams (Fell, 2005)	110
Figure 25: Example of Central Core Earth and Rockfill Dams (Fell, 2005)	111
Figure 26: Cross Section of a Typical Straight Mass Concrete Gravity Dam (Fell, 2005)	112
Figure 27: Rolled Concrete Dam (Department of Water Affairs and Forestry, 2014)	115
Figure 28: Cross Section of Borinquen Dam 1E (URS Holdings, Inc., 2009)	117
Figure 29: Permeability and Laboratory Testing Method for the Main Soil Types (Fell, 2005).....	125
Figure 30: Contractor Test Fill Number 7 Gradation Tests Results Plotted Against the Specified Gradation Requirement	146
Figure 31: ACP Test Fill Number 7 Gradation Tests Gradation Test Results Plotted Against the Specified Gradation Requirement	146
Figure 32: Plot of Plasticity Index and Liquid Limit with plasticity index requirement.....	148
Figure 33: Plot of Plasticity Index and Liquid Limit with suggested liquid limit requirement	148
Figure 34: Contractor Test Results, Plot Of Shear Strength Vs Moisture Content	151
Figure 35: Contractor Test Results, Plot Of Shear Strength Vs Sand Cone Moisture Content	151
Figure 36: ACP Test Results, Plot Of Shear Strength Vs Moisture Content	152
Figure 37: ACP Test Results, Plot Of Shear Strength Vs Sand Cone Moisture Content	152
Figure 38: Contractor Test Results, Plot Of Shear Strength Vs Compaction.....	153
Figure 39: ACP test results, plot of shear strength vs compaction	153
Figure 40: Plot of Maximum Dry Density vs Optimum Moisture Content achieved in the laboratory following ASTM D 698, standard test method for laboratory compaction characteristics of soil using standard effort	154
Figure 41: Proctor Compaction Curve with Point Showing the Dry Density Achieved In the Compacted Soil Lift.....	168
Figure 42: Percent Compaction Vs Water Content Of The Soil Samples Used To Generate The Laboratory Compaction Curve And Of The Tested Sample In The Field, With The Sand Cone Test.....	169
Figure 43: Correlation between Shear Strength and Dry Density of Test Fill Number 7 Results and Compacted Clay Core Test Results.....	170

Figure 44: Correlation between Shear Strength and Percent Compaction of Test Fill Number 7 Results and Compacted Clay Core Test Results.....	170
Figure 45: RIBA Plan of Work (Cooper, 2008).....	175
Figure 46: Excavator 336 D Schematic ("CATERPILLAR 336D L HYDRAULIC EXCAVATOR", 2007).....	178
Figure 47: 773 Rock Mover Schematic ("CATERPILLAR 773 ROCK MOVER", 2007).....	179
Figure 48: 740 Articulated Dump Truck Schematic ("CATERPILLAR 740 ARTICULATED DUMP TRUCK", 2007)	180
Figure 49: D6 Dozer Specification ("CATERPILLAR D6 EARTH MOVER", 2007).....	181
Figure 50: D8 Dozer Specifications ("CATERPILLAR D8 EARTH MOVER", 2007)	182
Figure 51: Sheepsfoot Roller (Sheepsfoot Roller, 2014).....	183
Figure 52: Full Hydraulic Vibrating Roller (Vibrating Roller, 2014)	184
Figure 53: Wacker Neuson Plates & Rammer (Plates & Rammer, 2014)	184
Figure 54: Smooth Wheeled Roller (Smooth Wheeled Roller, 2014)	185
Figure 55: Observational Vantage Points.....	190
Figure 56: Construction Rockfill Schedule	194
Figure 57: Construction Residual Soil Schedule.....	195
Figure 58: Filter Layer Zone 3B Embankment Construction	195
Figure 59: Filter Layer Zone 5 Embankment Construction	196
Figure 60: Filter Layer Zone 6 Embankment Construction	196
Figure 61: Filter Layer Zone 3A Embankment Construction	197
Figure 62: Projected days to completion	201
Figure 63: Current and Proposed Hauling Routes	203
Figure 64: Current Travel Route from Arial View.....	204
Figure 65: Miraflores II Geological Survey	205
Figure 66: Zone 3 Hauling Route Quality	207

Table of Tables

Table 1: Embankment Stability Criteria (URS Holdings, Inc., 2009).....	7
Table 2: Grout Mixture Components and Quantities (URS Holdings, Inc., 2009).....	8
Table 3: Comparing Angle Of Deflection Vs Minimum Bend Radius (Bray, 2014).....	46
Table 4: Turning Basins	46
Table 5: Various Discharge Methods for Trailing Suction Hopper Dredgers (Bray, 2014).....	50
Table 6: Constants for Production Equation (Turner, 1984).....	51
Table 7: Specifications of Breughel Dredger.....	65
Table 8: Pacific Entrance Channel Operating Restrictions.....	69
Table 9: Summary of Unnecessary Features.....	77
Table 10: Summary of Needed Dredging Features.....	78
Table 11: Embankment Stability Criteria (URS Holdings, Inc., 2009).....	85
Table 12: Grout Mixture Components and Quantities (URS Holdings, Inc., 2009).....	90
Table 13: Typical Grout Stages Used in Borinquen Dam 1E.....	96

Table 14: Association of Lugeon Values and Properties and Conditions of Rock Masses (United States Society on Dams, 2010)	99
Table 15: Effectiveness of Foundation Grouting in Terms of Grout Take and Lugeon Values	100
Table 16: Deere Classification System Based on Grout Take (URS Corp. Inc., 2009).....	103
Table 17: Features of Zoned Earthfill Embankment Dam (Fell, 2005).....	109
Table 18 Features of Central Core Earth and Rockfill Dam (Fell, 2005).....	111
Table 19: Modes of Failure for Concrete Gravity Dams (Fell, 2005).....	113
Table 20: Embankment Dam Zones Description and Function (Fell, 2005).....	118
Table 21: Embankment Dam Typical Construction Material (Fell, 2005)	118
Table 22: Materials Available For Dam Construction (URS Holdings, Inc., 2009).....	121
Table 23: Summary of volumes of PAC-4 Excavated Material and recommended Embankment use (URS Holdings, Inc., 2009).....	121
Table 24: Zone 1 Material Gradation Requirements (Acp, 2009).....	130
Table 25: Zone 1 Embankment Materials - Quality Control Laboratory and Field Testing (ACP, 2009)...	133
Table 26: Water Content Range, Optimum Moisture Content and Percent Compaction of the Samples That Failed and Passed the Shear Strength Requirement	149
Table 27: Ranges and Averages of the Optimum Moisture Contents and the Maximum Dry Densities Achieved In the Laboratory.....	155
Table 28: Contractor and ACP Test Result Average Moisture Contents from Moisture Content Test and Sand Cone Test.....	155
Table 29: ACP and Contractor Test Fill Number 7 Average Undrained Shear Strengths, the Moisture Contents, the Optimum Moisture Content, the Percent Compaction and the Differences of the Various Moisture Contents Form the Optimum	156
Table 30: Quality Control Laboratory and Field Testing Types, Frequencies and ASTM Standards, For Zone 1 of the Embankment	159
Table 31: Minimum Required Frequency of Construction Testing For Zone 1 Earthfill (Fell, 2005)	160
Table 32: Shear Strength.....	164
Table 33: Plasticity Index	164
Table 34: Material Gradation.....	164
Table 35: Water Content.....	165
Table 36: Soil Classification.....	165
Table 37: Shear Strength.....	166
Table 38: Plasticity Index	166
Table 39: Material Gradation.....	166
Table 40: Moisture Content	166
Table 41: Soil Classification.....	167
Table 42: Scheduled Placement of Material.....	193
Table 43: Filter Layers Production Rate and Construction Mass Balance	197
Table 44: Zone 1 Stock Pile Mass Balance	199
Table 45: Projected Date of Completion.....	201
Table 46: Projected Construction Cost	202
Table 47: Efficiency of Hauling Cycle	208

Table 48: zoned earthfill and central core earth and rockfill embankment dam main features (Fell, 2005)	217
Table 49: results of Zone 1 test fill number 7	220
Table 50: Water Content and Shear Vane Test Results	223
Table 51: Gradation Test Results of the 3 Main Sieves Adressed In The Specifications	223
Table 52: Liquid limit (LL), Plastic Limit (PL) and Plasticity Index (PI) test results	223
Table 53: Proctor Compaction Test and Sand Cone Test Results	224
Table 54: Soil Classification test result	224

Table of Equations

Equation 1: Dredge Law I	51
Equation 2: Dredge Efficiency (Bray, 1978)	52

Acknowledgements

From the very beginning of our project, we have been supported and guided by many people throughout the course of completing our research. We are very thankful for the amount of time, encouragement, and support we received. Our project would not have been possible without many of these people.

From Worcester Polytechnic Institute, we would like to first thank our advisors, Dr. Tahar El Korchi PhD and Dr. Aaron Sakulich PhD for their guidance as we worked on our project. From the ACP, we would like to thank Project Manager, Jorge Fernandez, Quality Assurance Manager, Pedro Lopez and Contract Officer Representative, Victor Wong for welcoming our project group into the Marieta and Corazol offices and assisting us in any way possible along the duration of our project.

Lastly, our project group would like to thank the consultants from URS Corp. Inc., Geotechnical Engineers, Dr. Ramon Martinez and James Toose who offered their help and shared their knowledge on numerous complex topics.

Introduction

The Panama Canal is one of the main passageways connecting the Northern and Southern Hemispheres. Spanning the canal is the Gatun Lake, one of the largest man-made fresh water lakes in the world, which functions as the navigational channel for the canal. Since September 2007, the Autoridad del Canal de Panama (ACP; Panama Canal Authority) has been expanding the canal's operations by adding a new, larger set of locks than can now accommodate massive Post-Panamax ships. To access this new set of locks, the Borinquen Dams will create a Pacific Access Channel separating the Gatun Lake from the Miraflores Lake. This project investigates four matters relating to both the maintenance of the current navigational channels and the excavation and construction of the Borinquen Dam 1E.

The first project examines the efficiency of the dredging operations for the area encompassing the Miraflores Reach. The Miraflores Reach includes the area from the Pacific entrance of the canal to the Miraflores Locks. Recommendations for increased functionality will be made from research done during this project.

The final 3 projects are related to the Borinquen Dam 1E. The first of the Dam 1E projects involves the production of the As-Built drawings for the foundation stitch grouting areas along the alignment of the Dam 1E. Information gained on site was assessed to formulate the most efficient procedure to produce 3D profile drawings of the foundation grouting. The next project involves analyzing the current design and specifications and formulating improved design specifications based on in-field testing. The original design and specifications were compared to see the difference in recommended design criteria and overall design and of the Dam 1E. The final Dam 1E project involves a progress analysis of the Dam 1E construction. Analyses were done to estimate the projected cost and time to completion based on several recommended improvements to routes and equipment scheduling.

Ultimately, recommendations were drawn and conclusions made to design improved methods for the completion of each respective task. The chapters that follow are separated based on individual projects. Each project contains background information necessary to understand major concepts regarding each project, a method of data collection and, a composition of findings, discussions and recommendations. Finally, conclusions were drawn for the overall predominant recommendations made from the combination of all four projects.

Literature Review

Background



Figure 2: Geographic Map of Panama (452 Encyclopedia of world Geography)

Panama is a small but important Central American country between Costa Rica and Colombia connecting South America and North America. The Panama Canal cuts across the smallest width of the S shaped country connecting the Pacific and the Atlantic Oceans and making the country extremely important to maritime trade as seen in the figure below.



Figure 3: Panama Canal Trade Routes (Panama Canal Authority, 2014)

The Gatun Lake, one of the largest manmade lakes in the world, was created specifically for the Panama Canal in order to tame the Chagres River for easier use in the lock design of the canal. It is located across this neck which only spans 81.6 km (50.7 mi).

Natural Conditions

Panama is a mountainous region dominated by the continental divide which was created by the Central American volcanic belt. The region lies along the divide where the Cocos Plate has cut under the North American and Caribbean Plates, creating a line of volcanos from Guatemala to Panama. Mountains run along both the Pacific and Caribbean coast lines east of the Panama Canal. West of the Canal are the Serrania de Tabasara Mountains, which run along the spine of the country with narrow coastal planes to the North and South sides.

Panama has a very tropical climate as it is positioned just above the equator at 9° N. Throughout the year it is hot and humid with temperatures ranging from 26°C to 27° C. The average precipitation is 177cm (69.7in) with a rainy season May through November. Most of the rain falls in short, concentrated daily showers. The climate is dominated by the northeast trade winds, which in contact with the mountains causes the rainy season in the winter and summer. The rainy season is supported by the belt of low atmospheric pressure which typically lies over the equator but travels north to Panama from May through July. South of the mountains, on the pacific side of the country, Panama experiences a rain shadow with a drier climate.

Environment

Due to the hot, wet conditions, the country is dominated by tropical rainforest and is rich with plant and animal life. Approximately 40 percent of the country is forested, with the exception of the Canal Zone which has experienced a 50 percent reduction in woodland since the 1940s. The forest that intercepted rainfall and stabilized the soil surrounding the Gatun Lake in 1952 has since disappeared, leaving bare hills which are much more susceptible to severe soil erosion. The lake is now threatened by heavy siltation from the stripped soil as well as sewage and industrial pollution.

The Panama Canal Authority, as part of the Expansion Program, identified several potential environmental impacts including, but not limited to loss of vegetative cover, deforestation, disturbance of wildlife and loss of habitat, alteration of aquatic environment and resources ("Environmental Impact Study- Category III", 2007). Several of these impacts, especially deforestation may lead to potential risks including micro climate change, loss of potential carbon capture, deterioration of air quality, increase noise and vibration pollution, increased odor, increase landslide and cave in risks, increased soil erosion, increased sedimentation, soil compaction and pollution, deterioration of water quality.

Through recognizing all potential risks, the ACP is better prepared to combat and mitigate these impacts. The ACP developed a Mitigation Plan that includes continuous monitoring of impact, a mitigation plan for all potential environmental degradation, and a follow up plan to ensure effective mitigation. As an example, for every hectare of deforestation during the construction process, two hectares of forests will be planted to mitigate the potential air quality effects.

Panama Canal Construction

The Isthmus of Panama has been an important maritime trade and transportation opportunity since its original discovery. The Isthmus, which had been populated by indigenous population for hundreds of

years, was first traversed in 1513 by Vasco Nuñez de Balboa. Balboa was unaware of the thin width of the Isthmus, but had rather been in search of increased gold profit from neighboring indigenous tribes. The Holy Roman Emperor Charles V first commissioned the development of a passageway across the isthmus with a land survey in 1534. The planned route followed the Chagres River similar to the route used today but, at the time of the survey, safe passage via maritime travel was judged impossible by the Panama regional governor. It wasn't until much later that

The French Attempt

The French were the first to attempt to build a waterway connecting the Atlantic Ocean to the Pacific. After having recently completed the Suez Canal in 1869, the French government saw the economic opportunity in a Panamanian Canal. The government appointed the same Chief Engineer Ferdinand de Lesseps who had worked on the Suez Canal to lead “La Société Internationale du Canal Interocéanique” (The International Society of the Interoceanic Canal, SICI). Ferdinand de Lesseps, without having done a geotechnical assessment of the land, assumed that the Panama Canal would require a similar design to the Suez Canal. The simple excavation method used for the Suez Canal was catered to the even elevation and sandy soil type of the Isthmus of Suez. However, the topography of Panama is much more mountainous with its lowest points well above sea level. This misjudgment, in addition to a poor understanding of the climate, caused a plethora of problems for de Lesseps and his company.

Panama has a tropical maritime climate with two seasons, rainy and dry. The dry season lasts from January to May while the rainy season is far more prolonged and extends from May to January. Throughout the rainy season, the team was faced with frequent floods and mudslides. One of the most serious threats to the completion of the Canal was tropical diseases common to the region such as malaria and yellow fever. At this time, little was known about how these diseases were contracted, and thus little was done to impede the epidemic. Due to deaths from injury and disease, a steady and competent workforce was challenging to maintain.

The overall stubbornness of Lesseps to disregard the engineering surveys and his cohorts greatly hindered the progress of the Canal. The decision to construct a sea-level Canal through the rocky Central American region doomed the project to fail. In 1881 the project was estimated to cost a total of \$120,000,000, equivalent to \$2.85 billion today, and take 6 years to complete- a full 4 years shorter than it took to construct the Suez Canal. These estimates were highly unrealistic and gave false hope to the success of the project. Soon after construction began, the design flaws became apparent and the original estimates

were found to be impractical. Regardless, the company continued to press on. After 8 years of overspending and over 22,000 deaths, SICI went bankrupt after having only completed approximately 40 percent of the Canal.

In 1892 it became clear that de Lesseps was hiding the hardships from the French in order to maintain his image in the public eye. De Lesseps bribed French officials to conceal his company's financial status from the public. When this came to light, it ruined several political careers and caused the collapse of SICI.

After the scandal, the New French Canal Company was created to complete the Canal. The new company realized that the sea level Canal was highly impractical and decided to build a two-level, lock-based Canal. By this point, however, the project was irredeemable and the French began to look for a buyer in 1902.

American Construction

Theodore Roosevelt was elected President of the United States of America in 1901. President Roosevelt saw potential in a passageway across Central America as a key strategic hold for the United States. Through the Spooner Act, the United States purchased the assets of the New French Canal Authority for \$40,000,000 in 1903, roughly \$1 billion today, giving the U.S. access to the land the French had previously owned.

The Hay-Herran Treaty signed on January 22nd, 1903 was ratified by the United States Senate, but not by the Senate of Colombia. If the treaty had been ratified, it would have allowed the United States a lease on the Isthmus of Panama or the eventual construction of the Panama Canal. Rather than attempting the long and tedious process of renegotiating the treaty with Colombia, the U.S. began to back a separatist movement in Panama. Hoping that an independent Panama would allow the Americans to complete the Canal, the United States supported, both politically and militarily, the planned uprising in Panama which led to its independence. After a successful secession, on November 3rd 1903, the Hay-Bunau-Varilla Treaty was signed on November 18, 1903, just 15 days later. The treaty was negotiated by John Hay and Philippe Bunau-Varilla then sent to Panama to be ratified. This treaty gave the United States control over the Panama Canal Zone in 1903. Because of Panama's newly founded independence and complete lack of government, military, or resources, they had little choice in ratifying the treaty since refusal would have meant the withdrawal of U.S. support (ABPanama, 2014).

The Americans had a daunting task ahead of them in trying to complete the Canal. John Frank Stevens was selected as Chief Engineer of the project. Stevens recognized the oversights of SICI and supported a lock design for the Canal. He improved the sanitation of the area, reducing the spread of disease.

William Crawford Gorgas was appointed Chief Sanitation Officer and greatly improved the living conditions of the workers and natives. Gorgas focused his efforts on ridding the Canal Zone of mosquitoes, the carrier of malaria and yellow fever. He began by reducing sources of stagnant water in order to prevent mosquitoes from laying eggs. When draining stagnant water was not possible, Gorgas added oil or pesticides to standing bodies of water. Bug screens were added to all buildings to prevent mosquitoes from entering. If a worker were to contract a tropical disease he was immediately quarantined and his living quarters would be fumigated in order to stop further spread. Through his efforts the number of deaths due to tropical disease was greatly reduced and yellow fever was virtually wiped out in the Canal Zone.

Stevens also recruited a large, more sustainable work force and improved safety using new drilling and dirt removal equipment. Fed up with the increasing administrative pressure from Washington and his peers, Steven chose to resign and was replaced by his right-hand man, George Washington Goethals.

Goethals divided the Canal into three divisions (Atlantic, Pacific, and Central) in order to increase productivity. The Atlantic division was responsible for construction of the bulkhead at Limon Bay, the Gatun Locks, and the Gatun Dam. The Pacific division had similar responsibilities, constructing a bulkhead in the Panama Bay and building the Miraflores and Pedro Miguel locks and dams. The Central division was responsible for everything in between the other two divisions. This included excavation the Culebra (Snake) Cut, later renamed the Gaillard Cut, an artificial valley that connects the Gatun Lake to the Gulf of Panama, consequently linking the Atlantic to the Pacific. The excavation of the Culebra Cut was considered one of the greatest engineering feats of its time due to the sheer scale of the work. After countless setbacks, deaths, and scandals during the initial French attempt, the Canal opened on August 15th 1914. The Canal cost the United States about \$375 million to construct, roughly \$8.6 billion today.

Gradual Upgrading and Expansion Projects

The Panama Canal has undergone several important improvements in an effort to maintain its competitive edge as an international maritime trade route. The current expansion project, however, is the largest project since the original construction in 1914. This project, which will be discussed later in greater detail, will double the current traffic capacity of the Canal with a third, larger lane which can accommodate larger

Post-Panamax vessels. The Canal currently operates at 85 percent capacity handling a maximum annual traffic of 14,000 vessels annually. The Canal is also limited to vessels with a maximum capacity of 5,000TEUs (twenty foot- equivalent-units). As of 2007, approximately 25 percent of all vessels are larger than the carrying capacity of the Canal. In order to meet current and future demands, the Canal must upgrade in order to accommodate these vessels both in size and in number of lanes.

Since the completion of the Canal in 1914, Panama and the surrounding region has thrived from the economic inflow. The international maritime business spurred economic development due to transportation opportunities such as trade, commerce, finance, logistics, insurance, and other services. The sum of all these direct and indirect revenues associated with the Canal is referred to as the Canal Economic System. Several subsequent industries include service to vessels in transit, logistics services, ports, cruise ship tourism, and many others. In order to protect the wide variety of economic ventures that rely on the success and sustainability of the Canal, several key construction projects have been undertaken to expand the safety and usability of the route.

In 1939, the construction of the third set of locks was originally conceived. The Tripartite Committee composed of representatives from the United States, Japan, and Panama recommended the construction of a third set of locks to accommodate vessels up to 150,000 Dead Weight Tons. However, several years into the planning of the upgrade, the world became preoccupied with the outbreak of World War II and the construction came to an abrupt halt. Following the war, the United States changed the naval strategy to station separate fleets on the Atlantic and Pacific sides of the Canal, reducing the necessity for the upgrade. The US Army Corps of Engineers (USACE) evaluated the current conditions of the Canal and recommended several improvements that contributed to the Canal Modernization Program which required an investment of \$1.5 billion and took precedence over the third lock expansion.

Several important improvement projects increased the operation and navigational safety of the Canal. The first of these expansions was the construction of the Madden Dam from 1931 to 1935. This dam enclosed the Alhajuela Lake and better controlled flooding conditions from the Chagres River Watershed. The first project completed under the Canal Mobilization Project was the widening of the Gaillard Cut from 91.5m to 152m in response to an increase of larger vessel transit. The Gaillard Cut is an excavated gorge approximately 8 miles long creating an artificial channel across the Continental Divide. Following the widening of the Gaillard Cut completed in 1957, several infrastructure enhancement projects were undertaken including the replacement of the locomotive fleet in 1964, and a high-mast lighting system was installed at the locks to increase safety for vessels traveling at night. The next expansion project was

the deepening of the Gatun Lake navigational channel in 1970. Following the Gatun Lake project, several more improvement projects were completed including the 1990-2000 replacement of all lock locomotive tracks, replacement and increase of the locomotives fleet with modern and powerful units, and increase and modernization of the tugboat fleet.

Panama Canal Authority

The Panama Canal Authority (ACP, Autoridad del Canal de Panamá) is the branch of the Panamanian government which is in charge of the management, administration, operation, preservation, maintenance, and modernization of the Canal. The Authority is organized under the terms of the National Constitution and the Organic Law, which enables it to be financially autonomous. In accordance with the Torrijos-Carter Treaties, enacted on December 31st, 1999, the Panama Canal Authority took over the administration of the Canal from the Panama Canal Commission. (About ACP, 2014)

The Fourth Treaty

The third and fourth treaties are known as the Torrijos - Carter Treaties. These treaties were signed by the United States and Panama in Washington, D.C., on September 7th, 1977 which abolished the Hay–Bunau-Varilla Treaty of 1903 and assured that Panama would acquire the Panama Canal after 1999, which would end the control that the U.S. had exercised over the Canal since 1903 (Torrijos–Carter Treaties, 2014).

The third treaty is officially titled the Neutrality Treaty. The U.S. retained the permanent right to defend the Canal from any threat that might interfere with its continued neutral service to ships of all nations (Torrijos–Carter Treaties, 2014) .

The Panama Canal Commission was formed from the fourth treaty, titled the Panama Canal Treaty. This treaty joined the forces of Panama and the United States to run the Canal's organization and management. Furthermore, it provided that on December 31st, 1999 Panama would assume full control of Canal operations and become primarily responsible for its defense. Thus the Panama Canal Authority (ACP) was born (Torrijos–Carter Treaties, 2014).

The Panama Canal Commission

The Panama Canal Commission managed the Canal between the years of 1977 and 1997. The nine members of the governing board were appointed by the President of the United States, five of whom were American and four Panamanian. The Panama Canal Commission was an independent entity funded

by the revenues derived from the operation of the Panama Canal. However, during the 20 years of the Panama Canal Commission management, the nation received \$10 million annually from the United States as fixed payments. Additionally, the Commission received \$10 million in adjustable payments in order to account for inflation. This supplementary support helped fund public services provided by the Panamanian government to the old Canal Zone. Panama also was permitted to receive a portion of the tolls paid by the ships traversing the Canal. The Commission ceased to exist on December 31st, 1999 when the Canal Authority took complete control of all operations. (Records of the Panama Canal, 2014) (Canal history, 2014)

Organic Law

The Organic Law was established on June 11th, 1997 with the purpose of providing the Panama Canal Authority with standards regarding its organization and operation. Additionally, the law establishes the Canal Authority as a leader in the social and economic development of the country, without discriminating against any participants. The Organic Law and the National Constitution provide the framework for the regulations regarding the work done on the Canal. (Organic Law, 1997)

Important Articles of the Organic Law

- Article 1: “The Panama Canal Authority is an autonomous, legal entity established and Organized under the terms of the National Constitution and this Law.” (Organic law, 1997)
- Article 3: “The Canal is an indisputable patrimony of the Panamanian nation. Therefore, it may not be sold, assigned, mortgaged, or otherwise transferred” (Organic law, 1997)
- Article 5: “The fundamental objective of the roles of the Authority is that the Canal must always remain open to the peaceful transit of vessels from all nations, without discrimination. Since the international public service provided by the Canal is extremely essential, its operation should not be interrupted for any reason.”
- Article 120: “Any regulation adopted by the Authority concerning water resources in the Canal watershed shall have, among others, the following purposes: To manage the water resources for the operation of the Canal and the supply of water for consumption by surrounding communities. To safeguard the natural resources of the Canal watershed, especially in critical areas, for the purpose of preventing a reduction in the indispensable supply of water to which the above paragraph refers” (Organic law, 1997).

Panama Canal Authority Organizational Structure

The Administrator (CEO), Jorge L. Quijano, is the highest-ranking executive officer and legal representative of the Authority. He is responsible for the administration and the implementation of the policies and decisions of the Board of Directors. The Administrator is appointed for a term of seven years and may be re-elected for one additional term. The Administrator and a Deputy Administrator are under the supervision of an 11-member Board of Directors. The responsibilities of the members of the board of directors overlap to guarantee independence from succeeding government administrations (About ACP, 2014).

Panama Canal Authority Board of Directors

The Panama Canal Authority Board of Directors is responsible for supervising the management and establishing the policies related to the operation, improvement and modernization of the Canal. The board of directors is comprised of 11 directors. There is a Director, the Minister of State for Canal Affairs, who chairs the Board of Directors and is appointed by the President of the Republic. In addition, there is another Director, freely appointed or removed by the Legislative Branch. Finally, there are nine Directors appointed by the President of the Republic, along with the consent of the Cabinet Council and the authorization of an absolute majority of the members of the Legislative Assembly. The Directors serve for a term of nine years, and they can only be removed if they commit any criminal offenses to the Public Administration as stated in Article 20 of the Organic Law (ACP Board of Directors, 2014). The President of the Republic can also suspend or remove any Director, having the consent of the Cabinet Council and the Legislative Assembly Directors, for any physical, mental, and/or administrative incompetence. (Organic Law, 1997)

The chairman of the Panama Canal Board of Directors is Robert R. Roy, since 2012. He has been a member of the board of directors since 1998. (ACP Board of Directors, 2014)

ACP Advisory Board

The Panama Canal Authority established the Advisory Board in December of 1999, in accordance with Article 19 of the Organic Law. Article 19 states that because the Canal services international parties, an Advisory Board including foreign representatives is necessary to maintain a diverse and therefore objective view. (Organic law, 1997)

The purpose of the Advisory Board is to serve as a consultant for the Canal's business, and provide guidelines and recommendations to the Board of Directors and the Canal administration. The Advisory Board is composed of highly recognized professionals with broad experiences, specifically in the business, trade, telecommunications, construction, academia, and banking sectors. The Board of Directors meets

with the Advisory Board annually, though the meetings may occur more frequently as necessary. (ACP Advisory Board, 2014)

Vision, Mission and Values

The Canal Authority states that through the revenue of the Canal it will put as much effort as possible to improve the nation's standard of living, welfare and development. The Authority recognizes the importance of creating long lasting relationships with customers and providing high quality service (ACP Corporate Mission, 2014). The Authority strives to create a friendly work environment, by valuing diversity and by giving the opportunity to employees to contribute, learn, grow, be promoted, and well compensated for brilliance and determination. The Authority views their employees as "the most important resource in achieving service excellence" (ACP Corporate Mission, 2014).

Expansion Program

In 2007, construction on the largest expansion in the history of the canal began. The \$5.25 billion Panama Canal Expansion Project consists of 4 essential components for the improvement of infrastructure, and water quality and supply. These components include the installation of Post-Panamax locks, the excavation of the Pacific Access Channel (PAC), improvements to navigational channels, and raising the water level of the fresh water Gatun Lake. The focus of this study is on the construction of the Borinquen Dam 1E for the creation of the Pacific Access Channel to the new locks and the continuous maintenance dredging.

Post-Panamax Locks

Currently, the locks have begun to be installed. Figure 4 below is a rendered drawing of the proposed Post-Panamax Locks with connected water saving basins.

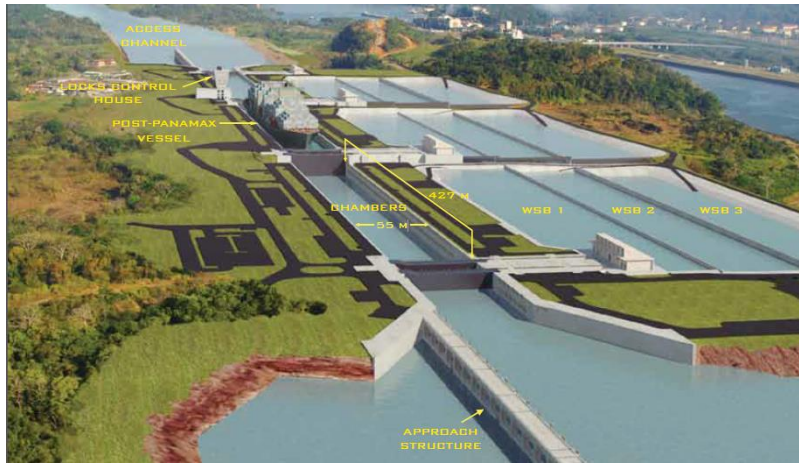


Figure 4: Rendered Drawing of Post-Panamax Locks

The new lock system consists of eight double rolling gates which enclose three chambers with three water saving basins attached to each chamber. The water saving basins assist in reducing the volume of water used by the new locks by saving 60 percent of the water used during lockage. The water saving basins are attached to its respective chamber though culverts regulated by water valves.

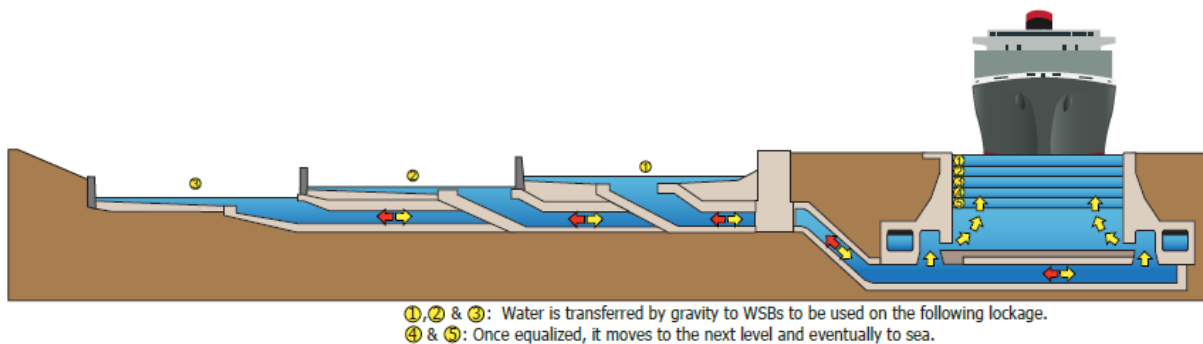


Figure 5: Connection between Lock Chambers and Water Saving Basins

Dredging

After the major excavation projects were completed, the dredging of the different navigational channels began in April 2008. The Belgian company, Dredging International, widened the Pacific entrance to the Canal to a width of 225 meters (738ft) and increased the depth of the channel from 12.5 to 15.5 meters (41.0 to 50.9ft) below mean low water level. Dredging International also participated in the partial construction of the Pacific Panamax locks. The construction company Jan De Nul n.v. was awarded with the contract for dredging the Pacific and Atlantic access channels. Each entrance navigation channel was widened from about 200 meters (656.2ft) to a minimum of 225 meters (738.2 ft) on both the Atlantic and

Pacific sides. This contract also included the deepening of the Pacific and Atlantic channels to 16.76 meters and to 16.1 meters (55ft-52.8ft) respectively.

Pacific Access Channel and the Boriquen Dam

The Boriquen Dams will create a channel to the new Panamax Locks called the Pacific Access Channel (PAC) such that the PAC will be at the same level as the Gatun Lake, 11 meters (36.1ft) above the Miraflores Lake.

The construction of the PAC was divided into 4 parts. The stage of construction in which the Boriquen Dam 1E is being constructed is referred to as PAC-4. The first stage of construction consisted of leveling Paraiso Hill from its original elevation of 136 meters to 46 meters (446.2 to 150.9ft). A total of 7.3 million cubic meters (25.3 million cubic feet) was removed from 2007 and 2010. Additionally, this first stage of construction included the clearing of 416 hectares of firing ranges, also referred to as MEC (Munitions and Explosives Concern) since the area had previously been used for U.S. military purposes. It also included the relocation of 3.6 kilometers (2.23mi) of the Boriquen Road which runs along the proposed Boriquen dam and the relocated Cocoli River. The figures below show the initial excavation efforts in PAC-1.



Figure 6: Flattening of the Paraiso Hill



Figure 7: Relocation of the new Boriquen Road

In the second phase of construction, PAC-2, an additional 7.4 million cubic meters of material was removed in addition to the 3.5km (2.17mi) rerouting of the Cocoli River, and 1.3km (0.81mi) rerouting of the Borinquen road. The third phase of construction, PAC-3, completed the leveling of the Paraiso Hill from 46 to 27.5 meter (150.9- 90.22ft) elevation and cleared an additional 190 hectares of MEC areas.

In the current phase of construction, PAC-4, 26 million cubic meters (918.² ft) of unclassified materials will be excavated in addition to the construction of the 2.3km (1.4mi) Boriquen dam. This dam will separate the waters of the Miraflores Lake and that of the new PAC. This excavation and dam construction contract

Pacific Entrance Maintenance Dredging

Introduction

Dredging is an integral part of the welfare and maintenance of all waterways. As time goes on existing channels become more shallow and narrow. This is primarily due to sediment being deposited by passing vessels and tidal currents. Access channels and turning basins require frequent maintenance dredging in order to sustain appropriate depths and widths for safe vessel travel (Bray, 2004).

The Autoridad del Canal de Panama (ACP) performed maintenance on the Pacific Entrance of the Canal. This was an extensive dredging project where the ACP reduced the slopes of the Canal banks and deepened the center by removing loose sediment. The purpose of this study was to determine the progress being made by the Canal Authority sub-contractor and design more efficient ways for the project to be done.

The ACP owns several dredgers; however none of the Authority's dredgers are suitable for the type of work that was done. To compensate for this, the ACP enlisted the aid of a sub-contractor to complete the necessary work. The contract for this work was awarded to Dredging International, a Belgian dredging company that has worked with the ACP in the past. The vessel used by Dredging International, the Brueghel, was a trailer suction hopper dredge. The dredge is shown below.



Figure 8: Brueghel Trailer Suction Hopper Dredge (Dredging International, 2014)

In this section of the report, the work done by Dredging International was analyzed in order to develop a comprehensive procedure to improve the current dredging operation as well as future maintenance plans.

Background

Dredging has been used for thousands of years to expand and deepen waterways for maritime transit. Since its origins, dredging has become a multi-million dollar industry with massive advancements in vessels and technology.

Navigational Improvement Dredging

There are many uses for dredging such as construction and reclamation, beach nourishment, and flood control (Bray, 2004). The purpose of the maintenance dredging on the Panama Canal was for increased navigational capacity. Vessels with high maneuverability carry out most navigational improvement dredging. Exceptional mobility is necessary to maneuver in areas where high traffic and currents are present. The following elements comprise navigational improvement dredging.

Depth

The minimum necessary channel depth that a vessel requires for navigation is determined through weight, shape, and vessel traffic (Bray, 1978). Navigable waters will be dredged according to use, and vessel size. Due to economic restraints, some larger vessels may be limited to times of high tide due to inadequate channel depth. When constant vessel traffic is expected throughout both high and low tide, the channel must be dredged to a greater depth in order to accommodate large vessel transit at low tide. Several measures are used to quantify the channel depth including water reference level, lowest water level, admissible draught, vertical ship movement, net underkeel clearance, sounding accuracy, and other components. These are discussed in more detail below.

Water Reference Level and Lowest Water Level

Water in a channel is usually measured at the water reference level. This is a fixed point that must be within 5% of the average deviation of historically recorded low tide (Bray, 1978). The lowest water level is where the water level is recorded at the absolute historically lowest tide (Bray, 1978). To find this level, meteorological conditions such as high-pressure and wind must be taken into account.

Admissible Draught, Vertical Ship Movement, and Net Underkeel Clearance

The draught is the measurement of how deep in the water a ship travels. This is known as the applicable load line zone. The admissible draught is the deepest depth at which a ship may safely travel. A non-specified vessel's admissible draught must be estimated using a maximum possible value (Bray, 1978). Draught can be calculated using the weight or length of the ship.

In addition to admissible draught, vertical ship movement must also be considered to determine how deep the vessel will drag under the water. Wakes and waves in the navigable channel can cause 'squatting' where, due to the drop in pressure around the vessel from irregular flows of water, the vessel will sink lower into the water (Bray, 1978).

The safety margin between the lowest calculated position of the vessel's hull and the highest probable elevation of the channel bed is the net underkeel clearance (Bray 1978). This clearance is usually measured at .3-.5 meters for softer sand or clay sea beds but is doubled in rocky sea beds.

Sounding Accuracy and Other Components

Due to a number of factors including tidal range, instrument accuracy, and vessel movement, there are some inaccuracies in bathymetric surveys of the water depth. Due to this uncertainty, it is assumed that the seabed is at least .15 meters above the shallowest depth shown on the survey.

Other components including sediment deposits, dredging tolerances, and navigable depth are additional safety factors that must be considered. The measurements for each is determined on an individual basis.

Width

In addition to depth, the width of channels must be maintained to ensure proper maneuverability and navigation. The three measures for minimum width requirements are "maneuvering, ship clearance, and bank clearance lanes" (Bray, 1978).

The Maneuvering Lane

In order for vessels to have adequate maneuverability, a minimum width was required. Commonly, the maneuvering lane will be 1.6 to 2.0 times the beam width of the vessel (Bray, 1978). Cross-currents may push vessels. This turning due to natural conditions is called the yaw of the vessel. The channel must be dredged an additional width to account for yaw up to 5° for large vessels or up to 10° for smaller vessels (Bray 1978).

The Ship Clearance Lane

For multi-lane channels, a ship clearance lane must separate maneuvering lanes. The lane is generally 30 meters wide but is dependent on a variety of factors including the current, channel depth, channel shape, vessel speed, and many more.

Bank Clearance

There must be a distance between the maneuvering lanes and the bank of the channel. This value tends to be 1.5 to 2.0 times the beam of the vessel but also depends on the type of channel, underkeel clearance, and vessel speed (Bray, 1978).

Bends

Minimum widths of the channel greatly increase during bends due to the increased width of the maneuvering lanes. Average bend radii are determined by the angle of deflection of the bend and follow this general chart:

Table 3: Comparing Angle Of Deflection Vs Minimum Bend Radius (Bray, 2014)

Angle of Deflection	Minimum Bend Radius
25° or less	3 x Length of Vessel
25° to 35°	5 x Length of Vessel
35° or more	10 x Length of Vessel

Basins and Maneuvering Areas

Basins and maneuvering areas have specific size and shape depending on numerous factors of the individual navigable channel. The stopping distances, turning basins, and berths of vessels will determine how these areas will be shaped.

Stopping Distances

To enter the channel or to port, a vessel has a minimum speed. Vessels are required to decrease to a specified speed in order to enter a channel or to bring the vessel to port. The distance required to slow the vessel from this speed to zero is known as the stopping distance. When a vessel is slowing down by putting the engines astern, the vessel's lateral movement must be accounted for. The shorter the stopping distance, the wider the channel must be (Bray, 1978).

Turning Basins

Many vessels must turn before leaving the channel or port. In order to maneuver safely, the following diameter of turning basin must be possible:

Table 4: Turning Basins

Type of Assistance	Diameter of Turning Basin
With Tug Assistance	2 x Length of Vessel

Without Tug Assistance	4 x Length of Vessel
------------------------	----------------------

Berths

Allowances must be made for overshoot when a vessel is maneuvering to a berth of shallower water. The width of the tidal berth should be 1.25 times larger than the beam of the vessel. In addition, underkeel clearance must be .3 to .5 meters in smooth sea bed conditions and double when the seabed is rocky.

Trailing Suction Hopper Dredger

The Panama Canal maintenance-dredging project will use a trailing suction hopper dredger. Also known as a 'trailer', this dredger has the ability to load a hopper contained within its structure by means of pumps while the vessel is moving ahead (Bray, 1978). The ability to move while dredging is helpful in the Canal as the vessel must navigate one of the busiest waterways in the world. In order to unload, the vessel will either have a pump discharge or a bottom-discharge to unload sediment below its location. This ability to bottom-discharge is especially useful in the maintenance dredging when the sediment is transported out to sea to be released. If the vessel is dumping at sea, bottom-discharge is the fastest discharge method.

The main advantages of a trailing suction hopper dredger are:

- Relative immunity to weather and sea conditions
- Independent operation
- Minimal effect on other shipping
- The ability to transport dredged material over long distances
- Relatively high rate of production
- Simple, and hence inexpensive, mobilization procedure

Meanwhile, the disadvantages are:

- Inability to dredge strong materials
- Inability to work in very restricted areas
- Sensitivity to concentrations of debris
- Dilution of dredged materials during the loading process

An average dredger will have the capacity to hold 750 to 10,000 cubic meters of sediment. A dredge was usually rated by its size capacity.

Site Characteristics and Conditions

When beginning a dredging operation, it is important to make sure the conditions are ideal in order to ensure that the operation goes as smoothly as possible. Some of the factors that must be taken into consideration are the location of the dredging and dumping sites, the weather conditions, the water conditions, traffic, and the seafloor conditions.

Location

The location of the dredging and dumping sites is integral to the dredging process. If the distance between the sites is considerable then it is easiest to transport material using a hopper dredge. If a hopper dredge is unavailable then barges are used to transport the material from a stationary dredge at the work site to the dumping site.

Water Conditions

The depth of the water at the site could restrict access and reduce production capabilities of the dredger. Trailer suction hopper dredges are only capable of excavating material below their hull (Bray, 1978). This means that the water must be at an adequate depth in order for the dredger to operate. Tidal movements could potentially affect access to the dredging and dumping sites. Strong currents could disrupt the contact between the ground and suction head, thus slowing productivity. The currents also affect the maneuverability of the dredge. In confined sites, it is important for dredges to be able to make tight turns. These turns also impact the time it takes for a dredge to load material.

Dredging Production Cycle

The production cycle for trailing suction hopper dredgers consist of three stages: load, discharge, and sail to working area. This section goes in depth into each part of the production cycle.

Load

The loading portion of the dredging cycle consists of not only loading time but also the turning time. Both of these factors must be taken into consideration in order to optimize efficiency.

Loading Time

The loading time is the time it takes for the dredger to become full of sediment. It is dependent on many factors including the rate at which the hopper is loaded and the type of sediment being dredged. Loading times are a good way to measure dredging efficiency as the faster the load can be dredged, the faster another trip can be made (Bray, 1978).

Turning Time

The turning time refers to the time it takes for the dredger to turn itself into a position to dredge. Turning time is counted as wasted time because no sediment will be dredged during turning. Both the severity of the turn and the number of turns will affect the non-productive time during loading (Bray, 1978).

Discharge

The discharge time depends mainly on the type of discharge being used, with bottom-discharge being the most commonly used.

Bottom-discharge

The bottom-discharge method uses hinged or sliding doors located at the bottom of the vessel to release the sediment into the water it is sailing over. Hinged doors have potential to be damaged in shallow water if there is not enough clearance for them to open so sliding doors powered by hydraulic cylinders may be used (Bray, 1978). Bottom valves can also be used in place of doors to discharge sediment. They can be operated in shallow waters in which hinged doors could not operate and provide greater safety. A split hull discharge is used to combat the problems mentioned in the other bottom discharge methods. Heavy hinges at deck level join two halves of the hull, making the hull itself be the door that releases the sediments. The split hull design has the fastest discharge rate but it is impractical for very large dredgers due to high construction costs (Bray, 1978).

Pump-discharge

Pump-discharge systems are primarily used during the reclamation of land. Material is dredged and sent through a system of pipework and valves to remove water. The soil/water mixture is pumped through a nozzle that sprays the material up to 100 meters from the dredger (Bray, 1978). For example, dredging can occur immediately behind a seawall for beach reclamation.

Scraper and Bucket Wheel Discharge

Large buckets moved by winches feeding to belt conveyers are commonly used to discharge aggregates to quays (Bray, 1978). These systems are mainly confined to marine aggregate mining operations in order to have a shore discharge of material.

Sidecasting or Boom-discharge

Sidecasting discharge dredgers operate without a hopper. The dredged material is pumped through a horizontal elevated pipe. The material can be moved up to 90 meters and is primarily used for

maintenance of long navigation channels where disposal offsite would not be feasible. The main issue with the side casting discharge method is that sediment is only moved 90 meters and may return quickly.

Table 5 below summarizes the different discharge methods for a trailing suction hopper dredger.

Table 5: Various Discharge Methods for Trailing Suction Hopper Dredgers (Bray, 2014)

Discharge Method	Application
Bottom Door: hinged	Clean silts, sands and soft clays, calm water
Bottom Door: sliding	Clean silts, sands and soft clays in shallow water, or rough seas
Bottom Valve	Clean silts, sands and soft clays in rough seas
Split Hull	Any material, including those containing boulders, or debris, for disposal in shallow water and moderate seas
Pump	Silts, sands where disposal was to onshore areas, or material was to be used for land reclamation
Scraper	Shore discharge of dredged aggregates
Grab	Shore discharge of dredged aggregates

Sail to Work Area

Sailing time is the amount of time it will take to reach the discharge or dredging area. This value is found by dividing the length of the trip by the velocity of the vessel when loaded. Despite having a full load, it takes about the same amount of time to sail to the discharge as it does to sail to the dredging area. Other factors that can affect the sailing time include weather, waves, current, boat traffic, and navigational restrictions (Bray, 1978).

Dredging Contract

Nearly all dredging work is performed under conditions stated in the dredging contract. There are many variable factors in a dredging process that must be discussed and included in the contract documents. The overall objectives of the contract documents are:

- To describe accurately the work to be done and the conditions under which it is to be done
- To apportion risk
- To provide a fair and equitable method of payment for that work when it is satisfactorily completed

Dredging Production

To determine the progress of dredging operations, quantity of materials dredged must be ascertained. Additionally, payment to the dredging company is based on the volume of sediment moved. When measuring the production of a dredger it is important to realize it is expressed in solids transported. Water is the medium used to transport the sediment but is not counted in the dredged volume. Dredge Law I is used to determine the relationship of water to production and solids. The following formula is used in daily dredge calculations:

$$yd^3/hr = GPM \times \text{Average Percent Solids} \times .297$$

Equation 1: Dredge Law I

To find cubic yards per hour you must multiple U.S. gallons per minute of slurry times average percent solids in the slurry times a constant of .297. The .297 value is a constant derived from the conversion to yd^3/hr . It is also important to consider the different units that are used to measure dredging volume. Table 6 below provides constants for indicated production units and flow units:

Table 6: Constants for Production Equation (Turner, 1984)

Production Units	Flow Units			
	American GPM	Liters/Sec	M ³ /hr	Imperial GPM
Yd ³ /hr	.297	4.71	1.31	.357
M ³ /hr	.227	3.60	1.00	.272
Short tons/hr	.442	7.01	1.95	.531
Long tons/hr	.401	6.36	1.77	.482

Gross tons/hr	.395	6.26	1.74	.474
---------------	------	------	------	------

Dredge Efficiency

Production is directly proportional to average percent solids. To understand dredge efficiency the relationship of maximum percent solids and cavitation must be defined.

Maximum Percent Solids vs. Cavitation

Maximum percent solid is defined as the highest practical, instantaneous percent of solids the hydraulic system can transport, without cavitating the pump (Turner, 1984). Cavitation occurs when the natural barometric pressure is no longer capable of overcoming the losses in the suction line at the rate the pump demands. The dredging operator must control the pickup of solids so as not to have cavitation while maintaining a high percentage of solids. The maximum percent solids coincide with maximum instantaneous production. These definitions are import especially when considering the dredge efficiency given in the following formula:

$$Dredge\ Efficiency = \frac{average\ percent\ solids}{maximum\ percent\ solids}$$

Equation 2: Dredge Efficiency (Bray, 1978)

It is important to realize that 100% dredge efficiency is not when the suction is picking up 100% solid materials rather it is dredging at a maximum percent solids rate.

Dredging Methodology

The goal of this project was to aid the Panama Canal Authority in the completion of the maintenance dredging project. To accomplish this goal our team will complete the following two objectives:

Objective 1: Determine if the dredging project was on schedule

Objective 2: Research and design possible improvements to operations

In this chapter we will discuss the specific methodological strategies we will use to achieve these objectives.

Objective 1: Determine Completion Time of Dredging Project

For this section of the paper, the progress of the maintenance dredging project was determined. The dredging operation began on November 5th, 2014 and will end on January 7th, 2015. Due to the time limitations of this project, the progress of the maintenance was measured through the end of November 2014. The first 5 weeks of the project focused on data collection while the last two focused on analysis and drawing conclusions. To determine the progress of the project, daily production reports and the dredging schedule were analyzed.

Daily Production Reports

The team collected and reviewed the daily reports submitted to the ACP by Dredging International. The material dredged per day and sailing distances to and from the dredging site were determined by analyzing the daily reports. Figure 9 below was an example of the daily reports. All calculations and graphs used in the results utilized data from the daily reports.

	DAILY REPORT			Doc. Name	Daily Report	
	Originated by:	NEG	Checked by:	SVM	Doc. No.	CMC303136.acp.DR 003
				Report Date	7/11/2014	
				Approved by:	SVM	Rev.1

3) EQUIPMENT												
		Vessel		TSHD Breughel								
		Status		Operation								
		Operational Hours		21:30								
		Disposal site		Tortolita South								
Trip	Dredge area	Type of material	Density (t/m3)	Start time (time)	Dredging excluding turning (hh:mm)	Turning (hh:mm)	Sailing loaded (hh:mm)	Dumping (time)	Dump box	Duration of dump (hh:mm)	Estimated volume (m³)	Sailing to dredge area hh:mm
9	CS 2-3	Mud	1.4	00:00	0h 30m	0h 10m	1h 10m	1:50	111	0h 10m	4617	1h 10m
10	CS 2-3	Mud	1.4	3:20	0h 35m	0h 15m	1h 10m	5:40	82	0h 10m	4739	1h 10m
11	CS 2-3	Mud	1.4	7:00	0h 35m	0h 30m	1h 10m	9:15	188	0h 10m	5152	1h 10m
12	CS1-2-3	Mud	1.4	10:45	0h 35m	0h 30m	0h 55m	12:45	190	0h 10m	5205	1h 10m
13	CS 4	Mud	1.4	14:05	0h 40m	0h 05m	0h 50m	15:40	196	0h 10m	4851	0h 55m
14	CS 2-3	Mud	1.4	15:50	0h 40m	0h 10m	0h 55m	18:30	188	0h 10m	5965	0h 35m
15	CS 2-3	Mud	1.4	18:40	0h 40m	0h 10m	1h 00m	22:35	200	0h 10m	5171	0h 40m
16	CS 2-3	Mud	1.4	22:45	0h 25m							

Figure 9: Daily Report Example from Dredging International

Using this information, the production levels of Dredging International were estimated.

Dredging Schedule Assessment

The second step in finding the progress of the operation relied on using the baseline schedule published by Dredging International. The contractor who was tasked with performing the dredging work was given 154 days to widen and deepen a 14.2 km portion of the channel near the Pacific entrance of the Panama Canal. Figure 10 shows a sample schedule for dredging operations.

Planned		32d 07-Dec-14 23	08-Jan-15 16	\$0.00
Project Milestones		32d 07-Dec-14 23	08-Jan-15 16	\$0.00
FE8020	Sector 1 (Contractual Handover)	0d	07-Dec-14 23*	\$0.00
FE8030	Sector 2 (Contractual Handover)	0d	07-Dec-14 23*	\$0.00
FE8040	Sector 3 (Contractual Handover)	0d	07-Dec-14 23*	\$0.00
FE8050	Sector 4 (Contractual Handover)	0d	08-Jan-15 16*	\$0.00

Figure 10: Sample Schedule for Dredging Operations

In addition to using the operation schedule, dredging specifications were also addressed. By using data on total volume to be extracted and the number of days for extraction, benchmarks for progress completion were set. The comparison between these benchmarks and the daily reports were used to determine if the progress was on schedule.

Objective 2: Identify Operation Improvements

The main focus of the research was on improving the dredging operation. With tight schedules and even stricter budgets, there was a slim margin for error on the project. After receiving two million dollars in fines for a late start, finishing on time is imperative. In order to make recommendations for improving the dredging operation, project delays were analyzed.

Site Details

The maintenance dredging project is focused on the Pacific Entrance of the Canal. With this in mind, research was focused on this portion of the canal. How the ACP divided the work site and the amount of work needed to be done in each sector was determined.

Dredging Delay Analysis

Within the daily reports submitted by Dredging International are details of every delay that occurred. Included with the length of time for each delay was a description of the cause. Measuring delays and determining the cause was critical to finding ways to improve efficiency. The main focus of the analysis of this project was to find where delays could be eliminated or reduced.

Dredging Operation Details

Although all aspects of the dredging operation were researched thoroughly, many details were illuminated through an interview with Dredging International. On November 27th, a tour of Dredging International's Breughel was taken. At that time, the crew was interviewed regarding the dredging operation. Specific aspects of the maintenance dredging operation were discussed as well as problems faced. Both items were important for designing a more efficient maintenance dredging operation.

Application of Best Practices

Besides reducing delays, determining dredging best practices was crucial to developing recommendations. There are numerous types of dredgers, all with varying advantages and disadvantages. In order to make recommendations on the current dredging operation, research was needed to determine if best practices could be followed.

Future Canal Authority Dredging Operations

Another avenue to improve efficiency of future operations was for the Canal Authority to use its own dredger for maintenance operations. To fully determine this possibility, and the benefits and drawbacks of not hiring an outside company like Dredging International, research was done on both the ACP's

current operations and future operations. Detailed costs of owning and operating dredging equipment were researched and compared to contracted costs. The need for future operations was also determined to see the quantity of equipment and type of dredger the ACP would need. Using these research factors the benefit of the Canal Authority owning and operating its own dredging equipment for maintenance projects was determined.

Recommendations

After research of best practices and analysis of dredging operations, recommendations were made. The recommendations were organized into two different categories: recommendations for the current dredging project, and for recommendations for future projects. Since the project has a completion date of December 7th for 3 out of the 4 Sectors, many recommendations may be applied to future dredging projects.

Findings, Discussions, and Recommendations

This chapter of the report details the research findings, discussion on the findings, and recommendations for improving Canal dredging operations. The chapter is divided into two main sections: dredging production analysis, and investigation of dredging efficiency. In these two sections, the quantity and efficiency of work were identified.

Dredging Production Analysis

The data received from Dredging International spans from the beginning of the project, November 5th, 2014 to November 30th, 2014. The information gathered was used to complete an analysis of the work done by the dredging contractor. The dredge operates 24 hours a day for three weeks at a time. Once every three weeks the dredge must halt all work for 24 hours in order to refuel. In order to determine if the project will be completed on time, calculation for daily design volumes were compared to daily volume reports.

Minimum Daily Load Volume

The following are the calculations to determine the volume of material that must be dredged daily. The first calculation was for the minimum design depth.

Sector I

Sector I has a design depth that was unknown, due to the uncertain sea floor conditions. To make an accurate estimate of the design volume to be removed, the average percentages of design to maximum volumes were applied to Sector I.

Sectors II-III

Design Depth = 502,700 m³

Maximum Paid = 678,299 m³

Design to Maximum Paid Ratio = $502,700/678,299 = .74$

Sector IV

Design Depth = 731,640 m³

Maximum Paid = 924,828 m³

Design to Maximum Paid Ratio = $731,640/924,828 = .79$

Average Ratio of Design to Maximum Paid $(.74 \times 2 + .79)/3 = .76$

Sector I

Using the average ratio of design to maximum paid volume, the design for Sector I calls for the removal of 354,261 m³ of sediment.

Average Daily Volume Design

In order to determine the daily volume needed to reach the design specifications, two pieces of information was needed. First, the total design volumes for Sectors I, II, and III were calculated. Secondly, the total number of working days was determined. The total design volume/number of working days will give you the volume per day needed for design completion.

Sectors I, II, III

The total design volume for Sectors I, II, and III was the individual volumes from Sector added together:

$$354,261 \text{ (I)} + 502,700 \text{ (II+III)} = 856,961 \text{ m}^3$$

The dredging project started on the 5th of November. For Sectors I, II, and III, the project was set for a handover on December 7th. Factoring in vacation days, the total number of working days was 31.

Total Volume/Total Days = Volume per Day for Completion

$$856,961/31 = 27,644 \text{ m}^3$$

For Sector IV, daily volume benchmarks were determined differently. Since the Sector IV dredging operation was set for completion on January 7th, 2015, it could be dredged at a slower rate than the other Sectors. To determine the daily volume needed for reaching the design qualifications we added the total number of days and multiplied by the rate of dredging. From November 5th to December 7th, Sector IV was dredged at a production level “x”. From December 8th to January 7th, Sector IV was dredged at a “4x” rate because it was the only Sector being dredged. The calculations for total days of dredging are as followed:

Sector IV

November 5th – December 7th, 31 working days at “x”

December 8th – January 7th, 27 days at “4x”

$$x \cdot 31 + 4x \cdot 27 = 731,640 \text{ m}^3$$

$$139x \text{ days} = 731,640 \text{ m}^3$$

$$x = 5264 \text{ m}^3 \text{ per day}$$

Since “x” was equal to 5264 m³ per day, from November 5th – December 7th 5,264 m³ must be dredged per day. After December 7th, 21,056 m³ must be dredged per day.

Total Design Volume per Day

By adding the total design volumes per day of Sectors I, II, III, IV, the total design volume per day was determined.

$$27,644 \text{ m}^3 \text{ (I, II, III)} + 5,264 \text{ m}^3 \text{ (IV)} = \mathbf{32,908 \text{ m}^3}$$

In order to meet daily production benchmarks, and to finish the project on time, Dredging International must dredge 32,908 m³ of sediment per day.

Maximum Daily Load Volume

The same calculations were also completed for maximum daily load volume. Using the maximum paid volume, Dredging International will receive a limit for daily average daily volume load can be set. All values for days will be the same as calculating the design volumes, but the volume calculations will change.

Sectors I, II, III

Maximum paid volume was determined by adding the individual maximum paid volumes from Sectors I, II, and III

$$468,599 \text{ (I)} + 502,700 \text{ (II, III)} = 1,146,898 \text{ m}^3$$

If the total number of dredging days is constant, the daily volume limits can be calculated:

$$1,146,898 \text{ m}^3 / 31 \text{ days} = 36,997 \text{ m}^3$$

Sector IV

The maximum paid volume for Sector IV was 924,828 m³

Like the calculations for design daily volume, two different rates of dredging for November 5th were used— December 7th and December 8th – January 7th. With this in mind the maximum volume per day was calculated.

$$x*31 \text{ days} + 4x*27 \text{ days} = 924,828 \text{ m}^3$$

$$x = 924,828 \text{ m}^3 / 139 \text{ days} = 6653 \text{ m}^3 \text{ per day}$$

Since “x” was equal to 6,653 m³ per day, from November 5th – December 7th 6,653 m³ was the limit to be dredged per day. After December 7th, 26,614 m³ was the limit.

Total Maximum Volume per Day

By adding the total maximum volumes from all of the Sectors, the total maximum volume per day can be determined.

$$36,993 \text{ m}^3 \text{ (I, II, III)} + 6,653 \text{ m}^3 \text{ (IV)} = \mathbf{43,650 \text{ m}^3}$$

The maximum paid volume Dredging International can remove was 43,650 m³ per day. If they continually dredge material at a rate that was greater, they will not be paid for all of their work, reducing the production of the operation.

Dredging Volume Production

On average, the dredge was able to make about eight trips per day. The chart below shows the average amount of sediment dredged per trip in m³.

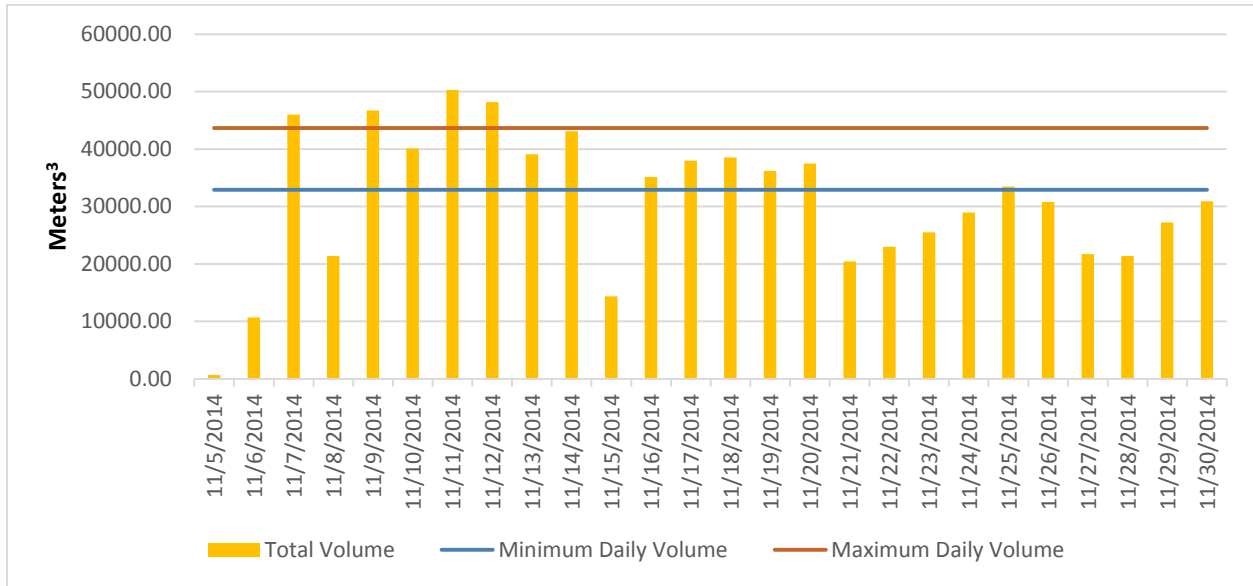


Figure 11: Average Volume per Trip

Based on the 1,234,340 m³ design volume the contract requires to be dredged, it was determined that Dredging International must dredge at least 32,908 m³ per day. If they fail to maintain the production, the project may not be completed on time. As the above graph shows, the dredging contractor consistently manages to dredge the above the required minimum amount, meaning they will most likely complete the project on schedule.

Although the average amount of material dredged stays fairly consistent, the overall amount of material dredged per day greatly varies. This was because although the average number of trips per day was eight, the number of trips per day was not consistent. For example, November 5th was the first day of the project so only one trip was made. Conversely, eleven trips were made on November 7th. The below graph shows the total volume of the material dredged per day in cubic meters.

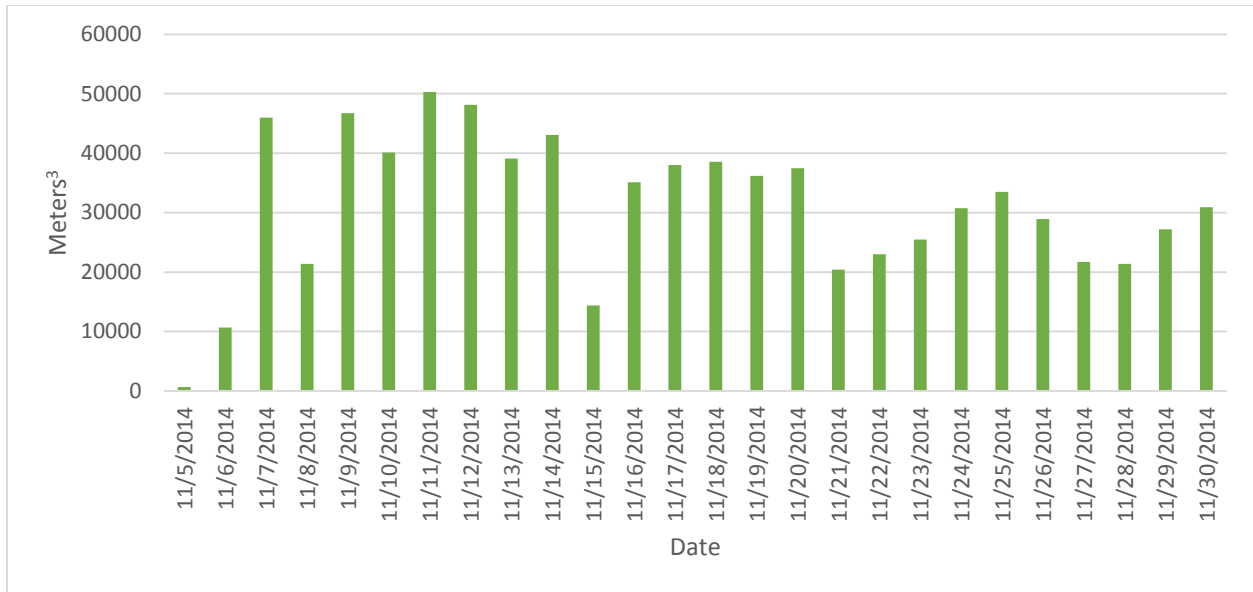


Figure 12: Total Volume per Day

The graph above shows the total volume dredged per day. There are several days where production was significantly hindered due to a variety of reasons. One trend was that as the project advanced, production levels began decreasing. One reason for this it that as the dredge excavates the softer surface materials, harder material and rocks begin to compromise the surface. In order to reach the design depth it may be necessary to dredge the harder material as well. Since the trailing suction hopper dredge was meant to only handle soft sediment, it takes substantially more time to excavate.

Investigation of Dredging Efficiency

The second part of the project was to design a more efficient way to perform the Pacific Entrance maintenance dredging. In order to make well educated recommendations on how to improve the operation, details of the project and progress updates were found. The following list of project elements was looked at to determine dredging efficiency:

- Site Assessment
- Dredging Operational Detail
- Challenges to the Operation
- Project Delay Analysis
- Application of Best Practices
- Canal Authority Dredging Need

Site Assessment

The site Dredging International was working in was the Pacific Entrance of the Panama Canal. The entrance is about 14.2 km long and 192 meters wide. The passage must be widened to a minimum width of 225 meters and deepening it to at least a 16.3 meter depth. There are four sectors of the Pacific Entrance that contractor must dredge. Below is a picture of the work area.

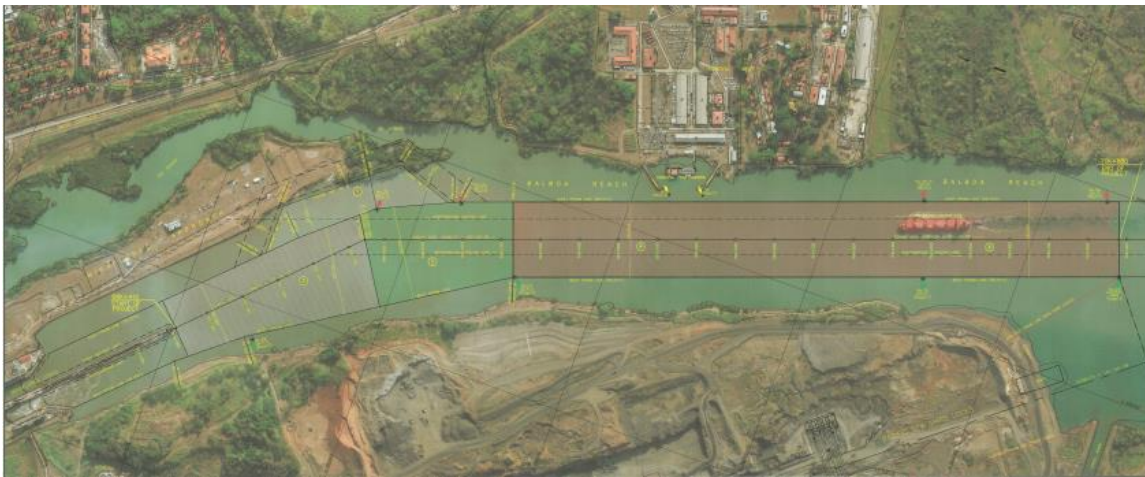


Figure 13: Dredging Sectors of the Pacific Entrance of the Panama Canal

Sector 1: New Fuel Facility Basin

Sector 1 of the work area was located outside of the main navigation channel and will be a new fuel facility for the ACP. Unlike the other sectors, there was no mandatory design depth of this portion of the channel. It was meant to be dredged at equivalent production speeds as sectors 2 and 3. Work was to begin on November 4th, 2014 and end on December 7th, 2014. In that time Dredging International plans on Dredging to the maximum paid depth of 14.8m and along the walls creating a slope of 10m from the base of the slope. The maximum amount of sediment to be removed was 468,599m³. The soil in sector one mostly consists of soft marine sediment and clay. There are also a large number of stones in sector 1. Because sector 1 isn't part of the main navigation channel, there will not be any traffic restrictions for dredging. This means that dredging can be done 24 hours a day in this sector.

Sectors 2 and 3: Miraflores Reach (North and South)

Sector 2 was a 450m expanse of the channel with a design depth of 12.92m. Sector 3 was also 450m and has a design depth of 14.2m. Dredging for sector 2 and 3 was to happen simultaneously and was also scheduled to begin on November 4th, 2014 and end on December 7th, 2014. Between the two sectors, 502,700m³ will be excavated. The maximum amount to be removed was 678,299m³. Dredging operations

are not allowed in either sectors while there was south bound traffic. The shallow west and east banks of the sectors also make dredging very difficult during low tide times.

Sector 4: Balboa Reach

At 1.5km, sector 4 was the longest sector, and has the most volume of material to be removed. The design depth and maximum paid depth of this sector was 14.20m. Dredging will begin on December 7th, 2014 and continue until January 15th, 2015. The design calls for 731,640m³ of sediment to be removed. The maximum allowable amount was 924,828m³. Theoretically, there can be traffic in this portion of the channel 24 hours a day. This will make scheduling dredging times very difficult and it was up to the pilot's discretion as to when it was acceptable to dredge.

Disposal Site: Tortolita South

Because the dredged material was mostly mud and sediment, which is unfit to be used for beach restoration or most other projects, the ACP dumped it at the Tortolita South disposal site. The site for the dredged material was 15km from the dredging area. Tortolita South was about 1.32 million square meters and can hold between 8.45 and 9.56 million cubic meters of sediment. There was a disposal grid in place to ensure that the site was filled evenly and gradually. The ACP also supplied Dredging International with an access channel to the disposal site. This was to prevent sediment from being deposited outside of the approved area. It also has the added benefit of keeping traffic out of the dredge's way as it goes to unload material. Below is an illustration of the Tortolita South dumping site and access channel.

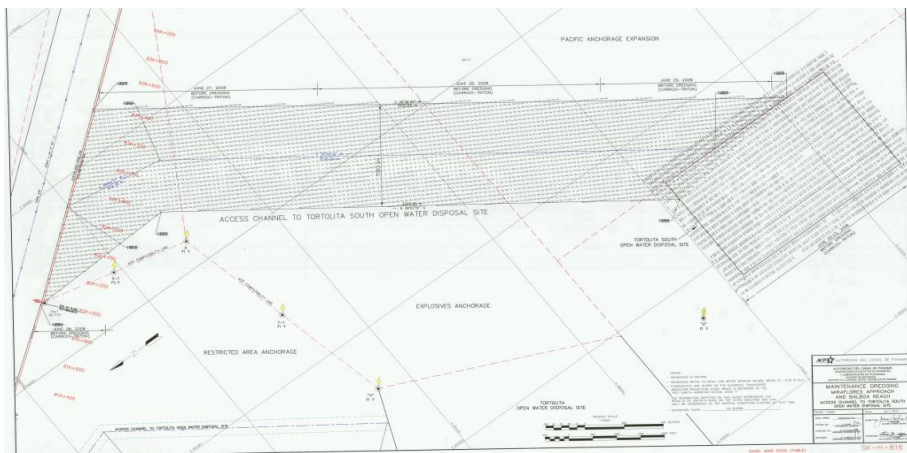


Figure 14: Tortolita South Dumping Site and Access Channel

Dredging Operation Details

In order to provide recommendations on improving the efficiency of the maintenance dredging, a detailed description of the Breughel's operation was needed. On November 27th, the team toured Dredging International's Breughel to learn more about the specific operational details concerning the maintenance dredging. Key equipment used on the dredger was identified and explained on the tour and in an interview with the ships project coordinator, the details of which can be found in Appendix H. The following overview of the dredging operation was compiled below.

General Details

The general details of the dredging operation are components that do not fall into the main three categories of loading, sailing, and discharge. They are still vital to the operation and must be researched thoroughly to determine possible inefficiencies.

Dredger Facts

The following table, taken from Dredging International's website, details the specifications of the Breughel dredger.

Table 7: Specifications of Breughel Dredger

Dimensions	Length	122.19 m
	Breadth	28.00 m
	Moulded Depth	9.80 m
	Draught Maximum	9.10m
Dredging Depth Maximum		28.00 m/43.00 m
Diameter	Suction Pipe	1,200 mm
	Discharge Pipe	1,000 mm
Hopper Capacity		11,796 m ³
Loading Capacity		18,397 tons
Maximum Speed Loaded		14.90 knots
Power	Total Installed with D.R.A.C.U.L.A	12,197 kW
	Total Installed	11,037 kW

	On Pumps Dredging	3,250 kW
	On Pumps Shore Delivery	6,000 kW
	Propulsion Sailing	2 x 4,000 kW
	Propulsion Trailing	6,200 kW

Shifts

The Breughel dredger works on a schedule of 12 hour shifts, seven days a week. They run from midnight to noon and then noon till midnight. Though some crew members live aboard the vessel, others must take a taxi boat and climb aboard the dredger. During this changeover there is some delay in dredging operation.

Refueling

The Breughel dredger uses diesel fuel to power its vessel and dredging operation. Its large tank has can hold enough fuel to power the ship for three weeks. However, due to its massive size, refueling leaves the ship unable to dredge a full 24 hours while refueling.

Loading

The loading phase is the most complicated and time consuming aspect of the dredging cycle. The efficiency of the dredging operation hinges on many factors specific to the site description such as traffic, sediment conditions, and tides. The following was a detailed account of the loading process performed by the Breughel Dredger.

Sediment Removal

The Breughel dredger uses a single 1200mm suction hose to pick up sediment. The draghead is also equipped with hydraulic jets that are used to loosen sediment. Though more compacted sediment can be broken up and drawn into the suction hose, the Breughel works most effectively in areas with loose material. The draghead is maneuvered by the ship captain, who also controls the suction velocity.

Calculating Volume

The maintenance dredging project cost is based on volume of sediment dredged. Since the draghead will pick up slurry that includes water along with the sediment, an accurate way to determine the volume of the sediment was needed. To ensure the volume measurements are accurate, the Breughel uses multiple methods in its calculations including:

- On board equipment to measure density of slurry and volume it collects

- Estimates from surveys of how much sediment must be removed
- On board equipment to measure discharge volume and density

These three methods are compared and fall within 10% of each other.

Traffic Maneuvering

Though maneuvering the vessel around traffic was an important aspect for all stages of the dredging operation, it was the most relevant during loading. Because the draghead must be securely at the bottom of the channel while dredging, the vessel must be in position before dredging occurs.

The Breughel does have many tools to lessen the effects of traffic on the dredging operation. The most helpful was offered by the Canal Authority, which gives a schedule of all vessel activity through the Canal for the current day and one day in advance. Additionally, there are tracking systems that will show the location of every vessel in and around the canal. The vessels name, dimensions, and velocity and bearing all are available.

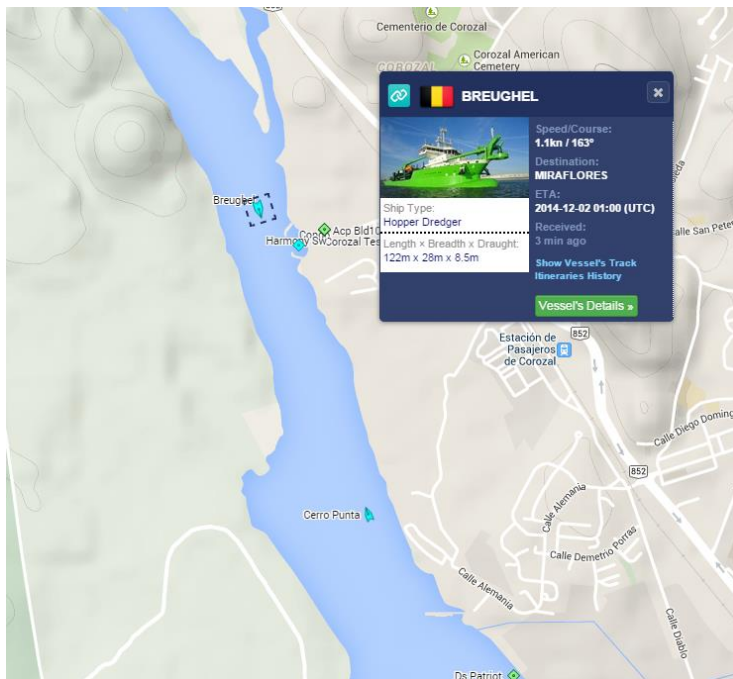


Figure 15: Vessel Information Traveling the Panama Canal (Shiptracking AIS, 2014)

The use of these tools was critical to the Breughel's successful completion of the dredging project. Even when there are times of 2-way Canal traffic the Breughel still manages to dredge when there was space.

The only way they can accomplish this was by using the GPS technology to utilize the time there was a lull in vessel traffic.

Dredging Location

Another factor in the loading process was where the Breughel will dredge at any given time. After talking with members of the crew, we found that the team was not worried about the deadline to complete Sectors I, II, and III. Their focus however was to make sure their production levels were as high as they could be. If there was traffic blocking Sectors I, II, or III, the Breughel would dredge in Sector IV, even though the finish date was not until January. This approach to the dredging project does work because the team was on schedule to complete the first three sectors on time.

Overflow

The Breughel currently does not use its overflow valve in the dredging operation. Without the use of such a device, whatever the suction hose extracts from the channel will be dumped into the discharge area. A concern of this practice was that the draghead must be secure to the bottom of the channel so the hopper will not fill up with slurry of low sediment concentration. Since the project completion and payment are based on sediment volume, slurry with high water content will reduce efficiency.

Sailing

There are few options for the sailing to and from the discharge area. The Breughel has speed limitations set forth by the Canal Authority for safe operation. Also, the distance between the dredging site and the discharge area was a fixed length. Once the production starts to slow from either the hopper being full or too much traffic the vessel sets a course for discharge. Deciding when to sail to the discharge area was a decision the captain makes with the help of the crew to ensure high levels of production.

Discharge

The Breughel uses a bottom-door discharge method. This process takes about 10 minutes and releases the slurry in the hopper directly downwards towards pre-marked disposal sites. To ensure there was enough room at the disposal site for the completion of the project, there are sections where sediment will not be released until the end of the project.

Challenges

There are some challenges that Dredging International must overcome in order to complete the project on schedule. The three main problems that the contractor faced are, traffic, material, and coordinating the sectors being dredged.

Traffic

Due to the channel receiving heavy volumes of traffic there were very limited times that the dredge was allowed to work in the channel. The contractor must be able to plan accordingly for when they will not have access to parts of the channel. The table below shows when traffic was permitted through certain parts of the channel.

Table 8: Pacific Entrance Channel Operating Restrictions

From Station	To Station	Hours	Traffic Conditions
8P+500	9P+968	0:00-24:00 (24 hours)	No Traffic
68K+410	71K+900	0:00-3:00 17:00-22:00 (8 hours)	Two-Way Traffic
		3:00-10:30 15:00-17:00 22:00-24:00 (11.5 hours)	One-Way Traffic
		10:30-15:00 (4.5 hours)	No Traffic
71K+900	82K+000	0:00-2:30 17:30-22:30 (7.5 hours)	Two-Way Traffic
		2:30-10:00 15:30-17:30 22:30-24:00 (11 hours)	One-Way Traffic
71K+900	82K+000	10:00-15:30 (5.5 Hours)	No Traffic
82K+000	84K+250	0:00-24:00	Two-Way Traffic

		(24 hours)	
--	--	-------------------	--

Dredging International must allow a minimum clearance of 122m in the navigation channel during “one-way” periods. This means that the prism lines outside of the navigation channel will still be available to work in. However the navigation channel will not be accessible for work during these times. This prevents sectors 2 and 3 from being dredged during one-way traffic hours. During times when “two-way” traffic was passing through the Pacific Entrance Channel, no work could be done, severely limiting production hours for Dredging International. Areas that were not easily accessible were dredged when there was no traffic on the canal, giving Dredging International the necessary time to maneuver through the access channel. As the project continued and the canal widened, the dredge was able to work on the edges of the channel while traffic as passing through.

Daily traffic schedules were only posted one day in advanced, giving the contractor a very limited timeframe to plan their daily operations. Coordination from traffic control, the contractor, and the ACP dredging division was paramount for the timely completion of this project. There was always an ACP pilot aboard the dredge to make decisions on behalf of the ACP and to ensure that the dredge was not disrupting traffic.

Tide

Tide also played a major role in the maintenance dredging operation. The water level in the entrance can fluctuate by about 5 meters due to the tide. This means that some shallower areas of the channel are inaccessible during certain points of the day, forcing the contractor to work on other areas. The tide also effects how the contractor dumps material at the disposal site. In order to ensure that the sediment was evenly distributed throughout the site, tide and currents must be considered. Strong currents could carry sediment into different parts of the dumping site or even out of the site completely.

Types of Material

Dredging International was only contracted to remove sedimentary materials. While the trailer suction hopper dredge excels at excavating sand and other loose materials, it struggles to handle rock. As the contractor digs deeper, they face harder materials. In some cases in order to reach the design depth it may be necessary to excavate some rocks and larger aggregates. Because the dredger was not designed to handle these materials the process was slowed.

Dredging Delay Analysis

Delays also significantly impacted production values. Pacific Entrance Channel remained open and operational while the project was underway. This means that the dredging must not at all interfere with the daily operations of the channel. The delays to the project, and the cause of the delays were tracked. Below is a graph showing the average amount of time, in minutes, wasted in delays per day.

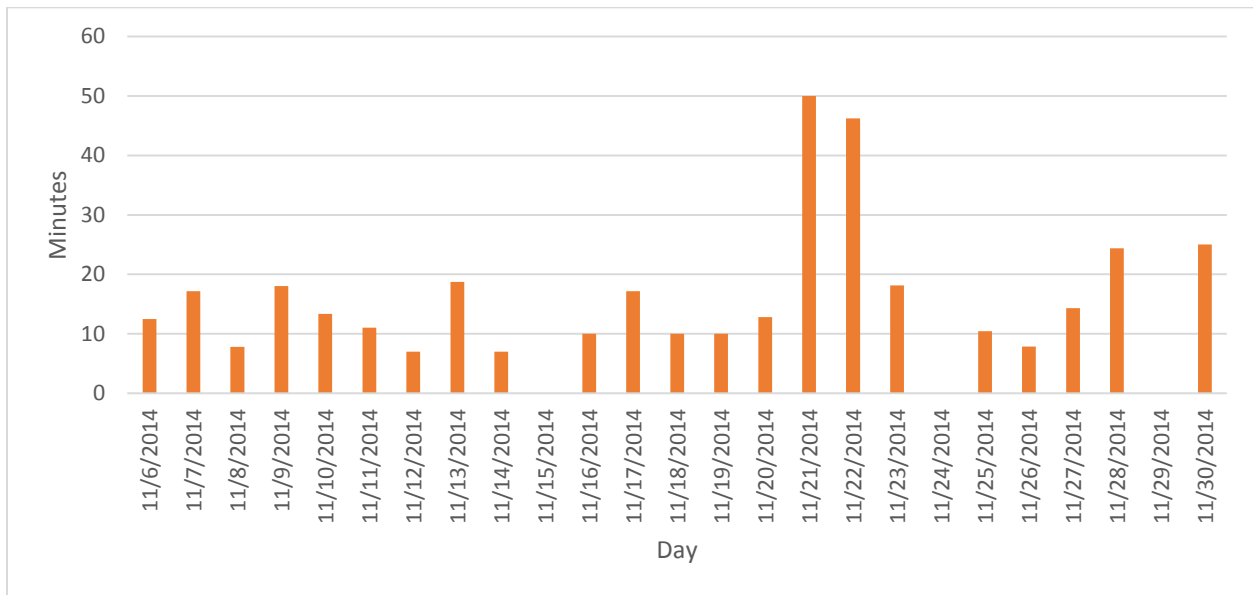


Figure 16: Average Delay per Day

Most of the delays were under 20 minutes, which in a 24 hour operation was only a minor setback. However, there are some outliers. On November 15th operation was halted for about 15 hours due to maintenance on the engines. This was deemed an outlier and removed from the chart.. On the 21st and 22nd more minor maintenance was done, thus delaying production.

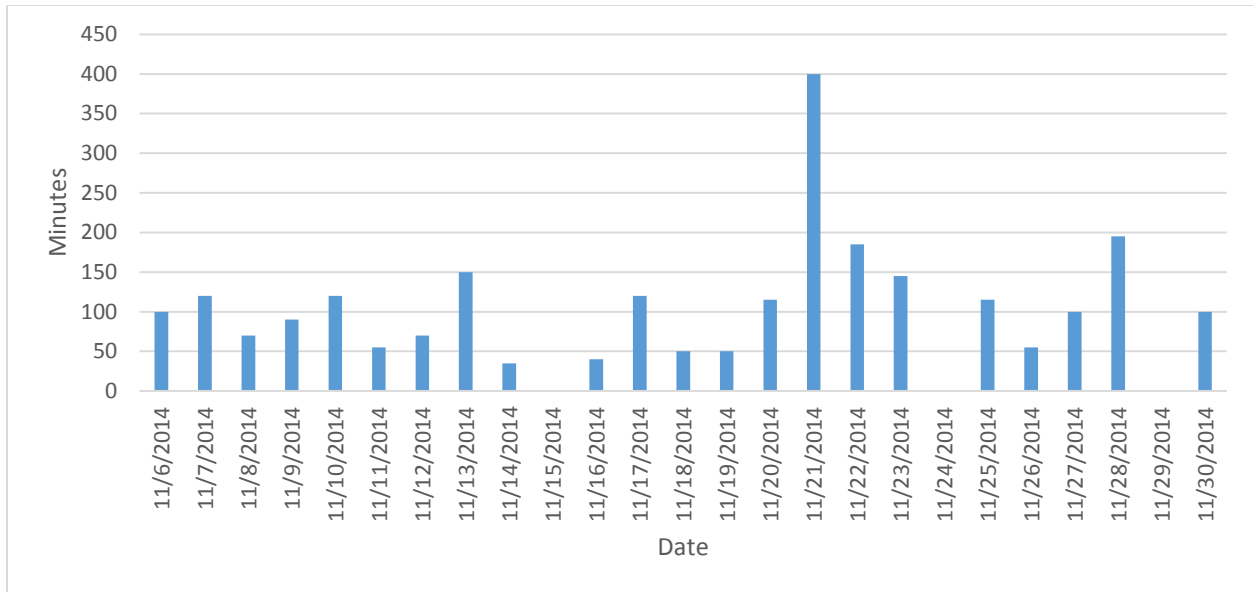


Figure 17: Total Delay per Day

Overall about 57 hours in production time was lost due to delays. While a vast majority of the delays were due to traffic, some were caused by maintenance, pilot changes, and collisions. Maintenance and collision delays are unforeseen circumstances, making them hard to prevent and even harder to predict. However traffic delays are easier to foresee. These traffic delays could be mitigated through better communication between the ACP traffic division and Dredging International.

Application of Best Practices

After researching dredging practice and the site description, it was concluded that trailing suction hopper dredgers are the best fit for the maintenance dredging. Even though the type of vessel has been decided, there are still numerous options for the completion of the project.

General Operations

The general operation of the maintenance dredging project has little room for optimization. The shift schedule and refueling times were both set in stone for the current project. Any changes in the general operations would have to take place in future maintenance dredging projects.

Loading

The loading stage of the dredging operation has many different elements. To further improve the dredging project, these elements were isolated and studied to determine if they were being completed with the utmost efficiency.

Overflow

The Breughel dredger was equipped with an overflow valve that can be used to increase the slurry density. Since the sediment has a higher density than water, as the hopper becomes full with slurry, the top levels that have low concentrations of sediment are released back into the channel. This technique was used to increase the productivity of a dredger as the hopper will hold slurry with a higher content of sediment, the volume of which was used to measure productivity.

With the maintenance dredging of the Canal however the use of an overflow was nearly impossible. Since the sediment being collected by the trailing suction draghead was of a relatively low density, it takes a long time to settle. If the overflow were to be used, the density of the slurry exiting the hopper would have a water content that was not much higher than the original slurry. Additionally, environmental concerns of dumping slurry with higher sediment contents means the Breughel must first seek approval before using the overflow valve. These two problems make the use of an overflow valve impractical for the maintenance-dredging project.

Sediment Removal

The Breughel dredger focused less on what sector it was dredging, and more on the production values it was able to achieve. Even though Sector IV has a completion date a month after the other three sectors, it was still dredged when the opportunity arose. By focusing on production rather than just dredging Sectors I, II, or III, the Breughel was exemplifying best practice for dredging efficiency.

Sailing

The second aspect of the operation was sailing. Sailing from the site to discharge area and sailing from discharge area back to the site was combined as one component due to their similarities. Though the options in this process were very limited, it was still necessary to explore any way for the sailing aspect to become more efficient. Improving sailing speed and minimizing sailing distance was the focus for improving sailing time.

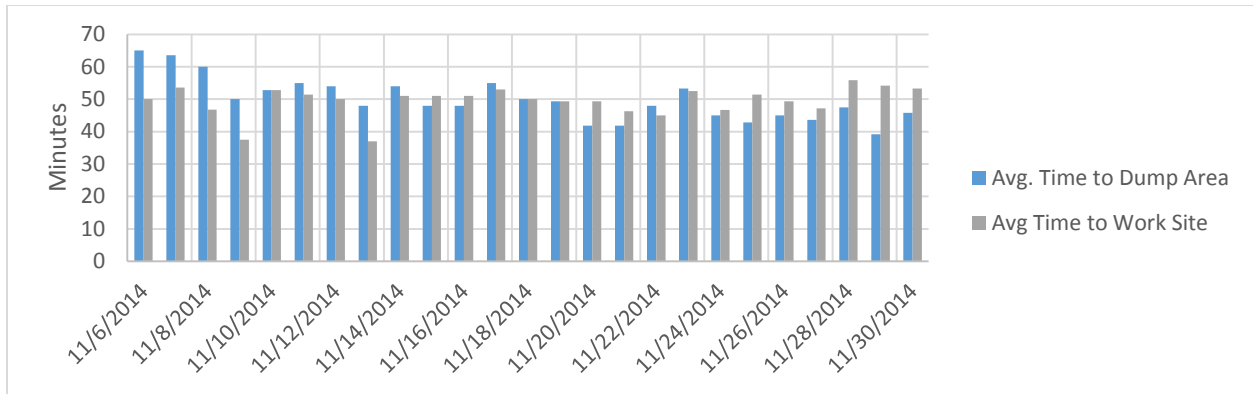


Figure 18: Average Sail Time

The above figure illustrated the time the dredge spent in transit. On average the dredge spent about the same amount of time travelling to and from the dump site, 48 and 49 minutes respectively. The transit time remains constant despite the change in sediment load.

Sailing Speed

The sailing speed discussed was not dependent on the distance as it was the velocity of the vessel and not the time it takes to travel. It was also dependent on many external factors including Canal restrictions, vessel traffic, and weather conditions. The sailing speed of the Breughel was a fixed speed and any change in efficiency from going faster would be negligible.

Sailing Distance

The current distance between the dredging site and discharge area was a fixed length. For the current maintenance project, discharging at sea was the most feasible option. The only way to reduce the sailing distance was to have a different method of discharge.

Discharge

Another area that has potential for improvement was the discharge of sediment from the dredger. The operation currently uses a dump site in the Pacific Ocean, meaning the vessel must sail to and from this site to discharge sediment. To improve sailing time between these two sites, alternative means of discharge must be considered. In addition to its bottom door, the Breughel also has the ability to pump its sediment to other barges for transportation or use a pump to shoot the slurry back to land for reclamation.

Barge Transportation

An additional way to transport sediment from the project site was by use of a barge. By pumping slurry into the barge instead of the dredger's hopper, there would be no waste of time in sailing to the discharge area. If the Breughel used a barge, they would save over 45 minutes that was required to sail and unload the hopper.

However, since the Breughel must dredge while navigating a busy channel, having an additional vessel would only complicate matters. The cost of having a barge would also offset any savings from the increase dredging production. Finally, given the size of the project and the time allotted for completion, there was no pressure to increase dredging production, especially at the cost of adding vessels and their crew to the project.

Sediment for Land Reclamation

Another idea to eliminate the need for a disposal site was to use the dredged sediment for land reclamation. Again, as with barge transportation, there are multiple issues that would eliminate any increase of production. First, according to Bray, the best granular fill material was superficial sandy deposits (Bray, 1997). The fine, clay-like sediment found at the Pacific Entrance navigational channel would not be ideal. Additionally, because the overflow valve of the Breughel cannot be used due to environmental concerns and practicalities mentioned previously. Thus, the slurry that would be pumped to shore would contain high concentrations of water and would not benefit the land reclamation.

Another problem with using sediment for land reclamation was that there are currently no places where land reclamation was needed. The goal of the project was to widen and deepen the navigational channel, adding land to the sides of the channel would be detrimental to the purpose of the operation. Even though the nearby Borinquen Dam project uses clay in its design, the clay found at the bottom of the channel would not meet the strict regulation requirements of the project. Though land reclamation was a great way to increase the value of a dredging operation, in the maintenance dredging of the Canal this would not be the case.

Canal Authority Dredging Need

In addition to improving the efficiency of the current maintenance dredging operation, the ACP has tasked us to determine the best possible way to complete future dredging projects. One major change that the ACP could implement was to have its own internal dredging division complete the task. To

examine the possibility of completing maintenance dredging internally, ACP dredging capacity was compared to their dredging need.

ACP Dredging Fleet

Dredging operations have been active in the Panama Canal for over 100 years. In addition to dredging work that was completed by outside dredging contractors, the ACP also owns and operates four dredgers. The following information was gathered from the ACP website under authorization of our sponsor.

The Mindi dredger is a cutter-suction dredge that can remove soft and medium hard sediment. It is a steam powered vessel that was originally bought for seven million dollars in 1943. Throughout its over 70 years of service it has undergone many improvements. In 1979, it was converted to run on diesel power, improving its power and dredging ability. Other improvements include adding modern electronic tools and a new control booth operator in 1985. Additionally, a computerized control was added in 1997 and GPS was installed. The new and improved Mindi dredger has been used in recent projects related to the modernization and expansion of the Canal.

A second dredger in the ACP fleet is called the Rialto M. Christensen, named in honor of an engineer who worked on the Canal. It is a mechanical bucket dredger and was commissioned in 1977. At a cost of six million dollars, it was one of the largest bucket dredgers in the world. It has the ability to remove material to a depth of 18 meters, and has a dredging capacity of 11 cubic meters. The bucket dredger was used for many projects related to the expansion of the Canal. It is especially used where the material being dredged is hard and the work cannot be completed by a suction dredger.

The third dredger, purchased in 2008 from IHC Engineering Business Ltd, is called the Quibian I. Like the Mindi, it is a cutter suction dredger that is used for the dredging of soft to medium sediment. It is a modern dredger that has up to date computer tools such as GPS and electronic controls.

Rounding out the ACP dredging fleet is the Aleman Zubieta dredger. It is a hydraulic backhoe dredger that was built in 2011 by IHC Engineering Business Ltd. It has the ability to dredge to a depth of 18 meters with an 11 cubic meter volume bucket.

Maintenance Dredging Cost

Dredging International was awarded a lump-sum contract. It was paid based on the volume of dredged material. Based on the maximum paid volume (2,071,726 m³) and the contracted \$5.96 per cubic meter,

Dredging International can be paid a maximum of \$12,347,490. Based on the minimum paid volume of 1,234,340 m³, the minimum Dredging International will be paid at least \$7,365,666.

Possible Expansion of Dredging Fleet

After reviewing the current ACP dredging fleet and the costs associated with hiring outside companies to do maintenance dredging, there was potential for the ACP to purchase additional dredgers. If a purchase was to be made, a trailing suction hopper dredger, much like the Breughel, would be the best addition to the dredging division. All the benefits of such a dredger mentioned early in this report still apply. Since the dredging fleet would primarily complete maintenance dredging, most of the sediment that needs to be removed is soft clay-like material. Rocks and other hard material are primarily found in new excavations where sediment has had time to settle and harden.

Recommendation 1: The ACP should purchase a trailing suction hopper dredger to use on future maintenance dredging operations.

The second objective of our project was to research and design possible improvements to the Pacific Entrance maintenance dredging operation. Though we originally believed these changes would be implemented on the current dredging project, giving the timing of our findings our recommendations are geared towards the improvement of future maintenance dredging projects done by the ACP. After a review of our findings and research, we recommend the ACP invests in the purchase of a trailing suction hopper dredger.

Even if it was decided to buy a trailing suction hopper dredger, there are still many options on the exact specifications of the vessel. Dredging prices vary based on a number of factors including age, size, and ability. Trailing suction hopper dredgers have prices that vary from \$4,000,000 to \$32,000,000 (Dredging Brokers). The more features a dredger has, such as a land reclamation pump or overflow valve, the more expensive it will be. Due to the nature of the Canal maintenance dredging operations, many of these features are an expensive luxury that was not needed. For the Canal to optimize its dredging fleet a trailing suction hopper dredger of small size, with a bottom door discharge method would be ideal. Though a smaller dredger would have a lower capacity, it would continually be in use, making it more cost effective. Table 9 below summarizes extra features that would not be needed on a dredger purchased by the Panama Canal.

Table 9: Summary of Unnecessary Features

Feature	Reason It Was Not Needed
Overflow	Environmental restrictions prohibit overflow valve use without permit
Land Reclamation Pump	The sediment of the Pacific Entrance was not suitable for land reclamation.
Large Hopper	A smaller vessel would be ideal for navigating the busy channel and reduce traffic delay.

Alternatively, when considering the purchase of a dredger there are many factors that should be taken into account. Many capabilities of the Breughel have proven to be very effective in increasing the production of the dredging operation. The following table lists different abilities that any dredger the ACP purchases must have.

Table 10: Summary of Needed Dredging Features

Feature	Reason It Was Needed
Maneuverability	Any dredger that operates in the busy channels of the Panama Canal must have high maneuverability to reduce traffic delays.
Bottom Door Discharge	Since the hopper must be emptied at sea a bottom door discharge was needed to complete the task at the fastest rate.
Modern Computer Technology	GPS and other computer tracking software was needed to ensure the dredger’s ability to communicate with other vessels. By having the information of other vessels, traffic delays can be reduced.

Conclusion

Maintenance dredging was a frequent and much needed task in order to ensure the operation of the Pacific Entrance of the Panama Canal. Through a cooperative effort from the ACP and Dredging International, the maintenance dredging project will be completed on schedule.

Based off of the daily reports submitted by Dredging International, the design depth of the project was identified. After conducting research and observations, the practices used by Dredging International and the ACP were determined. The gathered information was analyzed and opportunities for improvement were identified. The data collected was not only relative to this project, but may also be applied to future dredging projects on the Panama Canal.

Development of As-Built Stitch Grouting Drawings for Borinquen Dam 1E

Introduction

The new Pacific Access Channel (PAC) requires the construction of 2 earth and rockfill dams that will separate the higher PAC water level from the lower Miraflores Lake water level. One of these dams, the Borinquen Dam 1E is currently under construction and extends 2.4 kilometers from the existing Pedro Miguel Locks to Fabiana Hill. Dam 1E is underlain by a complex geologic setting which comprises of sedimentary rock of the La Boca Formation and Pedro Miguel Agglomerate. Seepage through the foundation of Dam 1E will be controlled by using foundation permeation grouting. The objective of permeation grouting is to intersect and seal fractures and voids within the rock mass in order to decrease the hydraulic conductivity of the foundation and consequently, seepage under the dam. The foundation seepage control for Dam 1E involves the use of a two-row grout curtain with holes inclined in the opposing directions along the entire dam alignment.

The complex geology of the Dam 1E site also includes extensive faulting, including the Pedro Miguel Fault which crosses the dam footprint at the south abutment on the slope of Fabiana Hill. Stitch grouting is being used to treat major faults of shear zones and other geologic features exposed during foundation excavation and cleanup. Areas to be stitch grouted are designated by the Grout Specialist and details of the stitch grouting design are provided via Design Field Instructions (DFIs). Stitch grouting entails drilling lines of grout holes across the shears at several locations within the core foundation area. Each stitch grout line includes at least five or 6 holes that cross the shear zone at various depths and angles appropriate for the orientation of the feature being grouted. The Dam 1E stitch grouting program includes the treatment of 10 faults and geologic structures identified along the dam foundation.

Background

This section of the report regards the production of As-Built drawings which demonstrate the response of the rock foundation to permeation grouting. The stability of the foundation of Borinquen Dam 1E is extremely important since the dam obliquely crosses the Pedro Miguel Fault, which is one of the largest faults in the world.

In this section, the various reasons of dam failure are discussed including poor site selection, poor design/construction, seismic activity and, impact. The design of the foundation by the contractor on the project is also discussed including the design criteria and specifications for the dam and detailed description of the various foundation stability efforts implemented.

Dam Failure

The failure of a large dam has the potential to cause more fatalities and destruction than any other man-made structure. Such drastic death tolls and damage are a result of the destructive power of flood waves which are released from the sudden collapse of a large dam (Woodward, 2004). While larger dams have greater holding capacities than smaller dams, smaller dams have proven to be less reliable with about 1 percent of failures each year. The Teton Dam in Idaho stood at a height of 95 meters and caused 14 deaths when it collapsed in December 1976. The significantly smaller Johnstown dam in Pennsylvania was 23 meters high and resulted in 200 deaths when it collapsed in May 1889 (Goldsmith, 1984).

For these reasons, all aspects of the dam design need to be carefully assessed to account for any and all discrepancies that may occur during construction and during the life time of the dam. The main reasons for dam failure can be categorized into four factors. These are poor site selection, poor design/construction, seismic activity and impact/collision.

Poor Site Selection

With more dams being built every day, less appropriate sites for dam erection are available. This limitation in site locations will cause dam construction to occur in less suitable places. When choosing a dam site location, the following parameters should be taken into account (Becue, 2012):

1. **Topography and inflow of water into the catchment area** –the site location needs to be able to have a source of water to fill the catchment area during dry weather periods. Lakes bounded by dams are used for recreational purposes and hydroelectric power sources; therefore, functionality needs to continue during dry weather.

2. **Morphology of the river valley** – a dam needs to be naturally associated with the environment in which it is constructed in. The ideal area in which a dam should be constructed includes a narrow location whereas valley widens upstream of where the future dam is to be built. On the other hand, such site locations are hardly found due to the lack of a narrow section in the natural structure of a valley or due to the choice of the site location not completely being dependent on engineering conditions.
3. **Geological and geotechnical conditions** – the presence of joints, faults and shear zones, along with the permeability, nature, and strength of the rock are also deciding factors when choosing a dam site location. The foundation needs to be sound or well compacted to ensure a stable structure.

The failure of the St. Francis Dam falls into this category. After its failure, studies showed that some of the foundation rock had disintegrated when the rock became fully saturated shortly after the reservoir was filled (Woodward, 2004). For the reasons listed above, the location of a dam site needs to be carefully selected before design can even begin.

Poor Design/Construction

The design and construction of a dam is another factor which contributes to the success or failure of a dam. From the design standpoint, besides the geological setting described earlier, designers need to take into account the functionality of the dam (URS Holdings, Inc., 2004).

An important factor when designing a dam is the holding capacity. This information is necessary so proper measures can be taken to avoid overtopping. Overtopping occurs when the actual capacity of the dam exceeds the maximum capacity and water flows above the highest level of the dam (Bates, 1984). This may cause permanent damage to the dam. In the past 75 years, overtopping has accounted for approximately 30 percent of dam failures in the U.S. Most dams will likely not withstand sustained overtopping of a foot or more without a high probability of failure. The Gibson dam in northern Montana overtopped on June 6-8, 1964 was due to the breakdown of the spillway gate system causing 2 of the 6 spillway gates to not fully open. The dam was overtopped so that the capacity reached about 3 feet over the crest of the dam and lasted for over 20 hours. The design of the dam was improved in 1981 which implemented a cofferdam which allows overtopping (U.S. Department of the Interior Bureau of Reclamation, 2013).

From the construction standpoint, a dam may fail due to substandard workmanship. If the foundation is not fully stabilized, or the dam material is not compacted properly, the dam can be displaced over time which can ultimately lead to failure (Goldsmith, 1984). This was the case for the Gleno Dam in Italy which failed on December 1st, 1923. Soon after the reservoir was filled for the very first time, a buttress on the dam cracked and the dam failed killing approximately 350 persons. After failure, research discovered that the dam probably failed due to the use of inferior building materials and poor workmanship. The concrete used was of poor quality and it was not allowed to fully cure before the reservoir was opened. Also, for reinforcement, anti-grenade scrap netting from World War I, was used in most of the reinforced concrete structures on the dam.

Seismic Activity

Another cause of dam failure is seismic activity. This is a major concern in the case of the Panama Canal since the Borinquen Dam 1E is positioned across the Pedro Miguel, Limon, and secondary faults (Seismological Society of America, 2010). Paleoseismic studies done by Rockwell, et al., demonstrates that “both the Limon and Pedro Miguel faults are seismically active, having a relatively short recurrence rate for large earthquakes, with displacements in the range of 1.5 to 3 meters (4.9 to 9.8 feet)” (Seismological Society of America, 2010). For these reasons, the ACP has specified that the Borinquen Dam should be able to withstand a colossal 2,500 quakes a year with little to no damage to the dam. These stability efforts are demonstrated by producing a sound foundation and dam core to absorb shocks caused by such tremors. The Lower San Fernando Dam in California, USA failed during an earthquake in 1971. This earthquake resulted in the liquefaction of the fill in the dam, resulting in the collapse of the upstream section of the dam. The only reason a catastrophic flood did not occur was because the reservoir level was well below the maximum capacity at the time that the earthquake occurred (Woodward, 2004).

Impact/Collision

The final failure mechanism discussed in this report is impact and collision. Similar to seismic activity, there are design specifications stated for the case that a ship collides with the dam. These will be discussed in more detail in the Embankment Design Section of this paper.

Foundation Design

As previously mentioned, the west branch of the Pedro Miguel Fault has been mapped obliquely across the Borinquen Dam 1E footprint underlying approximately 200 meters of the dam between stations 1+950 and 2+150 shown below in Figure 19 (URS Holding, Inc., 2009). Many of the boring logs along the entire

length of the dam indicate the presence of crushed rock, sheared and altered as a result of rock movement. These indications are referenced in the logs as shear zones, fault zones, gouge, and slickensides on fracture faces.

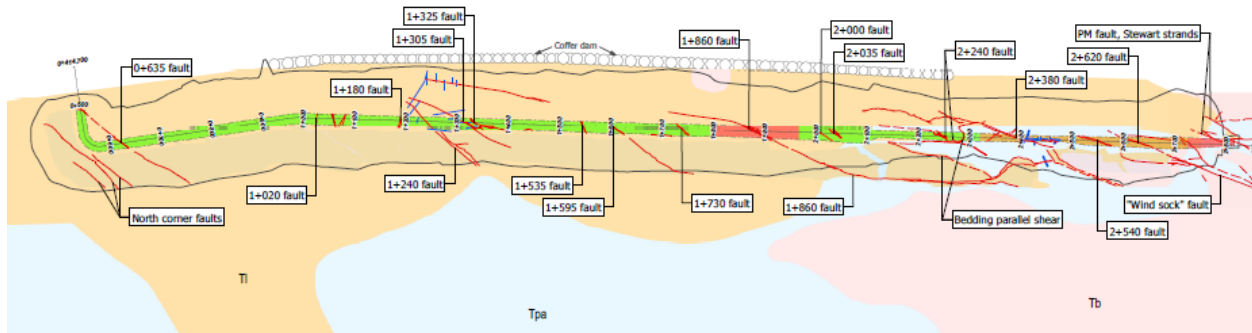


Figure 19: Faults and Shear Zones Located within the Footprint of Borinquen Dam 1E (URS Holding, Inc., 2009)

Therefore, the Borinquen Dams have several design constraints due to some of the failure mechanisms previously discussed. The design of the dams posed challenges including (United States Society on Dams, 2011):

1. Variable foundation conditions with occasional weak features;
2. A high seismic hazard, including possible surface fault rupture across the dam foundations; and
3. Potential for grounding of Post-Panamax-size ships against the inboard face of the dams.

Design Criteria

For these reasons, it is important that the dam foundation has sufficient strength for static and seismic stability, avoids liquefaction potential, minimizes settlements, and allows treatment to control seepage (URS Holdings, Inc., 2009). These requirements were displayed in the design criteria set by the ACP.

Stability Criteria

Moderate inboard (Pacific Access Channel Side) and outboard (Miraflores Lake Side) slopes stand at a ratio of 3:1. These slopes provide seismic stability by limiting embankment deformations under earthquake shaking. Table 11 below dictates the stability criteria stated by the ACP. The water surface elevation under various loading conditions must acquire the minimum factors of safety shown below. These values for the safety factors are derived from the stability analyses conducted using Spencer's method of slices. The results of the slope stability analyses indicate that the safety factors exceed minimum acceptable values for all stages of dam construction, long-term, and operating conditions.

Table 11: Embankment Stability Criteria (URS Holdings, Inc., 2009)

Load Condition	Slopes	Water Surface Elevation		Minimum Acceptable Factor of Safety
		Inboard Side	Outboard Side	
End of and During Construction	Inboard and Outboard	Empty	Maximum Operating Level	1.3
Long Terms, Steady Seepage	Outboard	Maximum Operating Level	Maximum Operating Level	1.5
Maximum Flood Surcharge Pool	Outboard	New Locks Design Flood Level	Maximum Operating Level	1.4
Rapid Drawdown	Inboard	Maximum Operating Level	Maximum Operating Level	1.3

Seismic Criteria

The two branches of the Pedro Miguel Fault that are mapped within the Borinquen Dam 1E footprint are found to be active. The east branch of the Pedro Miguel Fault is mapped along the north side of Fabiana Hill where it intersects the southeast section of the Borinquen Dam 1E at station 2+800. The second branch is located at station 2+100. There is a third branch of the Pedro Miguel Fault which is mapped to the west of the dam footprint. While this third strand does not intersect the dam footprint, it may still be a threat to the stability of the dam due to mapping errors and secondary faults that may be associated with this strand (URS Holdings, Inc., 2009).

Designers have acknowledged the possibility of concealed faults which would not have been recorded during the initial mapping of the fault. These faults may be revealed during excavation. With this, the dam design must be incredibly sturdy to accommodate potential displacement along the concealed faults. The seismic design criteria allows for up to 1 meter of lateral slip within the Pedro Miguel Fault zone and 3 meters outside of this fault zone, as well as 0.5 meter of thrust slip anywhere within the dam 1E foundation.

Seismic dam deformation must not compromise the ability of the structure to retain the Gatun Lake, lead to overtopping, or require emergency response that impedes the operation of the canal.

Foundation Treatment

In order to fulfill the criteria previous stated for the foundation of Dam 1E, several steps need to be taken. Firstly, the area needs to be excavated until sound rock is found and that surface needs to be treated. Next, the area needs to be properly dewatered, and cutoff walls installed in the necessary areas. Lastly, grout curtains need to be created to control seepage (URS Holdings, Inc., 2009).

Foundation Excavation Objectives

Excavation will be required to remove all soils including fill, alluvium and residual soils. These materials will need to be removed from the foundation as they are considered to be unacceptable for static and seismic stability. To meet foundation objectives, the following needed to be achieved (URS Holdings, Inc., 2009):

1. The Zone 1 core, Zone 5 chimney filter, and Zone 6 chimney drain will be created on slight to moderate, or less weathered rock that has the following strength classifications:
 - a. For agglomerate of the Pedro Miguel Formation, the minimum strength will be medium hard
 - b. For La Boca and tuff of the Pedro Miguel Formation, the minimum strength will be soft
2. The Zone 3 inboard shell/blanket filters/ transitions and outboard shell will be created on highly or less weathered rock that has the following strength classifications:
 - a. For Pedro Miguel Agglomerate, the minimum strength will be soft
 - b. For La Boca Formation, the minimum strength will be soft

Foundation Surface Treatment

Surface treatment of the core foundation is necessary in order to achieve the following (Fell, 2005):

1. Decrease the surface leakage of grout, thereby improving grouting effectiveness;
2. Prevent loss of joint filling materials;
3. Prevent piping of embankment materials into foundation fractures and piping along the foundation contact; and
4. Decrease the potential for differential settlement and stress concentration that might result in embankment cracking.

To achieve a relatively smooth surface free of irregularities, protrusions, overhangs, pinnacles, and steep rock surfaces, rock shaping needs to be done. These imperfections could cause cracking of the embankment or impede the placement and compaction of the clay core. This process involves the following steps (Fell, 2005):

1. The surface of the rock foundation needs to be cleaned for inspection and geologic mapping;
2. Dental excavation and dental grouting need to occur along with concrete backfilling to seal discontinuities and prevent piping; and
3. The surface of the rock foundation then needs to be cleaned again prior to embankment fill placement.

For smaller fractures where more massive and less fractured agglomerate is exposed to the core foundation, slush grout will be used. A layer of shotcrete will be applied in areas that are highly fractured or in sheared areas of Pedro Miguel agglomerate and other harder rocks in the Zone 1 foundation where treatment with slush grout would not be practical. In areas which are more susceptible to deterioration when exposed, such as the La Boca formation, a 300 millimeter sacrificial layer of rock will be left in place during the grouting process and will be removed just before the placement of the embankment fill (URS Holdings, Inc., 2009).

Dewatering

For the Borinquen Dam 1E, groundwater levels can reach as much as 18 meters above the dam foundation. The groundwater levels in this area are controlled by the Gatun and Miraflores Lakes. These levels will need to be lowered to approximately 1 meter below the dam foundation prior to excavation in order for the embankment construction to occur in dry environments. These groundwater levels will be reduced using a combination of dewatering wells and dewatering trenches. The dewatering wells will be installed along the outer limits of the work; whereas dewatering trenches will be excavated as work progresses. Dewatering wells function by drawing down groundwater to stabilize provisional excavation in the residual soil, fills, and alluvium, as well as control the seepage into the excavation (URS Holdings, Inc., 2009).

Pedro Miguel Dam Cutoff Wall

The Pedro Miguel Dam (PMD) is situated between the Paraiso Hill and the Pedro Miguel Locks. This dam retains the Gatun Lake and was first constructed in 1911 during the original canal construction, and currently acts as a cofferdam during the excavation of both the Pacific Access Channel (PAC) and the north

end of Dam 1E. In the past, the PMD has functioned safely for a maximum head differential of approximately 10 meters between the Miraflores and Gatun Lakes. During the construction of Dam 1E, this head differential will rise to 18 meters (URS Holdings, Inc., 2009). Therefore, something needs to be done to control the seepage through the PMD which could result in the failure of underlying piping and foundation, as well as the ultimate release of the Gatun Lake. A solution to this problem would be the construction of a cutoff wall. A cutoff wall is an impermeable wall placed beneath or within a dam that prevents or impedes seepage (Bates, 1984).

The PMD cutoff wall that was constructed is approximately, 458 meters long and extends from elevation 28 meters to 10 meters. This cutoff wall penetrates five soil types. These include Pedro Miguel Dam select fill, Pedro Miguel rock/waste fill, Upper pre-1907 fill, Basal pre-1907 fill, and Residual soil overlying La Boca Formation sandstone (URS Holdings, Inc., 2009).

When designing the PMD, there were 6 options (URS Holdings, Inc., 2009):

1. Steel Sheetpiles with Jet Grouting
2. Deep Soil Mixing (DSM) with Jet Grouting
3. Jet Grouting
4. Plastic Concrete Secant Piles
5. Slurry Trench Cutoff Wall
6. Grouted Wall

These options were assessed based on performance, construction risk, constructability, and cost. Based on these assessments, the Slurry Trench Cutoff Wall was chosen because of its high performance and low cost. A cement bentonite slurry was selected for the design and contains cement to stabilize the trench during the excavation phase. After completion of the excavation, the cement causes the slurry to harden to a strength comparable to that of a stiff clay, thereby eliminating the need for trench backfilling. The 28-day compressive strength of the cement bentonite backfill ranges between 200 to 400 kPa which is a comparable compressive strength to the surrounding soil material (URS Holdings, Inc., 2009).

Foundation Seepage Control

Seepage will be controlled using foundation grouting throughout the entire dam. Foundation grouting consists of drilling a line of holes from the cutoff level of the dam into the dam foundation (Bates, 1984). Cement slurry or chemicals are forced into these holes under pressure into the rock defects including joints, fractures, bedding partings and faults. Grouting aims to accomplish the following (Fell, 2005):

1. Reduce leakage through the dam foundation;
2. Reduce seepage erosion potential;
3. Reduce uplift pressures; and
4. Reduce settlements in the foundation.

Foundation grouting takes two forms: Curtain Grouting and Consolidation (Fell, 2005). Curtain grouting, specifically permeation grouting is used in the Borinquen Dam 1E. Permeation grouting functions by creating a narrow barrier or curtain in highly permeable rock. This grouting method usually consists of a single row of grout holes which are drilled and grouted to the base of the permeable rock.

Number of Grout Rows

For larger dams multiple rows are needed to increase the likelihood of grout interception by neighboring holes. For Borinquen Dam 1E, a two row grout curtain is used. These holes are drilled at 20 degrees from the vertical in opposing directions. These grout rows are referred to as Row A and Row B and are aligned 1.5 meters offset from the dam alignment on the outboard and inboard sides of the dam, respectively (URS Holdings, Inc., 2009).

Stitch Grouting

In areas where shear zones are present in the foundation, additional measures need to be taken to ensure that the foundation meets the stability criteria set by the ACP. These efforts are referred to as stitch grouting. Stitch grouting uses angled fans of grout holes that are crisscrossed at various depths and locations (Weaver, 2007)

While Rows A and B span the entire length of the dam, stitch grout holes are only placed in areas which need additional permeation grouting for reinforcement. These areas are determined by a grouting specialist who has a vast background in geology and general engineering. The grouting specialist designates the location, angle, and number of rows needed in each stitch grout area. While grout holes in Rows A and B are placed at 20 degrees from the vertical on opposing sides, the angles at which stitch grout holes vary based on the geology of the rock and the slip, dip, and thrust of the fault in question. Typically for this project, 3 or 4 rows are used for stitch grouting. These rows are referred to as Line 1, Line 2, Line 3, and Line 4. Line 1 is offset 3 meters on the outboard side of Row A and Lines 2 and 3 are offset 3 and 6 meters on the inboard side of Row B (URS Holdings, Inc., 2009).

Depth of Grout Curtain

The depth of the grout curtain is determined by the depth to relatively impervious rock. To test permeability, Lugeon tests are done. Lugeon tests are an in-situ test which estimates hydraulic conductivity. Water at a constant pressure is injected into the rock mass through a slotted pipe bounded by packers which hinder the escape of the water. From this test, the amount of water in a determined area can be recorded. Along the alignment of the dam, several areas produced rock masses with permeability reaching 62 Lugeons. The ideal permeability of sound rock, in accordance with design specifications is any value less than 10 Lugeons. Lugeon tests have shown that rock masses that acquire this permeability is found 15 to 20 meters below the surface (URS Holdings, Inc., 2009).

Grout Hole Spacing

The spacing of grout holes depends on closure. The aim of grouting is to achieve complete closure or to have interception of grout to ensure the absence of gaps. Initially, holes 24 meters in depth are drilled 24 meters apart for research and geological mapping purposes. These holes are referred to as Super Primary holes. Primary holes are drilled approximately 15 meters deep at intervals of 6 meters. Super Primary and Primary holes are mandatory. Subsequent split holes (secondary, tertiary, and quaternary etc.) are drilled half way through the necessary span. Remediation holes are drilled and grouted between Rows A and B in order to ensure closure by attaining a Lugeon value of 5 (URS Holdings, Inc., 2009).

Grout Materials

The grout mixture used consists of Type III Portland cement, superplasticizer, bentonite and water. Type III cement is used since it is finer cement and is capable of penetrating finer cracks. Superplasticizer is used to reduce premature flocculation of the grout particles and reduce viscosity by helping to disperse grout particles. Bentonite is used to produce a stable mixture. In the case of the Borinquen Dam 1E, five grout mixtures are used. The mix components and quantities are displayed below in Table 12.

Table 12: Grout Mixture Components and Quantities (URS Holdings, Inc., 2009)

	Mix 1	Mix 2	Mix3	Mix 4	Mix 5
Water (Lts.)	55	30	21	13	16
Cement (kg)	85	85	85	85	85
6% Bentonite Soln. (Lts.)	48	48	48	48	45

R-1000 Additive (kg)	3.55	2.12	2.12	1.918	1.692
Density (g/cm³)	1.44 – 1.46	1.55 – 1.57	1.60 – 1.63	1.66 - 1.68	1.66 - 1.68
Viscosity (s for 100 cm³ sample)	≤35	40 - 50	60 - 70	>95	≤118
2-Hour Settlement	<2%	<2%	<2%	<2%	<2%

Methodology

The goal of this section of the project was to design a simple set of instructions for producing As-Built drawings for the stitch grout holes drilled and grouted in the construction of the foundation for Dam 1E. The purpose of these drawings is to visually demonstrate the layout of the stitch grout holes and give additional information on each hole including the grout take, dates of drilling and grouting and, the stages in which the holes are drilled. To successfully accomplish this goal, five objectives were developed. These objectives include:

Objective 1: Become well versed on the terminologies and calculations

Objective 2: Identify internal resources available

Objective 3: Become familiarized on the databases provided

Objective 4: Determine the layout of the final drawings

Objective 5: Determine mode of process presentation

To accomplish these objectives, terminologies and necessary calculations associated with grouting need to be identified. Also, resources that would be available to the ACP employee that would be developing these drawings needed to be acknowledged. The relevant information and the resource pool were used to provide a step by step instruction manual for producing these drawings, inclusive of tips and solutions to problems encountered during my experience with the database and software.

In this chapter, the methodological approach taken to accomplishing the aforementioned objectives are discussed. Under each objective, the method of data collection used, along with justification for its use is discussed.

Objective 1: Become well versed on the terminologies and calculations

The first objective was to become well versed on the terminologies and calculations. This objective was satisfied by conducting interviews and document analysis.

Semi-structured interviews were conducted with Dr. Ramon Martinez and Dr. Sarah Kemp who are both grouting specialist. Kemp is the Supervising Engineering Geologist and Foundation Grouting Specialist at URS Corporation since 1978. She focuses on water resources and hydroelectric projects worldwide and is a Certified Engineering Geologist and Licensed Professional Geologist in California, Washington, Oregon,

Idaho and North Carolina. Kemp's extensive experience in the grouting industry for the past 36 years has deemed her a reliable source of information regarding foundations and grouting. Dr. Ramon Martinez is a Geotechnician at URS Corporation. Martinez has a PhD in geotechnical engineering and was a college professor at the Technological University of Panama in the civil engineering department. He has over 18 years of experience in the field of geotechnical engineering working with URS. He was involved in the management and direction of geotechnical and interdisciplinary projects in the US and various Latin-American countries. His experience centers on soil engineering and the creation of geotechnical structures. Martinez is a credible source of information given his experience in the geotechnical field. Also, being a professor in Panama, he was able to provide clear and concise explanations of concepts and terminologies.

Document analysis was another initiative taken in accomplishing this objective. Textbooks on the geotechnical engineering of dams were read to familiarize myself it background topics regarding foundations. These textbooks were also a great source of explaining the derivation of equations and the variables associated with these equations. These books are written by engineering geologists and foundation engineers who have experience in these areas. These books were found in the library of the URS consulting office and are the same books that Martinez and Kemp reference in their designs. After the basics were solidified, design reports and internal studies were studied to apply the information obtained from the text books. These reports were written by URS consultants including Martinez and Kemp. They include specific information on the foundation design for the foundation of Dam 1E including methods used, geotechnical constraints, and construction efforts. These documents were valuable sources of information as it was possible to understand the conditions under which the dam is being built and the justification of the foundation methods used.

Objective 2: Identify internal resources available

This objective was accomplished by conducting interviews with CAD technicians on site. CAD technicians were able to give me information on the versions of AutoCAD provided by the ACP and also provided me with the Visual Based Macro used to interpret Microsoft Excel databases to produce the As-Built drawings. These CAD technicians have been working with the ACP since the beginning of the expansion project and were well versed on the different types of software that were available and their capabilities.

Objective 3: Become familiarized on the databases provided

This objective was accomplished by document analysis and interviews. Interviews were conducted with technicians from the control rooms on site. These technicians have been working night and day, recording information on the drilling and grouting process and presenting it in individual databases for each hole. These technicians were able to explain where the figures in the database come from and how this data is analyzed before reaching the CAD technician who produces the As-Built drawings. These persons are qualified to speak on this topic since they have experience in the grouting field in similar projects in Spain and Italy.

The database of compiled grouting log information and associated equations were also studied using Microsoft Excel files provided by the consultant and the technicians from the control room. This database was well sorted in numerous tabs, each calling information from the main database and selecting specific fields to be displayed. This database is where all the numerical data necessary for the As-Built drawings was found. Grouting logs provided by the control room technicians were also analyzed. These databases contain detailed information on each hole. In the case that information went missing in the main database, it was possible to utilize these grouting logs to “fill in the blanks.”

Objective 4: Determine the layout of the final drawings

This final objective was accomplished by conducting an interview with Martinez. The drawing production protocol used on this project was discussed and a custom border for the drawings was formulated. The scale, layout, colors and, organization of drawings were also discussed to ensure that the drawings were presented in a systematized and neat manner.

Objective 5: Determine mode of process presentation

This objective was accomplished independently. After working with the resources made available, including databases and system software, options for presentation of the process were analyzed. These presentation styles were assessed in order to evaluate which mode would be the easiest for a technician to understand. This technician would be producing these drawings and therefore the design processes needs to be catered to them, in terms or terminologies used and level of experience with the software and databases used.

Project Restraints

During this project, there were few constraints. These restraints were mostly administrative. It took several weeks for a computer with the associative databases and system software to be provided. Other

than this single limitation, there were no issues receiving information. The employees from both ACP and URS Corp. Inc. were all helpful and willing to assist.

Findings and Recommendations

During this seven week internship, grouting terminologies and calculations have been researched and discussed, software and databases associated with the production of As-Built drawings have been identified, and the layout of these drawings have also been identified.

Finding 1: Grouting is performed in stages no longer than 6 meters to more effectively target fractures and other defects in the rock

To ensure that grout holes are drilled and grouted properly, grouting needs to take place in stages to avoid the collapsing of a hole due to the irregular grout flow and rock permeability and quality (Fell, 2005). The length of each stage is determined based on several factors. These include:

1. Data on the geological setting and depths at which the permeability of the rock is likely to change
2. The minimum length that is worth drilling; if shorter lengths are selected, increased costs will be incurred due to additional stages
3. The maximum allowable pressure that the rock in the zone closest to the surface is able to withstand

In the case of the grouting done on Dam 1E, the following stages stated in Table 13 below are typically used in holes up to 16 meters deep (Fell, 2005):

Table 13: Typical Grout Stages Used in Borinquen Dam 1E

Stage	Depth Range (m)
1	1 – 6
2	6 – 11
3	12 - 16

There are two main methods used on this project to drill and grout a hole in stages (URS Holdings, Inc., 2009). These methods are listed below in order of the most common procedure used:

1. **Upstage Grouting** – This procedure includes drilling the grout hole to the full depth, setting the packer at the top of the bottom stage and then grouting that stage. This procedure is repeated in

succession until the hole is fully grouted. This procedure is illustrated below in Figure 20. This method is the least expensive method since the drill rig only needs to be positioned at the hole once. The downfalls to using this method is the possibility of the hole collapsing or being enlarged during the placing of the packer due to poor rock quality (Fell, 2005). In the case of the Dam 1E, this has not been a significant issue except for rare occasions where downstage grouting was required.

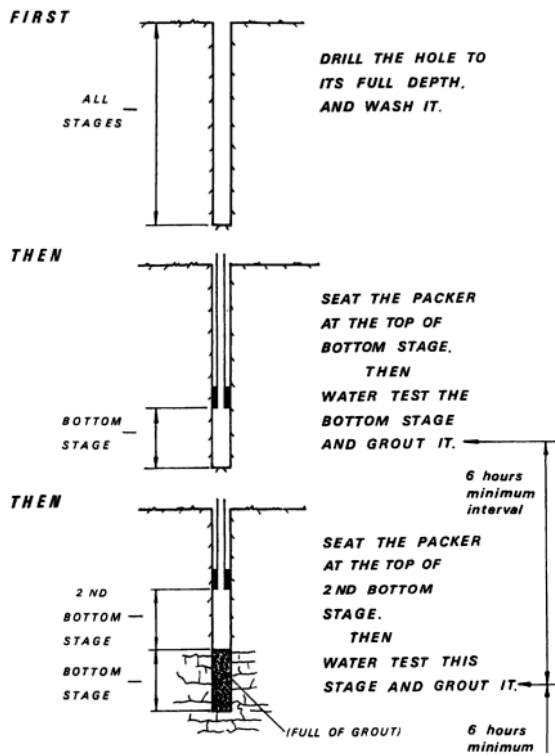


Figure 20: Upstage Grouting Procedure (Fell, 2005)

2. **Downstage Grouting** – In this procedure, the top stage is drilled and grouted first by setting the packer at the base of this stage, then allowed to cure. The same is done for the second stage and all other stages thereafter. This procedure is illustrated below in Figure 21. This method is used for holes that are prone to collapsing. The downfall for this method is that bleeding of the grout cannot be accomplished except at the ground surface. The cost is also higher for this method since it requires the drill rig to be available intermittently for each of the grouting stages (Fell, 2005).

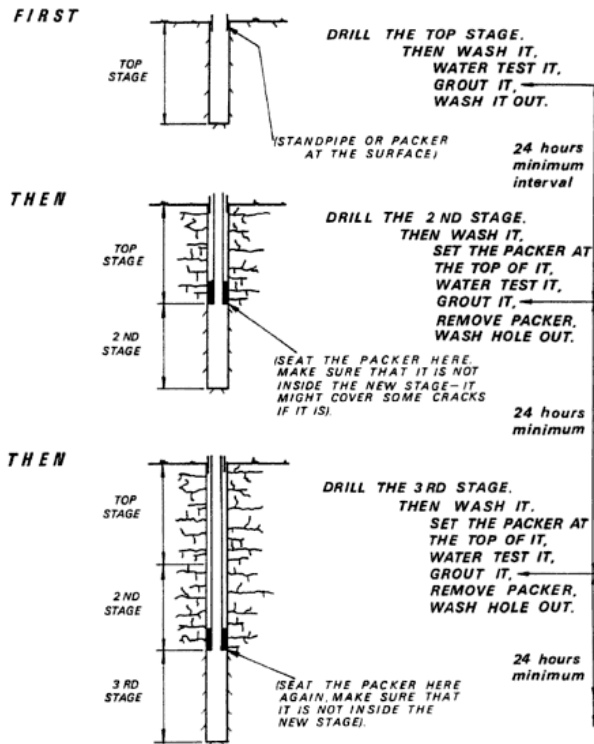


Figure 21: Grouting Downstage with Packer Procedure (Fell, 2005)

Finding 2: Mix 1 is the most common mix being used

As stated in the Background Chapter, there were five different grout mixes proposed by the Contractor. The least viscous mix was Mix 1 with a target viscosity of 33 Marsh seconds and the most viscous mix was Mix 5 with a target viscosity of 117 Marsh seconds. The project specifications allow a $\pm 10\%$ variation in the mix viscosity. However, only the first three mixes were routinely used: Mix 1 and Mix 2 to grout the rock, and Mix 3 mostly for backfilling the holes. The thinner mixes were used because of their ease of flowing into smaller rock fractures and other defects. In all holes, Mix 1 was used first; however, if the holes took large quantities of grout, the thicker Mix 2 was used to seal the openings more easily.

Finding 3: Results from Lugeon tests and grout take values in verification holes are used to prove the effectiveness of foundation grouting on the rock masses

As mentioned in the Background Chapter, the Lugeon test measures rock mass permeability. This test consists of isolating a section of a drilled hole and pumping water under pressure into that section through a slotted pipe bounded by inflatable rubber balloons called packers until the flow rate for any given pressure is constant (United States Society on Dams, 2010). The configuration of this test is shown below in Figure 22.

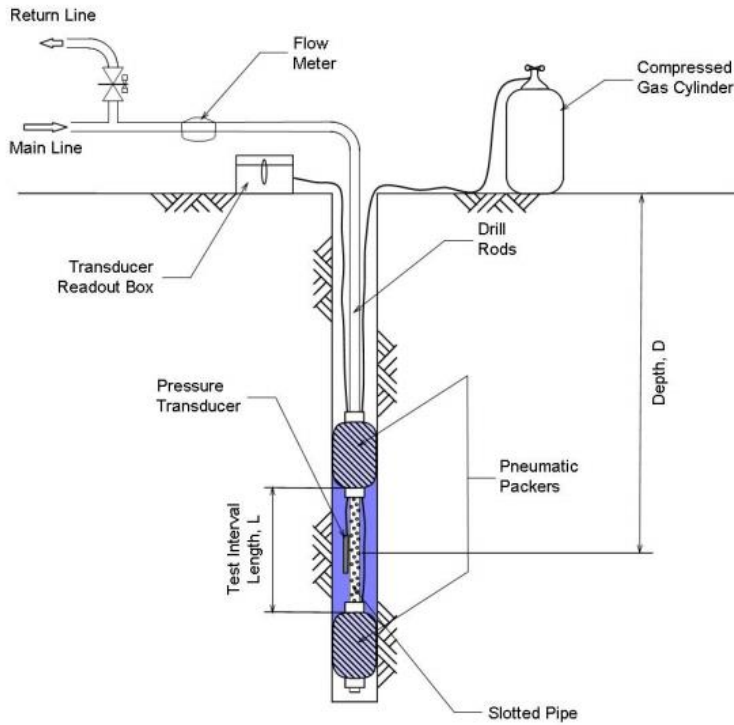


Figure 22: Configuration of Lugeon Test (United States Society on Dams, 2010)

As stated in the Background Chapter, areas within the footprint of the Borinquen Dam 1E were found to have Lugeon values of up to 62 Lugeons in testing performed during the project design phase. According to the United States Society on Dams (USSD), rock masses with Lugeon values ranging 50-100 are classified as having high hydraulic conductivities ranging from 6×10^{-4} – 1×10^{-4} cm/sec and have many open discontinuities. The aim of permeation grouting is to seal these fractures to produce a final Lugeon value of around 5 (URS Holdings, Inc., 2009). Table 14 below, from the USSD describes the properties and classifications associated with Lugeon values.

Table 14: Association of Lugeon Values and Properties and Conditions of Rock Masses (United States Society on Dams, 2010)

Lugeon Value	Classification	Hydraulic Conductivity (cm/sec)	Condition of Discontinuities in Rock Mass
<1	Very Low	$< 1 \times 10^{-5}$	Very tight
1 – 5	Low	$1 \times 10^{-5} - 6 \times 10^{-5}$	Tight

5 – 15	Moderate	$6 \times 10^{-5} - 2 \times 10^{-4}$	Few partly open
15 – 50	Medium	$2 \times 10^{-4} - 6 \times 10^{-4}$	Some open
50 – 100	High	$6 \times 10^{-4} - 1 \times 10^{-3}$	Many open
>100	Very High	$> 1 \times 10^{-3}$	Open closely spaced or voids

Lugeon tests are initially conducted on production grout holes in Rows A and B prior to grouting. Lugeon tests are also conducted on verification holes drilled along the centerline of the dam alignment after the grouting of Rows A and B (and stitch grouting, if appropriate) is completed. The purpose of two sets of Lugeon tests is to confirm that there is full closure of the grout curtain produced by Rows A and B. In theory, before Rows A and B are grouted, a higher Lugeon value is attained. After production grouting is done, Lugeon values in verification holes are supposed to be lower than the values obtained in Rows A and B prior to grouting, hence demonstrating full closure of the grout curtains. As an example, Table 15 below displays data directly from the grouting database which show the effectiveness of foundation grouting at this location. The centerline verification hole demonstrates that after Rows A and B were grouted, full closure was achieved since it could not take any more grout.

Table 15: Effectiveness of Foundation Grouting in Terms of Grout Take and Lugeon Values

Row/Station Location	Grout Take (kg/m)	Lugeon Value
Row A @ ~Station 1+865	~68	~20
Row B @ ~Station 1+865	~99	0
CL Verification @ ~Station 1+865	0	0

While Lugeon tests in verification holes are often used to demonstrate the reduction in hydraulic conductivity of a rock mass, this is not always the case. This is mainly due to the fact that water is less viscous than grout. Thus, water during Lugeon testing may still be capable of moving through very small cracks where grout could not penetrate, which may produce relatively high Lugeon values. For this reason,

the amount of grout take in verification holes is also used to evaluate the effectiveness of permeation grouting. While a grout hole may absorb more water resulting in a high Lugeon value, if it takes little to no grout, then full closure of the curtain segment being evaluated can be assumed. In terms of grout take, the Dam 1E closure criterion requires a minimum grout take of 35 kg/m or less, which corresponds to Moderately Low (ML) cement consumption in the Deere Classification System (1982), which will be further described later in this chapter.

Finding 4: Remediation holes may also be drilled and grouted if verification testing shows that the initial production grouting was ineffective

In the case that verification holes do not meet the Lugeon and/or grout take acceptance criteria, additional grouting efforts may be required until full closure is accomplished. This condition was observed in limited areas of the Borinquen Dam 1E project. Additional holes, called remediation holes, are split spaced drilled and grouted along the alignment of the production rows or along the dam centerline. There were cases where there was not sufficient space to place split spaced holes resulting in some of the remediation holes being offset by a maximum of 0.5 meters from the original alignment. Figure 23 below demonstrates the placement of production, verification, and remediation holes in a relatively short segment of the curtain.

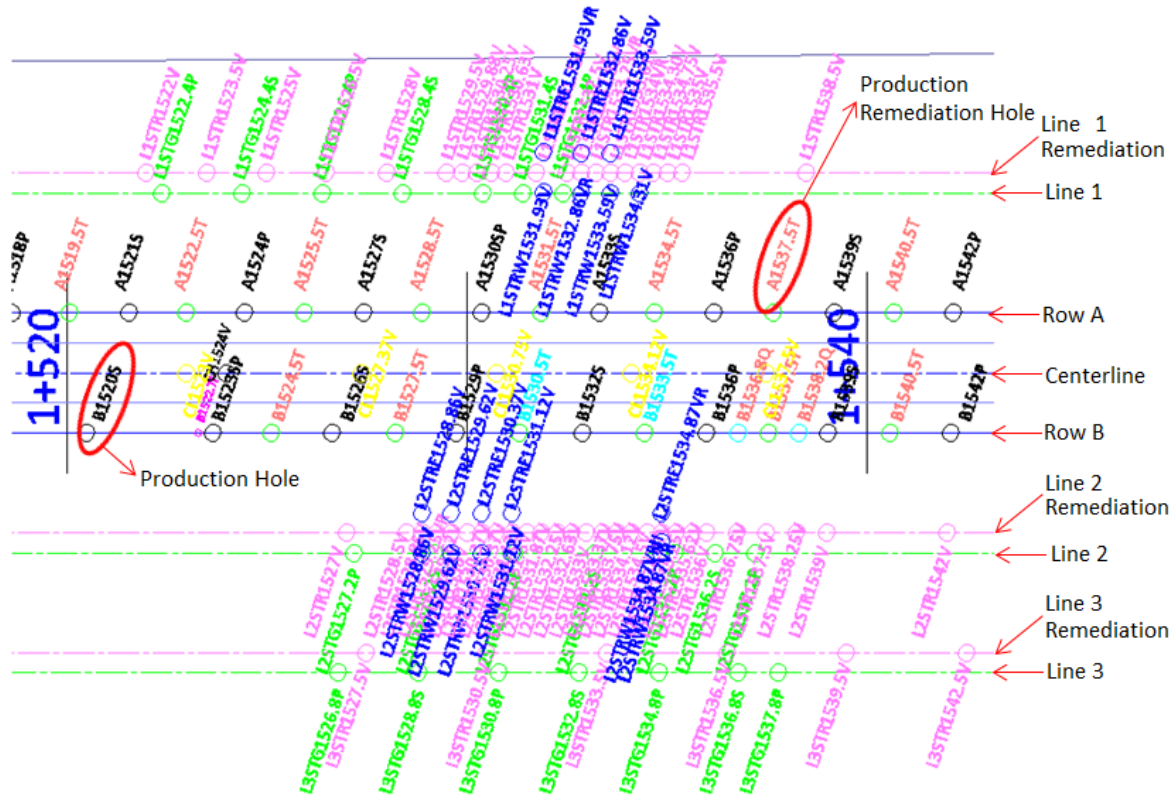


Figure 23: Location of Production, Verification and Remediation Holes

The process by which all production, verification, and remediation holes are drilled and grouted is basically the same.

Finding 5: All drawings should be at a scale of 1:100 on a 22"x34" sheet to make the drawings readable at half size (11" x 17")

The As-Built drawings produced for the Dam 1E project will be used in reports and for final engineering records. In AutoCAD, the page layout used for full size drawings is a 22"x36" sheet (ANSI D size). The scale used on this sheet size is 1:100 so that the drawings can be clearly read. For the production of reports, half-size drawings (11"x17" sheets) will be used at a scale of 1:200. This scale was found to be adequate for the readers of the report. In the case of the As-built drawings for the stitch grout areas, these scales also allowed incorporating of the grout holes plan view on the same sheet. While the As-Built drawings show the profile of the grout holes, it is beneficial for the plan view to also be shown so that the relatively location of grout holes on rows or lines can be easily appreciated. This plan view will also be presented at a scale of 1:100 on full size drawings.

Finding 6: The Deere Classification is used to color code the As-Built drawings based on grout take

For presentation purposes, as-built drawings prepared for the Dam 1E project use the Deere “Grout Absorption Intensities” Classification System (1982). This system assigns a descriptive classification of the grout take for each stage in each hole, as described in the table below. In addition, the Deere classification allocates a specific color for the amount of grout taken in each stage, as shown in Table 16 below, so that the data can be easily interpreted when viewing the drawings.

Table 16: Deere Classification System Based on Grout Take (URS Corp. Inc., 2009)

Grout Take (kg/m)	Deere Classification	Color
0 – 25	Low (L)	Magenta
26 – 50	Moderately Low (ML)	Blue
51 – 100	Moderate (M)	Green
101 – 200	Moderately High (MH)	Yellow
201 – 400	High (H)	Orange
Greater than 400	Very High (VH)	Red

This color-code system improves the presentation of the final drawings by distinctly associating a color with a grout take range so the reviewer can more easily understand the data being presented. As stated in the Background Chapter, the database containing all the drilling, grouting and Lugeon data is extremely extensive and complex. It is, therefore, necessary to present the information in the as-built drawings in a clear, graphical fashion, minimizing the need for words or tables which can clutter the drawings.

Finding 7: The extensive grouting data from thousands of holes can be more easily interpreted when sorted by Row and then by Station

There are approximately 6000 grout holes drilled and grouted to stabilize the foundation of the Dam 1E. The drilling and grouting information for all of these holes is compiled in a very large database in Microsoft Excel. When producing As-Built drawings, these data need to be organized in such a way such that similar

lines or rows are grouped together. In addition, it would also be advantageous to sort the data by station successively so that the person producing the drawings can evaluate the data in the order in which each hole was grouted. Also, if holes are organized by station, the program used to interpret the data can more easily function, reducing the program run time. Sorting the data in this fashion can assist in observing if there is grouting/drilling information missing. This sorting technique also assists in easily separating the information into smaller databases based specific criteria (e.g., stitch grouting of a specific fault zone), which can be more readily read and interpreted by the Visual Basic macro that will be shortly discussed.

Finding 8: Approach used to create As-Built drawings from an Excel database using an AutoCAD Visual Basic macro

As-built drawings of the grout holes are prepared to provide a visual representation of the dam foundation response to permeation grouting and Lugeon testing.

After drilling, grouting and ground surface elevation information is organized into smaller secondary databases, a Microsoft created Visual BASIC program can be run. This program reads this information from an excel database and plots the data in AutoCAD. Visual BASIC is a high functioning programming language which has advanced from the former DOS language called Beginners' All-purpose Symbolic Instruction Code (BASIC). This code is an English-like code that is incorporated in several Microsoft based software, such as Microsoft Excel. In past years, BASIC functioned in a text-only setting to run calculations and create user interfaces within Microsoft. The more recent Visual BASIC, as its name suggests, functions in a visual or graphical setting where an object oriented programming approach is taken. The program doesn't need solely text code to function, but can utilize pre-programmed commands to produce a simpler software package. Therefore, it can be said that Visual BASIC software comprises of numerous subprograms which can function independently, simultaneously or, linked in various orders. Because this Visual BASIC program functions using pre-programmed icons, it can now integrate the use of Microsoft external and internal software programs, such as AutoCAD and Microsoft Excel.

The As-Built drawings require the use of this Visual BASIC program or macro to interpret Microsoft Excel databases created by the drafter from the main grouting database to produce visual connotations in AutoCAD. The process to create the As-Built drawings utilizing the aforementioned software is shown in the Appendix.

Finding 9: A instruction manual would be the most effective mode of presentation for the process design

After working with the databases and software provided by the ACP and assessing the employee structure of the work place it was determined that the technician producing the As-Built drawings may have limited to vast AutoCAD experience. From personal experience working with AutoCAD, it was decided that the best method of presenting this process design was to create and procedure or instruction manual. This method of presentation would allow for detailed instructions, inclusive of pictures and tips for the technician who would be producing the drawings. Also, it was noted that a majority of the technicians speak Spanish and would benefit from having this manual in Spanish. Using an instruction manual would allow easy translating from English to Spanish.

Finding 10: The instruction manual used to produce the As-Built drawings for stitch grout areas worked very well and proved to be more efficient than manually preparing the drawings directly with AutoCAD

To facilitate the process of developing As-built drawings of stitch grout holes using the Visual Basic macro that manipulates and interprets an extensive grouting database, an instruction manual was produced and tested. Following the instructions in this manual have proven to be more efficient than the process previously being used. Upon the beginning of the 7-week internship, it was estimated that it would only be possible to complete As-Built drawings for about three of the ten stitch grout areas provided. At the end of the internship, it was possible to complete As-Built drawings for eight of the ten areas. This is approximately three times the amount of work expected. The process developed has proven to produce drawings of equal quality in less time.

Recommendations

From the findings previously discussed, it is recommended to use the instruction manual created from information gathered from this internship. This instruction manual and be found in Appendix K. The As-Built drawings created using this instruction manual can be found in Appendix L. Using this instruction manual reduced the time taken to create the drawings and resulted in an equal quality drawing. It is also recommended that this drawing be translated in Spanish, in the case that a Spanish-speaking technician is producing the drawings.

Conclusion

The production of As-Built drawings to demonstrate the response of permeation grouting is a critical, intricate and, extensive task leading to the completion of the project. There are approximately 6000 permeation grout holes used in the foundation of the Borinquen Dam 1E. The purpose of this project is to develop a process to produce As-Built drawing profiles of the Dam 1E stitch grout holes completed to date.

The procedure was presented in the form of a detail instruction manual, inclusive of detailed guidelines, demonstrative images and, tips. This instruction manual, entitled “Process Manual for Generating As-Built Drawings for Stitch and Production Grouting” aims to be extremely simple and reduce production time dramatically. In order to improve the efficiency of the production of these drawings, it was advised to follow this process since it provides insight on how to manipulate the massive database of grouting information.

This instruction manual has been test by peers with limited exposure to AutoCAD and the grouting database. These peers have agreed that this method offered easy-to-understand instructions and vivid visuals which resulted in easy production of drawings.

Embankment Construction: Compaction of Zone 1 Materials

Introduction

The purpose of this study is to conceptually assess the type of structure options available for the construction of Borinquen Dam 1E, design alternative compaction control requirements for zone 1 earthfill and evaluate the actual compaction achieved in the field against the specification and the personally designed specification.

The Panama Canal Authority (ACP) is proposing to construct four embankment dams, Borinquen Dams 1E, 2E, 1W & 2W, as part of the Pacific Access Channel (PAC) that will connect and allow navigation from the Gallard Cut section of the Panama Canal to the new Pacific Post-Panamax Locks. The Borinquen Dam 1E site is located on the west bank of the Panama Canal between the Pedro Miguel Locks and the Miraflores Locks. The length of Dam 1E is approximately 2.3 km. The Dam's function is to retain the water in the Gatun Lake from the Miraflores Lake. The Miraflores lake elevation is 16.5m and the Gatun Lake elevation is 27.13 m. The dam crest will be a minimum of 32.00 m in elevation and will be 30 m in width. The cofferdam crest elevation will be at elevation 18m (URS Holdings, Inc., 2009).

An earth and rockfill embankment dam needs to be impermeable for it to retain water. For this to occur the compaction of the earthfill needs to be done adequately following certain criteria, for it to achieve suitable low permeability. Therefore, close supervision of the compaction procedures done in the field, of laboratory testing procedures and careful evaluation of laboratory test results is vital for achieving adequately low permeabilities. Supervision of the compaction and testing procedures, analysis of test results and produced recommendations, will improve the efficiency of the compaction process of Zone 1 earthfill and help refine the overall safety and successful outcome of the construction of Borinquen Dam 1E.

Dam Structures

Both embankment dams and concrete dams were taken in consideration for the construction of Borinquen Dam 1E. Embankment Dams are built with earth fill or with earth and rock fill. These dams are usually constructed in areas where there is a suitable quantity of earth and/or rock materials available (International commission of large dams, 2014).

Dams built of concrete are called gravity dams. The entire weight of a Gravity dam is what resist the force of the stored water. Years ago, a few dams were built with masonry blocks and concrete. Today however gravity dams are built with mass concrete or roller compacted concrete (RCC) (International commission of large dams, 2014).

According to the resident engineer James Toose, the main types of Dams taken in consideration were:

- 1) Zoned earthfill embankment dam
- 2) Central core earth and rockfill embankment dam
- 3) Roller compacted concrete dam (RCC)
- 4) Mass concrete gravity dam

Zoned Earthfill Embankment

The table below displays the summary of the main features of a zoned earthfill embankment dam. The summary includes: the description of the zones, the degree of control of internal erosion and piping, the pore pressures for stability, and the suitability of the dam in relation to consequences of failure classification (Fell, 2005).

Table 17: Features of Zoned Earthfill Embankment Dam (Fell, 2005)

Zone description	Zone 1 is earthfill. Zones 1-3 are made of burrow pit run alluvial silt/sand/gravel; or weathered and low strength rock, compacted to form silt/sand/gravel.
Degree of filter control of internal erosion and piping	Moderate (poor to good). All seepage will be intercepted by Zones 1-3. Depends on particle size distribution of Zones 1-3 to act as a filter to Zone 1.

Degree of control of pore pressures for stability	Good provided Zones 1-3 is much higher permeability than Zone 1
Consequences of failure classifications to which suited – new dams	Very low to significant, depending on material particle size distributions and construction control.

The figure below displays examples of earthfill dams.

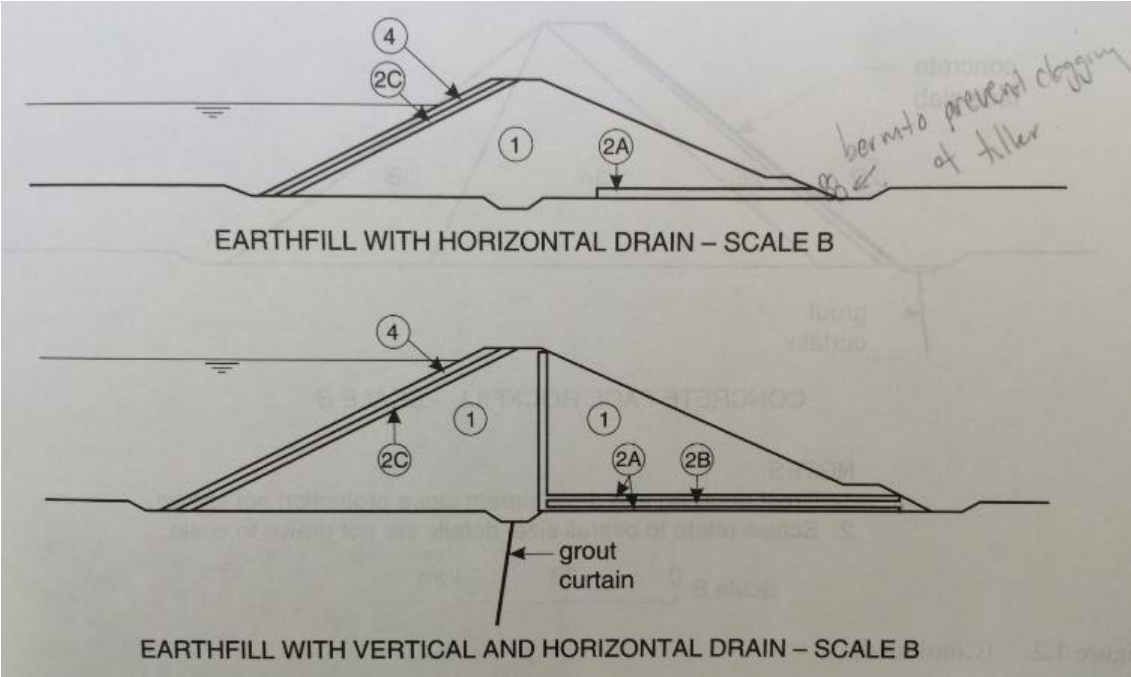


Figure 24: Examples of Earthfill Dams (Fell, 2005)

Central Core Earth and Rockfill Dam

The table below is a summary of the main features of a central core earth and rockfill embankment dam. The summary includes: the description of the zones, the degree of control of internal erosion and piping, the pore pressures for stability and the suitability of the dam in relation to consequences of failure classification (Fell, 2005).

Table 18 Features of Central Core Earth and Rockfill Dam (Fell, 2005)

Zone description	Zone 1 earthfill, Zones 2A & 2B filters, Zones 3A & 3B rockfill.
Degree of filter control of internal erosion and piping	Very good, seepage in earthfill and from cracks is intercepted by the filters and discharged in the rockfill
Degree of control of pore pressures for stability	Very good provided the rockfill is free draining.
Consequences of failure classifications to which suited – new dams	Significant to extremes. Likely to be too complicated and costly for dams less than 20m high.

The figure bellow displays examples of central core earth and rock fill dams.

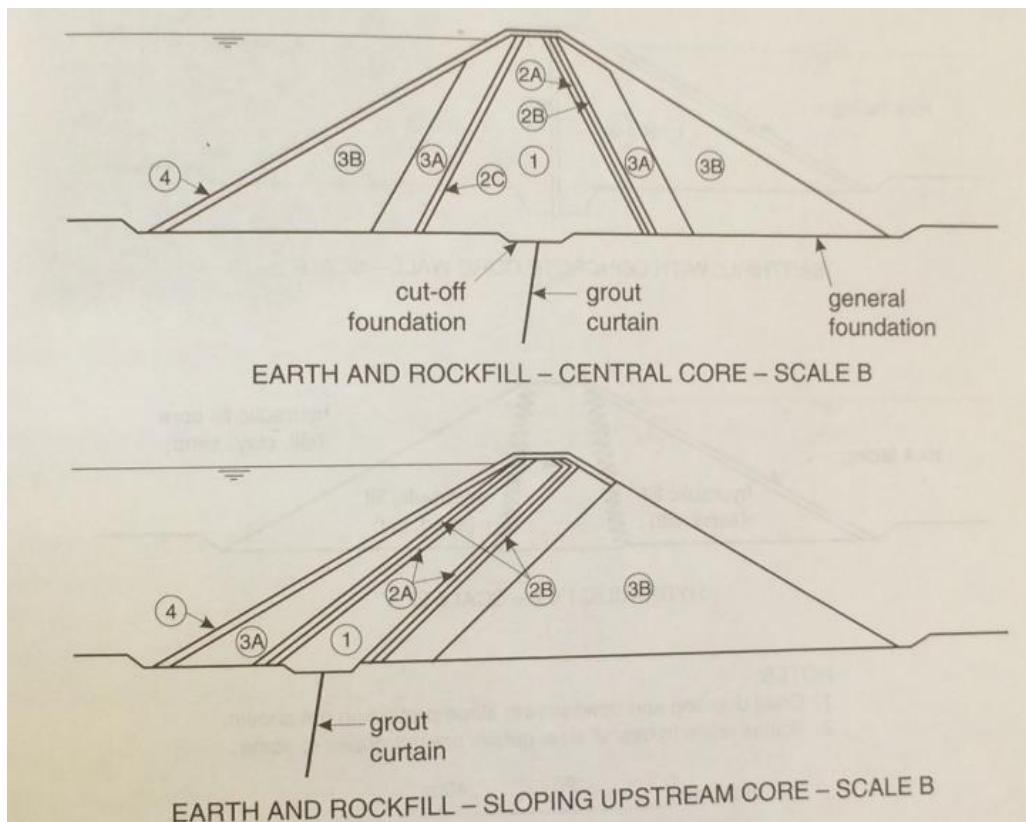


Figure 25: Example of Central Core Earth and Rockfill Dams (Fell, 2005)

Mass Concrete Gravity Dam

A mass concrete gravity dam is a structure in which its own weight resists and withstands the forces imposed on it by the water its holding. Its weight is also what prevents it from overturning. Concrete gravity dams are durable, and require very little maintenance (Hazart, 2012).

The Figure below displays a cross section of a typical straight mass concrete gravity dam structure.

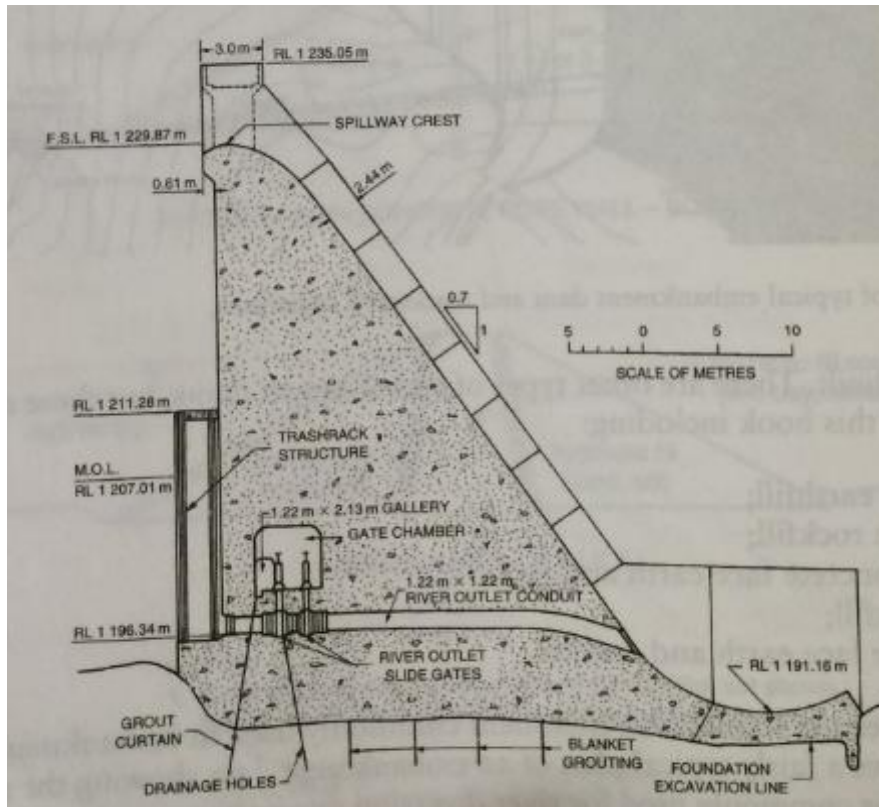



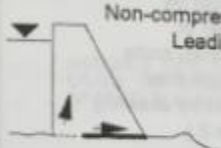
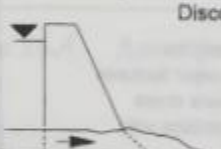
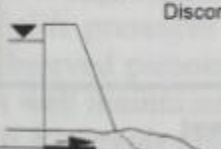
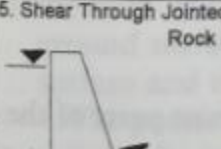
Figure 26: Cross Section of a Typical Straight Mass Concrete Gravity Dam (Fell, 2005)

Today, Concrete gravity dams feature galleries inside them, from which grout holes are drilled. This is done to reduce the permeability of the foundation. Grouting and drainage holes together reduce the uplift pressures inside the dam foundation (Fell, 2005).

Concrete gravity dams can be constructed on any dam site that has strong enough natural foundation that can withstand the large weight of the concrete dam structure. Therefore it is essential, in the design of concrete gravity dams, to have an accurate assessment and evaluation of the geological structure of the foundation. The focus should be on the analysis of its strength and compressibility (Fell, 2005).

The table below illustrates typical kinematically feasible failure modes (Fell, 2005).

Table 19: Modes of Failure for Concrete Gravity Dams (Fell, 2005)

MODE	CONDITION	COMMENTS
<p>1. Shear Along Concrete-rock Contact</p> 	<ul style="list-style-type: none"> - shear along weak discontinuity at or near concrete-rock contact. 	<ul style="list-style-type: none"> - possible if weak foundation or poor excavation and/or foundation treatment. - infrequent mode. Check for condition, particularly in old dams.
<p>2. Development of Non-compression Zone Leading to Shear</p> 	<ul style="list-style-type: none"> - development of non-compression zone (a computed crack) leading to shear along the concrete-rock contact. 	<ul style="list-style-type: none"> - can occur even in good foundations.
<p>3. Shear Along Planar Discontinuity</p> 	<ul style="list-style-type: none"> - shear along planar or near planar discontinuity in rock foundation. 	<ul style="list-style-type: none"> - frequent mode.
<p>4. Shear Along Stepped Discontinuities</p> 	<ul style="list-style-type: none"> - shear along stepped discontinuities or along surfaces stepping down between parallel discontinuities in rock foundations. 	<ul style="list-style-type: none"> - if step(s) not large then failure surface may be approximated by an inclined plane. - large steps may require higher level of analysis.
<p>5. Shear Through Jointed Rock Mass</p> 	<ul style="list-style-type: none"> - shear through highly jointed rock mass (both natural or blast induced) or through weak rock mass. 	<ul style="list-style-type: none"> - infrequent mode - may be used in moderately jointed rock masses (with an adjustment to assigned Hoek-Brown rock mass strength) - failure surface usually approximated as being planar.

Additionally, it is indispensable after the analysis of the foundation's geological structure is complete to identify potential failure modes specific to the results of the analysis (Fell, 2005).

Furthermore, for the construction of a concrete dam, suitable aggregates in sufficient quantity and at a reasonable cost must be available. Usually aggregates are processed from natural deposits of either sand

or gravel, though they can also be crushed from appropriate rocks (United States Bureau of Reclamation, 1976).

Roller Compacted Concrete

Roller compacted concrete, RCC, is concrete that has a specific type of mix design. It has much of the same ingredients as regular concrete but with different ratios. Its main feature is the partial substitution of fly ash with Portland cement. The RCC mix consists of: water, cement/fly ash, sand, coarse aggregates and several additives. It is important to note that it contains much less water than conventional concrete. The outcome of the design is a drier mix than normal concrete, which has no-slump. RCC is delivered by dump trucks, placed and spread with bulldozers and successively compacted by vibratory rollers (Abu-Khashaba, 2013).

RCC is different from conventional concrete mainly because of its consistency. In order to achieve effective consolidation, the mix must be dry enough to prevent sinking of the rollers which compact it while being wet enough to allow the binder mortar to be adequately distributed. The use of fly ash combined with the low cement content, causes less heat of hydration during curing in comparison with conventional concrete mixture (Mehta, 2005).

Roller Compacted Concrete Dam Applications

RCC is used to construct dams. Horizontal Layers of RCC are assembled lift after lift, the result of which is a concrete staircase. Once a layer is placed and hardened, another layer can be immediately placed since the previous layer is already capable of supporting the equipment needed to place the next layer (U.S. Army corps of Engineers, 2000).

RCC dams have both cost and time benefits. These benefits are primarily due to the construction techniques, which has placed them in a position to be an effective economical alternative to conventional mass concrete and embankment dams. A central advantage is the lower material costs. Other improvements are the reduced construction times which are accredited to the higher rates of concrete placement and the lower expenses attributed to the less post-cooling time. The overall process makes very high production rates possible because it consists of a continuous placement of the concrete design mix, which is not the case when dealing with regular concrete. The high production rates along with the lower quantity of material required (compared with concrete and earth embankment dams), significantly shorten the total construction time period of the project. Shortening the project time period has many additional benefits, some of which are: reduced administration costs, possible earlier project benefits and

possibility of constructing dams on sites that have specific types of seasons that limit construction. Essentially, constructing a RCC dam offers several economic and time advantages (U.S. Army corps of Engineers, 2000).

The Figure below displays a typical Rolled Compacted Concrete Dam.



Knellpoort Dam 1988, Department of Water Affairs and Forestry

Figure 27: Rolled Concrete Dam (Department of Water Affairs and Forestry, 2014)

Design Requirements for the Embankment of Borinquen Dam 1E

There are several main design requirements specific to the construction of Borinquen Dam 1E that limit the selection of alternative designs. The design requirements specific to Borinquen Dam 1E are (URS Holdings, Inc., 2009):

1. The seismic loads due to possible fault displacement in the dam foundation caused by strong earthquake shaking;
2. Ship grounding due to possible impact by Post-Panamax ships;

3. Construction practicality considering the availability of materials and the length of the rainy season;
4. Consideration of dam types constructed in areas of similar environmental and seismic conditions;
5. Construction risks mainly related to potential schedule delays and increasing costs; and
6. Possible comparative construction costs.

Based on the evaluation of alternative designs, it was concluded that the central vertical core earth and rockfill dam is the best suited type of dam structure for the geologic and seismic conditions found in the PAC area. It is also the most cost effective alternative. Therefore the central core and rockfill dam alternative was chosen for the final design (URS Holdings, Inc., 2009).

Embankment Design

There are several types of embankment dams. The designs vary in regards to the degree of in-built conservatism, which usually relates to the degree to which seepage inside the dam is controlled. The seepage in embankment dams is controlled by the filters and drains, by the free draining rockfill and by the grouting done in the foundation (Fell, 2005).

General design criteria

The following is the general design criteria used to construct Borinquen Dam 1E.

Embankment configuration

The Configuration of the embankment structure of Dam 1E is based on: “materials availability, static and seismic stability investigation, ship grounding analysis, constructability and maintenance considerations” (URS Holdings, Inc., 2009).

The dam includes a central vertical earth core, which controls the seepage through the dam, and rockfill shells. The total embankment volume is estimated to be 4,920,000 m³, of which the core would be 460,000 m³. To prevent piping of the clayey residual soil core materials and to transfer seepage away from the dam embankment, an arrangement of filters and drains has been included in the design (URS Holdings, Inc., 2009).

For the embankment to be appropriate for the dam, it was designed to have the ability to accommodate for fault displacements without loss of structural integrity. This way it can still safely discharge potential leakages without erosion or failure. This was done by including plastic core materials bordered with thick filters of materials without cohesion and large zones of cohesionless and pervious materials up and

downstream of the core. Furthermore, to prevent damage to the core from possible ship grounding, the filter and drain zones were designed to be located sufficiently away from the inboard slope within the rockshell zone (URS Holdings, Inc., 2009).

Embankment Zoning

The embankment is divided in to several sections with different functions and made with different types of materials.

Overview

The zone dimensions of the dam are based on several considerations. The first is the potential constructability, the second is the potential for high seismic shaking which results in embankment deformations, and the third is fault rupture potential. The figure below is the schematic cross section of Borinquen Dam 1E. The embankment zones can be observed (URS Holdings, Inc., 2009).

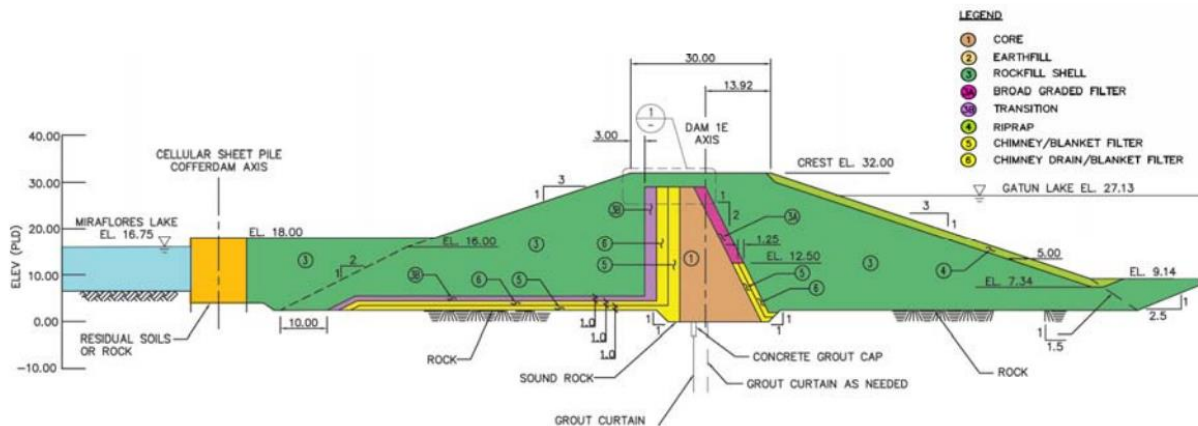


Figure 28: Cross Section of Borinquen Dam 1E (URS Holdings, Inc., 2009)

General Embankment Zone Description and Function

The table below describes the characteristics and functions of general embankment dam zones (Fell, 2005).

Table 20: Embankment Dam Zones Description and Function (Fell, 2005)

Zone	Description	Function
1	Earthfill (“core”)	Controls seepage through the dam
2A	Fine filter (or filter drain)	(a) Controls erosion of Zone 1 by seepage water, (b) Controls erosion of the dam foundation (where used as horizontal drain), (c) Controls buildup of pore pressure in downstream face when used as vertical drain
2B	Coarse filter (or filter drain)	(a) Controls erosion of Zone 2A into rockfill, (b) Discharge seepage water collected in vertical or horizontal drain
2C	(i) Upstream filter	Controls erosion of Zone 1 into rockfill upstream of dam core
	(ii) Filter under rip rap	Controls erosion of Zone 1 through rip rap
2D	Fine cushion layer	Provides uniform support for concrete face; limit leakage in the event of the concrete face cracking or joints opening
2E	Coarse cushion layer	Provides uniform layer support for concrete face. Prevents erosion of Zone 2D into rockfill in the event of leakage in the face
1-3	Earth-rockfill	Provides stability and has some ability to control erosion
3A	Rockfill	Provides stability, commonly free draining to allow discharge of seepage through and under the dam. Prevents erosion of Zone 2B into coarse rockfill
3B	Coarse rockfill	Provides stability, commonly free draining to allow discharge of seepage through and under the dam
4	Rip rap	Controls erosion of the upstream face by wave action, and may be used to control erosion of the downstream toe from backwater flows from spillways

Embankment Dam typical construction materials

The table below, obtained from Robin Fell’s “Geotechnical Engineering of Dams”, outlines typical construction materials used for different zones in embankment dams. Good dam engineering is related to the use of the materials available at the project site area. However, this rule is not followed in the search for filters that might require materials with precisely specified gradations (Fell, 2005).

Table 21: Embankment Dam Typical Construction Material (Fell, 2005)

Zone	Description	Construction material
1	Earthfill	Clay, sandy clay, clayey sand, silty sand, possibly with some gravel. Usually greater than 15% fines passing 0.075mm, preferably more.
2A	Fine filter	Sand and gravelly sand, with less than 5% fine passing 0.075mm. Fines should be non-plastic. Manufactured by crushing, washing, screening and recombining sand-gravel depositions and quarried rock
2B	Coarse filter	Gravelly sand and sandy gravel, manufactured for zone 2A. Zones 2A and 2B are required to be dense, hard durable aggregates with similar

		requirements to that specified for concrete aggregates. They are designed to restrict particle size grading limits to act as filters.
2C	Upstream filter and filter under rip rap	Sand gravel/gravelly sand, well graded (100% passing 75mm, not greater than 8% passing 0.075mm fines non plastic). Usually obtained as crusher run or gravel pit run with a minimum of washing, screening and regarding. Relaxed durability and filter design requirements compared to zones 2A and 2B
2D	Fine cushion layer	Silty sandy gravel well graded, preferably with 2%-12% passing 0.075 mm to reduce permeability. Obtained by crushing and screening rock or naturally occurring gravels or as crusher run. Larger particles up to 200 mm are allowed by some authorities, but segregation and internal instability is likely to result.
2E	Coarse cushion layer	Fine rockfill placed in 500mm layers to result in a well graded sand/gravel/cobbles mix which satisfies filter grading requirements compared to zone 2D
3A	Rockfill	Quarry run rockfill. Free possibly with oversized removed in quarry or dam. Preferably dense, strong, free draining after compaction, draining after compaction, but lesser, but lesser properties are often accepted compacted in 0.5-1m layers with maximum particle size equal to compacted layer thickness.
3B	Coarse rockfill	Quarry run rockfill. Preferably dense, strong, free draining after compaction, but lesser, but lesser properties are often accepted compacted in 1.5-2.0m layers with maximum particle size equal to compacted layer thickness.
4	Rip rap	Selected dense durable rockfill sized to prevent erosion by wave action. In earth and rockfill dams often constructed by sorting larger rocks from adjacent 3A and 3B Zones. In earthfill dams ether selected rockfill or a wider zone of quarry run rockfill may be used

Description of embankment zones used for Borinquen Dam 1E

Below are the summarized descriptions of the embankment dam zones used for Borinquen Dam 1E (URS Holdings, Inc., 2009).

- **“Zone 1- core:** The core of the dam will be constructed with clayey residual soils with at least 35% fines. The downstream slope of the core will be vertical and the upstream slope will be 0.5H: 1V” (URS Holdings, Inc., 2009).
- **“Zone – 2:** (not used in dam 1E)” (URS Holdings, Inc., 2009).
- **“Zone 3 – Rockfill:** basalt rockfill will be used to provide strong, free-draining shell zones. Basalt used for rockfill will be moderately or less weathered that is medium hard to very hard rock” (URS Holdings, Inc., 2009).
- **“Zone 3A – Broad-Graded Filter:** the Zone 3A filter is broadly graded to provide a transition between the core and inboard rockfill zone 3. Zone 3A will be processed (crushed, screened and washed) from sound basalt” (URS Holdings, Inc., 2009).
- **“Zone 3B – Transition:** Zone 3B will separate the Zone 6 chimney drain from the outboard rockfill Zone 3. Zone 3B will be processed (crushed, screened and washed) from sound basalt” (URS Holdings, Inc., 2009).
- **“Zone 4 – Riprap:** a riprap layer of sound basalt rock will be placed on the inboard slope for protection against wave erosion” (URS Holdings, Inc., 2009).
- **“Zone 5 – filter:** the zone 5 chimney filter will be located on the outboard side of the core to prevent piping of the core into zone 6 chimney drain. Zone 5 will also be placed on the foundation to prevent piping of weathered basalt and La Boca Formation into the zone 3 rockfill. Zone 5 will be placed against the lower part of the inboard side of the core zone to protect the core against piping into zone 6 during construction, when the Pac will be empty and dewatering behind the cofferdam may be discontinued. These materials will be obtained from processed (crushed, screened and washed) sound basalt” (URS Holdings, Inc., 2009).
- **“Zone 6 – Drain:** the zone 6 chimney drain will be located outboard of the zone 5 chimney filter to prevent piping of zone 5 into the zone 3B transition and on the inboard side to prevent piping of zone 5 into the zone 3 rockfill. These materials will be obtained from processed (crushed, screened and washed) sound basalt” (URS Holdings, Inc., 2009).

Materials available for Dam Construction

The table below displays the materials available for the construction of the Dam (URS Holdings, Inc., 2009).

Table 22: Materials Available For Dam Construction (URS Holdings, Inc., 2009)

Zone No. ¹	Description	Material	Source
1	Core	Clayey soils (min 35% fines)	Residual Soils
2 (not used in Dam 1E)			
3	Rockfill Shell	Rockfill	Pedro Miguel Agglomerate & Basalt
3A	Broad Graded Filter	Processed Fine Rockfill	Sound Basalt
3B	Transition	Processed Fine Rockfill	Sound Basalt
4	Riprap	Rock	Sound Basalt
5	Filter	Processed Clean Sand	Sound Basalt
6	Drain	Processed Clean Sandy Gravel	Sound Basalt

Quantities of Material Needed For The Embankment

The table below summarizes the volumes of excavated material needed for the construction of the various zones of the Borinquen Dam 1E (URS Holdings, Inc., 2009).

Table 23: Summary of volumes of PAC-4 Excavated Material and recommended Embankment use (URS Holdings, Inc., 2009)

Excavated Material	Use in Embankment	Residual Soil Borrow Areas		PAC-4 Contract	
		In-Situ Volume (m ³)	Volume Available for Embankments (m ³)	In-Situ Volume (m ³)	Volume Available for Dam 1E (m ³)
Fill	Dispose	-	-	4,920,000	-
Residual Soil (PAC)	Core	-	-	1,070,000	960,000 ¹
Residual Soil (Dam)	Dispose	-	-	1,320,000	-
Zone A	Core	335,000	285,000 ⁴	-	-
Zone B	Core	730,000	695,000 ³	-	-
Zone C ²	Core	660,000	630,000 ³	-	-
Zone D	Core	180,000	155,000 ⁴	-	-
Sound Basalt	Rockfill/Riprap/Transition/Filter/Drain	-	-	3,540,000	4,600,000 ⁵
Weathered Basalt	Dispose	-	-	550,000	-
Sound Agglomerate	Rockfill	-	-	10,710,000	13,950,000 ⁵
Weathered Agglomerate	Dispose	-	-	2,030,000	-
La Boca Formation	Dispose	-	-	1,540,000	1,940,000 ⁶
Cucaracha Formation	Dispose	-	-	670,000	-
Total		1,905,000	1,765,000	26,350,000	21,450,000

Notes:

Design of Clay Core

The following are the specific construction constraints relative to Borinquen Dam 1E along with the specific criteria design requirements for the construction of the clay core of the dam.

Specific project constrains regarding the clay core

Investigations to identify additional burrow areas around the Pacific Access Channel (PAC) were performed because of concerns that the available residual soils used for core material would not be sufficient. Five potential burrow areas were locate and investigations were performed by excavating test pits. The types of materials found for construction of the core were only residual soils formed through the weathering of the underlying bedrock. Consequentially, the dam will be made out of clayey residual soils, which have been successfully used for the construction of core zones in other embankment dams around the world (URS Holdings, Inc., 2008).

All of the estimated volume of residual soil excavated from the PAC excavations and the other identified burrow areas may not be suitable to use for construction of the core for various reasons. One concern is that potential material losses during clearing and stripping operations may occur. Losses may happen due to the failure of identifying the top of the residual soil during excavation, in areas in which various types of material cover the residual soil. Burrow sites may contain oversized materials that need to be removed. Furthermore, the excavated residual soils might not meet the specified gradation requirement for the core of the dam (URS Holdings, Inc., 2008).

There are other issues regarding the in-situ moisture contents of the residual soils at the burrow areas. The values of the moisture contents might be near or above the optimum moisture content range, due to the high precipitation conditions. As a consequence, some core materials could be placed significantly above the optimum moisture content. This is a major issue for compaction, since the residual soils are medium to highly plasticity silts or clays that are difficult to compact effectively when wet. Hence, the excavated residual soils must be adequately processed before getting placed in the embankment. Occasionally parts of the burrow areas were found with moisture contents lower than the optimum. In this case the residual soil must be processed until it reaches the optimum moisture content or greater. This process should be done in the burrow area before the material is brought and placed in the embankment (URS Holdings, Inc., 2008).

The compaction of test fills, made of the residual soil for the core zone, was performed to evaluate its overall workability. The performance of different types of compaction equipment was noted, with an evaluation of different methods for moisture conditioning. Different lift thicknesses were tried, and the number of passes required to reach adequate compaction were listed (URS Holdings, Inc., 2008). Several

challenges were faced during the construction of the test fills. The most challenging aspects were the control of the material's moisture content while it was being placed and the control over the material's gradation requirements. The gradation of the residual soils is mostly controlled by the amount of weathering they were subjected to, which is significantly variable in the different burrow areas. The material will need to be processed in order to acquire suitable conditions for the core. Controlling the moisture content of the soil during placement is also a major challenge because of the high precipitation circumstances at the site. A high water content affects a material's workability and consequentially restricts the ability to compact it. The result could be a soil having a lower strength, a lower dry density, a higher compressibility, and a higher permeability (URS Holdings, Inc., 2009). All of these potential outcomes of the core's characteristics could be detrimental to the overall outcome of the project. These circumstances will definitely be a significant challenge during dam construction. Therefore, close control of construction operations must be done to prevent the moisture content from becoming too high to adequately compact the core material while working in wet weather conditions. Maintaining an adequate water content in periods of high precipitation is and will always be a severe problem. Impervious materials should never be placed on embankments during rain, however, construction operations may continue successfully when rain stops. Additionally, the moisture content of material just placed on the embankments can be reduced by disking, after the rain has stopped, before rolling can commence. (URS Holdings, Inc. "draft", 2008)

Criteria requirements for the Zone 1 Core

Following are the criteria requirements, with their respective details, for construction of Zone 1 core of the embankment of Borinquen Dam 1E.

Requirements for Core Construction

The core of the dam must be (1) impermeable, (2) have sufficient strength, (3) be sufficiently ductile and flexible and (4) have an adequately low compressibility (Toose, 2014).

Overview

According to the department of the U.S. Army Corps of Engineers: "The density, permeability, compressibility, and strength of impervious and semi-impervious fill materials are dependent upon water content at the time of compaction" (U.S Army Corps of Engineers, 2004). As a consequence, the embankment design is notably influenced by the natural water content found in the materials at the

burrow sites and the processes done to dry or moisturize the material before it is brought to the fill (U.S Army Corps of Engineers, 2004).

The embankment and foundation materials of dams that are constructed on weak, compressible foundations should have similar stress-strain characteristics. If the stress-strain characteristics of the embankment and the foundation of an embankment dam are significantly diverse, the outcome of a possible failure are extremely high. For prevention, the embankment needs to be designed in correlation with the foundation, so that neither will be strained beyond their respective peak strengths. By following these specific design criteria, the stage where progressive failure starts to happen is not going to be reached, preventing failure. Additionally, embankments should be constructed with more plastic materials, making them capable to adjust efficiently to settlements if compaction wet of the optimum water content is performed (U.S army Corps of Engineers, 2004).

Requirement Details

Impermeable

For a soil to be impermeable it needs to have an adequate permeability coefficient “k” (hydraulic gradient), which is not a fundamental property of the soil. It depends on several factors: particle size distribution, particle shape and texture, mineralogical composition, void ratio, degree of saturation, soil fabric, nature of fluid and type of flow and temperature. In embankment dam engineering these factors have varying degree of influence, the main factors are the gradation, the type of material and the degree of compaction (Fell, 2005).

The figure below shows, in general terms, the ranges of permeability that can be encountered, the drainage characteristics, the soil types, test methods and permeability classifications.

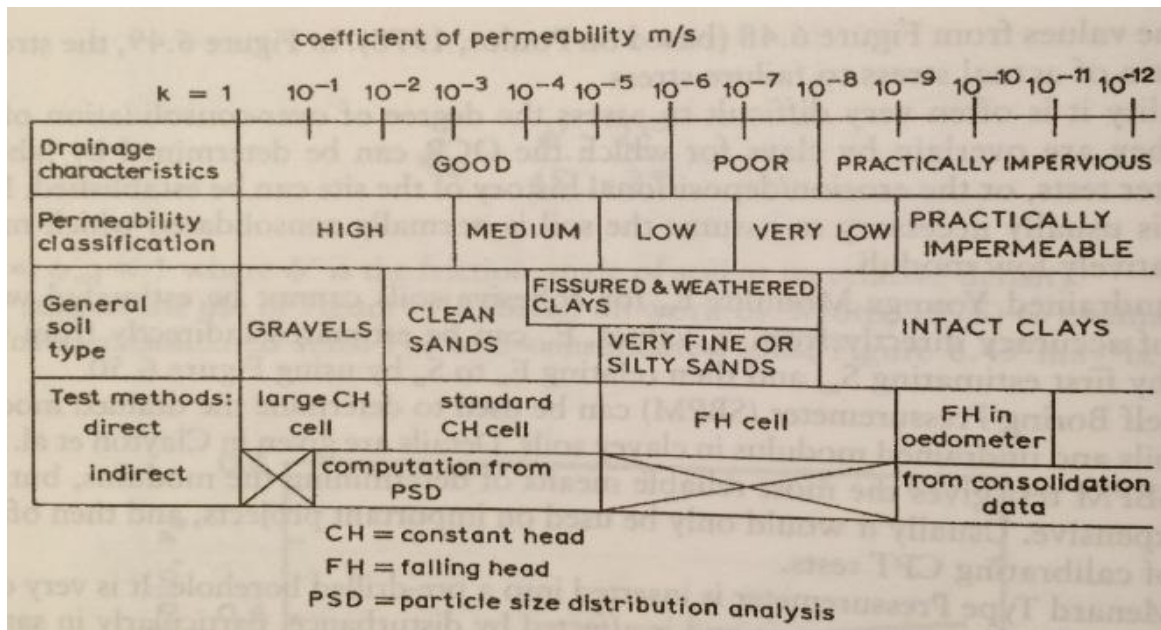


Figure 29: Permeability and Laboratory Testing Method for the Main Soil Types (Fell, 2005)

The core of Dam 1E must have a low permeability, which should range around 10^{-6} m/s to 10^{-7} m/s. This can be achieved if the material chosen for the core of the dam has a significant amount of fines and is placed and compacted adequately. Compaction must be done at a specific water content in order to achieve an adequate percent compaction compared to standard maximum dry density (URS Holdings, Inc., 2009).

Description of the several factors affecting permeability of soil

The permeability of a soil is dependent on its *particle size distribution*. Fine grained clay soils have much lower permeability compared to the permeability of coarser soils, like sands and gravels. Studies have concluded that the finer particles in soils largely determine their permeability (Fell, 2005).

The *shape and texture of particles* in a certain soil have a smaller amount of influence on their permeability. Elongated particles have the tendency to have a lower permeability compared to rounded ones. Whereas rougher textured particles have a lower permeability respect to smooth ones (Fell, 2005).

The *Mineralogical composition* of sand and gravels has a treasurable effect on their permeability. On clays it has a more pronounced affect. For example, Montmorillonite clays are finer grained, which means that

they have a much greater tendency to absorb water, leading to a lower permeability compared to other clays, like Kaolin clay (Fell, 2005).

A soil's *void ratio* has an important effect on its permeability. For example, Cohesive soils, compacted to a high density ratio will have lower permeability than those compacted to a low density ratio (Fell, 2005).

When a soil becomes partially *saturated*, its permeability is significantly reduced. This happens because of the reduction of the total area of pores filled with water. The pores filled with water are mainly the finer pores, since water is easily removed from the larger pores because of their low suction potential. The remaining finer water filled pores have a naturally lower permeability (Fell, 2005).

The fabric of soil can affect its permeability. This is particularly important for soils where stratification or layering of different soil types could potentially lead to different permeability along strata of specific soil lifts. Furthermore, the permeability of recomputed soil is also affected by the soil fabric. The water content at which a clay is compacted affects its permeability. In particular a cohesive soil compacted dry of optimum moisture content results in a less oriented structure, which leads to a relatively high permeability (Fell, 2005).

The permeability of a soil is also dependent on the intrinsic properties of the *permeating fluid*. For dam projects where water is the permeating fluid, the small variations in viscosity are not significant, compared to other variables (Fell, 2005).

The *type of flow* of the permeating fluid also affects the permeability of the soil. Darcy's Law $q = kiA$ is used to describe the flow of a fluid through a porous medium. The basic assumption in the calculations are that the soil through which the liquid flows, is saturated and that the flow is laminar. This is generally the case for flows through and beneath a dam (Fell, 2005).

Strength

The minimum shear strength requirement for the core of Dam 1E is based on its static stability requirements, on the event of a possible seismic load and on construction loads. To achieve the shear strength requirement the soil must be adequately compacted, since with compaction a soil's shear strength increases. It must also be compacted with a moisture content that is above optimum, so that the material is workable enough to be compacted adequately to achieve the required shear strength.

However, the water content must not be too high above optimum or the material will be more difficult to compact, resulting in lower strength (Toose, 2014).

Ductile / Flexible

The soil must be adequately ductile and flexible. This is required so that the soil can be placed and compacted properly. It is also required because the core can deflect and deform a certain amount but still remain functional and intact. In the event of a displacement caused by an earthquake, which is likely to occur since the Dam is being built on several faults, the core must remain intact. To achieve this, a specific type of material must be used and adequately compacted wet of optimum moisture content. A suitable Plasticity Index must also be chosen in order to have a sufficiently plastic soil. This will allow it to deform along with the displacement caused by a seismic load. A ductile and flexible material will also reduce possible damages due to settlements (Toose, 2014).

Compressibility

Compressibility is the property of a substance capable of being reduced in volume by application of pressure (Hilf, 1991). The core of the material must have a low compressibility to avoid large future settlement and differential settlement that could be extremely detrimental to the dam. The compaction must be done above moisture content for the material to be compacted adequately, but not too high above optimum or the material will end up having a high compressibility (Toose, 2014).

Specifications Required for Central Core Earth and Rockfill Dam Construction

The requirements for construction of Zone 1 clay core state that the core must be impermeable, have sufficient strength, be sufficiently ductile and flexible, and have an adequately low compressibility.

According to Fell's "Geotechnical Engineering of Dams", to abide to these requirements the following aspects of the material placed in Zone 1 need to be specified:

- The source and Soil Classification.
- The maximum particle size and particle size distribution
- The Atterberg limits
- The shear strength
- The water content placement range
- The density ratio (percent compaction)

Design specifications of clay core

Following are all the detailed specifications regarding the Zone 1 core of Borinquen Dam 1E

Literature review on the compaction of soils and testing methods

Research on soil compaction characteristics has been performed in order to better understand the soil compaction requirements specific to the project.

Soil compaction

Soil has been used throughout history as a construction material. Its widespread availability and relatively low price make it a valuable option for use in construction of foundations and embankments of dams. Compacting soil changes its physical properties. Compaction is the process in which a soil sample (made of solid soil particles, air and water) is reduced in volume due to the application of loads. The process involves the expulsion of air but not a significant change in the amount of water, thus the moisture content of the soil (ratio of weight of water to weight of dry soil) remains the same. Since the amount of air is reduced and the water content remains unchanged, the degree of saturation increases. However, in most soils, the expulsion of all of the air is not possible to achieve by the means of compaction, hence 100% degree of saturation is not reached (Hilf, 1991).

When soil is used as a construction material its significant engineering properties are shear strength, compressibility, permeability and flexibility. With compaction shear strength increases, compressibility and permeability decrease. When considering soils for their compaction characteristics, they can be classified and divided in to two separate groups, cohesive soils and cohesionless soils. Cohesive soils are those that contain a sufficient quantities of silt or clay to make them practically impermeable after they are properly compacted. Such soils are composed of a combination of varieties of clays, silts, silty sands, silty gravels, clayey sands and clayey gravels. These soils are included in the USCS classification groups CH, CL, MH, ML, SC, SM, GC, GM. Cohesionless soils, on the other hand, are the relatively clean sands and gravels that remain penetrable even if they are well compacted. Soil groups SW, SP, GW, GP and boundary groups represent these soils (Hilf, 1991).

Testing methods

The control of the degree of compaction of embankments in earth dams is exceptionally important, due to the fact that the shear strength of a compacted cohesive soil depends on the density and water content

at the time of shear. Therefore, several quality control tests are performed on the compacted soil to assess the percent compaction obtained and other soil characteristics, to determine whether the design specifications are being met or not. Testing is vital to ensure that the safety and successful outcome of the project will be reached (US army corps of Engineers, 1995).

Tests are performed following ASTM standards. The tests performed in this project include:

- ASTM D 2216, Standard test method for laboratory determination of Water (Moisture) Content of Soil and Rock by Mass.
“A test specimen is dried in an oven at a temperature of 110 ± 5 °C to a constant mass. The loss of mass due to drying is considered to be water. The water content is calculated using the mass of water and the mass of the dry specimen” (ASTM international, 2014).
- ASTM D 2573, Standard test method for field vane shear test in cohesive soil
“The vane shear test consists of placing a four-bladed vane in the undistributed soil and rotating it from the surface to determine the torque required to shear a cylindrical surface with the vane. This torque, or moment, is then converted to the unit shearing resistance of the failure surface by limit equilibrium analysis. Friction of the vane rod and instrument are either minimized during readings by special casings or housing, or else accounted for, and subtracted from the total torque to determine the torque applied to the vane” (ASTM international, 2014).
- ASTM D 4318 standard test method for Liquid Limit, Plastic Limit & Plasticity Index of soils (ASTM international, 2014).
- ASTM C 136, standard test method for sieve analysis of fine and coarse aggregates
“A Sample of Dry aggregate of known mass is separated through a series of sieves of progressively smaller openings for determination of particle size distribution” (ASTM international, 2014).
- ASTM D 698, standard test method for laboratory compaction characteristics of soil using standard effort.
“A Soil at a selected water content is placed in three layers into a mold of given dimensions, with each layer compacted by 25 or 56 blows of 5.5 lbf rammer dropped from a distance of 12 in, subjecting the soil to a total compactive effort of about 12,400 ft-lbf/ft³. The resulting dry unit weight is determined. The procedure is repeated for a sufficient number of water contents to establish a relationship between the dry unit weight and the water content for the soil. This

data, when plotted, represents a curvilinear relationship known as the compaction curve. The values of optimum water content and standard maximum dry unit weight are determined from the compaction curve” (ASTM international, 2014).

- ASTM D1156, Standard test method for density and unit weight of soil in place by sand cone method.

“A test hole is hand excavated in the soil to be tested and all the material from the hole is saved in a container. The hole is filled with free flowing sand of a known density, and the volume is determined. The in place wet density of the soil is determined by dividing the wet mass of the removed material by the volume of the hole. The water content of the material from the hole is determined and the dry mass of the material and the in-place dry density are calculated using the wet mass of the soil, the water content, and the volume of the hole” (ASTM international, 2014).

Specifications for Zone 1 for Borinquen Dam 1E

Below are the required material characteristics specific for the Zone 1 core earthfill.

Material type, plasticity index and gradation requirements

Materials for Zone 1 will be residual soils and may not contain organic material. Zone 1 materials must have a plasticity index (PI) of at least 10 as determined by ASTM D 4318. Materials need to meet the gradation requirements organized in the table below (ACP, 2009).

Table 24: Zone 1 Material Gradation Requirements (Acp, 2009)

U.S Standard Sieve Size	Percent Passing (by weight)
6" (150 mm)	100
¾" (19.0 mm)	>70
No. 200 (0.075 mm)	>35

Water content and undrained shear strength requirement

Based on the test fill results, it was concluded that the compaction water content of Zones 1 needs to be between 2% and 12 % above the optimum moisture content to achieve adequate compaction (ACP, 2009).

The adequate shear strength assumed for the design of the core is 60 kPa. The Zone 1 material must be placed and compacted to achieve a minimum untrained shear strength of 75 kPa, measured with a field vane following ASTM D 2573 standard. Absolutely no strength test results may be allowed lower than 60 kPa, at least 9 out of 10 consecutive strength test results must be higher than 75 kPa, and the running average of any 5 consecutive tests must be higher than 75 kPa (ACP, 2009).

Zone 1 Summarized Key Specification Requirements

1. Material to be residual soil and not contain any organic material
2. PI > 10
3. 100% passing 6" (150mm) sieve, ≥ 70% passing ¾" (19mm) sieve, and > 35% passing No.200 (0.075 mm) sieve
4. Minimum undrained shear strength = 75 kPa
5. Compaction water content range between +2% and +12% of OMC

Processing of Burrow Materials

Some burrow sites may contain oversize material (≥6") which will need to be removed. Oversize material must be removed at the burrow source before it is brought to the fill area of the dam. The removal of oversized material at the dam fill site is not allowed (URS Holdings, Inc., 2009).

Materials from the burrow sites may also potentially not meet the minimum fines requirement of 35%. In order to meet this specification the materials are going to require selective excavation and blending, which should be done before it is hauled to the dam site. Additionally, measures will need to be taken to protect the burrow areas from excessive water infiltration during the rainy season. Since burrow materials that are rained on excessively need to be discarded at designated disposal areas, there will be a reduction in the amount of material available to be used in the embankment (URS Holdings, Inc., 2009).

Zone 1 lift placement and compaction procedures

A lift is an individual layer of Embankment material placed at the specified thickness and compacted in the specified manner before placing the next layer (Lift) (ACP, 2009).

From the analysis of the test fills, it has been deduced that adequate characteristics of the core are obtained spreading Zone 1 materials in level, continuous lifts. The lifts should be placed with a slope of

about 5% from the horizontal towards the inboard toe, to help with the drainage process and to allow rain to runoff when the embankment is rained on. The materials need to be also placed in lifts that do not exceed 22.5 cm in thickness. After each lift of Zone 1 is leveled and blended adequately, compaction can begin and needs to be done with a minimum of 8 passes of the compaction equipment. According to the outcomes of the Zone 1 clay core test fills, it was noted that it is important that all compaction passes of any portion of the lift are made with the same type of compaction equipment (ACP, 2009). Additionally, the tamping-foot compactor was found to lead toward a better bonding between lifts compared to the dozer's compaction performance, meaning it is recommended that the former is used (URS Holdings, Inc., 2009).

If any potential fill materials are found to have a moisture content outside of the acceptable specified range, they need to be conditioned before the compaction process can begin or discarded at designated disposal sites. However, no water should be added to materials in the fill with moisture contents that are in the acceptable range before compaction. Furthermore, in order to ensure an evenly distributed moisture content within the placed lift and overall fill, the materials in each lift should be thoroughly mixed with a disk before compaction can begin (URS Holdings, Inc., 2009).

During precipitations, it will be necessary to protect the core material of the embankment in the stockpiles and in the burrow areas from increasing their moisture content. Sealing the surface of the compacted fill with a roller so that the water does not seep through but runs off due to the 5% slope should be done. Trimming the material on the fill surface that has been rained on will be required before compaction or the placement of a next fill. Materials exposed to more than 3 mm of rainfall will need to be removed from the fill, stockpiles, and burrow pits. The depth of trimming removal of the fill will be at least 50mm. However, if it is observed that the material at less than 50mm in depth is not affected by rain fall it can be acceptable to remove less than 50mm (ACP, 2009). The removed material will needs to be disposed of in designated disposal areas, or placed in areas away from the embankment where it can dry (URS Holdings, Inc., 2009).

Quality control

Tests to determine whether or not the compaction of Zone 1 meets specifications will be done in the lift below the last compacted lift. The table below summarizes the type of test, the ASTM standard and the frequency of testing to be performed on Zone 1 material of the embankment (ACP, 2009).

Table 25: Zone 1 Embankment Materials - Quality Control Laboratory and Field Testing (ACP, 2009)

Test Type	Testing Frequency	ASTM Test Method
Undrained Shear strength	1 test per lift per 100m horizontally and 1 test per lift per 200m horizontally thereafter. Three individual field vane tests shall be performed at each test location, the three results shall be averaged and compared to the specified undrained shear strength.	ASTM D 573
Water content	At each undrained shear strength test	D2216
Gradation	Every tenth undrained shear strength test location	C 136
Atterberg limits	Every tenth undrained shear strength test location	D 4318
Optimum moisture content	Every tenth undrained shear strength test location	D 698

The ASTM D 1156 standard sand cone test, which is required to be done in the test fills, is occasionally performed on the compacted Zone 1 core as an additional check for quality control. It is done to not rely solely on the vane shear test result. The density and water content of the material obtained with the sand cone test is compared to the laboratory generated compaction proctor curve ASTM D 698. The curve shows the maximum dry density and the optimum moisture content. The percent compaction in the soil is calculated and reflections on whether or not the specifications are being met can be done (ACP, 2009).

Methodology

To complete this project a series of three objectives was established. This chapter explains in detail each objective and the methods used to accomplish them.

Objectives

1. Select most adequate type of structure to be used for construction of Borinquen Dam 1E;
2. Develop compaction requirements and testing specifications for Zone 1 clay core based on the results of Zone 1 test fill number 7; and
3. Evaluate the actual compaction achieved in the field, based on analysis of test results, against project specifications and personally designed specifications.

Objective 1: Select most adequate type of structure to be used for construction of Borinquen Dam 1E

The types of dam structures that were analyze for the possible construction of Borinquen Dam 1E were:

- 1) Zoned earthfill embankment dam;
- 2) Central core earth and rockfill embankment dam;
- 3) Mass concrete Gravity dam; and
- 4) Roller Compacted Concrete Dam (RCC)

Procedure performed to choose the most suitable type of dam for construction of Borinquen Dam 1E

Following are procedures performed in order to analyze the available dam structure options and select the best one.

Research on types of dams

Research was performed, and is located in the background chapter, on each possible type of dam candidate for construction of Borinquen Dam 1E. The research consisted of identifying the main functionalities of the dams and their major advantages and disadvantage. The focus of the research was on: the types, quantities and cost of materials needed for construction, the overall time and cost of construction and the necessary circumstances needed for each type of dam to be constructed. The main necessary circumstances being: foundation strength, permeability, compressibility of soil used for embankment, seismic performance and climate condition constraints (URS Holdings, Inc., 2009).

Summary of the main critical design criteria requirements specific to the site and construction of Borinquen Dam 1E

The main constraints specific to the project were summarized from the background chapter and displayed and organized in lists and tables, which can be found in appendix A, to better analyze the dam options available and therefore choose the most suitable one.

Comparison between the main characteristics of researched dams

To make a final selection on the type of dam to be used, there was first a comparison between embankment and concrete dams. A table contrasting the main features of the two types of embankment dams was generated, and can be found in Appendix A. The table helped in the analysis of the dams main advantages and disadvantages used to make select the better option. The main features of the mass concrete gravity dam and RCC dam were enumerated as well, in Appendix A, to help with their analysis and selection.

Comparison with design requirements specific to the construction of Borinquen Dam 1E

Following analysis, one type of embankment dam was chosen because it was regarded as being more adequate concerning the design constraints of Borinquen Dam 1E. The same was done for the concrete dam types. The two remaining dam options, one embankment dam and one concrete dam, were analyzed between each other based on their main features and necessities. They were also at the same time contrasted with the project's design criteria requirements, to choose which dam structure was the best possible option. Specific focus was directed toward: the type and quantity of material necessary for construction and toward the amount of material available at the site for the type of dam; toward the foundations strength characteristics and the ones necessary for the two dam options; and toward the seismic considerations of both the site and of the dam options.

Selection of type of Dam structure

A certain type of dam structure was chosen for the project following the analysis, since it was found to be the best possible choice for Borinquen Dam 1E based on the design requirements of strength, seismic conditions and availability and cost of materials.

Objective 2: Development of compaction requirements and testing specifications for Zone 1 clay core, based on the results of Zone 1 test fill number 7.

Below are the detailed procedures followed to analyze the results of Zone 1 test fill number 7 and successively produce the compaction requirements and testing specifications for the Zone 1 clay core.

The scope of Zone 1 test fill number 7

The objectives of Zone 1 test fill number 7 were to acquire information on the engineering properties of the available materials excavated from the burrow areas. With the gained information, the development of the various construction specifications was possible. The soil was taken from more than one of the identified burrow areas (URS Holdings, Inc., 2009).

Outline of methodological procedure followed to analyze test fill results and come up with adequate specifications

- Received laboratory test results for test fill number 7 from the resident engineer James Toose.
 - The tests performed on test fill were: sieve analysis tests, Atterberg limit tests, specific gravity tests, vane shear strength tests, moisture content tests, compaction tests and sand cone density tests. All the tests were done following the ASTM standards.
- Results were organized in table form to better visualize, compare and understand their meaning, they can be found in Appendix B.
- Test results were summarized and separated based on who performed them: the Contractor or the ACP.
- Research was performed regarding the specifications and quality control of earthfill for construction of embankment dams, from Robin Fell's "Geotechnical Engineering of Dams".
- The analysis of the results was based on the design requirements for Borinquen Dam core construction previously outlined in the background.
- As described in the background the requirements for the clay core are that it must be impermeable, have sufficient strength, be sufficiently ductile, and flexible and have an adequate compressibility. With this knowledge at hand, and from the research regarding core specifications, it was concluded that several specific characteristics and properties of the material needed to be specified to meet the requirements. These being: the source and Soil Classification of the earthfill material, the maximum particle size and particle size distribution, the Atterberg limits, shear strength, water content variation, and density ratio (Fell, 2005).

- Based on the research and analysis of the findings of test fill number 7, the source and soil classifications to be used in the core material were specified.
- Based on the performed research, the knowledge about the project's constraints and its importance, particle size distributions requirements were generated.
- All of the gradations test results were plotted on a single graph with the generated gradation specification to determine whether the materials excavated and used in the test fill meet the requirement.
- Based on the performed research, the knowledge about the project's constraints and its importance, a specification for the plasticity index of the soil was produced.
- Additionally, the requirement of a specific liquid limit was discussed, based on performed research and project constraints.
- A plot of the plasticity index against the liquid limit was generated to see whether a certain trend existed. It also checked whether the material used in the test fill could comply with the generated specification, meaning it could be used for the construction of the core of the dam.
- Analysis of the required strength for the core of the dam to be able to have static stability, withstand seismic loads, and endure construction loads requires advanced knowledge of earth dam behaviors related to earthquakes. Therefore, I was told by the resident engineer James Toose to keep the specification at 75 kPa (Toose, 2014).
- Since the table in the results shows that four samples failed the shear strength requirement, a new table was constructed. It displayed the shear strength test results that failed next to their respective water content ranges, corresponding optimum moisture content ranges, and percent compaction achieved. In the same table, the values of ranges of water content, optimum moisture content, and compaction of the samples that passed the shear strength requirement were inserted. This was done to better visualize the soils behavioral properties and to try and find possible reasons for the shear strength failure.
- Research was performed on typical specification requirements of adequate water content placement ranges and density ratios.
- Several graphs were generated to understand correlations between different properties of the material to come up with adequate specifications.
- The generated plots were: shear strength vs water content, shear strength vs sand cone water content, shear strength vs percent compaction, and maximum dry density vs optimum moisture content.

- Tables were made to better visualize and analyzed properties of the material. The tables produced were:
 - Ranges and averages of the optimum moisture contents and the maximum dry densities achieved in the laboratory. For both the ACP and Contractor tests.
 - Average moisture content test results and average sand cone moisture content test results. For both the ACP and Contractor tests.
 - ACP and Contractor undrained shear strengths, moisture contents, optimum moisture contents, percent compactions and differences of the various moisture contents from the optimum.
- Material engineering property characteristics, anomalies, and trends between material properties were identified.
- Analysis of the material properties of the soil samples that failed the shear strength test were contrasted and compared with the other engineering properties of the samples that passed the shear strength test.
- Specifications were generated based on the outcomes of the analysis along with the consideration of the project constraints (available material and climatic conditions), the performed research on testing specifications, the importance of the project's safety, and whether specifying certain requirements is feasible or not.
- Developed specifications were summarized and enumerated and can be found in appendix B.
- Specifications regarding the type of quality control tests and their frequency, required for Dam 1E core, were developed based on the outcomes of the analysis of test fill number 7, on research regarding type and frequency of embankment dam testing specifications and on advice from the resident engineer James Toose
- A table which displays the developed quality control testing types and frequencies was generated.

Objective 3: Evaluation of actual compaction achieved in the field and comparison to project specifications and personally developed specifications

Below is the procedure followed to supervise the compaction procedure of Zone 1 and to supervise the tests being done in the field and in the laboratory for quality control. The tests performed on the compacted clay core were enumerated and the results were organized in tables in appendix C. The Zone 1 key specifications were also summarized in appendix C.

Observation of compaction of lifts in the field

Alex Manternach was brought to the field by inspector Sebastien Bonnart, who explained the whole process involved in the compaction of the Zone 1 clay core. Together we observed and supervised the trimming of the soil that had been rained on. We then oversaw the placing, disking, leveling, compacting and sealing of the core material. We made sure that the process was being done correctly according to the specifications. I was also brought to observe the stock pile of zone 1 material where it is dumped after being excavated from the burrow areas. There are specifications to follow regarding the stockpiles as well, Sebastien and I made sure that they were being followed correctly.

Supervision of field testing

After a lift of Zone 1 core had been placed, disked, leveled, compacted and sealed properly, Sebastien and I supervised the quality control and quality assurance testing being done by the Contractor and by the ACP. We supervised the performance of the vane shear tests and the sand cone tests done on the Zone 1 compacted lift. We also supervised the various soil samples being taken for the tests that need to be performed in the laboratory. We made sure everything was being done according to the specifications and according to the ASTM standards.

Supervision of laboratory testing

Sebastien Bonnart brought me to the laboratory where we supervised the tests being done to see whether the ASTM standard procedures were being followed correctly. The test procedures that Sebastien Bonnart and I supervised, in the both the contractors' and ACP's laboratories, were: the water content determination, the liquid and plastic limit determination, the sieve analysis, the soil compaction procedure (to generate the proctor curve) and the procedures followed to calculate the density of the soil by sand cone method.

Procedure regarding how to analyze and the results of the field and laboratory tests.

The test results were received and the values were organized in tables. Project specifications and personally generated specifications were also inserted in the tables to help determine whether the project specifications were being met.

The gradation and Atterberg limit test results were analyzed to determine whether the specifications were being met. The reported value of the shear vane strength was compared to the specified strength to determine if it was adequate. The reported moisture content was analyzed to see if it fell within the range stated in the specifications. The outcome of the soil classification was compared to the specification to see whether it was adequate.

A proctor compaction curve was made with the same 5 values of moisture content and dry density used in the laboratory compaction test. The point where maximum dry density and optimum moisture content occurs was also plotted along with the zero air voids line and the 95% saturation curve. On the same graph the point corresponding to the value of moisture content and dry density determined with the sand cone test was inserted. This was done to visualize graphically the dry density achieved compared to the MDD, where the sample is respect to the zero air voids line curve and weather the sample falls within the specified range of acceptable moisture contents. A plot with the percent compaction contrasted with the water content was also produced to better comment on the achieved percent compaction.

The reported undrained shear strength was correlated, through a graph, with the reported dry density and with the percent compaction achieved. The values in the graph were contrasted with the results of test fill number 7. This was done to understand the relationship between density and shear strength. The differences between the test fill and the actual clay core of the dam, along with the importance of these material properties in the resulting performance of the clay core were analyzed. Comments were made regarding their relationship.

Findings and Recommendations

Analysis was performed to:

- Select most adequate type of structure to be used for construction of Borinquen Dam 1E.
- Develop compaction requirements and testing specifications for Zone 1 clay core, based on the results of Zone 1 test fill number 7.
- Evaluate the actual compaction achieved in the field, based on analysis of test results, against project specifications and against personally designed specifications.

Recommendations were produced based on the findings of the analysis that was performed.

Objective 1: Selection of best type of dam structure to use for construction of Borinquen Dam 1E

Below are the various findings and discussions regarding the selection of the best type of dam to be used for building Borinquen Dam 1E. Recommendations on what type of structure is most appropriate for the construction of Borinquen Dam 1E are stated at the end of the discussed findings.

Finding 1: Selection of best type of embankment dam option

The two types of embankment dams were compared to one another and with Borinquen Dam 1E design criteria requirements. The central core earth and rockfill dam was chosen as the best suitable embankment dam option.

Analysis

The central core earth and rockfill embankment has a better degree of filter control of internal piping and erosion and it has a better control of pore pressures for stability compared to the zoned earth fill embankment. This can be deduced from the table comparing their main features, displayed appendix A. It also has less future settlement if construction is done adequately (Toose, 2014). Furthermore, the zoned earthfill is less suitable for earthquakes since the clay core in earth and rockfill embankments has a better degree of control over seismic displacements (Toose, 2014). Additionally, material suitable for the clay core embankment was found in the surrounding borrow areas (URS Holdings, Inc., 2008). Therefore, between the two considered embankment structures, the zoned earthfill embankment was excluded and the central core earth and rockfill dam was chosen.

Finding 2: Selection of best type of concrete dam option

The two types of concrete dams were compared between each other and with Borinquen Dam 1E design criteria requirements. The Roller Compacted Concrete dam was chosen as the best suitable concrete dam option.

Analysis

The advantages that RCC dams have over the mass concrete gravity dam are: lower material cost due to the specific mix design, lower costs and time benefits attributed to the higher rates of concrete placement, less post cooling time and the ability to transport the RCC concrete mix design with dump trucks. These advantages significantly shorten the construction period of the project, compared to that of construct mass concrete gravity dams. This brings additional benefits like reduced administration costs, earlier project benefits and possible construction at sites that have season constraints. Therefore the Roller Compacted Concrete Dam was evaluated to have more advantages in respect to the mass concrete gravity dam, which was excluded for the construction of Borinquen Dam 1E.

Finding 3: Selection of best type of dam structure option

The central core earth and rockfill embankment dam and the Roller Compacted Concrete dam possibilities were contrasted with each other and with Borinquen Dam 1E design criteria requirements. The central core earth and rockfill embankment dam design was chosen for the construction of Borinquen Dam 1E.

Analysis

More suitable material available for the clay embankment was found in the surrounding borrow areas in respect to aggregates needed for the RCC mix design (URS Holdings, Inc., 2009) (Toose, 2014). The dam site is underlain by volcanic and sedimentary rock of Miocene age. The main formations identified in the construction location are La Boca formation fault and a minor occurrence of the Pedro Miguel Formation fault. Several boring logs indicate that there is rock that has been crushed, sheared and altered as a result of movement of the rock masses (URS Holdings, Inc. "GIR", 2009). La Boca Formation is composed of a sedimentary assemblage of different rock types and their hardness ranges from very soft to medium hard rock. Many high percentage of highly plastic clay minerals were found in the formation, which cause it to be susceptible to slaking (URS Holdings, Inc. "GIR", 2009). Therefore the analysis of the foundation has concluded that the design of an embankment dam is more suitable, since it could not bear the weight of a concreted dam; the foundation is too weak (Toose, 2014).

Furthermore, according to the outcomes of the seismic analysis, it was concluded that the dam is likely to undergo fault displacement and possible deformations, including crest settlements. Therefore is was

concluded that, the central core earth and rockfill embankment is more capable of resisting the possible deformations caused by seismic displacements, without loss of structural integrity and hence functionality, compared to the concrete dams (URS Holdings, Inc., 2009). This can be achieved if the materials used for the clay core of the embankment dam are chosen to have an adequate ductility and plasticity. This will allow the core to deflect along with a seismic displacement. Concrete on the other hand has a limited capability of being ductile and flexible, it has a higher potential to crack in the event of a seismic displacement (Toose, 2014).

Recommendations

The performed research and the analysis of options available for the selection of best type of dam structure to be used for building Borinquen Dam 1E was based on: the design requirements of strength, static and seismic stability, seepage performance, environmental constraints, cost and time of construction and material availability. The outcome found that the central core earth and rockfill embankment dam design is recommended for the construction of Borinquen Dam 1E since it is the best suitable type of structure to be used and most cost efficient.

Objective 2: Developed compaction requirements and testing specifications for Zone 1 clay core, based on the results of Zone 1 test fill number 7.

Following are the findings and discussions regarding the compaction requirements and testing specifications for Zone 1 clay core. The developed recommendations regarding the compaction requirement and quality control specification are summarized at the end of the enumerated and discussed findings.

Finding 1: Specified requirement for the source and Soil Classification of the core material

The outcomes of the analysis determined that for the core to be impermeable, have sufficient strength, be sufficiently ductile and flexible, and have an adequately low compressibility, the material to be used should be residual soil. The high plasticity silt (MH) and clay (CH) are suitable for use as core material. The silty gravel (GM) and clayey sand (SC) are adequate alternatives but MH and CH soils are preferred.

Analysis

The classification of the residual soils derived from basalt and other agglomerate residual soils, based on the United Soil Classification System (USCS), which have formed through intensive weathering of underlying bedrock (URS Holdings, Inc. "draft", 2008), placed in test fill number 7 are displayed in appendix B. They are mostly classified as elastic silt (MH). Only two of the soils were differently classified as silty gravel (GM)

and dense sandy clay (CH), according to the contractor test results. On the other hand according to the ACP test results, the residual soil placed in the test fill number 7 classified as clayey sand (SC). For the dam to be adequately impermeable, as described in the background section, the soil placed in the core must have a “permeability coefficient” of at least 10^{-6} m/s, which can be achieved if the material has significant amount of fines and is placed and compacted adequately at a specific water content, so that it can reach a satisfactory percent compaction. Therefore, the high plasticity silty (MH) and clayey (CH) residual soils that have formed are definitely suitable for use as core material since they are weathered clays with a high percent of fines, with more than 50% passes the No. 200 sieve (ASTM international “USCS”, 2014). The silty gravel (GM) and Clayey Sand (SC) should preferably not be used since they have a lower percent of fines, more than 50% retained on the No. 200 sieve (ASTM international “USCS”, 2014). However, they could still be used if the gradation test results show that they have a sufficient quantity of fines to meet the design requirement and if the Atterberg limit test results also show that they have an adequate plasticity index, which meets the design requirement.

Finding 2: Specified requirement for the maximum particle size and particle size distribution

The outcomes of the analysis determined that for Zone 1 to be impermeable, have sufficient strength, be sufficiently ductile and flexible, and have an adequately low compressibility, the material gradation specifications should be: 100% passing 3” (75mm) sieve, $\geq 70\%$ passing $\frac{3}{4}$ ” (19mm) sieve and $> 25\%$ passing No.200 (0.075 mm) sieve. The test fill number 7 materials all pass the specified gradation requirement.

Analysis

The maximum particle size (of gravel and rock fragments) to be placed in the earthfill is limited to ensure that compaction outcome is not affected. Usually the choice is to specify the maximum particle size to not be greater than a size in the range of 3” (75mm) to 6” (150mm) (Fell, 2005). For this specific dam the soil used for the core should be specified to have a 100% passing requirement of the 3” (75mm) sieve. This requirement was chosen to ensure that adequate compaction is achieved and, as a consequence, effective permeability and strength are met. Since the constraints of this project deal with high precipitation, there is a high possibility that a significant portion of the core materials might be placed significantly above the optimum moisture content. Thus, requiring a 100% passing the 3” sieve is an additional aid toward securing an adequate compaction. Both the contractor’s and the ACP’s test fill results comply with this requirement.

The particle size distribution is further defined to ensure that there is sufficient silt and clay fines passing the No. 200 (0.075 mm) sieve. This helps guarantee the low permeability of the core. It is normal to require at least 15% passing in the No.200 (0.075mm) sieve. For most clays, sandy clays and clayey sands this

requirement will already be met (Fell, 2005). For the core material of Borinquen Dam 1E it should be specified that at least 25% of the material pass the No. 200 (0.075mm) sieve. An additional 10% has been added to the normal requirement of 15% fines to ensure that the low permeability and adequate compaction of the core is achieved. This precaution has been added since precipitation at the site is very frequent and compaction above optimum moisture content is likely to occur, which could compromise the outcome of the strength, compaction and permeability of the core. The contractor's results, of the gradation tests performed on test fill number 7, show that all of the soil samples pass this requirement, in fact they all have percent passing values, of the No. 200 sieve, which are greater than 70%. The ACP gradation analysis test results still satisfy the requirement even though they have a much lower percent passing the No. 200 sieve, 46% and 34%, compared to the contractor's results.

Furthermore in order for the soil to comply with ASTM D698 standard, for the laboratory compaction of soils using standard effort, the soil used for the core of the embankment must retain a maximum of 30% on the $\frac{3}{4}$ " (19 mm) sieve (ASTM international, 2014). The percent passing the $\frac{3}{4}$ " (19 mm) sieve is also specified to prevent areas of nested coarse material within the core, since the residual soil can be gap-graded (Fell, 2005). Thus the soil must have 70% passing the $\frac{3}{4}$ " (19mm) sieve requirement. According to the gradation test results of both the contractor and the ACP the soil used in test fill number 7 comply with this requirement.

The tables below show the Contractor's and the ACP's various gradation test results done on the soil of test fill number 7. The vertical blue lines are the specified sieves 3", $\frac{3}{4}$ " and No. 200. The horizontal blue lines are the percent passing requirement of the respective specified sieves, 100%, 70% and 25%. The red dotted line is the required gradation specification.

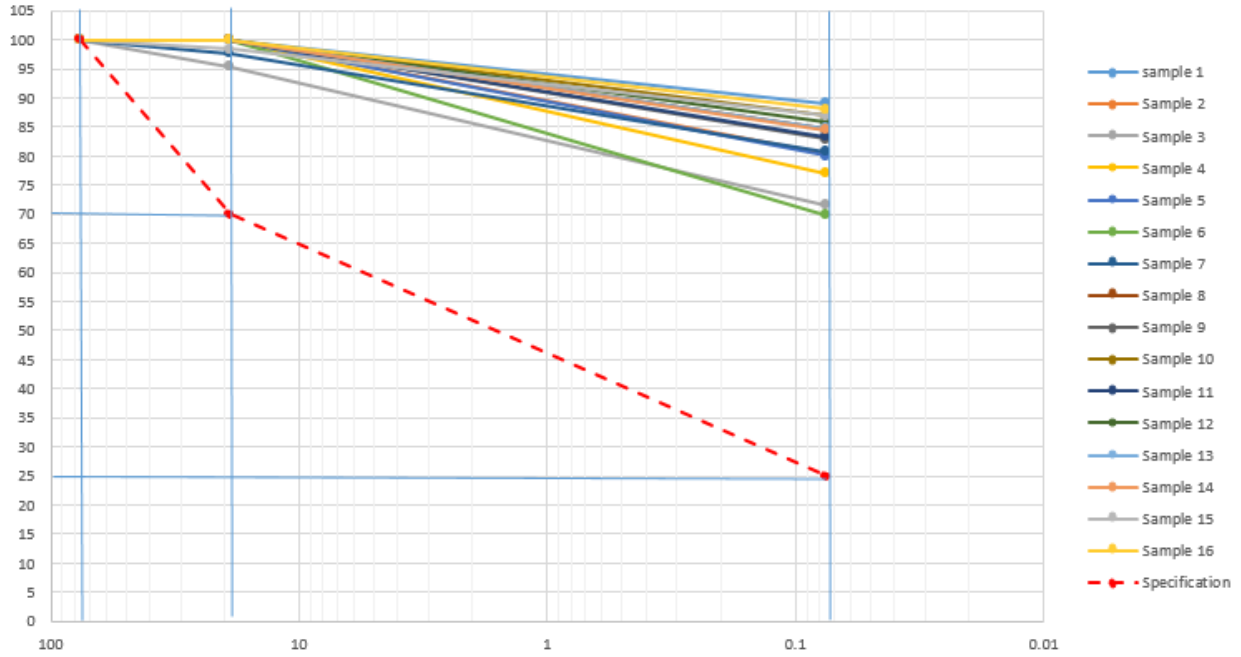


Figure 30: Contractor Test Fill Number 7 Gradation Tests Results Plotted Against the Specified Gradation Requirement

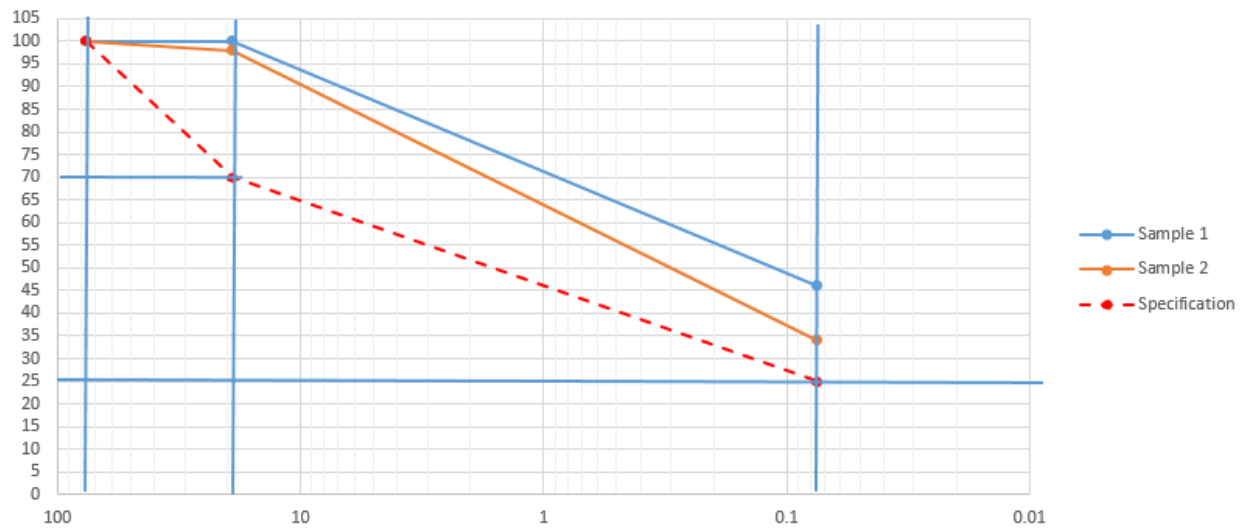


Figure 31: ACP Test Fill Number 7 Gradation Tests Gradation Test Results Plotted Against the Specified Gradation Requirement

The graphs visually show that all the tested soil samples pass the specified gradation requirement, since all the samples gradations fall above the red dotted specification line.

Finding 3: Specified requirement for the Atterberg limits

The outcomes of the analysis determined that for the core to be impermeable, have sufficient strength, be sufficiently ductile and flexible to remain functional after seismic displacements, and have an adequately low compressibility, the lower bound on the plasticity index should be 10 and the upper bound on the liquid limit does not need to be specified.

Analysis

A minimum plasticity index is usually specified in clay materials used in dam constructions and sometimes a maximum liquid limit is specified as well. The upper bound on the liquid limit value needs to be specified because of the possible presence in the borrow area of particularly high plasticity clays, which may be difficult to compact. Nevertheless, there is evidence that shows that higher plasticity clays are more likely to be less erodible making it advantageous to have in the core of the dam. The crucial issue is to specify limits which can be satisfied with the materials present in the borrow area (Fell, 2005). Further research was done regarding the upper limit of the liquid limit and it was found that most clays are suitable to be used for earthfill as long as they are inorganic and insoluble. However clays with a liquid limit values above 80 should generally not be used (U.S army Corps of Engineers, 2004).

The plasticity index required for the material to be placed in Zone 1 should be specified to be higher than 10. This is done in order to avoid certain types of soil material and to have an adequately plastic soil which will be able to deflect adequately in the event of a seismic load and allow the core of the dam to still remain functional (Toose, 2014).

The graph below shows the Plasticity Indices (PI) and the Liquid Limits (LL) of the test fill. A trend can be observed in which, as the liquid limit increases so does the plasticity index. The red line is the specified plasticity index requirement of 10.

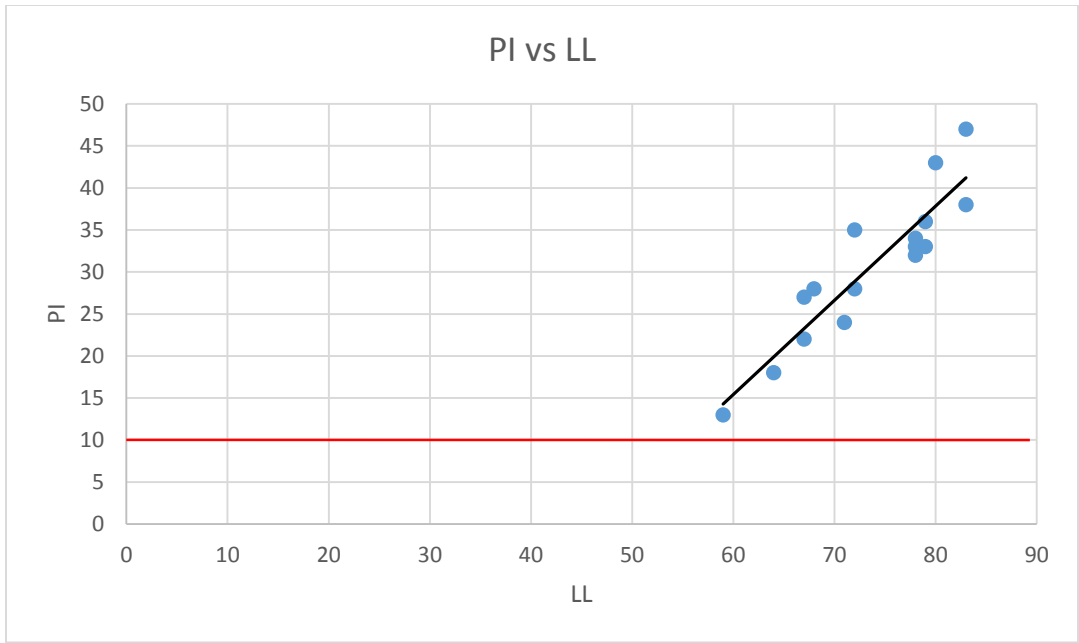


Figure 32: Plot of Plasticity Index and Liquid Limit with plasticity index requirement

The graph above shows that all of the test fill number 7 results comply with the specifications, all of the tested materials had a plasticity index that was above 10.

The graph below shows the Plasticity Indices and the Liquid Limits of the test fill along with the suggested upper bound of 80 for the liquid limit.

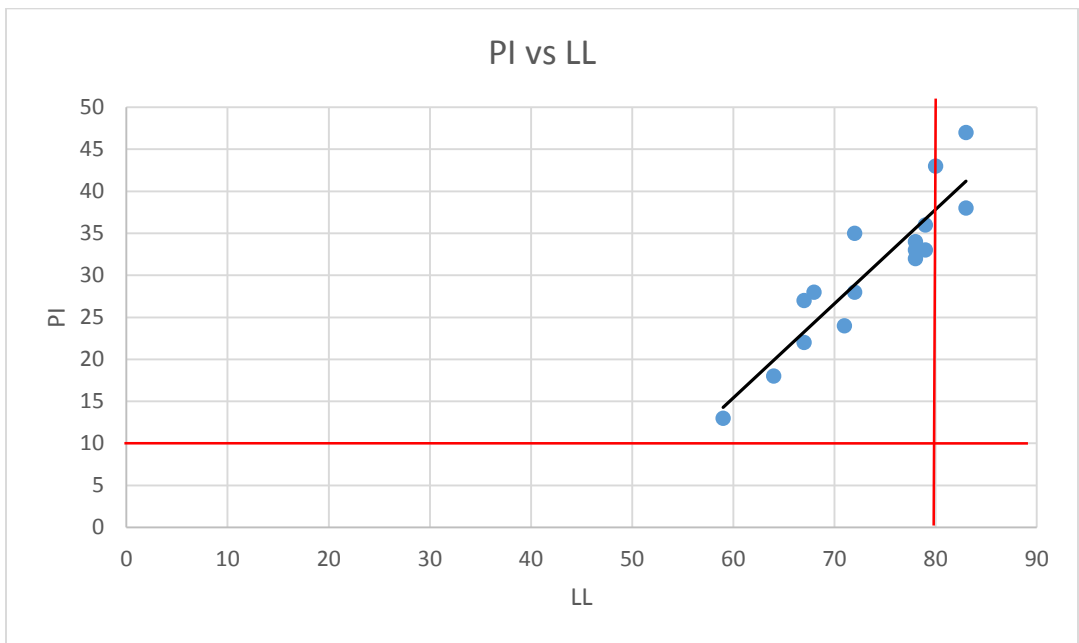


Figure 33: Plot of Plasticity Index and Liquid Limit with suggested liquid limit requirement

If the upper bound requirement on the Liquid Limit (LL) would be specified to be 80 some samples of test fill number 7 would fail the specifications and others would be very close to not meeting the requirement as well. This could potentially exclude large amounts of material from the borrow areas that could probably be suitable for placement in the core of the dam. Therefore, there will be no specified upper bound on the liquid limit.

Finding 4: Specification requirement for the strength

The outcomes of the analysis determined that for the core to have sufficient strength for static stability, seismic stability in the events of earthquake displacements and withstand construction loads, the specified minimum undrained shear strength should be 75 kPa.

Analysis

The strength of Zone 1 must be sufficient enough to have static stability, to withstand seismic loads (which may occur since the dam is built on several faults) and to endure construction loads (Toose, 2014).

To satisfy these requirements the shear strength of the core material must be higher than 60 kPa, according to the previously done stability analysis (URS Holdings, Inc., 2009) (Toose, 2014). To make sure that this value is met throughout the entire core of the dam it is specified that the material placed, compacted and then tested for shear strength, with a shear vane, must have a resulting value of 75 kPa or higher to be acceptable (URS Holdings, Inc., 2009). According to test fill number 7 results, the shear strength of the placed and compacted soil was achieved except for the last set of four vane shear tests performed by the contractor, which were all below the 75 kPa limit. The table below summarizes the water content range, optimum moisture content and percent compaction of the samples that failed and passed the shear strength requirement.

Table 26: Water Content Range, Optimum Moisture Content and Percent Compaction of the Samples That Failed and Passed the Shear Strength Requirement

Required shear strength	Water content range (%)	Optimum Moisture Content Range (%)	Compaction (%)
75 kPa			
Shear strength failed	43 - 49.5	36 - 40	92 - 94
Shear strength passed	40 - 47	35 - 38	88 - 100

It was hard to understand the reason for shear strength failure in those specific samples, since other samples which passed the shear strength requirement had a lower percent compaction than the ones which failed and were compacted at higher moisture contents respect to the samples that failed.

To understand the soil's correlations between its strength, water content and density properties, and, to generate adequate specification requirements for percent compaction and water content placement range, further analysis has been performed.

Finding 5: specified requirements for water content and density ratio

The outcomes of the analysis determined that for the core to be compacted adequately, have sufficient strength, be sufficiently ductile and flexible, and have an adequately low compressibility, the compaction water content range should be specified to be between +2% and +8% of Optimum Moisture Content (OMC) and the dry density ratio should not be specified.

Analysis

According to the authors of "Geotechnical Engineering of Dams": "it is common practice to specify a density ratio $\geq 98\%$ of standard maximum dry density, with a water content between OWC -1% and OWC +1, or OWC and OWC +2%, where OWC is standard compaction optimum water content" (Fell, 2005).

The requirement of a density ratio $\geq 98\%$ is reasonable and compatible only if compaction is done at the previously stated water content ranges, very close to the optimum. There are no advantages in specifying higher density ratios than 98% since they are almost impossible to achieve. Furthermore, they could be detrimental, since the soil might have to be compacted dry of optimum. For dams to be constructed in wet climates, for relatively small dams, and when dealing with soils that are difficult to compact effectively, it is reasonable to have the compaction requirement be lower, for example 95%. However, in these cases compaction must always be done above optimum moisture content (Fell, 2005).

The lab test results from test fill number 7 were analyzed and various graphs were generated to understand the correlation between different properties of the material in order to come up with water content range and density specifications. The shear vane strengths were plotted against the material's water content. Both the water content calculated with a sample (taken at the same time and location where the vane shear test was performed) and the water content resulting from the sand cone test (also done at the same time and location of the shear vane test) were used.

Below are the plots of shear strength vs moisture content of both the contractor's and the ACP's test results. The x-axis values are the moisture contents (%) and the y-axis values are the shear strengths (kPa). The red line in the plot indicates the minimum required shear strength specification of 75 kPa.

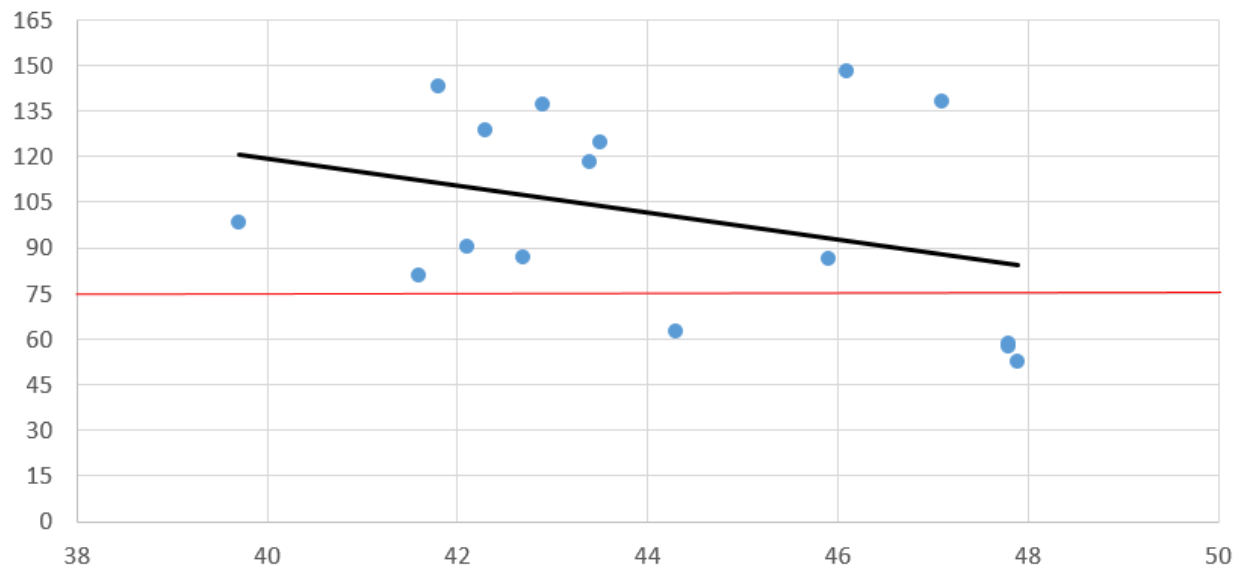


Figure 34: Contractor Test Results, Plot Of Shear Strength Vs Moisture Content

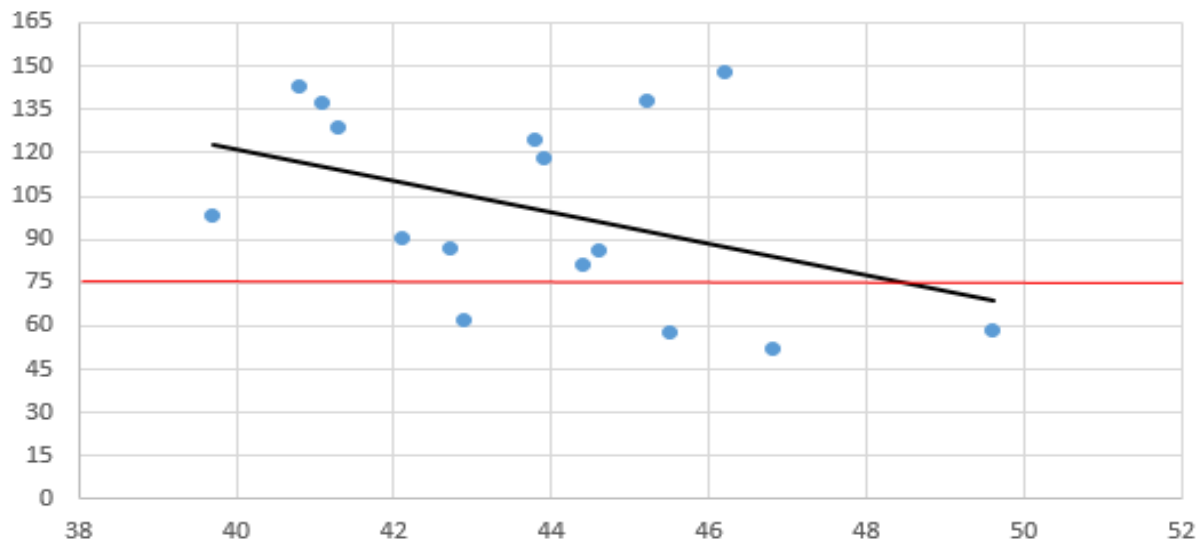


Figure 35: Contractor Test Results, Plot Of Shear Strength Vs Sand Cone Moisture Content

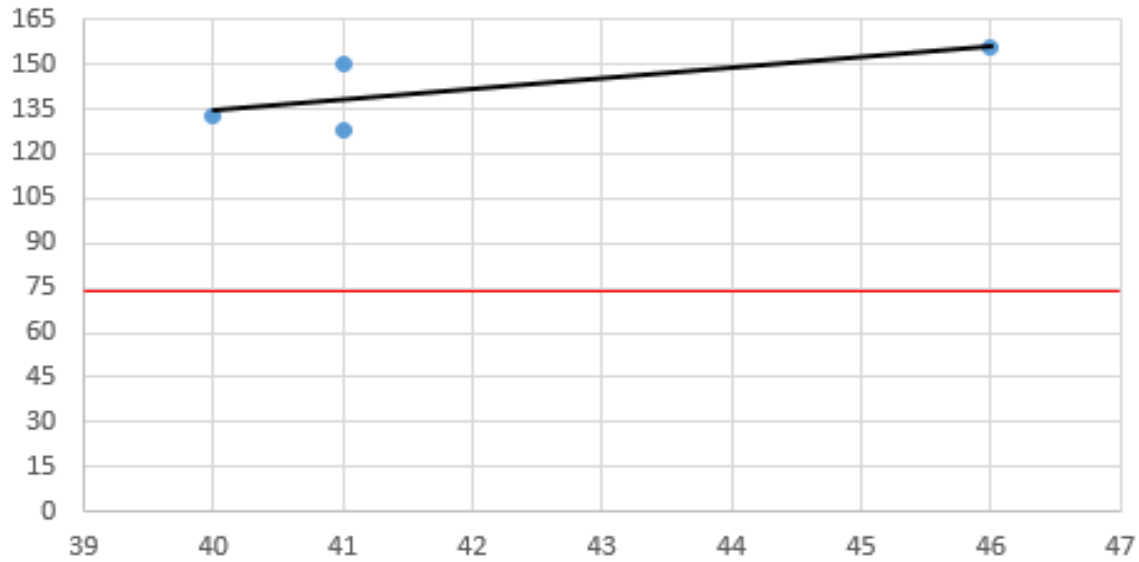


Figure 36: ACP Test Results, Plot Of Shear Strength Vs Moisture Content

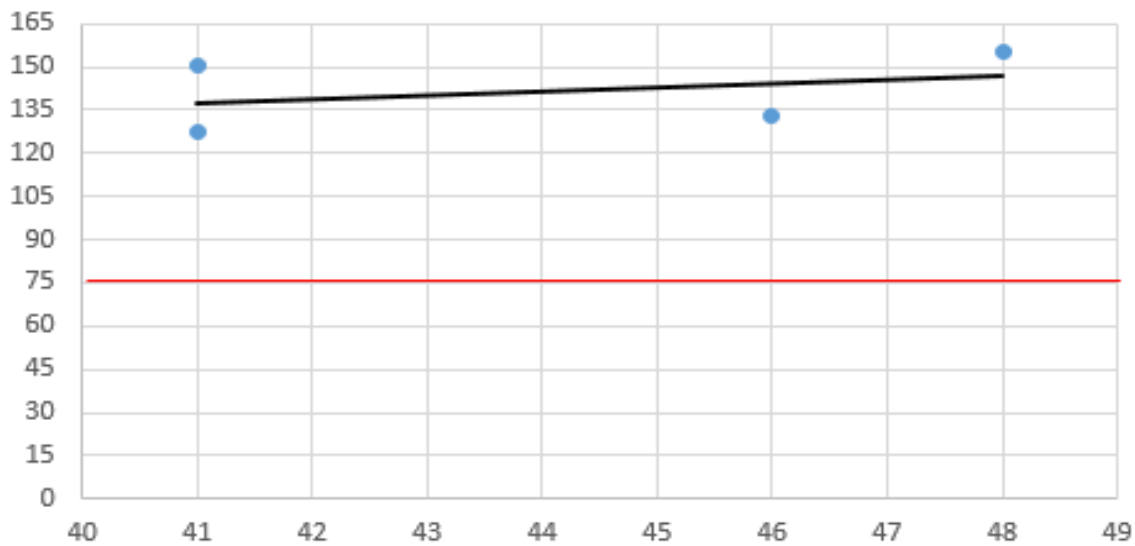


Figure 37: ACP Test Results, Plot Of Shear Strength Vs Sand Cone Moisture Content

From the generated graphs it can be observed that the compaction of the soil was done at a water contents which ranged between 40% and 48%. A correlation between the shear strength and the water contents of the soils is displayed. The graphs with the data received from the contractor show that as the water content of the soil increases the strength decreases. The opposite trend is shown when the data received from the ACP is plotted. Since the contractor has done more tests an assumption can be made that the trends shown by the plots generated with those results are more accurate. Therefore, in conclusion, as the water content in the soil increases its shear strength decreases.

The plots also visually illustrate that a portion of the soil failed the shear strength requirement of 75 kPa. These locations had a water content that ranged between 43% and 49.5%. Assumptions could be made that the soil was compacted too wet of optimum moisture content. However, other soil samples, tested for shear strength, achieved the required 75 kPa strength and had water contents in the same range as the ones that failed, or above.

The vane shear strengths were also plotted against the percent compactions achieved in the soil. The graphs can be seen below. The y-axis values are the shear strengths (kPa) and the x-axis values are the percent compactions (%). Again, the red line indicates the 75 kPa shear strength requirement.

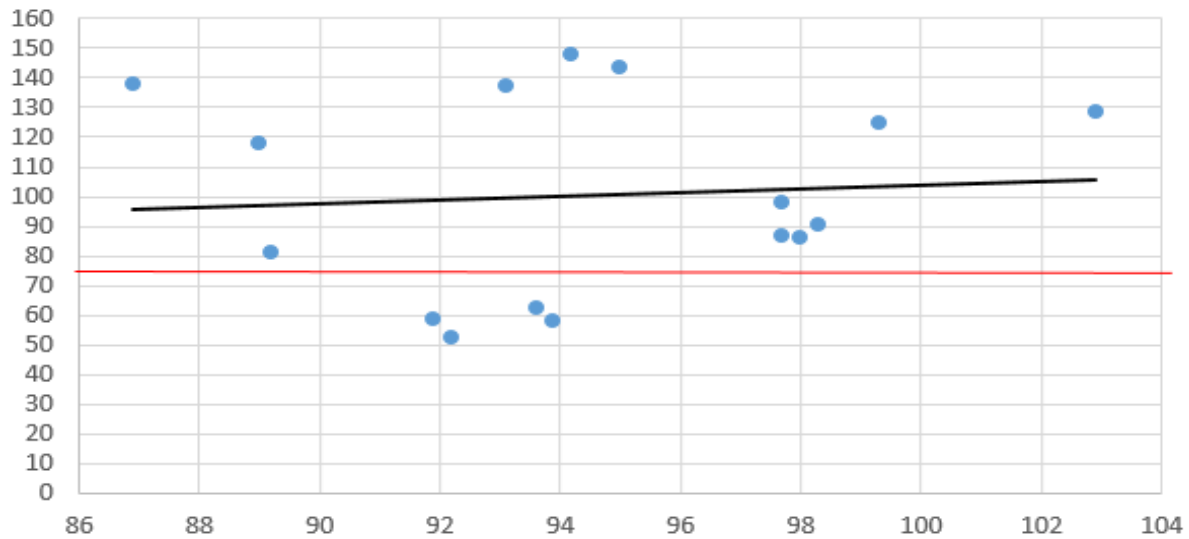


Figure 38: Contractor Test Results, Plot Of Shear Strength Vs Compaction

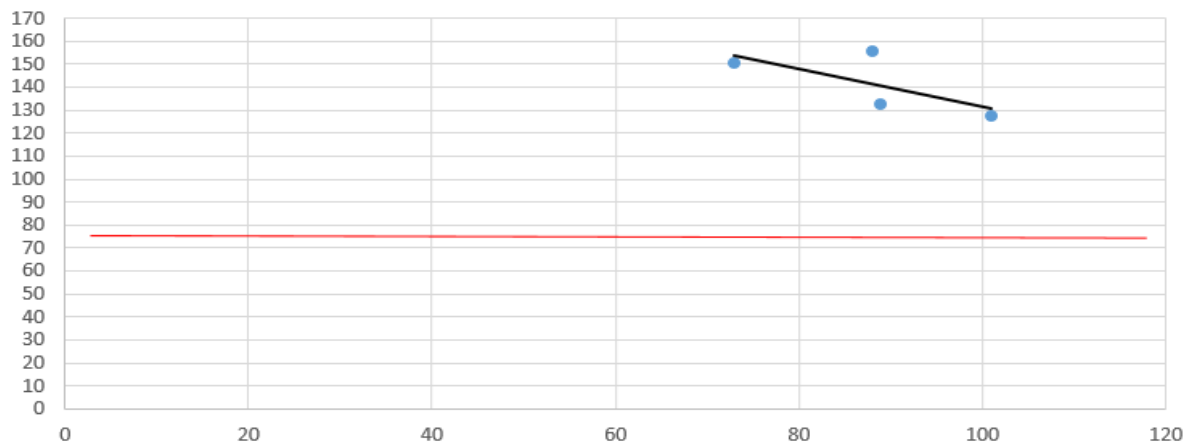


Figure 39: ACP test results, plot of shear strength vs compaction

According to the plotted data received from the contractor, the soil's shear strength increases along with its percent compaction. On the other hand the trend displayed by the ACP's data shows that the shear strength decreases as compaction increases. Since more tests have been performed by the contractor on the test fill, the results give a more accurate interpretation of the actual material properties of the soil. Hence, in conclusion, shear strength increases along with the percent compaction achieved. The plot of the contractor's values also visually illustrates that a portion of the soil samples tested failed to satisfy the shear strength requirement of 75 kPa around a compaction percentage range of 92% to 94%; even though some compacted samples achieved the required shear strength below 90% compaction.

Proctor compaction curves were generated in the laboratory to determine the optimum moisture content and maximum dry density of the compacted soil of test fill number 7. Below are all the optimum moisture contents plotted with their respective maximum dry densities. The MDD values are on the y-axis and the OMC values are on the x-axis.

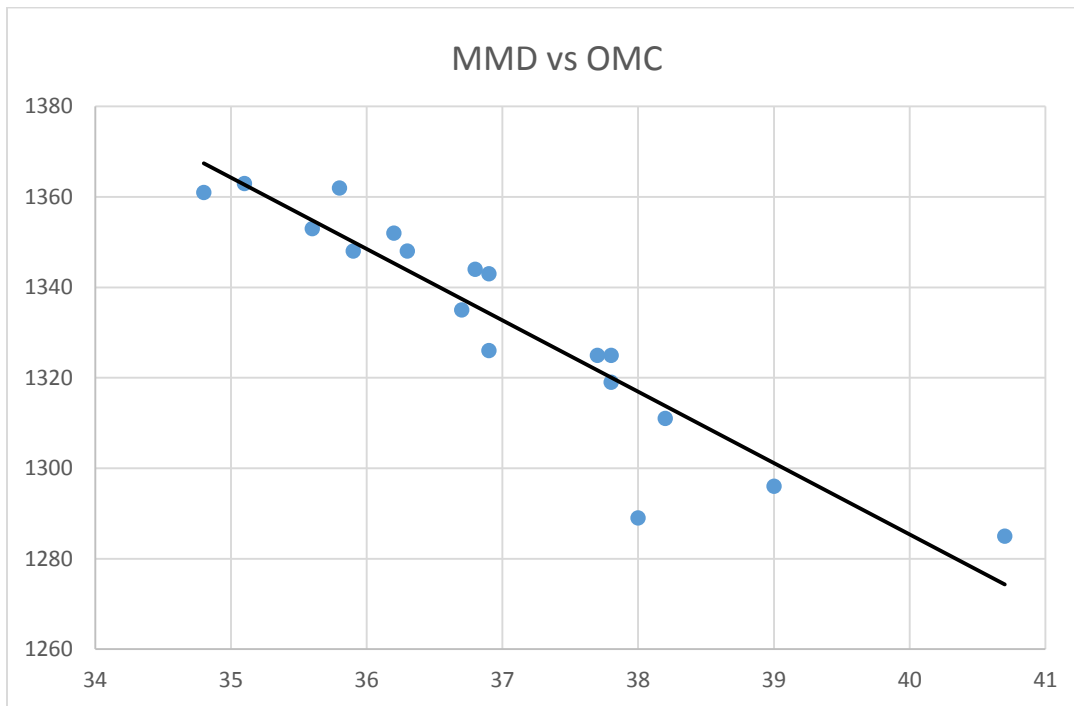


Figure 40: Plot of Maximum Dry Density vs Optimum Moisture Content achieved in the laboratory following ASTM D 698, standard test method for laboratory compaction characteristics of soil using standard effort

It can be observed that the optimum moisture content ranges from approximately 35% to 39%, that the maximum dry density ranges approximately between 1290 Kg/m³ and 1360 Kg/m³ and that as the optimum moisture content increases, the maximum dry density decreases.

Several tables, contrasting different material characteristics, were constructed to better analyze their properties and try to come up with both a water content placement range and a density specification. The table below displays the different ranges and averages of the optimum moisture contents along with the maximum dry densities, of the Contractor and of the ACP test results.

Table 27: Ranges and Averages of the Optimum Moisture Contents and the Maximum Dry Densities Achieved In the Laboratory

	OMC range (%)	Average OMC (%)	MDD (kg/m³)	Average MDD (kg/m³)
Contractor	34.8 - 40.7	36.8	1285 - 1363	1337.5
ACP	38 - 39	38.5	1289 - 1296	1292.5

It can be observed that the maximum and minimum values of optimum moisture content, obtained by the laboratory standard compaction, differ by about 6% for the contractor and 1% for the ACP test results. More testing has been done by the contractor, in order to have a wider range of values which would result in more realistic assessments. The same can be said for the ranges of the maximum dry densities. However, the average moisture contents and the average maximum dry densities do not differ enough to consider the test results to be incorrect. The average optimum moisture contents differ by about 2%, which is adequate considering the impossibility of perfect testing and for the soil to always have the same optimum moisture content.

The table below displays the differences between the averaged moisture content results of the soil samples used for the sand cone test and from the samples taken from the field for the moisture content test, of both the Contractor and the ACP.

Table 28: Contractor and ACP Test Result Average Moisture Contents from Moisture Content Test and Sand Cone Test

	Average Sand cone test moisture content (%)	Average moisture content test (%)
Contractor	43.8	44.2
ACP	44	42

Slight differences in the values can be observed but overall the values are similar enough to state that the testing has been done correctly and that the average moisture content of the tested soil ranged between 42% and 44%.

The table below displays the average undrained shear strengths, the moisture contents, the optimum moisture content, the percent compaction and the differences of the various moisture contents from the optimum, of both the Contractor and ACP test results. The last four sets of data values are bolded to better visualize the samples that failed the shear strength requirement.

Table 29: ACP and Contractor Test Fill Number 7 Average Undrained Shear Strengths, the Moisture Contents, the Optimum Moisture Content, the Percent Compaction and the Differences of the Various Moisture Contents Form the Optimum

Average undrained shear strength (kPa)	Difference of Sand Cone Moisture Content From OMC	Difference of Moisture Content From OMC	Moisture content from Sand Cone Test (%)	Moisture content (%)	OMC (%)	Compaction (%)
ACP test results						
150.54	2	2	41	41	39	73
155.65	9	7	48	46	39	88
132.68	8	2	46	40	38	89
127.58	3	3	41	41	38	101
Contractor test results						
128.65	5.7	6.7	41.3	42.3	35.6	102.9
137.27	6.3	8.1	41.1	42.9	34.8	93.1
143.23	5.7	6.7	40.8	41.8	35.1	95
147.88	8	7.9	46.2	46.1	38.2	94.2

86.87	4.9	4.9	42.7	42.7	37.8	97.7
90.18	5.4	5.4	42.1	42.1	36.7	98.3
80.9	8.1	5.3	44.4	41.6	36.3	89.2
98.14	3.9	3.9	39.7	39.7	35.8	97.7
137.93	7.5	9.4	45.2	47.1	37.7	86.9
124.67	7.9	7.6	43.8	43.5	35.9	99.3
86.21	7.7	9	44.6	45.9	36.9	98
118.04	7	6.5	43.9	43.4	36.9	89
52.39	9	10.1	46.8	47.9	37.8	92.2
62.33	6.1	7.5	42.9	44.3	36.8	93.6
58.35	8.9	7.1	49.6	47.8	40.7	91.9
57.69	9.3	11.6	45.5	47.8	36.2	93.9

This table was generated to better visualize and analyze the characteristics of the soil which failed in shear strength and compare them to the ones that had adequate shear strengths.

Some soil samples fail in shear strength at certain moisture contents and achieved percent compactions where other test samples passed. Therefore, it is hard to exactly correlate and determine when the soil is going to have a satisfactory shear strength at a certain moisture content and at an adequate compaction. However, it can be observed that a few of the samples that failed the shear strength requirement of 75 kPa had high moisture contents above optimum, which were of 9%, 10% and 11%. Hence, to avoid the potential shear failure compacting at these high moisture contents should be avoided. Some soil samples failed at moisture contents of 6% and 7% above optimum, but many other tested samples passes the shear strength requirement and also achieved shear strength way above 100 kPa. Therefore these particular samples can be treated as anomalies. To be extremely sure that the shear strength failure does not occur, compaction below 6% should be specified. However, this would exclude several of the test results to not meet specifications and hence the entire placement process would become more difficult

and the construction of the dam would be slowed down significantly. Additionally, due to the precipitation frequency of the site area, materials will most likely be found at the borrow area and placed at moisture contents high above the optimum; it will be difficult to place them at moisture contents close to the optimum value. If an exceedingly restrictive water content range is specified, several available material might not meet the requirement and will not be used. This could lead to material shortages. Furthermore, specifying unnecessary restrictive water content ranges for soil placement would probably lead to disputations between the engineer and the contractor (Fell, 2005). Thus, to be adequately safe but still be able to use reasonable amounts of material, the materials should be compacted at moisture contents between +2% and +8% of OMC. The +2% is specified based on the fact that the material needs to be compacted wet of optimum moisture content, so that the material is workable enough to be compacted adequately. Water content is to achieve the required shear strength, so that the material can achieve an adequate ductility and flexibility, and so that the material can achieve an adequate low compressibility. The +8% is chosen to ensure that the shear strength requirement will always be met, since most of the compacted lifts that failed, failed above 8%. It is also chosen so that compaction can still be feasible since the materials from the borrow site are going to have moisture contents above optimum and since placement and compaction is going to be performed in wet weather conditions. The specified limits need to be realistically achievable to match the materials available at the site and to suite the weather condition constraints during placement and compaction (Fell, 2005). Furthermore the upper bound of 8% is specified so that the material is not too moist which would not allow for an adequate compaction to be achieved and to ensure that the compacted material does not end up having a high compressibility, which would lead to high future settlements and, as a consequence, potential damages to the dam.

Based on the displayed results it is difficult to determine an adequate specification for the percent compaction to be achieved in the soil. The results that failed in shear strength all had a percent compactions above 90%. The results that passed had similar percent compactions and also percent compactions below 90%. The percent compaction should not be specified since it is hard to find a precise correlation between shear strength and percent compaction based on the test fill number 7 results. Compaction is going to be done at significantly high percentages above optimum, so it would be feasible to specify a compaction percentage that has a high potential to not meet specifications even though the shear strength requirement does. If percent compaction is specified and a test result of a certain lift of compacted soil complies with shear strength but not with percent compaction, it will have to be removed which is costly and would slow down the entire project. This would make the whole compaction and testing process harder, more time consuming and more costly. Furthermore, specifying

a percent compaction requirement for quality control would require proctor tests along with sand cone tests to be done. These test take more time to perform and there is more room for error compared to the shear vane test which gives an instant result in the field and does not require laboratory procedures. There are benefits of specifying the testing of the undrained shear strength of the compacted clay core. The advantages of this method of testing technique are that it is useful where soils are to be compacted significantly wet of optimum and that the testing procedure is easy and the results are instant (Fell, 2005). Anyhow, the percent compaction and dry density achieved in the soil should be calculated occasionally, as a verification of the shear vane strength test. It is easy and convenient to mainly rely on one test, since it is more efficient in regards with time and cost, but testing for density should be done intermittently to check the validity of the shear strength test results and to not only rely on one specific test for quality control.

Finding 6: Specification requirements for quality control

Test types, frequencies and ASTM standards that need to be followed for quality control procedures, were developed, based on the outcome of the performed analysis. The produced specifications are organized in the table below.

Table 30: Quality Control Laboratory and Field Testing Types, Frequencies and ASTM Standards, For Zone 1 of the Embankment

Test Type	Testing Frequency	ASTM Test Method
Shear strength	1 test per lift per 100m horizontally and 1 test per lift per 200m horizontally thereafter	D 573
Water content	At every shear test location	D2216
Gradation	Every 1500 m ³	C 136
Atterberg limits	Every 1500m ³	D 4318
Optimum moisture content	Every 1500m ³	D 698

Testing for dry density and percent compaction achieved is specified to be done every 10,000m³ and whenever the field inspector feels the need to perform it, to check the validity of the shear vane test. It is done with the sand cone tests ASTM D 1556.

Analysis

After the specifications for the placement and compaction of Zone 1 have been developed testing specifications on frequency need to be produced for quality control, to check whether the specifications are being met. Testing performed to investigate that the specified requirements are being met is externally important. However, testing is costly and time consuming. Tests cannot be performed at all times and at all locations. Hence, a frequency criteria must be adopted. Several aspects influence the frequency and locations of testing for quality control, it depends primarily on the type of material and importance of the compacted fill relative to the entire project. Impervious core material require more control compared to random fill (US army corps of Engineers, 1995). The authors of “Geotechnical Engineering of Dams” give some useful examples regarding frequency of testing required for some large fill dams in various countries (Fell, 2005). By analyzing the testing frequency of the zoned earth and rockfill embankment of the Ranger Mine Tailings Dam in Australia (135,000 m³ core), which has a relatively similar fill volume to the Borinquen Dam 1E (460,000 m³ core), conclusions regarding testing frequency can be drawn (Fell, 2005). The table below shows the minimum required frequency of construction testing for the clay core of the Ranger Mine Tailings Dam in Australia (Fell, 2005).

Table 31: Minimum Required Frequency of Construction Testing For Zone 1 Earthfill (Fell, 2005)

Type of test	Particle size	Atterberg limits	Water Content	Density Ration	Shear strength
Frequency	Every 3000 m ³	Every 3000 m ³	1 per shift	Every 500 to 1000 m ³	Some

The drawn conclusions are also based on the critical importance of the effectivity of Zone 1 and on the fact that the Borinquen Dam 1E core earthfill will be compacted at moisture contents significantly wet of optimum and in climates where precipitations are frequent. Therefore, it has been decided that quality control tests should be done more frequently, respect to the ones done on standard dams, to ensure safety and proper outcome of the core’s functionalities.

For the Borinquen Dam 1E core earthfill it should be specified to use an undrained shear strength test, following the ASTM D2573 standard for field vane shear test in cohesive soils, to test if the required 75 kPa strength in the compacted soil is reached. The resident engineer James Toose suggested to leave the frequency of the vane shear test as the one specified in the final design report of Borinquen Dam 1E (Toose, 2014). Thus the vane shear test should be performed as follows: “1 test per lift per 100m horizontally of zone 1 core and 1 test per lift per 200m horizontally of zone 1 thereafter” (ACP, 2009).

The testing frequency for the particle size, Atterberg limits and the optimum moisture content was chosen to be done every 1500 m³, twice the testing frequency done on the Ranger Mine Tailings Dam in Australia; since the functionality of the core is very important for the successful outcome of the project. Also, since the material is being placed at moisture contents significantly high above the optimum, it has been specified that the moisture content test needs to be done at every shear strength test location. This will ensure that the material gets placed within the specified range of acceptable moisture contents, which might be challenging due to the frequent precipitations at the site and the already moist soil from the borrow areas.

The percent compaction and dry density achieved in the soil should be calculated occasionally, with the sand cone tests ASTM D 1556, as a verification of the validity of the shear vane strength test. The testing for dry density and percent compaction achieved should be done whenever the field inspector feels the need to perform it, to check the validity of the shear vane test. It should also be done every 10,000 m³ as a safety measure to ensure that compaction is being done correctly, since the outcome of the core’s functionality is extremely important for the overall success of the dam project.

Summary of Zone 1 personally generated specifications

- 1) Material to be residual soil (MH and CH preferred, GM and SC alternatives)
- 2) 100% passing 3” (75mm) sieve, ≥ 70% passing ¾” (19mm) sieve and > 25% passing No.200 (0.075 mm) sieve
- 3) PI > 10
- 4) Minimum undrained shear strength = 75 kPa
- 5) Compaction water content range between +2% and +8% of OMC

Recommendations

The engineering properties of the materials to be used for the clay core of Borinquen Dam 1E were deduced from the analysis performed on the results of Zone 1 test fill number 7. Consequentially,

compaction requirements and quality control testing frequency specifications for Zone 1 were developed. They were developed by contrasting the soil's engineering properties and the soil's performance in the compacted test fill with: the design criteria requirements of the dam, the importance of the clay core's functionality and the previously done research on general testing criteria specifications for embankment dam clay cores.

The recommended personally developed specifications were: (1) material to be residual soil (MH and CH preferred, GM and SC alternatives), (2) 100% passing 3" (75mm) sieve, \geq 70% passing $\frac{3}{4}$ " (19mm) sieve and $>$ 25% passing No.200 (0.075 mm) sieve, (3) PI $>$ 10, (4) Minimum undrained shear strength = 75 kPa, (5) compaction water content range between +2% and +8% of OMC.

The specifications are similar to the actual project specifications except for some slightly more restrictive aspects. 100% passing the 3" sieve instead of 100% passing the 6" sieve and placement of soil to be done at a maximum of +8% above OMC instead of a maximum of +12% above OMC.

The recommended personally developed quality control test types and frequency specifications were the same as the present ones, except for the frequency criteria adopted for the Atterberg limit, gradation and optimum moisture content tests. The criteria was based on the volume of fill that is placed (tests done every 1500m³ of fill placed), in comparison with the actual testing frequency which is based on the frequency of the undrained shear strength test (every tenth undrained shear strength test). It was additionally recommended to test for density by performing the sand cone test, as a verification of the vane shear strength test's validity, every 10,000 m³ or whenever a field inspector requires it to be necessary.

Objective 3: Evaluated Field and Laboratory Test Results

The following section details the findings and discussions regarding: the field and laboratory supervisions and the results of the field and laboratory quality control tests performed on a compacted lift of Zone 1. The developed recommendations regarding the quality of the field and laboratory tests are summarized in the conclusion of this section.

Finding 1: outcome of field compaction supervision

During the supervision of the Zone 1 compaction process, Sebastien Bonnart had to intervene and require the contracted workers to use a tamping-foot compactor instead of the dozer. It was found that a tamping-foot compactor created a better bonding between lifts during the compaction of the test fills, which is why it is recommended in the specifications (ACP, 2009). The workers had decided to use the dozer since it was located closer than the tamping-foot compactor. Accordingly, supervision of the procedures completed on the field was recommended.

Finding 2: outcome of laboratory test supervision

The supervision of the tests being done in the laboratory concluded that the ASTM procedures were being followed correctly except for the sand cone test. Sebastien Bonnart realized that during the calibration process, determination of the Bulk Density of the sand used for the test was being done by following the “alternative method B” and not the “preferred method A” in the ASTM standard D 1556 Annex (ASTM international, 2014). The “Preferred method A” was then performed for the calibration of the sand density. The results of the calculated dry density were compared to the ones previously obtained by following “alternative method B”. Differences were found, and so the results were then plotted on the generated proctor compaction curve to measure the percent compaction achieved in the soil. The density results obtained by following “preferred method A” were determined to be more accurate, since the dry density values of the soil from “alternative method B” fell above the zero air voids line on the proctor curve. This means that the soil was achieving more than 100% compaction, which is not possible. From then on, the determination of the bulk density of sand when performing the sand cone test was done by using the “preferred method A”. Accordingly, supervision of the laboratory testing procedures was recommended to be done occasionally.

Finding 3: Comparaison between vane shear test for strength and sand cone test for density

During field supervisions of quality control procedures, the vane shear test was acknowledged to be easily executable in the field and provide instantaneous adequate results. Alternatively, the sand cone test was considered to have a less accurate performance compared to the vane shear test. There is more room for

error when performing it in the field due to the fact that an exact amount of soil needs to be excavated and retained before the sand gets dumped in to the excavated hole. It is more costly to perform, with respect to the vane shear test, since it requires very fine sand that cannot be reused for multiple tests. The sand for the Borinquen Dam 1E project is shipped from Alberta, Canada, and is very expensive (Bonnart, 2014). There is also more room for error during the density determination of the soil in the lab. Furthermore, the test results take more time to get produced with respect to the vane shear test. Thus, testing for strength with the vane shear test was determined to be a better choice to be used as the primary quality control method.

Finding 4: evaluation of whether the tested lift of Zone 1 met project specifications

The evaluation of the test results against the project’s specifications concluded that all the core’s specified designed criteria requirements were adequately met.

Analysis

Below are several tables which display the results of the tests performed on Zone 1, along with the required specification and whether the specification was met or not.

Table 32: Shear Strength

Specified Shear Vane Strength (kPa)	Shear Vane Strength test Result (kPa)	Check
>75	87.83	Ok

Table 33: Plasticity Index

PI specification	PI test result	Check
>10	22	Ok

Table 34: Material Gradation

	100% passing 6" Sieve		>70 % passing 3/4" Sieve		>35% passing No. 200 Sieve	

Specification		Check		Check		Check
Percent passing results	100%	Ok	98.50%	Ok	81%	Ok

Table 35: Water Content

Water Content specified range	OMC Result (%)	Water Content Result (%)	Percent above optimum	Check
+2% to +12% of OMC	38	46.8	8.8	Ok

Table 36: Soil Classification

Specification on material to be used for clay ore	Result of Soil Classification	Check
Residual soil with no organic material	MH	Ok

According to the values displayed in the tables, all of the test results adequately met the project's specifications.

Finding 5: evaluation of whether the tested lift of Zone 1 met personally developed project specifications

The evaluation of the test results against the personally developed project specifications concluded that all of the core's specified designed criteria requirements were adequately met except for the required water content.

Analysis

Below are several tables which display the results of the tests performed on Zone 1, along with the personally designed specifications and whether the specifications were met or not.

Table 37: Shear Strength

Specified Shear Vane Strength (kPa)	Shear Vane Strength test Result (kPa)	Check
>75	87.83	Ok

Table 38: Plasticity Index

PI specification	PI test result	Check
>10	22	Ok

Table 39: Material Gradation

Specification	100% passing 3" Sieve	Check	>70 % passing 3/4" Sieve	Check	>25% passing No. 200 Sieve	Check
Percent passing results	100%	Ok	98.50%	Ok	81%	Ok

Table 40: Moisture Content

Water Content specified range (%)	OMC (%)	Water Content Result (%)	Percent above optimum (%)	Check
+2% to +8% of OMC	38	46.8	8.8	Fail

Table 41: Soil Classification

Specification on material to be used for clay ore	Result of Soil Classification	Check
Residual soil (MH & CH preferred, GM & SC alternatives)	MH	Ok

According to the values displayed in the tables, all of the test results adequately met the personally designed specifications, except for the moisture content. The soil’s calculated moisture content resulted in being 8.8% above the calculated optimum moisture content. The designed water content placement range specifies a maximum of 8% above the optimum. However, since all of the other test results comply with the specification, especially the shear strength requirement, it can be concluded that this particular lift is adequate for the safety and functionality of the dam and will not need to be removed.

Finding 6: degree of compaction achieved in tested Zone 1 lift material

An 88.9% compaction was achieved by compacting the soil at a moisture content of 47.2%, +9.2% of OMC. It was considered to be a low value since it is below 90%. However, the shear strength requirement and the other specified requirements were met. Also, the percent compactions resulting from test fill number 7 all ranged approximately between 87 % & 98% and most of the samples had achieved satisfactory shear strengths and other adequate specified properties. Based on these circumstances, it was concluded that the dry density, and the percent compaction of the tested lift, was adequate for Zone 1 to be functional.

Analysis

The sand cone test was performed to calculate the dry density of a compacted lift of Zone 1. The resulting dry density was used, along with the maximum dry density value determined with the proctor compaction curve, to calculate the percent compaction achieved in the field. The dry density of the soil was found to be 1172.2 kg/m³ while the maximum dry density was 1318 kg/m³. The percent compaction achieved in the soil is $100\% \times (1172.2) / 1318 = 88.9\%$. The percent compaction to be achieved is not defined in the design specifications or in the personally developed specifications, so it was not trivial to determine whether 88.9% compaction was an adequate value. It was acknowledged as a low value since it is below 90%, yet it can be recognized as adequate. The percent compactions that resulted from the test fill number 7 all ranged approximately between 87 % & 98%, and most of the tested samples had achieved satisfactory shear strengths.

Below is a generated graph that displays the proctor compaction curve, done with the same five values of moisture content and dry density used in the laboratory compaction test. The point of maximum dry density and optimum moisture content are in red, and the 100% and 95% saturation curves are in yellow and green respectively. On the same graph the point corresponding to the value of moisture content and dry density determined with the sand cone test was also inserted, in green. The vertical purple lines display the compaction water content range of +2% & +12% of OMC.

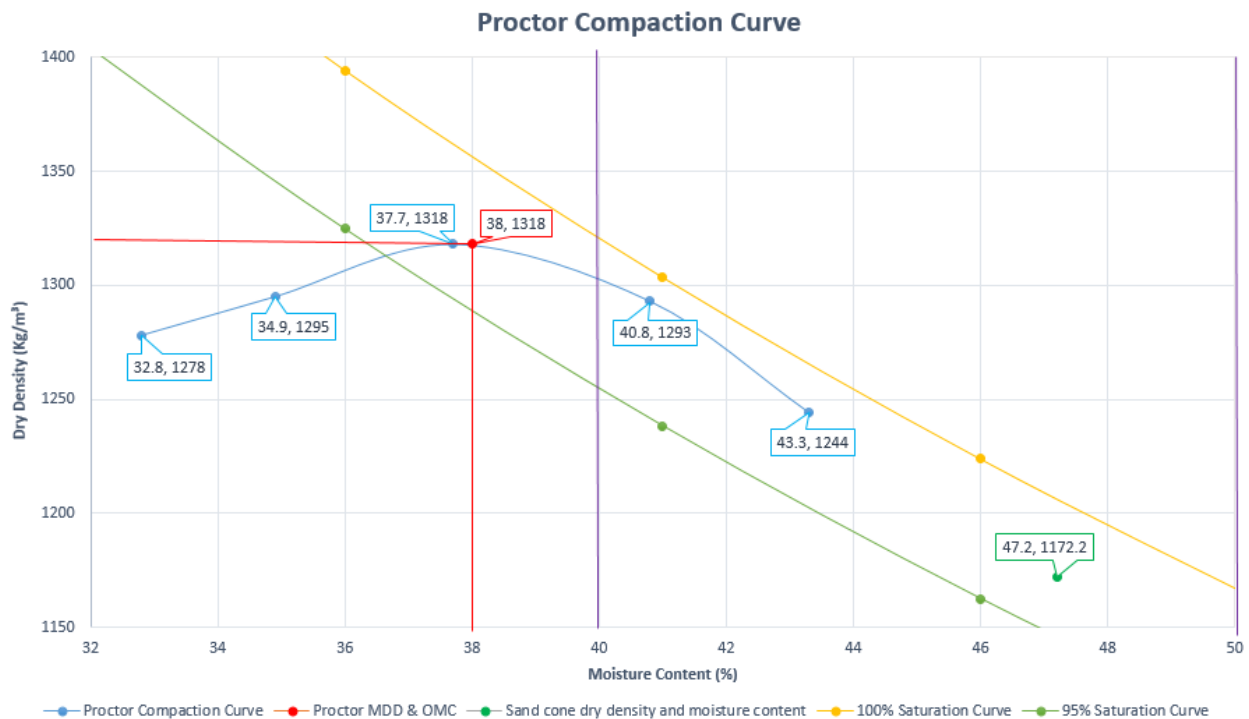


Figure 41: Proctor Compaction Curve with Point Showing the Dry Density Achieved In the Compacted Soil Lift

It can visually be observed that the compacted soil sample is below the 100% saturation curve which is accurate since 100% saturation is almost impossible to achieve. The sample is above the 95% saturation curve meaning that the sample has less than 5% air voids. The moisture content of the sample is around 9% above optimum. This graph helps visually display the difference between the sample's moisture content and the optimum, and the difference between the maximum dry density and the dry density achieved in field after compaction. The dry density of the tested lift, which was compacted way above the OMC, is much lower than the dry densities achieved with samples compacted near the OMC.

The graph below displays the percent compactions of the samples used to generate the laboratory proctor compaction curve and the percent compaction of the tested lift, against their respective percent water

contents. The values of compaction and water content of the test fill number 7 were also included. The three red lines display the 98%, 95% & 90% compaction marks.

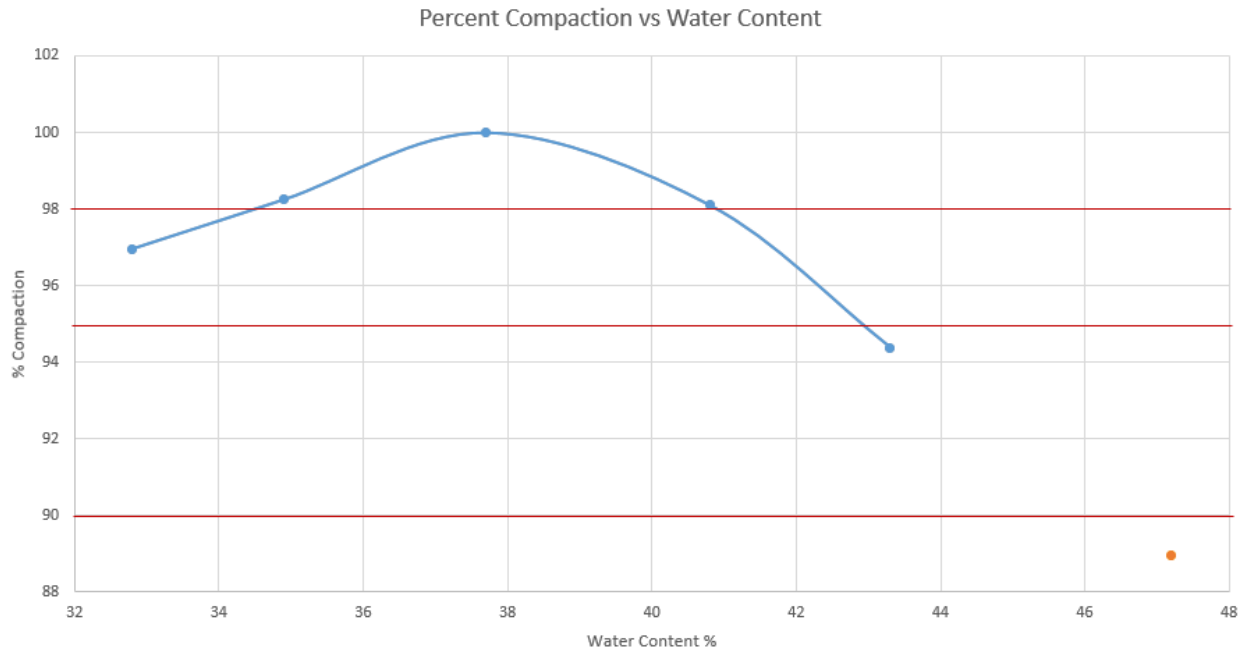


Figure 42: Percent Compaction Vs Water Content Of The Soil Samples Used To Generate The Laboratory Compaction Curve And Of The Tested Sample In The Field, With The Sand Cone Test.

The differences between the percent compactions achieved at +2% to +4% and -2% to -4% of OMC, and the one achieved in the field can be visually seen. The percent compaction at optimum moisture content is 100%. The percent compaction starts to decrease as the percent water content increases and as the percent water content decreases, from the optimum. It can be observed that compaction done way above OMC yields a much lower percent compaction, with respect to the soil compacted near OMC. An 88.9% compaction was achieved by compacting the soil at a moisture content of 47.2%, +9.2% of OMC.

Analysis of correlation between dry density and undrained shear strength

The graphs below display the correlation between the resulting dry density and the undrained shear strength, and the percent compaction and undrained shear strength, of the compacted clay core compared to the results of test fill number 7.

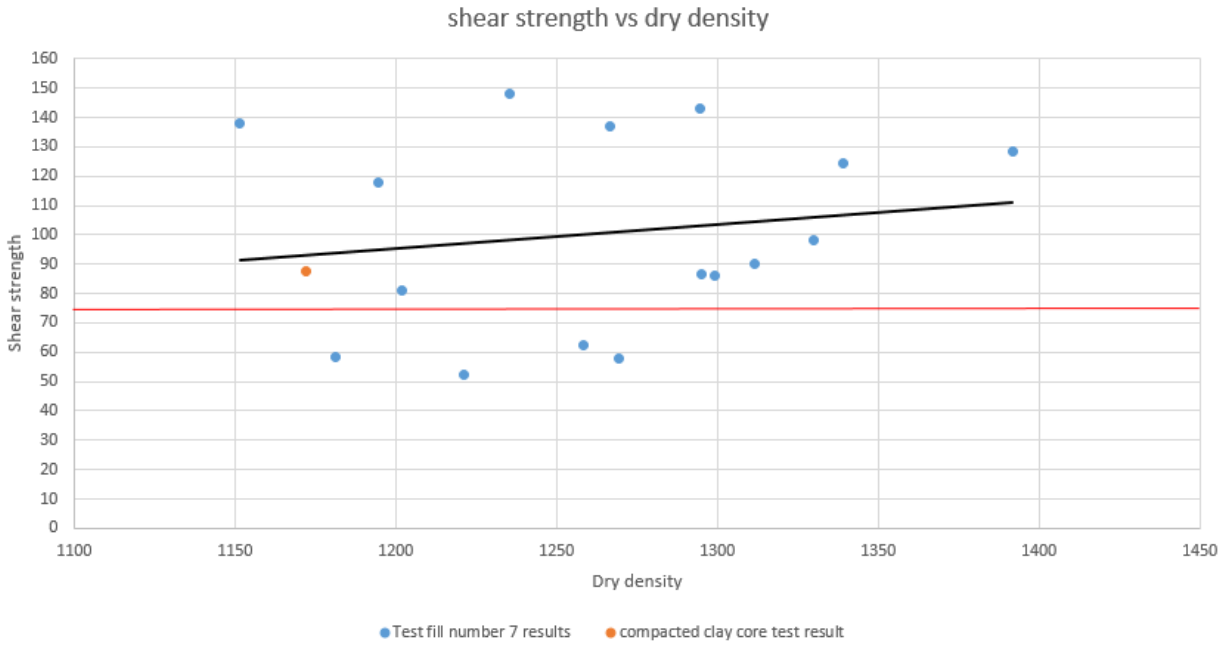


Figure 43: Correlation between Shear Strength and Dry Density of Test Fill Number 7 Results and Compacted Clay Core Test Results

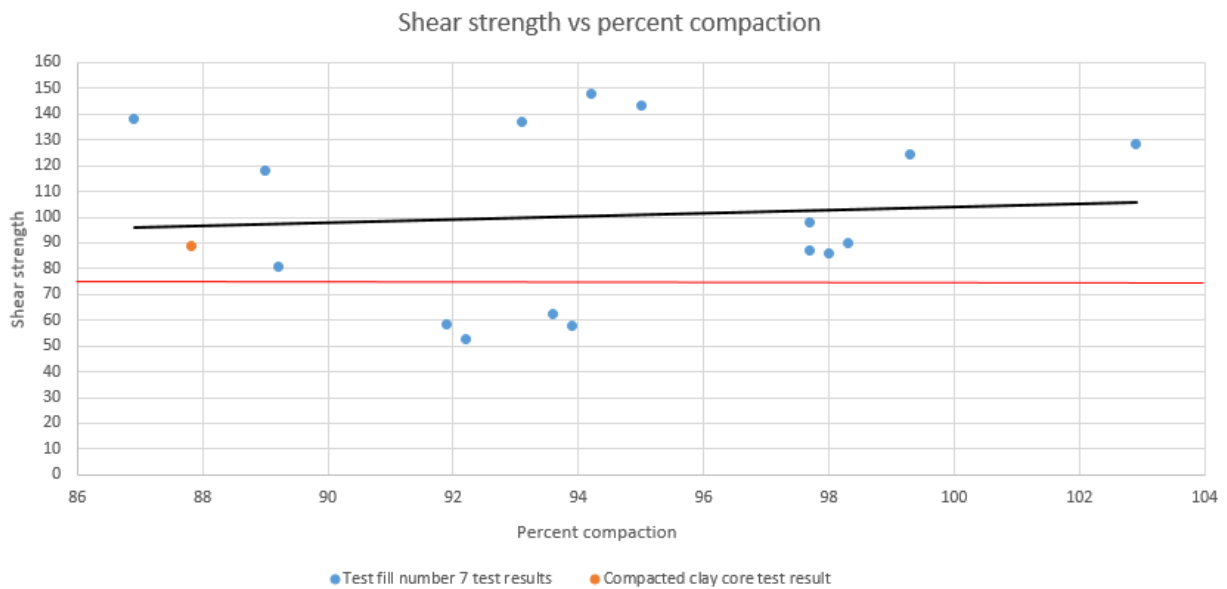


Figure 44: Correlation between Shear Strength and Percent Compaction of Test Fill Number 7 Results and Compacted Clay Core Test Results

It can be seen that the specified shear strength requirement was met and that the dry density of the soil in the clay core of the dam had lower values compared to almost all of the outcomes of test fill number 7. The dry density was even lower than the values of the four samples that failed in shear strength in the

test fill. The same can be observed when the percent compaction and the shear strength values are compared. The percent compaction was relatively lower compared to almost all the results from test fill number 7. However, since the shear strength requirement was met, along with the other specified requirements, it can be concluded that the dry density and the percent compaction of the placed and compacted soil were adequate values for Zone 1 to be functional. In conclusion, the clay core will be adequately impermeable, have sufficient strength, be sufficiently ductile and flexible and have an adequate low compressibility.

Recommendations

From observing and supervising the Zone 1 construction procedures, it was recommended that constant supervision of the placing, trimming, diskings, leveling, compacting, sealing of Zone 1 be done to ensure that the specifications are being followed correctly.

From going to the laboratory and supervising the tests being performed, it was determined that precisely following the ASTM test procedure is vital for the outcome of correct test results. It was recommended that supervision of laboratory testing procedures is done occasionally, especially when the outcomes of the test results are suspicious. It is key to always critically analyze laboratory test results and never blindly trust them or take them for granted. It was recommended to constantly check whether test results make sense, since a wrong test result could go by undetected. If this were to happen, specifications could possibly not be met and the dam's future functionality could be compromised.

From the supervisions of the quality control testing procedures, it was deduced that testing for strength with the vane shear test is more efficient than testing for density with the sand cone test. Testing for strength with the vane shear test was determined to be the preferable choice as the primary quality control testing method. Alternatively, it was recommended to test the dry density of the compacted soil and calculate the percent compaction with the sand cone test every 10,000m³ and whenever a field inspector considers it to be required, as a check to determine whether adequate compaction is being achieved. This was recommended due to the placement of the soil at moisture contents high above optimum levels that could compromise the soils engineering property performances.

The analysis performed on the test results of Zone 1 determined that all of the project's specifications were met. On the other hand not all of the personally developed project specifications were met. The percent water content placement range requirement, of the personally developed specifications, failed. The soil's calculated moisture content resulted being +8.8% of OMC, 0.8% above the specified requirement. However, since all the other test results complied with the specifications, including the shear

strength requirement, it was reasoned that the functionalities of this particular tested lift were still adequate for the safety and functionality of the dam. This result meant a removal of the tested lift was not needed.

Analysis was done on the values of dry density and percent compaction of the Zone 1 lift found by performing the sand cone test. It was established that the values were lower compared to many of the tested samples of test fill number 7. Since there is no specified percent compaction requirement in neither the project specifications, nor the personally developed specifications, it was difficult to deduce whether the 88.9% achieved percent compaction of Zone 1 was adequate. However, since the shear strength requirement was met along with the other requirements, and since the test fill number 7 percent compactions ranged between 87 % & 98% and most samples had achieved satisfactory shear strengths, it was established that the dry density and the percent compaction of the compacted Zone 1 lift were adequate. It was important to note that even though achieving a percent compaction value lower than 90% is unusual for most embankment dam projects, it is still adequate for the clay core of Borinquen Dam 1E. Zone 1 will usually still be impermeable, have sufficient strength, be sufficiently ductile, maintain flexibility, and have an adequately low compressibility, even though it achieves percent compaction values slightly lower than 90%, as long as it achieves the specified values for the other test results, especially shear strength.

The personally developed specifications for Zone 1 were more restrictive compared to the actual project specifications. It was concluded that the more restrictive specifications did not improve the dam's safety and functionality. The specifications might only make the compaction procedure more challenging and prolong the overall project construction time; mainly due to materials having inadequate moisture contents that would not allow them to be placed in the fill. Another reason is due to the higher possibility of losing time by removing lifts that would fail to meet the specifications. Therefore, the personally developed specifications were not recommended and it was inferred that the current project specifications are adequate.

Conclusion

The objectives were achieved and the outcome of the project was successful.

It was determined that the central core earth and rockfill embankment dam structure, which is being used to construct Borinquen dam 1E, is the best possible option for the projects circumstances. It was determined that the compaction requirements and testing specifications for quality control are accurate for the construction of Borinquen dam 1E. Furthermore, they are more suitable than the more restrictive personally developed specifications. It was determined that the compaction being achieved In Zone 1, meets the project's specifications and is satisfactory for the outcome of the core's functionalities.

It was recommended that the Zone 1 field compaction processes and the laboratory test procedures be carefully supervised. It was recommended that lab test results be precisely and critically evaluated before their approval. It was recommended to do the sand cone density test more often than what is currently being done, in order to test the validity of the vane shear strength result, to confirm that adequate compaction is being achieved and to know for certain that the outcome of the core's functionalities will be achieved.

Borinquen Dam 1E Construction Management Process Analysis

Introduction

The purpose of this study was to analyze the construction of the Borinquen Dam 1E in order to better understand the limiting factors to rate of construction, develop recommendations with which to optimize the construction process, and predict the time of completion. In the field, the excavation, processing, and laying of materials including rockfill, backfill, the clay core, and the filter layers were timed, and the hauling route observed. The following report presents the results of the field observations, analysis, and potential operational improvements. The analysis will include potential improvements to be incorporated into a redesign of the route as well as a cost analysis. Optimization methods include route length and quality, equipment scheduling, as well as additional excavation and construction fronts, and increased working days. These recommendations will be presented to the project management team in order to increase rate of construction and ensure the construction is completed on time and in the most cost efficient manner.

Background

Construction Process Models

Construction management is a balance of time, quality, and cost. Thorough and early planning is the most effective technique in controlling the balance of cost and schedule of a project while ensuring a quality product. In order to develop an adequate plan, significant testing and surveys must be completed to optimize predictability. With adequate planning, variations may be reduced which can lead to significant cost savings (Cooper, 2008). There are two basic scheduling plans that can be utilized in the construction operation, the RIBA Plan of Work, and the British Property Federation model.

The RIBA Plan of Work was developed in 1964 as a standard method for construction operations. It is generally applied to buildings because it is not generic enough for many implementations. The RIBA Plan organizes the operation into a logical sequence as seen in the figure below. This plan ensures that

decisions are made in an effective and timely fashion throughout the course of the project. This project model anticipates adjustments based on the size and complexity of the project.

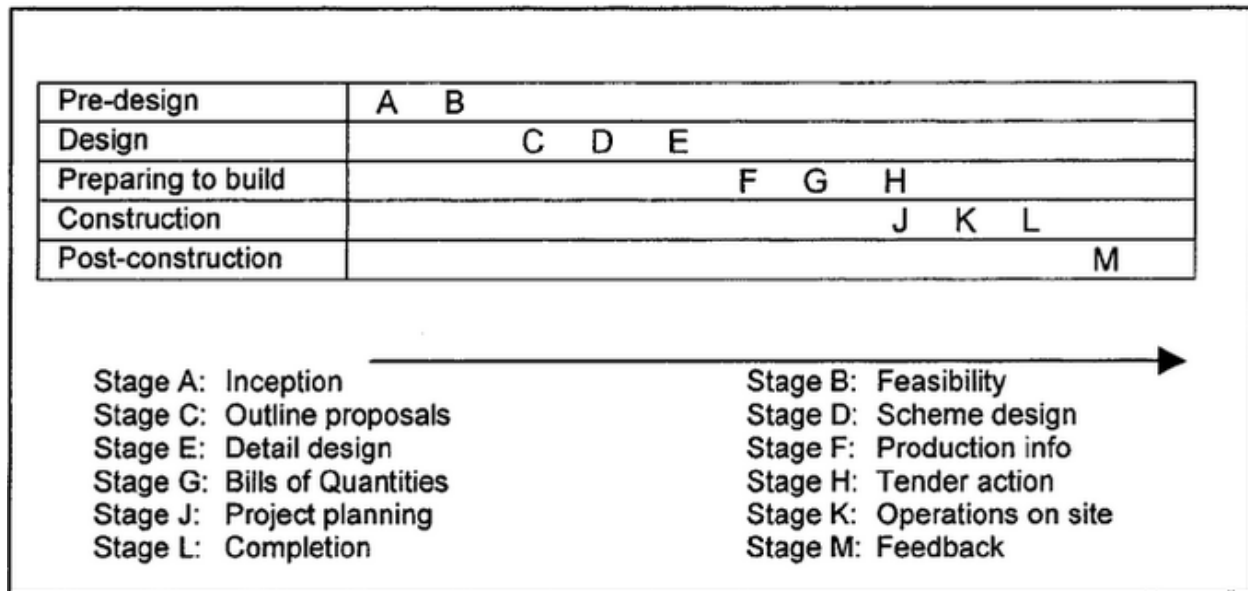


Figure 45: RIBA Plan of Work (Cooper, 2008)

The project must progress sequentially from A to M, one stage before the next, however the design and construction is not essentially linear. This method can have deficiencies for this reason. The sequential divides may cause breaks in communication and coordination between the organization, design, industry, and construction bodies.

The British Property Federation (BPF) model developed to solve the inadequacies of previous operational designs which barred communication and is superior to the RIBA model on this front. The model contains all parties involved in the construction project including but not limited to the client, design consultants, contactors, subcontractors, and suppliers. Both formal and informal relationships among each party are highlighted to facilitate stronger and more candid communication. In addition, the value for cost is more translucent for the client and the client has the opportunity to decide whether to continue the project at the end of each stage. The five flexible stages include the concept, preparation of the brief, design development, tender documentation and tendering, and construction. This model tends to be superior in that it produces structures faster and more cost efficiently. It also removes overlap of effort between design team members. Because more preparation is put into the plan, there are fewer variations on site, therefore developing a superior, less expensive product faster. (Cooper, 2008)

Primavera Project Planner

Primavera Enterprise has a number of programs that utilize the concepts developed through RIBA and BPF planning methods. Primavera Project Planner is a multi-project planning, scheduling, and control software; Progress Reporter is an online software used for distributing assignments, and updating progress; Portfolio Analyst is used for portfolio analysis; and Primavision is used for developing an executive summary of the current project progress and portfolios.

Primavera is an effective tool in planning projects that may be large scale, long term, and large budget. It is the current leading provider for project management service (Business Editors).

Design and Construction Coordination

A cost efficient project will ensure design and construction coordination to optimize communication and thus predictability. This coordination can only be developed in a truly co-operative project environment which facilitates teamwork and communication and effective use of technologies (Cooper, 2008).

Therefore, although the design of the Earth Dam has been completed long before construction begins, coordination between the designing contractor and construction personnel is necessary throughout the process to ensure that the design intent is fulfilled despite any unforeseen obstacles. Throughout construction, the structure must be assessed and any adaptations incorporated into a constantly evolving design. This can be accomplished through instructions to construction engineers in a preconstruction orientation, in addition to regular on site visits, tests and reporting by the designers (Mcmahon, 2004). At the Borinquen Dam, both the contractor and design team perform regular testing in addition to the client, the ACP to not only ensure the Dam is being constructed to the intent of the design, but also that construction is occurring on efficiently schedule.

Pre-Construction Orientation

The designing engineers must prepare a report for the field personnel including the construction engineers. The U.S. Army Corps of Engineers suggests the title of "Engineering Considerations and Instructions for Field Personnel," but a similar title may be used for this project. This report shall explain the design based on assumed site conditions such that field personnel can be best prepared to identify any changes in the field that may be directly observed once excavation has begun especially in critical sections of the embankment. This report shall additionally explain all necessary quality assurance testing instructions and "be augmented by appropriate briefings, instructional sessions, and laboratory testing sessions (ER 1110-2-1150)" (Mcmahon, 2004). This report shall be used to lead a preconstruction

orientation by the designing engineers for the construction engineers during which all concepts, intents, and assumptions of field conditions will be presented.

On-Site Visits

During the construction process, designing engineers should perform on-site visits at several important milestones to ensure the following:

“(1) Site conditions throughout the construction period are in conformance with design assumptions and principles as well as contract P&S.

(2) Project personnel are given assistance in adapting project designs to actual site conditions as they are revealed during construction.

(3) Any engineering problems not fully assessed in the original design are observed, evaluated, and appropriate action taken” (McMahon, 2005).

Additional visits may be necessary by design engineers when cut off trenches, foundations, abutments for dams and appurtenant structures, tunnels, or borrow areas are excavated, materials are placed for the dam embankment early in construction, or any field conditions appear that are significantly different from those assumed in the design in accordance with ER 1110-2-112 (McMahon, 2004).

Owner and Contractor Coordination

In order to expedite construction and avoid obstacles arising from communication errors, a partnership may be developed between the owner and contractor external from the actual construction contract. Prior to beginning construction, the owner may take steps to recognize common interests in order to establish an open, trusting atmosphere. Through increased familiarity, and understanding of mutually beneficial goals, the partnership may promote more candid communication and cooperation. The owner should personally contact the contractor in order to establish a new relationship. Both parties should then establish joint goals and detailed steps with which to achieve those goals. Both parties should take time to foresee any delineation from these objectives, consider processes with which to avoid such problems in the future, evaluate performance, and promote cooperation throughout construction (McMahon, 2004).

Construction Considerations

Several aspects of construction may impact the quality and efficiency of the operation including but not limited to the following: equipment, hauling roads, construction schedule, and wage rates.

Equipment

The first considerations for the construction of a small earth dam are the plant and equipment necessary for the operation. Important factors in selecting equipment include hauling distances and conditions, quantity of material to be hauled, and time and expense budgets (Stephens, 2010).

Excavators

Two models of Caterpillars Hydraulic Excavators were used for the Borinquen Dam. Excavators' capacities range based on the size of the bucket attached to the boom stick. This can be adjusted based on material or environmental conditions. The below figure shows the schematics of an excavator model 336D.

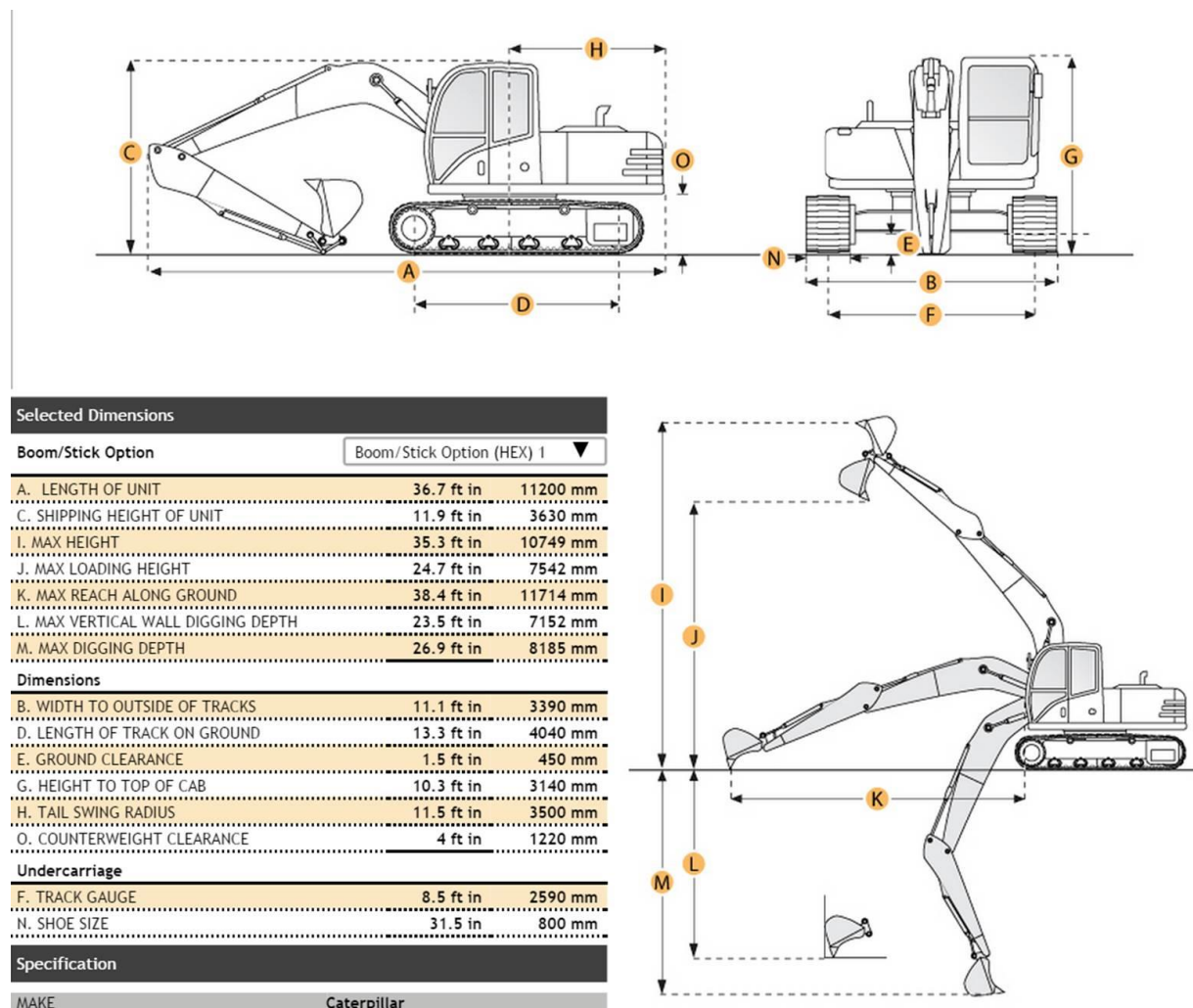
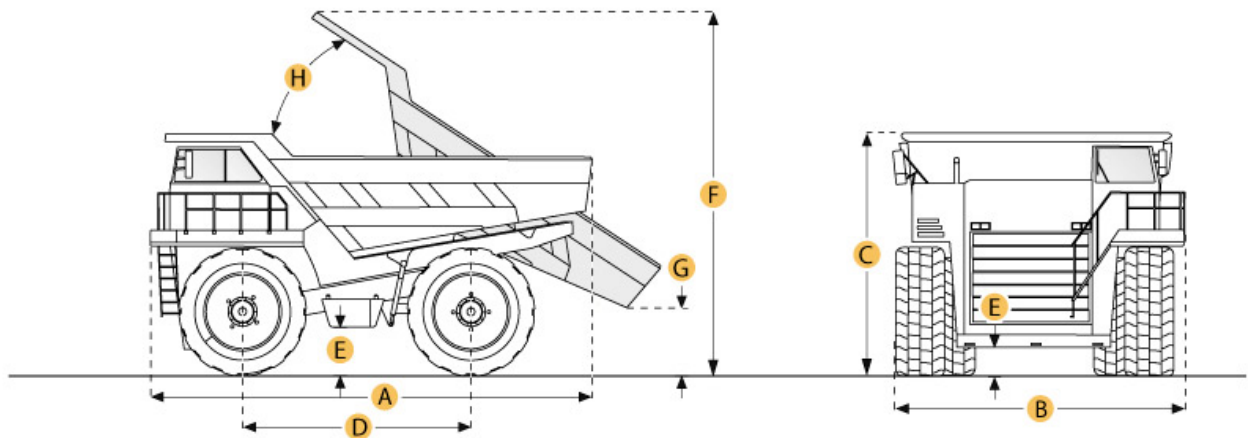


Figure 46: Excavator 336 D Schematic ("CATERPILLAR 336D L HYDRAULIC EXCAVATOR", 2007)

Hauling Vehicles

In order to transport material from the excavation site to the stock pile for treatment, then to the embankment for construction, hauling trucks are necessary. The selection of the trucks depends on the route environment, and can control the efficiency of a project. The larger the hauling truck, the larger load it can move and the heavier the vehicle. Off road routes must be built for most large hauling vehicles because they are beyond the allowable weight for highways. The following vehicles are the three types used in the construction of the Borinquen Dam: rock movers, articulated trucks, and small trucks.



Selected Dimensions

Dimensions

A. OVERALL LENGTH	28.6 ft in	8712 mm
B. OVERALL WIDTH	12.5 ft in	3800 mm
C. OVERALL HEIGHT	13.8 ft in	4200 mm
D. WHEELBASE	13 ft in	3962 mm
F. DUMP HEIGHT	27.2 ft in	8300 mm
G. DUMP GROUND CLEARANCE	11.3 ft in	3450 mm

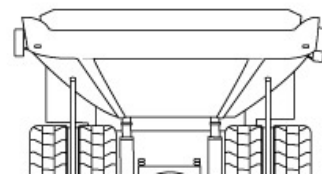
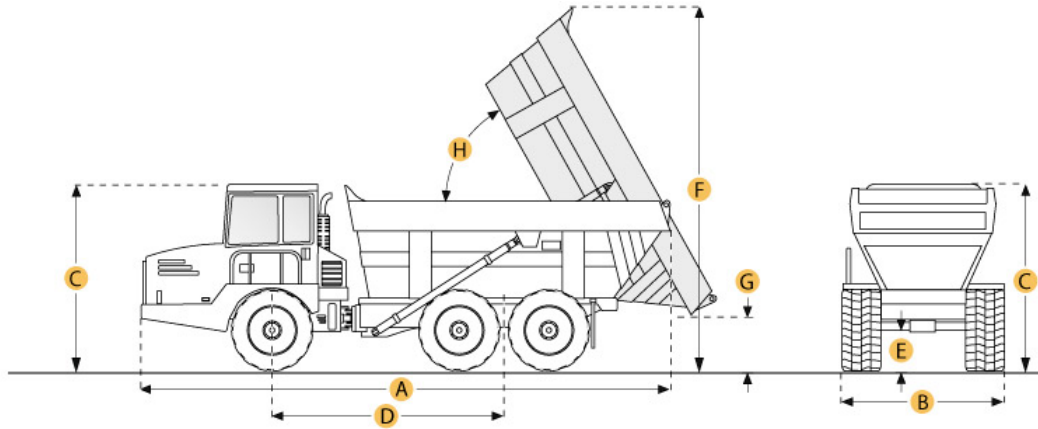


Figure 47: 773 Rock Mover Schematic ("CATERPILLAR 773 ROCK MOVER", 2007)

Pictured above is the Caterpillar model 773 Rock Mover. This is an enormous, off road only vehicle that has a capacity of 41 cubic yards. This truck is primarily used for hauling rockfill from the stock pile and blasting source to the embankment. It carries the most material most efficiently, but is much heavier at approximately 83,000kg loaded than the 740 with only approximately 46,000kg loaded. It also has a larger turning radius than articulated trucks and smaller trucks.



Selected Dimensions

Dimensions

A. OVERALL LENGTH	35.7 ft in	10889 mm
B. OVERALL WIDTH	11.3 ft in	3430 mm
C. OVERALL HEIGHT	12.3 ft in	3745 mm
D. WHEELBASE	17.1 ft in	5227 mm
E. GROUND CLEARANCE	1.9 ft in	577 mm
F. DUMP HEIGHT	23.2 ft in	7068 mm
G. DUMP GROUND CLEARANCE	2.3 ft in	697 mm

Dump

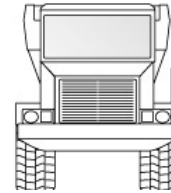


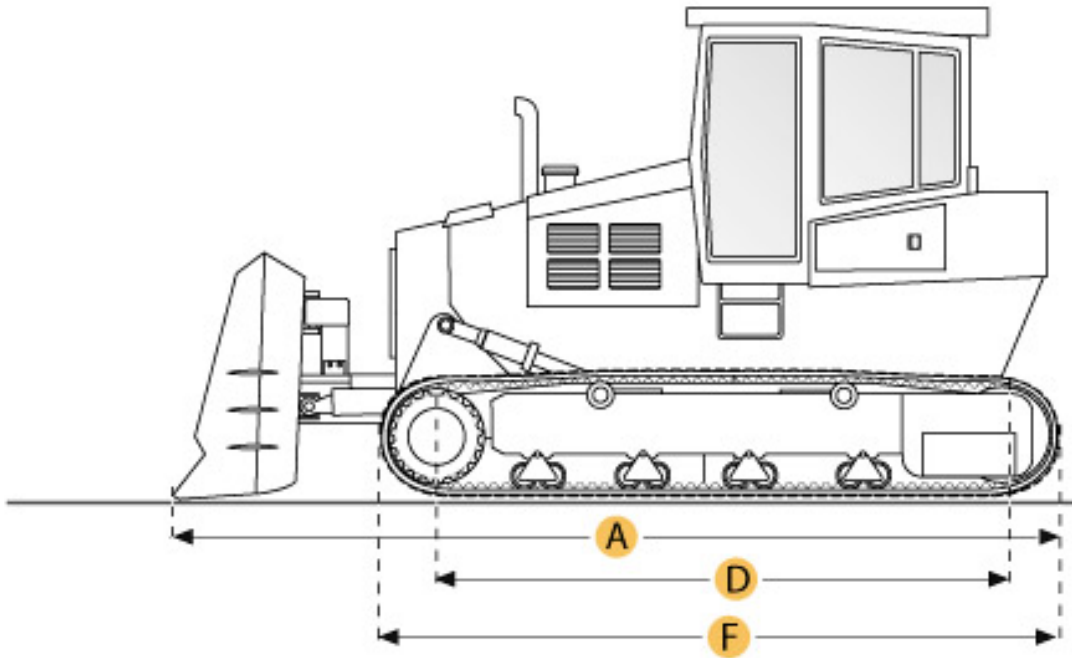
Figure 48: 740 Articulated Dump Truck Schematic ("CATERPILLAR 740 ARTICULATED DUMP TRUCK", 2007)

Pictured above is the Caterpillar model 740 Articulated Dump Truck. This is an off road only vehicle that has a capacity of 30 cubic yards. This truck is primarily used for hauling zone 1 material because it has more maneuverability than a 773 Rock Mover and is more equipped for the terrain of the current hauling route.

Caterpillar has several options for smaller trucks and their schematics range significantly. However, the key factor that makes small truck as a necessity is that they can travel on state and federal roads.

Earth Movers

Earth movers are essential to construction because they manipulate the materials at the stock pile and at the embankment such that the material can be effectively processed or shaped for construction. Bull dozer models D6 and D8 were the two types of earth movers used on the Borinquen Dam. Their schematics are shown in the Figures 49 and 50.



Selected Dimensions

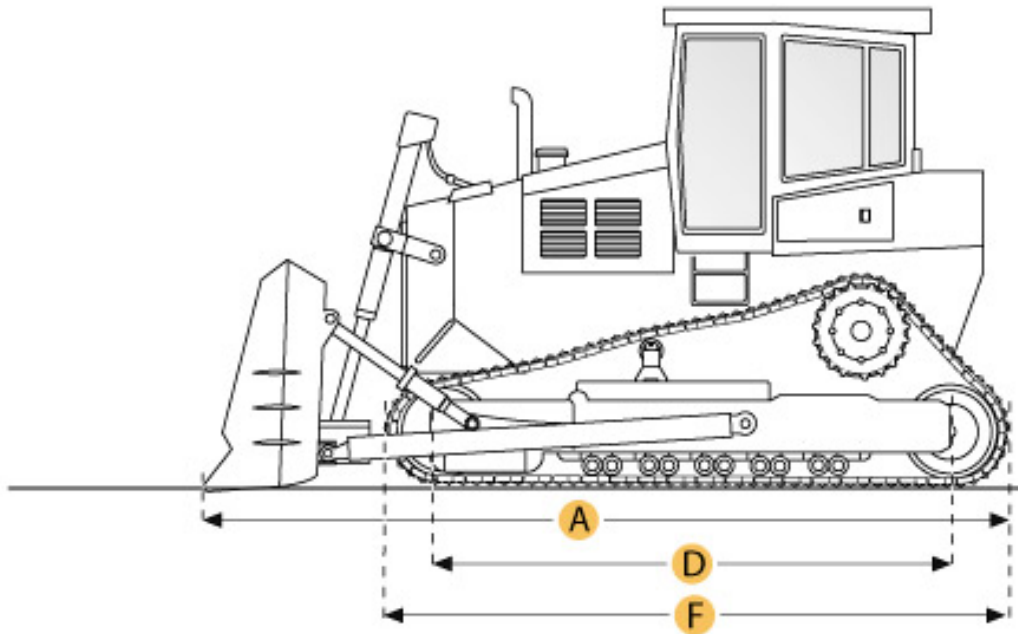
Dimensions

B. WIDTH OVER TRACKS	6.5 ft in	1980 mm
C. HEIGHT TO TOP OF CAB	9.5 ft in	2900 mm
D. LENGTH OF TRACK ON GROUND	6 ft in	1830 mm
E. GROUND CLEARANCE	1.2 ft in	360 mm
F. LENGTH W/O BLADE	10.5 ft in	3200 mm

Undercarriage

G. TRACK GAUGE	5 ft in	1520 mm
H. STANDARD SHOE SIZE	16 in	406 mm

Figure 49: D6 Dozer Specification ("CATERPILLAR D6 EARTH MOVER", 2007)



Selected Dimensions

Dimensions		
A. LENGTH W/ BLADE	20.4 ft in	6220 mm
B. WIDTH OVER TRACKS	9.3 ft in	2840 mm
C. HEIGHT TO TOP OF CAB	12.7 ft in	3870 mm
D. LENGTH OF TRACK ON GROUND	10.6 ft in	3220 mm
E. GROUND CLEARANCE	1.5 ft in	456 mm
F. LENGTH W/O BLADE	16.2 ft in	4950 mm
Undercarriage		
G. TRACK GAUGE	7.5 ft in	2280 mm
H. STANDARD SHOE SIZE	22 in	560 mm

Figure 50: D8 Dozer Specifications ("CATERPILLAR D8 EARTH MOVER", 2007)

The D6 Dozer is smaller than its D8 counterpart with a capacity of 3.3m³. The D8 dozer has a capacity of 11m³ and can be much more efficient, although it has less versatility due to its larger size and weight.

Compactors

Compaction is the essential final component in increasing the shear strength of the earth dam and increase long term stability. Several methods for compaction may be employed for small dams depending on the expected load. Smaller dams that are low budget and not time sensitive can achieve a high level

of compaction using livestock. Similarly, zones that do not require a precise level of compaction may be compacted to a significant degree under the load of hauling trucks and bull dozers. The different zones of the dam require varying levels of compaction and therefore different methods may be used for each zone. Zones that require higher level of compaction may use rollers (Stephens, 2010). Several types of rollers may be used for compaction including sheepsfoot rollers, vibrating rollers, rammers and plates, and smooth wheeled rollers. Sheepsfoot rollers are superior to other rollers in compacting dry clay and can compact up to 200mm of loose soil with 6-12 passes at a speed of 3-6km/h. This type of compactor needs to be cleaned regularly to ensure that it is effectively churning the soil and distributing water through the construction surface.



Figure 51: Sheepsfoot Roller (Sheepsfoot Roller, 2014)

Vibrating rollers are most suited for the compaction of sandy soils or where significant compaction is required for a resulting high density. They are also very useful in small scale work such as cutoff compaction, or trench work.



Figure 52: Full Hydraulic Vibrating Roller (Vibrating Roller, 2014)

Rammers and Plates are used for the same purpose as vibrating rollers. They can also be used in small scale projects with limited space for maneuverability and in special projects such as trench work, and around delicate areas such as concrete structures, or pipe work.



Figure 53: Wacker Neuson Plates & Rammer (Plates & Rammer, 2014)

The Smooth Wheeled Rollers are most suited for compacting lower layers and reducing air-spaces. This roller requires less passes at the same rate as other rollers, although its size and weight can make it less versatile.



Figure 54: Smooth Wheeled Roller (Smooth Wheeled Roller, 2014)

The Sheepsfoot roller is the most well suited roller for clay compaction as smooth wheeled rollers may create seepage paths between clay layers. If a sheepsfoot roller is not available, the dam should be built in much thinner lifts to ensure good bonding during compaction (Stephens, 2010). Sheepsfoot and vibrating rollers as well as manual rammers are used for various applications on the dam.

Rental Equipment

Construction equipment is expensive equity for a contracting business, and rentals can be a better option. Purchasing and owning equipment is expensive upfront and may incur expensive maintenance. The contractor needs to have a large company with regular work in order to afford the expense without losing capital while the equipment is not being used. Additionally, because equipment is highly specialized for specific types of jobs in terms of size, weight, capacity, or specialty sensors or extensions the contractor may need a diverse fleet in order to ensure regular use of the equipment. Renting equipment gives the contractor more versatility with low up front cost. Renting equipment also give the contractor more flexibility to increase or decrease the fleet at various points in construction. A set rental rate also helps contractors prepare bids and make cost projections. The contractor also avoids storage and transportation costs and can rent the newest, most effective models.

The contractor at the Borinquen Dam is renting excavation, hauling, dozing, compaction, water, and several other pieces of equipment. The primary equipment used for each zone and at each stage of construction is summarized below with the associated rental rates.

Zone	Activity	# Fronts	Equipment	#	Rental Fee	
					\$/Day	\$/Month
1/ 1A	Source	2	CAT Excavator 336	2	\$60.00	\$480.00
	Stock Pile	2	Water Truck	2	\$66.73	\$533.80
			CAT Dozer D6	2	\$40.00	\$320.00
			Vibrating Compactor Hamm 3520	1	\$32.00	\$256.00
	Hauling	2	CAT 740	10	\$75.00	\$600.00
	Embankment	3	Water Truck	3	\$66.73	\$533.80
			Vibrating Compactor Hamm 3520	3	\$32.00	\$256.00
			CAT Dozer D6	6	\$40.00	\$320.00
			Manual Compactor	12	\$4.00	\$32.00
						\$1,374.65
3	Hauling	4	CAT 773	16	\$85.95	\$687.60
			CAT 740	3	\$75.00	\$600.00
	Embankment	4	CAT Dozer D8	6	\$75.00	\$600.00
			Vibrating Compactor Hamm 3520	6	\$32.00	\$256.00
			CAT Backhoe 416	3	\$27.00	\$216.00
			Water Truck	3	\$66.73	\$533.80
						\$2,328.39
4	Hauling	4	CAT 773	16	\$85.95	\$687.60
			CAT 740	3	\$75.00	\$600.00
	Embankment	1	CAT Excavator 336	1	\$60.00	\$480.00
			CAT Dozer D8	1	\$75.00	\$600.00
						\$1,735.20

3B, 3A, 5,6	Hauling	4	Small Trucks	27	\$10.00	\$80.00
	Embankment	3	Vibrating Compactor Hamm 3520	3	\$32.00	\$256.00
			Water Truck	3	\$66.73	\$533.80
			Manual Compactor	15	\$4.00	\$32.00
					\$431.19	\$9,143.87
Total					\$5,869.43	\$361,187.19

Wage Rates

In Panama, the wage laws enforced under the Constitution of Panama favor the worker. The laborer has the following basic rights:

1. “The right to a minimum salary, and to periodic adjustment thereof;
2. the right to nondiscrimination and to equal jobs;
3. the right to form unions and to strike;
4. the right to limit the daily work shift to a maximum of 8 hours and the weekly work schedule to a maximum of 48 hours;
5. maternity protections for the female worker; and
6. the right not to be dismissed from employment without just cause” (Arauz de Grimaldo, 2012).

These rights are the minimum protection ensured to workers.

Cost Saving Techniques

Several cost saving techniques may be used in order to increase value efficiency from design through construction oversight.

Cost Optimization in Design

In order to ensure the most cost effective design, the U.S. Army Corps of Engineers suggests a technique called “Value Engineering” which suggests that a multidiscipline engineering team design alternate design for several portions of the project. These alternate designs will be compared to those of the design teams for immediate and long term technical and cost efficiency (Mcmahon, 2004).

Location and Availability of Materials

Zones should be designed in the embankment in order to optimize the amount of locally sourced materials from required excavation and borrow areas with the shortest hauling distance for the highest cost

efficiency and least waste. It is also highly economical to consider a zone for which indeterminate soils can be used since considerable amounts will most likely be removed from excavation sites. This material can be used as a downstream drainage zone called a Random Zone of the embankment to control seepage. A transition zone may be necessary between the pervious zones and the random zone (McMahon, 2004).

Methodology

The goal of this section of the project was to develop a process analysis of the construction of the Borinquen Dam 1E. This process analysis investigated key points in the operation including the excavation, processing, hauling, and construction of each zone of the Dam. From this analysis, potential points for improving production and cost efficiency were identified. In order to accomplish this goal, several objectives were developed including the following:

1. Gain an understanding of the construction process and identify central crutches to productivity;
2. Gain metrics on the current construction process at the identified crutches;
3. Analyze the efficiency of the current process and develop recommendations for the contractor at several key points including:
 - a. Excavation,
 - b. Stock Piling,
 - c. Construction, and
 - d. Hauling;
4. Project the time of completion and the associated cost at the current rate of construction and after recommendations.

Prior to beginning the methods associated with these objectives, an introduction to the Borinquen Dam 1E project was given including a tour of the active construction of the embankment, the material stockpiling, and the excavation from the sources. Additionally, significant research into the design of the dam, equipment, scheduling practices, and other construction management tools were researched. The researched information in addition to supportive information provided by ACP employees were used to develop a frame for the process analysis. Using that frame, field work and interviews could be effectively used to maximize resources and achieve the following objectives.

Objective 1: Gain an understanding of the Construction Process and Identify Central Crutches to Productivity

The first objective was to gain an understanding of the construction process in order to focus the study. This objective was completed by analyzing the process via on site tours and interviews.

Pedro Lopez, the head of the safety and quality department, is the supervisor of this project and delegated to several quality assurance field engineers. Christian Cáceres is the quality engineer focused on the source excavation of the residual soil for zone 1; Eduardo Epifanio is the quality engineer focused on the

construction of the embankment; and Eduardo Chui is the quality engineer focused on the processing of material for the filter layers of the embankment. Several emails were prepared and sent to each of the engineers regarding the contractor’s submitted schedule and the actual excavation, processing, and placement of material on the dam in order to gain a better understanding of the process. Although the primavera files were confidential, the excel files Eduardo Epifanio, Eduardo Chui, and Christian Cáceres had developed were provided.

After having gained insight on the operation in general, two main zones were identified as central to the progress of construction: the excavation of zone 1 residual soil and the placement of zone 3 rockfill. This is further discussed in the findings of this report. The emphasis of this study was focused on these two zones.

Objective 2: Metrics on the Current Construction Process

The construction process of zone 1 excavation and zone 3 construction was observed from several vantage points at each of the following checkpoints: excavation, processing, and placement. The points are identified on the map below. The black stars signify observation points for rockfill and the white points signify observation points for residual soil.

Figure 55: Observational Vantage Points



Having a general understanding of the process, a table was developed to regularly examine the process with the following column headers:

Truck #	Arrival Time	Dumping Initiation	Comments
---------	--------------	--------------------	----------

In addition, the date, shift, starting and ending time, equipment model information, weather conditions, and route observations were recorded.

The construction equipment was timed down to the second using a stop watch. The field work was performed from vantage point a safe distance from moving vehicles and with a clear view of the equipment.

This information was recorded during both the morning and afternoon shifts and on several dates at each location to ensure adequate replication. Using this information, the total cycle time of each truck could be averaged by recording the truck at initial arrival time and each subsequent arrival time.

$$\overline{Cycle} = \frac{T_n - T_{n-1}}{n}$$

Additionally, the time spent in cue could be recorded by subtracting the time at which the truck arrived from the when it began to dump or fill depending on the location.

$$T_{waiting} = T_{Initiate Dumping} - T_{Arrival}$$

Objective 3: Analyze the Efficiency of the Current Process and develop Recommendations for the Contractor

The efficiency of the operation can be judged at several points including, but not limited to, equipment selection, scheduling of equipment and laborers, and route length and quality.

The efficiency in light of route length and quality was identified using an aerial map of the site and driving the routes with a quality engineer. Observations on the quality of the road were taken and the route identified on the map for several of the cycles. Additionally, note was taken as to more efficient potential routes and poor quality of routes.

The efficiency in light of scheduling was calculated using the waiting time.

$$Cycle Efficiency = \frac{(\overline{Cycle} - T_{waiting})}{\overline{Cycle}} \times 100$$

Low cycle efficiency due to hauling truck wait time means that there are more trucks in the cycle than necessary which may be reduced or used in an alternate location. The number of hauling trucks for each operation should be chosen based around the excavator load time or the bull dozer spreading time

assuming. For optimal cost efficiency, the ideal ratio of equipment should be considered to reduce wait time and increase cycle efficiency.

Having gained metrics on the construction process, and an understanding on construction management standard practices, recommendations to the contractor were then developed where inefficiencies were noted.

Objective 4: Project Time of Completion and Cost Efficiency

Using the heaped volume of the trucks, a basic process analysis was prepared. The amount of material hauled was averaged per hour from which and an approximate projection was developed assuming a 90% efficient cycle and a standard 6 day work week with 2 shifts. This projection was updated in order to project time of completion based on recommendations to the contractor.

The cost efficiency of the cycle was estimated considering the rental fees of the equipment and wage rates of laborers over each period of time based on cycle efficiency. This information was gathered through informal interviews with Christian Cáceres. An email was prepared for Cáceres that requested information on the following topics:

1. Standard wage rates for laborers and operators based on skill level and equipment certifications
2. Equipment owned by contractor and maintenance cost or equipment available for rental and rental fees
3. Delegation of equipment among source, stock pile, embankment, and shop
4. Volume of stock pile and mass balance based on excavation on construction rates

From this information, a cost analysis was developed based on the time to completion and recommended changes to current scheduling, equipment, and routes.

Results

Finding 1: The contractor is placing less material on the embankment than scheduled

The contractor submits a schedule that includes monthly benchmarks for placement of material. A summary of this information to date can be seen in the below table.

Table 42: Scheduled Placement of Material

Material	Placed	BL7-3	Diff.
Zona 1/1A	233,371.12	301,362.65	- 67,991.53
Zona 5	208,067.10	225,225.64	- 17,158.54
Zona 6	174,234.43	173,013.83	1,220.60
Zona 3A	11,309.29	17,318.77	- 6,009.48
Zona 3B	154,994.70	134,177.49	20,817.21
Zona 3	1,456,370.95	1,594,980.29	- 138,609.34
Zona 4	82,235.62	82,988.06	- 752.44
Backfill Total	335,787.00	442,827.75	- 107,040.75
Total	2,656,370.21	2,971,894.48	- 315,524.27

In the above table, each zone is listed in the left column, followed by the placed material according to ACP surveys. The BL7-3 column refers to the third revision of the Base Line scheduled placement submitted by the contractor. The third column is the difference between the two: the red numbers represent a deficit and the green a surplus. A complete table of the Contractor's monthly schedule and the ACP monthly surveys can be found in Appendix J.

It was found that the contractor in general has not been placing the amount of material as scheduled.

Finding 2: The contractor has not been placing Zone 1 and Zone 3 materials as scheduled over the past 5 months

The information from the contractor’s schedule and the survey data was compiled into several graphs representing the data collected from the beginning of construction. These figures visually compare the placed material and to the schedule on a monthly and cumulative basis.

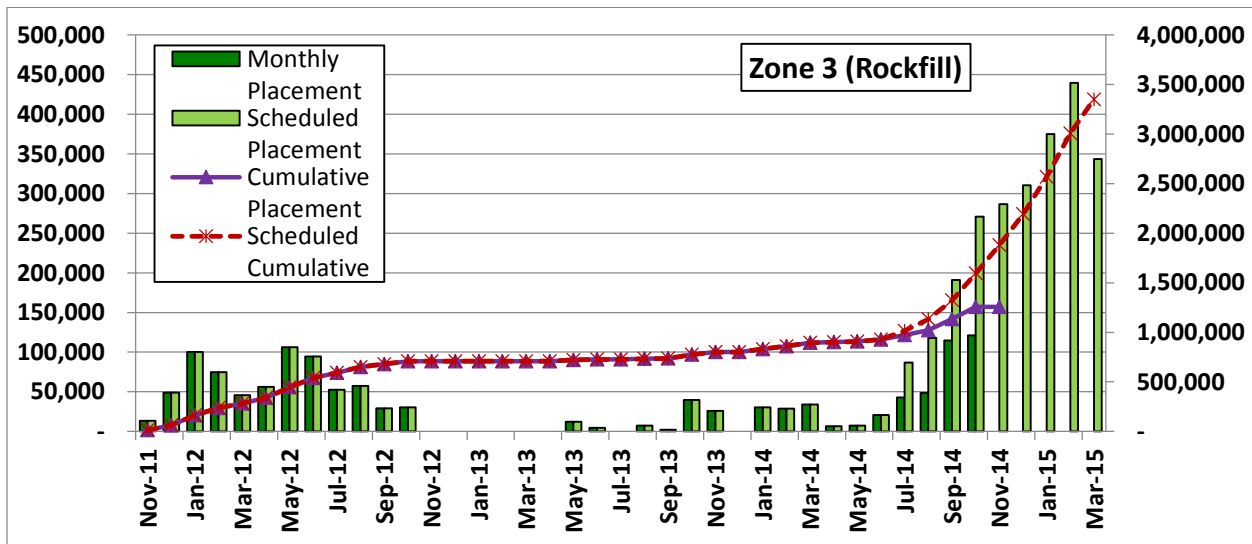


Figure 56: Construction Rockfill Schedule

The above figure shows the Contractor’s submitted schedule and the monthly history of construction of rockfill. In the past, the contractor has met the submitted schedule from November 2011 through May 2014. However, in recent months the contractor has not placed material at the scheduled rate. If this pattern continues, the contractor may not complete the construction on time.

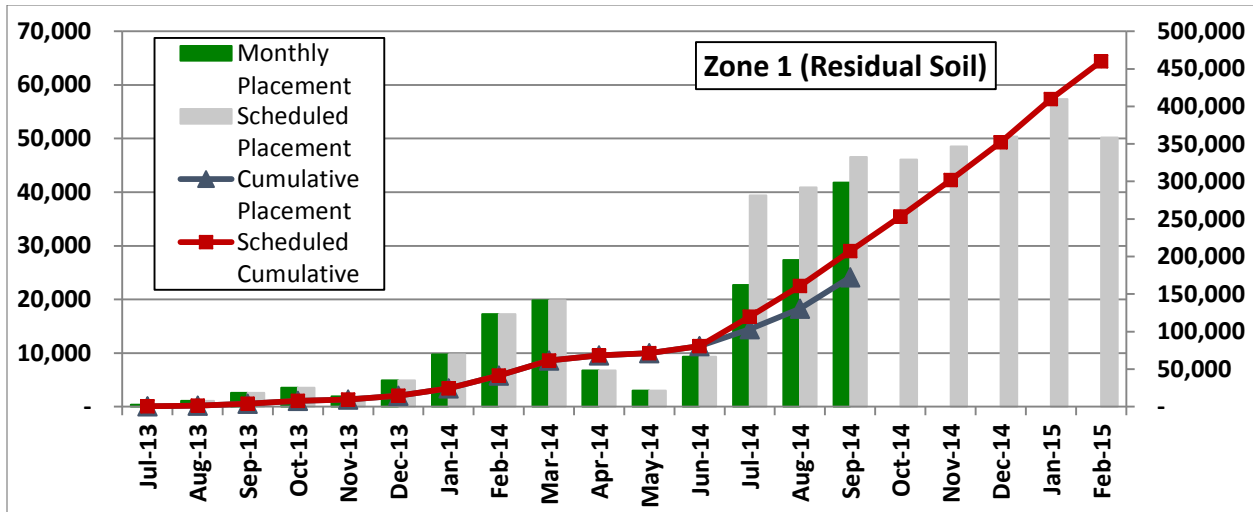


Figure 57: Construction Residual Soil Schedule

The above figure shows the Contractor’s submitted schedule and the monthly history of construction of residual soil. In the past, the contractor has met the submitted schedule from July 2013 through June 2014. However, in recent months the contractor has not placed material at the scheduled rate. If this pattern continues, the contractor may not complete the construction on time.

However, the filter layers including zones 3A, 3B, 5, 6 are fairly on track with the scheduled construction as can be seen in the following figures.

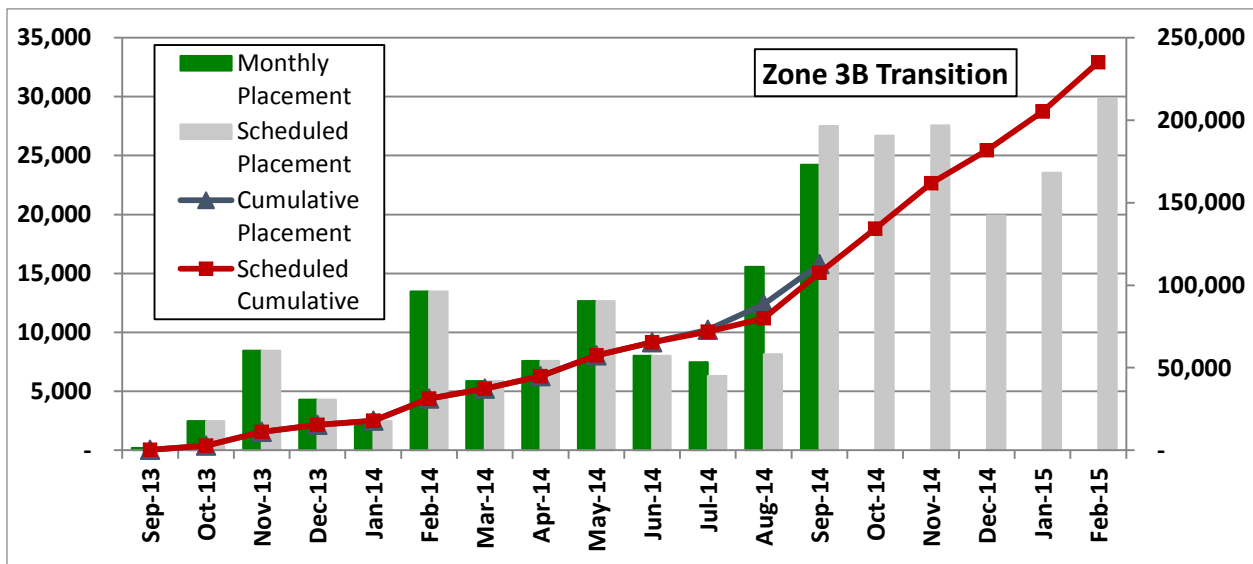


Figure 58: Filter Layer Zone 3B Embankment Construction

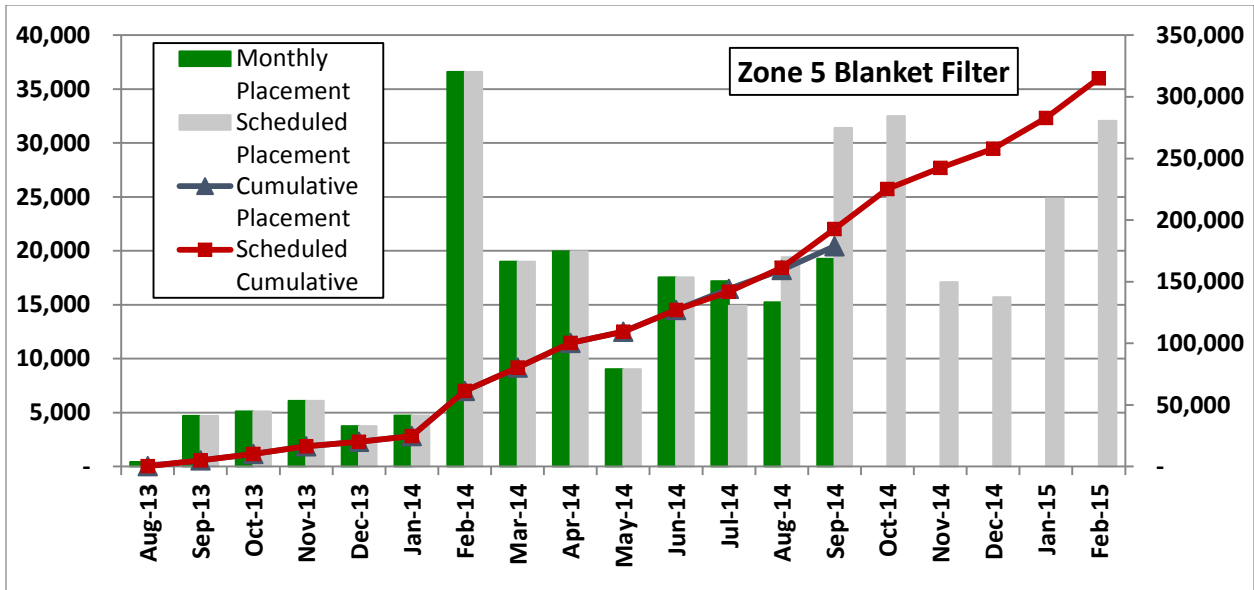


Figure 59: Filter Layer Zone 5 Embankment Construction

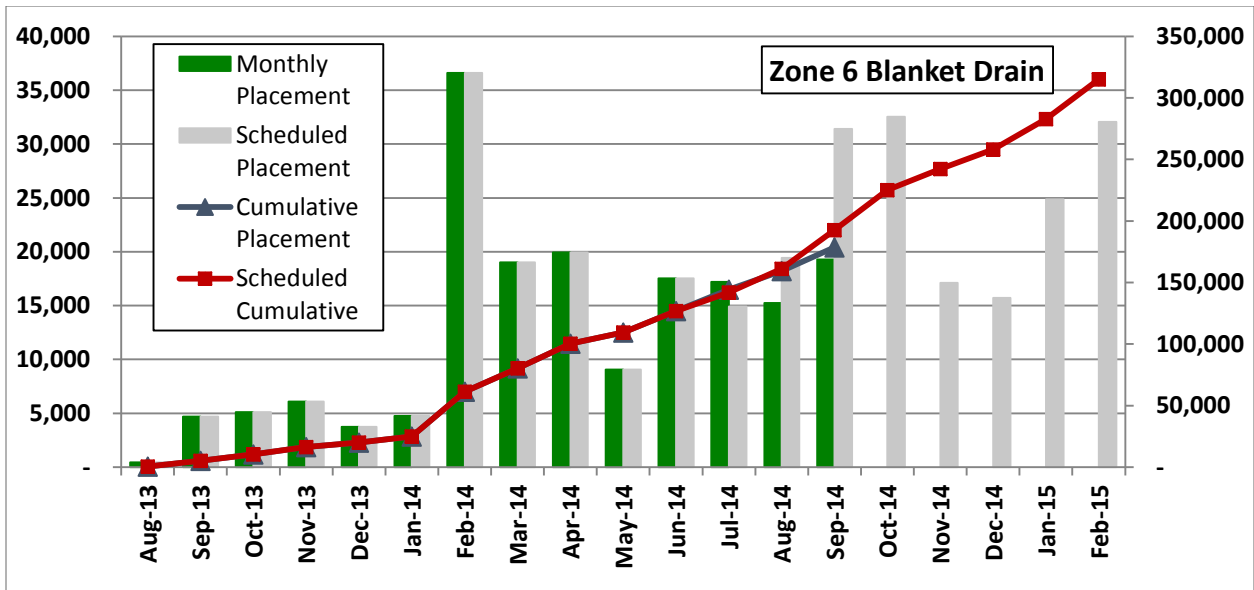


Figure 60: Filter Layer Zone 6 Embankment Construction

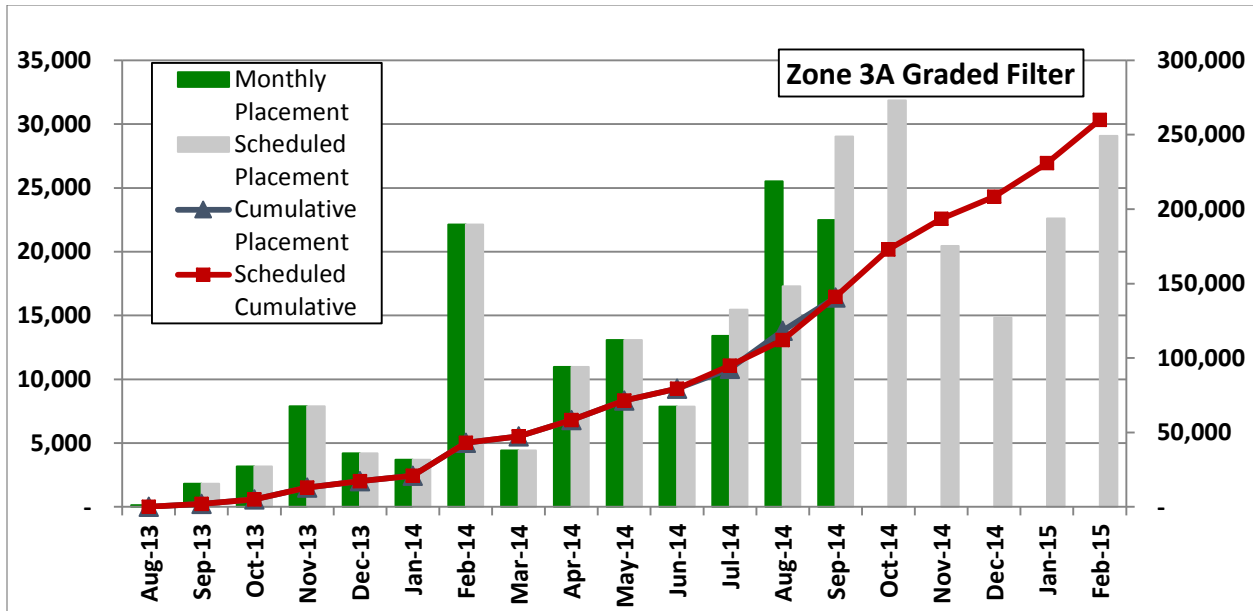


Figure 61: Filter Layer Zone 3A Embankment Construction

From the above figures, it was found that the contractor has been placing the filter layers as scheduled, but has not been placing zones 1 and 3 material as scheduled.

Finding 3: The contractor is not producing enough material for the filter layers to place as scheduled

The contractor submits a schedule that includes monthly benchmarks for processing of materials. A summary of this information can be found in the table below.

Table 43: Filter Layers Production Rate and Construction Mass Balance

	Zone 3B	Unit	Equation	DEC	JAN	FEB	MAR	APR
	[6]							
A	Stock Piled	m ³	C-D	50,000.00	48,096.00	43,940.80	8,174.80	69.46
B	Production	m ³		22,000.00	24,100.00	-	-	-
C	Available	m ³	A+B	72,000.00	72,196.00	43,940.80	8,174.80	69.46
D	BL7	m ³		23,904.00	28,255.20	35,766.00	8,105.34	-
	Zone 5							

A	Stock Piled	m ³	C-D	33,000.00	43,834.80	38,780.79	18,567.73	(4,735.39)
B	Production	m ³		33,000.00	30,000.00	25,000.00	25,000.00	4,800.00
C	Available	m ³	A+B	66,000.00	73,834.80	63,780.79	43,567.73	64.61
D	BL7	m ³		22,165.20	35,054.01	45,213.06	48,303.12	-
Zone 6								
A	Stock Piled	m ³	C-D	37,000.00	32,475.00	18,211.25	(4,146.25)	(13,622.50)
B	Production	m ³		14,000.00	14,000.00	14,000.00	14,000.00	13,700.00
C	Available	m ³	A+B	51,000.00	46,475.00	32,211.25	9,853.75	77.50
D	BL7	m ³		18,525.00	28,263.75	36,357.50	23,476.25	-
Zone 3A								
A	Stock Piled	m ³	C-D	10,000.00	11,172.20	144.31	(9,008.39)	(1,838.31)
B	Production	m ³		22,000.00	22,000.00	31,000.00	31,000.00	2,000.00
C	Available	m ³	A+B	32,000.00	33,172.20	31,144.31	21,991.61	161.69
D	BL7	m ³		20,827.80	33,027.89	40,152.70	23,829.92	-

In the above summary table, the four filter zones, zone 3b, 3a, 5, and 6 are listed. Under each, the current volume of the stockpile and the projected monthly production are listed. The available material is the sum of these two volumes. BL7 refers to the Base Line scheduled placement submitted by the contractor. Where this row is in green, the contractor has enough available material to place on schedule. Where this row is in red, the contractor will exhaust the available material. In this case, the remaining material will be placed in the following month. A complete table of the contractor's monthly schedule for production and the associated production plants can be found in Appendix I.

It was found that although the contractor will not always produce enough of every material to place as intended, there will be enough material produced to continue construction within a month of the intended schedule.

Finding 4: The stock piled zone 1 residual soil is being drawn from faster than excavation can restock it

The rate of excavation was recorded on a daily basis. A full table of the collected data can be found in Appendix J. A summary table was developed to compare the rate of excavation and the rate of construction on a monthly basis. It was assumed, based on an estimation in an interview with Christian Cáceres, that the starting stock pile had 91,000m³ of residual soil such that the stock pile in was approximately 30,000m³ at the beginning of December. 5,390.00m³ have been produced since the beginning of December through 12/14.

Table 44: Zone 1 Stock Pile Mass Balance

	Zona 1	Equation	SEP	OCT	NOV	DEC	JAN	FEB	
A	Stock Piled	C-D	91,000.00	61,647.16	54,197.16	29,932.16	-15,047.84	-72,424.84	
B	Production		12,415.00	38,626.00	24,276.00	5,390.00	-	-	
C	Available	A+B	103,415.00	100,273.16	78,473.16	35,322.16	-15,047.84	-72,424.84	
D	BL7		46,557.00	46,076.00	48,541.00	50,370.00	57,377.00	50,227.00	
E	Actual Placement		41767.84						

The above table shows that, assuming that the stock pile was at 30,000m³ at the beginning of December, the current construction rate must double in the last two weeks of December or the stock pile will become completely exhausted. Not only will this slow construction in December, but it will also significantly decrease construction in subsequent months since the residual soil needs to be stockpiled and processed to reach the appropriate water content before it can be brought to the embankment for use in construction.

It was found that the current rate of excavation using three fronts on the Miraflores II formation is not adequate to supply material for the scheduled construction.

Finding 5: The two main zones that will most likely slow construction are zone 1 and zone 3

After having gained insight on the operation in general, two main zones were identified as central to the progress of construction: the excavation of zone 1 residual soil and the placement of zone 3 rockfill.

Zone 1 is the impervious clay core. It is constructed in 200mm lifts and must be compacted at a precise water content to ensure optimal strength for the lifetime of the entire structure. This zone is therefore central to the progress of construction. The stockpile for this zone, however, is quite low and not being maintained at a sustainable rate for construction to continue at the rate of the other zones.

Zone 3 is the pervious rockfill shell of the dam. It is by far, the largest volume of any zone requiring 3,340,000m³ of material. For comparison, Zone 3 requires more than seven times the material of the next largest zone, Zone 1 with 460,000m³ of material. In addition to being the largest zone of the Dam, it also plays a key role in the construction process. Each lift of each zone is compacted from exterior inward, meaning that the material for the zone 3 lift must be laid prior to that of the other layers. Therefore, when the Zone 3 material is not placed in a timely manner, all of construction is delayed.

Along the length of the dam, a thin wall of Zone 3 has been constructed in order to maintain the rate of construction for the rest of the dam while the majority of rockfill has yet to be placed. Because the rockfill has the least precise compaction requirements, it is also used as a road. It is used for excavators, bull dozers, water trucks, inspection vehicles, and hauling trucks with materials for the other zones. However, at some points the wall is not wide enough for two lane traffic and hauling trucks must wait to pass, slowing the construction of the dam.

Because of these reasons, the recommendations were focused on these two zones.

Finding 6: Projected time of completion and most cost effective completion time

Because Zone 1 is subject to so much variability, a projection was developed based on the productivity of Zone 3 placement the following figure visually displays days to completion. This projection is developed based on several assumptions including:

1. Mild, dry, and predictable weather conditions
2. Plentiful availability of all materials for all zones
3. Availability of equipment
4. Availability of workers
5. 90% efficiency
6. Following all recommendations including shorter hauling routs and ideal scheduling

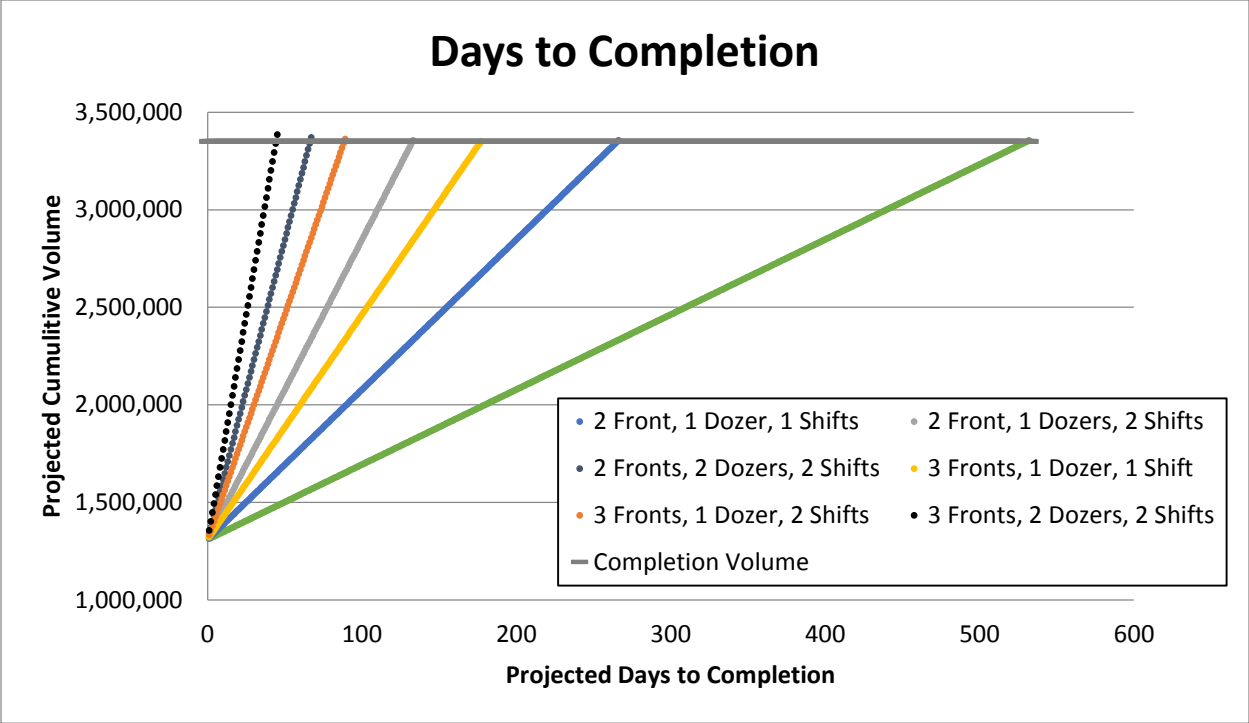


Figure 62: Projected days to completion

For additional clarity, a table was also developed with the associated date of completion assuming a start date of December 1st 2014.

Table 45: Projected Date of Completion

Day of Completion (Start 12/1/2014)					
Fronts		2 Front		3 Front	
Shifts	Work Week	1 Dozer	2 Dozer	1 Dozer	2 Dozer
Day	6 Day Week	13-Feb-15	4-May-15	29-Jan-15	5-Mar-15
	7 Day Week	24-Aug-15	13-Apr-15	25-Jan-15	26-Feb-15
Day & Night	6 Day Week	4-May-15	16-Feb-15	5-Mar-15	21-Jan-15
	7 Day Week	13-Apr-15	4-Feb-15	26-Feb-15	13-Jan-15

Three dates are highlighted in red in the above table. If the contractor continues to follow the current rate of production, using 2 fronts with one dozer at each front, than construction of zone 3 may be completed May 4th, 2015. Zone 3 is scheduled to complete March 2015, but as earlier figures suggest, the contractor has not been performing as scheduled in recent months. The January 15th 2015 completion date is the projected completion date assuming the contractor follows all the Zone 3 recommendations, increases construction fronts to 3 continuously for both sifts and doubles the construction at each front with two bull dozers. The third date is the suggested balance of the two, suggesting that the contractor increase fronts from 2 to 3, and therefore increase the rate of construction in order to complete construction within the schedule.

Table 46: Projected Construction Cost

Cost to Completion					
Fronts		2 Front		3 Front	
Shifts	Work Weeks	1 Dozer	2 Dozer	1 Dozer	2 Dozer
Day	6 Day Week	798,676.97	2,654,462.69	646,735.86	2,115,075.18
	7 Day Week	3,030,363.82	2,203,908.79	618,601.36	1,648,644.21
Day & Night	6 Day Week	1,829,413.66	1,179,833.68	1,363,450.33	908,531.79
	7 Day Week	1,521,051.34	1,081,459.84	1,068,395.72	824,322.11

The above table represents the cost to complete the construction for the associated date of completion. The two bolded totals are the two suggested dates from the previous table. These costs to completion were determined using the equipment rental fees which include laborers, maintenance, fuel, transportation, and all other costs associated with the equipment.

It was found that the most cost effective completion would be to increase equipment such that the construction is completed in February 2015.

Recommendations

The recommendations for this report are based around the observation of several key limitations for productivity including the Excavation of Zone 1 material, and placement of Zone 3 material on the

embankment. The following recommendations reflect potential opportunities to accelerate these processes and expedite production

Zone 1

Based on the observations from this study, several key points in the construction process may help improve the rate of construction. The hauling route, equipment scheduling, and excavation efforts may be improved in order to increase the rate of construction.

Recommendation 1: Improve Length and Quality of Hauling Route

It is recommended that the hauling route length be improved. From observations it was identified that the hauling route from the current excavation source for zone 1 residual soil could be decreased by approximately 1.1km.

The following figure shows the current hauling route, approximately 3.7km, in red and the proposed travel route, approximately 2.5km, in blue. The updated route would require minimal construction and run parallel to the highway.



Figure 63: Current and Proposed Hauling Routes

The hauling route travels all the way around the hill when it could travel a potentially shorter route of 2.5km, saving approximately one third the original distance.

Additionally, the hauling route quality slows the speed at which the trucks can travel. The current route, as seen in the blow figure, is built upon fairly unstable soil that is highly susceptible to erosion due to the excavation process.



Figure 64: Current Travel Route from Arial View

The route is narrow and at points two lane travel is impossible. Additionally, it winds around hills that decrease visibility. An upgraded route that decreases the travel time on potentially unstable and unsafe roads would increase the rate of travel and decrease the number of trucks necessary for an efficient cycle.

Recommendation 2: Increase Excavation Fronts

As observed by this study, there are approximately three open excavation fronts at the Miraflores Formation II where residual soil is being excavated. There is always one excavator at the top of the hill and one on the side of the hill. The third excavator primarily works on the top of the hill, but depending on the soil conditions may move to the side of the hill. The locations are marked by stars in the above figure. The white stars denote the excavation locations on the side of the hill and the black stars denote the excavation locations on the top of the hill. The usable material is being excavated from the side of the hill, but the top of the hill must be further excavated to avoid collapse while accessing the good material. The following figure is an initial geological survey of Miraflores II with elevation lines and roads mirroring those in the above image. Several areas with potentially usable material are colored in green.

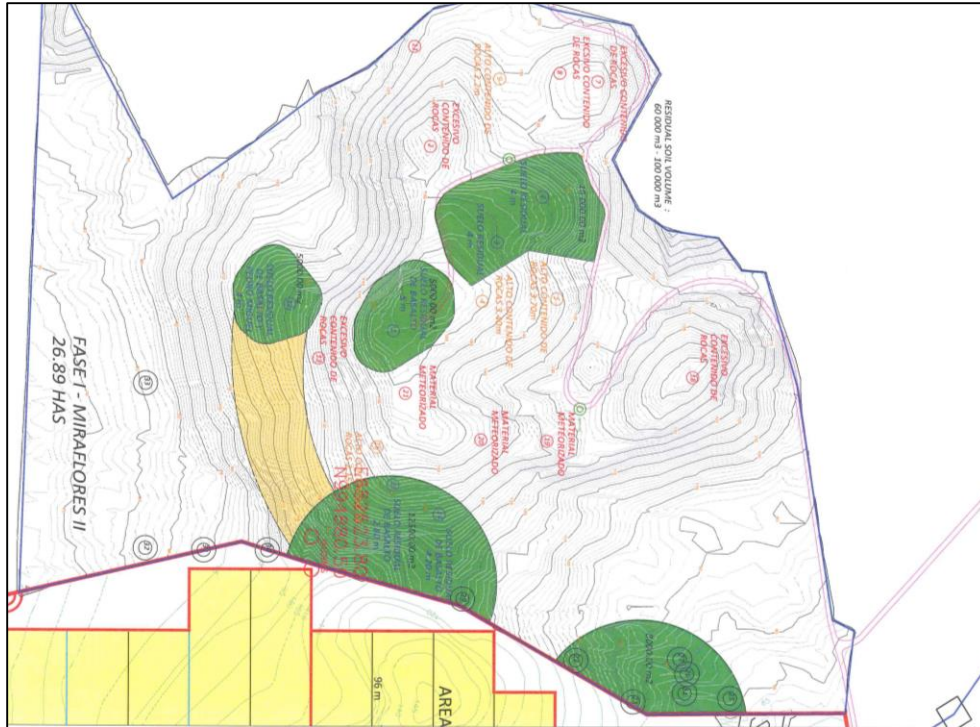


Figure 65: Miraflores II Geological Survey

It is recommended that additional excavators be used to open additional fronts in order to access greater amounts of material and maintain the stockpile. It is recommended that the excavator fleet be increased such that two excavators can work continuously on the side of the hill and three excavators can work continuously at the top of the hill in order to gain access to additional areas with usable material.

Recommendation 3: Utilize Bull Dozers to Assist the Excavators

It was observed that a significant amount of wait time at the source is due to the excavator moving material in order to place it into the hauling trucks. Trucks waited an average of four minutes and forty two seconds for the previous truck to be loaded and as much as eight minutes. This wait time could be decreased through the use of a small D6 bull dozer to assist the excavator in creating piles to load. As seen in the equipment table above, the D6 dozer costs \$40 per day and \$320 per month. The use of this dozer would decrease the bull dozer loading time and therefore the cycle time overall. With a shorter cycle time, the number of trucks necessary for the cycle could be decreased therefore decreasing the overall cost. Potentially, two less 740 articulated hauling trucks would be necessary which cost \$75 per day and \$600 per month each.

It is recommended that where needed, a D6 bull dozer be used to support excavators in order to decrease wait time of hauling trucks and increase cycle and cost efficiency of the operation.

Zone 3

Similar to Zone 1, based on the observations from this study, recommendations at several key points in the construction process may help improve the rate of construction. The hauling route, equipment scheduling, and embankment fronts may be improved in order to increase the rate of construction. Individual, several or all recommendations may be used in coordination.

Recommendation 1: Widen placement planes at embankment to allow for increased maneuverability of hauling trucks and bull dozer.

At the embankment, hauling trucks dump along the placement plane of the embankment, creating on site piles of their loads. The bull dozer then spreads these piles over the placement plane extending the length of the embankment at a higher level. The average wait time varies significantly depending on the front and shift from one, to several minutes. However, at times, hauling trucks waited in cues up to six trucks long because the plane was not wide enough to allow for adequate maneuverability of the bull dozer and the hauling trucks to unload.

It is recommended that the placement planes along the embankment be widened in order to accommodate more on site stock piles and increase maneuverability so the hauling truck wait time can be decreased and number of necessary trucks be decreased.

Recommendation 2: Use multiple bull dozers to expedite spreading of material

On all but one occasion, it was observed that one bull dozer was used per embankment placement plane. However, it was observed that when two bull dozers were used on a wider plane, the hauling trucks and bull dozers worked together more seamlessly. Each vehicle had less wait time as each hauling truck cycle and bull dozer supported one another when hauling trucks arrived out of order or bull dozers fell behind. The placement plane was also twice as wide allowing for greater maneuverability in the shared negative space where the bull dozers were not spreading material.

It is recommended that placement planes be widened in order to accommodate two bull dozers each and associated hauling trucks be increased to accommodate two bull dozers at each front.

Recommendation 3: Use Compactor to improve temporary construction road surface conditions

It was observed that hauling vehicles travel significantly faster on well-traveled roads than on the temporary construction roads. These roads are highlighted in red in the figure below.



Figure 66: Zone 3 Hauling Route Quality

The portion of the route highlighted in red represents the regularly traveled route that is naturally compressed throughout travel. However, the portion of the route highlighted in black does not appear through the observations of this report to be as well maintained. Hauling trucks travel along this route to bring zone 3 material to the North and Center Fronts currently. Up to 2km of this route could be compacted to a greater degree to which hauling truck travel time could be decreased for trucks delivering loads to the north and center fronts.

It is recommended that a compactor be used along the temporary hauling route highlighted in black on a weekly basis.

Recommendation 4: Optimize number of hauling trucks and route distance to construction sites

It was found that the number of hauling trucks for each cycle is not optimally efficient as seen in the below table.

Table 47: Efficiency of Hauling Cycle

Zone 3									
Rout		Current				Proposed			Saved
Hauling Cycle Efficiency: Route Distance									
Point A	Point B	Cycle Time	Distance	Rate	Efficiency	Cycle Time	Distance	Time	Distance
Stock Pile	South Front	24:28	4.3	5:41	74%	18:12	3.2	6:16	1.1
Hauling Cycle Efficiency: Wait Time									
Vantage Point	Point B	Cycle Time	Wait Time	# Trucks	Efficiency	Cycle Time	# Truck		# Truck
North Front	Stock Pile	19:12	3:55	5	80%	15:17	3		2
South Front	Stock Pile	24:28	0:16	6	99%	24:12	6		0

It was observed, that when the optimal number of trucks were uses, hauling truck wait time and bull dozer wait time was decreased significantly.

It is recommended that the hauling trucks traveling to the south route use the shortest route that the hauling trucks use for the north and center fronts. It is also recommended that 3 trucks be used for the north front, and 6 for the south front in order to operate in optimal cycles.

Recommendation 5: Increase Embankment construction sites to three fronts continuously: North, South, and Center

It was observed that embankment construction is active on only one to two fronts at a time. Embankment construction is limited by the amount of material that can be transported. Therefore it is recommended that the hauling truck, and bull dozer fleets be increased such that active embankment construction can occur on all three fronts.

Conclusions

The potential points for improvement in the construction process were identified through interviews with ACP quality engineers, on site observations and data collection, research, and data analysis. In this analysis, the excavation rate and production rate of materials, and the hauling route and scheduling efficiency of equipment were investigated. From this analysis, it was identified that the most likely zones to delay the construction process were Zones 1 and 3. Several recommendations were developed to be presented to the Project Management Team to potentially improve the efficiency of these two zones. The Project Management team is working with the contractor to apply these and similar recommendations to ensure a timely completion. Time to completion was projected based on several of these recommendations for Zone 3 construction because Zone 3 represents the zone with the largest volume. The associated cost to completion for each date was also developed based on recommended changes to the equipment fleet and rental rates. It was projected that at the current rate of construction as observed over the past month, the date of completion may be May 4th 2015, two months after the contractor's submitted schedule. It was concluded that the superior combination of recommendations for best time and cost efficiency was for the contractor to increase the width of the placement planes to accommodate 2 dozers each at the two current fronts and increase the associated equipment fleet likewise for a February 15th completion date assuming a 6 day work week. This updated schedule is projected to cost the contractor \$1,179,833.68 starting from December, 1st 2014.

Conclusion

Over the past eight weeks, our project team has conducted extensive research with the Panama Canal Authority. This research focused on both the maintenance dredging operation of the Pacific Entrance of the Panama Canal and the Canal Expansion Program.

The maintenance dredging research sought to answer two main questions, whether or not the project would finish on time and if it was being completed in the most efficient way. Through the analysis of daily reports, overview of dredging operations, and research and application of best practices, both objectives were accomplished. First, it was determined that the maintenance dredging project would be completed on time. Additionally, the team recommended the ACP invest in a trailing suction hopper dredger.

For section regarding the production of the As-Built drawings for the stitch grouting sections of the Borinquen Dam 1E, an instruction manual was developed. This instruction manual was tested on peers for ease of interpretation and simplicity. At the end of this research project, this instruction manual was proved to produce quality drawings which complied with URS Corp. Inc. specifications in a timely matter. While it was estimated by the consultants that only three of the ten stitch grout sections would have been produced during this internship, eight were completed. This justifies that the process designed was functional and efficient.

The research on embankment construction and compaction of clay core determined that the central core earth and rockfill embankment dam structure is the best possible option for construction of Borinquen Dam 1E. It was additionally found that the compaction requirements and testing specifications for quality control are accurate for the construction of Borinquen dam 1E. Furthermore, the testing specifications are more suitable than the more restrictive personally developed specifications. It was recommended that the Zone 1 field compaction processes and the laboratory test procedures be carefully supervised and the lab test results be precisely and critically evaluated before their approval.

When looking at of construction of the Borinquen Dam 1E, the two zones most likely to delay project productivity are Zone 1 and Zone 3. Several recommendations were developed to optimize the construction of these zones. Based on the construction of Zone 3, it was determined that, at the current rate of construction as observed by this study, construction will complete two months behind schedule. In order to complete construction on time and with optimal cost efficiency, it was recommended that the contractor double current construction at each of the two current construction fronts with two bull dozers each. The projected cost to contractor is approximately \$1.2 million from a December 1st start date.

Our research project has produced many recommendations and conclusions over the past eight weeks. When combined, the research conducted will be useful for the ACP to improve existing operations and future projects. It is our hope that this research will have a lasting, positive effect on the Panama Canal Authority.

References

- Abu-Khashaba Mohamed, El-Ashaal A.Adam, (2013) *Investigating the possibility of constructing low cost roller compacted concrete dam*, Alexandria Engineering Journal,
- Arauz de Grimaldo, M. (2012, December 20). A Primer on Panama's Labor & Employment Law. Retrieved December 1, 2014, from <http://terralex.org/publication/80128a914f>
- ASTM international, *Annual book of ASTM standards, Section: 4 Construction*, (n.d.) Retrieved from URS Oakland library on November 13th, 2014
- ASTM international, *Annual book of ASTM standards: Standard Practice for Classification of Soils for Engineering Purposes (United Soil Classification System [USCS])* (n.d.) Retrieved on November 10th, 2014
- Autoridad Del Canal De Panama (ACP) (2009), *Notification to Bidders*, Solicitation No. 94278, Pacific Access Channel, Project No. PAC-4, Division 35: Waterway and Marine Construction, Section 35 73 16 – Embankment Dam Construction
- Bates, R. (1984). *Dictionary of geological terms* (3rd ed., Anchor Books ed.). Garden City, N.Y.: Anchor Press/Doubleday.
- Becue, J. (2012). Choice of site and type of dam. Retrieved December 9, 2014, from <http://www.barrages-cfbr.eu/BackUp/Info/documentation/texte/pb2002/anglais/pb2002-c1-p17.pdf>
- Bray, Nick. *Dredging for Development*. The Hague, Netherlands: International Association of Dredging Companies, 1997. Print.
- Bray, R. N., A. D. Bates, and J. M. Land. *Dredging: A Handbook for Engineers*. London: Arnold, 1997. Print.
- Business, E. (2000, September 11). Primavera Introduces Primavera Enterprise 2.0. Retrieved December 1, 2014, from <http://ezproxy.wpi.edu/login?url=http://search.proquest.com.ezproxy.wpi.edu/docview/445874411?accountid=29120>
- CATERPILLAR 336D L HYDRAULIC EXCAVATOR. (2007, January 1). Retrieved December 1, 2014, from <http://www.ritchiespecs.com/specification?type=Construction+Equipment&category=Hydraulic+Excavator&make=Caterpillar&model=336D+L&modelid=104720>
- CATERPILLAR 773 ROCK MOVER. (2007, January 1). Retrieved December 1, 2014, from <http://www.ritchiespecs.com/specification?type=Construction+Equipment&category=Rock+Truck&make=Caterpillar&model=773&modelid=93322>

CATERPILLAR 740 ARTICULATED DUMP TRUCK. (2007, January 1). Retrieved December 1, 2014, from <http://www.ritchiespecs.com/specification?type=Construction+Equipment&category=Articulated+Dump+Truck&make=Caterpillar&model=740&modelid=91910>

CATERPILLAR D6 EARTH MOVER. (2007, January 1). Retrieved December 1, 2014, from <http://www.ritchiespecs.com/searchresults?type=Construction+Equipment&category=Shovel&make=Caterpillar>

CATERPILLAR D8 EARTH MOVER. (2007, January 1). Retrieved December 1, 2014, from <http://www.ritchiespecs.com/searchresults?type=Construction+Equipment&category=Shovel&make=Caterpillar>

Cooper, R., Aouad, G., & Lee, A. (2008). Construction project process models. In *Process Management in Design and Construction* (p. 206). John Wiley & Sons.

Department of water affairs and forestry, (1988) Knellpoort Dam, Retrieved December 3, 2014, from: <http://www.ewisa.co.za/misc/damfsknelpoort/0012564.JPG>

Environmental Impact Study- Category III. (2007, July 1). Retrieved December 1, 2014, from <http://micanaldepanama.com/expansion/wp-content/uploads/2014/04/chap-08.pdf>

Fell, R., MacGregor, P., & Bell, G. (2005). *Geotechnical Engineering of Dams* (p. 912). Great Britain: A.A. Balkema Leiden, The Netherlands, a member of Taylor & Francis Group plc.

(2013, February 20). Flood Overtopping Failure of Dams. *FEMA Workshop*. Lecture conducted from U.S. Department of the Interior Bureau of Reclamation.

Goldsmith, E., & Hildyard, N. (1984). Chapter 9. In *The Social and Environmental Effects of Large Dams* (Vol. 1). UK: Wadebridge Ecological Centre, Worthyvale Manor Camelford, Cornwall PL32 9TT.

Hazrat, A., Rabiul, A., Naimul, H., & Jahangir Alam, M. (2012). *Comparaison of Design and Analysis of Concrete Gravity Dam*. Chittagong, Bangladesh.

Hilf W. Jack, Ph.D, PE, Consulting Engineer, (1991), *Foundation Engineering handbook, second edition, Compacted Fill*, Colorado.

Huston, John. *Hydraulic Dredging: Theoretical and Applied*. Cambridge, MD: Cornell Maritime, 1970. Print.
International commission of large Dams, Technology of dams. (n.d.). Retrieved December 4, 2014, from http://www.icold-cigb.org/GB/Dams/technology_of_dams.asp

James Toose, Borinquen Dam 1E [Personal interview], December 5th, 2014

Mehta P.K. & Monteiro P.J.M., (2005) *Roller Compacted Concrete, Concrete: Microstructure, Properties, and Materials*.

Mcmahon, J. (Ed.). (2004). General Design and Construction Considerations for Earth and Rock-Fill Dams. *Engineering and Design*, 130-130. Retrieved December 1, 2014, from http://www.publications.usace.army.mil/Portals/76/Publications/EngineerManuals/EM_1110-2-2300.pdf

Panama Canal Authority (Ed.). (1998, January 1). Trade Routes. Retrieved December 1, 2014, from <https://www.pancanal.com/eng/op/routes.html>

Plates & Rammer. (n.d.). Retrieved December 10, 2014, from <http://www.smplantsales.co.uk/img/listings/1509.png>

Richardson, M. J., and Steve L. Richardson. *The Dynamics of Dredging*. Irvine, CA: Placer Management, 2001. Print.

Sebastien Bonnard, Borinquen Dam 1E [Personal interview], November 10th, 2014

Seismological Society of America. (2010, November 19). Panama Canal, Panama City at risk of large earthquake, says new research. ScienceDaily. Retrieved December 9, 2014 from www.sciencedaily.com/releases/2010/11/101118141543.htm

Sheepsfoot Roller. (n.d.). Retrieved December 10, 2014, from http://www.bluelinerental.com/sites/default/files/imagecache/equipment_image/equipment/image/Sd115_63_2.jpg

Smooth Wheeled Roller. (n.d.). Retrieved December 10, 2014, from <http://www.cn-changlin.com/products/3-3.jpg>

Stephens, T. (2010). Dam Construction. In *Manual on Small Earth Dams* (Vol. A Guide to siting, design and construction, p. 114). Rome: Food and Agriculture Organization of the United Nations.

Turner, Thomas M. *Fundamentals of Hydraulic Dredging*. Centreville, MD: Cornell Maritime, 1984. Print.

Turner, Thomas M. *Fundamentals of Hydraulic Dredging*. New York, NY: ASCE, 1996. Print.

United States Bureau of Reclamation. (2012). Dam Safety. *Flood Overtopping Failure of Dams and Levees*. Retrieved December 9, 2014.

United States department of the interior bureau of reclamation (1976), *Design of Gravity Dams*, Denver, Colorado.

U.S. Army Corps of Engineers,(2000), *Engineering and Design: Roller-Compacted Concrete*, Washington, DC

URS Holdings, Inc., (2009). *Final Design Report: Borinquen Dam 1E B.2.6*. URS Holdings, Inc., pp.1 -37

URS Holdings, Inc., (2009). *Final Design Report: Borinquen Dam 1E B.2.6*. URS Holdings, Inc.

URS Holdings, Inc. (2008), Final Design Report on in-situ Construction Materials Dam 1E, *Geotechnical Interpretive Final Report (GIR)*.

URS Holdings, Inc. (2008) Draft Report On Test Burrow Pits And Compaction Field Tests Zone 1 Test Fill, Borinquen Dam 1E

URS Holdings, Inc. (2009), *Task A.1.4 Foundation Materials – Dam 1E Final Geotechnical Interpretive Report (GIR)*

U.S. Army Corps of Engineers (2004). *General Design and Construction Considerations for Earth and Rock-Fill Dams*, Washington, DC.

US army corps of Engineers (1995), *Construction control for earth and rockfill dams*, Washington, DC

USSD Representatives. (Director) (2011, April 11). 21st Century Dam Design — Advances and Adaptations. *31st Annual USSD Conference*. Lecture conducted from United States Society on Dams, San Diego, CA.

Vibrating Roller. (n.d.). Retrieved December 10, 2014, from http://image2.cccme.org.cn/i_supply/2009/9/8/2011010213012010285_145008143.jpg

Vibrating Roller. (n.d.). Retrieved December 1, 2014, from http://image2.cccme.org.cn/i_supply/2009/9/8/2011010213012010285_145008143.jpg

When It's Right to Rent. (2014, January 1). Retrieved January 1, 2014, from http://www.cat.com/en_US/articles/cat-dealers/paving-rent-time.html

Woodward, R. (2004, January 1). Dam Failures. Retrieved December 9, 2014, from <http://members.optusnet.com.au/~engineeringgeologist/page18.html>

Weaver, K., & Bruce, D. (2007). *Dam foundation grouting* (Rev. and expanded ed.). Reston, Va.: ASCE Press.

Appendix A: Summary of the main critical design criteria requirements specific to the site and construction of Borinquen Dam 1E

- The Borinquen Dam 1E must be statically stable and must account for the possibility of seismic loads due to fault displacements (URS Holdings, Inc., 2009).
- Both the concrete and embankment dam must be able to remain intact and functional after the possible amounts of displacements predicted, hence its structural integrity should not be compromised (URS Holdings, Inc., 2009).
- Seismic displacements must not compromise its ability to retain the Gatun Lake at any time (URS Holdings, Inc., 2009).
- If an embankment dam is chosen, the zones must remain functional after seismic deformations (URS Holdings, Inc., 2009).
- In the case of a central core earth and rockfill embankment, the core of the dam must be able to accommodate for fault displacements without piping, thus still remain impermeable (URS Holdings, Inc., 2009).
- The dam should have low future maintenance (URS Holdings, Inc., 2009).
- The dam should maintain PAC water level with minimum seepage (URS Holdings, Inc., 2009).
- The dam should be cost effective. Therefore, it should be done with the materials available around the site area, for both concrete and earthfill embankment options (URS Holdings, Inc., 2009).
- The construction of the dam should take in to consideration the constraints caused by the rainy season present at the site (URS Holdings, Inc., 2009).
- The dam should account for the possibility of ship grounding and consequentially not loose functionality or structural integrity (URS Holdings, Inc., 2009).
- If a concrete dam is chosen, analysis of the foundations strength and compressibility must be done to determine whether it can hold the weight of the dam, and analyze all the possible types of kinematically feasible failure modes (Fell, 2005).
- If an embankment dam is chosen it must be sufficiently impermeable, strong enough for static and seismic stability, ductile and flexible enough to remain functional after seismic displacements caused by earthquakes and have low compressibility to prevent damage from settlements (Toose, 2014).

Further discussion on general criteria for adopting a type of dam structure

The selection of the type of dam to be used at a particular site location is affected by many factors. In most cases the predominant consideration is to build the safest structure for the lowest price. The most economic design usually is the one which uses construction materials found in the proximities of the construction site, without excessive modification and processing from the burrow pit. This is especially crucial for earthfill and rockfill embankment dams, which require the use of a large volume of material (Fell, 2005). It is important to take note that it is difficult to construct earthfill embankments in wet weather circumstances, particularly when precipitations are relatively continuous without high evaporation. In any case, the construction in wet weather situations must not ruin the quality of the structure (Fell, 2005).

Table comparing advantages and disadvantages of types of embankment dams available for the construction of Borinquen Dam 1E

The main researched and reported features of zoned earthfill and central core earth and rockfill embankment dams were: the description of the zones, the degree of control of internal erosion and piping, the pore pressures for stability and the suitability of the dam in relation to consequences of failure classification. These features are organized in the table below.

Table 48: zoned earthfill and central core earth and rockfill embankment dam main features (Fell, 2005)

	Zoned earthfill embankment	Central core earth and rockfill embankment
Zone description	Zone 1 is earthfill. Zones 1-3 are made of burrow pit run alluvial silt/sand/gravel; or weathered and low strength rock, compacted to form silt/sand/gravel.	Zone 1 earthfill, Zones 2A & 2B filters, Zones 3A & 3B rockfill.
Degree of filter control of internal erosion and piping	Moderate (poor to good). All seepage will be intercepted by Zones 1-3. Depends on particle size distribution of Zones 1-3 to act as a filter to Zone 1.	Very good, seepage in earthfill and from cracks is intercepted by the filters and discharged in the rockfill
Degree of control of pore pressures for stability	Good provided Zones 1-3 is much higher permeability than Zone 1	Very good provided the rockfill is free draining.

Consequences of failure classifications to which suited – new dams	Very low to significant, depending on material particle size distributions and construction control.	Significant to extremes. Likely to be too complicated and costly for dams less than 20m high.
---	--	---

Further characteristics of embankment dams:

- The materials need to be available at the site to be cost effective
- Embankment dams do not require extremely strong foundations however the foundation needs to be treated correctly (Fell, 2005).
- The dams offer a certain degree of flexibility in the event of seismic displacements and will still remaining functional depending on the type of embankment chosen and on how large the displacement is.
 - Zoned earthfill has less degree of flexibility when accounting for displacements caused by earthquakes compared to central core earth and rockfill embankment dam (Toose, 2014).

Main features of concrete dam structures

The main researched and reported features of mass concrete gravity dams and RCC dams were: durability, maintenance, foundation strength and compressibility requirements, availability of suitable aggregates, and advantages compared to each other and compared to embankment dams.

Mass Concrete Gravity Dam Main Features

- Mass concrete gravity dams are durable and require low maintenance (Hazard, 2012).
- They require a strong foundation with low compressibility to hold up the weight of the structure (Fell, 2005).
- They do not allow for a large degree of flexibility and ductility needed in the event of fault displacements (Toose, 2014).
- They need suitable aggregate material at low cost, preferably found at the site area (Fell, 2005).

RCC Dam Main Features:

- Roller compacted concrete dams are durable and require low maintenance once constructed (Toose, 2014).
- They require a strong foundation with low compressibility to hold up the weight of the structure (Toose, 2014)

- They do not allow for a large degree of flexibility and ductility needed in the event of fault displacements (Toose, 2014).
- They have lower material costs compared to mass concrete gravity dams, due to the different type of mix design which also enables it to be hauled to the site with dump trucks thus reducing transportation time and cost; and lower material cost compared to embankment dams due to reduced material quantities. However to be cost effective the material needs to be available at the site area (U.S. Army corps of Engineers, 2000).
- They have high production rates due to: faster rates of concrete placement, less heat of hydration and less post-cooling compared to regular concrete dams. The outcomes are less construction time and lower costs compared to concrete and embankment dams, which include: reduced administration cost, possible earlier project benefits and the use of dam sites that have limited construction seasons due to freezing or wet weather conditions (U.S. Army corps of Engineers, 2000).
- RCC dams, compared to embankment dams, offer the alternative of constructing the spillway in the main structure of the dam. Whereas, the embankment dams normally require that spillways to be constructed in an abutment. This can significantly reduce the cost and time of construction (U.S. Army corps of Engineers, 2000).

Appendix B: Summary of the Contractor's Lab Test Results

Test Fill Number 7 Results

The results from test fill number 7 are organized and enumerated to better understand their meaning.

Tabularized results for tests performed on Zone 1 test fill number 7

Below is table which displays the results obtained from the tests performed, by the ACP and by the contractor, on Zone 1 test fill number 7.

Table 49: results of Zone 1 test fill number 7

Compaction (%)	Dry Density (kg/m ³)	Moisture content from Sand Cone Test (%)	Moisture content (%)	Average undrained shear strength (kPa)	USCS Soil Classification	LL	PI	OMC (%)	MDD (kg/m ³)	Percent passing 3/4" (19.5m m) sieve	Percent passing No. 200 (0.075m m) sieve
ACP test results											
73	952.3	41	41	150.54	SC	62	33	39	1296	100	46
88	1137.8	48	46	155.65	SC	62	33	39	1296	100	46
89	1148.04	46	40	132.68	SC	68	35	38	1289	98	34
101	1302.3	41	41	127.58	SC	68	35	38	1289	98	34
Contractor test results											
102.9	1391.6	41.3	42.3	128.65	MH	72	28	35.6	1353	100	89
93.1	1266.7	41.1	42.9	137.27	MH	68	28	34.8	1361	100	80.44
95	1294.6	40.8	41.8	143.23	GM	67	27	35.1	1363	95.4	71.6
94.2	1235.3	46.2	46.1	147.88	MH	72	35	38.2	1311	100	77

97.7	1295.1	42.7	42.7	86.87	MH	78	34	37.8	1325	100	80
98.3	1311.7	42.1	42.1	90.18	MH	67	22	36.7	1335	100	69.8
89.2	1201.7	44.4	41.6	80.9	MH	78	32	36.3	1348	97.7	80.7
97.7	1329.7	39.7	39.7	98.14	MH	79	33	35.8	1362	100	84.6
86.9	1151.4	45.2	47.1	137.93	MH	64	18	37.7	1325	100	82.9
99.3	1339.1	43.8	43.5	124.67	MH	83	38	35.9	1348	100	86.9
98	1299.2	44.6	45.9	86.21	MH	59	13	36.9	1326	100	83.3
89	1194.7	43.9	43.4	118.04	MH	71	24	36.9	1343	100	85.8
92.2	1221.1	46.8	47.9	52.39	MH	78	33	37.8	1319	100	84.8
93.6	1258.4	42.9	44.3	62.33	MH	79	36	36.8	1344	100	84.4
91.9	1181	49.6	47.8	58.35	CH	83	47	40.7	1285	98.4	86.9
93.9	1269.2	45.5	47.8	57.69	MH	80	43	36.2	1352	100	88

Summary of the Contractor's Lab Test Results

- According to the Atterberg Limit and Gradation test results performed by the contractor, the residual soil derived from the weathered basalt placed in the test fill number 7 classified as elastic silt (MH). Only two of the soils were differently classified as silty gravel (GM) and dense sandy clay (CH).
- The fines content (percent passing the No. 200 sieve) was between 70% and 90%. The percent passing the No. 4 in sieve was always above 90% and the percent passing the No. 10 in sieve was always above 100%.
- The ranges of the liquid limits (LL) and plasticity indices (PI) were between 59 to 83 and 13 to 47, respectively.
- The moisture content ranged between 41.6% and 47.8%
- The moisture content calculated from the sand cone test ranged between 40.8% to 49.6%
- The dry density calculated with the sand cone test ranged between 1151.4 kg/m³ to 1391.6 kg/m³
- The maximum dry density and optimum moisture content ranged between 1285 kg/m³ to 1363 kg/m³ and 34.8% to 40.7%, respectively.

- The compaction achieved was between 87% and 99%
- Four vane shear tests were done on four different soil samples. Samples 1, 2 & 3 were all above 75 kPa and ranged between 80 kPa to 150 kPa. Whereas the vane shear test results for Sample 4 were all below 75 kPa and ranged between 50 kPa to 60 kPa.

Summary of the ACP's Lab Test Results

- According to the Atterberg Limit and Gradation test results performed by ACP, the residual soil derived from basalt placed in the test fill number 7 classified as Clayey sand (SC)
- The fines content (percent passing the No. 200 sieve) of test sample number 1117 was around 46%. The percent passing the ¾" and 3" sieves was 100%. The percent passing the No. 200 sieve of test sample 1119 was around 34%. The percent passing the ¾" sieve was around 98% and the percent passing the 3" sieve was 100%
- The liquid limit and plasticity index for test sample No. 1116 & 1117 were 62 and 33. The liquid limit and plasticity index for test sample No. 1118 & 1119 were 68 and 35.
- The moisture content, tested on four samples, ranged between 40% and 46%
- The moisture content calculated from the sand cone test ranged between 41 to 48%
- The dry density calculated with the sand cone test ranged between 59 pcf to 81 pcf, 952.3 kg/m³ to 1302.3 kg/m³
- The maximum dry density and optimum moisture content were 80 pcf - 81 pcf (1289 kg/m³ - 1296 kg/m³) and 38% - 39 %.
- The compaction achieved was between 73% and 100%
- The vane shear test results were all above 75 kPa and ranged between 127.58 to 155.65 kPa.

Summary of Zone 1 personally generated specifications

- 1) Material to be residual soil (MH and CH preferred, GM and SC alternatives)
- 2) 100% passing 3" (75mm) sieve, ≥ 70% passing ¾" (19mm) sieve and > 25% passing No.200 (0.075 mm) sieve
- 3) PI > 10
- 4) Minimum undrained shear strength = 75 kPa
- 5) Compaction water content range between +2% and +8% of OMC

Appendix C: Summary of Zone 1 specifications

Tests performed

The tests performed on the compacted clay core were (ASTM international, 2014):

- ASTM D 2573, Standard test method for field vane shear test in cohesive soil
- ASTM D 2216, Standard test method for laboratory determination of Water Content of Soil and Rock by Mass.
- ASTM D 4318 standard test method for Liquid Limit, Plastic Limit & Plasticity index of soils
- ASTM C 136, standard test method for sieve analysis of fine and coarse aggregates
- ASTM D 698, standard test method for laboratory compaction characteristics of soil using standard effort.
- ASTM D1156, Standard test method for density and unit weight of soil in place by sand cone method.

Test Results

Below are the field and laboratory test result nicely organized in tables.

Table 50: Water Content and Shear Vane Test Results

Water Content Result (%)	Shear Vane Strength Result (kPa)
46.8	87.83

Table 51: Gradation Test Results of the 3 Main Sieves Addressed In The Specifications

Gradation Test Results		
% passing 3" Sieve	% passing 3/4" Sieve	% passing No. 200 Sieve
100	98.50	81

Table 52: Liquid limit (LL), Plastic Limit (PL) and Plasticity Index (PI) test results

Atterberg Limits Test Results		
LL	PL	PI
66	44	22

Table 53: Proctor Compaction Test and Sand Cone Test Results

Sand Cone Test and Proctor Compaction Curve Results				
Water content (%)	Dry Density (kg/m ³)	MMD (kg/m ³)	OMC (%)	Compaction (%)
47.2	1172.2	1318	38	88.9

Table 54: Soil Classification test result

Soil Classification	
ASTM D 2487	MH
ASTM D 3282	A-7-5 GI=24

Summary of Zone 1 specifications

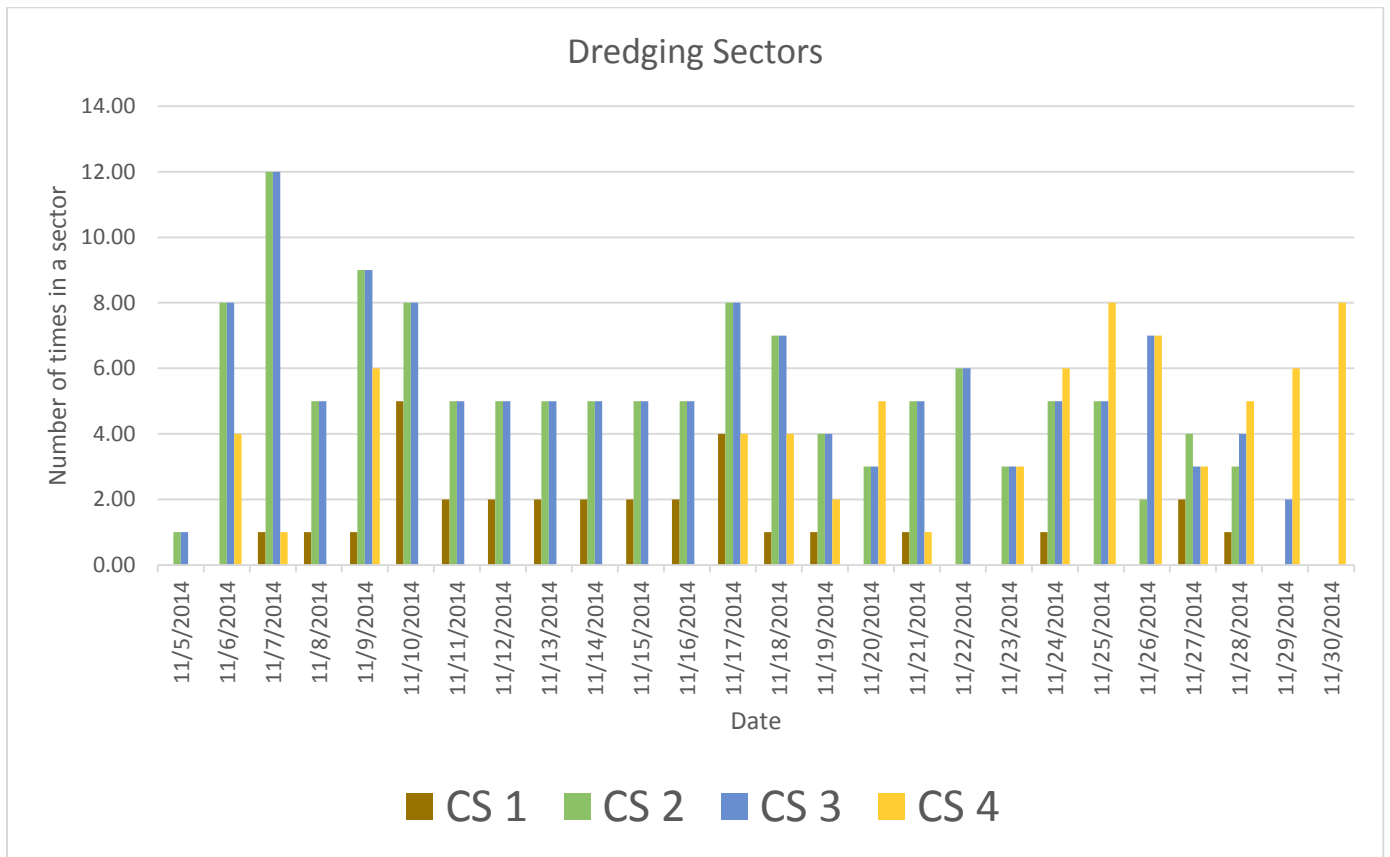
- 1) Material to be residual soil and not contain any organic material
- 2) PI > 10
- 3) 100% passing 6" (150mm) sieve, ≥ 70% passing ¾" (19mm) sieve and > 35% passing No.200 (0.075 mm) sieve
- 4) Minimum undrained shear strength = 75 kPa
- 5) Compaction water content range between +2% and +12% of OMC

Appendix D: Delay Summary

Delay Summary

Overall Delays		3405	
Date (DD/MM/YYYY)	Total Delay (minutes)	Avg. Delay (minutes)	
11/6/2014	100	13	
11/7/2014	120	17	
11/8/2014	70	8	
11/9/2014	90	18	
11/10/2014	120	13	
11/11/2014	55	11	
11/12/2014	70	7	
11/13/2014	150	19	
11/14/2014	35	7	
11/15/2014	925	231	
11/16/2014	40	10	
11/17/2014	120	17	
11/18/2014	50	10	
11/19/2014	50	10	
11/20/2014	115	13	
11/21/2014	400	50	
11/22/2014	185	46	
11/23/2014	145	18	
11/25/2014	115	10	
11/26/2014	55	8	
11/27/2014	100	14	
11/28/2014	195	24	
11/30/2014	100	25	

Appendix E: Dredging Sectors



Appendix F: Dredging Operations Summary

Date	Avg. Volume	Avg. Time to Dump Area	Avg. Turning	Total Volume	Avg. Time to dredging					Amount Earned				Minumum	Maximum
					Area	CS 1	CS 2	CS 3	CS 4	Total Time per day	Total Earned				
11/5/2014	653.00	65.00	5.00	653.00	50.00	0.00	1.00	1.00	0.00	175	\$ 3,891.88	3891.88	32,908	43,650	
11/6/2014	3147.40	63.57	15.71	10684.00	53.57	0.00	8.00	8.00	4.00	1245	\$ 63,676.64	\$ 67,568.52	32,908	43,650	
11/7/2014	4182.00	60.00	13.64	46002.00	46.82	1.00	12.00	12.00	1.00	1820	\$ 274,171.92	\$ 341,740.44	32,908	43,650	
11/8/2014	5350.00	50.00	7.50	21400.00	37.50	1.00	5.00	5.00	0.00	540	\$ 127,544.00	\$ 469,284.44	32,908	43,650	
11/9/2014	5193.33	52.78	6.11	46740.00	52.78	1.00	9.00	9.00	6.00	1460	\$ 278,570.40	\$ 747,854.84	32,908	43,650	
11/10/2014	5728.57	55.00	7.14	40100.00	51.43	5.00	8.00	8.00	0.00	1115	\$ 238,996.00	\$ 986,850.84	32,908	43,650	
11/11/2014	5588.44	54.00	9.00	50296.00	50.00	2.00	5.00	5.00	0.00	1335	\$ 299,764.16	\$ 1,286,615.00	32,908	43,650	
11/12/2014	5351.78	48.00	6.00	48166.00	37.00	2.00	5.00	5.00	0.00	1410	\$ 287,069.36	\$ 1,573,684.36	32,908	43,650	
11/13/2014	4883.88	54.00	9.00	39071.00	51.00	2.00	5.00	5.00	0.00	1285	\$ 232,863.16	\$ 1,806,547.52	32,908	43,650	
11/14/2014	4787.00	48.00	10.00	43083.00	51.00	2.00	5.00	5.00	0.00	1480	\$ 256,774.68	\$ 2,063,322.20	32,908	43,650	
11/15/2014	4790.33	48.00	11.00	14371.00	51.00	2.00	5.00	5.00	0.00	485	\$ 85,651.16	\$ 2,148,973.36	32,908	43,650	
11/16/2014	4392.38	55.00	10.00	35139.00	53.00	2.00	5.00	5.00	0.00	1245	\$ 209,428.44	\$ 2,358,401.80	32,908	43,650	
11/17/2014	4749.75	50.00	10.00	37998.00	50.00	4.00	8.00	8.00	4.00	1285	\$ 226,468.08	\$ 2,584,869.88	32,908	43,650	
11/18/2014	4818.50	49.38	10.63	38548.00	49.38	1.00	7.00	7.00	4.00	1285	\$ 229,746.08	\$ 2,814,615.96	32,908	43,650	
11/19/2014	4526.25	41.88	13.75	36210.00	49.38	1.00	4.00	4.00	2.00	1260	\$ 215,811.60	\$ 3,030,427.56	32,908	43,650	
11/20/2014	4685.00	41.88	11.88	37480.00	46.25	0.00	3.00	3.00	5.00	1270	\$ 223,380.80	\$ 3,253,808.36	32,908	43,650	
11/21/2014	4088.00	48.00	22.00	20440.00	45.00	1.00	5.00	5.00	1.00	850	\$ 121,822.40	\$ 3,375,630.76	32,908	43,650	
11/22/2014	3832.83	53.33	25.83	22997.00	52.50	0.00	6.00	6.00	0.00	1175	\$ 137,062.12	\$ 3,512,692.88	32,908	43,650	
11/23/2014	4250.50	45.00	21.67	25503.00	46.67	0.00	3.00	3.00	3.00	1025	\$ 151,997.88	\$ 3,664,690.76	32,908	43,650	
11/24/2014	4133.86	42.86	28.57	28937.00	51.43	1.00	5.00	5.00	6.00	1230	\$ 172,464.52	\$ 3,837,155.28	32,908	43,650	
11/25/2014	4186.25	45.00	13.75	33490.00	49.38	0.00	5.00	5.00	8.00	1245	\$ 199,600.40	\$ 4,036,755.68	32,908	43,650	
11/26/2014	4396.86	43.57	25.71	30778.00	47.14	0.00	2.00	7.00	7.00	1260	\$ 183,436.88	\$ 4,220,192.56	32,908	43,650	
11/27/2014	3616.33	47.50	42.50	21698.00	55.83	2.00	4.00	3.00	3.00	1160	\$ 129,320.08	\$ 4,349,512.64	32,908	43,650	
11/28/2014	3564.67	39.17	27.50	21388.00	54.17	1.00	3.00	4.00	5.00	1130	\$ 127,472.48	\$ 4,476,985.12	32,908	43,650	
11/29/2014	4535.17	45.83	13.33	27211.00	53.33	0.00	0.00	2.00	6.00	1015	\$ 162,177.56	\$ 4,639,162.68	32,908	43,650	

Appendix G: Delay Classification

Delay Classifications

Type of Delay	Number of Delay
Traffic	158
Other	2
Maintanance	2
Pilot Change	1
Breakdown	1
Total Numbers of Delays	164

Appendix H: Questions for Dredging International

- Confirm data on spreadsheet is correct
 - o All of the data on the spreadsheet is accurate
- How is volume calculated?
 - o Uses shipboard monitoring to determine density of slurry
 - o Density and flow rate is used to determine sediment volume
 - o Water does not count as volume
- When does ship sail to discharge area?
 - o Depends on when dredging becomes unproductive
 - o Generally, whenever it is full
- How does traffic effect navigation?
 - o Not as much effect as they once thought
 - o Learn to navigate around traffic
 - o Two way traffic makes it almost impossible to dredge
- Can you dredge during two way traffic time periods?
 - o It is possible but challenging
 - o Massive loss in productivity
- What is most challenging dredging aspect?
 - o Rocks
 - o Rocks cannot be picked up by trailing suction hopper dredger
- Dredging Cycle Explanation
 - o Dredge, Sail, Discharge, Repeat
- Type of discharge method
 - o Bottom door
 - o 10 minutes to completely empty
 - o Discharge strategically to not fill all dump zone area
- What times are the different shifts?
 - o Two 12-hour shifts
 - o 0:00-12:00, 12:00-24:00
- How do they know where to discharge?
 - o Computer GPS information

- Are there any uses for the sediment other than discharge?
 - o Given the type of sediment, no
 - o Loose sediment is not good for land relocation
- How does tide and current effect operation?
 - o Some areas are only reachable when it is high tide
 - o Low tide can make some area off limits

Appendix I: Contractor's Schedule

Month	Zone 1 and 1A	Zone 3B	Zone 5	Zone 6	Zone 3A	Zone 3	Zone 4
	Core	Transition	Chimney/ Blanket Filter	Chimney/ Blanket Drain	Broad- Graded Filter	Rockfill	Rip Rap
Nov-11						13,221.08	
Dec-11						62,212.13	
Jan-12						162,422.58	
Feb-12						237,017.24	
Mar-12						282,556.52	3,173.10
Apr-12						338,834.64	3,173.10
May-12						445,044.52	3,368.10
Jun-12						539,772.07	3,368.10
Jul-12						592,210.33	3,368.10
Aug-12						649,775.81	7,718.82
Sep-12						679,105.18	9,065.73
Oct-12						709,359.93	11,391.51
Nov-12						709,359.93	11,391.51
Dec-12						709,359.93	12,328.95
Jan-13						709,359.93	15,072.48
Feb-13						709,359.93	15,072.48
Mar-13						709,359.93	15,072.48
Apr-13						709,359.93	15,072.48
May-13						721,519.41	25,651.07
Jun-13						725,881.57	51,625.91
Jul-13	385.00					725,881.57	53,774.93
Aug-13	1,492.80		431.88	124.62		733,171.98	62,115.69
Sep-13	4,101.08	194.93	5,126.28	1,945.98		735,459.12	66,738.34
Oct-13	7,651.77	2,677.00	10,245.22	5,122.54		775,042.90	69,171.69
Nov-13	9,589.19	11,134.19	16,335.64	13,014.63		800,798.43	69,171.69

Dec-13	14,525.57	15,435.87	20,077.17	17,204.85		800,798.43	69,171.69
Jan-14	24,264.66	17,901.86	24,814.70	20,911.49	1,106.77	831,338.61	73,826.06
Feb-14	41,520.87	31,368.44	61,422.47	43,048.53	1,106.77	859,946.68	73,826.06
Mar-14	61,367.99	37,235.04	80,426.59	47,466.39	1,106.77	893,781.74	73,826.06
Apr-14	68,154.37	44,830.45	100,375.82	58,435.75	1,106.77	900,270.15	73,826.06
May-14	71,155.00	57,490.43	109,423.42	71,511.36	1,106.77	907,578.16	73,826.06
Jun-14	80,491.65	65,495.49	126,965.64	79,365.83	1,106.77	928,359.29	73,826.06
Jul-14	119,918.65	71,810.49	141,866.64	94,814.83	1,646.77	1,015,209.29	73,826.06
Aug-14	160,818.65	79,973.49	161,285.64	112,114.83	2,954.77	1,133,056.29	74,216.06
Sep-14	207,375.65	107,482.49	192,705.64	141,143.83	10,055.77	1,324,233.29	79,534.06
Oct-14	253,451.65	134,177.49	225,225.64	173,013.83	17,318.77	1,594,980.29	82,988.06
Nov-14	301,992.65	161,749.49	242,324.64	193,472.83	29,303.77	1,881,468.29	85,158.06
Dec-14	352,362.65	181,669.49	258,044.64	208,292.83	44,963.77	2,191,893.29	92,037.06
Jan-15	409,739.65	205,215.49	282,905.64	230,903.83	69,796.77	2,566,972.29	112,825.06
Feb-15	459,966.65	235,020.49	314,971.64	259,989.83	99,986.77	3,006,648.29	136,412.06
Mar-15						3,350,006.29	183,923.06
Apr-15							209,976.06

Appendix J: Surveyed Embankment Construction

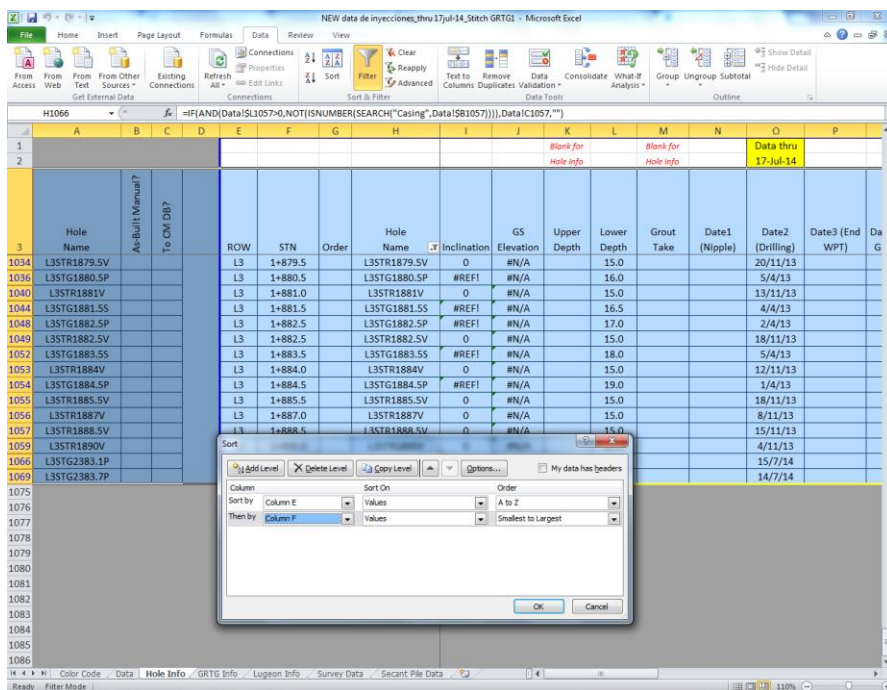
Month	Zone 1 and 1A	Zone 3B	Zone 5	Zone 6	Zone 3A	Zone 3	Zone 4
	Core	Transition	Chimney/Blank et Filter	Chimney/Blank et Drain	Broad- Graded Filter	Rockfill	Rip Rap
11-Nov						13,221.08	
11-Dec						62,212.13	
12-Jan						162,422.58	
12-Feb						237,017.24	
12-Mar						282,556.52	3,173.10
12-Apr						338,834.64	3,173.10
12-May						445,044.52	3,368.10
12-Jun						539,772.07	3,368.10
12-Jul						592,210.33	3,368.10
12-Aug						649,775.81	7,718.82
12-Sep						679,105.18	9,065.73
12-Oct						709,359.93	11,391.51
12-Nov						709,359.93	11,391.51
12-Dec						709,359.93	12,328.95
13-Jan						709,359.93	15,072.48
13-Feb						709,359.93	15,072.48
13-Mar						709,359.93	15,072.48
13-Apr						709,359.93	15,072.48
13-May						721,519.41	25,651.07
13-Jun						725,881.57	51,625.91
13-Jul	385.00					725,881.57	53,774.93
13-Aug	1,492.80		431.88	124.62		733,171.98	62,115.69
13-Sep	4,101.08	194.93	5,126.28	1,945.98		735,459.12	66,738.34
13-Oct	7,651.77	2,677.00	10,245.22	5,122.54		775,042.90	69,171.69
13-Nov	9,589.19	11,134.19	16,335.64	13,014.63		800,798.43	69,171.69
13-Dec	14,525.57	15,435.87	20,077.17	17,204.85		800,798.43	69,171.69
14-Jan	24,264.66	17,901.86	24,814.70	20,911.49	1,106.77	831,338.61	73,826.06

14-Feb	41,520.87	31,368.44	61,422.47	43,048.53	1,106.77	859,946.68	73,826.06
14-Mar	61,367.99	37,235.04	80,426.59	47,466.39	1,106.77	893,781.74	73,826.06
14- Apr	68,154.37	44,830.45	100,375.82	58,435.75	1,106.77	900,270.15	73,826.06
14- May	71,155.00	57,490.43	109,423.42	71,511.36	1,106.77	907,578.16	73,826.06
14-Jun	80,491.65	65,495.49	126,965.64	79,365.83	1,106.77	928,359.29	73,826.06
14-Jul	103,191.83	72,967.46	144,166.06	92,779.80	1,106.77	971,136.42	73,826.06
14-Aug	130,533.16	88,527.44	159,415.64	118,296.83	2,479.15	1,019,474.40	73,826.06
14-Sep	172,301.00	112,757.44	178,687.00	140,774.46	4,003.00	1,134,319.94	80,951.62

Appendix K: Instruction Manual for Generating As-Built Drawings for Stitch and Production Grouting

Step 1: Sort Injection Database by Row and then by Station

- In the database, select the “Hole Info” sheet tab at the bottom of the spreadsheet
- Press Ctrl+A to select all the data in the Hole Info sheet
- Under the “Data” tab in Microsoft Excel, click “Sort”
- The box shown will appear, click the dropdown arrow next to “Sort By” and select “Column E” which is the “Row” column.
- Click “Add Level” and select “Column F” in the dropdown menu next to “Then By”
- Repeat this process in the “GRTG Info” tab in the spreadsheet



Step 2: Create the Master Grout file

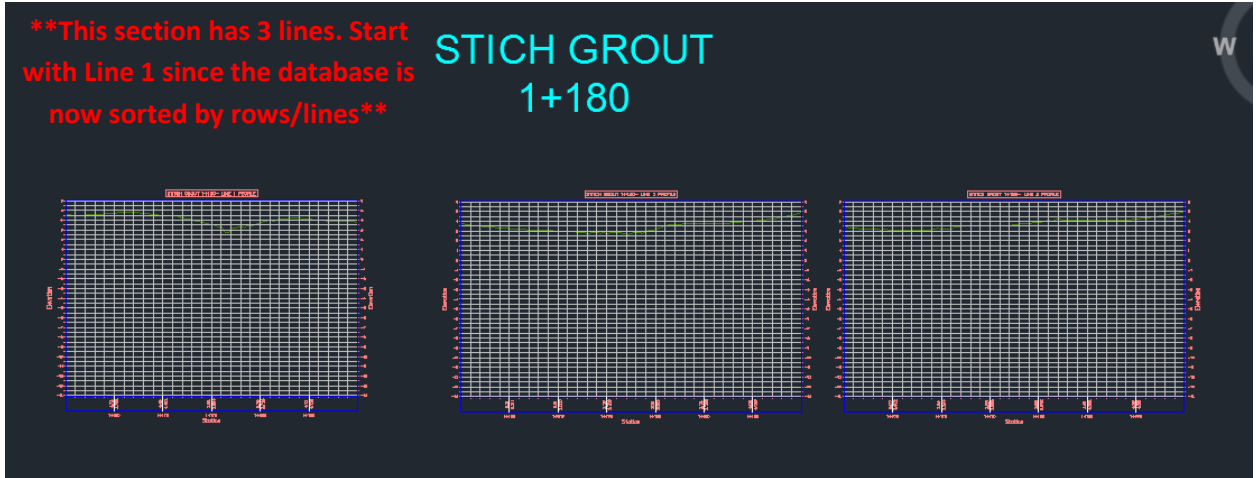
- From the original “GROUTING TEST FORMAT 02” file, right-click and select “Clipboard.” Select
- “Copy with a base point” and enter “0,0” when prompted for the base point
- In a new AutoCAD drawing, press Ctrl+V and enter “0,0” again when prompted for the base point
- Copy and paste this grid to suit the number of rows/lines. Ensure that the grids line up perfectly, one beneath the other

E. Save As “Master Grout File”

****This is the only time this step needs to be done****

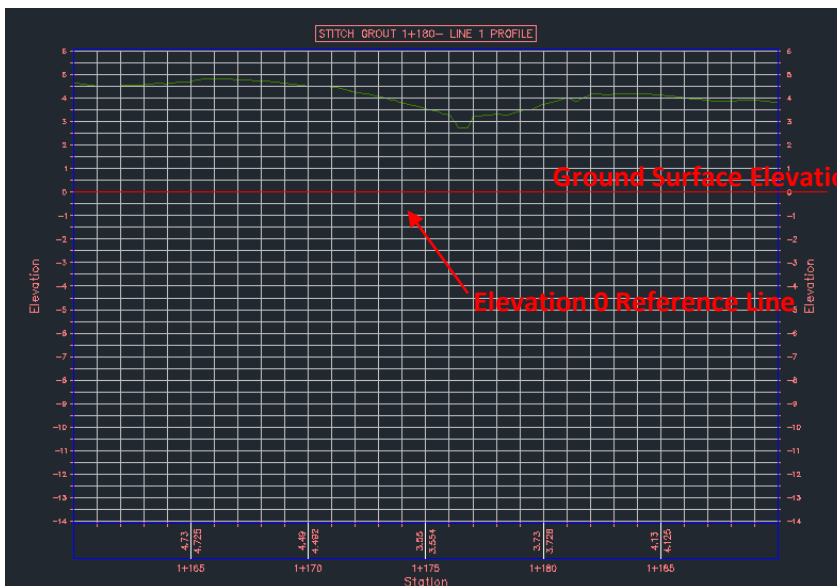
Step 3: Record Ground Surface Elevation for each hole

A. Open the AutoCAD drawing for the grouting section in which you are working on.



****There will be several profile drawings depending on how many lines/rows are in the section you are working in****

B. Draw a horizontal reference line where the elevation is 0 straight across the drawing.

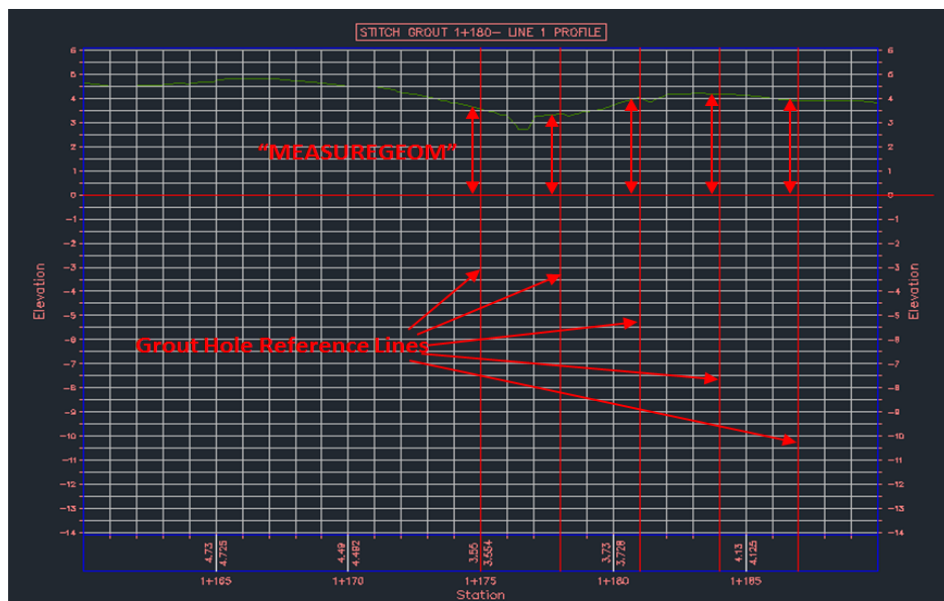


- C. Use the DFI report and database to determine which holes are present along the selected line

Hole Name	STN	Order Hole	Inclination	GS Elevation	Upper Depth	Lower Depth	Grout Take	Date1 (Nipple)	Date2 (Drilling)	Date3 (End WPT)	Date4 (End Grouting)	Date5 (Backfilling)
L1STG1175V	1+175.0	L1STG1175V	0	3.554	15.0				17/2/14			
L1STG1178V	1+178.0	L1STG1178V	0	3.393	15.0				18/2/14			
L1STG1181V	1+181.0	L1STG1181V	0	4.024	15.0				19/2/14			
L1STG1184V	1+184.0	L1STG1184V	0	4.191	15.0				11/2/14			
L1STG1187V	1+187.0	L1STG1187V	0	3.898	15.0				17/2/14			
L1STG1219V	1+219.0	L1STG1219V	0	6.659	15.0				20/2/14			
L1STG1222V	1+222.0	L1STG1222V	0	6.214	15.0				22/2/14			
L1STG1225V	1+225.0	L1STG1225V	0	5.496	15.0				13/2/14			
L1STG1226.5V									14			
L1STG1228V									13			
L1STG1229.5V									14			
L1STG1231V									14			
L1STG1234V									14			
L1STG1237V									14			
L1STG1306V	1+306.0	L1STG1306V	0	4.848	15.0				6/2/14			
L1STG1309V	1+309.0	L1STG1309V	0	4.992	15.0				8/2/14			
L1STG1310.5V	1+310.5	L1STG1310.5V	0	5.002	15.0				12/2/14			

- D. Use the “MEASUREGEOMETRY” command in AutoCAD to measure the distance between the elevation 0 reference line and the ground surface elevation line for each station in your database

TIP: Draw vertical reference lines at the stations where grout holes are present. These reference lines should cut the elevation 0 reference line and the ground surface elevation line.



- E. Enter the elevation for each hole into both the “Hole Info” and “GRTG Info” tabs in the database for the respective station

Hole Name	As-Built Manual?	To CM DB?	ROW	STN	Order	Hole Number	Inclination	GS Elevation	Upper Depth	Lower Depth	Grout Take	Date1 (Nipple)	Date2 (Drilling)	Date3 (End WPT)	Date4 (End Grouting)	Date5 (Backfilling)
L1STG1175V			L1	1+175.0		L1STG1175V	0	3.554	15.0	15.0			17/2/14			
L1STG1178V			L1	1+178.0		L1STG1178V	0	3.333	15.0	15.0			18/2/14			
L1STG1181V			L1	1+181.0		L1STG1181V	0	4.024	15.0	15.0			19/2/14			
L1STG1184V			L1	1+184.0		L1STG1184V	0	4.191	15.0	15.0			11/2/14			
L1STG1187V			L1	1+187.0		L1STG1187V	0	3.898	15.0	15.0			17/2/14			

ROW	STN	Order	Hole Number	Inclination	GS Elevation	Upper Depth	Lower Depth	Grout Take	Date1 (Nipple)	Date2 (Drilling)	Date3 (WPT)	Date4 (Grouting)	Date5 (Backfilling)
L1	1+175.0		L1STG1175V	0	3.55	15	10	0				17/2	
L1	1+175.0		L1STG1175V	0	3.55	10	5	0				17/2	
L1	1+175.0		L1STG1175V	0	3.55	5	1	0				17/2	
L1	1+178.0		L1STG1178V	0	3.33	15	10	0				18/2	
L1	1+178.0		L1STG1178V	0	3.33	10	5	0				18/2	
L1	1+178.0		L1STG1178V	0	3.33	5	1	0				18/2	
L1	1+181.0		L1STG1181V	0	4.02	15	10	0				19/2	
L1	1+181.0		L1STG1181V	0	4.02	10	5	0				19/2	
L1	1+181.0		L1STG1181V	0	4.02	5	1	0				19/2	
L1	1+184.0		L1STG1184V	0	4.19	15	10	0				11/2	
L1	1+184.0		L1STG1184V	0	4.19	10	5	0				11/2	
L1	1+184.0		L1STG1184V	0	4.19	5	1	0				11/2	
L1	1+187.0		L1STG1187V	0	3.90	15	10	0				18/2	
L1	1+187.0		L1STG1187V	0	3.90	10	5	0				18/2	
L1	1+187.0		L1STG1187V	0	3.90	5	1	0				18/2	

- F. Repeat this process for the remaining lines in the section that you are working on

Step 4: Create smaller spreadsheets with drilling and grouting information for each line

- A. Open the blank “DATA FEED GROUTING V06” file
 B. Copy and paste hole and grouting information into this table under the “GROUTHOLE INFORMATION” and “GROUTING INFORMATION” sections respectively

SOFTWARE INFORMATION				GROUTING DATA														
X	ID	COLOR	LAYER	ROW	STATION	ORDER	ID	INCLINATION	SURF. ELEV	UPPER DEPTH	LOWER DEPTH	METRIC VALUE (TAKE / LUGEON)	INFO 1 (DATE / NIPPLE)	INFO 2 (DRILL)	INFO 3 (W.TEST)	INFO 4 (GROUT)	INFO 5 (BACKFILL)	
GROUTHOLE INFORMATION																		
GH	001	GRAY	1+180L1	L1	1+175.0		L1STG1175V	0.00	3.55		15.00			17/2/14				
GH	002	GRAY	1+180L1	L1	1+178.0		L1STG1178V	0.00	3.33		15.00			18/2/14				
GH	003	GRAY	1+180L1	L1	1+181.0		L1STG1181V	0.00	4.02		15.00			19/2/14				
GH	004	GRAY	1+180L1	L1	1+184.0		L1STG1184V	0.00	4.19		15.00			11/2/14				
GH	005	GRAY	1+180L1	L1	1+187.0		L1STG1187V	0.00	3.90		15.00			17/2/14				
GROUTING INFORMATION																		
GR	101	MAGENTA	1+180L1	L1	1+175.0		L1STG1175V	0.00	3.55	15	10	0	17/2					
GR	102	MAGENTA	1+180L1	L1	1+175.0		L1STG1175V	0.00	3.55	10	5	0	17/2					
GR	103	MAGENTA	1+180L1	L1	1+175.0		L1STG1175V	0.00	3.55	5	1	0	17/2					
GR	104	MAGENTA	1+180L1	L1	1+178.0		L1STG1178V	0.00	3.33	15	10	0	18/2					
GR	105	MAGENTA	1+180L1	L1	1+178.0		L1STG1178V	0.00	3.33	10	5	0	18/2					
GR	106	MAGENTA	1+180L1	L1	1+178.0		L1STG1178V	0.00	3.33	5	1	0	18/2					
GR	107	MAGENTA	1+180L1	L1	1+181.0		L1STG1181V	0.00	4.02	15	10	0	19/2					
GR	108	MAGENTA	1+180L1	L1	1+181.0		L1STG1181V	0.00	4.02	10	5	0	19/2					
GR	109	MAGENTA	1+180L1	L1	1+181.0		L1STG1181V	0.00	4.02	5	1	0	19/2					
GR	110	MAGENTA	1+180L1	L1	1+184.0		L1STG1184V	0.00	4.19	15	10	0	11/2					
GR	111	MAGENTA	1+180L1	L1	1+184.0		L1STG1184V	0.00	4.19	10	5	0	11/2					
GR	112	MAGENTA	1+180L1	L1	1+184.0		L1STG1184V	0.00	4.19	5	1	0	11/2					
GR	113	MAGENTA	1+180L1	L1	1+187.0		L1STG1187V	0.00	3.90	15	10	0	18/2					
GR	114	MAGENTA	1+180L1	L1	1+187.0		L1STG1187V	0.00	3.90	10	5	0	18/2					
GR	115	MAGENTA	1+180L1	L1	1+187.0		L1STG1187V	0.00	3.90	5	1	0	18/2					
EOF																		

C. Save this file and name it with the following nomenclature:

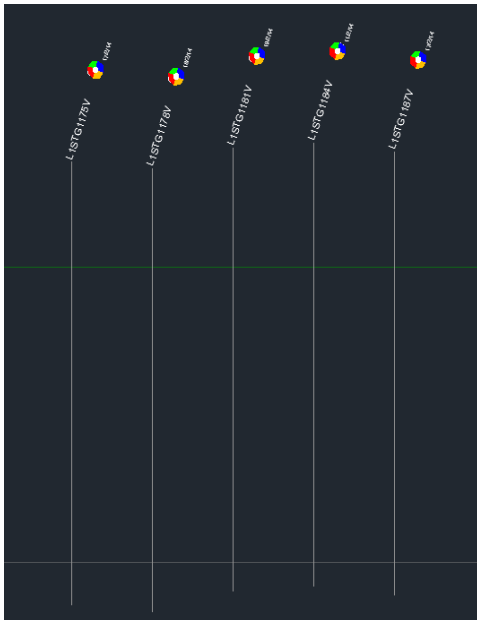
DATA FEED GROUTING V06_[Section][Line/Row]

eg. DATA FEED GROUTING V06_1+180L1

D. Repeat this process for the remaining lines in the section that you are working on

Step5: Run Visual BASIC Macro

- Open "GROUTING TEST FORMAT 02" file in AutoCAD. A grid will appear
- Run the command "VBALOAD"
- A box will appear asking you to select a program to run. Select "ProjectGRT V02"
- Click "Open"
- A box will come up asking you to enable the macros. Click "Enable Macros"
- Next, run the command "VBARUN"
- A box will appear asking to select the data model engine. Select the option ending in "GH" for grout hole.
- Return to your desktop and reopen AutoCAD
- You will then be prompted to choose a database for the program to run. Select the database that you have created in Step 3.
- The AutoCAD window will close. Wait a few seconds; reopen the AutoCAD window and the grout holes to appear on the grid.



- K. Run the command “VBARUN” again and select the data model engine ending in “GR” for grouting.
- L. Return to your desktop screen and reopen AutoCAD
- M. You will then be prompted to choose a database for the program to run. Select the database that you have created in Step 3 again
- N. Thicker, colored lines will appear on top of your grout hole lines



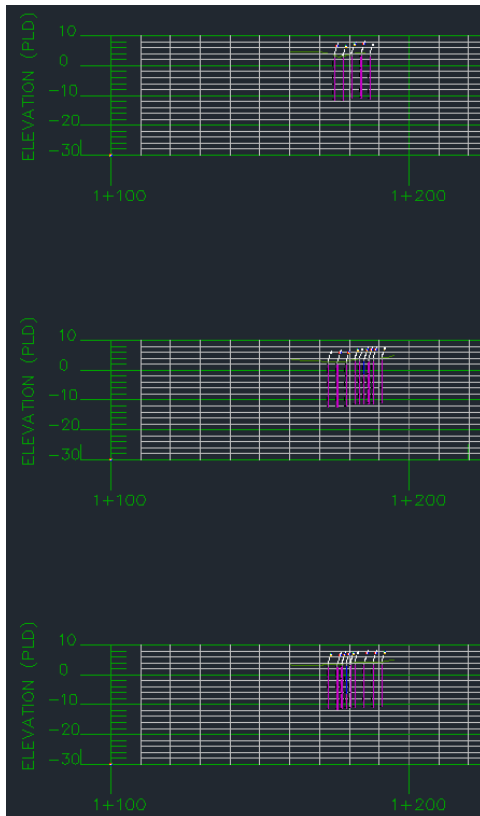
- O. “Save As” with the following nomenclature:
[Fault Number][Line/Row Number]

Eg. 1+180L1

Step 6: Populate the Master Grout file

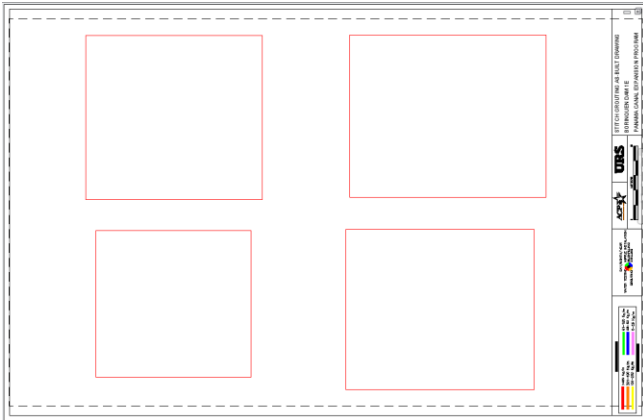
- A. In the file just created, run the command “EXPLODE”, click and drag from the left to the right to select the grout holes and their labels only.
- B. Right-click and select “Clipboard.” Select “Copy with a base point” and select a point which can be identified in both the current drawing and the Master Grout file
- C. In the Master Grout file press Ctrl+V and enter the same point previously selected when prompted for the base point
- D. Repeat this process for the remaining lines in the section that you are working on

****It is advised that you separate these grout holes by lines/rows****

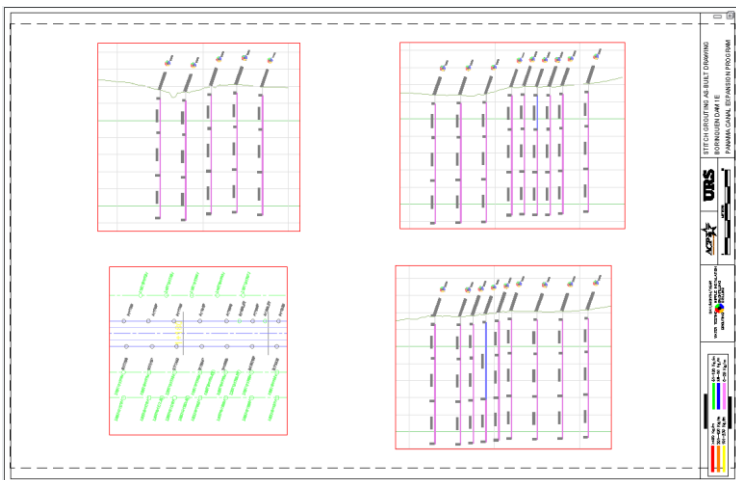


Step 7: Layout drawings on a 22"x34" sheet

- A. Open the URS Corp. Inc. title block
- B. Create a viewport layer and ensure that it is not printable in the Layer Manager
- C. Make this your current layer by double-clicking on it in the Layer Manager
- D. Under the Layout tab, create enough rectangular viewports in paper space to accommodate all the lines/rows necessary plus another viewport for the plan view



- E. Activate model space
- F. Run the function "XREF. Right-click in the box that appears, and select "Attach DWG" then scroll to the Master Grout file and the Grout Holes Plan View file
- G. Place these files anywhere in Model Space
- H. Return to Paper Space
- I. Activate each viewport and toggle until the appropriate line is within the viewport extents. You may need to alter the extents of the viewport to fit the drawing.
- J. Double click in these viewports and use the function "CH" to change the scales to 1:100



Appendix L: As-Built Drawings Produced

