Naalehu Large Capacity Cesspool Closure Project

Revised Preliminary Engineering Report

Prepared for

County of Hawaii, Department of Environmental Management

August 2023

2261 Aupuni Street, Suite 201
Wailuku, Maui, HI 96793

T: 808.244.7005

August 31, 2023

Ms. Brenda Iokepa-Moses
County of Hawaii Wastewater Division
108 Railroad Avenue
Hilo, HI 96720 153740.008

Subject: Naalehu Wastewater Treatment Plant Revised Preliminary Engineering Report

Dear Ms. Iokepa-Moses:

Brown and Caldwell (BC), in association with Engineering Partners, Inc. (EPI) is pleased to present the attached Revised Preliminary Engineering Report (PER) for the Naalehu Large Capacity Cesspool (LCC) Closure Project. Preparation of a Revised PER is required by the Revised Administrative Order on Consent (AOC) that became effective on August 22, 2022. The need for a Revised PER was precipitated by several items:

- A wastewater treatment process with a smaller and shallower footprint is preferable due to the presence of lava rock at the proposed wastewater treatment plant (WWTP) site. Mechanical treatment technologies in the form of package plants offer the opportunity to achieve these goals.
- The community, through community meetings and outreach, has not been receptive to the aerated lagoon technology that was formerly proposed.
- The Revised AOC allows the possibility of implementing individual wastewater systems (IWS) to close the LCCs.

The Revised AOC requires evaluation of four feasible options:

i. A package plant and new collection system.
ii. A package plant connected to the existing collection system.
iii. A maintenance contract model IWS program.
iv. An operating permit model IWS program.

This Revised PER consists of three parts:

- This introductory summary that provides comparisons of the four feasible options.
- Part A, by BC, which presents updated analysis of feasible options i and ii that are based on using a package plant based WWTP to service the Naalehu community and close the LCCs. BC is a nation-wide environmental engineering firm with local Hawaii offices located in Kamuela, Wailuku, and Honolulu. BC has been planning and designing WWTPs throughout the United States for over 75 years.
- Part B, by EPI, which presents a detailed analysis of feasible options iii and iv that are based on using IWS to service the Naalehu community and close the existing LCCs. EPI is a multi-discipline engineering and design firm based in Hilo. EPI has successfully designed IWS systems on Hawaii Island and is well-versed to address implementing IWS in the unique local soil and subsurface geological conditions in Naalehu.

Throughout this Revised PER the following terms are used:

> "Feasible options" refers to the four specific options (i, ii, iii, iv) listed above and in paragraph V.A.31.a of the Revised AOC.
>
> "Alternatives" and "project alternatives" refer to various combinations of systems or technologies that are evaluated within this Revised PER to determine preferences for the feasible options.

1. Comparison of Feasible Options

The four feasible options are compared below,

1.1 Protection of Human Health and the Environment

Table 1 compares the four feasible options with respect to protection of human health and the environment. The State of Hawaii Department of Health (DOH) regulates both WWTPs and IWS. All four feasible options are protective of human health and the environment when implemented in accordance with the applicable Hawaii Administrative Rules (HAR). Additional discussion is provided in Parts A and B.

Table 1. Protection of Human Health and the Environment

Feasible Option	Regulatory Authority	Variances	Protective of Human Health and the Environment?
i. Package plant and new collection system	HAR 11-62, Subchapter 2	Variance required by DOH for WWTP flow capacity	Yes
ii. Package plant connected to the existing collection system	HAR 11-62 Subchapter 2	Variance required by DOH for WWTP flow capacity	Yes
iii. A maintenance contract model IWS program	HAR 11-62, Subchapter 3	Variances may be required for some lots for setback distances, etc.	Yes
iv. An operating permit model IWS program	HAR 11-62, Subchapter 3	Variances may be required for some lots for setback distances, etc.	Yes

1.2 Capital Cost Comparison of Feasible Options

Table 2 summarizes the capital costs for the four feasible options. Details of the capital cost estimates are provided in Parts A and B. Note that the IWS capital costs per lot are presented as ranges; greater precision will not be available until designs are complete due to the site-specific nature of IWS implementation on existing developed properties.

Table 2. Capital Cost Comparison		
Feasible Option	Capital Cost ($ million)	Cost per Lot
i. Package plant and new collection system	$84.3	$413,000 [a]
ii. Package plant connected to the existing collection system	$74.2	$364,000 [a]
iii. A maintenance contract model IWS program	$5.8 - $29.1	$30,000 - $150,000 [b]
iv. An operating permit model IWS program	$5.8 - $29.1	$30,000 - $150,000 [b]

a. Based on a total of 204 lots in the WWTP service area.

b. Based on a total of 194 lots converted per the AOC.

As shown in the table, the IWS feasible options incur significantly lower capital costs than the package plant alternatives.

1.3 Long-Term Recurring Costs Comparison

Long-term recurring costs include operations and maintenance (O&M) costs; examples include labor, electricity, chemicals, and maintenance materials like spare parts. Another recurring cost is the need to replace and refurbish (R&R) equipment or systems when they reach the end of their useful life.

A summary of the long-term recurring costs for the four feasible options is presented in Table 3 for comparison. Additional detail is included in Parts A and B.

Table 3. Long-Term Recurring Costs Comparison		
Feasible Option	Annual O&M Cost [a]	R&R Cost (after 20 years) [a]
i. Package plant and new collection system	$886,000	$6,000,000
ii. Package plant connected to the existing collection system	$1,050,000	$6,000,000
iii. A maintenance contract model IWS program	$250,000	$5,800,000
iv. An operating permit model IWS program	$340,000	$5,800,000

a. Expressed in current (2023) dollars.

1.4 Life-Cycle Cost Comparison

An economic evaluation was prepared to assess the potential life-cycle costs associated with each project alternative. The economic evaluation consists of a net present value comparison. The net present value analysis includes capital, O&M, and R&R costs. An appropriate inflationary factor and discount rate are applied to obtain the net present value over a 30-year planning period. The net present value of an alternative represents the amount of money that would need to be set aside today (at a given interest rate) to pay the costs associated with the alternative over the entire planning period. The alternative with the lowest net present value is considered the most attractive from an economic perspective. Table 4 summarizes the life-cycle cost evaluation assumptions.

Table 4. Life-Cycle Economic Assumptions	
Description	**Value**
Year of analysis	2023
Planning period	30 years
Inflation rate	3.5 percent
Discount rate	5.0 percent
R&R cycle	20 years

Table 5 presents the life-cycle cost evaluation results.

Table 5. Life-Cycle Cost Evaluation Results			
Alternative	**Capital Cost ($M)**	**Net Present Value of O&M and R&R Costs ($M)**	**Life-Cycle Cost ($M)**
i. Package plant and new collection system	$84.3	$ 25.4	$ 109.8
ii. Package plant connected to the existing collection system	$74.2	$ 29.3	$ 103.5
iii. A maintenance contract model IWS program	$ 29.1	$ 10.2 a	$ 39.3
iv. An operating permit model IWS program	$ 29.1	$ 12.3 [a]	$ 41.4

a Includes replacement costs and IWS O&M costs paid directly by homeowners.

Figure 1 shows the results graphically. The IWS alternatives have significantly lower life-cycle costs than the package plant alternatives.

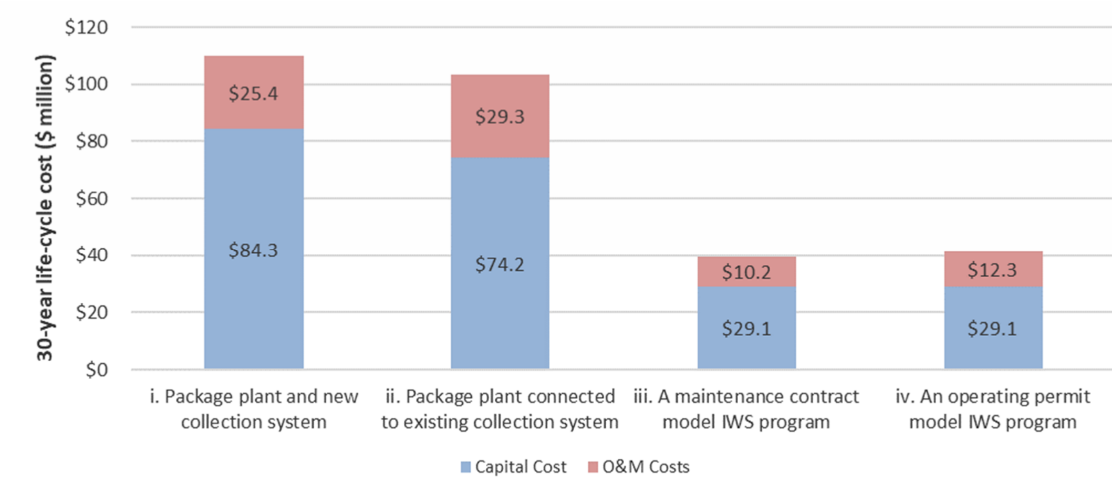


Figure 1. Life-Cycle Cost Comparison

Figure 2 shows the cumulative cash flow projections for the four feasible alternatives over the planning period, expressed in current (2023) dollars.

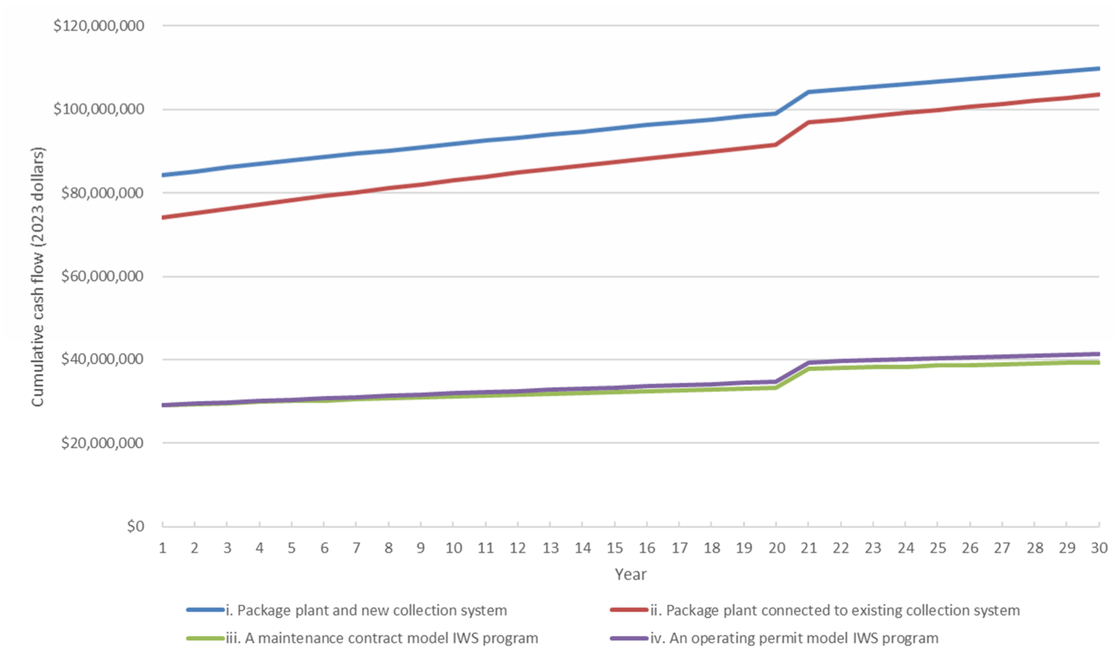


Figure 2. Cumulative Cash Flow Projections

1.5 Schedule

The Revised AOC requires the LCCs be closed no later than December 31, 2027. Parts A and B include preliminary assessments of implementation schedules. Table 6 provides a summary of the preliminary implementation schedule assessments. As discussed in Part A, it will be difficult to implement the WWTP approach to close the LCCs by the deadline, due to entitlement processes, environmental review, land acquisition, and materials supply challenges currently facing the Hawaii construction industry. A design/build approach could potentially reduce the implementation timeframe if equipment procurement and fabrication can occur in parallel with design. However, compliance with the Revised AOC deadline will be a significant challenge with Feasible Options i and ii without a schedule extension.

Per Part B, the IWS approach may be able to be implemented by the Revised AOC deadline. The IWS approach assumes that the County can address Hawaii Revised Statues (HRS) 343 environmental review requirement via an exemption, and that property access and any alterations to County regulations deemed necessary by the County are achievable within the timeframe.

Table 6. Summary of Preliminary Implementation Schedule Assessments				
Description	Feasible Options			
	i. Package Plant New Collection System	ii. Package Plant Existing Collection System	iii. Maintenance Contract Model IWS Program	iv. Operating Permit Model IWS Program
Entitlements and permitting	Q4 2024	Q4 2024	Q2 2024	Q2 2024
Design and construction	Q3 2028	Q3 2028	Q2 2026	Q2 2026
Estimated LCC closure	Q3 2028	Q3 2028	Q3 2026	Q3 2026
Revised AOC LCC closure milestone	December 31, 2027			
Risk of missing Revised AOC LCC closure milestone	High	High	Moderate	Moderate

Note: Q = quarter

2. Revised AOC References

The Revised AOC paragraph V.30.A.a lists information that must be included in this Revised PER. Table 7 provides references to the information within.

Table 7. Revised AOC Paragraph V.30.A.a Checklist				
Revised AOC Paragraph V.30.A.a Description	Report Reference Section for Feasible Options			
	i. Package Plant New Collection System	ii. Package Plant Existing Collection System	iii. Maintenance Contract Model IWS Program	iv. Operating Permit Model IWS Program
Description of project details for each feasible option	Part A, §2.3, §2.4, and §8	Part A, §2.3 and §8	Part B, §10	Part B, §10
Planning area description	Part A, Fig. 2-1	Part A, Fig. 2-1	Part A, Fig. 2-1	Part A, Fig. 2-1
Planning period	Part A, §7.2.3	Part A, §7.2.3	Part A, §7.2.3	Part A, §7.2.3
Description of planning phases	Part A, §7.2.3	Part A, §7.2.3	Part A, §7.2.3	Part A, §7.2.3
Owner and operator of facilities	Part A, §1-1	Part A, §1-1	County/In-house or 3rd-party service provider	Homeowner/3rd-party service provider
Location of facilities (including a map)	Part A, Fig. 2-1, Fig. 8-1	Part A, Fig. 2-1, Fig. 8-1	Part A, Fig. 2-1	Part A, Fig. 2-1
Design parameters for each feasible option	Part A, §2.3, §2.4, and §8	Part A, §2.3, and §8	Part B, Table 1.10	Part B, Table 1.10
Major unit processes:	Part A, §5	Part A, §5	Part B, §10	Part B, §10
Flow diagram	Part A, Fig. 8-2	Part A, Fig. 8-2	Part B, Fig. 1.1	Part B, Fig. 1.1
Pipe lengths, sizes, and locations	Part A, Appendix D	Part A, Appendix D	Part B, Appendix F	Part B, Appendix F
Design criteria	Part A, §8.3	Part A, §8.3	Part B, §6	Part B, §6
Project costs	Part A, §7	Part A, §7	Part B, §11, Appendix B	Part B, §11, Appendix B

3. Conclusions

From a technical perspective, all four feasible options represent viable ways to close the LCCs in Naalehu and will be protective of human health and the environment. The IWS feasible options (iii and iv) present significantly lower capital, long-term recurring, and life-cycle costs than the WWTP feasible options (i and ii). In addition, the IWS feasible options (iii and iv) offer greater potential to meet the LCC closure deadline contained within the Revised AOC. We anticipate that financing options, community engagement, burden on the Naalehu community, and alignment with County long term planning goals will need to be considered outside of this preliminary engineering report.

The County provided the following statement:

"COH will continue working with, planning department, community leaders, and developers for long term accomplishments of the CDP as this will take more time and effort than what was given within the AOC. Better planning decisions can be made as the COH can continue monitoring growth, work with planning department on the general plan 2045, finalize the cesspool conversion master plans, and be able to negotiate a better financial plan with all stakeholders aka developers, businesses, state government agencies, and the community at large."

Brown and Caldwell appreciates that the County has requested our services in assisting with this project. Should you have any questions, please do not hesitate to call Craig Lekven at 808.442.3301.

Very truly yours,

Brown and Caldwell

Craig C. Lekven, P.E., Project Manager
Wailuku, Hawaii

PART A: WWTP Approach

Part A
Naalehu Wastewater Treatment Plant
Revised Preliminary Engineering Report

Prepared for

County of Hawaii, Department of Environmental Management

August 2023

THIS WORK (PART A) WAS PREPARED BY ME OR UNDER MY SUPERVISION

April 30, 2024

Signature — *Expiration Date of the License*

CRAIG CARSTEN LEKVEN
LICENSED PROFESSIONAL ENGINEER
No. 13003-C
HAWAII, U.S.A.

Brown AND Caldwell

Table of Contents–Part A

List of Figures

List of Tables

List of Abbreviations

AACE	Advancement of Cost Engineering International
AOC	Administrative Order on Consent
BC	Brown and Caldwell
BOD_5	5-day biochemical oxygen demand
CCH	City and County of Honolulu
CDP	Kau Community Development Plan
CWA	Clean Water Act
DB	design/build
DBA	District Boundary Amendment
DBB	design/bid/build
DNA	deoxyribonucleic acid
DOH	Hawaii Department of Health
ELLF	end-of-lamp-life
FAI	Fukunaga & Associates, Inc.
ft^2	square feet
ft^3	cubic feet
GAC	granular activated carbon
gpcd	gallons per capita per day
gpd	gallons per day
gpm	gallons per minute
H_2S	hydrogen sulfide
HAC	Hawaiian Agriculture Company
HAR	Hawaii Administrative Rules
I/I	Infiltration and inflow
L	Liter
lbs	pounds
LCCs	large capacity cesspools
LPHO	low pressure high output
LUC	Land Use Commission
MBR	membrane bioreactor
Mgal	million gallons
mgd	million gallons per day
mL	milliliter
MLSS	mixed liquor suspended solids
NaOCl	sodium hypochlorite
N	nitrogen
NPDES	National Pollutant Discharge Elimination System
NPV	net present value
NRCS	Natural Resources Conservation Service
O&M	operation and maintenance
PER	Preliminary Engineering Report
psi	pounds per square inch
PWWF	peak wet weather flow
RGF	recirculating gravel filter
RNA	ribonucleic acid
SBRs	sequencing batch reactors
SES	sand equivalent size
SR	slow rate
SRT	solids retention time
TMK	tax map key
TN	total nitrogen
TSS	total suspended solids
UIC	Underground Injection Control
USEPA	United States Environmental Protection Agency
USSC	United State Supreme Court
UV	ultraviolet
WQS	water quality standards
WWPS	wastewater pump station
WWRF	Wastewater Reclamation Facility
WWTP	Wastewater Treatment Plant

Section 1

Introduction

This section summarizes the project background and describes key elements of the existing Naalehu wastewater collection system.

1.1 Background

The town of Naalehu is located in the Kau district of the Island of Hawaii. According to the 2020 United States Census, the town population is approximately 1,007 persons.

The Naalehu community was established as the result of the sugar operations of the C. Brewer Company. A portion of the community is serviced by a sewer system that was privately built, owned, and operated by the C. Brewer Company, which merged with Hawaiian Agriculture Company (HAC) in 1972. The wastewater collected by the sewer system discharges into large capacity "gang" cesspools (LCCs). Many years after its establishment, the private sewer system ownership was conveyed to the County of Hawaii Department of Environmental Management.

In 1998, the U.S. Environmental Protection Agency, promulgated regulations, 40 Code of Federal Regulations 144.14, that require the elimination of LCCs. Options to close the LCCs include construction of a new sewer collection system located within public right-of-way and replacement of the existing LCCs with a wastewater treatment plant (WWTP) to address the wastewater treatment and disposal needs of the Naalehu community. These centralized WWTP options are the subject of this Preliminary Engineering Report (PER). A separate report is being concurrently prepared that evaluates additional options using individual wastewater systems in lieu of a new collection system and WWTP to close the LCCs.

This report revises the 2018 PER for the Naalehu WWTP and summarizes the proposed facilities needed to treat and dispose of wastewater flow currently discharged to the LCCs, plus additional sewer connections. The PER presents the existing and estimated future flows and loads to the treatment plant, describes the proposed treatment processes, recommends needed upgrades for the WWTP to meet the future treatment levels, and provides an initial opinion of the cost to construct, operate, and maintain the improvements project.

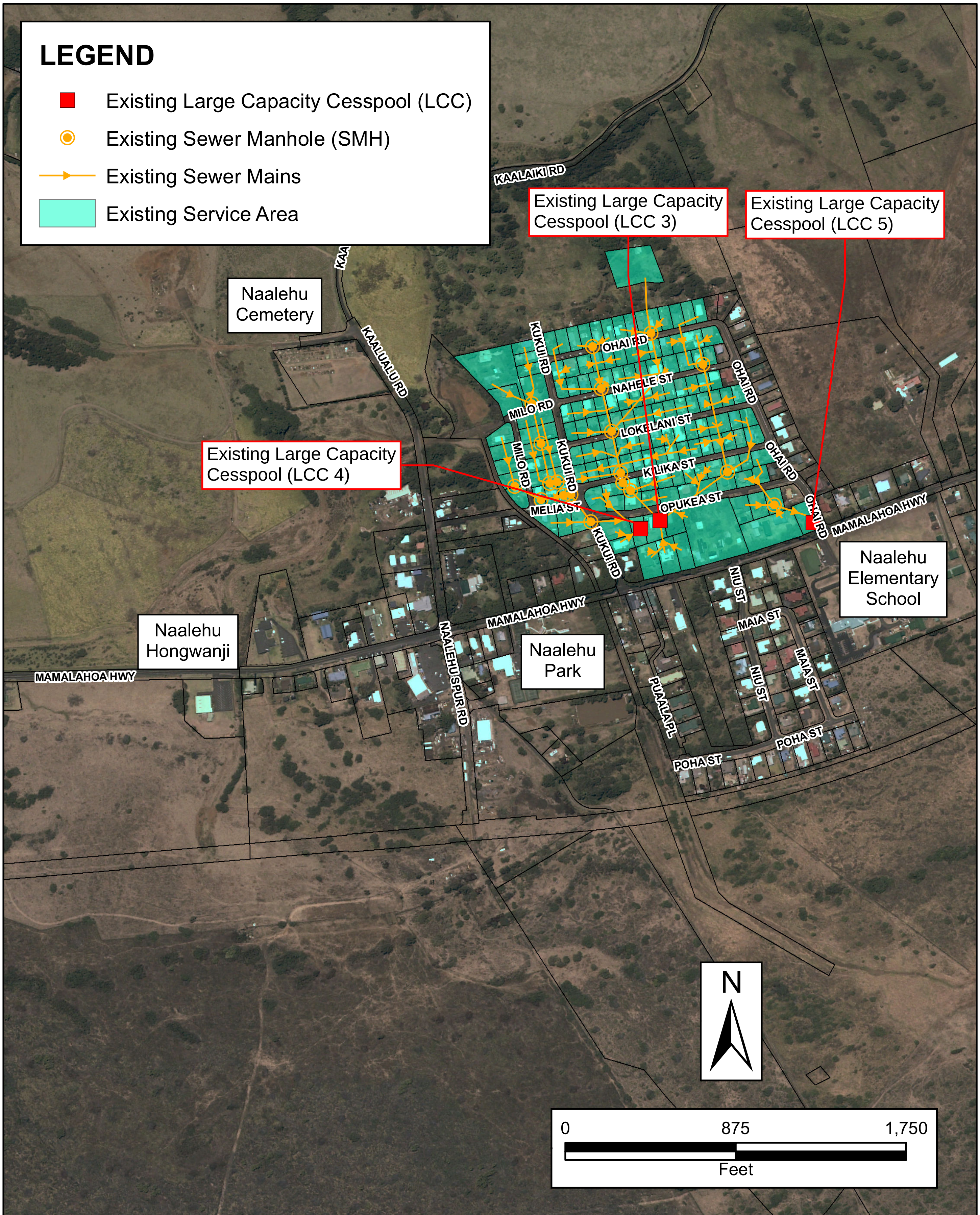
1.2 Existing System

The existing collection system is a network of gravity sewers that discharge to three existing LCCs. Figure 1-1 shows the existing Naalehu wastewater collection system and service areas for the LCCs. The LCCs in Naalehu are numbered 3, 4, and 5; LCCs 1 and 2 are located in the nearby town of Pahala. A detailed analysis of the existing wastewater collection system was completed by others (M&E Pacific, December 2004). The report concluded that the Naalehu community existing sewer system consists of approximately 5,288 linear feet of 6-inch-diameter and 15,500 linear feet of 4-inch-diameter pipelines. Residential laterals connect to 4-inch sewers that discharge into 6-inch sewer mains, predominately found in easements on private property, which transmit wastewater to the three LCCs. There are approximately eight manholes in the sewer system. More recently available information notes the size of piping to be between 3 and 8 inches with a few additional sewer manholes (Fukunaga and Associates, Inc., June 2013). There are no pump stations, and the system is not designed to collect stormwater.

1.3 Report Contents

The remainder of the revised PER is organized as follows:

- Section 2 describes the service area and collection system options.
- Section 3 describes flow and load projections for the new Naalehu WWTP.
- Section 4 evaluates potential options for effluent management and details the treatment requirements for the preferred option.
- Section 5 describes the evaluations conducted in support of the preliminary design of the Naalehu WWTP.
- Section 6 summarizes the solids management approach.
- Section 7 describes alternative treatment options under consideration.
- Section 8 provides a preliminary design for the proposed improvements.
- Section 9 presents the proposed implementation plan for the new WWTP.



Brown AND Caldwell	SCALE AS SHOWN JOB NO.: 153740	NAALEHU WASTEWATER TREATMENT PLANT Existing Naalehu Wastewater System	FIGURE 1-1

Section 2

Collection System

This section summarizes the existing and new collection system options for the Naalehu service area (LCC Closure Project compliance area), which was investigated by Fukunaga and Associates, Inc. (FAI) in May 2020 in efforts to close the existing large-capacity cesspools (LCCs). The figures and collection system layouts included within this section are reproduced from the 2020 FAI report, which was prepared in accordance with the original 2017 AOC. Should a new WWTP be the preferred option for implementation, an update to the collection system layout is anticipated to finalize the service area in accordance with the 2022 Revised AOC. FAI's Naalehu report is provided in Appendix D.

2.1 Service Area

A concrete flood canal divides Naalehu into East (Hilo) and West (Kona) sides. The majority of the Hilo side properties are residential, while majority of Kona side properties are commercial. Within the town of Naalehu, there is an existing wastewater collection system that services approximately 164 former C. Brewer Company (Brewer Company) house lots on the mauka (mountain) side of Mamalahoa Highway. The collection system is currently located within easements in private properties and is discharged to three LCCs.

The Kau Community Development Plan (CDP) indicates the sewer system may eventually be expanded to service the entire community; however, the collection system and wastewater treatment plant (WWTP) presented in this report will service the former Brewer Company properties currently connected to the LCCs and properties adjacent to the new collection system, including three properties requesting connection. Therefore, the proposed service area constitutes the LCC Closure Project compliance area.

Figure 2-1 shows the service area for the new WWTP, including the newly accessible properties.

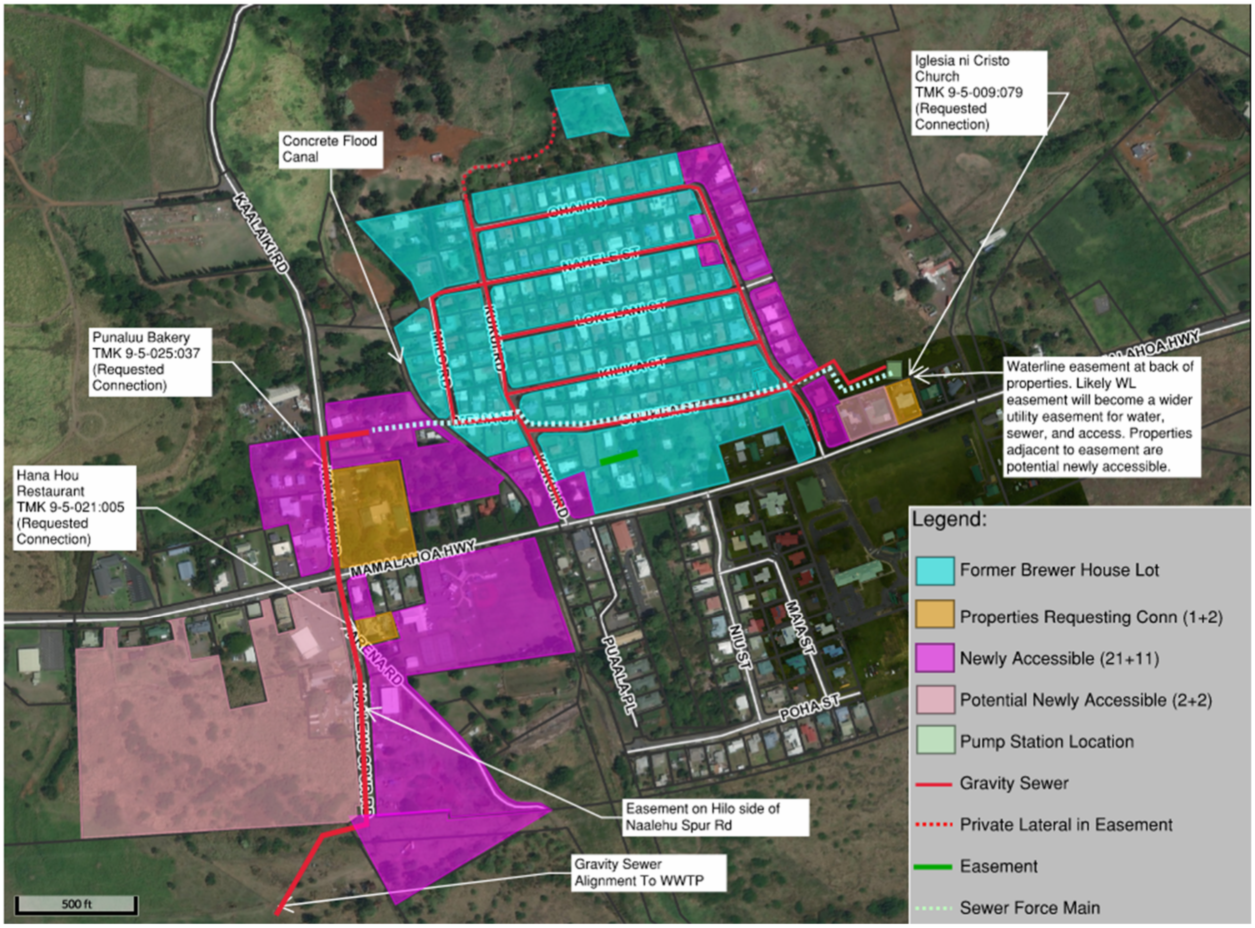


Figure 2-1. Naalehu WWTP Service Area

Table 2-1 provides a summary of the WWTP service area (LCC Closure Project compliance area) property types.

Table 2-1. Naalehu WWTP Service Area Summary

Property Type	Number of Parcels		
	Existing C Brewer	Newly Accessible	Total
Residential	159	25	184
Commercial	2	7	9
Church	1	1	2
Industrial	-	2	2
Agricultural	1	2	3
Residential/Commercial	1	1	2
Residential/Agricultural	-	1	1
Park	-	1	1
Total	164	40	204

Note: Service area = LCC Closure Project compliance area

2.2 Existing Collection System

In 2004, Brewer Company contracted M&E Pacific to perform a sewer system evaluation for the town of Naalehu. The results of this investigation determined that the existing sewer lines and manholes do not conform to the County sewer design standards. The existing sewer system was not constructed in the streets, but instead runs through easements located on private properties, with many collection lines running adjacent to or beneath the houses. The results of a smoke test performed during the 2004 sewer system evaluation identified at least 13 locations of line breaks and/or pipe defects and 12 household units with defective sewer vents. In addition, the existing sewer system is over 80 years old, long surpassing its expected lifespan, and will require extensive repair and rehabilitation if chosen to be reused.

The recommended alternative, which received overwhelming support from Naalehu voters in 2004, consists of constructing a new sewer system in the streets to meet the County sewer standards and to allow the collection system to be owned and operated by the County (M&E Pacific, December 2004).

Nearly 20 years have passed since the 2004 study was completed. In order to reuse the existing collection system into the future an updated condition assessment study is recommended to better identify system deficiencies. Substantial improvements will likely be necessary due to the age of the system. Reusing the existing collection system will require constructing the Phase 1 collection system scenario described below to connect to the proposed WWTP and close the LCCs.

2.3 Phase 1 Collection System Project

In efforts to close the existing LCCs, FAI conducted a collection system investigation in May 2020. The investigation recommends a two-phase approach. Phase 1 involves utilizing the existing collection system within the Brewer Company house lots and constructing new gravity sewers, wastewater pump station, and force main to transport sewage from the LCCs to the new WWTP. Phase 1 consists of the following:

1. Construct a new gravity sewer on Kaalaiki Road and Naalehu Spur Road to the WWTP located on a portion of Tax Map Key (TMK) (3) 9-5-007:016.
2. Construct a new pump station located on a portion of TMK (3) 9-5-008:048, and construct a new force main, which crosses an existing storm drainage channel at Melia Street, to connect to the Kaalaiki Road gravity sewer.
3. Construct a new gravity sewer on Opukea Street and Ohai Road to intercept existing flows entering the LCCs and divert sewage to the wastewater pump station (WWPS) and transport flows to the gravity sewer along Kaalaiki Road.

Figure 2-2 illustrates the transmission system layout established by FAI in May 2020 for the Phase 1 collection system.

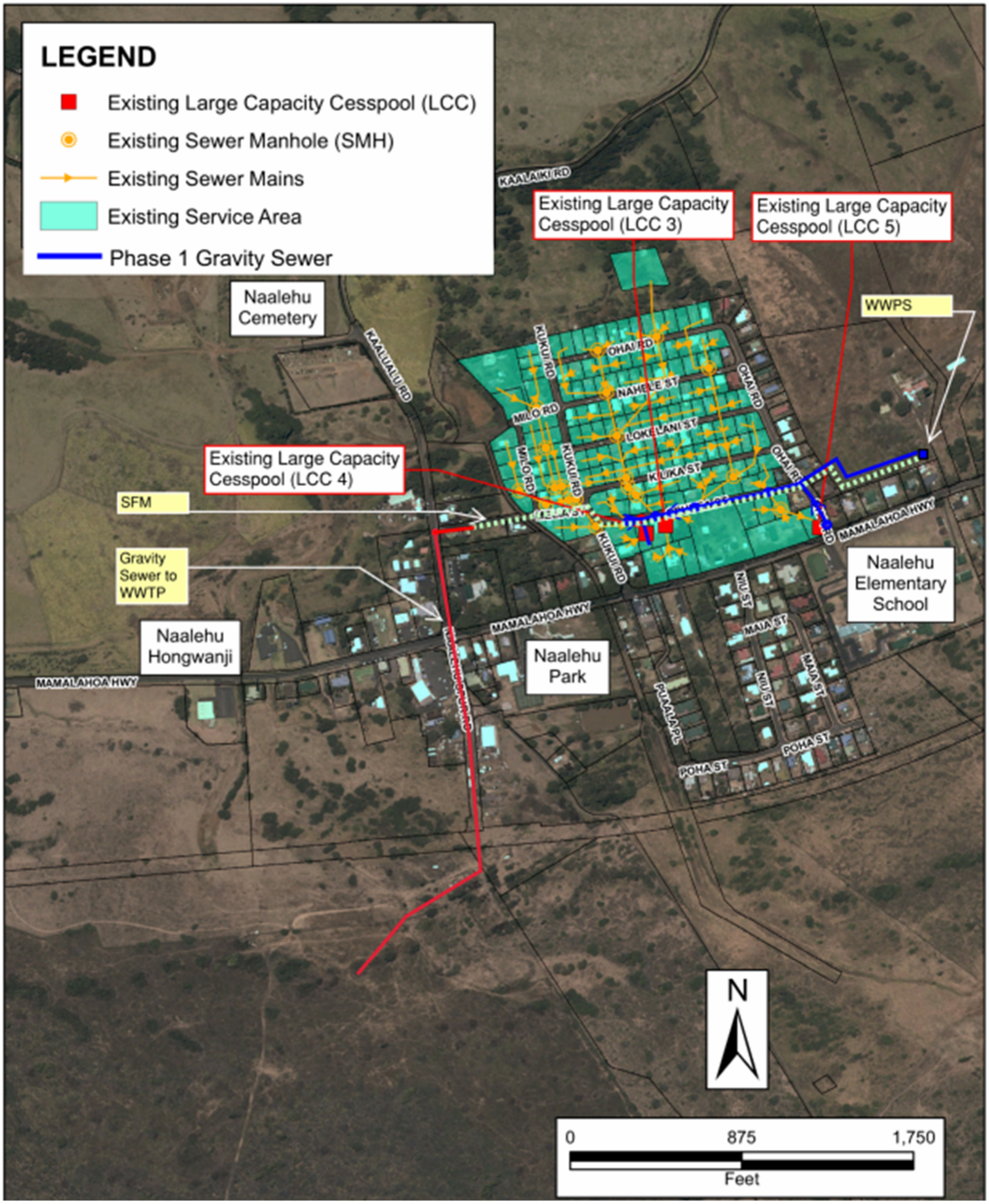


Figure 2-2. Naalehu Phase 1 Collection System Layout

2.4 Phase 2 Collection System Project

The Phase 2 collection system project consists of the installation of gravity sewers within the streets to replace the existing collection system. Figure 2-3 shows the former Brewer Company housing area Phase 2 collection system established by FAI in May 2020.

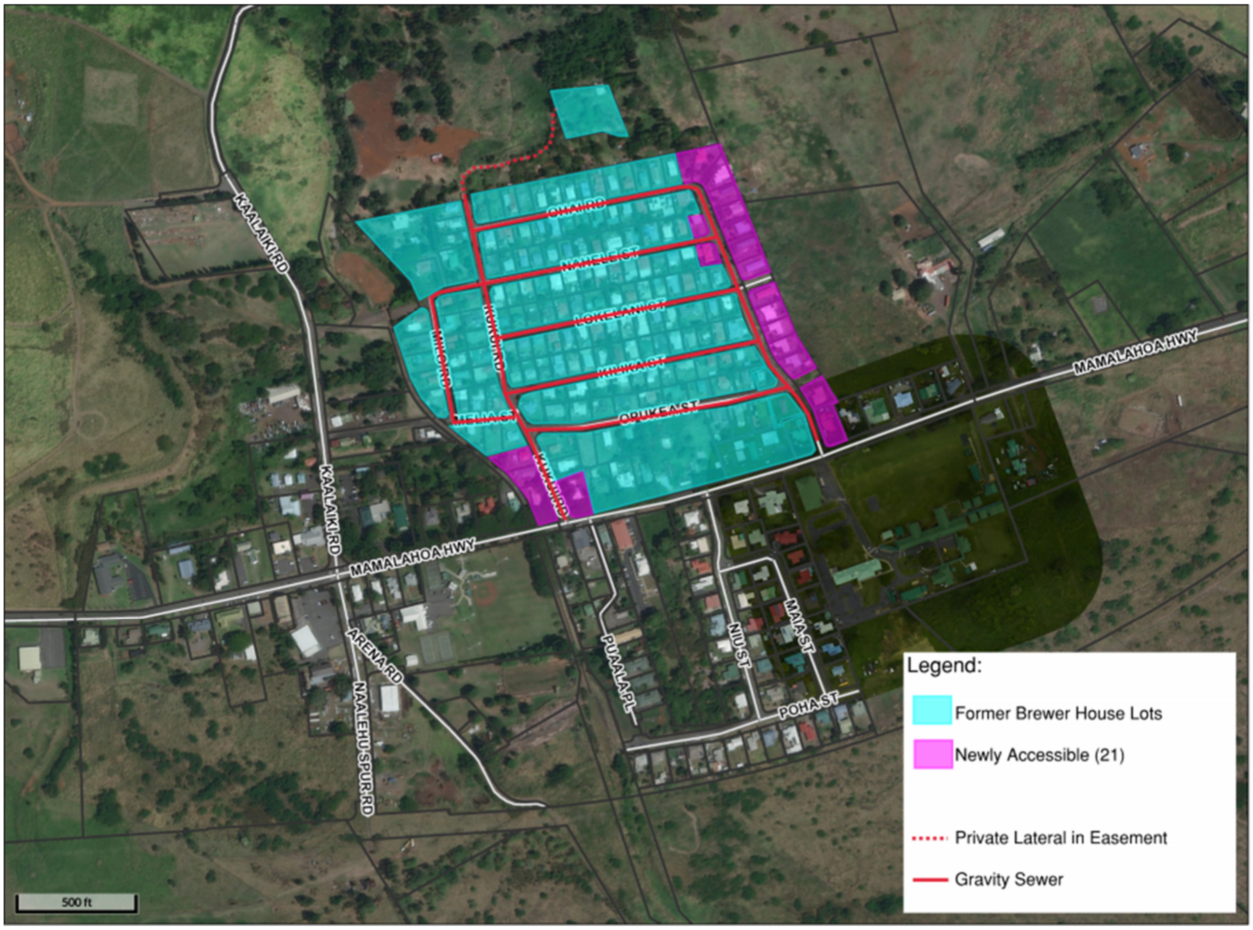


Figure 2-3. Naalehu Phase 2 Collection System Layout–Former Brewer Company House Lots

2.5 Collection System Alternatives

Two collection system alternatives were evaluated for Naalehu:

- **Phase 1 Only: Reuse Existing Collection System:** This alternative would construct the Phase 1 project to close the LCCs. The existing collection system would continue to be used.
- **Phase 1+2: All New Collection System:** This alternative would construct both Phase 1 and Phase 2 to create an all-new collection system for the community.

2.6 Collection System Costs

Table 2-2 summarizes the capital, operation and maintenance (O&M), equipment replacement, and life-cycle costs for the two collection system alternatives. The capital costs were calculated by FAI in their Naalehu Wastewater Collection System Improvements Technical Memorandum (Appendix D) conducted in May 2020, adjusted to current (June 2023) dollars.

Phase 1 capital costs include an additional $4.4 million from FAI's estimate to account for capital costs related to reusing the existing collection system, such as inspection, cleaning, and repairing of existing defects. Both scenarios include an estimated $400,000 WWPS equipment replacement cost after 20 years. The life-cycle costs consist of the 30-year net present value (NPV) of the capital, O&M, and 20-year equipment replacement costs. Additional detail is included in Appendix A.

Table 2-2. Collection System Alternatives Cost Summary

Collection System Alternative	Capital Cost ($M)	Annual O&M Cost ($/year)	WWPS Equipment Replacement (20-yr)	Life-Cycle Cost ($M)
Phase 1 Only: Reuse Existing Collection System	$30.0	$240,000	$400,000	$35.9
Phase 1+ Phase 2: All New Collection System	$40.2	$74,000	$400,000	$42.2

Table 2-3 provides a summary of capital cost assumptions used for the collection system cost analysis.

Table 2-3. Collection System Alternatives Capital Cost Estimating Assumptions

Description	Value
Estimate date	June 2023
Engineering News Record 20-Cities Average Construction Cost Index	13,345
Engineering, administration, and legal markup	25 percent
Estimating contingency for unknowns	20 percent

The life-cycle cost evaluation consists of a NPV comparison of the two alternatives. The NPV analysis includes capital, annual O&M, and periodic equipment replacement costs. The capital expenditure is assumed to occur in year 1, and annual O&M costs are incurred during years 2 through 30. A pump station equipment replacement project is assumed to occur in year 20. An appropriate inflationary factor and discount rate are applied to the cash flow projections to obtain the NPV over a 30-year planning period.

The NPV of an alternative represents the amount of money that would need to be set aside today (at a given interest rate) to pay the costs associated with the alternative over the entire planning period. The alternative with the lowest NPV is considered the most attractive from an economic perspective.

Table 2-4 summarizes the life-cycle cost evaluation assumptions.

Table 2-4. Life-Cycle Economic Assumptions

Description	Value
Year of analysis	2023
Planning period	30 years
Inflation rate	3.5 percent
Discount rate	5.0 percent
Equipment replacement cycle	20 years

The life-cycle costs are shown graphically on Figure 2-4. Reusing the existing Brewer Company collection system has the lowest capital and life-cycle costs.

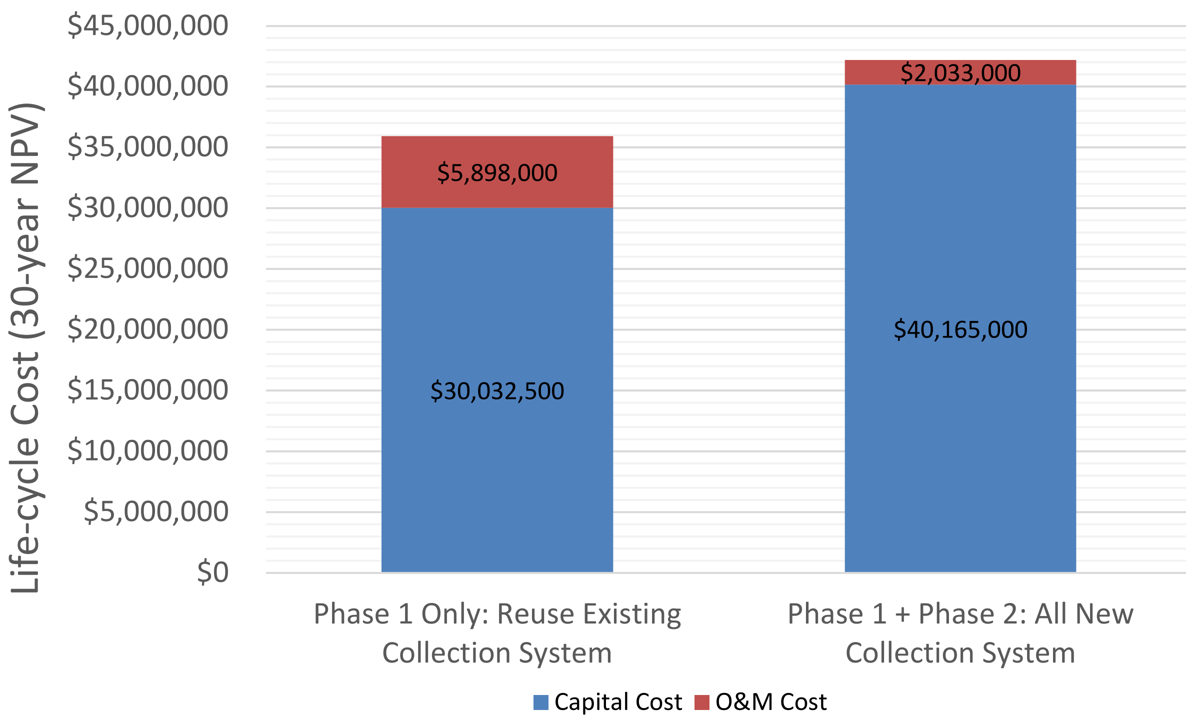


Figure 2-4. Life-Cycle Cost Comparison of Collection System Alternatives

2.7 Recommendation

Although reusing the existing collection system appears to incur lower life-cycle costs than constructing an all-new collection system, it is not recommended. Due to the advanced age of the existing collection system, the option would incur substantial financial, public health, and environmental risks to the County, as summarized below:

- The piping is at the end of its useful service life; catastrophic failures are likely to increase in frequency, resulting in greater risks to public health and the environment.
- Most of the system is located in backyard easements, making it difficult to access and maintain.
- System expansion to accommodate sewering additional areas of the town (in accordance with the Kau CDP) would not be feasible.
- The option does not address the Administrative Order on Consent requirement to connect additional properties that are currently not connected to the collection system to the WWTP.

A new conventional gravity sewer collection system constructed in the streets of the Brewer Company lot development (Phase 2) is a viable solution to meet the wastewater collection needs of the town of Naalehu. From a technical perspective, the Phase 1 + Phase 2 option is recommended for implementation should a new WWTP be constructed. It is anticipated that additional non-technical considerations such as financing options, community input, burden on the Naalehu community, and detailed alignment with County long term planning goals will be addressed outside of this preliminary engineering report.

The County provided the following statement:

"COH will continue working with, planning department, community leaders, and developers for long term accomplishments of the CDP as this will take more time and effort than what was given within the AOC. Better planning decisions can be made as the COH can continue monitoring growth, work with planning department on the general plan 2045, finalize the cesspool conversion master plans, and be able to negotiate a better financial plan with all stakeholders aka developers, businesses, state government agencies, and the community at large."

Section 3

Flow and Load Projections

This section summarizes the wastewater flow and load projections for the new Naalehu wastewater treatment plant (WWTP).

3.1 Flow Projections Based on City and County of Honolulu Standards

Section 11-62-24(b) of the Hawaii Administrative Rules (HAR) requires Counties to use their adopted wastewater flow standards to develop flow projections for WWTPs. Counties are to use the City and County of Honolulu (CCH) flow standards if they have not adopted their own standards. The County of Hawaii has not adopted its own flow standards, so wastewater flow projections were developed using current CCH wastewater standards (2017). Table 3-1 summarizes the flow projections.

Table 3-1. Naalehu WWTP Flows Based on 2017 CCH Standards

Description	Value (gallons per day)	Peaking Factor
Average dry weather flow	225,000	1.0
Peak day dry weather flow	446,000	2.0
Peak day wet weather flow [a]	563,000	2.5
Peak hour wet weather flow	480 gallons per minute (691,000 gallons per day)	3.1

a. Peak day wet weather flow is not part of the CCH standards but is an important WWTP design parameter. Peak day wet weather flow estimate was developed using an appropriate peaking factor.

The CCH standards were established for a major metropolitan area that includes vast areas of residential, commercial, and industrial development, with significant proportions of service areas near sea level elevations. Wastewater generation rates are generally lower in rural areas than in urban areas. The County's experience with the CCH flow standards on other projects (e.g., Honokaa WWTP) has illustrated that the standards are very conservative for small rural communities located at higher elevations on Hawaii Island. Therefore, the current wastewater standards based on urban Honolulu are likely overly conservative for rural communities like Naalehu.

3.2 Reduced Flows Based on Potable Water Records

The amount of wastewater generated within a residence will not exceed the amount of potable water used by the occupants. Therefore, potable water use records can be used to estimate wastewater generation rates within existing communities where no combined sewers are present. The County of Hawaii Department of Water Supply provided potable water use records for the parcels located within the service area from November 2017 through October 2022. Evaluation of the potable water use data is discussed in Appendix B. Analysis of the potable water use records indicates that an 80,000 gallons per day (gpd) monthly wastewater generation rate would reflect the current needs of the service area. Using a 2.5 peaking factor to estimate the maximum wastewater flow into the collection system results in a maximum wastewater flow of 200,000 gpd.

3.2.1 Dry Weather I/I Allowance

Groundwater can infiltrate into wastewater collection systems during dry weather, increasing flows to the WWTP. The 2017 CCH standards specify a dry weather infiltration and inflow (I/I) allowance of 35 gallons per capita per day (gpcd). The previous CCH standards (dated 1993) specified a dry weather I/I allowance of 5 gpcd for properties located above the groundwater table. Through the County's experience at Honokaa evaluating dry weather I/I for a rural collection system located in Hawaii Island's well-drained geology, at elevations hundreds of feet above sea level and a significant distance from the shoreline, we conclude that continued use of the 1993 standard for dry weather I/I is appropriate for Naalehu and using the 2017 standard would be overly-conservative. Further discussion is provided in Appendix B.

3.2.2 Wet Weather I/I Allowance

The 2017 CCH standards, which specify a wet weather I/I allowance of 3,000 gallons per acre per day, were used for all wet weather I/I calculations.

3.2.3 Reduced Flow Projections

Accurately quantifying flow projections for the Naalehu community is necessary to design an appropriately sized wastewater treatment and disposal facility. The WWTP design will provide sufficient capacity for the existing parcels within the service area, including newly accessible parcels, reflecting current development. This will allow the County to close the three LCCs. Furthermore, the design will provide sufficient area within the WWTP site for future expansion. Table 3-2 provides a summary of the calculated WWTP capacities for the reduced flow projections and for the flow projections for future development based on the 2017 CCH Standards.

Table 3-2. Naalehu WWTP Calculated Flow Capacity

Description	Reduced Flow Projections	Flow Projections Based on 2017 CCH Standards
Base sanitary flow	80,000 gpd	147,000 gpd
Peak hour sanitary flow	200,000 gpd	368,000 gpd
Dry weather I/I	12,000 gpd	78,000 gpd
Wet weather I/I	245,000 gpd	245,000 gpd
Average dry weather flow	92,000 gpd	225,000 gpd
Peak day dry weather flow	212,000 gpd	446,000 gpd
Peak day wet weather flow	322,000 gpd Peaking Factor = 3.5	563,000 gpd Peaking Factor = 2.5
Peak hour wet weather flow	317 gpm (457,000 gpd)	480 gpm (691,000 gpd)

HAR 11-62-23.1(i) requires the initiation of a facility planning process when the actual wastewater flows (measured at the WWTP) reach 75 percent of the design capacity of the WWTP, and implementation of the facility plan must be initiated when actual wastewater flows (measured at the WWTP) reach 90 percent of the design capacity. In anticipation of future development, we recommend the WWTP design be rated to treat an average dry weather flow of 125,000 gpd to avoid the potential of having to initiate a facility plan shortly after the project is constructed. Note that the

biological processes in the mechanical WWTP will need to be sized to treat the peak day dry weather flow of 212,000 gpd, not the average dry weather flow.

The proposed WWTP design capacity is based on actual water use data to establish wastewater generation rates, and rational assumptions to establish dry weather I/I allowances, and we believe it is appropriate for the existing conditions, while providing limited capacity for growth. Table 3-3 presents the recommended design capacity for the reduced flow projections.

Table 3-3. Recommended WWTP Capacity

Description	Value	Peaking Factor
Average dry weather flow	125,000 gpd	1.0
Peak day dry weather flow	212,000 gpd	1.7
Peak day wet weather flow	322,000 gpd	2.6
Peak hour wet weather flow	457,000 gpd (317 gpm)	3.7

3.2.4 Flow Variance

If the County pursues a WWTP approach (using the recommended capacity in Table 3-3) to close the LCCs then a DOH variance from HAR 11-62 requirements will be needed. The variance will need to be renewed every 5 years. The WWTP capacity needs should be re-evaluated upon application for the variance renewal.

3.3 Influent Characteristics

The properties within the existing service area are primarily residential, but do include several commercial, agricultural, and industrial zoned parcels. The wastewater characteristics of the WWTP influent are assumed to be similar to typical domestic wastewater. Table 3-4 provides a summary of the assumed influent characteristics.

Table 3-4. Summary of Assumed Influent Characteristics

Parameter	Value (mg/L)
5-day biochemical oxygen demand (BOD_5)	300
Total suspended solids (TSS)	300
Total nitrogen	40
Total phosphorus	7

3.4 Influent Mass Loads

Table 3-5 summarizes the projected loads to the WWTP, based on the proposed peak day dry weather capacity of 212,000 gpd and the influent characteristics presented above.

Table 3-5. Projected Peak Dry Weather Day Influent Mass Loads

Description	Value (lbs/day)
BOD_5	530
TSS	530
Total nitrogen	71
Total phosphorus	12

Section 4

Effluent Management Alternatives and Regulatory Requirements

Effluent management alternatives are evaluated in this section, followed by an assessment of regulatory requirements for the recommended effluent management system.

4.1 Effluent Management Alternatives

Effluent management alternatives are evaluated below.

4.1.1 Ocean Discharge

Ocean discharge of treated effluent is not considered a viable alternative for this small community due to the long distance to the shoreline (approximately 2 miles), high cost to construct an outfall, stringent receiving water quality standards, high receiving water monitoring cost, and difficulty and length of time required to secure the required permits. The coastal waters in the Naalehu area are classified as "AA" marine waters by State of Hawaii Department of Health (DOH). Hawaii Administrative Rules (HAR) 11-54 does not allow zones of mixing in waters up to a distance of 300 meters (1,000 feet) offshore if there is no defined reef area and if the depth is greater than 18 meters (10 fathoms). The water quality criteria for nutrients for Class AA embayments are listed in Table 4-1. If a mixing zone is not provided, then a WWTP discharging to the coastal waters would be required to treat water to meet the applicable water quality criteria. Treatment to the specified levels is not feasible with current technologies. Therefore, ocean discharge is not feasible without a mixing zone and an outfall at least 1,000 feet offshore would be required for Naalehu.

Table 4-1. Nutrient Water Quality Standards for Class AA Embayments

Parameter	Geometric mean not to exceed (µg/L)	Not to exceed the given value more than 10% of the time (µg/L)	Not to exceed the given value more than 2% of the time (µg/L)
Total nitrogen	200	350	500
Ammonia nitrogen	6	13	20
Nitrate + nitrate nitrogen	8	20	35
Total phosphorus	25	50	75

4.1.2 Subsurface Disposal via Injection Wells

Per HAR, Title 11, Chapter 23, disposal to groundwater via an injection well is not allowed mauka of the DOH Underground Injection Control (UIC) line. The UIC line in the Naalehu area is located along the shoreline. Since the town of Naalehu is located mauka of the UIC line, an injection well is not a viable alternative. In addition, per Environmental Protection Act 131, DOH is prohibited from issuing permits "for the construction of sewage wastewater injection wells unless alternative wastewater disposal options are not available, feasible or practical". Therefore, subsurface disposal via injection wells is not feasible.

4.1.3 Water Recycling

This section summarizes Brown and Caldwell's (BC's) evaluation of water recycling as the primary effluent management system.

4.1.3.1 Irrigation

An irrigation assessment was completed to determine the viability of water recycling as the primary effluent management system, assuming the recycled water would be used to irrigate nearby coffee trees or other agricultural crops.

Figure 4-1 presents a summary of the assessment, which shows there is typically no irrigation demand for 3 months of the year (November through January) due to high rainfall. In addition, the DOH requires that all water recycling programs have a 100 percent backup disposal system in place to handle flow that does not meet recycled water quality standards or when recycled water supply exceeds demand. Therefore, water recycling is not a viable primary or sole effluent management strategy for the community at this time. However, water recycling treatment, storage, and distribution systems could be added in the future. In addition to nearby irrigated agricultural reuse, the Naalehu Park and Naalehu Elementary School have significant areas of turf that could be considered for future irrigation with recycled water that meets the DOH "R-1" standards.

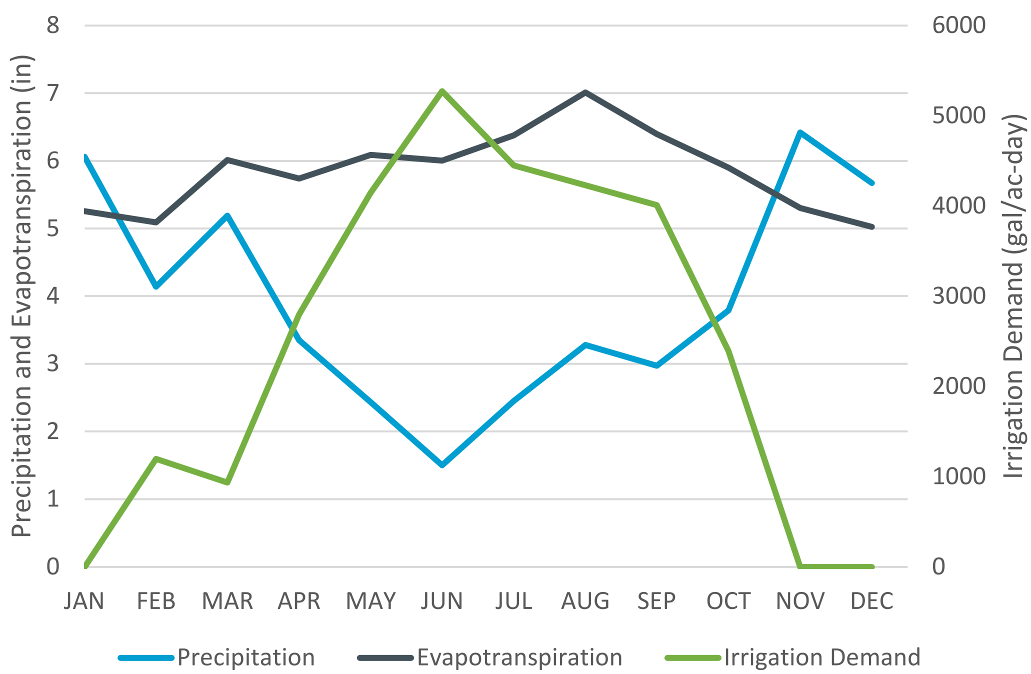


Figure 4-1. Irrigation Demand Assessment

4.1.3.2 Stock Water

The proposed WWTP site is located on a large ranch parcel. The County evaluated recycled water for stock watering purposes in November 2019 (BC, November 2019). The study concluded that recycled water that meets DOH R-1 standards could be used for stock watering purposes. The estimated peak stock water demand was 29,000 gallons per day (gpd), approximately 23 percent of the proposed average dry weather flow capacity of the WWTP. Stock water use appears to be feasible if a water recycling program is developed in the future.

4.1.4 Slow Rate Land Treatment

A potential project effluent management concept consists of Type 1 slow rate (SR) land treatment, which involves irrigation of vegetation with effluent. Type 1 slow rate land treatment differs from water recycling in that it is a disposal method, and effluent is typically applied in excess of the irrigation needs of the vegetation. The potential effluent management concept calls for grading the site to contain all precipitation and planting native Hawaiian trees within the effluent disposal area. Effluent would be applied using surface (flood) irrigation techniques.

The soils at the proposed WWTP location are suitable for SR land treatment. The proposed WWTP effluent management system will make use of an area containing Naalehu medial silty clay loam soil (Natural Resources Conservation Service [NRCS], 2018). This soil type is well drained with moderately high to high permeability. SR land treatment consists of irrigation of land and vegetation with effluent. Significant treatment is provided as the water percolates through the soil. The vegetation uses the nutrients in the effluent as fertilizer and transpires a portion of the applied water. SR land treatment serves as a means for final disposal of effluent.

4.1.5 Subsurface Drip Irrigation Disposal

Another effluent management concept is to retain the existing site topography along with the existing vegetation and use subsurface drip irrigation technology to apply the effluent within the effluent disposal area. The use of subsurface drip irrigation technology to disperse effluent at the site will allow the County to significantly reduce the amount of clearing, grubbing, and grading required to construct the facility, as compared to slow rate land treatment.

Drip irrigation technology has evolved to the point where non-clog emitters are available for subsurface applications of effluent. Non-clog subsurface emitters decrease the potential for the irrigation components to be clogged by roots.

Figure 4-2 illustrates the subsurface drip concept. Drip tubing with integral emitters is buried 6 to 9 inches below ground. Effluent emitters are typically designed to operate at a flow rate of 1 gallon per hour (gph) and are typically spaced every 2 feet along a drip line. Pressure compensating drip systems typically operate under pressures ranging from 10 to 45 pounds per square inch (psi).

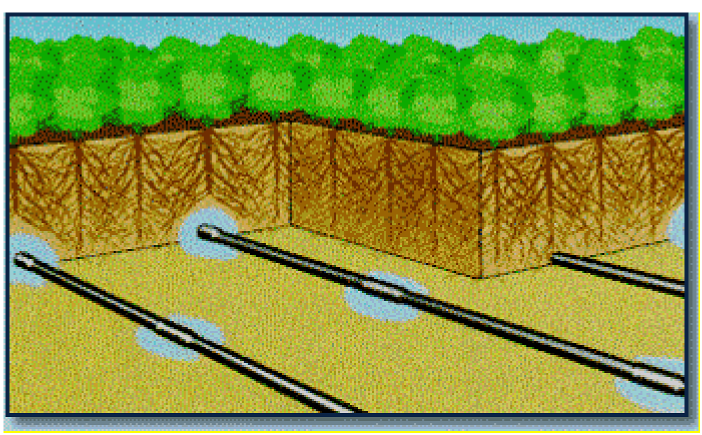


Figure 4-2. Subsurface Drip Irrigation Concept

(Courtesy of Geoflow, Inc.)

Subsurface drip irrigation technology incurs greater operation and maintenance than a surface irrigation system. The County would need to periodically flush the drip lines to remove debris. As described below, a significant number of drip lines are necessary to accommodate peak flow rates. In addition, periodic chlorination would be required to remove biological growth from the drip lines.

These O&M tasks would need to be completed on a regular schedule, because drip systems are buried and not readily accessible or observable. During periods of dry soil conditions, the County would need to inspect the disposal area for patches of wet soil that would indicate a localized failure that requires repair. Flow and pressure monitoring would also be useful tools for validating the status of the subsurface drip system. The disposal area would be divided into multiple irrigation zones, allowing a zone to be taken out of service for maintenance purposes. A fence would be constructed around the site to deter entry by humans and ungulates.

The subsurface drip system would slowly disperse effluent 6 to 9 inches below the ground surface, therefore operating as a subsurface disposal system similar to a leach field. Effluent is not intended to surface with a properly operating subsurface drip system. Precipitation falling on the site would either percolate into the soil or run off as surface drainage. The amount of runoff from the site would vary with the storm intensity; precipitation rates in excess of the infiltrative capacity of the site soils would result in runoff. The implementation of a subsurface disposal system will allow the existing grading to be retained, because stormwater runoff will not come into contact with, and therefore will not contain, effluent.

4.1.6 Leach Field

A leach field could potentially be constructed for subsurface disposal of treated effluent. Preliminary assessment of the concept based on the site soil characteristics (NRCS, 2018) and HAR 11-62 indicate approximately 14 acres of leach fields would be required to accommodate the anticipated flow, which includes a 100-percent redundant drain field per the DOH requirements. There is insufficient soil area available at the proposed WWTP site to construct a leach field of this size; therefore, this alternative is considered to be not feasible.

4.1.7 Ramifications of United States Supreme Court Opinion on County of Maui, Hawaii v. Hawaii Wildlife Fund et. al.

The United States Supreme Court (USSC) has published its opinion on the case of Hawaii Wildlife Fund, et al. vs. County of Maui regarding whether the injection wells at the Lahaina Wastewater Reclamation Facility (WWRF) are subject to regulation under the Clean Water Act (CWA) National Pollutant Discharge Elimination System (NPDES) program. The USSC ruled that a discharge of pollutants that originate from a point source but are conveyed to navigable waters by a nonpoint source (in the Lahaina situation, groundwater) are subject to regulation under the NPDES program if the nonpoint source is a “functional equivalent” of a point source discharge. Unfortunately, the term “functional equivalent” is not well defined at this time. The USSC decision offered two examples of what would and would not be considered functional equivalents to a direct discharge:

- A pipe ending a few feet from a navigable water and the pipe emits pollutants that travel those few feet through groundwater (or over the beach) to the navigable water would clearly be subject to NPDES regulation.
- If a pipe ends 50 miles from navigable waters and the pipe emits pollutants that travel with groundwater, mix with much other material, and end up in navigable waters only many years later, would not be subject to NPDES regulation.

The two examples represent the two extreme situations. The USSC also offered some vague guidance as to how situations between the two extremes could be handled. Specifically, seven relevant factors were listed that could be applied to specific situations:

1. Transit time.
2. Distance travelled.
3. The nature of the material through which the pollutant travels.
4. The extent to which the pollutant is diluted or chemically changed as it travels.
5. The amount of pollutant entering the navigable waters relative to the amount of the pollutant that leaves the point source.
6. The manner by or area in which the pollutant enters the navigable waters.
7. The degree to which the pollution (at that point) has maintained its specific identity.

The USSC opinion further states that “time and distance will be the most important factors in most cases, but not necessarily every case”. The USSC was not able to provide additional guidance as to what would constitute a functional equivalent to a direct discharge, leaving substantial regulatory uncertainty that requires resolution.

4.1.7.1 The Road to Regulatory Clarity

Regulatory clarity is important to public agencies tasked with protecting public health and the environment by managing wastewater from communities. Environmental protection projects are often costly, and it is important for public agencies to have a level of confidence that a project will meet regulatory requirements over the project lifecycle before public funds are expended. Unfortunately, the USSC opinion does not provide regulatory clarity. There are four potential ways that regulatory clarity can be achieved:

1. Regulatory guidance documents: The U.S. Environmental Protection Agency (USEPA) or State of Hawaii could issue regulatory guidance documents that establish criteria for determining if a particular discharge scenario is a functional equivalent to a direct discharge. Developing regulatory guidance documents would be the fastest way to address the problem but would also be the weakest approach because regulatory guidance documents aren’t subject to a public review process and don’t carry the authority of the options listed below.
2. Regulations: The USEPA and/or State of Hawaii could develop regulations that define conditions whereby a discharge is a functional equivalent to a direct discharge. Development of regulations takes longer than development of regulatory guidance documents because a public participation process is involved.
3. Legislation: The United States and/or State of Hawaii governments could enact legislation that defines what is functionally equivalent to a direct discharge.
4. Court rulings: The definition of what it means to be a functional equivalent to a direct discharge could be resolved via court cases. This would be the slowest pathway of the four.

Based on the above, it will likely take years to decades before the term “functional equivalent” is adequately defined to a point where agencies have regulatory clarity.

4.1.7.2 Complications of Discharges within the State of Hawaii

The USSC opinion is especially complicated for most forms of effluent discharge in the State of Hawaii, because of the unique hydrogeology of volcanic islands. Our islands are surrounded by the navigable waters of the Pacific Ocean, and water that percolates into the soil that does not evaporate or is not taken up by vegetation via evapotranspiration will eventually find its way to the groundwater that moves mauka to makai and into the ocean. Therefore, any pollutants that percolate to groundwater will eventually find their way to navigable waters.

A further complication is the State of Hawaii's water quality standards (WQS) that define ocean water quality criteria for discharge into the ocean. These standards are an order of magnitude lower (i.e., more restrictive) than what can be achieved with existing advanced treatment technology. Treatment technology is currently not available that can reliably produce effluent that meets the WQS. While technology is available that can purify wastewater to where it is safe to drink (at great expense), ironically that same purified water cannot not be discharged into the ocean in certain places or certain conditions because it would not meet the water quality standards the State has established for ocean waters.

4.1.7.3 United States District Court Ruling

Subsequent to the USSC decision, the U.S. District Court in Honolulu ruled the Lahaina WWRF injection wells are functionally equivalent to a direct discharge and are subject to the CWA and require an NPDES permit. The DOH is in the process of issuing an NPDES permit to the facility. The U.S. District Court Ruling establishes another data point in addition to the two provided by the USSC regarding whether a discharge is functionally equivalent to a direct discharge:

- The transit time from the Lahaina WWRF injection wells to the Pacific Ocean is a minimum of 84 days, reaches peak concentration in 9 to 10 months, and continues for 3 to 5 years after discharge to the injection wells. A dye tracer test was used to establish the transit time.
- The straight-line distance traveled from the injection wells to the Pacific Ocean is approximately 2,900 feet (0.55 miles), although the groundwater in the aquifer takes a non-linear route.

4.1.7.4 Functional Equivalent Risk

As discussed above, the USSC opinion establishes at least seven factors to be used to determine whether an indirect discharge is a "functional equivalent" to a direct discharge, and without regulatory action, legislative direction, or additional legal precedents, agencies that desire to implement an indirect discharge system are placed in a position of regulatory uncertainty. The risk can be eliminated by:

- Implementing a direct discharge system (e.g., ocean outfall) that is known to be subject to the NPDES program.
- Implementing a true zero-discharge system, i.e., evaporation.

All other forms of disposal carry risk of being a functional equivalent to a direct discharge. Site-specific factors need to be evaluated for each particular discharge system and location. However, each type of discharge system provides different degrees of treatment and environmental attenuation. Therefore, for a given site certain options present less risk than others.

Table 4-2 shows BC's opinion of the risk of various effluent management systems that have been discussed in relation to each other, assuming disposal systems that provide more treatment present less risk than disposal systems that provide less treatment.

Of the indirect discharge systems, discharge to groundwater via injection wells is shown as having the highest risk of being functionally equivalent to direct discharge because treated effluent is added directly to the groundwater in a discrete area.

Leach fields are shown to have lower risk than injection wells because effluent is applied near the surface over a large area and treatment occurs as the applied water percolates through the unsaturated soil.

Table 4-2. Relative Risk of Being Functional Equivalent to Direct Discharge		
Classification	Risk of Being Functional Equivalent to Direct Discharge	Effluent Disposal System
Direct discharge	Not applicable	Direct discharge to ocean
Indirect discharge	Highest risk	Discharge to groundwater via injection wells
		Leach fields
		Slow rate land treatment and subsurface drip disposal systems
	Lowest risk	Water recycling

Slow rate land treatment and subsurface drip disposal are shown as having lower risk because the effluent is applied over a large area at or near the surface, and the presence of managed vegetation allows a higher degree of treatment to occur before the water percolates to groundwater.

Water recycling in the form of irrigation carries the lowest risk because it produces the smallest volume of deep percolate of the options shown.

4.1.7.5 Functional Equivalent Risk at Naalehu

At the current time we believe the risk of any of the feasible effluent management alternatives for Naalehu (water recycling, slow rate land treatment, subsurface drip disposal, and leach field) being found to be the functional equivalent to a direct discharge are low due to the following considerations:

- The effluent management system(s) would be located at an elevation of approximately 680 feet above sea level, creating a large vadose zone that applied effluent would need to travel down before reaching the basal groundwater lens.
- The effluent disposal site is located nearly 2.5 miles from the shoreline, likely resulting in a long travel time.
- The effluent flows will be relatively small at this facility, and there are no documented groundwater quality problems in the area.

However, it must be noted that future court rulings, legal action, legislation, or regulatory actions could render the facility to be functionally equivalent to a direct discharge and subject to NPDES permitting requirements.

4.1.8 Recommendation

Subsurface drip irrigation system is the recommended method of effluent disposal for the Naalehu WWTP. Subsurface drip will incur lower capital cost and require less attention from WWTP operators with respect to vegetation maintenance than slow rate land treatment. Subsurface drip requires periodic maintenance chlorination to eliminate biofouling in the drip lines. Recommended design criteria for the subsurface drip irrigation system are presented in Table 4-3. The disposal system would be sized to handle the peak day wet weather flow of 322,000 gpd. An irrigation equalization and control tank are proposed to equalize higher peak flows and to allow discrete dosing of the irrigation zones.

HAR 11-62 requires a fully redundant subsurface disposal system. The design criteria listed in Table 4-3 are based on providing a subsurface drip system that is two times larger than needed in order to satisfy the HAR 11-62 requirement for redundancy. The drip system ould be divided into two separate systems so that the peak day wet weather flow can be disposed on the site using one system while the second system is out of service for maintenance.

Table 4-3. Recommended Subsurface Drip Disposal Design Criteria	
Description	Value
Average dry weather flow	125,000 gpd (87 gpm)
Peak day wet weather flow	322,000 gpd (224 gpm)
Irrigation equalization and control tank volume	20,000 gallons
Disposal area	5.2 acres
Subsurface drip emitters	1 gallon per hour, pressure compensating
Number of emitters needed for peak day wet weather flow	13,417 emitters
Number of systems	2 (1 active, 1 redundant)
Number of emitters provided to provide 2x redundancy	26,833 total emitters
Emitter spacing	2 feet
Drip line length per system	26,833 feet
Total drip line length	53,667 feet
Drip line spacing	4 feet
Drip line depth	6 to 9 inches
Number of irrigation zones	4 (2 per system)
Length of drip line per zone	13,417 feet
Flow per irrigation zone	112 gpm
Irrigation system monitoring	Flow meter(s) and pressure indicators

During high flow conditions the irrigation control system would open multiple irrigation zones to accommodate the disposal needs. Additional drip lines will need to be added when the WWTP capacity is expanded. The minimum spacing between drip lines is 2 feet, so there will be sufficient space between the initial drip lines to add additional drip lines as part of future expansion project(s).

4.2 Treatment Requirements

The DOH regulates subsurface drip irrigation disposal as “land disposal” per HAR 11-62. Table 4-4 lists the applicable effluent requirements for land disposal applicable to the project in effect at the time this report was prepared.

Table 4-4. Applicable HAR 11-62 Land Disposal Requirements		
Description	Value	HAR Reference
5-day biochemical oxygen demand (BOD_5)	30 milligrams per Liter (mg/L) monthly average 60 mg/L peak	11-62-26
TSS	30 mg/L monthly average 60 mg/L peak	11-62-26
Disinfection	Except for subsurface disposal systems, continuous disinfection of the treated effluent shall be provided.	11-62-24
Setbacks	Treatment units shall be not less than 25 feet from property lines nor less than 10 feet from any building.	11-62-23.1
Public accessibility control	6-foot-high fence surrounding treatment units	11-62-08

Section 5

Wastewater Treatment Evaluations

This section summarizes the evaluations conducted as part of developing the proposed wastewater treatment plant (WWTP).

5.1 Preliminary Treatment

The preliminary treatment system will include influent flow measurement, influent sampling equipment, screening, and grit removal.

5.1.1 Influent Flow Measurement

Influent flow measurement is recommended to allow assessment of flows and loads to the biological treatment process, and to assess the biological treatment process performance. A Parshall flume will be provided upstream of the screening system to continuously record influent flow rates. Parshall flumes work well for influent measurement because the flume can operate in an open-channel configuration, can accommodate wide ranges of flows, and is self-cleaning. A straight approach length of at least 20 times the flume throat width will be provided upstream of the flume to provide favorable hydraulic conditions.

5.1.2 Influent Flow Sampling

An automatic refrigerated composite sampler is recommended to allow influent composite samples to be collected. Influent composite samples, when combined with influent flow measurement, can be used to calculate influent mass loading rates to the WWTP to assess the treatment performance and optimization of aeration rates in the biological treatment process. Periodic influent sampling is also recommended to monitor for changes in the influent characteristics.

5.1.3 Screening

Screening is recommended to protect the downstream system operations from large objects, debris, wipes, and rags that can be present in wastewater. The industry trend is towards finer screening systems that remove greater amounts of debris from the waste stream; screens with 6-millimeter (1/4 inch) openings are frequently used for activated sludge treatment systems. Finer screens are used upstream of membrane bioreactors to remove hair that can foul the membranes. The screenings volume at the Naalehu WWTP is expected to be small, subsequently screenings disposal is expected to be infrequent; weekly at most. Therefore, the screenings must be washed of organic debris to prevent the accumulation of nuisance odors and flies in the screenings barrel or bag between screening disposal events.

5.1.3.1 In-Channel Cylindrical Screen

We recommend an in-channel cylindrical screen for this installation. The in-channel cylindrical screen combines screening, screenings washing, dewatering, compacting, and bagging/disposal within a single unit. The screening portion consists of an inclined screen basket inserted into the wastewater channel. The screening basket can consist of bars, perforated plates or sieves, depending on the application and clear opening required. The controls can be set to allow a mat to build up on the screening surface, allowing finer screening of the wastewater. Controlled by head loss, a rake arm starts rotating within the screen basket, pushing the screenings off the rake and into a perforated screenings hopper located at the screen's central axis. A shafted auger along the screen axis conveys the screenings from the hopper through an inclined tube, which dewaters and compacts the screenings. The tube includes a perforated dewatering section. The discharged screenings are about 40-percent dry and can be discharged into a bin or directly into a bagging system.

Figure 5-1 illustrates the process. Manufacturers include Lakeside and Huber. The key benefit to this system is the integrated screenings washing system, minimizing additional screenings handling and odor potential. For this installation, the headworks will include one in-channel cylindrical screen, plus a bypass channel with manually cleaned bar rack.

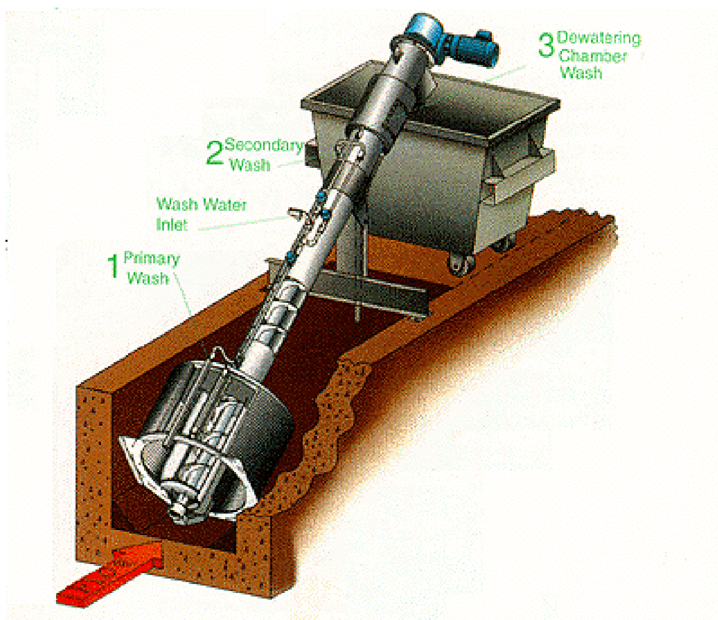


Figure 5-1. In-Channel Cylindrical Screen

5.1.4 Grit Removal

Grit is comprised of particles that are heavier than the organic biodegradable matter in wastewater. Grit particles can consist of sand, gravel, pebbles, silt, cinders, ground bone, eggshells, coffee grounds, and other materials. Grit in the wastewater collection and treatment system causes abrasive wear to mechanical equipment, piping, and appurtenances. Grit can also form deposits in pipelines, channels, and tanks, which reduces hydraulic capacity and can damage equipment. Removal of grit is very important to help prevent wear to downstream equipment, costly service interruptions and repair.

Grit removal systems usually are placed between screening and downstream treatment processes. At this point, the largest materials have been removed by the screens and will not interfere with grit handling equipment.

There are several types of grit removal methods, including induced vortex grit removal, aerated grit chambers, and lamella plate settlers. The type of grit removal chosen is mainly dependent on the size of the incoming grit particles and the desired capture rate. Removed grit must be washed, dewatered, and disposed.

5.1.4.1 Induced Vortex Grit Removal

Historically, vortex grit removal, or the circular grit chamber, has been the most widely used method for grit removal in the United States, Vortex grit removal relies on the principle that grit has a greater specific gravity than organic matter.

There are two configurations of vortex grit removal systems: a sloped bottom unit and a flat bottom unit. The sloped bottom unit relies on particle settling to remove grit. Flow enters the grit chamber tangentially to provide the longest flow path around the inside of the circular grit chamber. This longer flow path is designed to achieve a sufficient retention time to allow grit to settle. The sloped bottom funnels the settled grit into a hopper below the basin. A sloped bottom vortex grit unit cross section is shown in Figure 5-2.

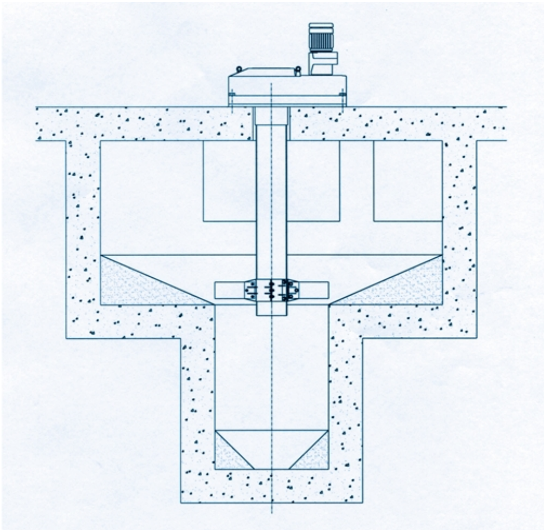


Figure 5-2. Sloped Bottom Vortex Grit Removal Cross Section

The flat bottom vortex system relies on hydraulic removal instead of specific gravity alone to remove grit from the wastewater stream. Flat bottom vortex systems use two paddles within the interior of the grit chamber that induce a toroidal flow pattern to move grit along the bottom towards the center. Once collected at the center of the grit chamber, a propeller forces excess grit down into the hopper. A flat bottom PISTA® Grit unit is shown in Figure 5-3.

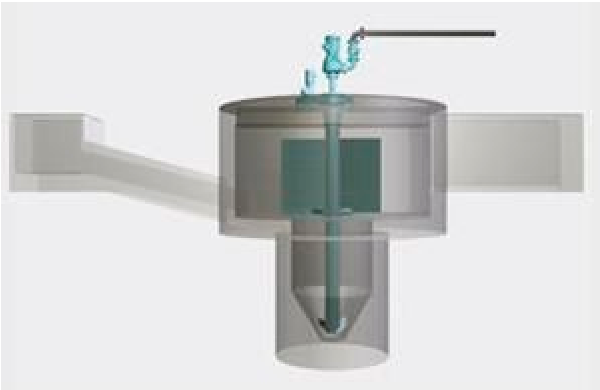


Figure 5-3. Flat Bottom PISTA® Grit Removal

5.1.4.2 Vortex Grit Removal Capture Rate

In BC's experience, it is necessary to de-rate vortex type grit removal units by a factor of 50 percent of the advertised capacity to achieve satisfactory performance, due to the short detention time in the chamber. At large flow rates, the small-chambered vortex type units tend to re-suspend smaller grit particles which become a problem for downstream processes. Table 5-1 lists some advantages and disadvantages of the vortex type grit removal.

Table 5-1. Induced Vortex–Advantages and Disadvantages

Advantages	Disadvantages
• Low maintenance • Low headloss • Small footprint	• Re-suspends/low capture rate of fines • Poor capture efficiency

5.1.4.3 Aerated Grit Removal

Aerated grit chambers are tanks that function specifically to remove inorganic solids from the wastewater stream. Aerated grit tanks are designed to induce sufficient vertical velocity in order to separate organic and inorganic solids. In theory, inorganic solids have a higher specific gravity than organic solids, and therefore require higher vertical velocities to keep them in suspension.

Air diffusers placed near one longitudinal tank wall induce a roll in the contents of the grit tank. This roll creates maximum velocities near the walls and lower velocities at the surface and bottom of the tank. The lower transverse horizontal velocities allow inorganic particles to settle out and be transported to the grit hopper by shear-induced currents.

Aerated grit chamber design is based on providing sufficient hydraulic detention time during peak wet weather flow (PWWF) conditions. In BC's experience it is necessary to provide at least 10 minutes of detention time to achieve satisfactory grit removal.

Aerated grit tanks can provide excellent grit removal with minimal headloss, but the chambers themselves require a larger footprint than induced vortex systems. Proper operation of aerated grit tanks can be difficult under varying hydraulic loads due to the need to make fine adjustments to the air diffusers. Figure 5-4 illustrates the particle settling action of an aerated grit chamber.

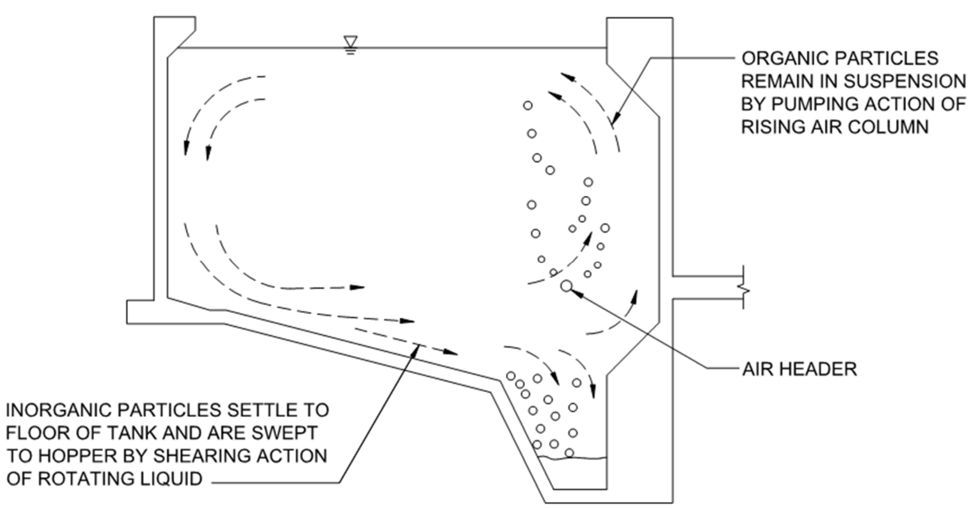


Figure 5-4. Aerated Grit Removal Schematic

Table 5-2 lists some advantages and disadvantages of aerated grit chambers.

Table 5-2. Aerated Grit Removal–Advantages and Disadvantages

Advantages	Disadvantages
• Low headloss • Once airflow is dialed in, the maintenance is low • Effective removal of fines • Provides additional aeration; “freshens” sewage prior to primary clarification. Reduces denitrification in primary clarifiers	• Large footprint • Requires fine tuning diffuser airflow for optimal performance • High capital cost • High O&M cost due to blowers

A variation of aerated grit removal technology that can be used in small WWTPs like Naalehu is an aerated grit trap. A small, aerated tank is provided to allow grit to settle. Aeration is provided to maintain organic solids in suspension and to “freshen” the influent. Accumulated grit is periodically removed using a Vactor truck.

5.1.4.4 Lamella Grit Removal

This proprietary technology from Eutek, called the HeadCell, consists of sloped trays stacked in deep tanks. Flow enters the tanks tangentially and establishes a vortex flow pattern. Solids settle onto each plate and fall toward an opening at the center of each plate. The grit collects at the cone shaped bottom of the tank where it is pumped to be washed and dewatered. Effluent flows out of the trays, over a weir, and into an effluent trough.

Grit capture is all done hydraulically and there are no moving parts. The headloss through each HeadCell is around one foot. HeadCells can be sized to provide up to 50 mgd of capacity within a single unit. With the stacked tray design, the HeadCells can achieve a 95 percent capture rate of grit 75 microns and larger. The multiple trays provide a large surface area for settling multiple size particles. The treatment capacity of the HeadCell is greater than other technologies with the same footprint. Figure 5-5 is an illustration of a section cut through the HeadCell process.

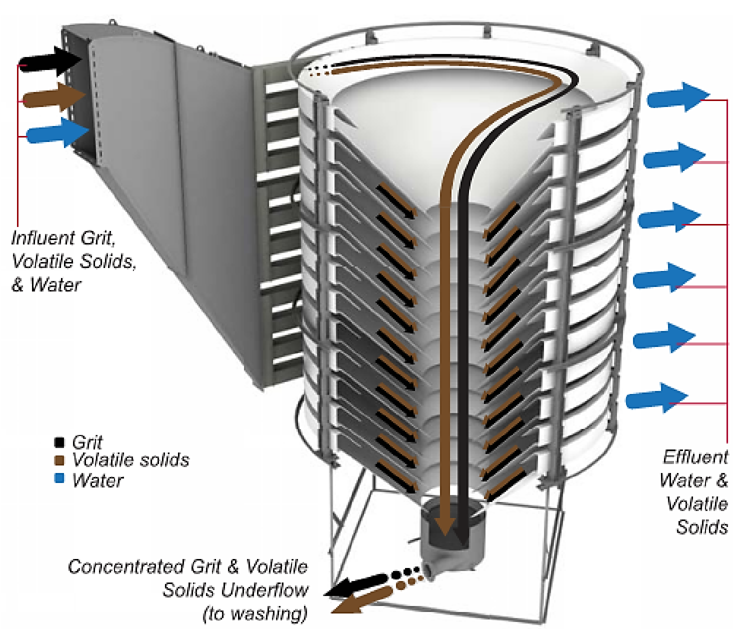


Figure 5-5. HeadCell Process Schematic

Image courtesy of Eutek

Table 5-3 lists advantages and disadvantages of the HeadCell.

Table 5-3. Lamella Plate Settling/HeadCell–Advantages and Disadvantages	
Advantages	**Disadvantages**
• Effective removal of fines • Small footprint • No moving parts • Low operating cost	• High capital cost • Short history of installations

5.1.4.5 Grit Particle Size Considerations

Most grit technologies and literature assume that grit is a clean sand or silica particle with a specific gravity of 2.65. In reality, grit particles are often coated with fats, grease, and organic material that reduce the particle's specific gravity. Grit particles with lower specific gravity have lower settling velocities, behaving like lighter and smaller grit particles.

The sand equivalent size (SES) is the size of a clean sand sphere that exhibits the same settling velocity as the coated grit particles. For example, a grease coated grit particle with a physical size of 200 microns may settle and behave like a clean particle with an SES of 150 microns.

5.1.4.6 Efficiency comparison

Each of the alternatives claims a minimum particle size and capture rate. These claims are based on the ideal, clean grit particle. As previously discussed, in reality grit particles are coated with fats and grease and do not exhibit the behavior of ideal grit particles. The capture rates have to be derated to reflect the SES of the particles. Table 5-4 compares the claimed minimum particle size captured of the alternatives discussed.

Table 5-4. Grit Capture Size Comparison	
Alternative	**Targeted Particle Size**
Induced Vortex	105 µm
Aerated Grit Removal	105 µm
HeadCell	75 µm

The HeadCell is able to remove the finest particles, with up to 95 percent removal of particles with a physical size down to 75 microns.

5.1.4.7 Grit Removal Recommendation

A simple aerated grit trap located downstream of the screening process is recommended for the Naalehu WWTP. Accumulated grit would be periodically removed using a Vactor truck, and dried onsite in a small drying bed. The dewatered grit would be disposed at the landfill.

An aerated grit trap provides adequate performance with a relatively uncomplicated process. Although a HeadCell grit removal system could potentially provide a slightly increased grit capture rate, that benefit is not likely to surpass its significantly higher costs and operational complexity. The capture rate of an aerated grit trap is sufficient to protect the downstream processes recommended in this report. High levels of grit removal are particularly important for anaerobic digestors, which are not anticipated for this facility.

5.1.5 Odor Control

A common location for foul odor is the headworks of a wastewater treatment plant. This odor is caused by hydrogen sulfide (H_2S), which is formed under anaerobic conditions of the wastewater collection system. Due to H_2S low solubility in wastewater, when there is an excessive concentration of H_2S in the wastewater or if there is turbulence, H_2S gas escapes into the atmosphere. This release produces the distinct rotten egg smell. In addition to H_2S, there are other foul odorous compounds that can be released from wastewater, such as ammonia, amines, diamines, mercaptans, skatole, and organic sulfides.

Treatment of foul odors can be approached in two ways: preventing odors through liquid treatment or controlling odors in the gas phase. While liquid treatment provides control of odors prior to their release, gas phase treatment involves the collection and treatment of gases once they have been released from wastewater. Treatment methods can be aimed at one type of odor or can treat a range of odors.

5.1.5.1 Biotrickling Filter

A biotrickling filter consists of a vessel containing plastic or foam media. Foul air is drawn through the media for treatment. A fixed film biomass is maintained on the media by circulating water over the media. Liquid fertilizer must be added to the circulating water to provide the nutrients (nitrogen and phosphorus) the biomass needs to grow. The biomass oxidizes odorous compounds from the foul air as it travels through the tower. A demister is provided to remove water droplets from the treated air stream.

Figure 5-6 shows a schematic diagram of a biotrickling filter. Biotrickling filters work best when H_2S concentrations are greater than 10 to 25 parts per million, which is greater than what is expected at Naalehu, due to the small size of the collection system.

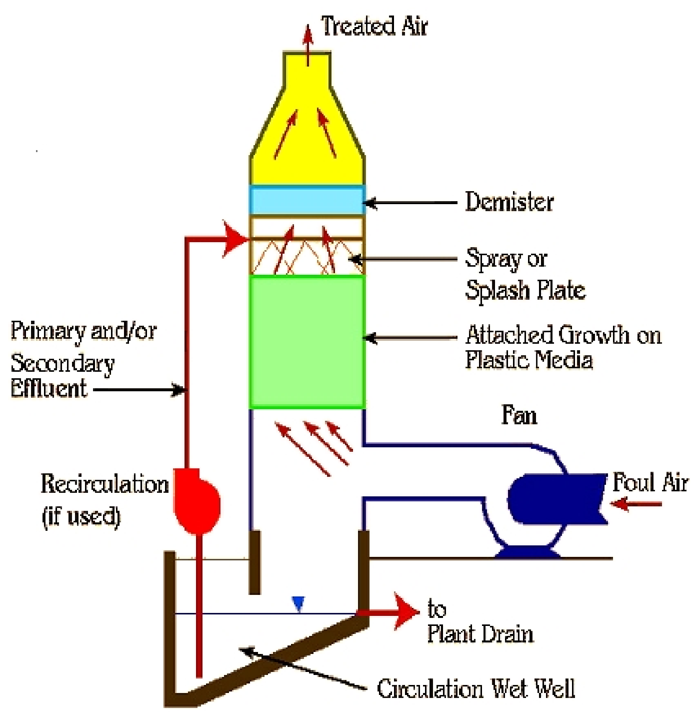


Figure 5-6. Biotrickling Filter

5.1.5.2 Granular Activated Carbon

A granular activated carbon (GAC) scrubber is recommended for the Naalehu WWTP headworks. A GAC scrubber passes odorous air through a bed of activated carbon, which adsorbs the odorous constituents within the pore spaces of the carbon.

Chemical oxidation or reduction of some compounds can also occur. As pore spaces become occupied, efficiency degrades, and the carbon must be replaced or regenerated. Carbon is most effective on higher molecular weight molecules such as the organic sulfur compounds, which makes it the technology of choice. Packaged GAC scrubbers are available for small headworks and vessels can be situated vertically, horizontally, or radially to optimize footprints and reduce structure elevation profiles. Figure 5-7 illustrates the process. The County currently operates GAC scrubbers at other facilities and purchases the GAC media in bulk to reduce costs.

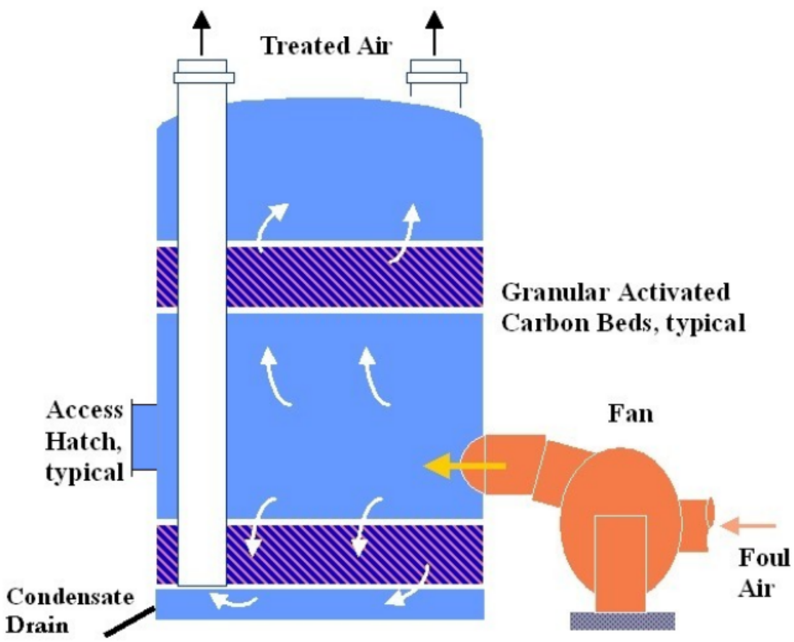


Figure 5-7. Activated Carbon Scrubber

5.1.6 Recommendation

The following are recommended for the Naalehu WWTP headworks:

- Parshall flume influent flow measurement
- Refrigerated automatic composite sampler
- In-channel cylindrical screen with integrated washer
- Aerated grit trap
- Covered channels with foul air collection and GAC scrubber

5.2 Secondary Treatment

Secondary treatment process provides 5-day biochemical oxygen demand (BOD_5), total suspended solids (TSS), and nutrient removal via biological treatment. This section provides descriptions of various secondary treatment options including advantages, disadvantages and applicability to the Naalehu WWTP. The treatment options are then screened to identify technologies for further evaluation.

5.2.1 Membrane Bioreactor

A membrane bioreactor (MBR) has the smallest footprint of the various biological treatment systems available and provides the highest quality effluent. An MBR basically combines an aeration basin with membrane filtration, eliminating the need for tertiary treatment if a very high-quality effluent is desired for water reuse purposes. Membranes provide an absolute barrier to large particles; TSS concentrations of the effluent (also known as "filtrate") are typically less than 1 milligrams per Liter (mg/L). Effluent from an MBR process can meet stringent water recycling turbidity requirements without an additional filtration process.

The main difference between MBRs and other biological treatment technologies is the method of separating the bacteria from the clean water. MBRs have thin membranes with many thousands of micro-perforations. Depending on the manufacturer, these perforations are 0.04 to 0.2 microns in diameter, too small for the passage of most microorganisms or other particles present in the wastewater, but large enough to allow the passage of water molecules.

Figure 5-8 is an illustration of an MBR. Figure 5-9 shows submerged MBR membranes in clean water.

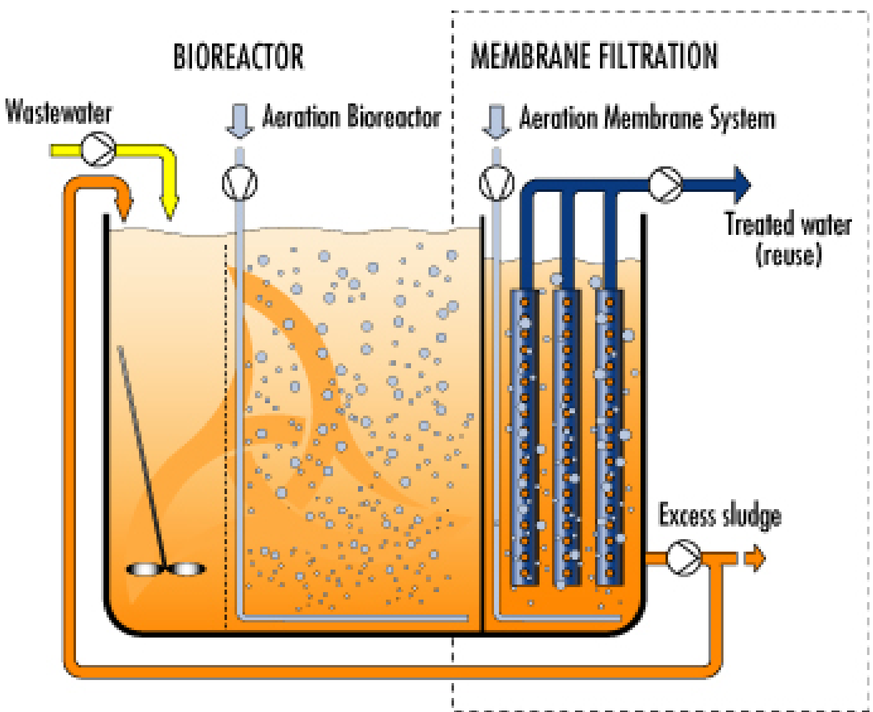


Figure 5-8. Membrane Bioreactor Illustration

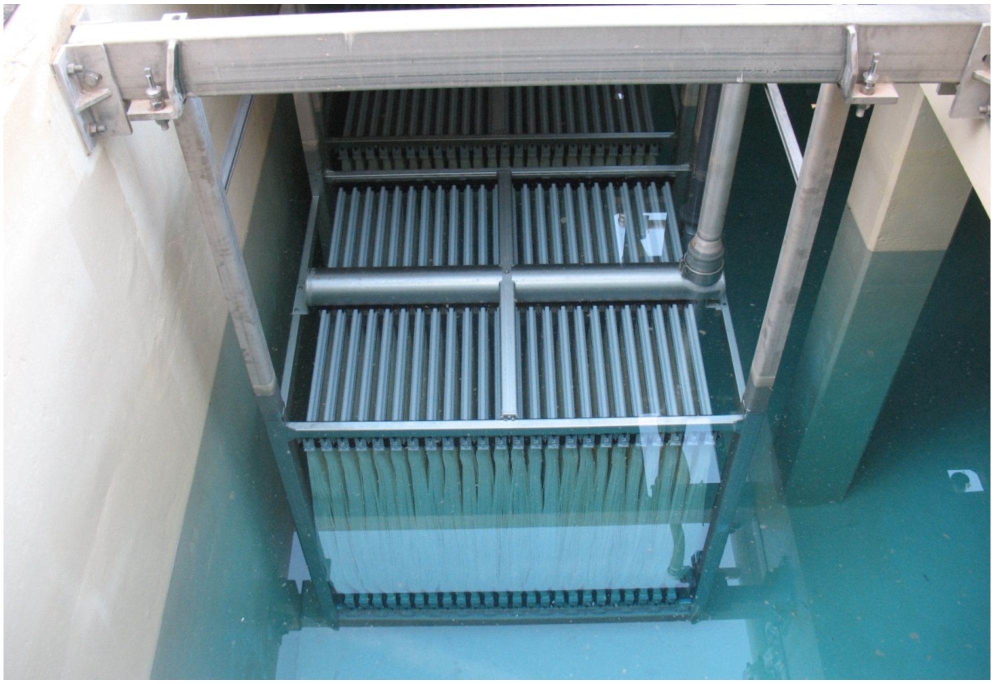


Figure 5-9. Membrane Cassettes at Johns Creek Environmental Campus

Fulton County, Georgia

Important considerations of an MBR system include:

- Small capacity MBRs can be purchased as a packaged treatment system.
- MBRs can be designed and programmed to achieve nutrient reduction.
- The membrane cost is significant, and membranes must be replaced every 10 to 15 years.
- Membrane fouling can occur with wastewater with high fats, oils, and grease levels.
- MBRs require the use of membrane cleaning chemicals, typically NaOCl and citric acid.
- The process requires a computer control system and is difficult to operate efficiently if the computer malfunctions.
- The process incurs high electrical power costs, relatively high costs for cleaning chemicals, and high overall O&M costs. Highly skilled labor is required for some of the O&M tasks.

The MBR process would produce an effluent that is of high quality and has a small footprint, but has high overall capital, O&M, and life-cycle costs. MBR is retained for further evaluation.

5.2.2 Sequencing Batch Reactor

Sequencing batch reactors (SBRs) are fill-and-draw systems that combine the processes of activated sludge in a single reactor. The reactor is filled with wastewater, where aeration, settling, and decanting occurs. By combining these processes, the need for secondary settling is not required. Denitrification can be achieved by incorporating an anoxic fill step in the cycle or a separate anoxic zone. A minimum of two SBR reactors are typically used for the process.

SBRs are capable of producing high quality effluent and are potentially space saving in that separate secondary sedimentation is not needed. However, SBRs are operated by a proprietary computer control system, cannot be operated in manual mode, and may require influent and/or effluent equalization (and thus increasing the footprint requirements). Considering these challenges, SBRs will not be considered further.

5.2.3 Nereda (Granular Activated Sludge) Process

The Nereda technology is a granular activated sludge process that utilizes proprietary granules in an SBR. Features of the process include simultaneous fill and draw, fast settling, and approximately 1/5 the footprint of traditional activated sludge systems. The process was developed in Europe and most current full-scale applications are located in Europe. In the U.S., the process is marketed by Aqua Aerobic Systems, Inc, according to the supplier website, there are currently only two full-scale operating systems treating municipal wastewater in the United States. One is a demonstration facility, and the other is a 3.6 mgd facility in Alabama that began operation in early 2020.

Figure 5-10is a conceptual illustration of the Nereda process. Due to the challenges listed for an SBR and the lack of long-term operational experience in the United States, the Nereda process is considered not appropriate for the Naalehu WWTP application.

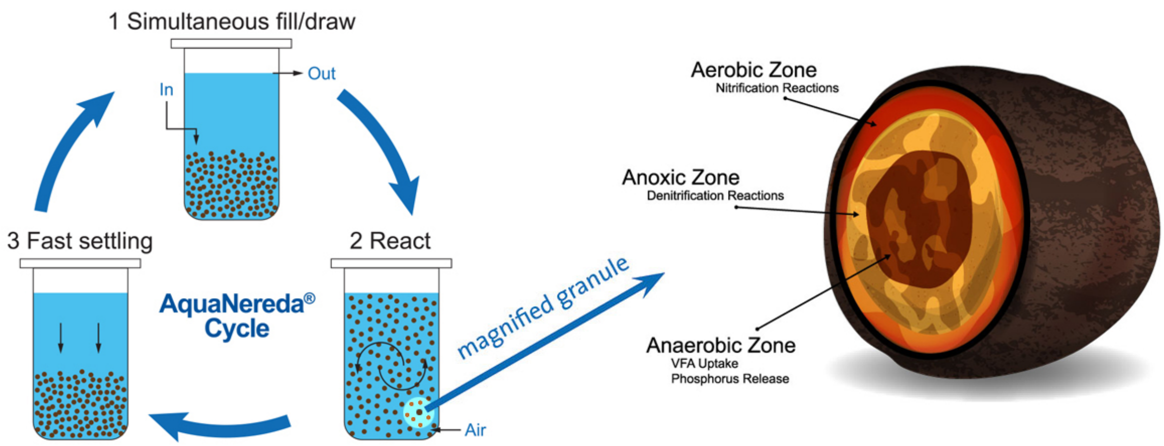


Figure 5-10. Nereda Process

Courtesy: Aqua Aerobic Systems

5.2.4 Oxidation Ditch

An oxidation ditch is a variation of the complete-mix extended aeration activated-sludge process. The process generally has a long solids residence time (SRT) and high mixed liquor suspended solids (MLSS) concentration, making it resilient to upset by peak organic loads. The typical SRT for oxidation ditches ranges from 15 to 30 days, and the MLSS is generally between 2,000 and 5,000 mg/L. Oxidation ditches are often oval in shape and have been called “racetrack” reactors. The depth of the ditch typically ranges from 4 to 12 feet. Mechanical aerators in the ditch provide aeration and mixing. Strategic placement of the aerators creates aerobic and anoxic zones within the oxidation ditch, for effective nitrification and denitrification. Biological phosphorus removal is also possible.

Oxidation ditches are usually preceded by preliminary treatment, such as screening and grit removal. Primary settling is typically not included upstream of oxidation ditch systems. Return activated sludge is pumped from the secondary clarifier back into the ditch.

Figure 5-11 presents a schematic of an oxidation ditch. Typically, rotating brush or disc mechanical aerators are used to move mixed liquor around the tank and to provide aeration. The aerators help mix scum into the water column for treatment. The rigorous mixing action of the mechanical aerators can generate off-spray. Oxidation ditches are not available as packaged treatment systems. Because of the large footprint requirements and non-availability of packaged treatment units, the oxidation ditch process is eliminated from further evaluation.

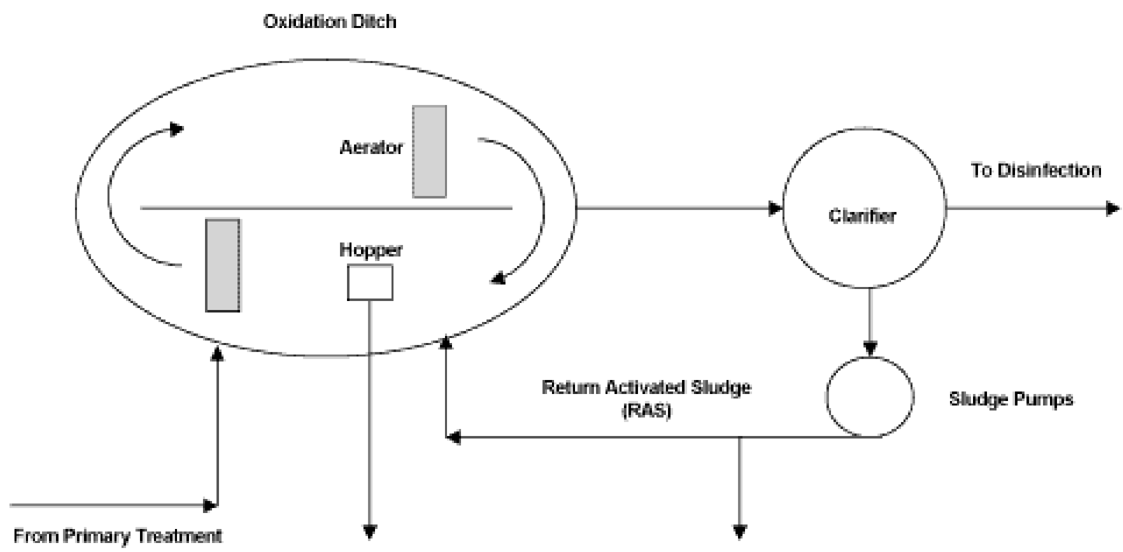


Figure 5-11. Typical Oxidation Ditch Schematic

5.2.5 Extended Aeration Activated Sludge Package Plant

Extended aeration is a less-complex system which can operate without primary treatment or anaerobic digestion. The treatment provides a completely mixed process operated at long hydraulic detention times and high sludge age. The process uses larger aeration tanks with extended solids retention times (SRTs) of over 20 days. Careful consideration needs to be given to the capacities of motors, pumps, and compressors in order to ensure the process can handle variations in flow. The basic extended aeration process schematic is shown in Figure 5-12.

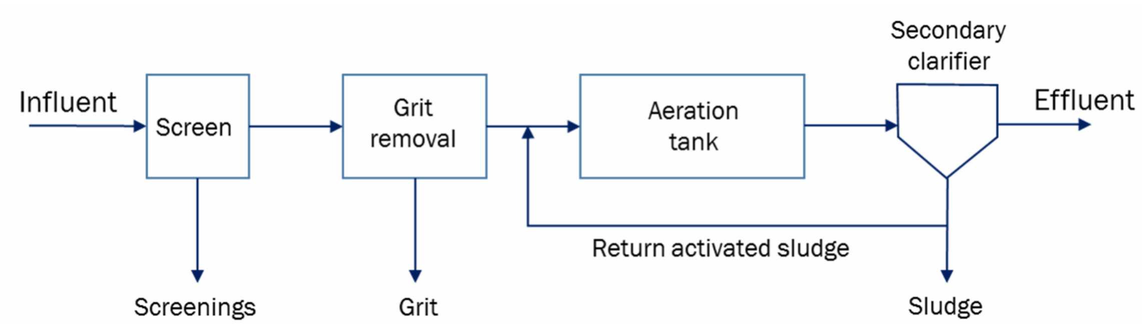


Figure 5-12. Extended Aeration Process Schematic

The process is generally limited to smaller WWTPs and is often used in prefabricated packaged plants. The range of typical SRTs on the mainland is 20 to 40 days, and the process generally operates with MLSSs between 2,000 and 5,000 mg/L. The long SRT and relatively high MLSS makes the process resistant to shock loading and stable but requires somewhat larger tanks and therefore incurs higher aeration costs for a given flow, compared to other forms of activated sludge. Sludge settling can be problematic in the tropics due to denitrification occurring in mixed liquor caused by the relatively high water temperatures. The process is similar to the oxidation ditch technology previously described but would use diffused aeration rather than mechanical aeration. The process is forgiving and resistant to shock loadings. Due to sludge settling challenges in the tropics this process will not be considered further.

5.2.6 Activated Sludge with Anoxic Selector

This process is similar to extended aeration but would employ a shorter SRT of less than 10 days and would operate at a MLSS concentration between 1,500 and 4,000 mg/L. Figure 5-13 shows a process schematic for this process. The Kihei WWRF and Wailuku-Kahului WWRFs on Maui operate with this process. The anoxic selector is typically sized to have a volume of approximately 10 to 30 percent of the total aeration basin volume. The process would not be as forgiving and resistant to shock loadings compared to the oxidation ditch and extended aeration processes due to the shorter SRT and lower MLSS concentration. But this option is available in a prefabricated package plants and would incur a smaller footprint than the oxidation ditch and extended aeration processes but would require operation and maintenance of blowers to provide air to the process. The fine bubble diffused aeration system would be more efficient than the mechanical aerators generally used in the oxidation ditch process. This process is retained for further evaluation.

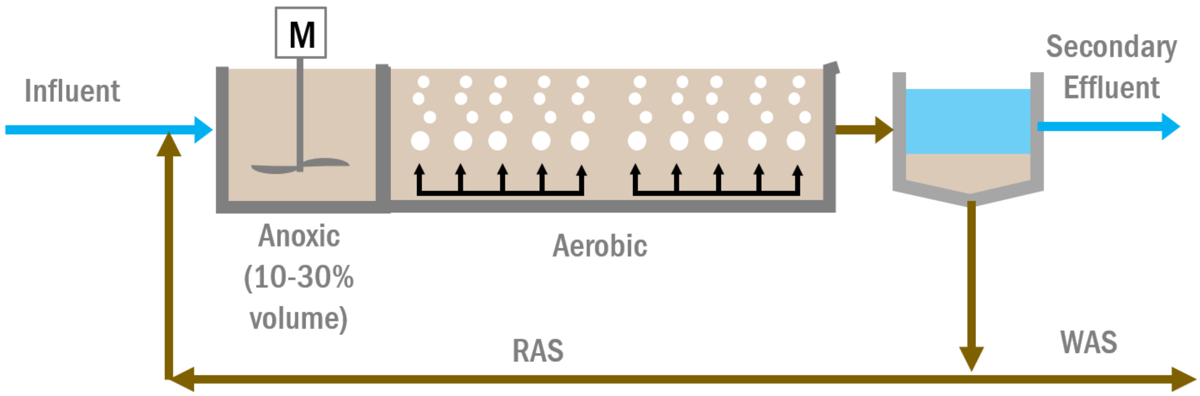


Figure 5-13. Activated Sludge with Anoxic Selector Process Schematic

5.2.7 Recirculating Gravel Filter

Recirculating gravel filter (RGF) technology is an effective technology to treat septic tank effluent wastewater. After collection and conveyance, the wastewater is treated, in this case using a recirculating pea gravel filter. RGFs are a relatively simple, but effective means to treat wastewater from small communities. RGFs have been used to treat flow rates up to 1.0 mgd. RGFs typically produce a nitrified effluent that contains less than 10 mg/L of BOD_5 and TSS (Crites and Tchobanoglous, 1998).

A schematic diagram of a RGF is shown in Figure 5-14. A septic tank is used to capture settleable and floatable solids. The septic tank effluent enters a recirculation tank. A dosing pump is used to apply wastewater in small doses to the top of the filter. The wastewater is treated as it percolates through the pea gravel media. A network of drainage piping collects the water at the bottom of the filter and returns it to the recirculation tank. A floating ball recirculation valve controls the return flow back to the recirculation tank or to the effluent disposal or reuse system. The dosing pump timer settings and recirculation tank volume are designed so that wastewater will typically flow through the filter for treatment an average of three to five times before being discharged. An example of a RGF system in use within a decentralized wastewater system can be found at the Stonehurst subdivision, located near Martinez, California (Crites, et. al. 1997).

For a community system with conventional sewers and Imhoff tank can be used in lieu of a septic tank. Imhoff tanks are designed to remove floatable and settleable solids, and also provides for some digestion of the removed materials.

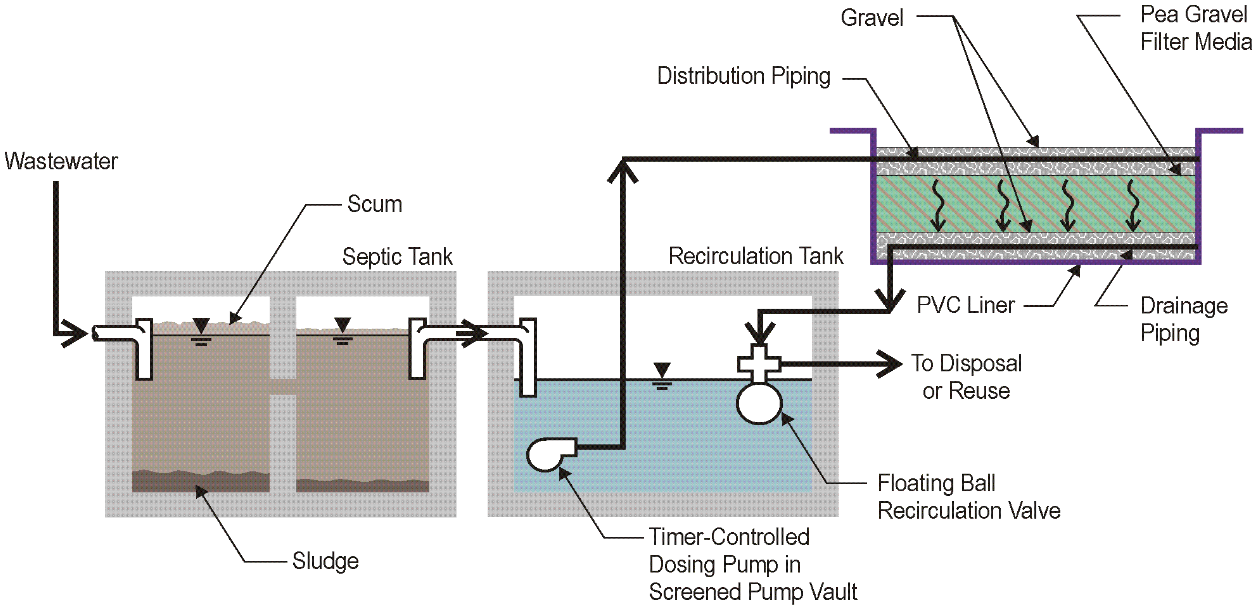


Figure 5-14. Recirculating Gravel Filter for Treatment of Septic Tank Effluent

5.2.8 Secondary Treatment Technology Screening

Table 5-5 provides a screening evaluation of the secondary treatment technologies described above. MBR, activated sludge with anoxic selector, and recirculating gravel filter are carried forward as project alternatives in Section 7.

Table 5-5. Screening of Secondary Treatment Options							
Criterion	MBR	SBR	Nereda	Oxidation Ditch	Extended Aeration	Activated Sludge with Anoxic Selector	Recirculating Gravel Filter
$BOD_5 \leq 30$ mg/L	X	X	X	X	X	X	X
TSS ≤ 30 mg/L	X	X	X	X	X	X	X
Nitrification	X	X	X	X	X	X	X
Total Nitrogen < 10 mg/L	X	X	X	X		X	
Anoxic selector	X	X	X	X		X	
Appropriate for remote island location	X			X	X	X	X
Appropriate for tropical climate	X	X	X	X		X	X
Aeration tank size	Small	Moderate	Small	Large	Large	Moderate	Not applicable, but largest overall footprint
Secondary clarifier size	None	None	None	Largest	Largest	Large	Not applicable, but largest overall footprint
Energy requirement	Highest	Moderate	Moderate	Moderate	Higher	Moderate	Low
Operational complexity	High	High	High	Moderate	Moderate	Moderate	Low
Available as packaged treatment system	X	X	X		X	X	
Fatal flaw		Proprietary control systems	Limited full scale installations in U.S.	Large footprint	Large footprint		
Carry forward in evaluations	X					X	X

5.3 Disinfection

Disinfection processes selectively kill pathogens or render them incapable of reproduction or harm to humans. Disinfection at WWTPs is employed for the purposes of protection of public health, reduction of organic matter, inorganics, nutrients, odor, aesthetics, and maintaining waste-assimilative capacity of receiving water bodies. The protection of public health through the control of disease-causing microorganisms is the primary reason for wastewater disinfection (WEF, 1996). As the last barrier of protection from pathogenic organisms, disinfection at WWTPs is an important process. To address disinfection, both sodium and calcium hypochlorite system and a UV system were evaluated.

5.3.1 Sodium Hypochlorite

Sodium hypochlorite (NaOCl) is the most commonly used form of liquid hypochlorite. It is an effective disinfectant at relatively low concentrations. Commonly known as “bleach,” NaOCl can be found in concentrations ranging from 1.5 to 15 percent. Household bleach usually has a NaOCl concentration of 3 to 6 percent. Swimming pool sanitizers usually have a NaOCl concentration of 11 to 15 percent. A concentration of 12.5 percent is commonly used for wastewater treatment and will degrade to 10 percent in 6 to 8 weeks. The product of degradation is chlorine gas. At low concentrations the product is relatively stable. The solution should be stored in a cool, dark area in a non-corrosive container. NaOCl is corrosive and toxic.

Scaling can occur in pipes and valves used to transport diluted NaOCl if hardness is present in the carrier water. The carrier water can be softened prior to mixing with the NaOCl or systems can be designed to transport undiluted (neat) NaOCl.

Once added to the wastewater, the NaOCl dissociates to form hypochlorous acid and hypochlorite. The combined amount of hypochlorous acid and hypochlorite is referred to as free chlorine. When reacted, the compounds formed with the free chlorine also have some disinfecting potential. These chlorine residuals are desired in some processes, but are toxic to aquatic life if the processed water is discharged to an open stream. Disinfection with hypochlorite can form DBPs similar to chlorine gas. For WWTPs, NaOCl can be delivered in bulk or generated on site.

5.3.1.1 Bulk Sodium Hypochlorite

Bulk NaOCl can be obtained in totes, drums, or smaller packages as desired. Currently, NaOCl is produced on Oahu and transported to neighbor islands in totes. NaOCl solutions are applied with a metering pump or suction injector. Table 5-6 lists advantages and disadvantages of bulk NaOCl.

Table 5-6. Bulk Sodium Hypochlorite–Advantages and Disadvantages

Advantages	Disadvantages
• Able to oxidize at relatively low concentrations • Use is common at WWTPs • Readily available • Simple O&M–application with metering pumps • Stored and used as a liquid	• Corrosive and toxic • Degrades quickly at high concentrations and temperatures • Forms DBPs • Scaling in pipes and valves can occur • High transportation cost due to weight of liquid • 12.5% NaOCl is considered a hazardous chemical. A release of 100 lbs (approximately 12 gallons) or more is considered a reportable quantity. • Air binding of pumps is possible at high temperatures due to off-gassing, but can be mitigated through proper system design.

The chlorine demand at the Naalehu WWTP is anticipated to be relatively small due to the small WWTP size and method of effluent disposal. Bulk NaOCl is not recommended due to cost and the chemical degradation rate in storage.

5.3.1.2 On-Site Generation

On site generation comprises mixing softened water with salt to form brine and then passing an electric current through the brine to produce NaOCl. When generated on site, the resulting solution is a relatively weak solution, usually around 0.8 percent. Because the process includes passing an electrical current through the solution, the electrical power demand can be quite high. Figure 5-15 is a schematic illustration of NaOCl generation system.

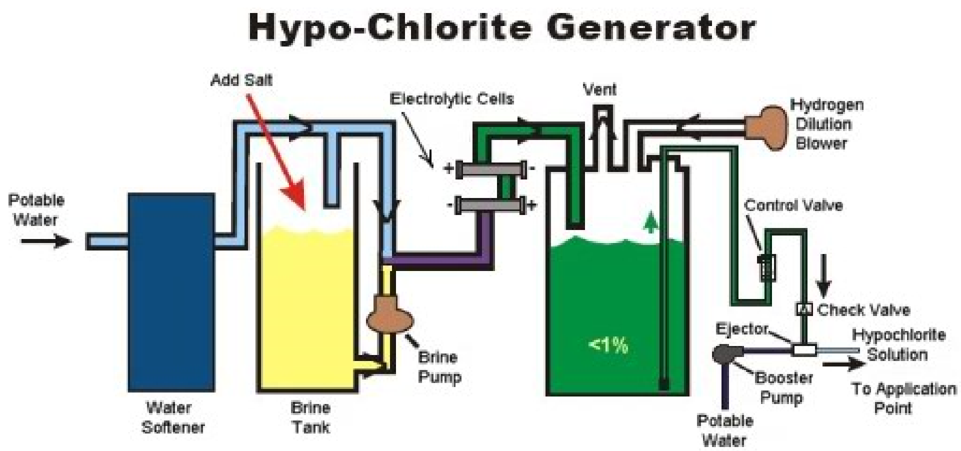


Figure 5-15. Schematic Diagram of an On-site NaOCl Generator

In addition to water hardness, the levels of silica present in the source water can cause build up problems in the on-site NaOCl generator. The level of pretreatment required is dependent on the water quality. A detailed water quality analysis would be necessary if this disinfection technology were chosen. Table 5-7 lists advantages and disadvantages of onsite NaOCl generation.

Table 5-7. Onsite Sodium Hypochlorite Generation–Advantages and Disadvantages

Advantages	Disadvantages
• Salt is readily available and inexpensive • Salt is transported as a solid • Relatively stable liquid product (low concentration) • 0.8% solution produced by on-site generation is not considered a hazardous chemical and is safer to handle than more concentrated solutions • Air binding in pumps due to off-gassing less likely to occur than with more concentrated solutions	• High electrical power demand • High maintenance requirements • Generation process generates flammable hydrogen gas • Disinfection by-products formed

Onsite NaOCl generation is not recommended at the Naalehu WWTP due to the high maintenance requirements.

5.3.2 Calcium Hypochlorite

Calcium hypochlorite is the most common solid form of hypochlorite used for disinfection. It can be found as a powder, granules, pellets, or as tablets in concentrations up to 70 percent. Calcium hypochlorite will degrade in strength at a rate of 3 to 5 percent per year. Once applied to the wastewater, the chemistry is similar to that for NaOCl (i.e., bleach). Calcium hypochlorite decomposes in an exothermic reaction if exposed to moisture.

The solid can be directly applied to wastewater at small WWTPs. Figure 5-16 shows a typical calcium hypochlorite feed system.

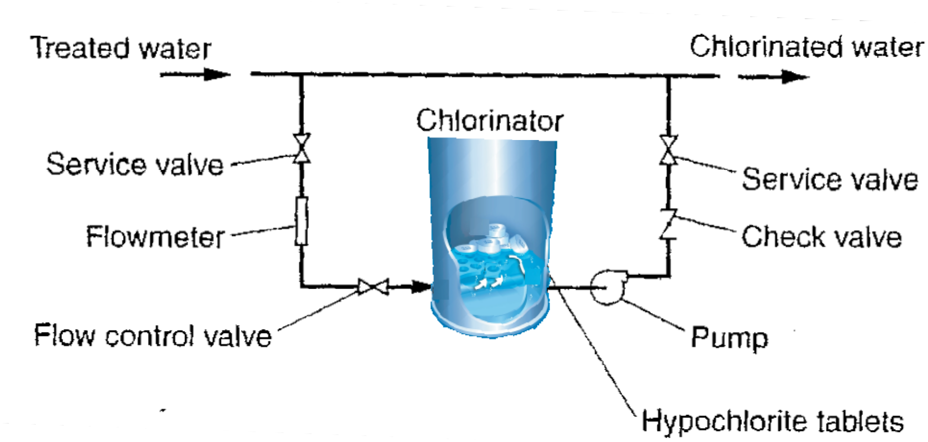


Figure 5-16. Typical Calcium Hypochlorite Feed System

The advantages of using calcium hypochlorite for disinfection at small, remote WWTPs is that it is available in concentrated form as powder, pellets, or tablets. This makes the transportation and storage of disinfectant optimal for small WWTPs. Table 5-8 summaries calcium hypochlorite characteristics.

Table 5-8. Calcium Hypochlorite Summary

Description	Characteristic
Transported form	Solid
Typical transported concentration	70%
Largest transported volume available	55 pound pails
Decay rate	Decays 3-5% per year
Hazards	Toxic if ingested (usually through dust or liquid form)
Storage constraints	Must be stored in a cool, dry, dark place
Special equipment	Tablet feeder
Particular issues	Heats and combusts if not stored properly, scaling in pipes, off gassing

5.3.2.1 Dose and Contact Time

The effectiveness of a chlorination system is highly dependent on the characteristics of the wastewater, the initial mixing and contact time, and the chlorine dose used. For nitrified effluent, the recommended dose is between 4 and 8 mg/L (Crites and Tchobanoglous, 1998).

Table 5-9 lists the chlorine demand for various flow conditions. Equipment will be sized to provide chemical feed at a rate of up to 100 lbs/day, which will ensure an adequate chlorine dose for peak wet weather discharge flows. The recommended minimum contact time for chlorination is 15 minutes (Ten States Standards Wastewater, Recommended Standards for Wastewater Facilities, 1997, Great Lakes–Upper Mississippi River Board of State and Provincial Public health and Environmental Managers).

Table 5-9. Chlorine Demand

Description	Flow (mgd)	Chlorine Demand (lbs/day)
Average dry weather flow	0.125	4-8
Peak day wet weather flow	0.322	11-22
Peak hour wet weather flow	0.457 (317 gpm)	15–31

5.3.3 Ultraviolet Light Disinfection

A common alternative to chlorine disinfection is ultraviolet light (UV). Ultraviolet systems destroy microorganisms by affecting their deoxyribonucleic acid (DNA) and ribonucleic acid (RNA) and impeding their ability to reproduce. A UV disinfection system is comprised of lamps, a reactor, and control panel. Wastewater can flow either parallel or perpendicular to the lamps in the reactor, while the control box provides a starting voltage and maintains the continuous current needed. Currently, most systems are equipped with an automated lamp cleaning system, to maintain lamp efficiency levels.

A UV system's effectiveness is dependent on the characteristics of the wastewater, the dose, and the exposure time. In the case of UV radiation, the most important factor is the transmittance of the water, which has a direct effect on the ability of UV light to penetrate through the liquid and reach microorganisms present at the required intensity. Ideally, the discharge undergoing treatment should not have a transmittance lower than 55 percent, with the intensity decreasing the farther the microorganisms are from the lamp. The optimum wavelength to effectively inactivate microorganisms is between 250 and 270 nanometers.

The main types of UV lamps used for wastewater disinfection are conventional low-pressure lamps, low pressure high output (LPHO) lamps and medium pressure lamps. Several UV systems include lamps with automated sleeve cleaning.

5.3.3.1 UV System Design Summary

A UV disinfection system requires about the same size footprint as chlorine at small WWTPs. Disinfection occurs as the organism is exposed to the UV radiation as the water flows past the UV lightbulbs. The Trojan UV3000+ system is used at numerous facilities across the U.S., including some treatment plants in Hawaii. The estimated cost included in this report are based on an assumed UV transmittance of 65 percent. The amalgam lamp used with the UV3000+ system has an end-of-lamp-life factor (ELLF) of 0.98 indicating little loss in UV light output over the life of the lamp. This ELLF has been tested and approved by the State of California and is also accepted by the State of Hawaii for reuse applications. The system would use LPHO lamps with automatic sleeve cleaning. LPHO lamps are energy efficient and the UV3000+ system is furnished with automatic sleeve cleaning devices to reduce labor requirements. Each UV lamp is enclosed in a quartz sleeve to separate it from the water medium. Each lamp draws 254 watts at full output and is driven by electronic ballast. The electronic ballast allows the lamps to be dimmed to conserve power based on

a control signal from a flow meter. The LPHO lamps will have a minimum life of 12,000 hours when operated in an automatic mode and limited to a maximum of 4 on/off cycles per 24 hours.
Table 5-10 summarizes the size and design criteria for the UV system required to treat the WWTP discharge.

Table 5-10. UV Disinfection Design Summary

Description	Value
Peak Hour Wet Weather Discharge	317 gpm
Minimum UV transmittance	65 percent
No. of UV channels	1
Design dose	35,000 µWs/cm2
Disinfection limit	30 MPN per 100 mL *(E.coli)*
Validation factors	0.98 end of lamp factor

MPN = most probable number

µWs/cm2 = microwatts per second per square centimeter

5.3.4 Cost Evaluation

A summary of capital and life-cycle estimated costs for both chlorination and UV disinfection is presented in Table 5-11 for comparison. Additional detail is included in Appendix A. The capital costs include the materials and equipment costs, construction costs, electrical, instrumentation and control, soft costs, and contingency. As shown in the table, the UV option incurs higher capital costs. The life-cycle costs look at the impact of the capital costs along with the annual operations and maintenance costs, including power, materials, chemicals, and labor costs over the next 30 years. The life-cycle costs for chlorination option appear to be about 55 percent of the UV option.

Table 5-11. Estimated Disinfection Costs

Description	Tablet Chlorination System	UV System
Capital Cost	$150,000	$1,100,000
Annual Operation and Maintenance	$25,000	$10,000
Life-cycle Cost (30-Year Net Present Value)	$740,000	$1,340,000

**Does not include annual labor.*

5.3.5 Disinfection Recommendation

A tablet chlorination feed system is the recommended disinfection option over the UV system for the Naalehu WWTP because it incurs lower capital and life-cycle costs. In addition, tablet chlorination will be more reliable than UV due to frequent “dirty power” conditions on the island. The County may elect to install a UV system at the Naalehu WWTP should they choose to pursue an R1 water recycling program in the future.

The proposed effluent management system (subsurface drip irrigation disposal) does not require a disinfection process to protect human health and the environment because the treated effluent is dispersed below the ground surface. However, periodic maintenance chlorination of the subsurface drip system will be required to reduce biofilm fouling within the drip lines.

Section 6

Solids Management

This section evaluates solids management options for the Naalehu WWTP.

6.1 Aerobic Digestion with Decant Thickening

Aerobic digestion consists of aerating sludge in a tank for an extended period of time. Volatile solids are oxidized in the process, stabilizing the sludge and reducing the total mass of solids that must be managed by recycling or disposal. Pathogen densities are also reduced. The process does not produce biogas. The aerobic digestion process requires substantial energy input in the form of aeration blowers, and therefore is not typically used at larger (i.e., greater than 10 million gallons per day [mgd]) wastewater reclamation facilities (WWRFs).

Many small (less than 5 mgd) wastewater treatment plants in the United States use aerobic digestion to stabilize solids, due to its relatively low capital costs, simplicity, and compatibility with the certain liquid treatment processes.

Aerobic digestion with decant thickening is a two-stage process that can be achieved in the same basin. The first stage includes a period of aerobic digestion as described above. In the second stage the blowers are turned off for a period of time to allow sludge to settle and thicken. Supernatant is then decanted off the top. The blowers are turned back on to continue the aerobic digestion process. This process is repeated a few times until the sludge reaches approximately three percent solids. It is then pumped to the next process.

Aerobic digestion with decant thickening is recommended for the proposed Naalehu WWTP due to its simplicity, low cost, and effectiveness for small WWRFs.

6.2 Anaerobic Digestion with Biogas Use

Anaerobic digesters are covered tanks equipped with mixing, heating, and biogas collection systems. Anaerobic bacteria in the digesters convert organic matter into methane, carbon dioxide, and water; pathogen densities are reduced; and a stabilized sludge is produced. Modern high-rate digesters are typically single-stage reactors. Mesophilic anaerobic digesters are typically operated at temperatures between 35 degrees Celsius (°C) and 38°C. Mesophilic digestion systems produce a Class B biosolids product if the solids retention time (SRT) is greater than 15 days.

Two-stage mesophilic anaerobic digestion, where digesters are operated in series, improves process performance. The second-stage anaerobic digester generally has less SRT than the first stage. The advantages of this process configuration are slightly improved volatile solids reduction, a product with reduced pathogen content, and less product odor potential.

The anaerobic digestion process generates biogas that can be used for digester heating and generation of electricity.

The mesophilic anaerobic digestion process requires primary sludge to operate effectively. Therefore, primary clarifiers are required for an anaerobic digestion process. WWRFs that do not have primary clarifiers must use other digestion technologies.

Anaerobic digestion is cost effective for facilities larger than 5 to 10 mgd. Anaerobic digestion is not considered to be an appropriate technology for a facility the size of the Naalehu WWTP.

6.3 Dewatering

A dewatering process is used to remove excess water from digested sludge to form a semi-solid "cake" product.

6.3.1 Centrifuge Dewatering

Centrifuges are a commonly used dewatering technology. Centrifuges provide the best dewatering performance of the dewatering technologies presented in this report but require the highest energy input to do so. High-solids centrifuge machines typically achieve anaerobically digested dewatered cake of approximately 23 to 28 percent total solids content. Aerobically digested sludge will typically have lower total solids content, on the order of 12 to 15 percent. The process is shown in Figure 6-1.

Centrifugal force of 500 to 3,000 times the force of gravity is applied to the biosolids within the centrifuge, separating liquid from the solids. The centrifuge has a solid bowl that spins at a high rate. Liquid sludge, conditioned with polymer, is introduced within the rotating bowl. The sludge spins with the bowl, separating into liquid and solid fractions. A screw conveyor mechanism spins within the rotating bowl at a slightly faster or slower speed than the bowl to facilitate moving the solids fraction towards one end of the bowl, where it is discharged. The centrate (removed liquid) is discharged through another port. The process operates continuously. Required ancillary equipment includes sludge feed pumps, polymer feed systems, and sludge cake conveyance systems.

Centrifuges are sized based on hydraulic and solids throughput. Machines are available to dewater sludge flow rates ranging from 25 gpm to 700 gpm. High-solids machines can produce a very well-dewatered material, if anaerobic digestion is used.

Centrifuges require a high level of operator due to the high rotational speed. For this reason, they are typically not used at small WWTPs like Naalehu.

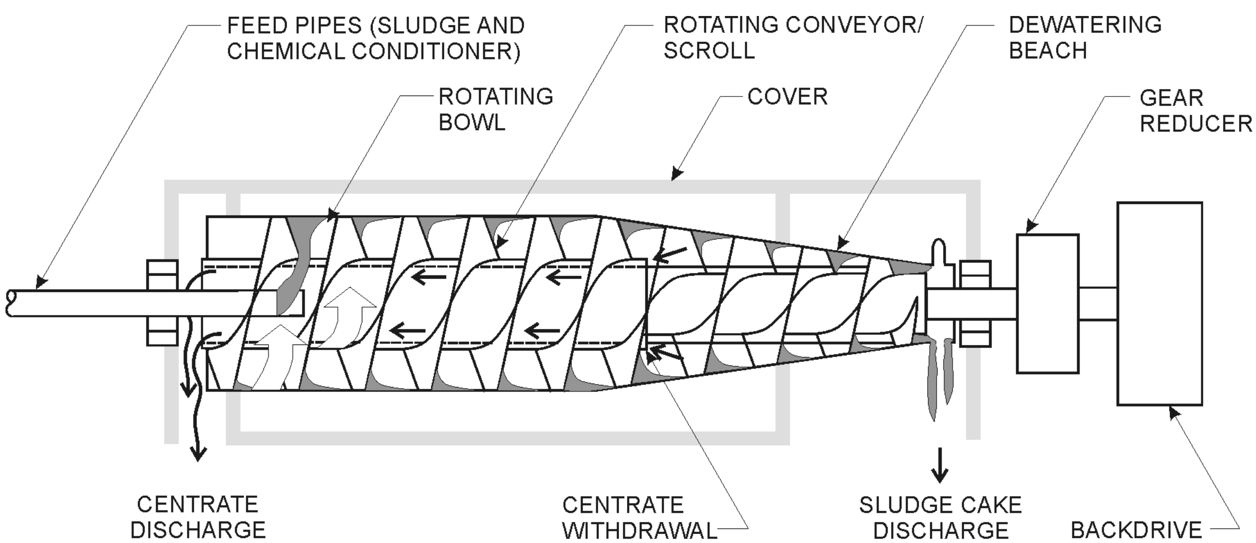


Figure 6-1. Centrifuge Dewatering

6.3.2 Screw Press Dewatering

The screw press represents a relatively new technology for dewatering municipal wastewater solids, although the technology has been used successfully in industrial, pulp and paper production, chemical, and food processing applications.

Figure 6-2 shows a diagram of a screw press. Thickened sludge, conditioned with polymer, is introduced to the machine in the head box at the inlet end. The mixture is conveyed from the inlet end to the outlet end of the press by the rotating screw. As the material is conveyed along the length of the press it is squeezed between the tapered screw shell and the screen drums. The dewatered solids exit the press at the discharge end and fall down the discharge box. The adjustable pressure cone provides back pressure within the machine, particularly when the machine is initially filled. For municipal wastewater solids applications, the pressure cone is typically not needed after the machine is filled; the dewatered sludge provides sufficient back pressure. The liquid that was forced out through the screens is returned to the liquid treatment process.

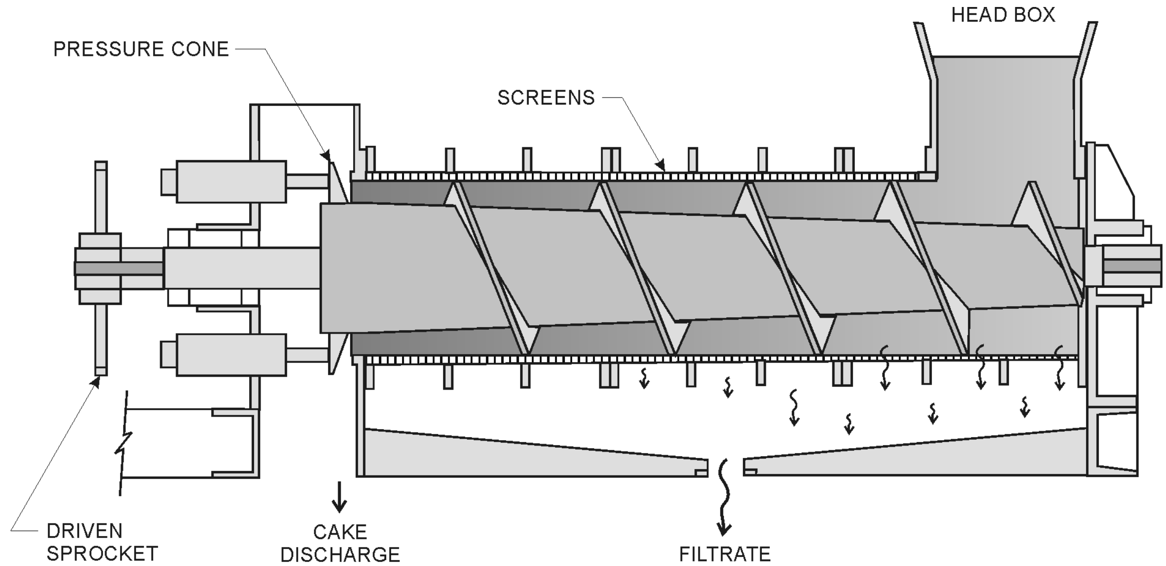


Figure 6-2. Screw Press Diagram

The screw press operates at a very slow rotational speed. The screw rotation is usually one-half of a revolution per minute or less for municipal wastewater solids. Water is slowly forced from the sludge by squeezing action–similar to a belt filter press–but for much longer periods of time. The solids retention time in a screw press can be on the order of 2 hours. The simplicity of screw presses makes them practical for small wastewater treatment plants, such as Naalehu.

6.4 Disposal

Dewatered solids, grit, and screenings would be trucked to the West Hawaii Landfill for disposal.

Section 7

Project Alternatives Evaluations

This section presents evaluations of the three project alternatives developed for the new wastewater treatment plant (WWTP).

7.1 Project Alternative Descriptions

Three project alternatives were developed as part of the tasks completed for this Preliminary Engineering Report. All three alternatives include a new C Brewer house lot collection system (i.e., Phase 1 + Phase 2), WWTP, and subsurface drip effluent disposal system.

7.1.1 Project Alternative 1: Activated Sludge with Anoxic Zone Package Plants

Project Alternative 1 is comprised of an activated sludge process with anoxic zone provided in the form of packaged treatment systems. A typical packaged treatment system of this type would include the following elements:

- Flow equalization
- Anoxic treatment zone
- Aerobic treatment zone
- Secondary clarifier
- Aerobic digester with decant thickening

Figure 7-1 presents a sketch of Project Alternative 1. Wastewater would receive preliminary treatment in the headworks before flowing into the packaged treatment system. Two package treatment units would be provided, each with 62,500 gallons per day (gpd) capacity. Effluent would flow into an irrigation equalization tank before being applied to the subsurface drip disposal system.

Digested solids would be dewatered using a screw press prior to disposal at the landfill.

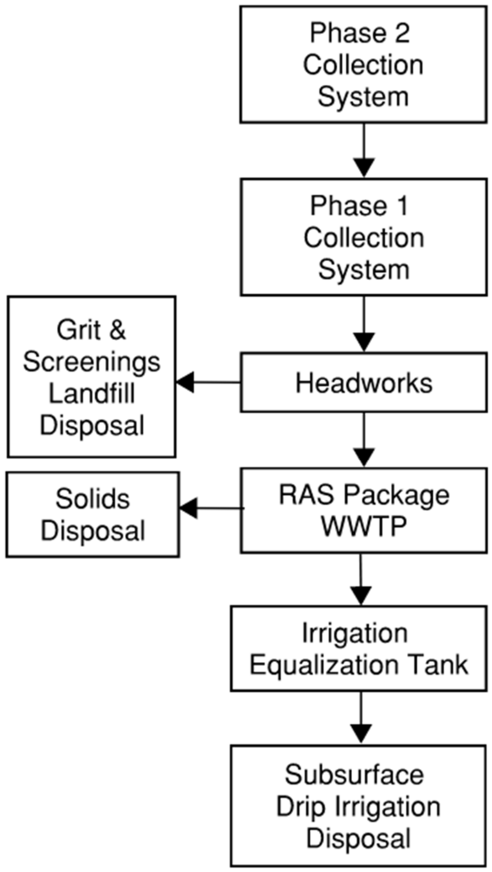


Figure 7-1. Project Alternative 1: Activated Sludge with Anoxic Zone Package Plants

7.1.2 Project Alternative 2: MBR Package Plants

Project Alternative 2 is similar to Project Alternative 1 but includes two MBR package plants to provide treatment. Figure 7-2 provides an outline of Project Alternative 2. The MBR technology would create effluent that could be recycled for irrigation and/or stock watering purposes, if desired in the future. However, recycled water distribution costs are not included in the evaluations below to allow all alternatives to be considered on an equal basis.

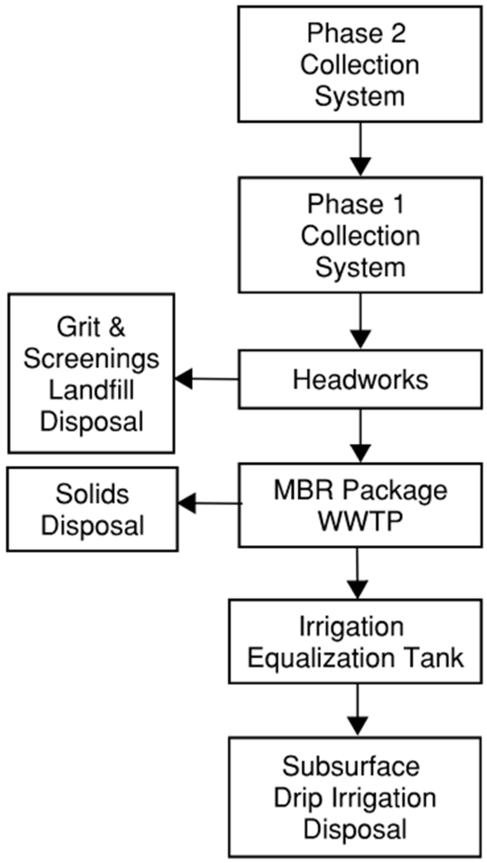


Figure 7-2. Project Alternative 2: MBR Package Plants

7.1.3 Project Alternative 3: Imhoff Tank/Recirculating Gravel Filter

Project Alternative 3 incorporates recirculating gravel filter treatment technology. Figure 7-3 provides a schematic of the project alternative. An Imhoff tank would be provided downstream of the headworks to remove grease and settleable solids prior to flowing into a recirculation tank. Recirculation pumps would distribute water from the recirculation tank over the surface of the pea gravel filter that provides secondary treatment. Water collected at the bottom of the filter would flow back to the recirculation tank. On average water would flow through the filter five times before disposal in the subsurface drip irrigation system as previously described.

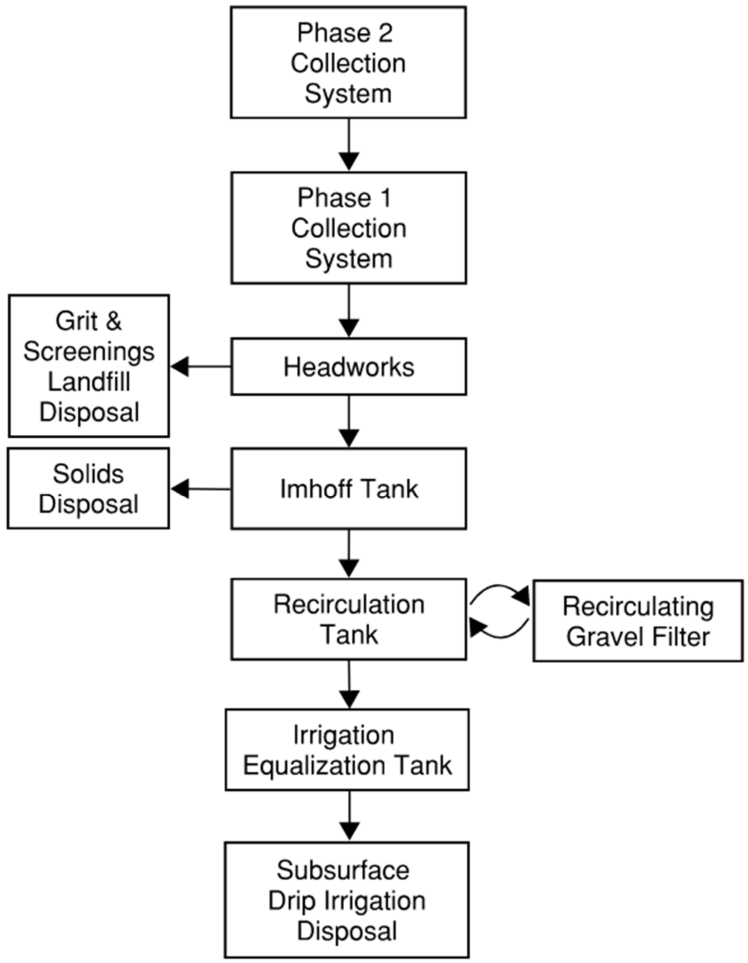


Figure 7-3. Project Alternative 3: Imhoff Tank/Recirculating Gravel Filter

7.2 Cost Evaluations

Capital, operations and maintenance (O&M), and life-cycle cost evaluations are presented in this section.

7.2.1 Capital Costs

Conceptual cost estimates were created for the three project alternatives. The cost estimates were developed using construction bids from similar projects, quantity take-offs, vendor quotes, and other sources. The costs were adjusted to account for economies of scale and construction inflation since the bid opening date. Where Hawaii costs were unavailable, U.S. mainland costs were used after adjustment to reflect Hawaii Island conditions.

In accordance with the Association for the Advancement of Cost Engineering International (AACE) criteria, these are Class 5 estimates. A Class 5 estimate is defined as a Conceptual Level or Project Viability Estimate. Typically, engineering is from 0 to 2 percent complete. Class 5 estimates are used to prepare planning level cost scopes or evaluation of alternative schemes, long range capital outlay planning.

Expected accuracy for Class 5 estimates typically ranges from -50 to +100 percent, depending on the technological complexity of the project, appropriate reference information and the inclusion of an appropriate contingency determination. In unusual circumstances, ranges could exceed those shown. Table 7-1 provides a summary of capital cost assumptions used.

Table 7-1. Capital Cost Estimating Assumptions

Description	Value
Estimate date	June 2023
Engineering News Record 20-Cities Average Construction Cost Index	13,345
Electrical and instrumentation markup	25 percent
Engineering, administration, and legal markup	25 percent
Estimating contingency for unknowns	20 percent

Table 7-2 provides a summary of the capital cost estimates, in current (June 2023) dollars. Detailed estimates can be found in Appendix A.

Table 7-2. Capital Cost Estimates Summary

Description	Alternative 1: Activated Sludge Package Plants	Alternative 2: MBR Package Plants	Alternative 3: Imhoff Tank/RGF
Collection system	$40.2 million	$40.2 million	$40.2 million
Drainage channel improvement	$19.9 million	$19.9 million	$19.9 million
Wastewater treatment	$20.7 million	$20.3 million	$29.6 million
Effluent disposal	$4.0 million	$4.0 million	$4.0 million
Totals	**$84.7 million**	**$84.3 million**	**$93.6 million**
AACE Class 5 estimate range	$42.4–$169.5 million	$42.2–$168.6 million	$46.8–$187.2 million

As shown in the table, all three project alternatives have similar capital costs, and can be considered equal at this level of analysis.

7.2.2 Operation and Maintenance Costs

O&M costs estimates were developed for the three alternatives. The O&M cost estimates include collection system maintenance, plus estimates of labor, electricity consumption, chemicals, maintenance materials and solids disposal for the WWTP. O&M assumptions are listed in Table 7-3. The O&M estimates are based on the WWTP average dry weather flow capacity.

Table 7-3. O&M Cost Assumptions

Description	Value
Average dry weather flow	125,000 gpd
Labor cost, loaded	$100,000/year/full time equivalent
Electricity cost	$0.45/kWh
Landfill tip fee	$116/wet ton
Maintenance materials	2 percent of equipment capital cost/year

The O&M estimates for the three project alternatives are summarized in Table 7-4. Details can be found in Appendix A. Per Table 7-4, Project Alternative 3: Imhoff Tank/Recirculating Gravel Filter incurs the lowest O&M cost, while Project Alternative 2: MBR Package Plants incurs the highest.

Table 7-4. O&M Cost Estimate Summary

Description	Annual Cost		
	Project Alternative 1: Activated Sludge Package Plants	Project Alternative 2: MBR Package Plants	Project Alternative 3: Imhoff Tank/RGF
Collection system	$74,000	$74,000	$74,000
Labor	$300,000	$300,000	$300,000
Electricity	$270,000	$300,000	$120,000
Chemicals	$27,000	$32,000	$27,000
Maintenance materials	$117,000	$113,000	$84,000
Solids disposal	$67,000	$67,000	$67,000
Totals	**$855,000**	**$886,000**	**$672,000**

7.2.3 Life-Cycle Costs

An economic evaluation was prepared to assess the potential life-cycle costs associated with each project alternative. The economic evaluation consists of a net present value (NPV) comparison. The NPV analysis includes capital, O&M, and equipment replacement costs. An appropriate inflationary factor and discount rate are applied to obtain the NPV over a 30-year planning period.

The NPV of an alternative represents the amount of money that would need to be set aside today (at a given interest rate) to pay the costs associated with the alternative over the entire planning period. The alternative with the lowest NPV is considered the most attractive from an economic perspective. The evaluation results are included in Appendix A.

Table 7-5 summarizes the life-cycle cost evaluation assumptions.

Table 7-5. Life-Cycle Economic Assumptions	
Description	**Value**
Year of analysis	2023
Planning period, years	30
Inflation rate, percent	3.5
Discount rate, percent	5.0
Equipment replacement cycle, years	20
Membrane replacement cycle, years	15

Table 7-6 summarizes the results of the life-cycle cost analysis.

Table 7-6. Life-Cycle Cost Analysis Summary

Description	Alternative 1: Activated Sludge Package Plants	Alternative 2: MBR Package Plants	Alternative 3: Imhoff Tank/RGF
Capital cost	$84.8 million	$84.3 million	$93.6 million
Annual O&M cost	$855,000	$886,000	$672,000
Equipment replacement cost (excluding membranes)	$6.3 million	$6.0 million	$4.6 million
Membrane replacement cost	N/A	$60,000	N/A
Life-cycle cost	$109.6 million	$109.8 million	$112.9 million
Comparison to lowest cost alternative	0%	+0.2%	+3.0%

Figure 7-4 shows the results in graphical form. As shown in the table and graph, the three project alternatives have similar capital costs. Alternative 1: Activated Sludge Package Plants and Alternative 2: MBR Package Plants incur nearly equivalent life-cycle costs due to the similarities in design and package plant capital costs. Project Alternative 3: Imhoff Tank/Recirculating Gravel Filter incurs the greatest life-cycle costs, largely due to the high capital cost associated with the Imhoff tank and recirculating gravel filter beds. At this level of analysis all three project alternatives can be considered to have similar life-cycle costs.

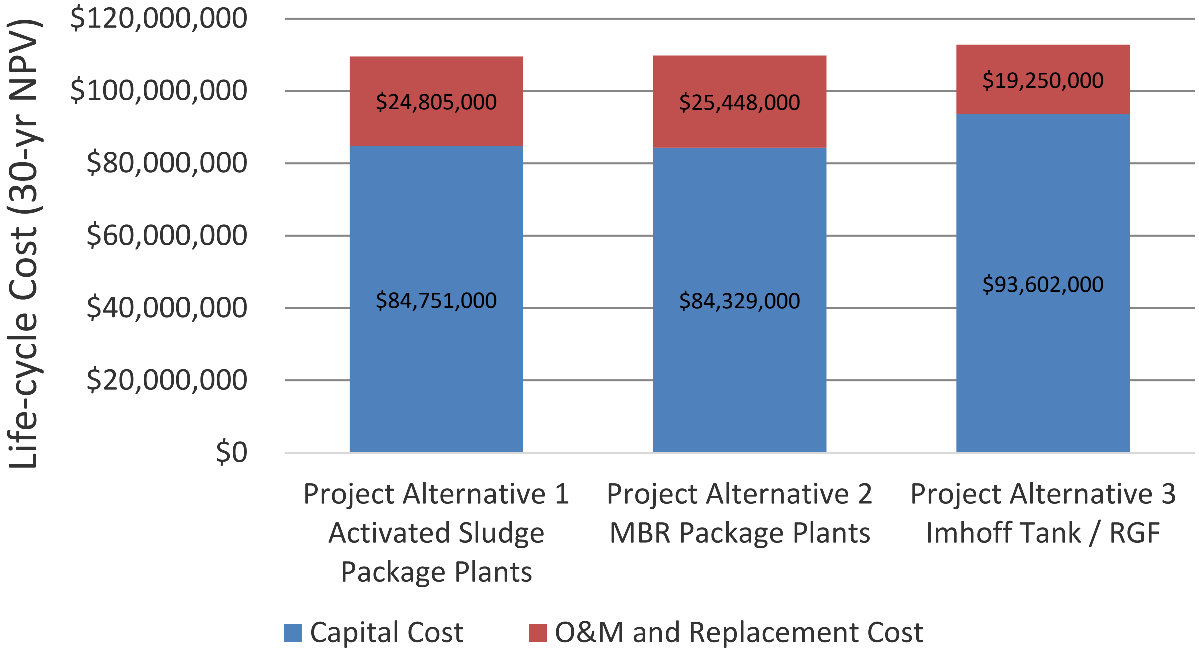


Figure 7-4. Life-Cycle Cost Evaluation Results

7.3 Non-Economic Evaluation

A non-economic evaluation was conducted to provide a qualitative comparison between the three alternatives.

7.3.1 Approach

Non-economic evaluations are generally subjective by necessity. Quantifiable measurements are used when available, but lack of information or difficulty and expense of obtaining information requires subjective assessments.

The project alternatives were scored in relation to one another and according to an evaluation matrix, described below. Each alternative was scored from 1 to 5 (5 = high/desirable, 1 = low/less desirable) for each evaluation criteria. The alternatives were not ranked in the scoring; alternatives could receive the same scores for any given criteria.

The evaluation criteria were weighted to reflect their overall significance for the project. The scores were multiplied by the criteria weights to develop a non-economic score for each alternative.

7.4 Non-Economic Evaluation Criteria

Table 7-7 shows the non-economic criteria chosen for comparing the three alternatives.

Table 7-7. Non-Economic Comparison Criteria

Category	Criteria	Description
Level of Service Measures	Effluent quality	The quality of the effluent produced with respect to BOD_5, TSS, nutrients, and turbidity.
	Potential for capacity expansion	Ability of the system to be expanded should additional capacity be required.
	Water recycling feasibility	The relative extent of modifications that would be needed to create R-1 recycled water to support a future water recycling program.
	Public perception/community concerns	The community's impression of the project and the perceived support.
Regulatory	Monitoring complexity	The relative difficulty of monitoring tasks required for the option chosen.
	Treatment adjustment potential	The ability to increase treatment to comply with future permit requirements and/or growth.
	Safety regulations complexity	The relative difficulty to comply with safety regulations including staff training, reporting, maintenance procedures.
	Environmental concerns	The extent of the project's potential environmental impacts should failures occur.
O&M Factors	Footprint	The physical space that the processes will occupy (affecting land acquisition, subdivision, and permitting).
	Safe work environment	The relative health and safety risk operation of given option will have on the employees; includes equipment access, chemical hazards, confined spaces, dust, etc.; the extent of measures required to ensure the health and safety of the employees.
	Maintenance complexity	The relative intensity of equipment maintenance requirements.
	Operations complexity	The relative intensity of the operations requirements.
Island Factors	Mainland delivery dependence	The relative dependence on regular deliveries of equipment, supplies, or spare parts from mainland sources.
	Mainland servicing dependence	The relative degree to which technology will require special servicing by mainland-based personnel.
	Power dependence	The relative degree to which the treatment processes depend on electrical power for operation.
	Chemical dependence	The relative dependence on chemical supplies, whether locally available or restricted by mainland delivery schedules and requirements.

The categories and criteria were developed using best engineering judgment and our understanding of the project, and the County of Hawaii Department of Environmental Management goals and concerns.

The weighting factors used for the non-economic comparison are listed in Table 7-8.

Table 7-8. Non-Economic Comparison Criteria Weighting Factors	
Category	**Criteria**
Level of Service Measures (25%)	• Effluent quality (30%) • Potential for capacity expansion (20%) • Water recycling feasibility (30%) • Public perception/community concerns (20%) *Category Total (100%)*
Regulatory (25%)	• Monitoring complexity (25%) • Treatment adjustment potential (25%) • Safety regulations complexity (25%) • Environmental concerns (25%) *Category Total (100%)*
Owner Factors (25%)	• Footprint (30%) • Safe work environment (25%) • Maintenance complexity (25%) • Operations complexity (20%) *Category Total (100%)*
Island Factors (25%)	• Mainland delivery dependence (25%) • Mainland servicing dependence (25%) • Power dependence (25%) • Chemical dependence (25%) *Category Total (100%)*
Overall Total 100%	

7.5 Non-Economic Evaluation Results

A score of 1 through 5 was given for each criterion, with 5 being the most favorable, and 1 representing the least desired option. The complete non-economic evaluation is included as Appendix C. The non-economic evaluation results are summarized in Table 7-9.

Table 7-9. Non-Economic Weighted Scores		
Alternative	**Score**	**Rank**
Project Alternative 1: Activated Sludge Package Plants	3.80	2
Project Alternative 2: MBR Package Plants	3.96	1
Project Alternative 3: Imhoff Tank/Recirculating Gravel Filter	3.53	3

As shown in Table 7-9, Project Alternative 2: MBR Package Plants, received the highest non-economic score. The higher score reflects the County's desire to standardize on MBR technology to provide the highest level of treatment at WWTP facilities and to facilitate future water recycling programs.

7.6 Conclusions and Recommendation

Figure 7-5 combines the economic and non-economic results into a single graph. As previously stated, the economic cost of the three project alternatives can be considered equivalent at this level of analysis. Project Alternative 2: MBR Package Plants, has the highest non-economic score and is recommended for implementation if the County proceeds with a centralized sewer system and WWTP for the community.

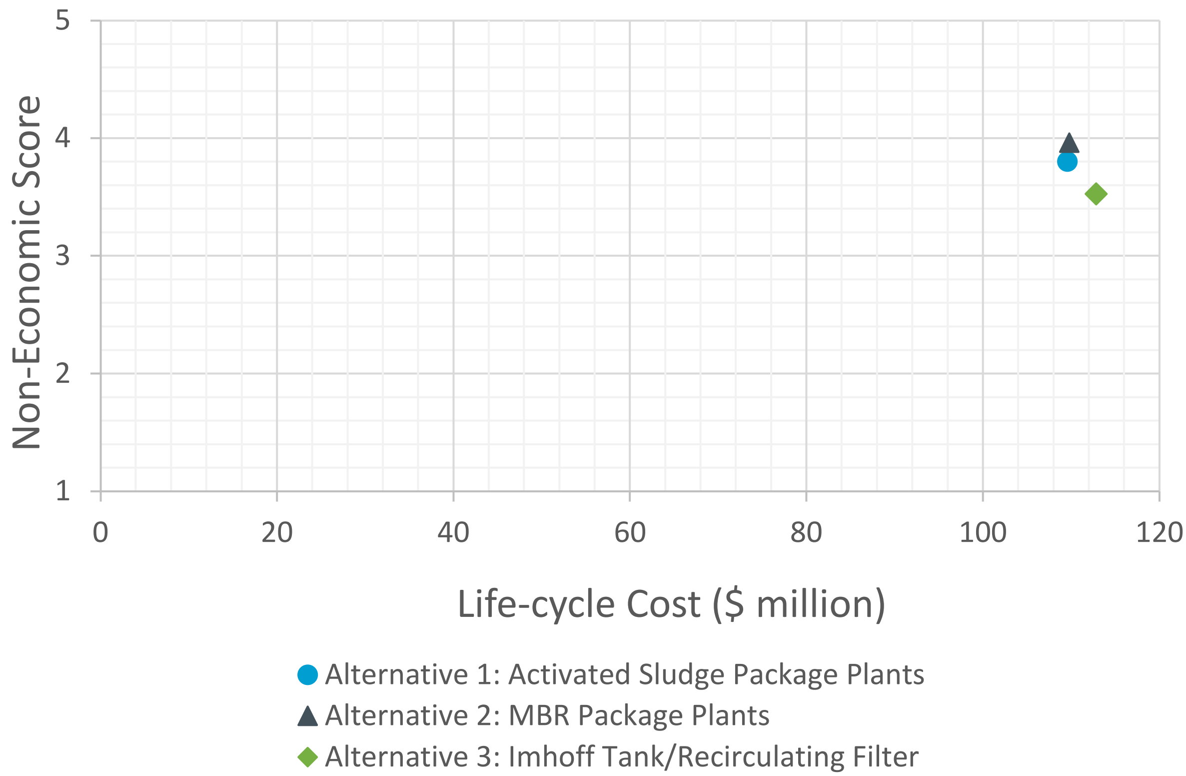


Figure 7-5. Combined Economic and Non-Economic Results

Section 8

Preliminary Design of Improvements

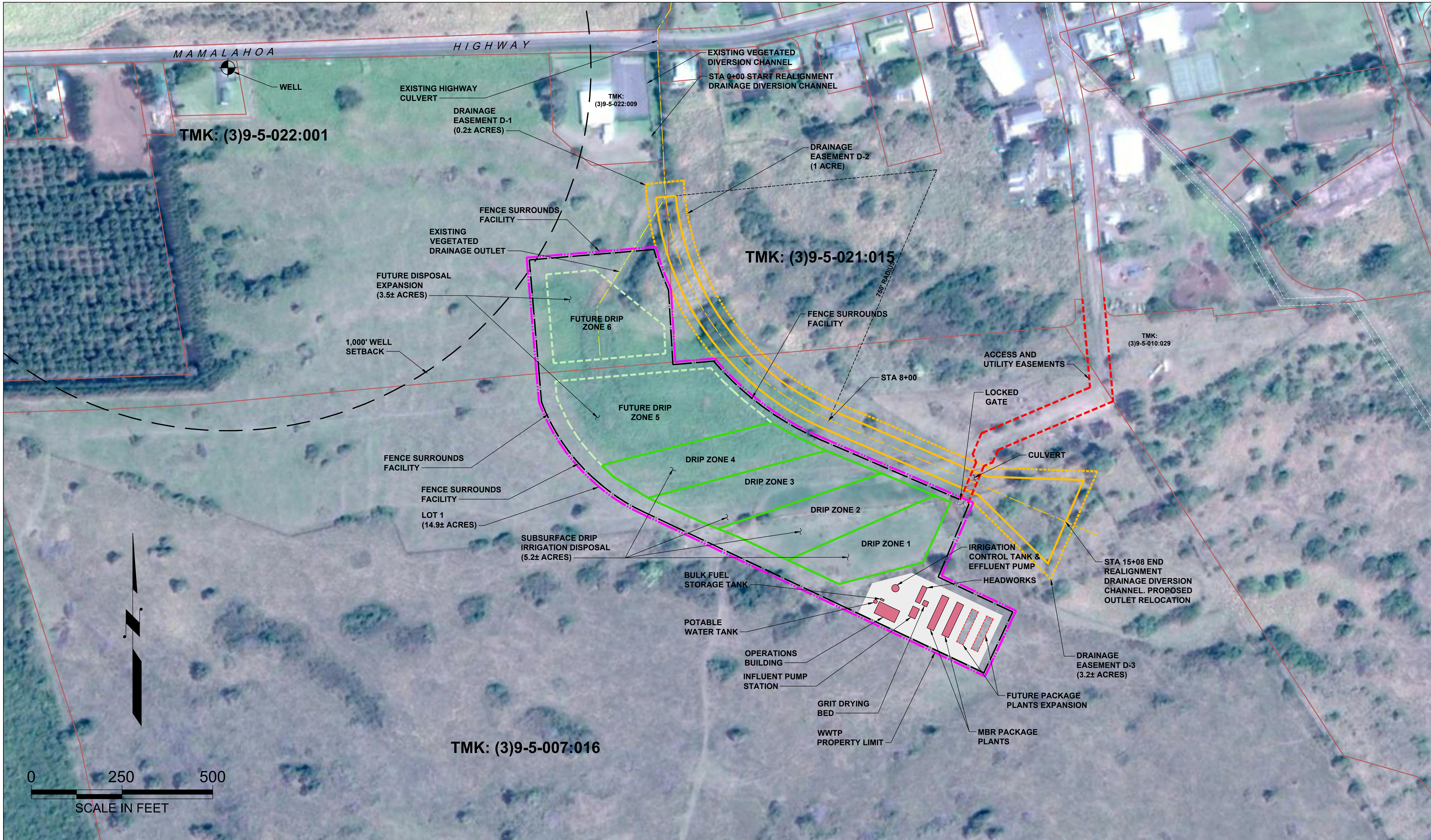
This section presents an overview of preliminary design improvements proposed for the new Naalehu wastewater treatment plant (WWTP) project.

8.1 Site Plan

Figure 8-1 provides a preliminary site plan of the WWTP project.

8.2 Process Schematic

Figure 8-2 provides a preliminary process schematic of the WWTP.



Path: C:\bcpw\d3321813
File Name: 153740-FIG-8-1-WWTP-PreliminarySitePlan
Plot Date: July 17, 2023 3:32 PM
Cadd User: Richard Sellona

Brown AND Caldwell

SCALE: 1" = 200'

153740

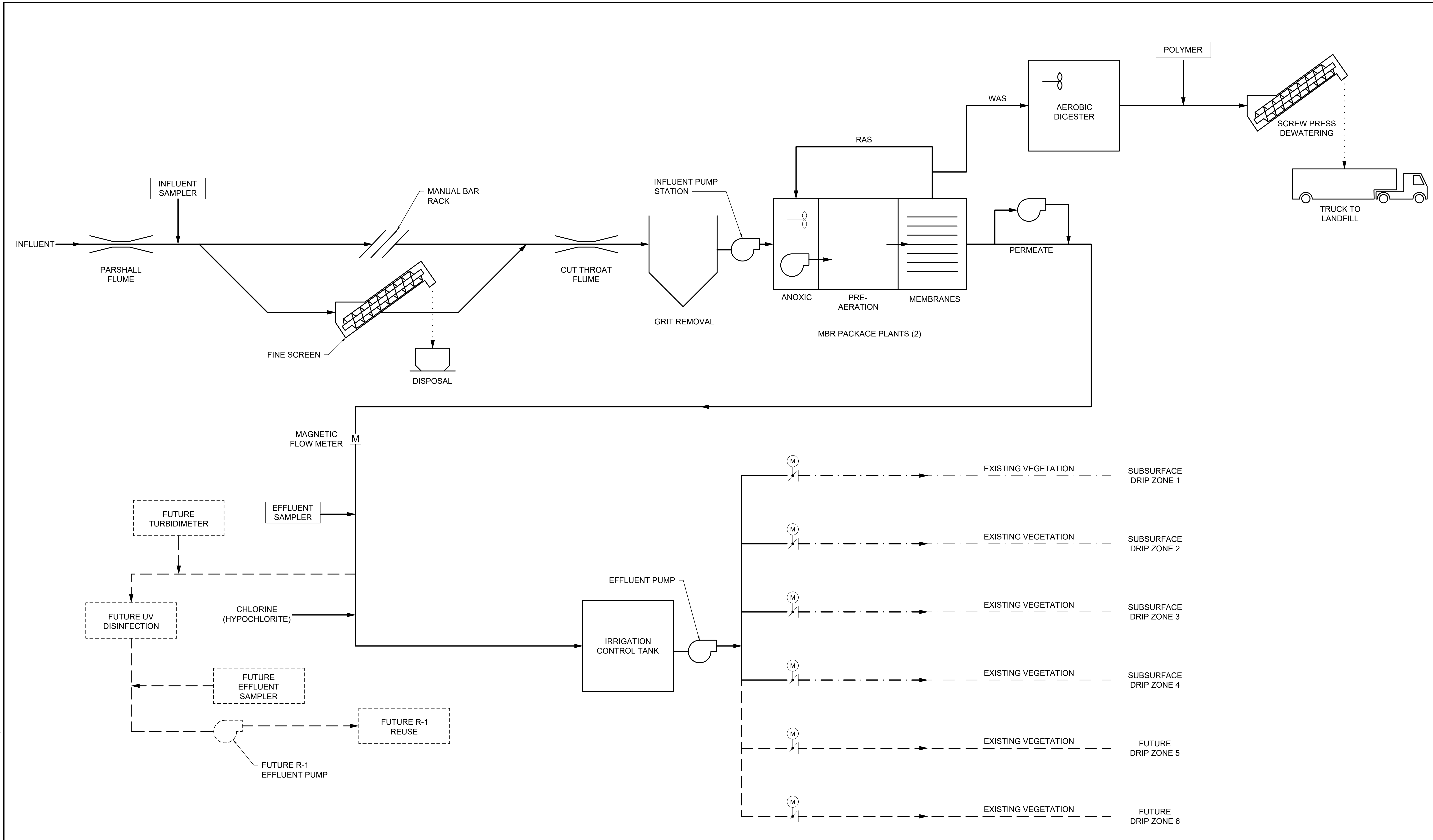
DATE: July 17, 2023

NAALEHU WASTEWATER TREATMENT PLANT

PRELIMINARY SITE PLAN

FIGURE

8-1



Brown AND Caldwell

SCALE: NONE

JOB NO: 153740

NAALEHU WASTEWATER TREATMENT PLANT

PROCESS SCHEMATIC

FIGURE

8-2

Path: C:\bcpw\d3321813
File Name: 153740-FIG_8-2 Plot Date: July 26, 2023 9:17 AM Cadd User: Richard Sellona

8.3 Preliminary Design Criteria

Table 8-1 lists the preliminary design criteria for the proposed WWTP.

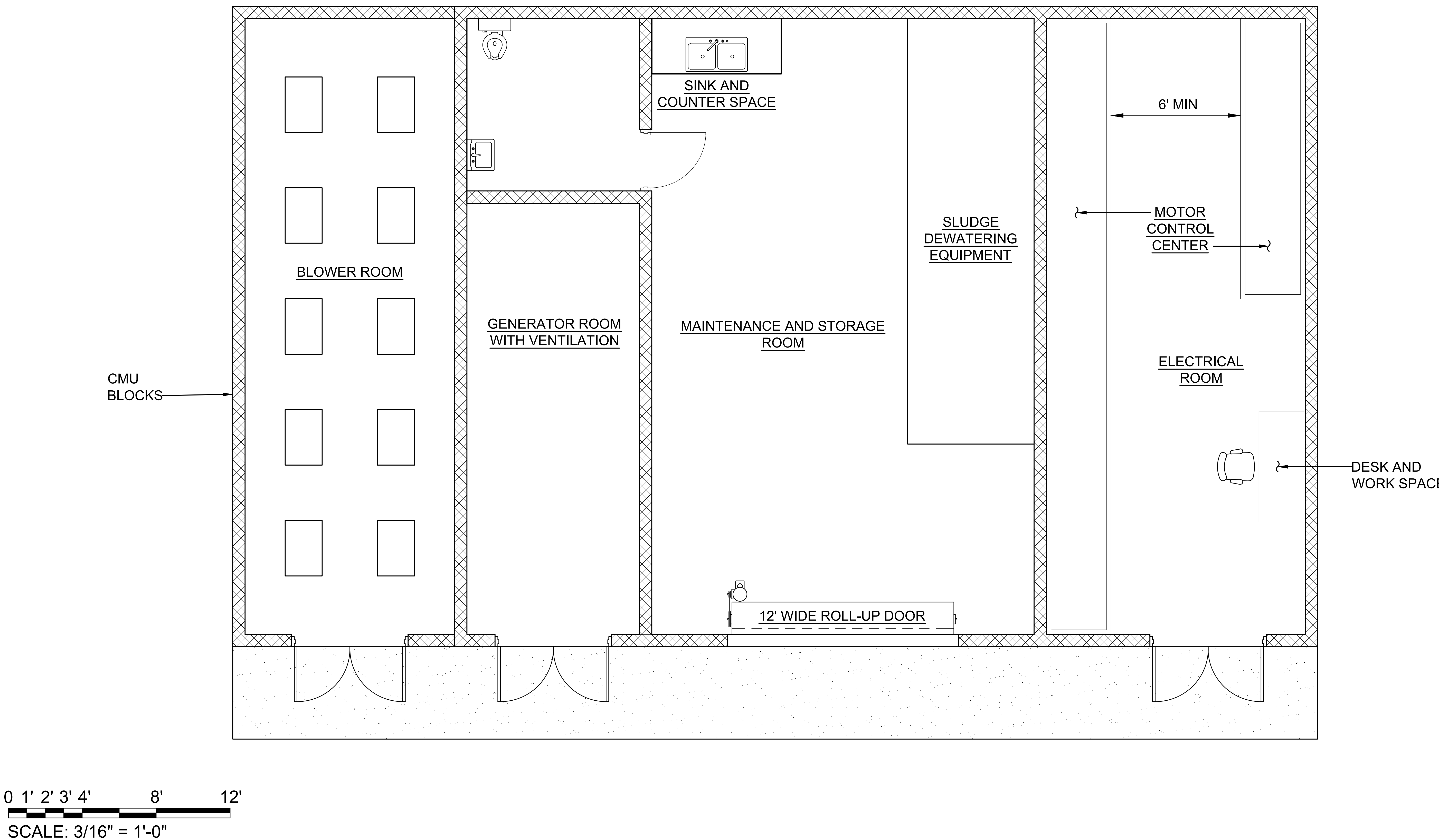
Table 8-1. Preliminary Design Criteria	
Description	**Value**
Influent Flow	
Average dry weather, gallons per day (gpd)	125,000
Peak day wet weather, gpd	322,000
Peak hour wet weather, gpd (gallons per minute [gpm])	457,000 (317)
Influent Characteristics	
Biochemical oxygen demand (BOD_5), mg/L	300
Total suspended solids (TSS), mg/L	300
Total Nitrogen (TN), mg/L	40
Odor Control–Granular Activated Carbon	
Airflow rate, air changes per hour	6
H_2S inlet concentration, parts per million	1-10
H_2S removal efficiency, percent	99
Media, type	High-capacity carbon
Mechanical Screens	
Number of units, each	1
Type	In-channel cylindrical
Screen opening size, inches/(millimeters)	0.125 /(3)
Maximum flow rate capacity, gpm	Greater than 480
Screening washing	Integral
Screening compaction	Integral
Bypass Screen	
Type	Manually-cleaned bar rack
Bar spacing, inches	1
Rake	Fabricated to Interlock with bars
Screenings Receptacle	
Type	55-gallon drum or bags
Screenings volume per million gallons (Mgal) treated, ft^3/Mgal	5
Estimated screenings quantity, ft^3/day	0.63
Disposal frequency, per week	1
Influent Flow Metering	
Type	Parshall flume
Maximum flow capacity, gpm	Greater than 480
Minimum straight upstream channel section	20 times the throat width
Influent flow sampling	Refrigerated automatic composite sampler

Table 8-1. Preliminary Design Criteria	
Description	**Value**
Grit Chamber	
Number of units, each	1
Type	Aerated grit trap
Volume, gallons	4,800
Detention time, minutes at Peak Hour Wet Weather Flow	10
Air supply, ft^3/minute	90
Removal	Vactor truck
Estimated average grit quantity, ft^3/day	1.4
MBR Package Plant	
Number of packaged biological treatment trains, each	2
Flow basis for biological design, gpd each	62,500
Anoxic tank working volume (excluding membranes), gallons each	2,000
Aerobic working volume, gallons each	10,000
Design SRT, days	7
Waste sludge removal, gpd each	1,500
Design mixed liquor suspended solids concentration in bioreactor, mg/L	≤ 8,000
Number of duty membrane blowers, per train	1
Number of duty process aeration blowers, per train	1
Aeration system, type	Coarse bubble diffused aeration
Mixed liquor recirculation rate, x Average Dry Weather Flow	4
Membrane cleaning dosing systems, types	Sodium hypochlorite, citric acid, and coagulant
Sludge Management System	
Number of units, each	1
Type	Incline screw press
Screw press capacity, gpm	45
Polymer dose, lbs/dry ton	20
Annual polymer use, lbs	621
Average amount of dewatered sludge, wet tons/day	0.71
Disposal frequency, per week	1
Maintenance Disinfection system	
Type	Chlorine
Form	Calcium hypochlorite tablets
Design chlorine dose, mg/L	8
Irrigation Equalization (Control) Tank	
Number of units, each	1
Type	Glass lined bolted steel
Volume, gallons	20,000
Drip flow, gpm	112
Effluent Flow Metering	
Type	Magnetic
Effluent flow sampler	Refrigerated automatic composite

Table 8-1. Preliminary Design Criteria	
Description	**Value**
Effluent Quality	
BOD_5, mg/L	≤ 5
TSS, mg/L	≤ 5
TN, mg/L	≤ 10
Turbidity, nephelometric turbidity units	≤ 0.2
Effluent Management System	
Type	Subsurface drip irrigation disposal
Number of units. each	2 (1 active, 1 redundant)
Distribution system	Non-clog subsurface drip emitters
Design percolation rate, inches/day	2.50
Design application rate, % of percolation rate	4
Total land application area, acres	5.2
Number of irrigation zones	4
Area per zone, sq. ft.	56,630
Flow rate per zone, gallons per minute	112
Drip rate per emitter, gallons per hour	1
Total number of emitters, each	26,833
Total length of drip line, ft	53,667
Depth of drip line, inches	6–9
Drip line spacing, ft	4.2
Emitter spacing, ft	2
Vegetation	Existing vegetation
Irrigation system monitoring	Flow meter(s) and pressure indicators
Stormwater site management	10-year, 1-hour storm

8.4 Preliminary Floor Plan

Figure 8-3 provides a preliminary floor plan for the proposed Operations Building.



Path: C:\bcpw\d3083617 File Name: 150440-FIG-OpsBldg Plot Date: July 13, 2023 12:53 PM Cadd User: Richard Sellona

Brown AND Caldwell	SCALE: 3/16" = 1'-0" JOB NO: 153740	NAALEHU WASTEWATER TREATMENT PLANT OPERATIONS BUILDING PRELIMINARY FLOOR PLAN	FIGURE 8-3

Section 9

Implementation Plan

This section describes the proposed implementation plan for the Naalehu wastewater treatment plant (WWTP).

9.1 Implementation Approach

The WWTP and collection system projects could be implemented using either a traditional design/bid/build (DBB) approach or a design/build (DB) approach, as discussed below.

9.1.1 Design Bid Build Approach

DBB is the traditional approach used by the County for implementing public works projects. The design is prepared by a consultant, and then bids are solicited from construction contractors. The County awards the contract to the lowest responsible bidder.

Advantages of the DBB approach are that the County retains maximum control over the design process, ensuring the project will meet its needs.

9.1.2 Design Build Approach

DB is an alternative delivery approach whereby the County would contract with an entity to both design and construct a facility that meets established project specifications. The combined WWTP and collection system projects are large enough monetarily for the County to consider a DB approach. The County would need to use a procurement process based on qualifications and cost to select the DB entity. The typical DB procurement process takes 9–12 months to complete. The DB bidders will need the County to complete the following prior to the DB procurement process:

- Complete geotechnical report
- Environmental assessment
- Land use entitlements
- WWTP land purchase

Advantages of DB implementation are:

- Possibility of reduced overall costs
- Design and construction can occur simultaneously, potentially reducing implementation time
- DB entity assumes the performance liability for the project, as defined in the project specifications

Disadvantages of the DB approach are that the County has limited experience with it, and the County would not have as much control over how the project is designed.

9.2 Implementation Schedules

Planning level implementation schedules were developed for both approaches.

9.2.1 Recent Change in State of Hawaii Land Use Commission Policy

The State of Hawaii Land Use Commission (LUC) recently changed its policy regarding the use of Special Permits for non-conforming uses. The proposed WWTP site is located in the Agricultural District as defined by the LUC. A WWTP is not an allowable land use in the Agricultural District. In the past the LUC has allowed Special Permits to be used for non-conforming uses. However, in response to litigation the LUC has recently changed its policy and now recommends that project proponents for permanent facilities (like a WWTP) pursue a District Boundary Amendment (DBA) from the LUC. The LUCs rationale is that permanent entitlement (i.e., a DBA) is more appropriate for a permanent facility like a WWTP, rather than a temporary entitlement like a Special Permit. Since the WWTP parcel is less than 15 acres the DBA can be processed by the County of Hawaii. However, the action will likely take longer to implement than a Special Permit.

9.2.2 Equipment Procurement Time to Impact Construction Schedule

The COVID-19 pandemic continues to impact the construction industry due to increased time to deliver equipment and other materials. Most significantly, the time for the MBR package plant supplier to manufacture their equipment was quoted at 55 weeks instead of a typical pre-pandemic time of approximately 26 weeks. Similar delays are being experienced on other construction projects in Hawaii, and it is reasonable to assume that other equipment suppliers will quote extended supply times. As a result, we now suggest that a construction schedule of 2 years is a reasonable expectation.

9.2.3 Implementation Schedules

Figure 9-1 presents implementation schedules for both approaches. At this time, both approaches may not enable the County to meet the Administrative Order on Consent (AOC) milestone schedule to close the LCCs. The DB approach offers greater potential to meet the milestone, because equipment procurement can possibly occur in parallel with design within a DB contract.

9.3 Recommendation

Given the Revised AOC deadline to close the LCCs, and the equipment procurement time impact to the construction schedule, a DB approach may offer a better opportunity to meet the deadline, because a DB entity could initiate equipment procurement while design activities progress. If the U.S. Environmental Protection Agency was able to grant a time extension, then either a DBB or DB approach can be taken. A traditional DBB approach would give the County greater control over the project outcome, and is the County's standard method for implementing projects.

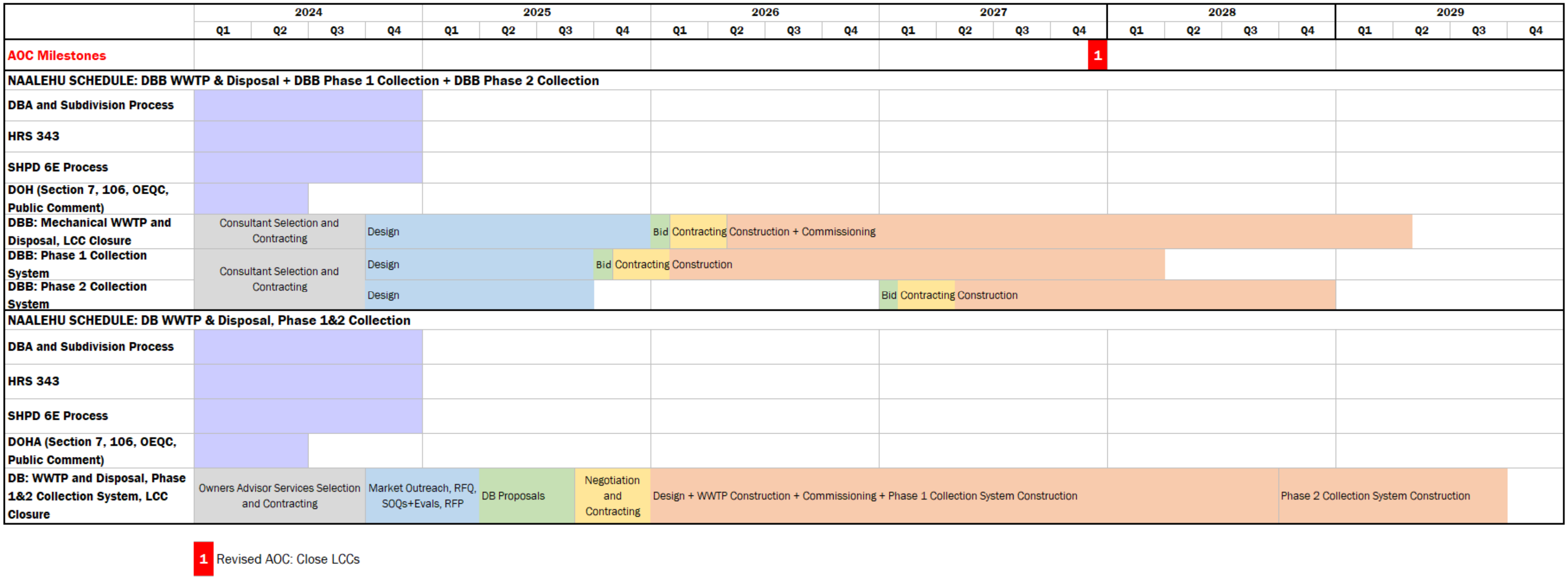


Figure 9-1. Implementation Schedules

Section 10

References

Brown and Caldwell (November 2019). Feasibility Study: Naalehu WWTP R-1 Stock Water, Prepared for County of Hawaii Department of Environmental Management Wastewater Division.

Crites, Ron, Craig Lekven, Steve Wert, George Tchobanoglous (1997). *A Decentralized Wastewater System for a Small Residential Development in California*. The Small Flows Journal. National Small Flows Clearinghouse. Volume 3, Issue 1, Winter 1997.

Crites, R. W. and G. Tchobanoglous (1998) Small and Decentralized Wastewater Management Systems, McGraw-Hill, New York.

Department of Wastewater Management, City and County of Honolulu, State of Hawaii. Design Standards of the Department of Wastewater Management, Volume 1 and 2. July 2017.

Great Lakes–Upper Mississippi River Board of State and Provincial Public Health and Environmental Managers. Recommended Standards for Wastewater Facilities. 1997.

Hawaii Administrative Rules, Title 11, Department of Health Administrative Rules.

Masa Fujioka & Associates. Letter Report, Probing for Large Cavities (Lava Tubes), Naalehu and Pahala Large Capacity Cesspool Sewerage System. January 9, 2007.

M&E Pacific, Inc. Kau Sewer System Evaluation, Kau, Island of Hawaii, Hawaii. December 2004.

SSFM International, Inc. Final Preliminary Engineering Report for Naalehu and Pahala Large Capacity Cesspool Conversion Projects, July 2007.

USEPA. “Process Design Manual, Land Treatment of Municipal Wastewater Effluents”. EPA/625/R-06/016. September 2006.

Water Environment Federation. *Wastewater Disinfection, Manual of Practice FD-10*. Water Environment Federation, 1996.

Appendix A: Cost Estimates

Naalehu WWTP
Revised Preliminary Engineering Report
Alternative Solutions Cost Summary

Alternative #1 - RAS Package Plants / Subsurface Drip

Collection System TOTAL		**$40,165,000**
Drainage Channel TOTAL		**$19,865,000**
Wastewater Treatment TOTAL		**$20,705,000**
Effluent Disposal TOTAL		**$4,016,000**
ALTERNATIVE #1 CAPITAL COST TOTAL		***$84,751,000***
ANNUAL O&M COSTS		
Collection system	$74,000	
Labor	$300,000	
Electricity	$270,000	
Chemicals	$27,000	
Maintenance materials	$117,000	
Solids disposal	$67,000	
Total Annual Operating Costs	***$855,000***	
EQUIPMENT REPLACEMENT COST (20 YEAR)	***$6,238,000***	

Alternative #2 - MBR Package Plants / Reuse / Subsurface Drip

Collection System TOTAL		**$40,165,000**
Drainage Channel TOTAL		**$19,865,000**
Wastewater Treatment TOTAL		**$20,283,000**
Effluent Disposal TOTAL		**$4,016,000**
ALTERNATIVE #2 CAPITAL COST TOTAL		***$84,329,000***
ANNUAL O&M COSTS		
Collection system	$74,000	
Labor	$300,000	
Electricity	$300,000	
Chemicals	$32,000	
Maintenance materials	$113,000	
Solids disposal	$67,000	
Total Annual Operating Costs	***$886,000***	
MEMBRANE REPLACEMENT COST (15 YEAR)	***$60,000***	
EQUIPMENT REPLACEMENT COST (20 YEAR)	***$6,005,000***	

Alternative #3 - Imhoff Tank / RGF / Subsurface Drip

Collection System TOTAL		**$40,165,000**
Drainage Channel TOTAL		**$19,865,000**
Wastewater Treatment TOTAL		**$29,556,000**
Effluent Disposal TOTAL		**$4,016,000**
ALTERNATIVE #3 CAPITAL COST TOTAL		***$93,602,000***
ANNUAL O&M COSTS		
Collection system	$74,000	
Labor	$300,000	
Electricity	$120,000	
Chemicals	$27,000	
Maintenance materials	$84,000	
Solids disposal	$67,000	
Total Annual Operating Costs	***$672,000***	
EQUIPMENT REPLACEMENT COST (20 YEAR)	***$4,574,000***	

Naalehu WWTP
Revised Preliminary Engineering Report
Unit Cost Estimates

Electrical & Instrumentation	25.0%	
Engineering, Admin, & Legal	25.0%	
Contingency	20.0%	
ENR CCI	13,345.00	June, 2023

Naalehu WWTP Capital Unit Costs	**Units**	**Unit Cost**
Environmental protection, BMPs	ac	$11,000
Site clearing	ac	$20,000
Site roads, guard rails, pavement	ac	$92,000
Perimeter fence	LF	$150
WWTP site grading	ac	$30,000
WWTP site drainage improvements	ac	$18,000
Drainage channel improvements	LS	$13,700,000
Plant water catchment/collection system	LS	$75,000
Process yard piping	LS	$250,000
Headworks (includes site/civil, structures, equipment & piping)	LS	$1,024,000
Influent pump station	LS	$1,670,000
Chlorine disinfection	LS	$150,000
RAS package plants	LS	$3,812,000
MBR package plants	LS	$3,579,000
Irrigation equalization tank	gal	$10
Effluent pump station	LS	$1,250,000
Subsurface drip irrigation line	LF	$10
Irrigation piping & valves	LF	$250
Imhoff tank	LS	$1,184,000
Recirculation tank	LS	$1,320,000
Recirculating gravel filter	LS	$6,591,000
Plant drainage system	ac	$40,000
Main generator (including process piping)	LS	$500,000
Maintenance/operations/electrical building	SF	$1,000
Phase 1 existing gravity collection system w/ new WWPS & force main	LS	$17,650,000
Reuse existing Brewer area gravity collection system	LS	$3,700,000
Phase 2 new gravity collection system w/ new WWPS & force main	LS	$27,700,000
Sludge dewatering system	LS	$860,000

Naalehu WWTP
Revised Preliminary Engineering Report
Unit Cost Estimates

Naalehu WWTP Operation, Maintenance, & Replacement Unit Costs	Unit Cost	Units
Equipment replacement cost	25%	of process capital cost
Package plant replacement cost	100%	of package plant captial cost
Maintenance materials	2%	of equipment capital cost
Gravity mainline collection system maintenance cost	$16,000.00	per mi
Force main collection system maintenance cost	$10,000.00	per mi
Membrane replacement cost	$1,950.00	per module + S&H & install
Solids disposal dumpster rental fee	$500.00	per week
Sanitary landfill tipping fee	$116.00	per wet ton
Hypochlorite tablet cost	$8.00	per lb
Dewatering polymer cost	$3.00	per lb
Diesel price	$6.06	per gallon

Naalehu WWTP
Revised Preliminary Engineering Report
Lump Sum Cost Estimates

Imhoff Tank	***Units***	***Unit Cost***	***Number of Units***	***Cost***
Excavation	CY	$300	450	$135,000
Bedding & backfill	CY	$100	35	$3,500
Concrete	CY	$1,500	160	$240,000
Piping & valves	LS	$50,000	1	$50,000
Cover plates	SF	$200	75	$15,000
Odor control	LS	$500,000	1	$500,000
Epoxy Coating	SF	$80	3,000	$240,000
			TOTAL	**$1,184,000**

Recirculating Gravel Filter	***Units***	***Unit Cost***	***Number of Units***	***Cost***
RGF bed excavation	CY	$300	11,800	$3,540,000
Bed liner	SF	$8	75,100	$600,800
20-inch PVC manifold pipe	LF	$200	400	$80,000
3-inch PVC lateral pipe	LF	$30	13,900	$417,000
6-inch PVC drainage pipe	LF	$60	3,700	$222,000
8-inch PVC recirculation pipe	LF	$80	200	$16,000
Gravel media	CY	$150	10,500	$1,575,000
6 in sand media under liner	CY	$100	1,400	$140,000
			TOTAL	**$6,591,000**

Recirculation Tank	***Units***	***Unit Cost***	***Number of Units***	***Cost***
Recirculation tank excavation	CY	$300	1,480	$444,000
Bedding & backfill	CY	$100	200	$20,000
Concrete	CY	$1,500	290	$435,000
Handrail	LF	$100	210	$21,000
Pumps & valves	ea	$100,000	4	$400,000
			TOTAL	**$1,320,000**

Naalehu WWTP
Revised Preliminary Engineering Report
Lump Sum Cost Estimates

Sludge Dewatering System	***Units***	***Unit Cost***	***Number of Units***	***Cost***
300 HP diesel dump truck	LS	$300,000	1	$300,000
Dewatering screw press	LS	$300,000	1	$300,000
Incline screw conveyor	LS	$80,000	1	$80,000
Polymer system	LS	$80,000	1	$80,000
Sludge feed pump & piping	LS	$100,000	1	$100,000
			TOTAL	**$860,000**

Reuse Existing Gravity Collection System	***Units***	***Unit Cost***	***Number of Units***	***Cost***
Inspection & cleaning	LS	$1,500,000	1	$1,500,000
Repair defects	LS	$2,000,000	1	$2,000,000
Archaeological monitoring	LS	$50,000	1	$50,000
BMPs	LS	$50,000	1	$50,000
Traffic control measures	LS	$50,000	1	$50,000
Pre- & post-construction inspections & documentation	LS	$50,000	1	$50,000
			TOTAL	**$3,700,000**

New Drainage Channel	***Units***	***Unit Cost***	***Number of Units***	***Cost***
Earthwork	CY	$250	49500	$12,375,000
Concrete culverts	LS	$1,250,000	1	$1,250,000
Archaeological monitoring	LS	$25,000	1	$25,000
Clear and grub	LS	$25,000	1	$25,000
BMPs	LS	$25,000	1	$25,000
			TOTAL	**$13,700,000**

Alternative #1 - RAS Package Plants / Subsurface Drip

Capital Cost Estimate

Naalehu WWTP Capital Cost Item Description	Units	General Unit Cost	Number of Units	COST
Collection System				
Phase 2 new gravity collection system w/ new WWPS & force main	LS	$27,700,000	1	$27,700,000
			Subtotal	$27,700,000
			Contingency @ 20%	$5,540,000
			Engineering, Admin, & Legal @ 25%	$6,925,000
			Collection System Total	**$40,165,000**
Drainage Channel				
Drainage channel improvements	LS	$13,700,000	1	$13,700,000
			Subtotal	$13,700,000
			Contingency @ 20%	$2,740,000
			Engineering, Admin, & Legal @ 25%	$3,425,000
			Drainage Channel Total	**$19,865,000**
Wastewater Treatment				
Environmental protection, BMPs	ac	$11,000	1.5	$16,500
Site clearing	ac	$20,000	1.5	$30,000
Site roads, guard rails, pavement	ac	$92,000	1.5	$138,000
Perimeter fence	LF	$150	4,100	$615,000
WWTP site grading	ac	$30,000	1.5	$45,000
WWTP site drainage improvements	ac	$18,000	1.5	$27,000
Plant water catchment/collection system	LS	$75,000	1	$75,000
Process yard piping	LS	$250,000	1	$250,000
Headworks (includes site/civil, structures, equipment & piping)	LS	$1,024,000	1	$1,024,000
Influent pump station	LS	$1,670,000	1	$1,670,000
Chlorine disinfection	LS	$150,000	1	$150,000
RAS package plants	LS	$3,812,000	1	$3,812,000
Plant drainage system	ac	$40,000	1.5	$60,000
Main generator (including process piping)	LS	$500,000	1	$500,000
Maintenance/operations/electrical building	SF	$1,000	2,150	$2,150,000
Sludge dewatering system	LS	$860,000	1	$860,000
			Subtotal	$11,422,500
			Electrical & Instrumentation @ 25%	$2,856,000
			Subtotal	$14,279,000
			Contingency @ 20%	$2,856,000
			Engineering, Admin, & Legal @ 25%	$3,570,000
			Wastewater Treatment Total	**$20,705,000**
Effluent Disposal				
Irrigation equalization tank	gal	$10	20,000	$200,000
Effluent pump station	LS	$1,250,000	1	$1,250,000
Subsurface drip irrigation line	LF	$10	54,000	$540,000
Irrigation piping & valves	LF	$250	900	$225,000
			Subtotal	$2,215,000
			Electrical & Instrumentation @ 25%	$554,000
			Subtotal	$2,769,000
			Contingency @ 20%	$554,000
			Engineering, Admin, & Legal @ 25%	$693,000
			Effluent Disposal Total	**$4,016,000**
			Alternative #1 TOTAL	**$84,751,000**

Alternative #2 - MBR Package Plants / Reuse / Subsurface Drip

Capital Cost Estimate

Naalehu WWTP Capital Cost Item Description	Units	General Unit Cost	Number of Units	COST
Collection System				
Phase 2 new gravity collection system w/ new WWPS & force main	LS	$27,700,000	1	$27,700,000
			Subtotal	$27,700,000
			Contingency @ 20%	$5,540,000
			Engineering, Admin, & Legal @ 25%	$6,925,000
			Collection System Total	**$40,165,000**
Drainage Channel				
Drainage channel improvements	LS	$13,700,000	1	$13,700,000
			Subtotal	$13,700,000
			Contingency @ 20%	$2,740,000
			Engineering, Admin, & Legal @ 25%	$3,425,000
			Drainage Channel Total	**$19,865,000**
Wastewater Treatment				
Environmental protection, BMPs	ac	$11,000	1.5	$16,500
Site clearing	ac	$20,000	1.5	$30,000
Site roads, guard rails, pavement	ac	$92,000	1.5	$138,000
Perimeter fence	LF	$150	4,100	$615,000
WWTP site grading	ac	$30,000	1.5	$45,000
WWTP site drainage improvements	ac	$18,000	1.5	$27,000
Plant water catchment/collection system	LS	$75,000	1	$75,000
Process yard piping	LS	$250,000	1	$250,000
Headworks (includes site/civil, structures, equipment & piping)	LS	$1,024,000	1	$1,024,000
Influent pump station	LS	$1,670,000	1	$1,670,000
Chlorine disinfection	LS	$150,000	1	$150,000
MBR package plants	LS	$3,579,000	1	$3,579,000
Plant drainage system	ac	$40,000	1.5	$60,000
Main generator (including process piping)	LS	$500,000	1	$500,000
Maintenance/operations/electrical building	SF	$1,000	2,150	$2,150,000
Sludge dewatering system	LS	$860,000	1	$860,000
			Subtotal	$11,189,500
			Electrical & Instrumentation @ 25%	$2,798,000
			Subtotal	$13,988,000
			Contingency @ 20%	$2,798,000
			Engineering, Admin, & Legal @ 25%	$3,497,000
			Wastewater Treatment Total	**$20,283,000**
Effluent Disposal				
Irrigation equalization tank	gal	$10	20,000	$200,000
Effluent pump station	LS	$1,250,000	1	$1,250,000
Subsurface drip irrigation line	LF	$10	54,000	$540,000
Irrigation piping & valves	LF	$250	900	$225,000
			Subtotal	$2,215,000
			Electrical & Instrumentation @ 25%	$554,000
			Subtotal	$2,769,000
			Contingency @ 20%	$554,000
			Engineering, Admin, & Legal @ 25%	$693,000
			Effluent Disposal Total	**$4,016,000**
			Alternative #2 TOTAL	**$84,329,000**

Alternative #3 - Imhoff Tank / RGF / Subsurface Drip

Capital Cost Estimate

Naalehu WWTP Capital Cost Item Description	Units	General Unit Cost	Number of Units	COST
Collection System				
Phase 2 new gravity collection system w/ new WWPS & force main	LS	$27,700,000	1	$27,700,000
			Subtotal	$27,700,000
			Contingency @ 20%	$5,540,000
			Engineering, Admin, & Legal @ 25%	$6,925,000
			Collection System Total	**$40,165,000**
Drainage Channel				
Drainage channel improvements	LS	$13,700,000	1	$13,700,000
			Subtotal	$13,700,000
			Contingency @ 20%	$2,740,000
			Engineering, Admin, & Legal @ 25%	$3,425,000
			Drainage Channel Total	**$19,865,000**
Wastewater Treatment				
Environmental protection, BMPs	ac	$11,000	2	$22,000
Site clearing	ac	$20,000	2	$40,000
Site roads, guard rails, pavement	ac	$92,000	2	$184,000
Perimeter fence	LF	$150	4,100	$615,000
WWTP site grading	ac	$30,000	2	$60,000
WWTP site drainage improvements	ac	$18,000	2	$36,000
Plant water catchment/collection system	LS	$75,000	1	$75,000
Process yard piping	LS	$250,000	1	$250,000
Headworks (includes site/civil, structures, equipment & piping)	LS	$1,024,000	1	$1,024,000
Influent pump station	LS	$1,670,000	1	$1,670,000
Imhoff tank	LS	$1,184,000	1	$1,184,000
Recirculation tank	LS	$1,320,000	1	$1,320,000
Recirculating gravel filter	LS	$6,591,000	1	$6,591,000
Chlorine disinfection	LS	$150,000	1	$150,000
Plant drainage system	ac	$40,000	2	$80,000
Main generator (including process piping)	LS	$500,000	1	$500,000
Maintenance/operations/electrical building	SF	$1,000	1,645	$1,645,000
Sludge dewatering system	LS	$860,000	1	$860,000
			Subtotal	$16,306,000
			Electrical & Instrumentation @ 25%	$4,077,000
			Subtotal	$20,383,000
			Contingency @ 20%	$4,077,000
			Engineering, Admin, & Legal @ 25%	$5,096,000
			Wastewater Treatment Total	**$29,556,000**
Effluent Disposal				
Irrigation equalization tank	gal	$10	20,000	$200,000
Effluent pump station	LS	$1,250,000	1	$1,250,000
Subsurface drip irrigation line	LF	$10	54,000	$540,000
Irrigation piping & valves	LF	$250	900	$225,000
			Subtotal	$2,215,000
			Electrical & Instrumentation @ 25%	$554,000
			Subtotal	$2,769,000
			Contingency @ 20%	$554,000
			Engineering, Admin, & Legal @ 25%	$693,000
			Effluent Disposal Total	**$4,016,000**
			Alternative #3 TOTAL	**$93,602,000**

Electricity cost

	$0.45	/kWh

Flow

ADWF:	0.125	mgd
	0.193404	cfs

Labor (common across all alternatives)

COH WWTP operator annual salary	$100,000	*including fringe benefits*
Number of employees/operators	3	*2 shifts: Wed - Sat / Mon - Fri + 1 supervisor*
Annual labor cost:	**$300,000**	

Electricity

Load	Duty Unit Count	Motor Size (hp)	Use Factor	Equivalent Continuous Load (hp)	Annual Power (kWh)	Alt 1 RAS PP (kWh)	Alt 2 MBR PP (kWh)	Alt 3 RGF (kWh)
Headworks								
Screens	1	2	20%	0.4	2,613	2,613	2,613	2,613
Grit blower	2	5	100%	10	65,323	65,323	65,323	65,323
Process tanks								
Anoxic zone mixers	2	5	100%	10	65,323	65,323	65,323	N/A
Aeration blower (main)	1	27	100%	27	176,373	176,373	176,373	N/A
Aeration blower (flow equalization)	1	13	100%	13	84,920	84,920	84,920	N/A
Imhoff tank odor control	1	2	100%	2	13,065	N/A	N/A	13,065
Recirculation tank pump	1	5	100%	5	32,662	N/A	N/A	32,662
Influent pump	1	10	30%	3	19,597	19,597	19,597	19,597
Effluent pump	1	15	40%	6	39,194	39,194	39,194	39,194
Secondary clarifier								
Clarifier mechanisms	2	1	100%	2	13,065	13,065	N/A	N/A
Membranes								
Membrane blower	2	5	30%	3	19,597	N/A	19,597	N/A
Permeate pumps	2	5	100%	10	65,323	N/A	65,323	N/A
Aerobic digestion								
Digester blowers	2	5	90%	9	58,791	58,791	58,791	58,791
Sludge dewatering								
Screw press feed pump	1	5	30%	1.5	9,798	9,798	9,798	9,798
Screw press	1	2	30%	0.6	3,919	3,919	3,919	3,919
Cake conveyor	1	2	30%	0.6	3,919	3,919	3,919	3,919
Miscellaneous								
Drainage return pumps	1	5	10%	0.5	3,266	3,266	3,266	3,266
Plant water pumps	1	5	100%	5	32,662	32,662	32,662	N/A
Fans	2	1	100%	2	13,065	13,065	13,065	13,065
Annual electricity consumption kWh:						591,829	663,685	265,213
Annual electricity cost:						**$270,000**	**$300,000**	**$120,000**

Chemicals

Hypochlorite Tablets

Daily chlorine demand @ ADWF	8.3	lbs/d	*assuming 8 mg/L dose, 15 min contact time @ PHWWF*
Annual hypochlorite demand @ ADWF	3,030	lbs/yr	
Hypochlorite tablet unit cost	$8	per lb	
Total annual hypochlorite tablet cost:	**$24,300**		*common across all alternatives*

Dewatering polymer

Daily dewatering polymer use	1.7	lbs/d	*assuming 20 lbs/dry ton dose*
Annual dewatering polymer use	621.0	lbs/yr	
Dewatering polymer unit cost	$3	per lb	
Total annual dewatering polymer cost:	**$1,900**		*common across all alternatives*

MBR cleaning chemicals

Sodium hypochlorite & citric acid cost:	**$5,000**	per yr	*Alternative #2 only*

Membrane replacement (Alt #2)

Membrane cost per module	$1,950	*material costs only*
Estimated additional costs per module	$550	*shipping & handling + installation costs*
Number of membrane modules	24	*modules (12 per unit)*
Membrance replacement cost (15 year):	**$60,000**	

WWTP maintenance materials

	Alt 1 RAS PP	Alt 2 MBR PP	Alt 3 RGF	
Package plant capital cost	$3,812,000	$3,579,000	N/A	
Process equipment capital cost	$8,104,000	$8,104,000	$16,694,000	*not including package plant*
Equipment replacement cost factor	25.0%	25.0%	25.0%	*replace after 20 years*
process equipment replacement cost	$2,026,000	$2,026,000	$4,173,500	*not including package plants*
Total WWTP equipment replacement cost:	**$5,838,000**	**$5,605,000**	**$4,174,000**	*includes 100% package plant replacement*
Maintenance materials cost factor	2.0%	2.0%	2.0%	
Total WWTP annual maintenance materials cost:	**$117,000**	**$113,000**	**$84,000**	

Sludge disposal

Daily dewatering flow:	680	gpd	
Daily dewatered sludge mass	0.71	wet tons/d	
West HI sanitary landfill tipping fee	$116.00	per wet ton	
Onsite disposal roll off dumpster size	10	cu yds	
Dumpster rental fee	$500.00	per week	
Annual dumpster rental fee	$26,000.00		
Disposal frequency	7	days	*Requires weekly disposal (once every 7 days)*
Diesel price (dollar per gallon)	$6.06	per gallon	
Employee labor cost per hour	$48.08	per hour	*based on 100K annual salary*
Distance Naalehu to Landfill (roundtrip)	165	mi	
Dump truck fuel economy	5.0	mpg	

Annual sludge disposal cost (truck to landfill)

Annual fuel cost	$10,400
Annual landfill tipping fee	$30,000
Annual Dumpster rental fee	$26,000
Total annual sludge disposal cost:	**$67,000**

Collection system

Phase 1: Reuse Brewer gravity collection system (Close LCCs) maintenance

Existing Brewer gravity sewer mainline	20,788	ft	*2004 Kau Sewer System Evaluation Report*
	3.94	mi	
Existing gravity mainline multiplier	3		
Gravity mainline maintenance cost	$48,000		
Annual mainline maintenance cost:	**$188,990**		
New gravity sewer mainline (FM to WWTP)	2,500	ft	
New gravity sewer mainline (LCCs to SPS)	1,950	ft	
New gravity sewer mainline TOTAL	4,450	ft	
	0.84	mi	
Gravity mainline maintenance cost	$16,000	per mi	
Annual gravity mainline maintenance cost:	**$13,490**		
Force main length	2,800	ft	*Fukunaga estimate*
	0.53	mi	
Force main maintenance cost	$10,000	per mi	
Annual force main maintenance cost:	**$5,310**		
WWPS process equipment capital cost	$1,670,000		*Assume equal to influent pump station*
WWPS process equipment replacement cost:	**$400,000**		*20 year replacement*
Total annual maintenance materials cost:	**$10,000**		
Total annual WWPS electricity cost	**$20,000**		
Total annual maintenance cost:	**$238,000**		

Phase 2: New Brewer gravity collection system maintenance

New Brewer gravity sewer mainline	10,140	ft	
New gravity sewer mainline (FM to WWTP)	2,500	ft	
New gravity sewer mainline TOTAL	12,640	ft	
	2.39	mi	
Gravity mainline maintenance cost	$16,000	per mi	
Annual gravity mainline maintenance cost:	**$38,310**		
Force main length	2,800	ft	*Fukunaga estimate*
	0.53	mi	
Force main maintenance cost	$10,000	per mi	
Annual force main maintenance cost:	**$5,310**		
WWPS process equipment capital cost	$1,670,000		*Assume equal to influent pump statior*
WWPS process equipment replacement cost:	**$400,000**		*20 year replacement*
Total annual maintenance materials cost:	**$10,000**		
Total annual WWPS electricity cost	**$20,000**		
Total annual maintenance cost:	**$74,000**		

County of Hawaii
Naalehu WWTP Revised PER
Alternatives Net Present Value Analysis

		Risk Premium	Sensitivity Adjustments (%)			Results	
Agency:	County of Hawaii		Benefits	Capital Costs	Other Costs	Capital Cost	30-year NPV
Project/Problem:	Naalehu Collection System						
Alternative 1	Phase 1 - Reuse C Brewer Collection System					$30,032,500	($35,930,046)
Alternative 2	Phase 2 - New C Brewer Collection System					$40,165,000	($42,197,986)
Alternative 3							
Alternative 4							
Alternative 5							

Year of analysis:	2023
Escalation rate:	3.50%
Discount rate:	5.00%

Select one
- (●) All entries in dollars
- ( ) All entries in thousands of dollars

Note: "Status quo" refers to Alternative 1

Make entries in yellow cells only

County of Hawaii
Naalehu WWTP Revised PER
Alternatives Net Present Value Analysis

Agency: County of Hawaii

Project/Problem: Naalehu Disinfection System

		Risk Premium	Sensitivity Adjustments (%) Benefits	Capital Costs	Other Costs	Results Capital Cost	30-year NPV
Alternative 1	Tablet Chlorination Feed System					$150,000	($738,500)
Alternative 2	Ultraviolet Light (UV) System					$1,100,000	($1,335,400)
Alternative 3							
Alternative 4							
Alternative 5							

Year of analysis: 2023
Escalation rate: 3.50%
Discount rate: 5.00%

Select one
- (●) All entries in dollars
- ( ) All entries in thousands of dollars

Note: "Status quo" refers to Alternative 1

Make entries in yellow cells only

County of Hawaii
Naalehu WWTP Revised PER
Alternatives Net Present Value Analysis

		Risk Premium	Sensitivity Adjustments (%)			Results	
			Benefits	Capital Costs	Other Costs	Capital Cost	30-year NPV
Agency:	County of Hawaii						
Project/Problem:	Naalehu WWTP Revised PER						
Alternative 1	RAS Package Plants / Subsurface Drip					$84,751,000	($109,555,755)
Alternative 2	MBR Package Plants / Subsurface Drip					$84,329,000	($109,776,643)
Alternative 3	Imhoff Tank / RGF / Subsurface Drip					$93,602,000	($112,851,053)
Alternative 4							
Alternative 5							

Year of analysis:	2023
Escalation rate:	3.50%
Discount rate:	5.00%

Select one
- (●) All entries in dollars
- ( ) All entries in thousands of dollars

Note: "Status quo" refers to Alternative 1

Make entries in yellow cells only

County of Hawaii DEM
Naalehu Revised AOC PER
Alternatives Net Present Value Analysis

Agency: County of Hawaii DEM

Project/Problem: Naalehu Revised AOC PER

		Risk Premium	Sensitivity Adjustments (%) Benefits	Capital Costs	Other Costs	Results Capital Cost	30-year NPV
Alternative 1	i. Package plant and new collection system					$84,329,000	($109,776,643)
Alternative 2	ii. Package plant connected to existing collection system					$74,197,000	($103,505,201)
Alternative 3	IWS management model 2A					$29,100,000	($38,859,287)
Alternative 4	iii. A maintenance contract model IWS program					$29,100,000	($39,321,375)
Alternative 5	IWS management model 3A					$29,100,000	($39,093,038)
Alternative 6	iv. An operating permit model IWS program					$29,100,000	($41,422,061)
Alternative 7							
Alternative 8							
Alternative 9							
Alternative 10							
Alternative 11							
Alternative 12							

Year of analysis: 2023
Escalation rate: 3.50%
Discount rate: 5.00%

Select one
- ◉ All entries in dollars
- ○ All entries in thousands of dollars

Note: "Status quo" refers to Alternative 1

Make entries in yellow cells only

Appendix B: Letter to Support Department of Hawaii Flow Variance Application

2261 Aupuni Street, Suite 201
Wailuku, Maui, HI 96793

T: 808.244.7005

July 18, 2023

Mr. Mark Grant
County of Hawaii Wastewater Division
108 Railroad Ave
Hilo, HI 96729-4252

Subject: Naalehu Wastewater Treatment Plant – Capacity Phasing

Dear Mr. Grant,

The County of Hawaii (County) is investigating options to reduce the size and impact of the Naalehu Wastewater Treatment Plant (WWTP). While previous planning and design efforts focused on an aerated lagoon secondary WWTP, the County is now evaluating a mechanical secondary treatment process to strive to meet the EPA Administrative Order on Consent (AOC) deadlines.

This letter provides information regarding a phased capacity innovative approach to the project scope that will further the County's goals. A phased WWTP capacity approach will require Department of Health (DOH) acceptance as it potentially falls outside of normally accepted practices and regulatory guidance.

Phased WWTP capacity implementation is proposed to reduce initial WWTP size while protecting human health and the environment. The phasing approach that the County is proposing requires an explanation of deviations from Hawaii Administrative Rules (HAR) 11-62, including utilization of potable water use data to estimate wastewater flows, peak flow prediction, and phasing implementation.

1.1 Background

HAR 11-62 requires WWTPs to be designed in accordance with County standards, or the City and County of Honolulu (CCH) standards if no applicable County standards exist. The County does not have its own WWTP standards, and therefore relies on the CCH standards.

Project planning and design to date has followed the CCH flow standards that were updated in 2017. Table 1 provides a summary of the flows to the Naalehu WWTP based on the 2017 CCH standards.

Table 1. Naalehu WWTP Flows Based on 2017 CCH Standards		
Description	**Peaking Factor**	**Value**
Average dry weather flow	1.0	225,000 gpd
Peak day dry weather flow	2.0	446,000 gpd
Peak day wet weather flow [a]	2.5	563,000 gpd
Peak hour wet weather flow	3.1	480 gpm (691,000 gpd)

Notes: gpd = gallons per day, gpm = gallons per minute

[a] Peak day wet weather flow is not part of the CCH standards but is an important WWTP design parameter. Peak day wet weather flow estimate was developed using an appropriate peaking factor.

The 2017 CCH flow standards include three elements:

1. Wastewater flow generation estimates based on equivalent population estimates (70 gallons per capita per day (gpcd)). This reflects the amount of wastewater that is expected to enter the sewer from residences and businesses and provide the main source of organic material mass to be removed by the WWTP process.
2. Dry weather infiltration and inflow (I/I) estimates based on equivalent population estimates (35 gpcd).
3. Wet weather I/I estimates based on service area acreage (3,000 gallons per acre per day (gpad)).

1.2 County Experience with the CCH Flow Standards

The CCH standards were established for a major metropolitan area that includes vast areas of residential, commercial, and industrial development, with significant proportions of service areas near sea level elevations. Wastewater generation rates are generally lower in rural areas than in urban areas. Typical flow rates in the United States range between 50 gpcd in rural areas and 120 gpcd in typical urban areas (Tchobanoglous, George, F. L. Burton, and H.D. Stensel, Wastewater Engineering: Treatment and Reuse / Metcalf & Eddy, Inc., 4th edition, 2003). The County's experience with the CCH flow standards on other projects (e.g., Honokaa WWTP) has illustrated that the standards are very conservative for small rural communities located at higher elevations on Hawaii Island. The observed monthly average dry weather flows at Honokaa are consistently less than 50 percent of the average dry weather flow design capacity of the WWTP. Peak day wet weather flow events have been near the WWTP facility capacity as designed per the 1993 CCH standards that were in effect at the time (1,250 gpad for sewers above the water table). Therefore, the current wastewater standards based on urban Honolulu are likely overly conservative for rural communities like Naalehu, as substantiated by flows observed at Honokaa WWTP and discussed in Section 1.5.

1.3 Service Area

Fukunaga & Associates, Inc (FAI) prepared a preliminary engineering report for the Naalehu wastewater collection system improvements in May 2020. The results of FAI's investigation established an updated service area for the proposed Naalehu WWTP that incorporates both the town's and County's needs. Figure 1 shows the updated WWTP service area, which includes properties that are currently connected to the three large capacity cesspools (LCC 3, 4, and 5), and properties that will be "newly accessible" to the collection system after the replacement collection system is constructed. LCC 1 and 2 are located in the neighboring town of Pahala.

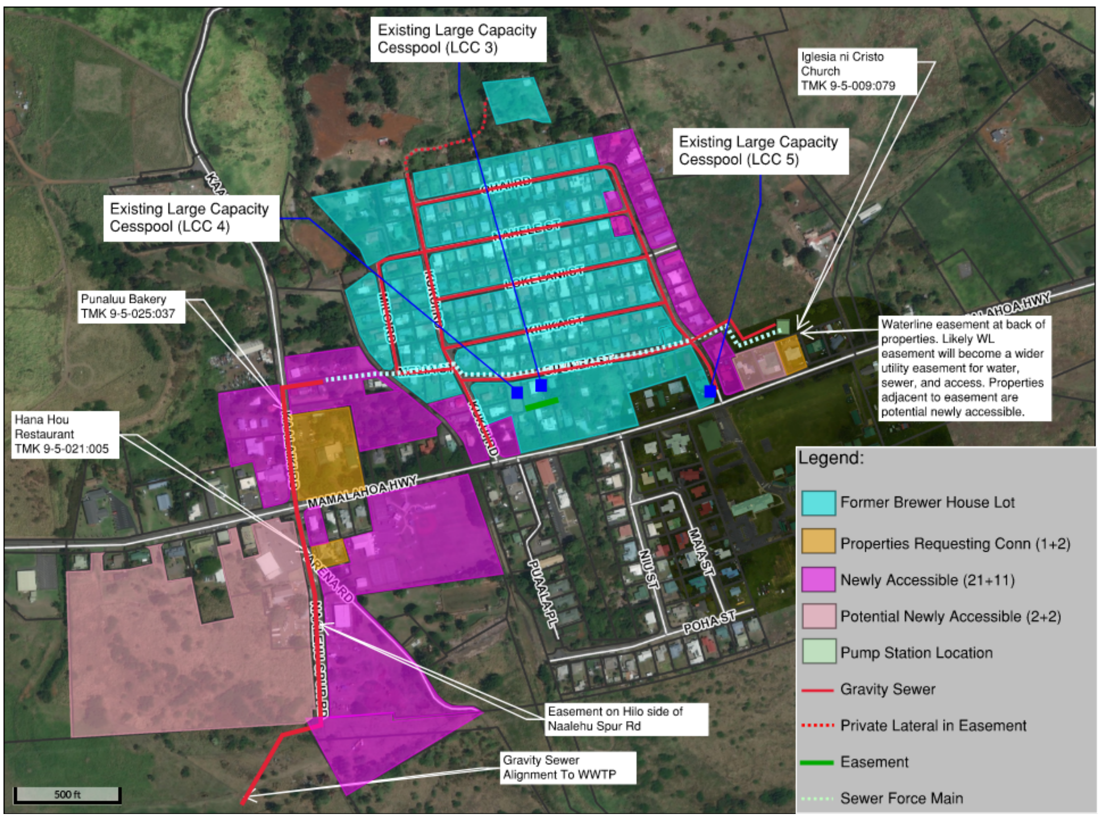


Figure 1. Naalehu WWTP Service Area Established by Fukunaga & Associates, Inc

Table 2 provides a summary of the WWTP service area.

Table 2. Naalehu WWTP Service Area Summary

Property Type	Number of Parcels		
	Existing C Brewer Lots	Newly Accessible Lots	New Collection Total
Residential	159	25	184
Commercial	2	7	9
Church	1	1	2
Industrial	-	2	2
Agricultural	1	2	3
Residential/Commercial	1	1	2
Residential/Agricultural	-	1	1
Park	-	1	1
Total	164	40	204

1.4 Potable Water Use

The amount of wastewater generated within a residence will not exceed the amount of potable water used by the occupants. Therefore, potable water use records can be used to estimate wastewater generation rates within existing communities where no combined sewers are present. The County of Hawaii Department of Water Supply (DWS) provided potable water use records for the parcels located within the service area from November 2017 through October 2022. Figure 2 provides an analysis of the potable water use records, with adjustments for properties for which no water use data was available. As shown figure 2, the maximum monthly average water use during the period of record was approximately 78,000 gpd in May and June 2018.

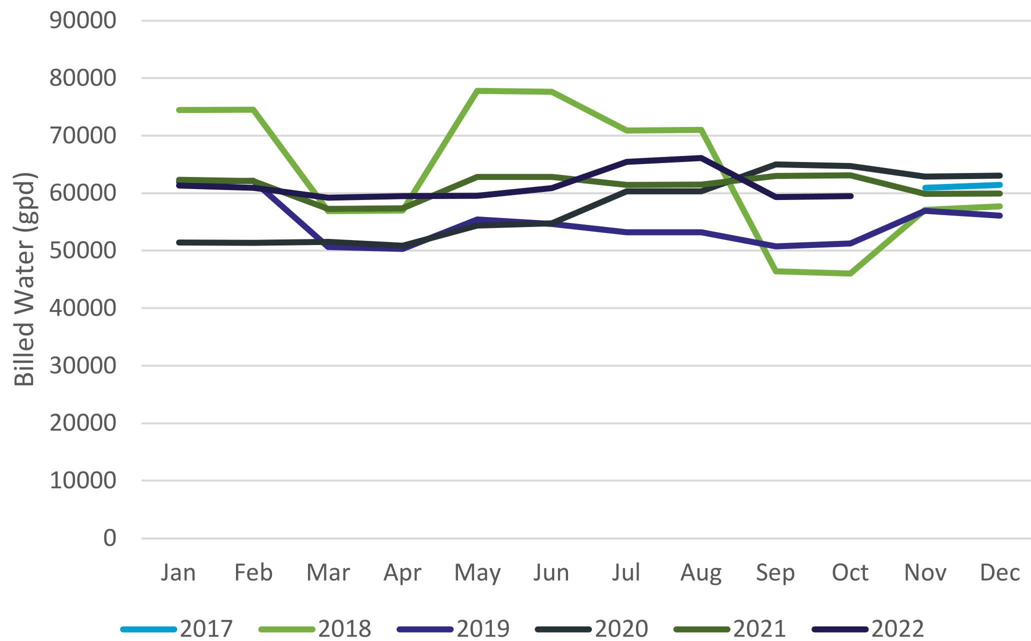


Figure 2. Average Potable Water Use in Naalehu WWTP Service Area, Nov 2017 – Oct 2022

Potable water is used both indoors and outdoors in residential areas, but only indoor uses enter the sewer system. Figure 3 shows the results of an irrigation demand analysis for Naalehu. Peak irrigation demands occur during the months of May through August, but inspection of potable water demands in Figure 2 shows little to no increase in potable water use during the driest months of the year. Therefore, it is reasonable to conclude that Naalehu residents do not use significant volumes of potable water for outdoor irrigation purposes, and most of the potable water that is supplied to the community by DWS ends up in the sewer.

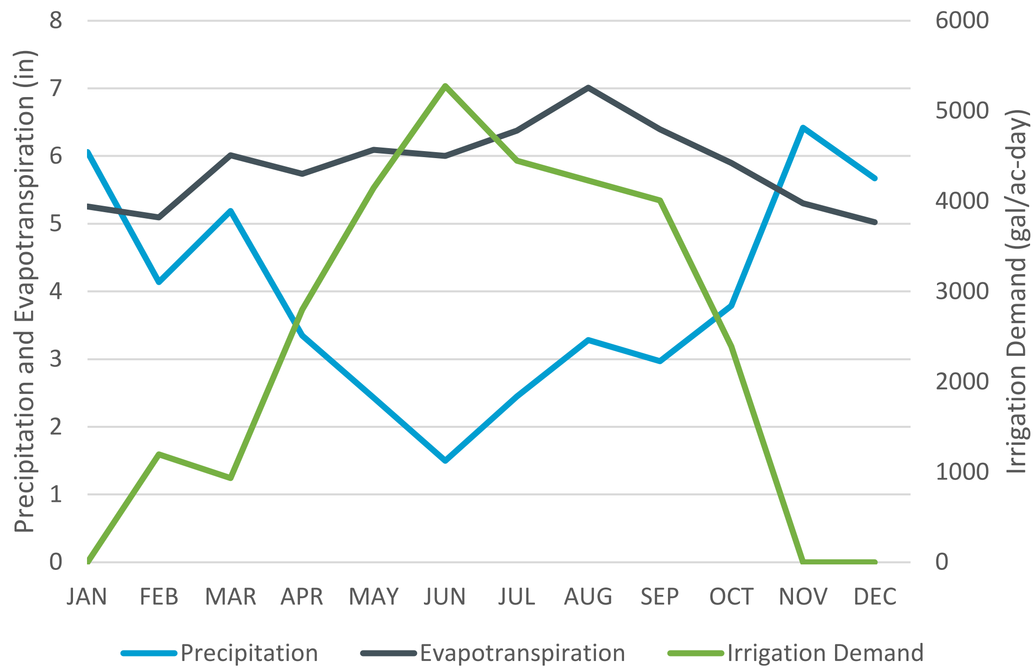


Figure 3. Naalehu Irrigation Analysis

Based on the above analyses, we believe assuming an 80,000 gpd monthly average wastewater generation rate in the capacity calculations reflect the current needs of the service area.

Daily wastewater flows can vary substantially from the monthly average flow. The CCH standards use a 2.5 peaking factor to estimate the maximum wastewater flow into the collection system. The 2.5 peaking factor is appropriate for Naalehu, resulting in a maximum wastewater flow of 200,000 gpd.

1.5 Dry Weather I/I Allowance

Groundwater can infiltrate into wastewater collection systems during dry weather, increasing the flow to the WWTP. The 2017 CCH standards specify a dry weather I/I allowance of 35 gpcd. The previous CCH standards (dated 1993) specified a dry weather I/I allowance of 5 gpcd for properties located above the groundwater table. The significant increase in the dry weather I/I allowance in the latest CCH standard will help the CCH to plan for increased dry weather I/I due to sea level rise effects in Honolulu. The Honokaa WWTP was designed for an average dry weather flow (ADWF) capacity of 200,000 gpd, which includes a dry weather I/I allowance based on the 1993 CCH standards. The County's experience at Honokaa has been that the 1993 dry weather I/I allowance is adequate for a rural collection system located in Hawaii Island's well-drained geology, at elevations hundreds of feet above sea level, and a significant distance from the shoreline. Figure 4 summarizes annual average effluent flows at the Honokaa WWTP, which include dry weather I/I. The high flows in July 2016 and August 2018 were due to Hurricanes Darby and Lane, respectively. We conclude that continued use of the 1993 standard for dry weather I/I is appropriate for Naalehu and using the 2017 standard would be overly-conservative.

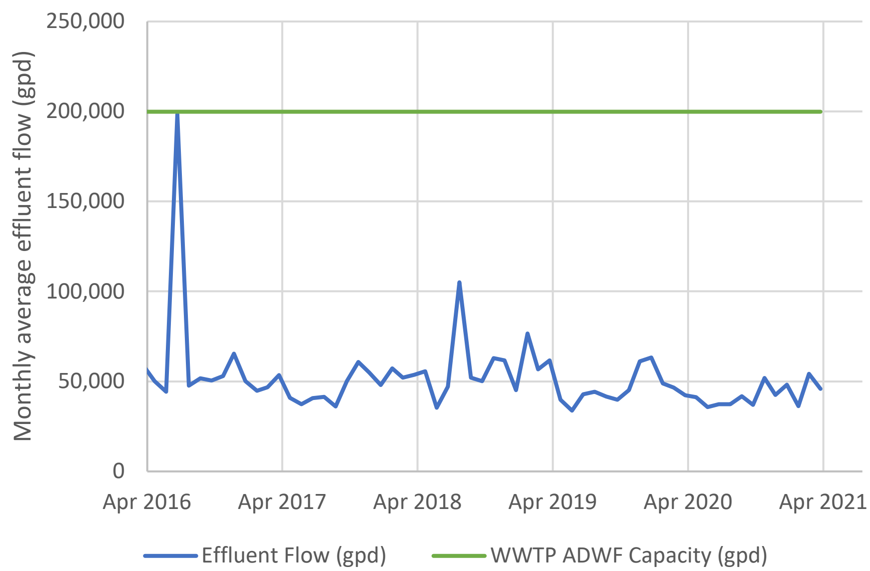


Figure 4. Honokaa WWTP Monthly Average Effluent Flows, Apr 2016 – Apr 2021

1.6 Wet Weather I/I Allowance

The 2017 CCH standards, which specify a wet weather I/I allowance of 3,000 gpad, are used for all wet weather I/I calculations.

1.7 Phased Capacity Implementation Recommendations

A phased approach towards implementing WWTP capacity is recommended to provide a flexible and appropriately sized wastewater treatment and disposal facility for the Naalehu community:

- Phase 1: Provide sufficient capacity for the existing parcels within the service area, including newly accessible parcels, reflecting current development. This will allow the County to close the LCCs. Provide sufficient area within the WWTP site for future expansion.
- Phase 2: When needed, expand the WWTP capacity to reflect the full CCH flow standards requirements for the existing service area. HAR 11-62 requires development of a facility plan when flows reach 75 percent of capacity and implementation of capacity-increasing measures at 90 percent.
- Phase 3: When needed, expand the WWTP capacity to reflect the recommendations included in the Kau Community Development Plan to sewer the entire community. This is not anticipated to occur within the near future and is not part of the current project's environmental assessment.

Table 3 provides a summary of the calculated capacities for the three-phase approach.

Table 3. Calculated Capacity Phasing

Description	Phase 1	Phase 2	Phase 3
Base sanitary flow	80,000 gpd	147,000 gpd	211,000 gpd
Peak base sanitary flow [a]	200,000 gpd	368,000 gpd	528,000 gpd
Dry weather I/I [b]	12,000 gpd	78,000 gpd	119,000 gpd
Wet weather I/I [c]	245,000 gpd	245,000 gpd	931,000 gpd
Average dry weather flow [d]	92,000 gpd	225,000 gpd	330,000 gpd
Peak day dry weather flow [e]	212,000 gpd	446,000 gpd	647,000 gpd
Peak day wet weather flow [f]	322,000 gpd PF=3.5	563,000 gpd PF=2.5	825,000 gpd PF=2.5
Peak hour wet weather flow [h]	317 gpm (457,000 gpd)	480 gpm (691,000 gpd)	1096 gpm (1,578,000 gpd)

[a] Base sanitary flow x 2.5 peaking factor.

[b] 5 gpcd for Phase 1, 35 gpcd for Phase 2 and Phase 3.

[c] 3,000 gpad

[d] Base sanitary flow + dry weather I/I.

[e] Peak base sanitary flow + dry weather I/I.

[f] Average dry weather flow x peaking factor (PF) shown. Peak day wet weather flow estimate was developed using an appropriate peaking factor.

[h] Peak base sanitary flow + dry weather I/I + wet weather I/I.

HAR 11-62-23.1(i) requires the initiation of a facility planning process when the actual wastewater flows reach 75 percent of the design capacity of the WWTP, and implementation of the facility plan must be initiated when actual wastewater flows reach 90 percent of the design capacity. **We recommend the Phase 1 WWTP be rated to treat an average dry weather flow of 125,000 gpd** to avoid the potential of having to initiate a facility plan shortly after the project is constructed. The peak day and peak hour flow capacities can remain as shown in Table 3. Note that the biological processes in the mechanical WWTP will need to be sized to treat the peak day dry weather flow of 212,000 gpd, not the average dry weather flow.

Per capita wastewater generation rate estimates were considered to check the validity of the proposed Phase 1 average dry weather flow capacity. The non-residential properties in the service area are expected to contribute approximately 50 percent of the total flows. Therefore, 50 percent of the average dry weather flow capacity can be allocated to the residential properties in the service area (92,000 gpd x 50% = 46,000 gpd). If we consider only the 184 residential properties in the service area, and assume four residents per residential lot, we have a total residential population of 736 persons (184 lots x 4 persons/lot = 736 persons). Dividing the residential flow capacity by the population yields a wastewater flow capacity of approximately 63 gpcd (46,000 gpd / 736 capita = 63 gpcd), which is within the typical values presented in Section 1.2 above.

The proposed Phase 1 capacity is based on actual water use data to establish wastewater generation rates, and rational assumptions to establish dry weather I/I allowances, and we believe it is appropriate for the Phase 1 existing conditions, while providing limited capacity for growth. Table 4 presents the recommended Phase 1 capacity.

Table 4. Recommended Phase 1 Capacity

Description	Value	Peaking Factor
Average dry weather flow	125,000 gpd	1.0
Peak day dry weather flow	212,000 gpd	1.7
Peak day wet weather flow	322,000 gpd	2.6
Peak hour wet weather flow	457,000 gpd (317 gpm)	3.7

If the County pursues a WWTP approach (using the Phase 1 capacity recommendation above) to close the LCCs then a DOH variance from HAR 11-62 requirements will be needed. The variance will need to be renewed every five years. The WWTP capacity needs should be re-evaluated upon application for the variance renewal.

Please call me at (808) 442-3301 if you have any questions.

Very truly yours,

Brown and Caldwell

Craig Lekven, P.E.
Project Manager

Appendix C: Non-Economic Evaluation

Naalehu WWTP Alternative Solutions
Non-Economic Evaluation July 2023

Category	Category Weight	Criteria	Criteria Weight	Raw Scores Alt #1 RAS	Raw Scores Alt #2 MBR	Raw Scores Alt #3 RGF	Weighted Scores Alt #1 RAS	Weighted Scores Alt #2 MBR	Weighted Scores Alt #3 RGF
Level of Service	25%	Effluent quality	30%	3	5	3	0.90	1.50	0.90
		Potential for capacity expansion	20%	5	5	2	1.00	1.00	0.40
		Water recycling feasibility	30%	3	5	2	0.90	1.50	0.60
		Public perception / community concerns	20%	3	5	3	0.60	1.00	0.60
Regulatory	25%	Monitoring complexity	25%	4	3	5	1.00	0.75	1.25
		Treatment adjustment potential	25%	5	5	3	1.25	1.25	0.75
		Safety regulations complexity	25%	4	4	4	1.00	1.00	1.00
		Environmental concerns	25%	4	5	3	1.00	1.25	0.75
O&M Factors	25%	Footprint	30%	5	5	2	1.50	1.50	0.60
		Safe work environment	25%	4	3	4	1.00	0.75	1.00
		Maintenance complexity	25%	4	3	4	1.00	0.75	1.00
		Operations complexity	20%	4	3	5	0.80	0.60	1.00
Island Factors	25%	Mainland delivery dependence	25%	3	3	4	0.75	0.75	1.00
		Mainland servicing dependence	25%	3	3	4	0.75	0.75	1.00
		Power dependence	25%	3	3	5	0.75	0.75	1.25
		Chemical dependence	25%	4	3	4	1.00	0.75	1.00
						Overall Score:	**3.80**	**3.96**	**3.53**

****Note* : 5 = high/desirable, 1 = low/not-desirable*

Appendix D: Naalehu Wastewater Collection System Improvements Technical Memorandum

Fukunaga & Associates, Inc., May 2020

FUKUNAGA & ASSOCIATES, INC.
CONSULTING ENGINEERS

MEMORANDUM

TO: Michelle Sorensen, Brown and Caldwell
Craig Lekven, Brown and Caldwell

FROM: Andrew Amuro

DATE: May 1, 2020

SUBJECT: Naalehu Wastewater Collection System Improvements
Final Preliminary Engineering

1. GENERAL PROJECT DESCRIPTION

The County of Hawaii (COH) is scheduled to close three large capacity cesspools (LCCs) in the town of Naalehu on the southeast side of the Big Island. The Naalehu town area is shown on **Figure 1**. To accomplish the closure, the COH has tasked Brown and Caldwell (B&C) with the planning and preliminary engineering for a wastewater treatment plant (WWTP) to serve the properties impacted by the LCC closure. Fukunaga and Associates, Inc. (FAI) has been tasked with investigating options for the collection system to convey the wastewater from the impacted properties to the proposed WWTP. The focus of the project is to close the LCCs as expediently and economically as possible; however, the Consent Order also requires 30 connections to the WWTP in addition to the Brewer properties currently served by the LCCs.

Three properties requested connection to the County sewer system and are presented in **Figure 1**. These properties are:

- Iglesia ni Cristo Church (TMK 9-5-009:079)
- Punalu'u Bakery (TMK 9-5-025:037; owned by La'i LLC)
- Hana Hou Restaurant (TMK 9-5-021:005; owned by Fujimoto, Drake S.)

These properties are not connected to the former Brewer sewer system. The church property is located on the Hilo side of the former Brewer house lots and can be connected to the collection system by adding a sewer within the State right-of-way (ROW) (see Section 2) or by adding a sewer within an easement behind the propert if the County chooses to extend the sewer to this property. The Punalu'u Bakery and Hana Hou Restaurant are located on the Kona side of the former Brewer house lots. This side of Naalehu town, to the west of the concrete-lined drainage canal, is considered the "commercial" side of town. The collection system can be designed to accommodate connection of these two commercial properties and will serve to define the western extent of the collection system.

2. GENERAL CRITERIA

The COH will be the owner of the collection system; therefore, the sewer system will meet COH standards and be accessible for maintenance. Preference is to construct sewers within the County ROW as much as possible unless other factors make placing the pipes within easements on private property or in the State ROW much more practical from economic and engineering standpoints. The COH prefers sewers not be within easements due to future access challenges when maintenance and repairs are needed. It is also recommended that the collection system not be located in a State ROW if possible due to the lengthy review and approval time needed for the Use and Occupancy Agreement (UOA). This was emphasized by the State Department of Transportation Highways Division (DOT-H) in their Environmental Assessment (EA) pre-consultation comment to stay out of the State ROW unless there is no other feasible means of routing the sewer. The collection system will require a crossing of the State ROW to convey the sewage from the community Mauka of the highway to the WWTP site that is Makai of the highway; however, DOT-H tends to take less time when reviewing ROW crossings, especially if they are done using trenchless installation techniques.

3. TOPOGRAPHY

The topography of the Naalehu area is presented in **Figure 2**. The proposed location of the wastewater treatment plant is upgradient of the lowest of the former Brewer properties. The ground elevation of the WWTP site is approximately 690 ft MSL while the ground elevation of the lowest Brewer house lot is about 645 ft MSL, an elevation difference of 45 feet. A gravity sewer at this depth is technologically possible using micro tunneling techniques for construction; however, a wastewater pump station will be more feasible to convey the wastewater to the WWTP due to the cost of micro tunneling a relatively short distance. The micro tunneling option can be investigated in the event that an acceptable wastewater pump station location cannot be found.

4. POTENTIAL WASTEWATER PUMP STATION LOCATIONS

As noted in the previous section, the former Brewer properties in Naalehu would most likely be served by a pump station due to the topography. When investigating potential sites for a wastewater pump station (WWPS), priority was placed on sites that are on County or State lands to minimize land acquisition issues, or large private parcels where a portion can be purchased by the County in hopes of minimizing impact to the landowner. The potential WWPS sites are listed in Table 1, and shown on **Figure 3**.

Table 1: Potential WWPS Sites

TMK	Owner	Current Use	Class	Size (sq ft)	Notes
9-5-008:012	Chinese Society Church	Church	Agricultural	186,001	Requires land from the owner and/or easement for WWPS site, utilities and road. Requires sewer and force main in State ROW. WWPS can be set back away from houses. Located at lowest point of the collection system. Within 550 feet of school property line. Longest distance for transmission to proposed WWTP location.
9-5-008:048	Kuahiwi Contractors Inc.	Storage/ Baseyard	Agricultural	534,307	Requires land from the owner and/or easement for WWPS site, utilities and road. WWPS can be set back away from most residents. Within 300 feet of school property line. Located near lowest point of the collection system requiring approx. 15 foot wet well.
9-5-009:006	State of Hawaii	Naalehu School	Residential	534,481	Site is frequented by the public. Residents have indicated sensitivity to wastewater facility at a school. Located at lowest point of the collection system.
9-5-021:023	County of Hawaii	Naalehu Park	Residential	279,176	Site is frequented by the public. Residents may be sensitive to wastewater facility at a park. Requires sewer in State ROW. Located upgradient of some of the former Brewer properties.
9-5-021:035	Thy Word Ministries – Waikoloa Faith Ctr	Vacant	Residential	134,949	Requires land acquisition from the owner and access easement or dedicated County road. Routing would most likely have sewer within State ROW. WWPS can be set back away from most houses.
9-5-024:007	‘O Ka’u Kakou Inc.	Senior Center	Residential/ Commercial	84,680	Facility currently in construction, timing to coordinate location could be difficult. Residents may be sensitive to wastewater facility within senior center campus.
9-5-024:011	County of Hawaii	LCC Location	Residential	10,347	Site is surrounded by houses. Facilities will be relatively close to property line. Residents may be sensitive to wastewater facility in residential area. Located up-gradient of the lowest Brewer properties requiring 30+ foot deep wet well.

Two of the potential sites appear to be more favorable than the others, the County site located within the former Brewer house lots (TMK 9-5-024:011) and the Kuahiwi Contractors site (TMK 9-5-008:048). The remaining parcels were eliminated from consideration at this time but can be re-visited in the future if needed. The Naalehu School and Naalehu Park sites would likely face the most public opposition due to proximity to areas frequented by the general population for recreation and education. The Chinese Society Church and Thy Word Ministries sites would require land acquisition after negotiation with the landowner and a UOA within the State ROW. Both processes would be time consuming and the land acquisition depends on the landowner's willingness to cede some of their land to the County. The new senior center parcel is currently in construction, making the timing to negotiate the WWPS site difficult and may possibly incur construction delays that would also factor into the land negotiations.

A. Pump Station on County Parcel TMK 9-5-024:011

The County site would be the most favorable from a timing standpoint. It does not require land acquisition and would minimize work in the State ROW by limiting construction to a single crossing of Mamalahoa Highway. However, since this site is up-gradient from the collection system low point, this site will require deeper sewers and a very deep pump station wet well. Construction on this site would be more challenging due to the narrow site and the existing sewers and LCC that would have to remain in operation until the collection system and WWTP are completed. These utilities, especially the existing sewer pipes, would have to be protected during construction as heavy vehicles and machinery enter and leave the site. The condition of the existing pipes may require additional protection such as supplemental concrete cover and/or partial replacement depending on condition. A proposed layout of a pump station on the County parcel is shown in **Figure 4**.

The need to trench for the deep influent sewer and the deep wet well may result in flow from the LCC being diverted to these trenches due to the lower resistance to flow through the ground to those excavated areas. Using mud walls for groundwater control would hinder the flow of sewage from the LCC into the ground and probably reduce the capacity of the LCC. Therefore, septage pumping for the duration of the pump station construction would most likely be required until the collection system and WWTP are complete. This would be a significant cost to the construction. In addition, should there be unforeseen delays in completion of the collection system and/or WWTP, the additional septage handling due to the delay could result in a significant construction change order for the additional pumping.

The on-site stormwater may be more difficult to contain due to the small area. Pervious pavement, if geotechnical investigation shows this type of pavement is permissible, would be used to minimize hard surfaces on the site. The only hard pavement area required would be the area designated for refueling of the generator. The grade of the site would be adjusted to minimize flow of off-site stormwater. Grade adjustment walls are anticipated along the property lines on the south and west sides of the parcel to raise the ground elevation.

This site is surrounded by residences directly adjacent to the property line on the east, south and west. The north side is bordered by Opukea Street. Due to the narrow site, the house structures are only about 60 feet from the proposed pump station location. The ambient noise from odor control ventilation fans and electrical switches could be a nuisance to the residents if not mitigated. The standby generator would also produce noise during testing and operation. This would require additional sound insulation for the generator and other equipment on the site.

A passive odor control system can be used to eliminate the need for a mechanical fan; however, passive systems rely on the collection system to be at a higher pressure than atmosphere to force the foul air through the odor control system. When the collection system becomes pressurized, odors can begin to leak out of the wet well hatches and nearby manholes so they must also be gasketed or sealed. A forced-air odor control system using a mechanical fan to draw air out of the pump station would keep the wet well and collection system at negative pressure, thus minimizing the chance that odors escape from nearby manholes and hatches.

Electrical and water utilities are located just off-site on Opukea Road. The HELCo transformer for the site can be located within an easement just off the County road. Water for fire protection could be from existing hydrants (pending analysis by a fire protection engineer). Water service is anticipated to be similar to a residential unit to provide wash down and general housekeeping hose bibbs.

This site poses several risks during construction that could add significantly to the cost and time to construct the pump station. The site is also narrow and cannot be configured optimally to mitigate noise, which could be significant to the homeowners directly adjacent to the site.

B. Pump Station on Portion of Kuahiwi Contractors, Inc. Parcel TMK 9-5-008:048

The Kuahiwi Contractors site is private property and would require time to negotiate the subdivision or easement for the WWPS site and access roadway; however, similar to the County site, work in the State ROW can be limited to a single crossing of the highway. The anticipated size of the site is about 8000 to 9000 square feet. A proposed pump station layout and site are presented in **Figure 5**. A larger area may be needed if soil conditions require a larger area to mitigate on-site stormwater flows. This is smaller than the County parcel above but the dimensions can be optimized to provide the area needed for the facility. This site is located at the low point of the collection system, which would keep the sewers and pump station wet well at more typical depths. Locating the WWPS on a portion of the Kuahiwi Contractors site would allow sewering of the Iglesia ni Cristo Church that requested connection but will place it at the farthest point from the WWTP, thus increasing the force main and sewer lengths required. An access road or driveway would also be needed to the site. A gravel drive would minimize the impervious surface area but can be susceptible to erosion during heavy rains.

The pump station would be placed close to the south property line, the low side of the parcel. The far south east or Hilo corner of the parcel is the lowest point where stormwater from this parcel and possibly some adjacent parcels collect before entering a drainage culvert under Mamalahoa

Highway. Anecdotal reports from residents indicate the highway floods at the storm drain during heavy or sustained rains. There is also a swale that appears to divert stormwater from cattle and horse pens on the site toward the far south east corner. It is recommended that the County not encroach into the far south east corner to maintain the current storm drainage pattern from the site. An aerial view of the area is shown in **Figure 6**. On-site stormwater can be contained by making the site large enough for impoundments and by using pervious pavement where possible depending on geotechnical investigation data. The only road area requiring hard surfacing is the area designated for re-fueling the generator. Grade adjustment walls are anticipated along the perimeter of the pump station site to raise the grade on the south side and lower the grade on the north side to even out the elevation of the site and minimize inflow of off-site storm water.

The site would be bordered on the south by residences unless the pump station is moved further interior of the parcel. If the pump station site is located against the south property line, the pump station could be as close as 80 feet from the house structures. Similar to the County parcel, the ambient noise from equipment could be a nuisance to the residents if not mitigated. This parcel would allow the County to obtain a more optimally configured site that would allow an additional motor control center (MCC) room that would not only mitigate noise issues from electrical switches, but also allow the valuable electrical equipment to be secured in a locked room. The MCC room can be integrated into the generator building. Noise from the odor control system could also be an issue, especially from the fan that would operate continuously. However, the site can be optimally sized to allow sound attenuating enclosures around the fan or around the entire odor control system. Therefore, a passive odor control system as discussed above for the County parcel site is probably not needed at this site.

Running electrical utility and communication lines would be more costly for this site since the closest pole would be on Ohai Road. HELCo would charge the County to run utility lines to their transformer near the pump station site. As an alternative, the HELCo transformer and meter can be placed within an easement near Ohai Road and the County would own the power lines going into the site. This option is mentioned in case HELCo prefers not to own the power lines going into the site or requires road improvements that would not be favorable to the County. This would require discussion with and approval by HELCo and would have to be analyzed further by an electrical engineer.

A potable water line would be required at the site for wash down and general housekeeping. In addition, a hydrant for fire protection would be needed. The potable water pipe in Naalehu appear to be 6-inch diameter. The new potable water line to the site would maintain the 6-inch size to maximize the water flow in case of fire or other emergency requiring water.

This site is preferred from an engineering standpoint because it is at the low side of the collection system and does not entail unknowns during construction such as breakage of existing utilities or need for septage pumping for unknown duration. It can also be optimally configured to mitigate noise and odors.

5 COLLECTION SYSTEM DESIGN

The town of Naalehu is divided by a concrete flood control canal into an east side (Hilo side) where the majority of the properties are residences, and a west side (Kona side) where the majority of the properties are commercial. The town is also divided in the Mauka (mountain) and Makai (ocean) directions by the Mamalahoa Highway. The collection system will convey the wastewater to the proposed WWTP that is located on the Makai-Kona side of Naalehu. The Naalehu collection system will be analyzed as two distinct systems, one serving the former Brewer house lots located on the Mauka-Hilo side to allow closure of the LCCs, and the gravity sewer located on the Kona side to transport the sewage to the proposed WWTP. An added benefit would be provision of sewer service to the two restaurants requesting connection. The church on the Hilo side that requested service falls outside of this area. The County has expressed a desire to accommodate their request; however, depending on the location of the WWPS, connection of the church may have to be deferred.

C. Former Brewer House Lots

The former Brewer house lots are located on the Mauka-Hilo side as shown in **Figure 7**. The houses are on a hillside with a slope of 8% to 9%, ranging in elevation from about 780 ft MSL to 645 ft MSL. The steep hillside results in houses located on the downhill side of the street being below the elevation of the street. Sewage from houses on the uphill side can flow easily by gravity to a sewer in the street. On the downhill side; however, the lateral serving the home is lower than a typical depth sewer (typically designed with 4 feet to 5 feet cover). Four options were investigated to address the low-lying properties; use of easements on private property, vacuum sewers, individual grinder pump stations at low-lying properties, and deep sewers to allow gravity flow from the low-lying properties.

i. Easements

The use of easements was eliminated as a viable option at this time. An initial investigation of the house lots along the backyard property lines where an easement would be most beneficial revealed extensive homeowner improvements. As part of sewer construction and easement terms, the improvements can be removed and the homeowners can be ordered to keep improvements off of the easements. However, over time it is likely that the homeowners' memory of the easement locations would fade and various improvements may be placed over the easement, hindering access by the County for repairs and maintenance.

ii. Vacuum Sewers

The use of vacuum sewers was also eliminated at this time. Vacuum sewers use a sealed sewer system that produces suction to move the wastewater from low or flat areas. Research into these types of systems revealed that it is not cost effective to sewer the uphill houses by gravity and the downhill houses by vacuum because of the need for a sealed piping system for vacuum service. Serving some house by conventional gravity and some by vacuum would require two sewer collection systems in the County ROW.

The vacuum interface valve that would be located on each property requires frequent maintenance checks to ensure proper operation. These checks should be done by trained maintenance personnel but since these valves would be on private property, the responsibility would fall on the homeowner to hire trained personnel or obtain training on the system. Another drawback is that the vacuum interface valves typically fail open to minimize the chance of spills. A failed valve will therefore result in the County-owned vacuum pumping system to run constantly (normal operation would be intermittent to save energy). County maintenance personnel would not be able to readily find the faulty valve since they would not have right-of-entry onto the individual properties. Also, since there is no spill or "problem" with the sewer at the individual property with the faulty valve, there is no incentive to correct the problem in a timely manner. Remote telemetry can be added to the vacuum interface valves to monitor the valves via SCADA but these would also require periodic maintenance on private property and would add to the cost of the system for monitoring at every property.

iii. Individual Grinder Pump Stations

The original concept developed for Brewer before the company dissolved was to have individual grinder pump stations at the low-lying properties. This type of system consists of a small pump station at each downhill home. As the pump station fills with sewage, the high wastewater level starts a pump that evacuates the station and discharges into the gravity sewer system on the street. Houses on the uphill side of the street would discharge sewage by gravity into the collection system. The advantage of individual grinder pump stations is the ability to keep the collection system as shallow as possible.

Disadvantages of this system are the need to maintain the pumps and valves on private property and the energy used to pump the wastewater. Another disadvantage is the potential for spills when there are mechanical failures such as, power outages, pump breakdowns, level switch malfunctions, and clogs within the pump. Similar to the vacuum sewer system, the mechanically intensive nature of individual grinder pump stations requires trained maintenance personnel to perform regular checks on the system. However, since the system belongs to the homeowner, it is their responsibility to either hire a maintenance service or obtain the necessary training. Unlike the vacuum system, the pump stations are not "fail safe" and will result in sewage backup in the home or a spill on the homeowner's property that would prompt them to seek corrective action. A rough life-cycle cost analysis of an individual grinder pump unit resulted in an annual cost burden of $850 per house on the homeowners.

The subsurface conditions of the area at the depths required for the sewers is yet to be investigated. If the deep gravity sewers (see below) are not feasible due to unfavorable conditions; such as, lava tubes, then the sewers may have to be kept as shallow as possible. This option is feasible and will be kept in consideration in the event that subsurface conditions require it, but is not the preferred option due to the disadvantages, especially the cost burden on the homeowners.

iv. Deep Gravity Sewers

The majority of the collection system sewer mains can be kept within the County or State ROW and fed by gravity if the pipes are deep below the low point of the low-lying parcels. Sewers constructed in the County of Hawaii must adhere to the *Wastewater System Design Standards of the City and County of Honolulu*, which requires review and approval for sewers deeper than 15 feet due to higher soil loading and potential difficulty when repairs are needed. A preliminary layout of a gravity sewer system, as shown in **Figure 7**, resulted in sewer depths of about 20 to 25 feet at the deepest point on the upper portions of Ohai Road and Nahele Street, and 15 to 18 feet at the deepest point on Lokelani Street and Kilika Street. These depths are based on running a lateral from the lowest point on a house lot up to the street. This was used as the basis because there are houses with sinks and outhouses at the back of the lot that are assumed to be connected to the existing collection system. The soil depth to rock in the area as reported by USGS and verified by past geotechnical investigations is 40 to 60 inches; therefore, excavation for the deep gravity sewers would be mainly in rock.

A sewer along the highway or easements would be needed to service the houses on the State highway between Kukui Road and Ohai Road. The depth, length and direction of flow for this sewer would depend on the location of a pump station. The depth of the sewer on Opukea Street depends on the location of a sewage pump station (see Section 4). As a worse case, the manhole fronting the County-owned parcel (TMK 9-5-024:011) on Opukea Street would have an invert about 30 feet deep if a pump station is constructed on that site due to the elevation of the house at the corner of Ohai Road and Mamalahoa Hwy (TMK 9-5-024:001), the lowest point of the former Brewer community. Although 30 feet is rather deep, it is not unheard of at the entrance to pump stations in Honolulu which are often 3 stories deep. At the Kuahiwi Contractors site (TMK 9-5-008:048) the ground elevation is about the same as the house at Ohai Road and Mamalahoa Hwy resulting in sewers that are at a more reasonable depth of about 15 feet.

This option is feasible and will be considered further. Despite the depth of the sewers in the County ROW, the advantage of not burdening the homeowners with the cost of maintaining the pumping system in Section 5-A-iii, above, makes this option preferred over the individual grinder pump option.

D. Gravity Sewer from Force Main to WWTP

The primary purpose of the sewer on the Kona side of Naalehu is to convey the wastewater from the Brewer house lots to the WWTP. A gravity sewer on Kaalaiki Road can be used to collect the wastewater discharged from a pump station servicing the housing area on the Hilo side of the drainage canal at either the County parcel on Opukea Street or the Kuahiwi Contractors site. The force main can be run entirely Mauka of the highway, cross the drainage canal, and discharge into a manhole when it reaches Kaalaiki Road. An easement would be required from the drainage canal to the County road. There is an existing access and utility easement located on a parcel owned by La'i LLC (TMK 9-5-025:039) that goes from Kaalaiki Road to the canal. This would be the recommended method of conveying the sewage to the WWTP to keep construction in the State ROW to a minimum. The force main route is discussed further in the next section.

Two commercial properties requested connection to the new collection system. These were the Punaluu Bake Shop (TMK 9-5-025:037) and Hana Hou Restaurant (TMK 9-5-021:005). These properties can be connected to the collection system via the gravity sewer running down Kaalaiki Road, across the highway, and down Naalehu Spur Road to the WWTP site. There are potentially an additional eight properties that would be newly accessible along this sewer route depending on the disposition of Naalehu Spur Road, which is privately owned by Kuahiwi Contractors, Inc. If the road is dedicated to the County or an easement in favor of the County spans the entire road width, then both sides of Naalehu Spur Road would be newly accessible. If the sewer easement is only on one side of the road, assumed to be the Hilo side to allow connection of Hana Hou Restaurant (TMK 9-5-021:005), then only the Hilo-side properties would be newly accessible. The proposed Kona-side collection system is presented in **Figure 8**.

E. Force Main from Brewer House Lots to Gravity Sewer on Kona Side

A force main will be needed to convey the wastewater from the WWPS that serves the former Brewer house lots to the gravity sewer going to the WWTP on the Kona side. The force main can be kept out of the State ROW by running the pipe along easements and County roads. **Figure 8** shows the possible force main routes from the County parcel and the Kuahiwi Contractors sites. The approximate force main lengths from the County parcel and Kuahiwi Contractors parcel are 1300 LF and 2800 LF, respectively.

Both WWPS options will require crossing the concrete drainage canal that divides the Kona side of Naalehu from the Hilo side. The force main can go either above the canal on a support structure, or under the canal. There is an existing pedestrian bridge crossing the canal that can be reconstructed and used to support the new force main; however, the County DEM is not in favor of this option due to the possibility of the structure impeding flow through the canal and ownership of the bridge structure being a burden on DEM personnel who are not set up for bridge inspection and maintenance duties. Routing the force main under the canal can be done by either cutting the concrete canal structure and laying the pipe in an open trench or tunneling under the canal. Cutting the structure poses some risk should a rain storm occur while the trench is open or repaired concrete is curing. Tunneling would be costly due to the depth and equipment needed to micro-tunnel or the shoring required to excavate under the structure. Further discussions with a geotechnical engineer will be needed to determine the best method to cross the canal while limiting risk during construction. For planning purposes, it will be assumed that the canal crossing will be approximately 100 LF using micro-tunneling as a worse case.

6 COLLECTION SYSTEM COSTS AND RECOMMENDATIONS

Planning level capital costs were developed for the collection system options developed above. The costs are presented in Table 2. Basis for the costs are presented in Table 3. Cost basis numbers are from recent past project construction and bid costs inflated to the current year.

Table 2: Collection System Costs

Deep Gravity Sewer at House Lots	$12 million
Individual Grinder Pumps at House Lots	$9 million
WWPS at County Parcel	$7.1 million + $1.3 million FM = $8.4 million
WWPS at Kuahiwi Contractors	$4.9 million + $2.8 million FM = $7.7 million
Canal Crossing (Tunneled)	$0.7 million
Gravity Sewer from FM to Proposed WWTP	$3.3 million

Table 3: Cost Basis

	Unit Cost
Deep WWPS at County Parcel	$7.1 million
Typical WWPS at Kuahiwi Contractors Parcel	$4.9 million
Gravity Sewers	$1100/LF
Deep Gravity Sewers in Rock	$1600/LF
Deep Sewers (Tunneled)	$7000/LF
Sewer Force Mains (not including tunneled section under canal)	$1000/LF

Note: Cost of individual grinder pump system based on design cost estimate of $5.6 million prepared in 2012 inflated to 2020 at 6% average inflation ($5.6m x 1.06^(2020-2012) = $9 million)

The overall recommendation for the collection system is to use deep gravity sewers for the former Brewer housing area. Two backyard easements would be used to connect four properties to the collection system. These easements have already been surveyed and are located on the south side of TMK 9-5-024:009 and TMK 9-5-024:010. These easements would expedite implementation by avoiding the need for a sewer in the State ROW to serve three of the properties. Two commercial properties requested connection to the new collection system. These were the Punaluu Bake Shop (TMK 9-5-025:037) and Hana Hou Restaurant (TMK 9-5-021:005). These properties can be connected to the collection system via a gravity sewer running down Kaalaiki Road, across the highway, and down Naalehu Spur Road. There are up to an additional fifteen properties that would be newly accessible along this sewer route. The newly accessible house lots along with the newly accessible properties along the sewer to serve the commercial lots brings the total newly accessible properties to 39, exceeding the number identified in the Consent Order. If the WWPS is located on the County parcel, the overall estimated cost of the system would be $24.4 million. If the the WWPS is located on the Kuahiwi Contractors site, overall estimated cost would be $23.7 million. The recommended collection system options are presented in **Figure 9 and 10**.

If using deep sewers in the former Brewer house lots is not feasible due to subsurface conditions, the estimated cost of the collection system would be $21.4 million if the WWPS is on the County parcel, and $20.7 million if the WWPS is on the Kuahiwi Contractors site. However, there would be added cost for the homeowners with parcels below the road of approximately $850 per year per house to maintain the pumps. A summary of the overall costs is presented in Table 4.

Table 4: Overall Planning Level Cost Estimates

	Deep Gravity Sewers	**Individual Pumps**
County WWPS at County Parcel	$24.4 million	$21.4 million
County WWPS at Kuahiwi Contractors Site	$23.7 million	$20.7 million

7 COLLECTION SYSTEM PHASING

The collection system can be constructed in phases to expedite the closure of the LCCs. As a minimum, the collection system would have to intercept the existing sewers entering the LCCs and convey the sewage to a pump station. To minimize the need for temporary facilities that would be removed when the collection system is completed in a second phase, it would be beneficial to construct the permanent WWPS as part of Phase 1. Phase 1 would therefore consist of the following:

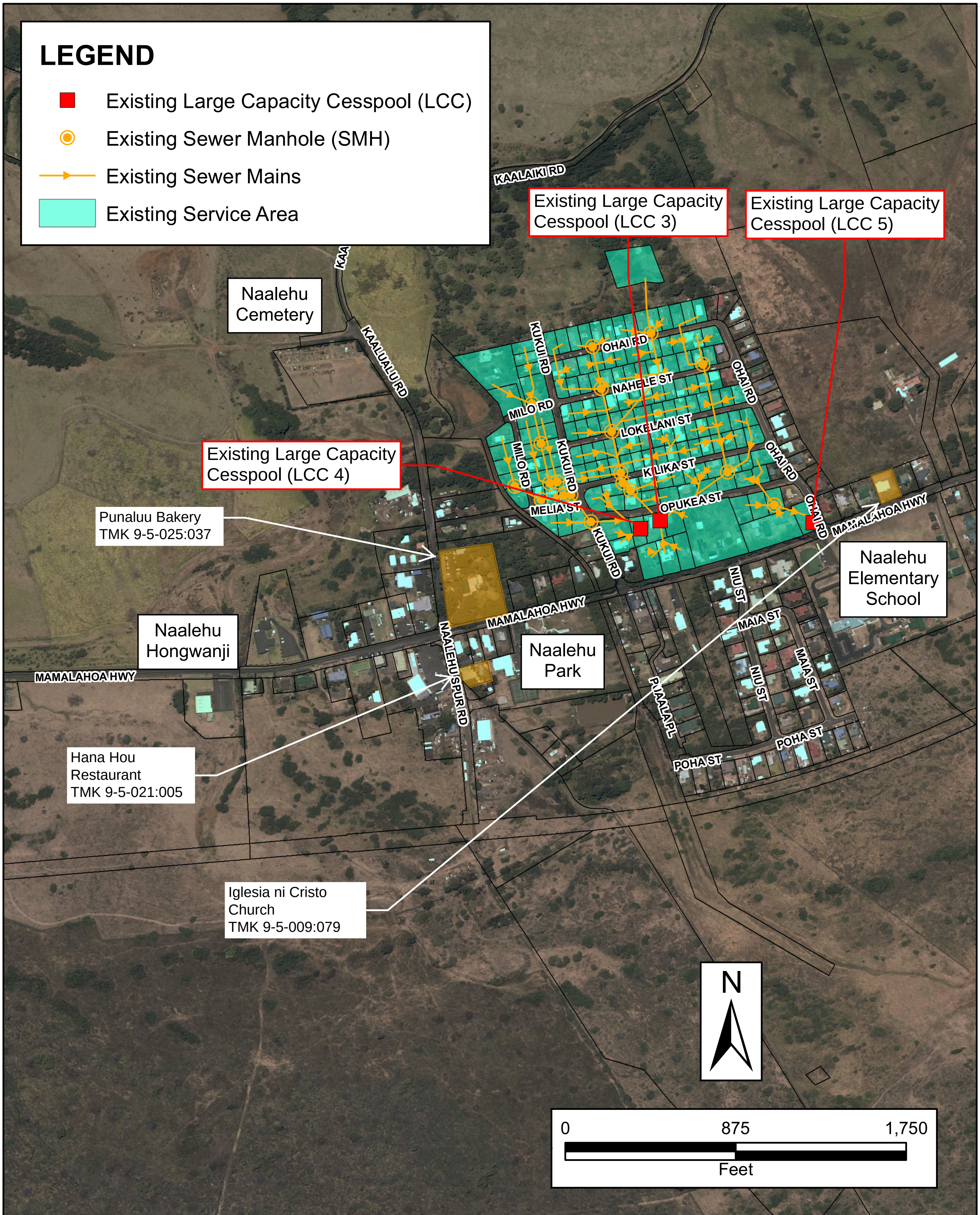
a. Construct Gravity Sewer on Kaalaiki Road and Naalehu Spur Road to the WWTP
b. Construct the WWPS and force main
c. Construct the gravity sewer on Opukea Street and Ohai Road to intercept existing sewers and connect to the WWPS

The sewer system for the proposed Phase 1 is presented in **Figures 11 and 12**. Sub-phases “a” and “b” can be done concurrently; however, intercepting the existing sewer system should be done at the end of Phase 1. This is so that the if there are issues encountered when constructing the sewers, the main WWPS and means of conveying the flow to the WWTP will be in place so that bypass pumping to the WWPS can be used if needed. If the main WWPS and conveyance sewers to the WWTP were not in place, the contractor would have a more difficult time with bypass pumping to the WWTP or would have to truck sewage if a problem occurs. The new WWPS would be close enough to allow temporary piping to be routed from the problem area into the wet well.

The cost to implement Phase 1 would be about $15.3 million with the WWPS on the County parcel, and about $15.1 million with the WWPS on the Kuahiwi Contractors site (not including land acquisition costs). A phased cost summary is presented in Table 5.

Table 5: Phased Cost Summary

	Deep Gravity Sewers		Individual Pumps	
	County Parcel	**Kuahiwi Parcel**	**County Parcel**	**Kuahiwi Parcel**
Phase 1	$15.3 million	$15.1 million	$12.3 million	$12.1 million
Phase 2	$9.1 million	$8.6 million	$6.1 million	$5.6 million
Total	$24.4 million	$23.7 million	$18.4 million	$17.7 million



From:

Brown AND Caldwell

SCALE AS SHOWN

JOB NO.: 151494

NAALEHU WASTEWATER TREATMENT PLANT

Existing Naalehu Wastewater System

FIGURE

1

Figure 2: Naalehu Topography

Elev 760± High Point

Elev 690± WWTP

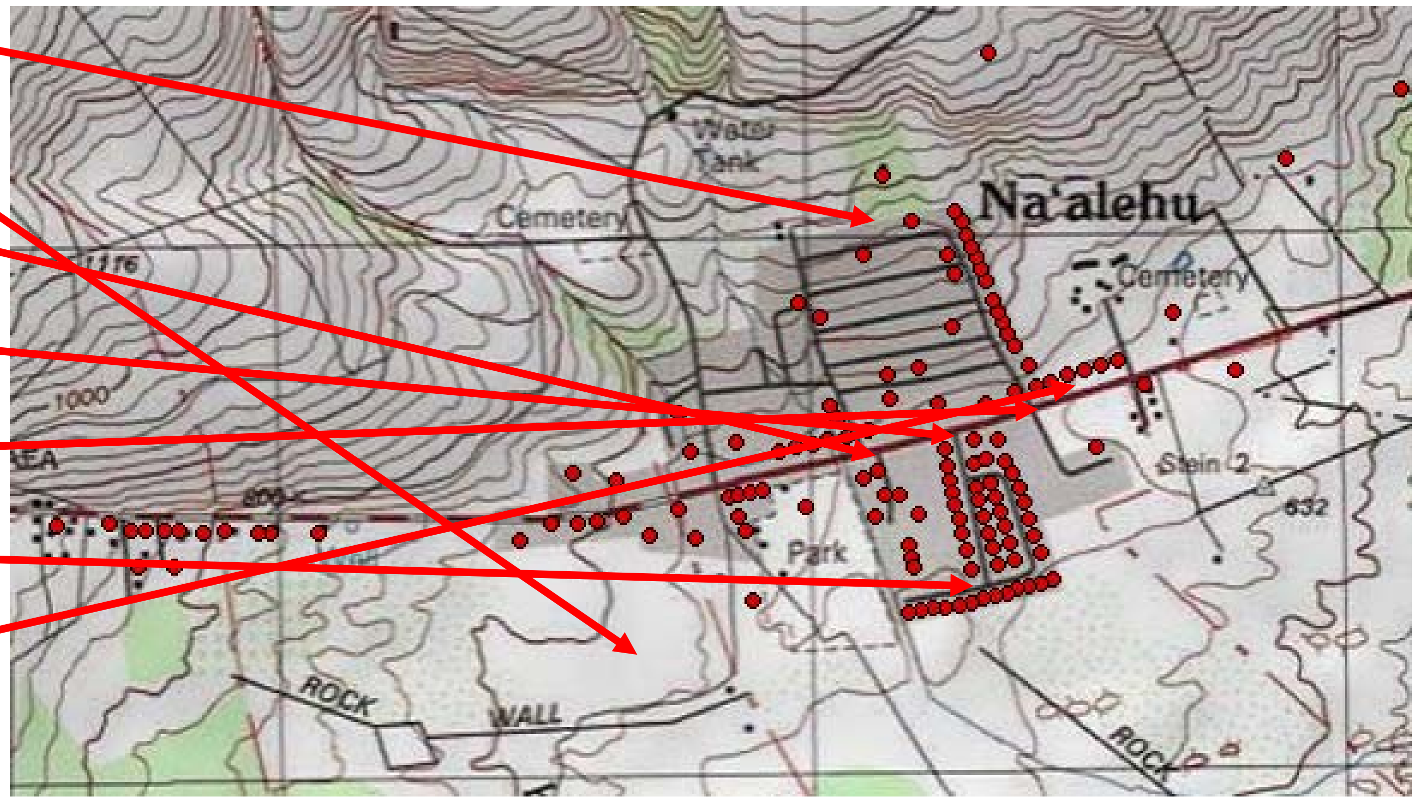
Elev 660± Kukui Int.

Elev 650± Niu Int.

Elev 645± Ohai Int

Elev 640± Niu/Poha Int.

Elev 640± Church



Source: Onsite Sewage Disposal Systems - Hawaii [arcgis_rest_services_Infrastructure_MapServer_26

Figure 3: Possible WWPS Locations

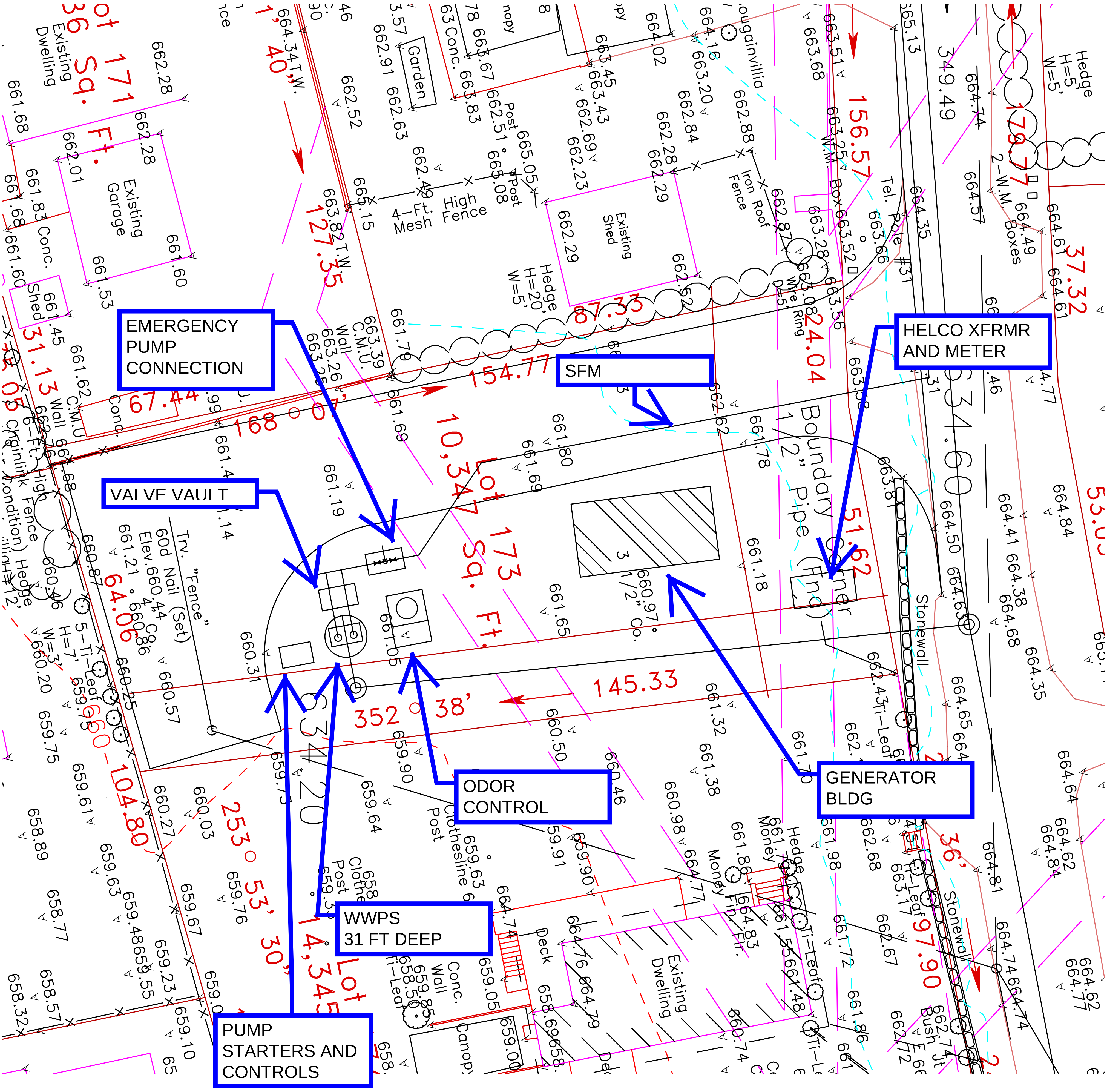


Figure 4
Naalehu Wastewater Collection System
Pump Station Layout on County Parcel

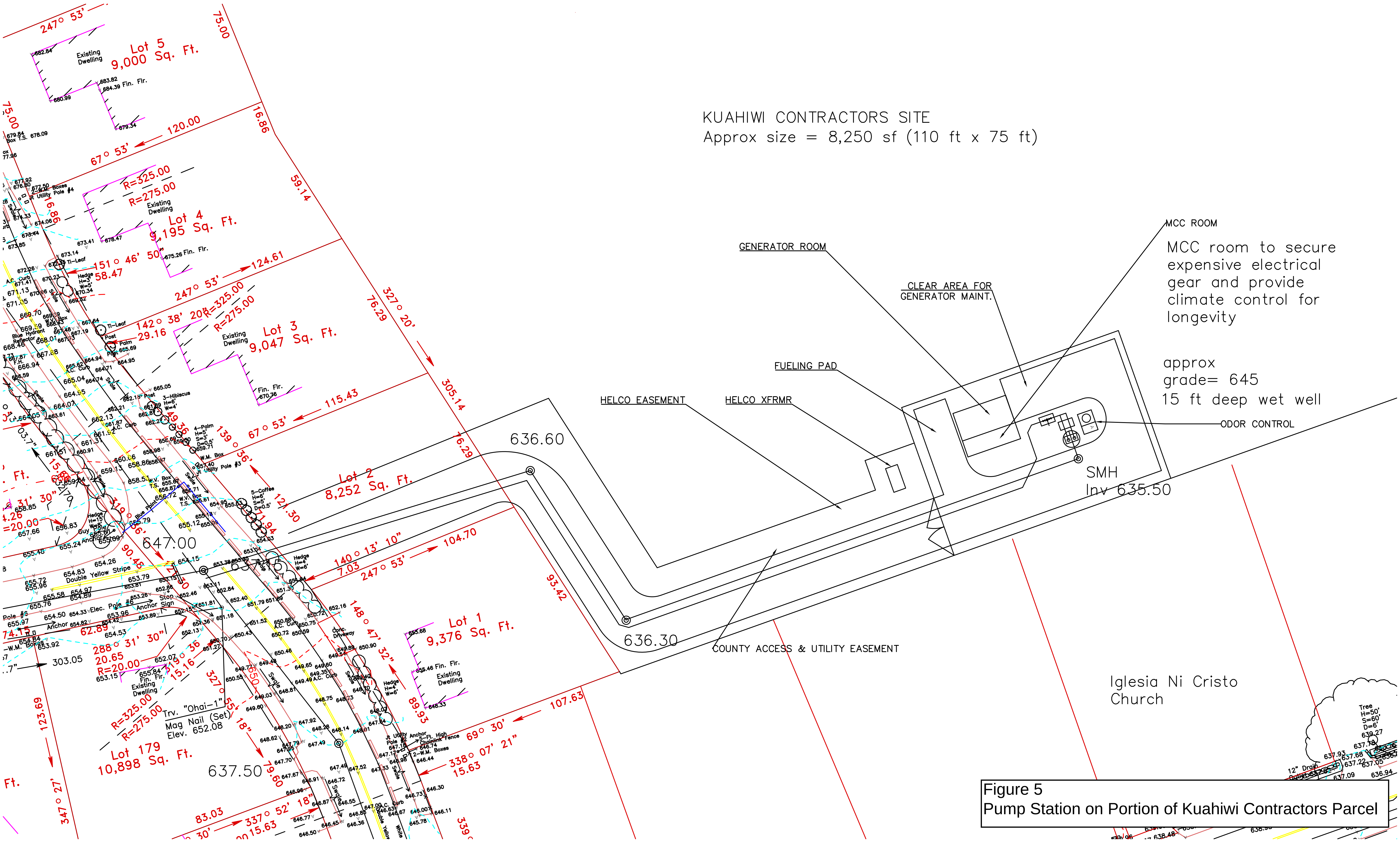


Figure 5
Pump Station on Portion of Kuahiwi Contractors Parcel

Figure 6
Kuahiwi Contractors Site

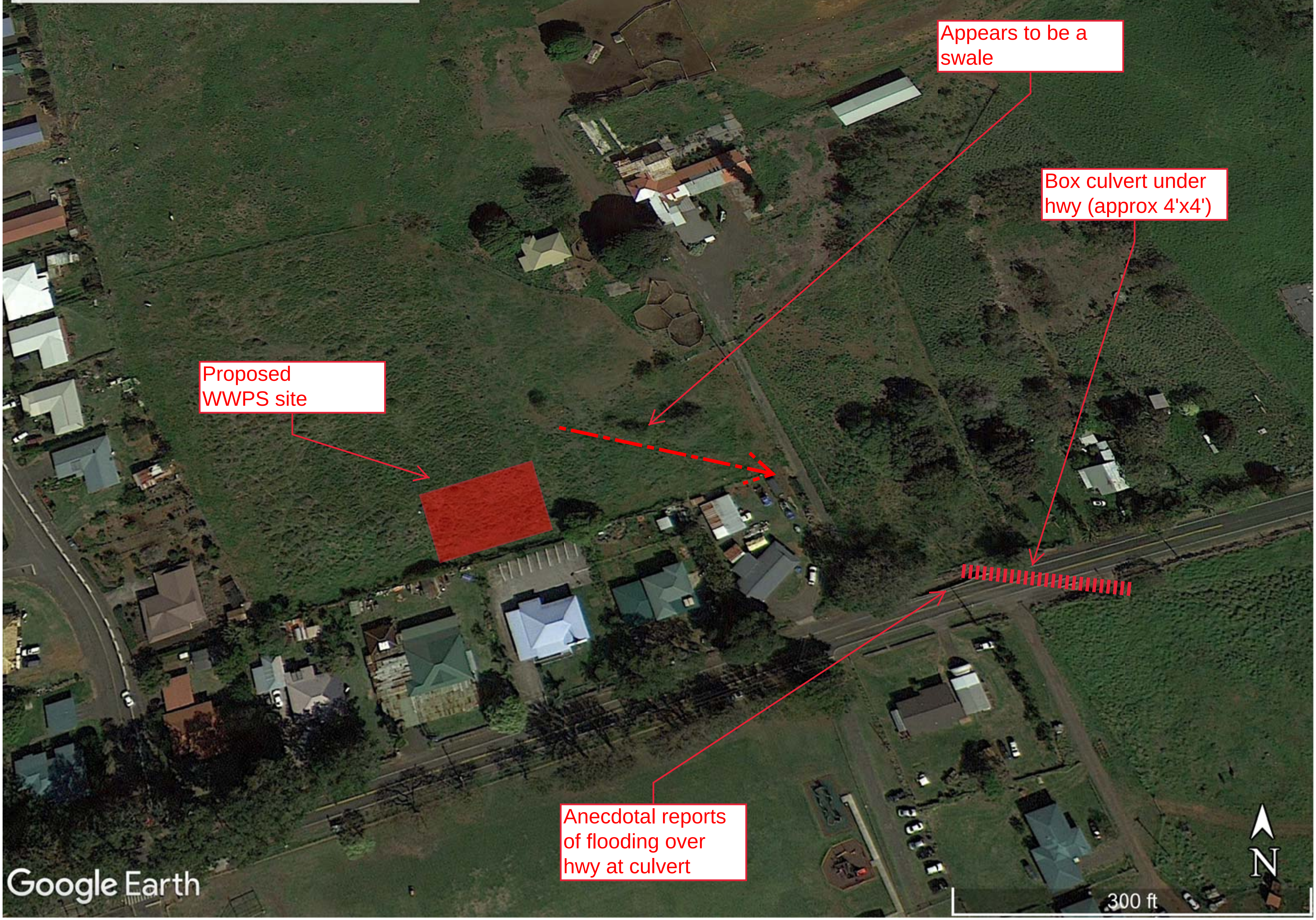
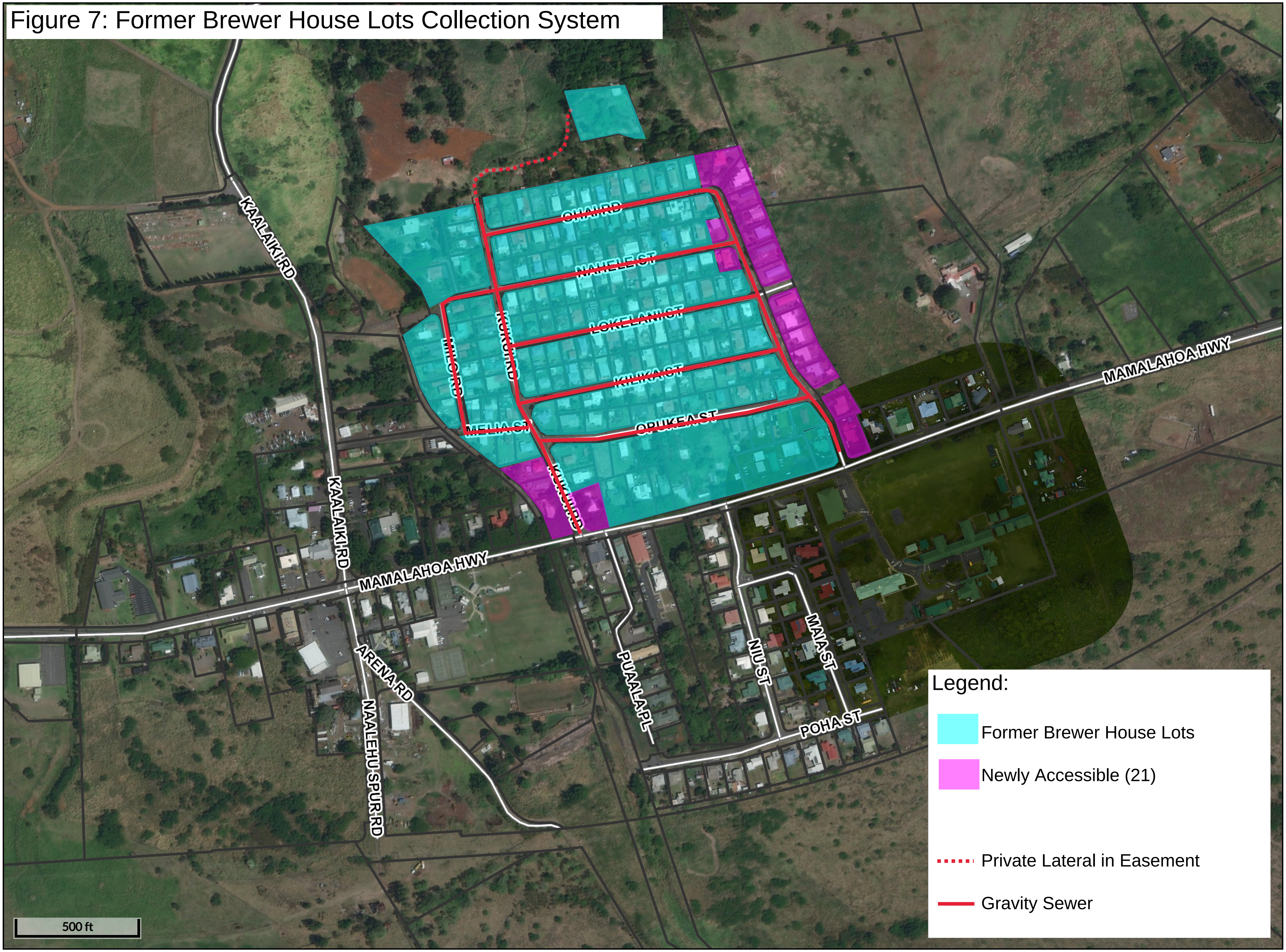


Figure 7: Former Brewer House Lots Collection System



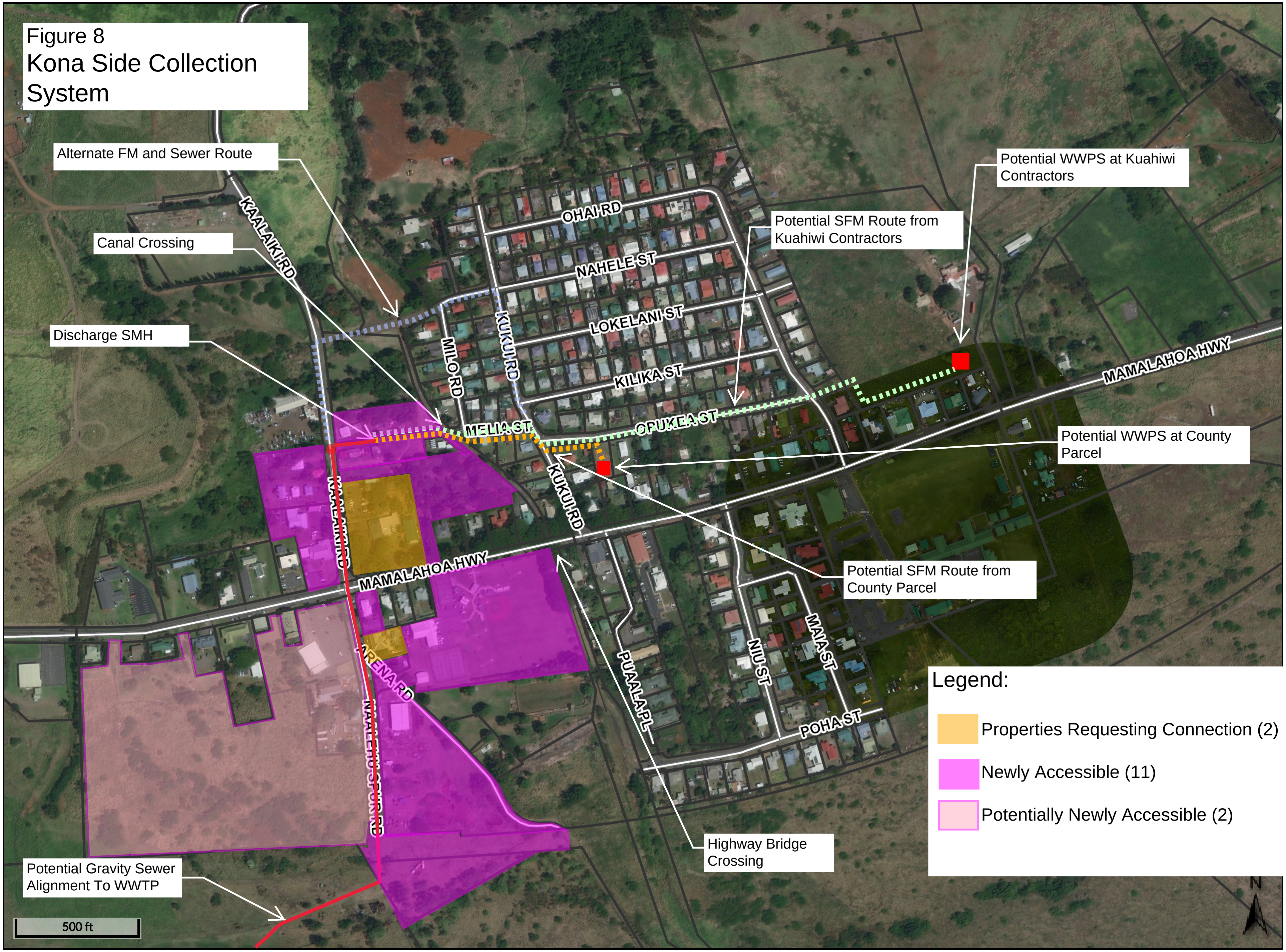


Figure 8
Kona Side Collection System

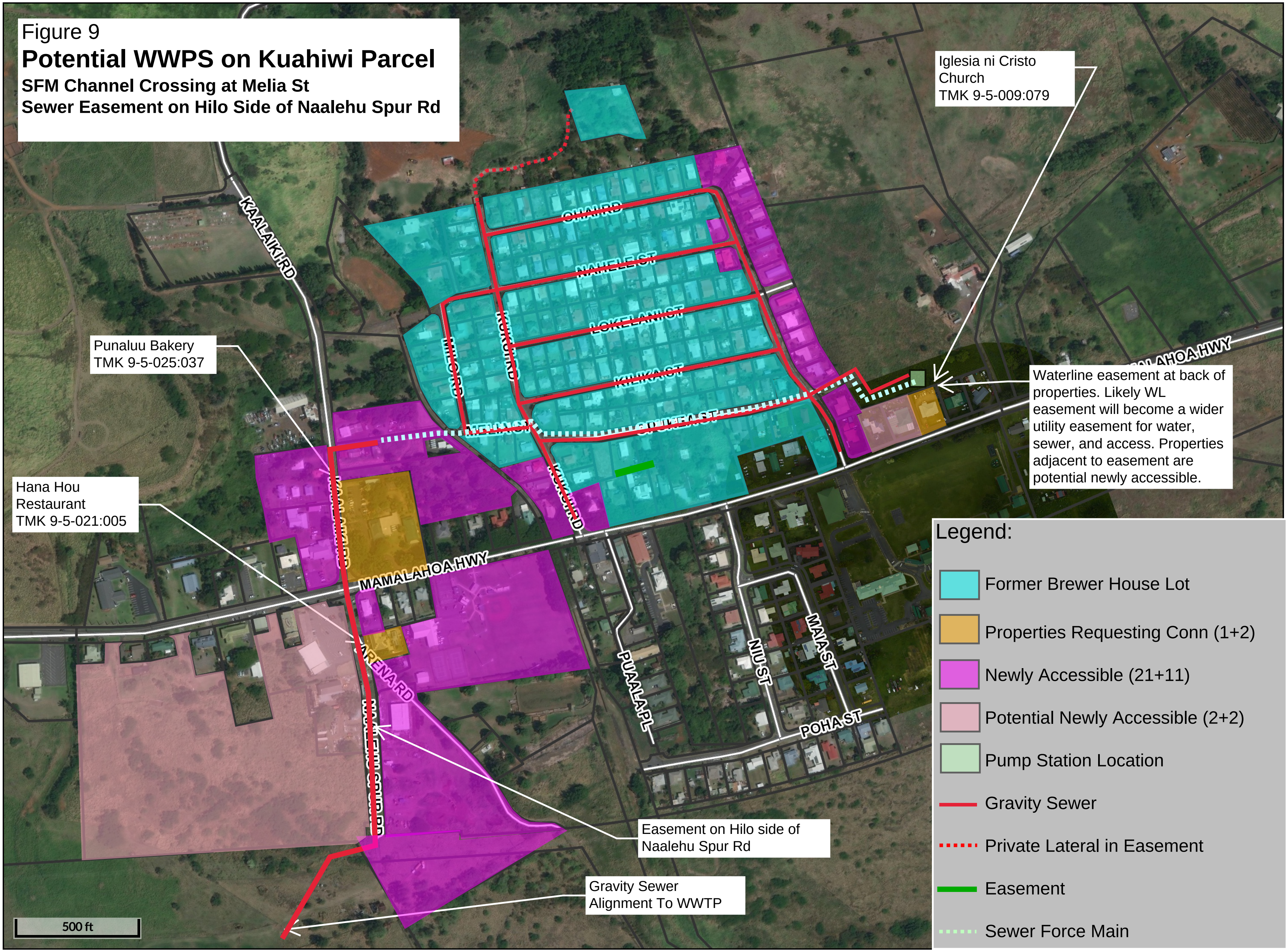


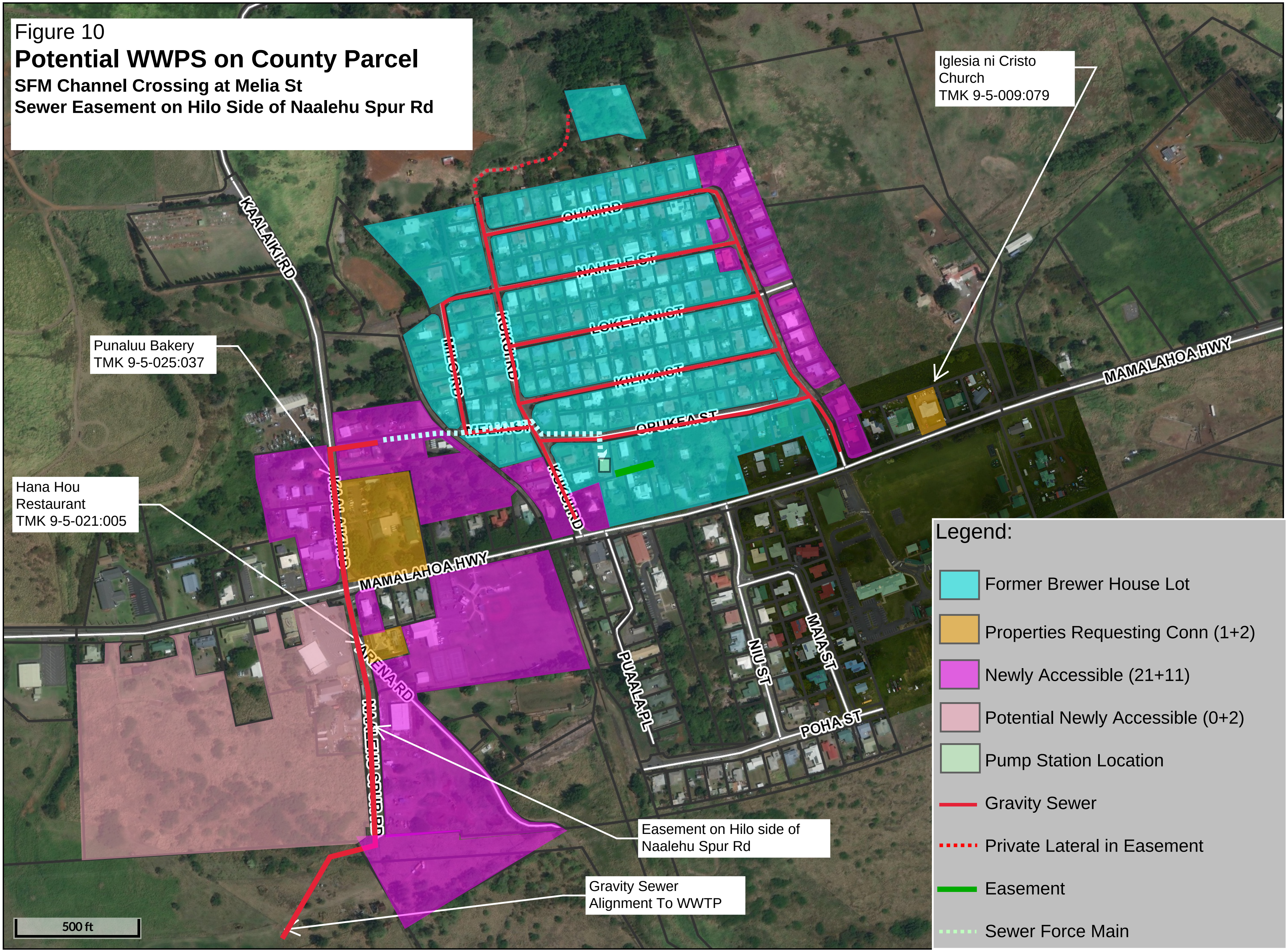
Figure 9
Potential WWPS on Kuahiwi Parcel
SFM Channel Crossing at Melia St
Sewer Easement on Hilo Side of Naalehu Spur Rd
Iglesia ni Cristo Church
TMK 9-5-009:079
Punaluu Bakery
TMK 9-5-025:037
Hana Hou Restaurant
TMK 9-5-021:005
Waterline easement at back of properties. Likely WL easement will become a wider utility easement for water, sewer, and access. Properties adjacent to easement are potential newly accessible.
Easement on Hilo side of Naalehu Spur Rd
Gravity Sewer Alignment To WWTP
KAALAIKI RD
OHAI RD
NAHELE ST
LOKELANI ST
KILIKA ST
MAMALAHOA HWY
ARENA RD
PUAALA PL
NIU ST
MAIA ST
POHA ST
500 ft
Legend:
Former Brewer House Lot
Properties Requesting Conn (1+2)
Newly Accessible (21+11)
Potential Newly Accessible (2+2)
Pump Station Location
Gravity Sewer
Private Lateral in Easement
Easement
Sewer Force Main

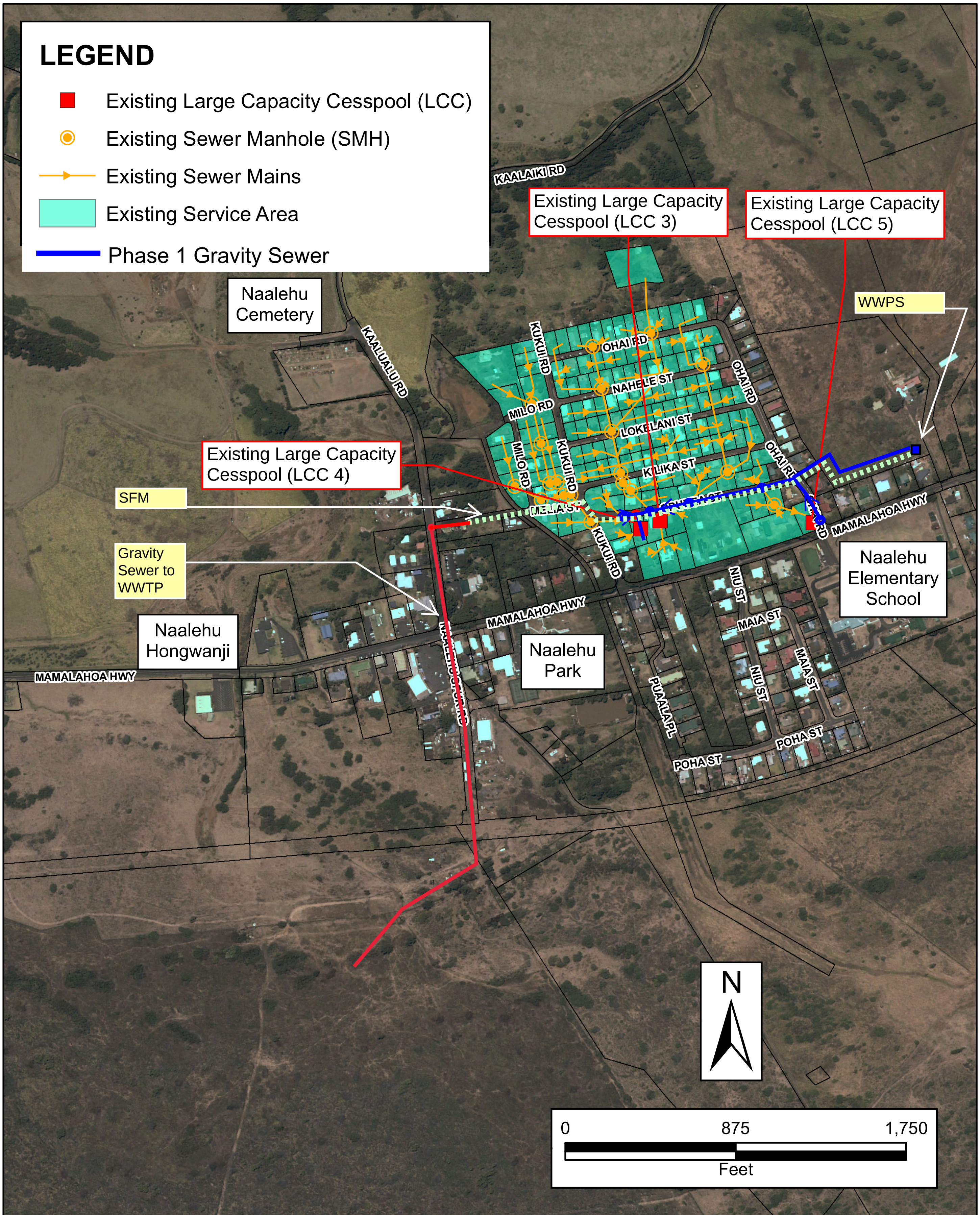
Figure 10

Potential WWPS on County Parcel

SFM Channel Crossing at Melia St

Sewer Easement on Hilo Side of Naalehu Spur Rd





From:

Brown AND Caldwell

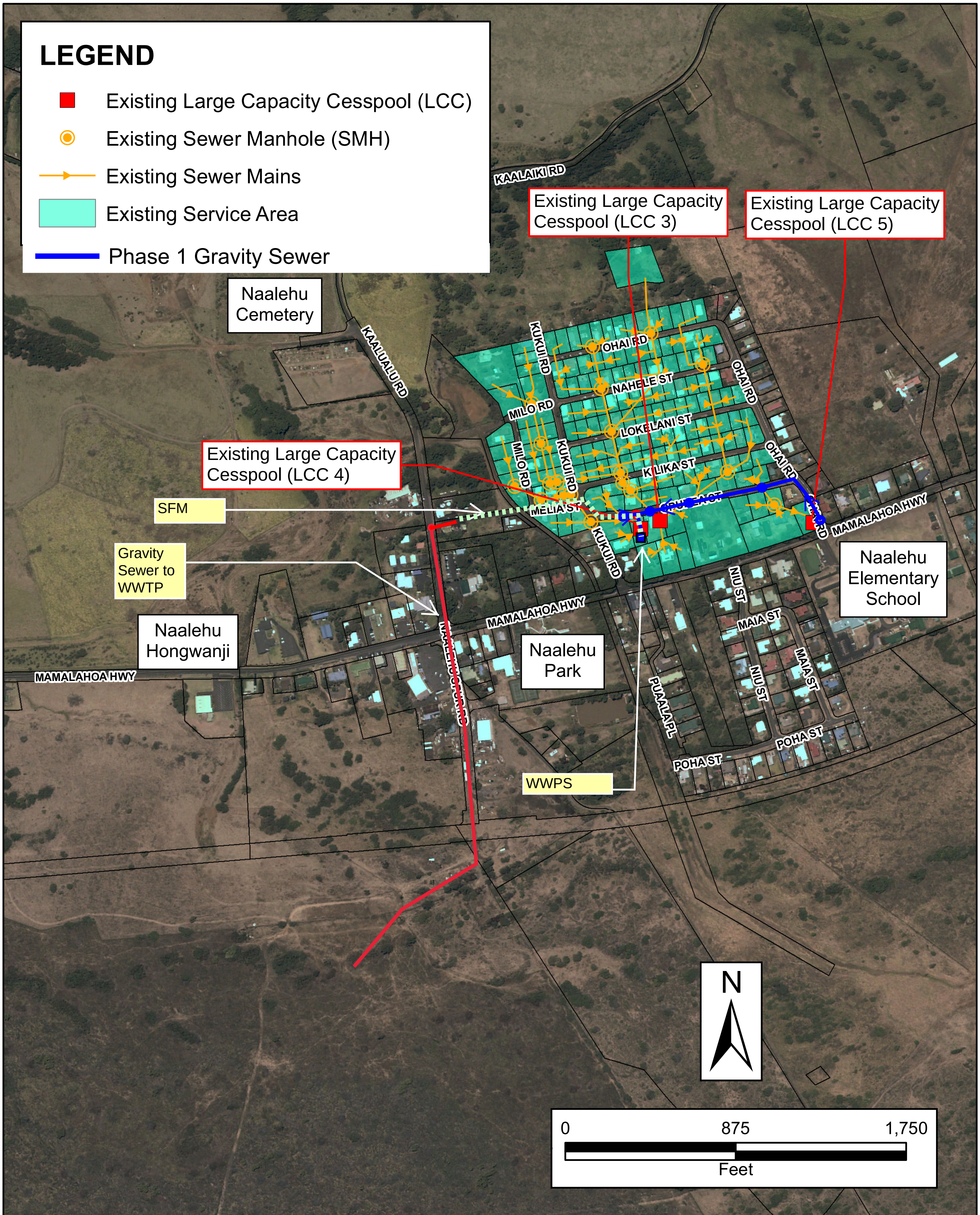
SCALE AS SHOWN

JOB NO.: 151494

Proposed Phase 1
WWPS at Kuahiwi Contractors Site

FIGURE

11



From:

Brown AND Caldwell

SCALE AS SHOWN

JOB NO.: 151494

Proposed Phase 1
WWPS at County Parcel

FIGURE

12

Appendix

Pump Station Cost Estimates

Pump Station Option Summary Table

County Parcel Vertical Shaft Machine	$	7,700,000	*
County Parcel Dig and Shore	$	7,100,000	
Kuahiwi Site Minimum VSM	$	6,100,000	* #
Kuahiwi Site Minimum Dig and Shore	$	4,900,000	#
Kuahiwi Site Large, Dig and Shore	$	5,600,000	#

Notes:

* Includes full cost of mobilizing VSM to Hawaii. If VSM is used for any other purposes (e.g., trenchless pipe installation, deep SMHs etc.) then mobilization cost will get distributed,

Does not include cost of stormwater control that may be required due to flooding issues at SE corner of site

County Parcel
Vertical Shaft Machine (Herrenknecht VSM 8000 at 4.5 m diameter)

DESCRIPTION	QTY	UM	MATERIAL	LABOR	EQUIPMENT	UNIT COST	TOTAL
Mobilization	1	LS				$ 800,000.00	$ 800,000
Clearing and grubbing	10,000	sf		$ 8.00	$ 3.00	$ 11.00	$ 110,000
Machine setup	1	LS				$ 60,000.00	$ 60,000
Wet well shaft	35	ft	$ 15,000.00	$ 12,000.00	$ 3,000.00	$ 30,000.00	$ 1,050,000
Haul Spoils	230	cy		$ 13.00	$ 8.00	$ 21.00	$ 4,830
Septage pumping (keep LCC low)	2	mo	$ 6,000.00	$ 4,800.00	$ 1,500.00	$ 12,300.00	$ 24,600
Septage hauling	2	mo		$ 14,400.00	$ 3,000.00	$ 17,400.00	$ 34,800
Concrete mat foundation	20	cy	$ 750.00	$ 225.00	$ 12.00	$ 987.00	$ 19,740
Wet well structure; 8' dia, 9" wall	31	ft	$ 500.00	$ 100.00	$ 250.00	$ 850.00	$ 26,350
Wet well base	1	ea	$ 1,750.00	$ 800.00	$ 300.00	$ 2,850.00	$ 2,850
Wet well top slab, 12'x12'x10"thk	4	cy	$ 950.00	$ 450.00	$ 85.00	$ 1,485.00	$ 5,940
Wet well hatch, ss and aluminum	1	ea	$ 2,300.00	$ 500.00	$ 90.00	$ 2,890.00	$ 2,890
Wet well interior coating	1000	sf	$ 85.00	$ 20.00	$ 10.00	$ 115.00	$ 115,000
Wet well exterior moisture prot	1045	sf	$ 12.00	$ 10.00	$ 2.00	$ 24.00	$ 25,080
Engineered backfill	140	cy	$ 47.00	$ 42.00	$ 20.00	$ 109.00	$ 15,260
Subm Pumps, 400 gpm, 80 ft TDH	2	ea	$ 3,000.00	$ 1,800.00	$ 180.00	$ 4,980.00	$ 9,960
Disch elbows and guides	2	ea	$ 1,500.00	$ 1,200.00	$ 180.00	$ 2,880.00	$ 5,760
Valve vault structure	1	ea	$ 925.00	$ 250.00	$ 180.00	$ 1,355.00	$ 1,355
Emergency pump conn vault	1	ea	$ 925.00	$ 250.00	$ 180.00	$ 1,355.00	$ 1,355
Gate Valve, 6"	5	ea	$ 910.00	$ 175.00	$ 85.00	$ 1,170.00	$ 5,850
Check valve, 6"	2	ea	$ 1,050.00	$ 175.00	$ 85.00	$ 1,310.00	$ 2,620
Quick connect hose coupling	1	ea	$ 400.00	$ 100.00	$ 85.00	$ 585.00	$ 585
90 bend, FExFE, 6"	3	ea	$ 1,005.00	$ 150.00	$ 85.00	$ 1,240.00	$ 3,720
Tee, FExFE, 6"	1	ea	$ 1,250.00	$ 225.00	$ 85.00	$ 1,560.00	$ 1,560
45 bend, MJ, 6"	4	ea	$ 935.00	$ 150.00	$ 85.00	$ 1,170.00	$ 4,680
Wye, MJ, 6"	2	ea	$ 1,270.00	$ 225.00	$ 85.00	$ 1,580.00	$ 3,160
Plug, MJ, 6"	1	ea	$ 685.00	$ 75.00	$ 85.00	$ 845.00	$ 845
DI pipe, FE, 6"	80	LF	$ 185.00	$ 25.00	$ 15.00	$ 225.00	$ 18,000

DI pipe, MJ, 6"	150	LF	$ 180.00	$ 25.00	$ 15.00	$ 220.00	$ 33,000
Sewer trench excavation, soft	270	cy		$ 25.00	$ 7.00	$ 32.00	$ 8,640
Sewer trench excavation, rock	110	cy		$ 175.00	$ 35.00	$ 210.00	$ 23,100
Haul Spoils	380	cy		$ 13.00	$ 8.00	$ 21.00	$ 7,980
Trench Shoring, 0-10 ft	2200	sf	$ 2.00	$ 5.00	$ 3.00	$ 10.00	$ 22,000
Trench Shoring 10-20 ft	2200	sf	$ 2.50	$ 7.00	$ 3.00	$ 12.50	$ 27,500
Trench Shoring 20-27 ft, rock	1540	sf	$ 3.50	$ 9.00	$ 5.00	$ 17.50	$ 26,950
Septage pumping (keep LCC low)	1.5	mo	$ 6,000.00	$ 4,800.00	$ 1,500.00	$ 12,300.00	$ 18,450
Septage hauling	1.5	mo		$ 14,400.00	$ 3,000.00	$ 17,400.00	$ 26,100
Influent sewer, 14" DI, lined	110	LF	$ 260.00	$ 45.00	$ 25.00	$ 330.00	$ 36,300
Reinf conc jkt	6.3	cy	$ 950.00	$ 350.00	$ 55.00	$ 1,355.00	$ 8,537
CLSM Backfill	285	cy	$ 300.00	$ 5.00	$ 10.00	$ 315.00	$ 89,775
Backfill	41	cy	$ 20.00	$ 42.00	$ 10.00	$ 72.00	$ 2,952
Deep manhole (27 ft)	1	ea	$ 7,500.00	$ 3,300.00	$ 1,500.00	$ 12,300.00	$ 12,300
Manhole interior coating	355	sf	$ 85.00	$ 20.00	$ 10.00	$ 115.00	$ 40,825
SMH exterior moisture protection	410	sf	$ 12.00	$ 10.00	$ 2.00	$ 24.00	$ 9,840
Odor control unit	1	ea	$ 7,150.00	$ 6,000.00	$ 400.00	$ 13,550.00	$ 13,550
Generator Bldg	320	sf	$ 250.00	$ 265.00	$ 10.00	$ 525.00	$ 168,000
Roofing membrane	5.8	sq	$ 3,000.00	$ 2,000.00	$ 170.00	$ 5,170.00	$ 29,986
Block sealant and paint	700	sf	$ 11.00	$ 10.00	$ 4.00	$ 25.00	$ 17,500
Acoustic insulation	700	sf	$ 90.00	$ 20.00	$ 5.00	$ 115.00	$ 80,500
Acoustic louvers	144	sf	$ 245.00	$ 50.00	$ 60.00	$ 355.00	$ 51,120
Generator w/base mounted tank	1	ea	$ 127,000.00	$ 35,000.00	$ 800.00	$ 162,800.00	$ 162,800
Conc inertia pad for generator	6	cy	$ 950.00	$ 450.00	$ 85.00	$ 1,485.00	$ 8,910
Conc fueling pad	3.5	cy	$ 750.00	$ 225.00	$ 12.00	$ 987.00	$ 3,455
Pervious pavement road	5000	sf	$ 50.00	$ 30.00	$ 9.00	$ 89.00	$ 445,000
3' Grade adjustment wall	360	LF	$ 625.00	$ 475.00	$ 5.00	$ 1,105.00	$ 397,800
Fence	426	LF	$ 45.00	$ 35.00	$ 5.00	$ 85.00	$ 36,210
Gate	1	ea	$ 950.00	$ 500.00	$ 30.00	$ 1,480.00	$ 1,480
2" water service	120	LF	$ 75.00	$ 9.00	$ 2.00	$ 86.00	$ 10,320
2" water meter box	1	ea	$ 250.00	$ 200.00	$ 15.00	$ 465.00	$ 465
Water meter	1	LS				$ 1,500.00	$ 1,500

General civil site work	10000	sf				$ 20.00	$ 200,000
HELCO xfrmr and fees	1	LS				$ 125,000.00	$ 125,000
SCADA and security system	1	LS				$ 95,000.00	$ 95,000
Electrical, incl instrumentation	1	LS				$ 377,310.60	$ 377,311
Adder for deeper sewers	2000	lf	$ 105.00	$ 235.00	$ 125.00	$ 465.00	$ 930,000
						SUBTOTAL	$ 5,906,945
						CONTINGENCY (30%)	$ 1,772,083
						TOTAL	$ 7,679,028

County Parcel
Conventional Dig and Shore

DESCRIPTION	QTY	UM	MATERIAL	LABOR	EQUIPMENT	UNIT COST	TOTAL
Mobilization	1	LS				$ -	$ -
Clearing and grubbing	10,000	sf		$ 8.00	$ 3.00	$ 11.00	$ 110,000
Mud walls for water control	75	cy	$ 600.00	$ 200.00	$ 200.00	$ 1,000.00	$ 75,000
Excavation soft material	296	cy		$ 164.71	$ 85.00	$ 249.71	$ 73,914
Excavation rock	80	cy		$ 500.00	$ 175.00	$ 675.00	$ 54,000
Haul Spoils	376	cy		$ 13.00	$ 8.00	$ 21.00	$ 7,896
Septage pumping (keep LCC low)	4	mo	$ 6,000.00	$ 4,800.00	$ 1,500.00	$ 12,300.00	$ 49,200
Septage hauling	4	mo		$ 14,400.00	$ 3,000.00	$ 17,400.00	$ 69,600
Shoring, 0-10 ft	800	sf	$ 45.00	$ 200.00	$ 120.00	$ 365.00	$ 292,000
Shoring 10-20 ft	640	sf	$ 65.00	$ 320.00	$ 200.00	$ 585.00	$ 374,400
Shoring 20-35 ft, rock	720	sf	$ 160.00	$ 320.00	$ 225.00	$ 705.00	$ 507,600
Concrete mat foundation	20	cy	$ 750.00	$ 225.00	$ 12.00	$ 987.00	$ 19,740
Wet well structure; 8' dia, 9" wall	31	ft	$ 500.00	$ 100.00	$ 250.00	$ 850.00	$ 26,350
Wet well base	1	ea	$ 1,750.00	$ 800.00	$ 300.00	$ 2,850.00	$ 2,850
Wet well top slab, 12'x12'x10"thk	4	cy	$ 950.00	$ 450.00	$ 85.00	$ 1,485.00	$ 5,940
Wet well hatch, ss and aluminum	1	ea	$ 2,300.00	$ 500.00	$ 90.00	$ 2,890.00	$ 2,890
Wet well interior coating	1000	sf	$ 85.00	$ 20.00	$ 10.00	$ 115.00	$ 115,000
Wet well exterior moisture prot	1045	sf	$ 12.00	$ 10.00	$ 2.00	$ 24.00	$ 25,080
Engineered backfill	295	cy	$ 47.00	$ 42.00	$ 20.00	$ 109.00	$ 32,155
Subm Pumps, 400 gpm, 80 ft TDH	2	ea	$ 3,000.00	$ 1,800.00	$ 180.00	$ 4,980.00	$ 9,960
Disch elbows and guides	2	ea	$ 1,500.00	$ 1,200.00	$ 180.00	$ 2,880.00	$ 5,760
Valve vault structure	1	ea	$ 925.00	$ 250.00	$ 180.00	$ 1,355.00	$ 1,355
Emergency pump conn vault	1	ea	$ 925.00	$ 250.00	$ 180.00	$ 1,355.00	$ 1,355
Gate Valve, 6"	5	ea	$ 910.00	$ 175.00	$ 85.00	$ 1,170.00	$ 5,850
Check valve, 6"	2	ea	$ 1,050.00	$ 175.00	$ 85.00	$ 1,310.00	$ 2,620
Quick connect hose coupling	1	ea	$ 400.00	$ 100.00	$ 85.00	$ 585.00	$ 585
90 bend, FExFE, 6"	3	ea	$ 1,005.00	$ 150.00	$ 85.00	$ 1,240.00	$ 3,720
Tee, FExFE, 6"	1	ea	$ 1,250.00	$ 225.00	$ 85.00	$ 1,560.00	$ 1,560

45 bend, MJ, 6"	4	ea	$ 935.00	$ 150.00	$ 85.00	$ 1,170.00	$ 4,680
Wye, MJ, 6"	2	ea	$ 1,270.00	$ 225.00	$ 85.00	$ 1,580.00	$ 3,160
Plug, MJ, 6"	1	ea	$ 685.00	$ 75.00	$ 85.00	$ 845.00	$ 845
DI pipe, FE, 6"	80	LF	$ 185.00	$ 25.00	$ 15.00	$ 225.00	$ 18,000
DI pipe, MJ, 6"	150	LF	$ 180.00	$ 25.00	$ 15.00	$ 220.00	$ 33,000
Sewer trench excavation, soft	270	cy		$ 25.00	$ 7.00	$ 32.00	$ 8,640
Sewer trench excavation, rock	110	cy		$ 175.00	$ 35.00	$ 210.00	$ 23,100
Haul Spoils	380	cy		$ 13.00	$ 8.00	$ 21.00	$ 7,980
Trench Shoring, 0-10 ft	2200	sf	$ 2.00	$ 5.00	$ 3.00	$ 10.00	$ 22,000
Trench Shoring 10-20 ft	2200	sf	$ 2.50	$ 7.00	$ 3.00	$ 12.50	$ 27,500
Trench Shoring 20-27 ft, rock	1540	sf	$ 3.50	$ 9.00	$ 5.00	$ 17.50	$ 26,950
Septage pumping (keep LCC low)	1.5	mo	$ 6,000.00	$ 4,800.00	$ 1,500.00	$ 12,300.00	$ 18,450
Septage hauling	1.5	mo		$ 14,400.00	$ 3,000.00	$ 17,400.00	$ 26,100
Influent sewer, 14" DI, lined	110	LF	$ 260.00	$ 45.00	$ 25.00	$ 330.00	$ 36,300
Reinf conc jkt	6.3	cy	$ 950.00	$ 350.00	$ 55.00	$ 1,355.00	$ 8,537
CLSM Backfill	285	cy	$ 300.00	$ 5.00	$ 10.00	$ 315.00	$ 89,775
Backfill	41	cy	$ 20.00	$ 42.00	$ 10.00	$ 72.00	$ 2,952
Deep manhole (27 ft)	1	ea	$ 7,500.00	$ 3,300.00	$ 1,500.00	$ 12,300.00	$ 12,300
Manhole interior coating	355	sf	$ 85.00	$ 20.00	$ 10.00	$ 115.00	$ 40,825
SMH exterior moisture protection	410	sf	$ 12.00	$ 10.00	$ 2.00	$ 24.00	$ 9,840
Odor control unit	1	ea	$ 7,150.00	$ 6,000.00	$ 400.00	$ 13,550.00	$ 13,550
Generator Bldg	320	sf	$ 250.00	$ 265.00	$ 10.00	$ 525.00	$ 168,000
Roofing membrane	5.8	sq	$ 3,000.00	$ 2,000.00	$ 170.00	$ 5,170.00	$ 29,986
Block sealant and paint	700	sf	$ 11.00	$ 10.00	$ 4.00	$ 25.00	$ 17,500
Acoustic insulation	700	sf	$ 90.00	$ 20.00	$ 5.00	$ 115.00	$ 80,500
Acoustic louvers	144	sf	$ 245.00	$ 50.00	$ 60.00	$ 355.00	$ 51,120
Generator w/base mounted tank	1	ea	$ 127,000.00	$ 35,000.00	$ 800.00	$ 162,800.00	$ 162,800
Conc inertia pad for generator	6	cy	$ 950.00	$ 450.00	$ 85.00	$ 1,485.00	$ 8,910
Conc fueling pad	3.5	cy	$ 750.00	$ 225.00	$ 12.00	$ 987.00	$ 3,455
Pervious pavement road	5000	sf	$ 50.00	$ 30.00	$ 9.00	$ 89.00	$ 445,000
3' Grade adjustment wall	360	LF	$ 625.00	$ 475.00	$ 5.00	$ 1,105.00	$ 397,800
Fence	426	LF	$ 45.00	$ 35.00	$ 5.00	$ 85.00	$ 36,210

Gate	1	ea	$ 950.00	$ 500.00	$ 30.00	$ 1,480.00	$ 1,480
2" water service	120	LF	$ 75.00	$ 9.00	$ 2.00	$ 86.00	$ 10,320
2" water meter box	1	ea	$ 250.00	$ 200.00	$ 15.00	$ 465.00	$ 465
Water meter	1	LS				$ 1,500.00	$ 1,500
General civil site work	10000	sf				$ 20.00	$ 200,000
HELCO xfrmr and fees	1	LS				$ 125,000.00	$ 125,000
SCADA and security system	1	LS				$ 95,000.00	$ 95,000
Electrical, incl instrumentation	1	LS				$ 379,844.85	$ 379,845
Adder for deeper sewers	2000	lf	$ 105.00	$ 235.00	$ 125.00	$ 465.00	$ 930,000
						SUBTOTAL	$ 5,455,754
						CONTINGENCY (30%)	$ 1,636,726
						TOTAL	$ 7,092,480

Kuahiwi Contractors Site, Minimum
Vertical Shaft Machine (Herrenknecht VSM 8000 at 4.5 m diameter)

DESCRIPTION	QTY	UM	MATERIAL	LABOR	EQUIPMENT	UNIT COST	TOTAL
Mobilization	1	LS				$800,000.00	$800,000
Clearing and grubbing	8,000	sf		$8.00	$3.00	$11.00	$88,000
Machine setup	1	LS				$60,000.00	$60,000
Wet well shaft	18	ft	$15,000.00	$12,000.00	$3,000.00	$30,000.00	$540,000
Haul Spoils	103	cy		$13.00	$8.00	$21.00	$2,163
Concrete mat foundation	20	cy	$750.00	$225.00	$12.00	$987.00	$19,740
Wet well structure; 8' dia, 9" wall	15	ft	$500.00	$100.00	$250.00	$850.00	$12,750
Wet well base	1	ea	$1,750.00	$800.00	$300.00	$2,850.00	$2,850
Wet well top slab, 12'x12'x10"thk	4	cy	$950.00	$450.00	$85.00	$1,485.00	$5,940
Wet well hatch, ss and aluminum	1	ea	$2,300.00	$500.00	$90.00	$2,890.00	$2,890
Wet well interior coating	500	sf	$85.00	$20.00	$10.00	$115.00	$57,500
Wet well exterior moisture prot	440	sf	$12.00	$10.00	$2.00	$24.00	$10,560
Engineered backfill	64	cy	$47.00	$42.00	$20.00	$109.00	$6,976
Subm Pumps, 400 gpm, 80 ft TDH	2	ea	$3,000.00	$1,800.00	$180.00	$4,980.00	$9,960
Disch elbows and guides	2	ea	$1,500.00	$1,200.00	$180.00	$2,880.00	$5,760
Valve vault structure	1	ea	$925.00	$250.00	$180.00	$1,355.00	$1,355
Emergency pump conn vault	1	ea	$925.00	$250.00	$180.00	$1,355.00	$1,355
Gate Valve, 6"	5	ea	$910.00	$175.00	$85.00	$1,170.00	$5,850
Check valve, 6"	2	ea	$1,050.00	$175.00	$85.00	$1,310.00	$2,620
Quick connect hose coupling	1	ea	$400.00	$100.00	$85.00	$585.00	$585
90 bend, FExFE, 6"	3	ea	$1,005.00	$150.00	$85.00	$1,240.00	$3,720
Tee, FExFE, 6"	1	ea	$1,250.00	$225.00	$85.00	$1,560.00	$1,560
45 bend, MJ, 6"	4	ea	$935.00	$150.00	$85.00	$1,170.00	$4,680
Wye, MJ, 6"	2	ea	$1,270.00	$225.00	$85.00	$1,580.00	$3,160
Plug, MJ, 6"	1	ea	$685.00	$75.00	$85.00	$845.00	$845
DI pipe, FE, 6"	80	LF	$185.00	$25.00	$15.00	$225.00	$18,000
DI pipe, MJ, 6"	150	LF	$180.00	$25.00	$15.00	$220.00	$33,000

Sewer trench excavation, soft	700	cy		$ 25.00	$ 7.00	$ 32.00	$ 22,400
Sewer trench excavation, rock	0	cy		$ 175.00	$ 35.00	$ 210.00	$ -
Haul Spoils	700	cy		$ 13.00	$ 8.00	$ 21.00	$ 14,700
Trench Shoring, 0-10 ft	9400	sf	$ 2.00	$ 5.00	$ 3.00	$ 10.00	$ 94,000
Trench Shoring 10-20 ft	1880	sf	$ 2.50	$ 7.00	$ 3.00	$ 12.50	$ 23,500
Trench Shoring 20-27 ft, rock	0	sf	$ 3.50	$ 9.00	$ 5.00	$ 17.50	$ -
Septage pumping (keep LCC low)	0	mo	$ 6,000.00	$ 4,800.00	$ 1,500.00	$ 12,300.00	$ -
Septage hauling	0	mo		$ 14,400.00	$ 3,000.00	$ 17,400.00	$ -
Influent sewer, 14" DI, lined	470	LF	$ 260.00	$ 45.00	$ 25.00	$ 330.00	$ 155,100
Reinf conc jkt	0	cy	$ 950.00	$ 350.00	$ 55.00	$ 1,355.00	$ -
CLSM Backfill	0	cy	$ 300.00	$ 5.00	$ 10.00	$ 315.00	$ -
Engineered Backfill	41	cy	$ 47.00	$ 42.00	$ 20.00	$ 109.00	$ 4,469
Manhole (10 ft)	3	ea	$ 3,900.00	$ 1,800.00	$ 600.00	$ 6,300.00	$ 18,900
Manhole interior coating	180	sf	$ 85.00	$ 20.00	$ 10.00	$ 115.00	$ 20,700
SMH exterior moisture protection	152	sf	$ 12.00	$ 10.00	$ 2.00	$ 24.00	$ 3,648
SFM trench excavation, soft	0	cy		$ 25.00	$ 7.00	$ 32.00	$ -
Haul Spoils	0	cy		$ 13.00	$ 8.00	$ 21.00	$ -
Trench Shoring, 0-10 ft	0	sf	$ 2.00	$ 5.00	$ 3.00	$ 10.00	$ -
SFM, 6" DI, coated and lined	0	LF	$ 180.00	$ 25.00	$ 15.00	$ 220.00	$ -
Engineered Backfill	0	cy	$ 47.00	$ 42.00	$ 20.00	$ 109.00	$ -
Odor control unit	1	ea	$7,150.00	$6,000.00	$400.00	$13,550.00	$13,550
Generator Bldg	520	sf	$250.00	$265.00	$10.00	$525.00	$273,000
Roofing membrane	8.4	sq	$3,000.00	$2,000.00	$170.00	$5,170.00	$43,428
Block sealant and paint	1105	sf	$11.00	$10.00	$4.00	$25.00	$27,625
Acoustic insulation	700	sf	$90.00	$20.00	$5.00	$115.00	$80,500
Acoustic louvers	144	sf	$245.00	$50.00	$60.00	$355.00	$51,120
Generator w/base mounted tank	1	ea	$127,000.00	$35,000.00	$800.00	$162,800.00	$162,800
Conc inertia pad for generator	6	cy	$950.00	$450.00	$85.00	$1,485.00	$8,910
Conc fueling pad	3.5	cy	$750.00	$225.00	$12.00	$987.00	$3,455
Pervious pavement road	5000	sf	$ 50.00	$ 30.00	$ 9.00	$89.00	$445,000
3' Grade adjustment wall	370	LF	$ 625.00	$ 475.00	$ 5.00	$1,105.00	$408,850
Fence	360	LF	$45.00	$35.00	$5.00	$85.00	$30,600

Gate	1	ea	$950.00	$500.00	$30.00	$1,480.00	$1,480
6" water service	500	LF	$ 120.00	$ 25.00	$ 15.00	$ 160.00	$ 80,000
6" detector check in vault	1	ea	$ 8,000.00	$ 6,000.00	$ 1,000.00	$ 15,000.00	$ 15,000
Fire standpipe	1	ea	$ 1,500.00	$ 900.00	$ 20.00	$ 2,420.00	$ 2,420
2" water service	120	LF	$ 75.00	$ 9.00	$ 2.00	$ 86.00	$ 10,320
2" water meter box	1	ea	$ 250.00	$ 200.00	$ 15.00	$ 465.00	$ 465
Water meter	1	LS				$ 1,500.00	$ 1,500
General civil site work	8000	sf				$20.00	$160,000
HELCO xfrmr and fees	1	LS				$225,000.00	$225,000
SCADA and security system	1	LS				$95,000.00	$95,000
Electrical, incl instrumentation	1	LS				$406,364.33	$406,364
Additional gravel road	17,000	sf	$1.75	$1.60	$0.75	$4.10	$69,700
						SUBTOTAL	$4,675,323
						CONTINGENCY (30%)	$1,402,597
						TOTAL	$6,077,920

Kuahiwi Contractors Site, Minimum
Conventional Dig and Shore

DESCRIPTION	QTY	UM	MATERIAL	LABOR	EQUIPMENT	UNIT COST	TOTAL
Mobilization	1	LS				$ -	$ -
Clearing and grubbing	8,000	sf		$ 8.00	$ 3.00	$ 11.00	$ 88,000
Excavation soft material	175	cy		$ 164.71	$ 85.00	$ 249.71	$ 43,699
Excavation rock	0	cy		$ 500.00	$ 175.00	$ 675.00	$ -
Haul Spoils	175	cy		$ 13.00	$ 8.00	$ 21.00	$ 3,675
Shoring, 0-10 ft	640	sf	$ 45.00	$ 200.00	$ 120.00	$ 365.00	$ 233,600
Shoring 10-18 ft	384	sf	$ 65.00	$ 320.00	$ 200.00	$ 585.00	$ 224,640
Shoring 20-35 ft, rock	0	sf	$ 160.00	$ 320.00	$ 225.00	$ 705.00	$ -
Concrete mat foundation	20	cy	$ 750.00	$ 225.00	$ 12.00	$ 987.00	$ 19,740
Wet well structure; 8' dia, 9" wall	15	ft	$ 500.00	$ 100.00	$ 250.00	$ 850.00	$ 12,750
Wet well base	1	ea	$ 1,750.00	$ 800.00	$ 300.00	$ 2,850.00	$ 2,850
Wet well top slab, 12'x12'x10"thk	4	cy	$ 950.00	$ 450.00	$ 85.00	$ 1,485.00	$ 5,940
Wet well hatch, ss and aluminum	1	ea	$ 2,300.00	$ 500.00	$ 90.00	$ 2,890.00	$ 2,890
Wet well interior coating	480	sf	$ 85.00	$ 20.00	$ 10.00	$ 115.00	$ 55,200
Wet well exterior moisture prot	445	sf	$ 12.00	$ 10.00	$ 2.00	$ 24.00	$ 10,680
Engineered backfill	136	cy	$ 47.00	$ 42.00	$ 20.00	$ 109.00	$ 14,824
Subm Pumps, 400 gpm, 80 ft TDH	2	ea	$ 3,000.00	$ 1,800.00	$ 180.00	$ 4,980.00	$ 9,960
Disch elbows and guides	2	ea	$ 1,500.00	$ 1,200.00	$ 180.00	$ 2,880.00	$ 5,760
Valve vault structure	1	ea	$ 925.00	$ 250.00	$ 180.00	$ 1,355.00	$ 1,355
Emergency pump conn vault	1	ea	$ 925.00	$ 250.00	$ 180.00	$ 1,355.00	$ 1,355
Gate Valve, 6"	5	ea	$ 910.00	$ 175.00	$ 85.00	$ 1,170.00	$ 5,850
Check valve, 6"	2	ea	$ 1,050.00	$ 175.00	$ 85.00	$ 1,310.00	$ 2,620
Quick connect hose coupling	1	ea	$ 400.00	$ 100.00	$ 85.00	$ 585.00	$ 585
90 bend, FExFE, 6"	3	ea	$ 1,005.00	$ 150.00	$ 85.00	$ 1,240.00	$ 3,720
Tee, FExFE, 6"	1	ea	$ 1,250.00	$ 225.00	$ 85.00	$ 1,560.00	$ 1,560
45 bend, MJ, 6"	4	ea	$ 935.00	$ 150.00	$ 85.00	$ 1,170.00	$ 4,680
Wye, MJ, 6"	2	ea	$ 1,270.00	$ 225.00	$ 85.00	$ 1,580.00	$ 3,160
Plug, MJ, 6"	1	ea	$ 685.00	$ 75.00	$ 85.00	$ 845.00	$ 845

DI pipe, FE, 6"	80	LF	$ 185.00	$ 25.00	$ 15.00	$ 225.00	$ 18,000
DI pipe, MJ, 6"	150	LF	$ 180.00	$ 25.00	$ 15.00	$ 220.00	$ 33,000
Sewer trench excavation, soft	700	cy		$ 25.00	$ 7.00	$ 32.00	$ 22,400
Sewer trench excavation, rock	0	cy		$ 175.00	$ 35.00	$ 210.00	$ -
Haul Spoils	700	cy		$ 13.00	$ 8.00	$ 21.00	$ 14,700
Trench Shoring, 0-10 ft	9400	sf	$ 2.00	$ 5.00	$ 3.00	$ 10.00	$ 94,000
Trench Shoring 10-20 ft	1880	sf	$ 2.50	$ 7.00	$ 3.00	$ 12.50	$ 23,500
Trench Shoring 20-27 ft, rock	0	sf	$ 3.50	$ 9.00	$ 5.00	$ 17.50	$ -
Septage pumping (keep LCC low)	0	mo	$ 6,000.00	$ 4,800.00	$ 1,500.00	$ 12,300.00	$ -
Septage hauling	0	mo		$ 14,400.00	$ 3,000.00	$ 17,400.00	$ -
Influent sewer, 14" DI, lined	470	LF	$ 260.00	$ 45.00	$ 25.00	$ 330.00	$ 155,100
Reinf conc jkt	0	cy	$ 950.00	$ 350.00	$ 55.00	$ 1,355.00	$ -
CLSM Backfill	0	cy	$ 300.00	$ 5.00	$ 10.00	$ 315.00	$ -
Engineered Backfill	41	cy	$ 47.00	$ 42.00	$ 20.00	$ 109.00	$ 4,469
Manhole (10 ft)	3	ea	$ 3,900.00	$ 1,800.00	$ 600.00	$ 6,300.00	$ 18,900
Manhole interior coating	180	sf	$ 85.00	$ 20.00	$ 10.00	$ 115.00	$ 20,700
SMH exterior moisture protection	152	sf	$ 12.00	$ 10.00	$ 2.00	$ 24.00	$ 3,648
SFM trench excavation, soft	0	cy		$ 25.00	$ 7.00	$ 32.00	$ -
Haul Spoils	0	cy		$ 13.00	$ 8.00	$ 21.00	$ -
Trench Shoring, 0-10 ft	0	sf	$ 2.00	$ 5.00	$ 3.00	$ 10.00	$ -
SFM, 6" DI, coated and lined	0	LF	$ 180.00	$ 25.00	$ 15.00	$ 220.00	$ -
Engineered Backfill	0	cy	$ 47.00	$ 42.00	$ 20.00	$ 109.00	$ -
Odor control unit	1	ea	$ 7,150.00	$ 6,000.00	$ 400.00	$ 13,550.00	$ 13,550
Generator Bldg	520	sf	$ 250.00	$ 265.00	$ 10.00	$ 525.00	$ 273,000
Roofing membrane	8.4	sq	$ 3,000.00	$ 2,000.00	$ 170.00	$ 5,170.00	$ 43,428
Block sealant and paint	1105	sf	$ 11.00	$ 10.00	$ 4.00	$ 25.00	$ 27,625
Acoustic insulation	700	sf	$ 90.00	$ 20.00	$ 5.00	$ 115.00	$ 80,500
Acoustic louvers	144	sf	$ 245.00	$ 50.00	$ 60.00	$ 355.00	$ 51,120
Generator w/base mounted tank	1	ea	$ 127,000.00	$ 35,000.00	$ 800.00	$ 162,800.00	$ 162,800
Conc inertia pad for generator	6	cy	$ 950.00	$ 450.00	$ 85.00	$ 1,485.00	$ 8,910
Conc fueling pad	3.5	cy	$ 750.00	$ 225.00	$ 12.00	$ 987.00	$ 3,455

Pervious pavement road	5000	sf	$ 50.00	$ 30.00	$ 9.00	$89.00	$445,000
3' Grade adjustment wall	370	LF	$ 625.00	$ 475.00	$ 5.00	$1,105.00	$408,850
Fence	360	LF	$ 45.00	$ 35.00	$ 5.00	$ 85.00	$ 30,600
Gate	1	ea	$ 950.00	$ 500.00	$ 30.00	$ 1,480.00	$ 1,480
6" water service	500	LF	$ 120.00	$ 25.00	$ 15.00	$ 160.00	$ 80,000
6" detector check in vault	1	ea	$ 8,000.00	$ 6,000.00	$ 1,000.00	$ 15,000.00	$ 15,000
Fire standpipe	1	ea	$ 1,500.00	$ 900.00	$ 20.00	$ 2,420.00	$ 2,420
2" water service	120	LF	$ 75.00	$ 9.00	$ 2.00	$ 86.00	$ 10,320
2" water meter box	1	ea	$ 250.00	$ 200.00	$ 15.00	$ 465.00	$ 465
Water meter	1	LS				$ 1,500.00	$ 1,500
General civil site work	8000	sf				$ 20.00	$ 160,000
HELCO xfrmr and fees	1	LS				$ 225,000.00	$ 225,000
SCADA and security system	1	LS				$ 95,000.00	$ 95,000
Electrical, incl instrumentation	1	LS				$ 407,214.53	$ 407,215
Additional gravel road	17,000	sf	$ 1.75	$ 1.60	$ 0.75	$ 4.10	$ 69,700
						SUBTOTAL	$ 3,785,292
						CONTINGENCY (30%)	$ 1,135,588
						TOTAL	$ 4,920,880

Kuahiwi Contractors Site, Large
Conventional Dig and Shore

DESCRIPTION	QTY	UM	MATERIAL	LABOR	EQUIPMENT	UNIT COST	TOTAL
Mobilization	1	LS				$0.00	$ -
Clearing and grubbing	8,000	sf		$8.00	$3.00	$11.00	$ 88,000
Excavation soft material	175	cy		$164.71	$85.00	$249.71	$ 43,699
Excavation rock	0	cy		$500.00	$175.00	$675.00	$ -
Haul Spoils	175	cy		$13.00	$8.00	$21.00	$ 3,675
Shoring, 0-10 ft	640	sf	$45.00	$200.00	$120.00	$365.00	$ 233,600
Shoring 10-18 ft	384	sf	$65.00	$320.00	$200.00	$585.00	$ 224,640
Shoring 20-35 ft, rock	0	sf	$160.00	$320.00	$225.00	$705.00	$ -
Concrete mat foundation	20	cy	$750.00	$225.00	$12.00	$987.00	$ 19,740
Wet well structure; 8' dia, 9" wall	15	ft	$500.00	$100.00	$250.00	$850.00	$ 12,750
Wet well base	1	ea	$1,750.00	$800.00	$300.00	$2,850.00	$ 2,850
Wet well top slab, 12'x12'x10"thk	4	cy	$950.00	$450.00	$85.00	$1,485.00	$ 5,940
Wet well hatch, ss and aluminum	1	ea	$2,300.00	$500.00	$90.00	$2,890.00	$ 2,890
Wet well interior coating	480	sf	$85.00	$20.00	$10.00	$115.00	$ 55,200
Wet well exterior moisture prot	445	sf	$12.00	$10.00	$2.00	$24.00	$ 10,680
Engineered backfill	136	cy	$47.00	$42.00	$20.00	$109.00	$ 14,824
Subm Pumps, 400 gpm, 80 ft TDH	2	ea	$3,000.00	$1,800.00	$180.00	$4,980.00	$ 9,960
Disch elbows and guides	2	ea	$1,500.00	$1,200.00	$180.00	$2,880.00	$ 5,760
Valve vault structure	1	ea	$925.00	$250.00	$180.00	$1,355.00	$ 1,355
Emergency pump conn vault	1	ea	$925.00	$250.00	$180.00	$1,355.00	$ 1,355
Gate Valve, 6"	5	ea	$910.00	$175.00	$85.00	$1,170.00	$ 5,850
Check valve, 6"	2	ea	$1,050.00	$175.00	$85.00	$1,310.00	$ 2,620
Quick connect hose coupling	1	ea	$400.00	$100.00	$85.00	$585.00	$ 585
90 bend, FExFE, 6"	3	ea	$1,005.00	$150.00	$85.00	$1,240.00	$ 3,720
Tee, FExFE, 6"	1	ea	$1,250.00	$225.00	$85.00	$1,560.00	$ 1,560
45 bend, MJ, 6"	4	ea	$935.00	$150.00	$85.00	$1,170.00	$ 4,680
Wye, MJ, 6"	2	ea	$1,270.00	$225.00	$85.00	$1,580.00	$ 3,160
Plug, MJ, 6"	1	ea	$685.00	$75.00	$85.00	$845.00	$ 845
DI pipe, FE, 6"	80	LF	$185.00	$25.00	$15.00	$225.00	$ 18,000

DI pipe, MJ, 6"	150	LF	$180.00	$25.00	$15.00	$220.00	$ 33,000
Sewer trench excavation, soft	700	cy		$ 25.00	$ 7.00	$ 32.00	$ 22,400
Sewer trench excavation, rock	0	cy		$ 175.00	$ 35.00	$ 210.00	$ -
Haul Spoils	700	cy		$ 13.00	$ 8.00	$ 21.00	$ 14,700
Trench Shoring, 0-10 ft	9400	sf	$ 2.00	$ 5.00	$ 3.00	$ 10.00	$ 94,000
Trench Shoring 10-20 ft	1880	sf	$ 2.50	$ 7.00	$ 3.00	$ 12.50	$ 23,500
Trench Shoring 20-27 ft, rock	0	sf	$ 3.50	$ 9.00	$ 5.00	$ 17.50	$ -
Septage pumping (keep LCC low)	0	mo	$ 6,000.00	$ 4,800.00	$ 1,500.00	$ 12,300.00	$ -
Septage hauling	0	mo		$ 14,400.00	$ 3,000.00	$ 17,400.00	$ -
Influent sewer, 14" DI, lined	470	LF	$ 260.00	$ 45.00	$ 25.00	$ 330.00	$ 155,100
Reinf conc jkt	0	cy	$ 950.00	$ 350.00	$ 55.00	$ 1,355.00	$ -
CLSM Backfill	0	cy	$ 300.00	$ 5.00	$ 10.00	$ 315.00	$ -
Engineered Backfill	41	cy	$ 47.00	$ 42.00	$ 20.00	$ 109.00	$ 4,469
Manhole (10 ft)	3	ea	$ 3,900.00	$ 1,800.00	$ 600.00	$ 6,300.00	$ 18,900
Manhole interior coating	180	sf	$ 85.00	$ 20.00	$ 10.00	$ 115.00	$ 20,700
SMH exterior moisture protection	152	sf	$ 12.00	$ 10.00	$ 2.00	$ 24.00	$ 3,648
SFM trench excavation, soft	0	cy		$ 25.00	$ 7.00	$ 32.00	$ -
Haul Spoils	0	cy		$ 13.00	$ 8.00	$ 21.00	$ -
Trench Shoring, 0-10 ft	0	sf	$ 2.00	$ 5.00	$ 3.00	$ 10.00	$ -
SFM, 6" DI, coated and lined	0	LF	$ 180.00	$ 25.00	$ 15.00	$ 220.00	$ -
Engineered Backfill	0	cy	$ 47.00	$ 42.00	$ 20.00	$ 109.00	$ -
Odor control unit	1	ea	$7,150.00	$6,000.00	$400.00	$13,550.00	$ 13,550
Generator Bldg	520	sf	$250.00	$265.00	$10.00	$525.00	$ 273,000
Roofing membrane	8.4	sq	$3,000.00	$2,000.00	$170.00	$5,170.00	$ 43,428
Block sealant and paint	1105	sf	$11.00	$10.00	$4.00	$25.00	$ 27,625
Acoustic insulation	700	sf	$90.00	$20.00	$5.00	$115.00	$ 80,500
Acoustic louvers	144	sf	$245.00	$50.00	$60.00	$355.00	$ 51,120
Generator w/base mounted tank	1	ea	$127,000.00	$35,000.00	$800.00	$162,800.00	$ 162,800
Conc inertia pad for generator	6	cy	$950.00	$450.00	$85.00	$1,485.00	$ 8,910
Conc fueling pad	3.5	cy	$750.00	$225.00	$12.00	$987.00	$ 3,455
Pervious pavement road	7000	sf	$ 50.00	$ 30.00	$ 9.00	$89.00	$623,000

3' Grade adjustment wall	484	LF	$ 625.00	$ 475.00	$ 5.00	$1,105.00	$534,820
Fence	480	LF	$45.00	$35.00	$5.00	$85.00	$ 40,800
Gate	1	ea	$950.00	$500.00	$30.00	$1,480.00	$ 1,480
6" water service	500	LF	$ 120.00	$ 25.00	$ 15.00	$ 160.00	$ 80,000
6" detector check in vault	1	ea	$ 8,000.00	$ 6,000.00	$ 1,000.00	$ 15,000.00	$ 15,000
Fire standpipe	1	ea	$ 1,500.00	$ 900.00	$ 20.00	$ 2,420.00	$ 2,420
2" water service	120	LF	$ 75.00	$ 9.00	$ 2.00	$ 86.00	$ 10,320
2" water meter box	1	ea	$ 250.00	$ 200.00	$ 15.00	$ 465.00	$ 465
Water meter	1	LS				$ 1,500.00	$ 1,500
General civil site work	15000	sf				$20.00	$ 300,000
HELCO xfrmr and fees	1	LS				$225,000.00	$ 225,000
SCADA and security system	1	LS				$95,000.00	$ 95,000
Electrical, incl instrumentation	1	LS				$475,340.03	$ 475,340
Additional gravel road	17,000	sf	$1.75	$1.60	$0.75	$4.10	$ 69,700
						SUBTOTAL	$ 4,307,588
						CONTINGENCY (30%)	$ 1,292,276
						TOTAL	$ 5,599,864

PART B: IWS Approach

PART B

Nāʻālehu Individual Wastewater System Preliminary Engineering Report

Prepared for
Brown and Caldwell
&
County of Hawaiʻi, Department of Environmental Management

August 2023

THIS WORK (PART B) WAS PREPARED BY ME OR UNDER MY SUPERVISION

Yen Wen Fang

Yen Wen Fang | April 30, 2024
Expiration Date of the License

ENGINEERING PARTNERS

Part B - IWS Approach

Table of Contents

List of Figures

List of Tables

Individual Wastewater System (IWS)

1. Introduction

In Hawaiʻi, Individual Wastewater System (IWS) is defined by the Department of Health (DOH) as a wastewater system for an individual property that receives less than 1,000 gallons per day of wastewater flow or serves five bedrooms or less. An IWS can be a cesspool, a traditional septic system, an aerobic treatment unit (ATU), or other means of treatment achieving National Sanitation Foundation (NSF) 40 quality effluent. Since 2016, the State of Hawaiʻi banned the use of new cesspools due to their poor treatment quality. Both ATUs and NSF 40 type systems are considered secondary treatment and will result in higher operating and maintenance requirements.

2. Different Types of IWS

2.1 Septic System

A typical IWS system consists of three main components: the septic tank, distribution box, and leach field (also known as absorption bed or drain field). The septic tank is typically buried underground and receives wastewater from the building's plumbing fixtures (Figure 1.1). From the septic tank which has one or two compartments, effluent flows to a distribution box, which evenly distributes the wastewater to the leach field. The leach field or absorption bed is a network of perforated pipes or chambers laid in trenches or beds, buried in the soil. A seepage pit, which is a vertical excavation typically lined with concrete perforated rings, can be used in place of an absorption bed when a variance application has been approved by the DOH. Typical IWS layout with an absorption bed, seepage pit etc. is outlined in Appendix F.

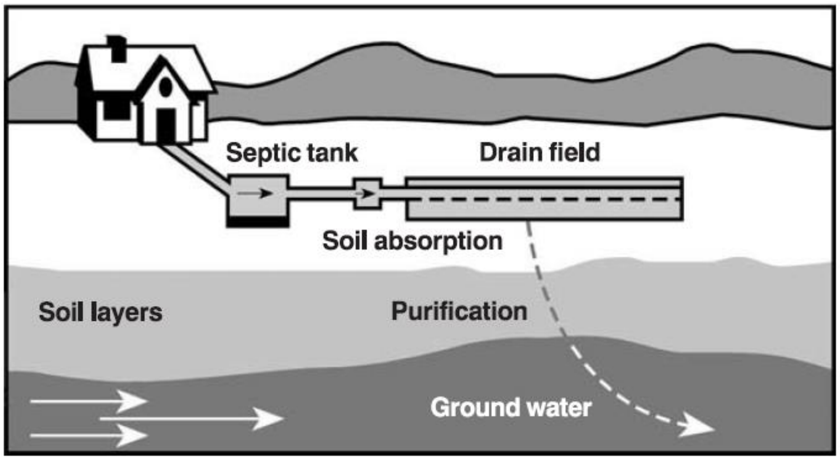


Figure 1.1: Typical IWS flow diagram.

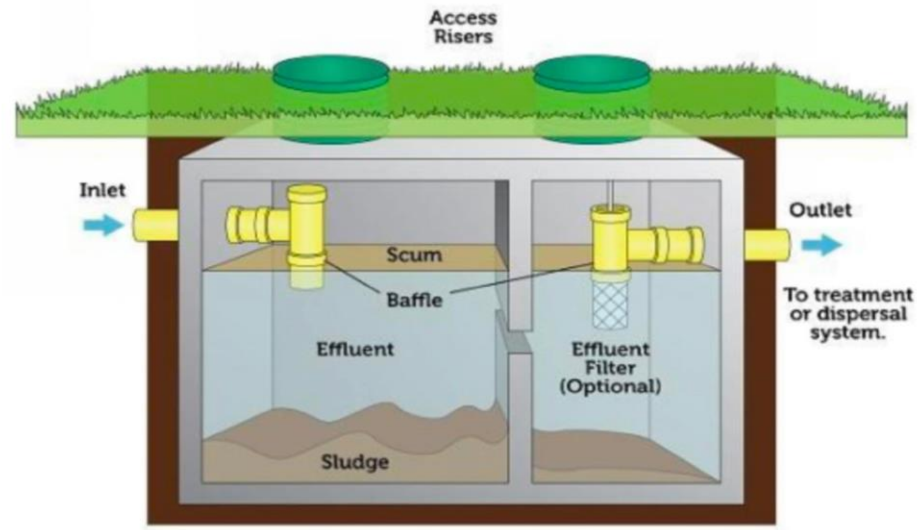


Figure 1.2: Side-view of a typical two-chambered septic tank (Carollo, 2021).

The underground septic tank is typically made of concrete, fiberglass, or plastic, and serves as a primary settling chamber to remove bulk solids and floatable substances such as oil and grease. Septic tanks usually have an inlet pipe where wastewater enters and an outlet pipe through which partially treated effluent flows out to the distribution box. Inside the septic tank, solid waste settles to the bottom forming sludge, while lighter materials like grease and oils float to the top as scum. The middle layer consists of partially clarified effluent.

The size of the septic tank is determined based on the number of bedrooms in the house or the daily wastewater flow rate. In Hawaiʻi, typical septic tank sizes for residential properties may range from 1,000 gallons to 1,250 gallons, or more, depending on the household's wastewater generation. Septic tanks are the most common conversion treatment technology in Hawaiʻi.

The tank contains a mixture of wastewater and anaerobic bacteria. While the primary function of the septic tank is to physically separate the solids and liquid into three layers: a top layer of scum, a middle layer of liquid effluent, and a bottom layer of sludge (Figure 1.2), the anaerobic bacteria further break down the solids that remain in the tank. The liquid effluent flows out of the tank and into a disposal system, where it is further treated by means of soil filtration and dispersed into the soil. The sludge and scum remain in the tank and must be periodically pumped out by a professional septic service approximately every three to five years, depending on usage. The drainfield can fail prematurely if periodic pumping is not completed. If solids overflow into the drainfield, they can clog up the soil pores physically and biologically due to excessive bio-mat growth.

The local IWS contractors are familiar with the nuances of septic system construction, which minimizes potential installation mistakes. The other advantages of the Traditional IWS are that it operates without the need for electricity, so operational cost is minimal and there are no mechanical or electrical components to repair or replace.

Residential septic systems on their own remove about half of the organics in the wastewater stream and none of the nitrogen. The overall level of treatment is thought to be much higher when well-maintained and operating with a fully functional absorption bed (Table 1.1).

Because the level of treatment is highly dependable on the soil filtration which can vary greatly from site to site, it is a common practice in Hawaiʻi to utilize a 3-foot soil replacement below the drainfield to simulate a good soil filtration layer when percolation rate is expected to be faster than one minute per inch.

Due to their simple design, septic systems are applicable for consideration to be used for the project and will be evaluated further. Septic systems also cause less disturbance to the public Right-of-Way (ROW) in comparison to the installation of a new wastewater collection system.

Table 1.1: Typical septic system performance in Hawaiʻi (Carollo, 2021).

Contaminant	Typical Raw Residential Wastewater [1]	Typical Septic Tank Effluent Quality [2]	Typical Effluent Quality Following Soil Absorption System [2]
Total Nitrogen, mg N/L [4]	14-40	39-82	~1
TSS (mg/L)	100-400	49-161	~4
BOD (mg/L)	100-400	132-217	<30
Fecal Coliform, MPN/100ml [3]	~10^6	1-10^6	~13

1 From Table 2-1 (Water Resources Center (WRRC) University of Hawaii-Manoa, 2008).
2 From Table 4-1 in the Onsite Wastewater Treatment Survey and Assessment Study (WRRC, 2008).
3 MPN/100mL = most probably number per 100 milliliters.

2.2 Residential Aerobic Treatment Units (ATU)

An ATU is a type of wastewater treatment system that utilizes oxygen and aerobic bacteria to break down and treat household sewage. ATUs come in many proprietary shapes and sizes but at a minimum, systems typically include a primary settling chamber (similar to a septic tank), an aeration chamber, where the wastewater is blended with air or oxygen while suspended bacteria/microbes are able to grow and thrive and a clarifier chamber, where microbes are allowed to settle out of the water (Figure 1.3). The effluent from the ATU will discharge into the drainfield similar to the Traditional IWS.

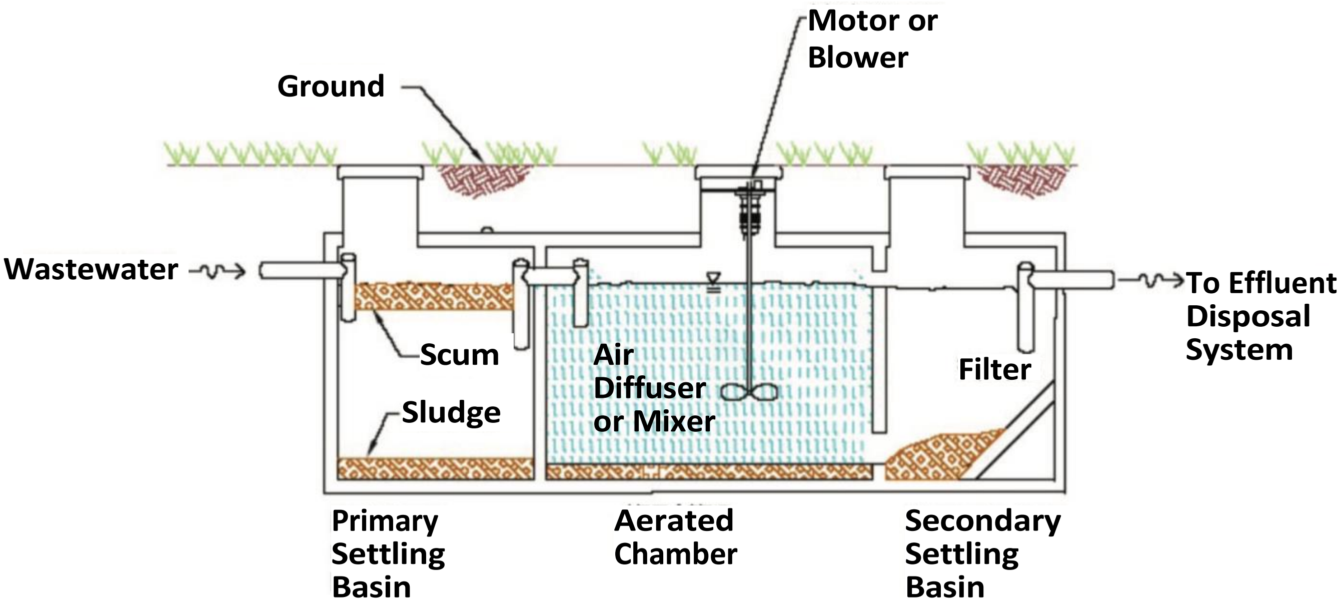
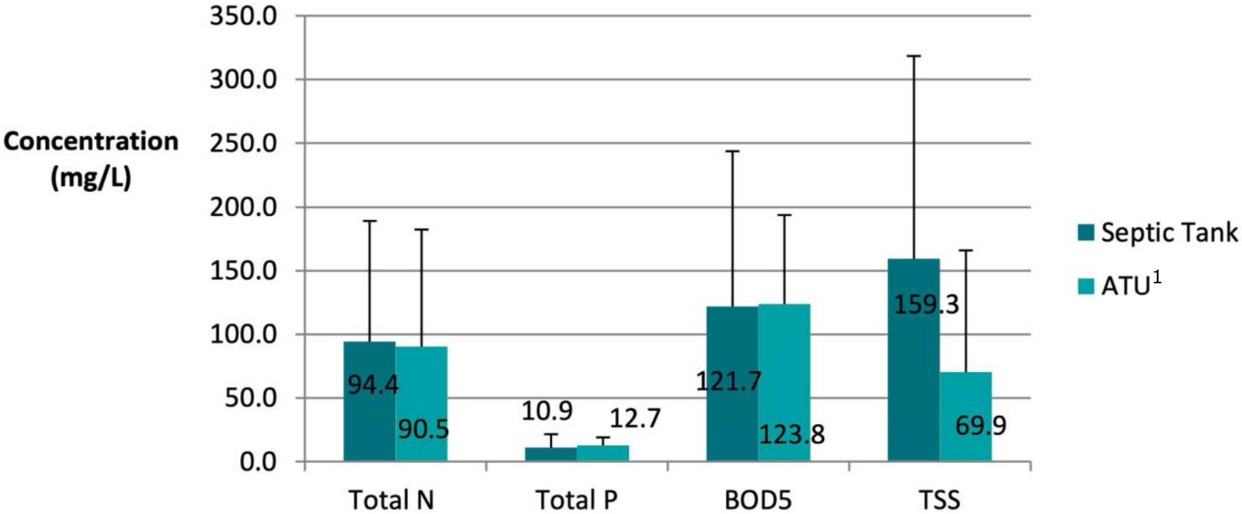


Figure 1.3: Side-view of typical aerobic treatment unit (Carollo, 2021)

With the introduction of air and/or oxygen into the wastewater within the settling chamber, ATU systems provide higher organic and nutrient removal rates than traditional septic systems, making ATUs optimal for operation upstream of sensitive receiving environments. Conversely, the added mechanical and electrical componentry leads to more frequent system downtime. Downtime can occur when mechanical parts such as blowers, pumps, and control systems need to be replaced, and when the ATU system requires cleaning, monitoring, and airflow adjustments. In addition, power outages can halt the treatment process due to the mechanical components' reliance of electricity, which can result in downtime until power is restored.

Without an effective maintenance strategy, the performance of ATUs in Hawaiʻi has been proven to be similar to septic tanks (Figure 1.4). ATUs are less affordable than septic tanks, as they have a higher up-front cost and a much higher O&M cost due to the requirement of contracting a certified wastewater treatment operator by the DOH, and due to the reliance of electricity. The continuous operation of mechanical components and exposure to moisture and organic matter can lead to ATUs having a shorter service lifetime than septic systems, as the components are more prone to wear and malfunction over time.

Installation of ATUs may be applicable for this project when a single dwelling has more than five bedrooms, in accordance with the DOH Chapter 11-62 guidelines.



[1] ATU effluent quality, if not properly maintained without an effective maintenance strategy, can perform similar to septic tanks in Hawai'i as stated in paragraph 2 in section 2.2.

Figure 1.4: Effluent quality of septic systems vs. ATUs in Hawaiʻi (Babcock, 2012)

2.3 Intermittent Sand Filter (ISF)

Intermittent Sand Filter (ISF) systems provide secondary treatment that effectively eliminate contaminants through the means of physical, chemical and biological treatment processes for primary treated wastewater or septic tank effluent (Figure 1.5). ISFs are made in a variety of packed-bed filters composed of sand or other granular materials (EPA, 2002). Septic tanks are used as a preliminary treatment step before sending wastewater to an ISF. The septic tank's primary function is to separate and settle out solids from the incoming wastewater. This helps to reduce the organic load and solids entering the ISF, thus extending its lifespan and improving its overall performance. Following the septic tank, the effluent is either pumped or gravity fed to the ISF.

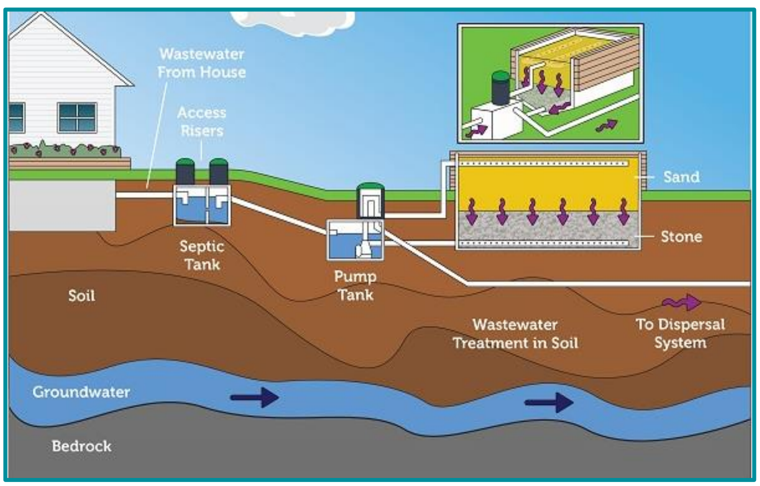


Figure 1.5: Pumped discharge ISF following a septic tank (EPA, 2023)

Wastewater is dosed onto the sand surface through a distribution network, percolating through the sand layer to reach the underdrain system. Filtered effluent is collected for further treatment or discharged to a leach field through a system of perforated pipes. These systems involve an excavation or structure lined with impermeable PVC on sand bedding filled with washed sand.

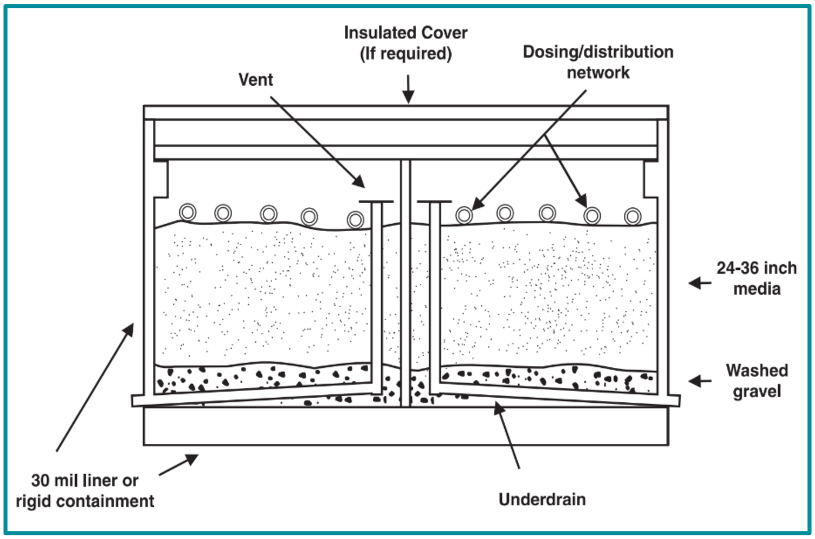


Figure 1.6: Generic, open intermittent sand filter (EPA, 2002)

Most biochemical treatment occurs in the top six inches, effectively removing suspended solids and carbonaceous biochemical oxygen demand (BOD). Microorganisms absorb wastewater constituents, nearly eliminating BOD. Decreasing carbonaceous BOD allows nitrifying microorganisms to thrive deeper, promoting nitrification.

There are several different types of ISFs:

- **Gravity Discharge ISFs**: These filters use gravity to distribute wastewater onto the sand surface usually on a sloped surface. Gravity allows for the effluent to exit the bottom of the sand filter, where it is transferred to a drainfield through a system of perforated pipes. The bottom of the sand filter needs to be several feet higher than the drainfield area.

- **Pumped Discharge ISFs**: Pumped discharge ISFs utilize a separate pump station to distribute treated effluent from the filter to a drainfield, allowing for more controlled and flexible effluent distribution regardless of topography, unlike gravity discharge ISFs. The integrity of the sand filter liner is safeguarded with discharge piping positioned above it.

- **Bottomless ISFs**: The bottomless ISF lacks an impermeable liner and doesn't release wastewater to a drainfield. Instead, it directly infiltrates into the soil beneath the sand.

ISFs present a more intricate setup that includes various components like a dosing tank, pump or siphon, distribution network, and a filter bed equipped with an underdrain system. Regular maintenance will increase the cost due to the electricity and labor for pumping/dosing, and overall O&M labor. There is a potential risk of the filter media in intermittent sand filters becoming clogged, and finding local replacements for the media could be challenging in Hawaiʻi.

For the Nāʻālehu LCC Replacement project, ISFs are not the most suitable IWS choice for residential installation. This is due to each dwelling having limited yard space to allow area for a septic tank, ISF, drainfield, and potentially a pump station. In addition, the sand needed for ISFs would be difficult to source in Hawaiʻi.

Usage Case: Sand filters are a practical alternative when site conditions hinder proper wastewater treatment and disposal through percolative beds/trenches. They are suitable for sites with shallow soil, poor permeability, high groundwater, and limited land.

2.4 Passive Treatment Beds

Passive treatment beds are a type of secondary, decentralized wastewater treatment technology designed to treat domestic or small-scale municipal wastewater. Passive treatment beds need to be installed together with a septic tank and do not require the use of electricity. This technology uses a natural process to prevent suspended solids from sealing the underlying soil by incorporating aeration and a larger surface area for bacterial treatment than traditional systems (Presby Environmental, Inc. 2017). Treated water is typically discharged directly to the soil below the treatment system via a soil absorption system (New Zealand Distributors, 2018).

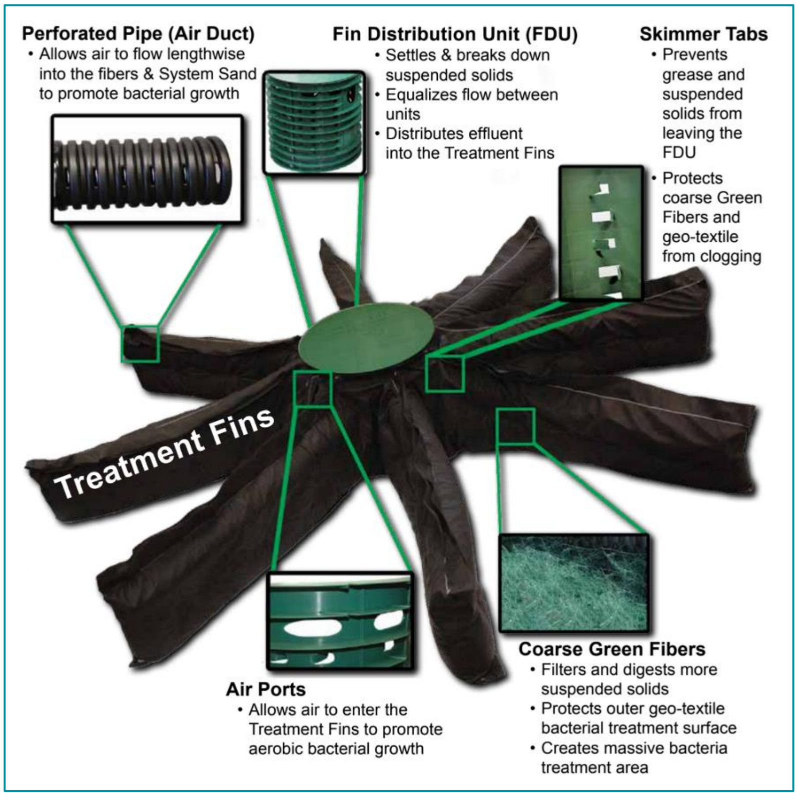
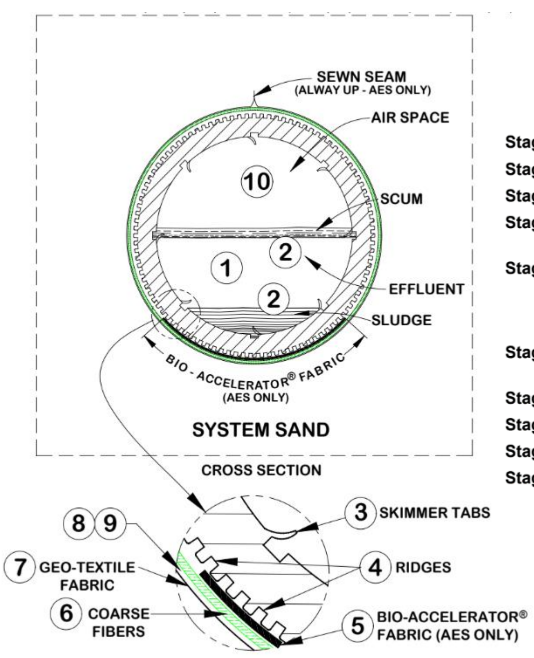


Figure 1.7: Schematic of Presby's Enviro-Fin Passive Treatment Bed (Presby Environmental)

Presby Environmental, Inc. specializes in the development and manufacturing of several passive wastewater treatment technologies, including Enviro-Fin (EF), Advanced Enviro-Septic (AES), and Enviro-Septic (ES), and Simple-Septic (SS). The Enviro-Fin system (Figure 1.7) consists of a centrally located Fin Distribution Unit (FDU), a basin and a sump made of plastic. The sump has a waterproof area at the bottom and small holes around it. Inside the sump, there is a pipe that helps equalize the wastewater between different parts of the system. The septic tank effluent flows into the FDU and then gets distributed to eight Treatment Fins that extrude radially from the center and treat the wastewater. Small holes in the plastic air ducts supply oxygen to the surfaces containing bacteria, which help break down the suspended solids in the wastewater. The system also has green plastic fibers packed underneath the air ducts, and specially designed fabric around the Treatment Fins and FDU, which increases the surface area for bacterial growth and wastewater treatment.

AES (Figure 1.8) consists of a pipe made of corrugated, perforated plastic and a Bio-Accelerator® fabric along its bottom. This fabric is surrounded by a layer of randomized plastic fibers and sewn geo-textile fabric. Together, these materials create an ecosystem within the pipe that efficiently treats the wastewater (Presby Environmental, Inc. 2017). The AES technology represents the evolution of Presby's ES system, incorporating the proprietary Bio-Accelerator® enhancement. This enhancement not only filters additional solids from the effluent but also accelerates the treatment processes, ensures even distribution, and provides a larger surface area for bacterial activity. Each foot of the AES pipe offers more than 40 square feet of total surface area for bacterial activity. The overall sizes of the AES system are similar to an absorption bed and depend on the number of bedrooms in a dwelling, and the percolation rate of the soil. Due to the higher rate of filtration, the pipes of an AES system can be installed in closer proximity compared to the perforated pipes in an absorption bed, which decreases the overall width of the disposal surface area needed.



Stage 1:	Warm effluent enters the pipe and is cooled to ground temperature.
Stage 2:	Suspended solids separate from the cooled liquid effluent.
Stage 3:	Skimmers further capture grease and suspended solids from the existing effluent.
Stage 4:	Pipe ridges allow the effluent to flow uninterrupted around the circumference of the pipe and aid in cooling.
Stage 5:	Bio-Accelerator® fabric screens additional solids from the effluent, enhances and accelerates treatment, facilitates quick start-up after periods of non-use, provides additional surface area for bacterial growth, promotes even distribution, and further protects outer layers and the receiving surfaces so they remain permeable. (AES only)
Stage 6:	A mat of coarse, randomly-oriented fibers separates more suspended solids from the effluent. (AES & ES only)
Stage 7:	Effluent passes into the geo-textile fabrics and grows a protected bacterial surface.
Stage 8:	Sand wicks liquid from the geo-textile fabrics and enables air to transfer to the bacterial surface.
Stage 9:	The fabrics and fibers provide a large bacterial surface to break down solids.
Stage 10:	An ample air supply and fluctuating liquid levels increase bacterial efficiency.

Figure 1.8: AES, Enviro-Septic, and Simple-Septic 10 stages of treatment.

The ES technology is composed in a similar fashion to the AES pipe but offers only 25 square feet of surface area for bacterial activity for every foot of pipe. The SS pipe is comprised of a single-layer geo-textile fabric, large diameter, gravelless pipe (LDGP) system, coupled with

Presby's patented skimmer tabs and cooling ridges which increase bacterial growth on the surface area of the fabric. For every foot of SS pipe, there is 15 square feet of total surface area for bacteria to grow.

Presby has claimed that each treatment system output effluent that has been tested and certified to NSF 40, Class I standards. The use of the Presby Passive Onsite Wastewater Treatment & Dispersal System is permitted under Section 11-62-35(b) of the Hawaiʻi Administrative Rules (HAR) by the Department of Health (DOH). The DOH acknowledges this is an innovative solution that could be beneficial to the State. In addition, the DOH requires that operational data be submitted upon request within the first twelve months after installation when Presby products are used in place of soil absorption systems.

In general, the cost of an AES or Enviro-Fin system may be comparable to or slightly higher than that of an ATU. Both systems require upfront investment for equipment, materials, and installation. However, it is important to consider that the long-term costs of maintenance and operation can differ between the two systems. ATUs typically require ongoing electrical power for aeration and additional maintenance, such as replacing mechanical components and monitoring systems. These factors can contribute to higher operational costs over time. On the other hand, AES systems are designed to operate passively without the need for electricity or mechanical components. They rely on natural processes and have fewer moving parts, which can lead to lower long-term maintenance and operational costs. Enviro-Septic or Simple-Septic systems have a lower system complexity than AES or Enviro-Fin Presby technologies and are generally less expensive alternatives but require a bigger footprint.

Presby offers both in-person and online training options, covering various aspects of system design, installation, maintenance, and regulatory requirements. Proper installation techniques, system components, troubleshooting, and best practices for achieving optimal performance of wastewater treatment systems are provided in the training.

While these passive treatment bed systems offer higher effluent quality without mechanical components, they are new to the market and are not familiar to local contractors. Furthermore, one of the key components of these systems is the requirement of "System Sand" which is a granular material with a very specific gradation. At the time of this report, it is not known if the local rock quarry can produce aggregates that meet that specification.

Based on the increased regulatory requirements, unfamiliarity of local contractors, and supply chain limitations for the aggregates needed, AES, EF, SS, and ES solutions may not be suitable treatment processes for the project.

2.5 Composting Toilet

A composting toilet is a type of toilet that uses a combination of heat and aerobic microbial action to break down human waste into a nutrient-rich compost. Composting toilets typically come in three varieties:

- **Individual Composting Toilets:** These waterless toilets combine human waste with bulking material such as sawdust, leaves, or peat moss in a single chamber (Figure 1.9). The waste dries and composts in-situ until the container fills and then needs to be hauled to a landfill that can process solid waste in accordance with DOH standards. Some composting toilets also use electrical or mechanical systems, such as an exhaust fan, to aid in the breakdown of waste and limit odors. These toilets do not require water or a connection to a sewer system, making them an eco-friendly alternative to traditional flush toilets.

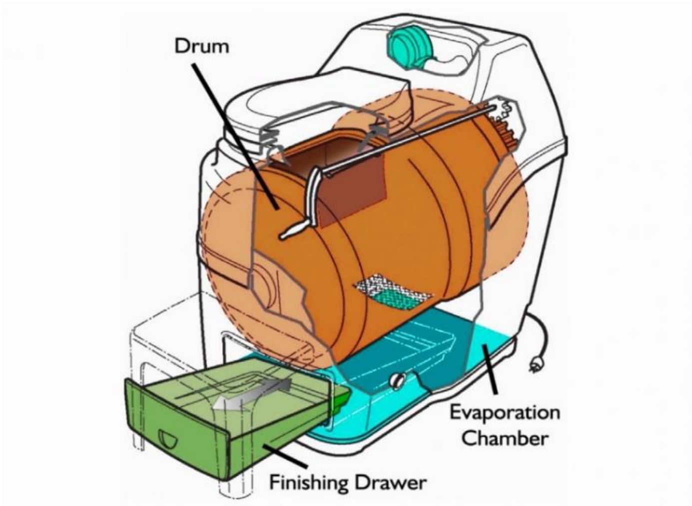


Figure 1.9: Component view of a typical individual composting toilet (Sun-Mar, 2023)

- **Urine Diverting Toilets:** Urine diverting toilets typically have two chambers: one for urine and one for solid waste (Figure 1.10). The urine is typically stored and used as a fertilizer, while the solid waste is broken down into compost.

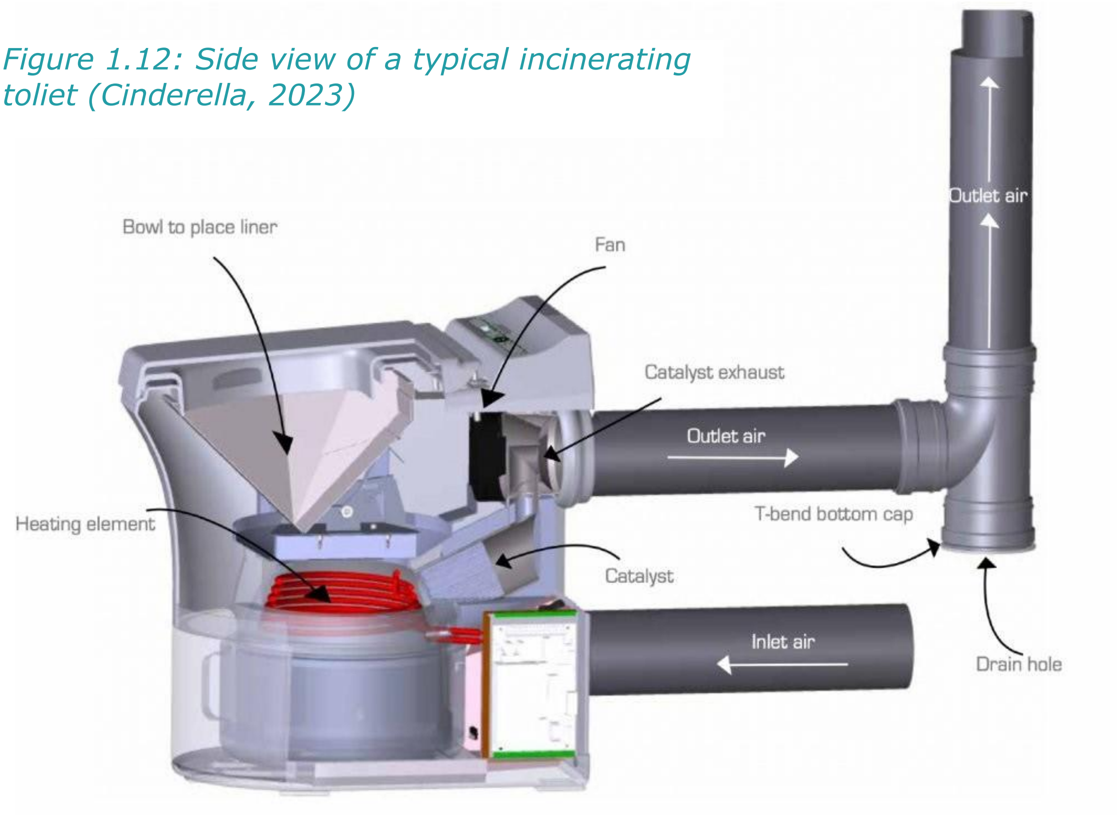
Figure 1.10: Top view of a typical urine separating toilet (Separett, 2023)

- **Central Composting Units:** These composters are installed outside the home and work in conjunction with pint-flush toilets. Toilet blackwater (along with a scoop of wood chips) flows through a 4” gravity sewer to the composter where it is introduced to a horizontal drum that collects the solids and allows liquids to drain into the base of the enclosure (Figure 1.11). The drum must be rotated in the forward direction once every two days to fluff the retained solids and rotated in the reverse direction once every two months to drop the retained solids into an aging drawer where the composting process is completed. The outputs are a drawer of composted manure and drained liquid. A single central composting unit can serve an entire home and solid wastes and any odors are kept entirely outdoors.

Figure 1.11: Top view of a central composting unit (Sun-Mar, 2023)

Composting toilets are reviewed and approved by the DOH on a case-by-case basis. Solids generated from composting toilets that are land applied must meet the requirements of HAR 11-62 Subchapter 4: Wastewater Sludge Use and Disposal. This treatment process may not be applicable for this project due to the regulations pertaining to the disposal of solid waste, which could require disposal at a municipal solid waste landfill unit in compliance with the sludge related conditions in a permit issued by the DOH. Septic tanks require less active involvement from homeowners and do not typically require regular emptying if properly sized and maintained. Composting toilets can also emit strong odors if not properly maintained.

Figure 1.12: Side view of a typical incinerating toliet (Cinderella, 2023)



2.6 Incineration Toilet

An incineration toilet, also known as a thermal toilet, is a type of toilet that uses heat to turn human waste into ash (Figure 1.12). The waste is placed into a combustion chamber where it is heated to high temperatures, typically around 800-1000 degrees Fahrenheit, by a gas or electric burner. This process kills any harmful bacteria and viruses and reduces the volume of waste by up to 90%. The ash that is left can be safely disposed of in a landfill or used as a fertilizer. Incineration toilets do not require any water, making them suitable for remote locations or areas with limited water resources.

They also produce very little smell and have no need for a septic system or connection to a sewer. However, they do require electricity or gas to operate, and can be relatively expensive to purchase and maintain. Further, they do not comprise a complete treatment solution. While the remaining household gray water can be disposed of with minimal treatment, a septic tank or other treatment unit is still required to treat kitchen blackwater, eliminating the economic advantage.

No matter the variety, composting toilets and incineration toilet technologies are significantly lower cost than septic tanks and other IWS. Unfortunately, they do not comprise a complete treatment solution, making them not applicable for this project.

2.7 Graywater Reuse

Graywater systems are systems that collect and reuse wastewater from sources such as sinks, showers, and laundry machines. The collected water is then treated and can be used for landscape irrigation once separated from blackwater. The wastewater streams can be separated through the implementation of two separate wastewater piping systems in the household. Graywater can be treated to a re-use level for distribution strictly below the soil surface or should be disposed of to an IWS or county sewer system (CS).

In Hawaiʻi, the use of graywater systems is regulated by the State Department of Health, which sets guidelines for the treatment and reuse of graywater. Overall, these systems are typically expensive to install and maintain as they are installed in addition to a traditional wastewater treatment system that's still required to treat household blackwater. For simplicity of design, graywater systems are less applicable for this project than a traditional septic tank system, as access to a CS is not readily available in Nāʻālehu, which would require the reuse system to have both a septic and graywater holding tank.

Usage Case: Used on lots with source separated plumbing that prioritizes wastewater reuse.

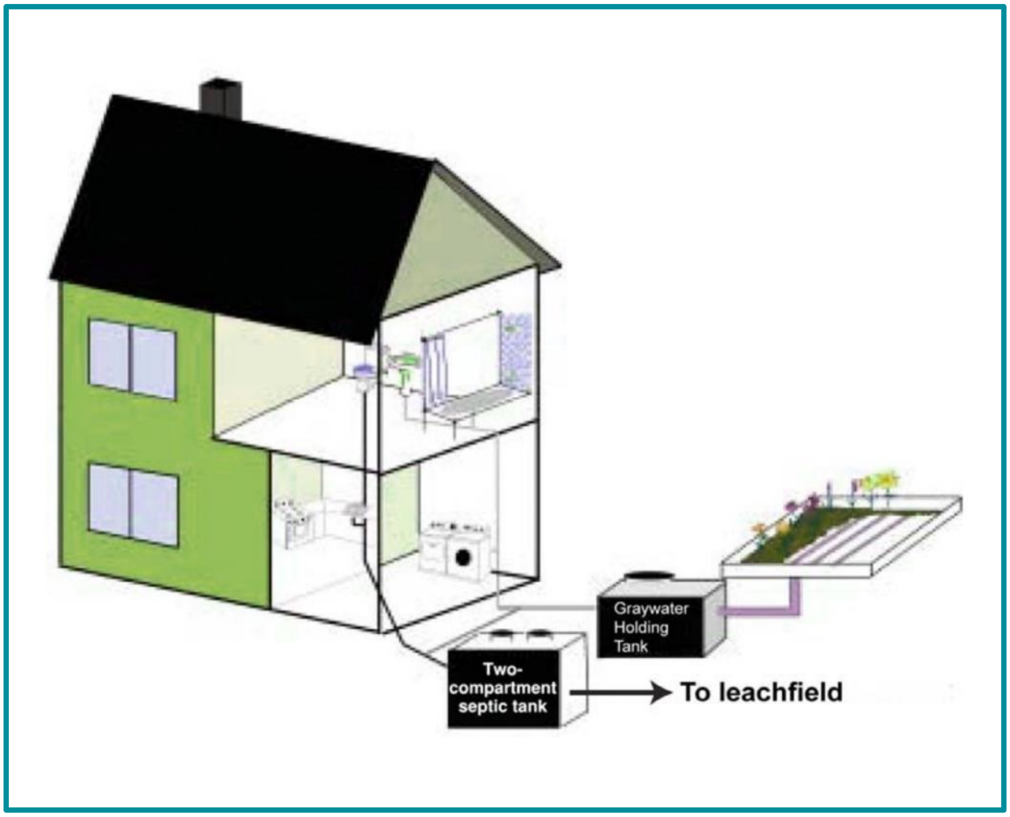


Figure 1.13: Typical graywater reuse system installed in parallel with a septic tank (DOH, 2009)

3. Types of IWS Disposal Systems

3.1 Absorption Bed

Absorption beds are the most common form of IWS disposal system installed in Hawaiʻi today. They consist of a network of perforated pipes, each a maximum of 100 feet long and laid in trenches 1.5-3 feet below the finished grade 4-6 feet apart (Figure 1.14). Each line is laid level to allow the gravity dispersal of treated effluent below and above each pipe, with a filter fabric covering the top of gravel, to prevent the gravel from clogging. A minimum of 6 inches of gravel is provided below each pipe. If the percolation rate is faster than one minute per inch, a depth of 3-foot soil replacement shall be installed to underlie the entire absorption bed. Soil replacement shall be washed #4 sand or cinder-soil mix with a percolation rate not faster than one minute per inch.

These systems are easy to maintain when following an effective treatment system and microorganisms in the soil offer an added degree of treatment to the effluent as it filters through the upper oxic layers of the soil matrix. Absorption beds, however, have a significant space requirement, which increases with decreasing hydraulic conductivity of the soil. Additionally, absorption beds can only be installed on a grade of less than 8%.

While conventional perforated pipe adsorption beds are not traffic rated, companies such as Infiltrator offer a chambered dispersion product with an H-20 load rating that can also reduce the absorption bed space requirement by 17%.

Absorption beds are applicable for this project when space permits, as they utilize natural soil and microbial processes to treat and filter wastewater, minimizing the environmental impact. Additionally, the relatively simple design and lower maintenance requirements make them cost-effective and practical for residential properties.

Usage Case: Used on typical lots without spatial, groundwater level, grade, or percolation rate constraints. Installed on flat/mild sloped terrain.

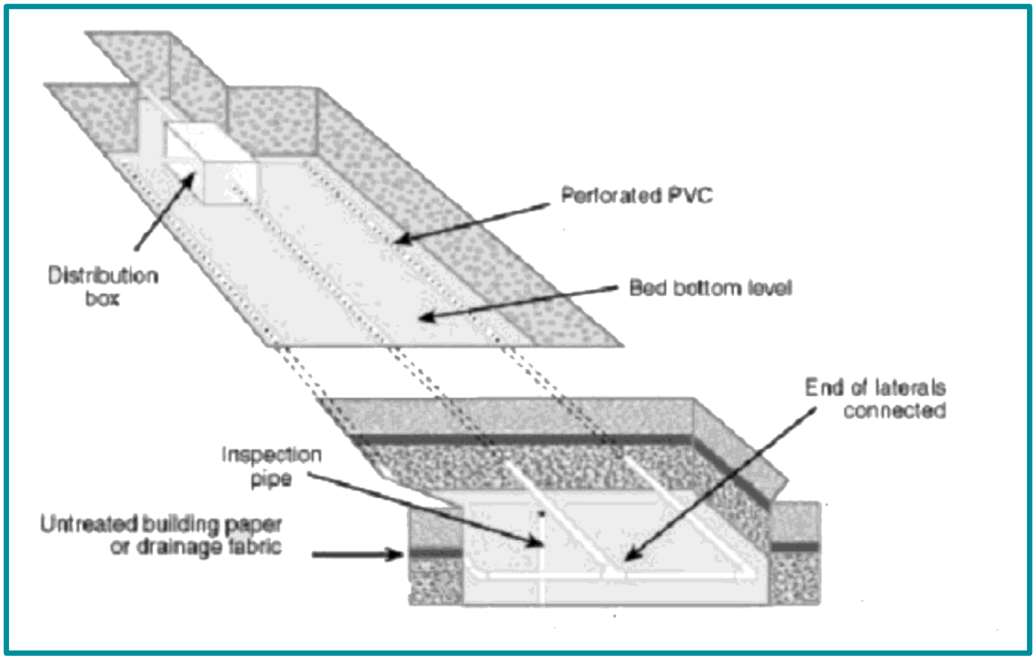


Figure 1.14: Typical absorption field installed following a septic tank (Babcock, 2019)

3.2 Leach Filed

Leach fields consist of absorption trenches installed parallel to the contour line on ground slopes up to 12%, making them suitable for steeper terrain. These trenches are non-traffic rated, and consist of gravel, which serves as a natural filter that permits gradual seepage into the adjacent soil. Perforated PVC pipes are commonly used to distribute wastewater into the trenches. Additionally, filter fabric lining can be installed to prevent the gravel from becoming clogged with soil or debris.

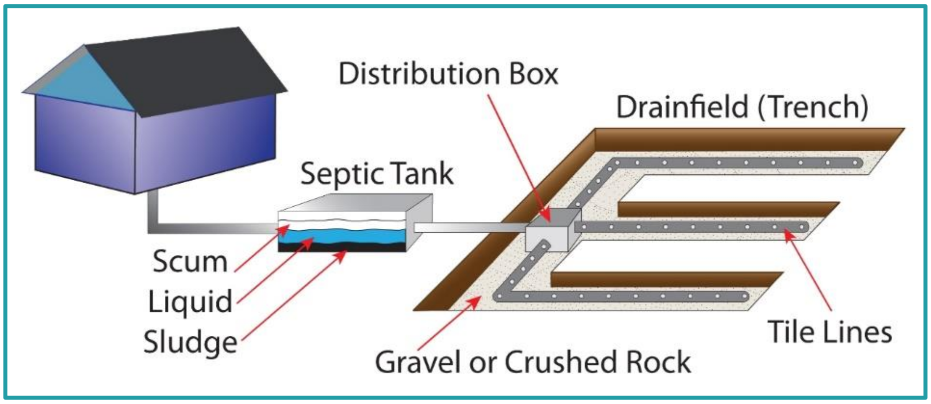


Figure 1.15: Leach filed schematic (Nuflw Wide Bay, 2015)

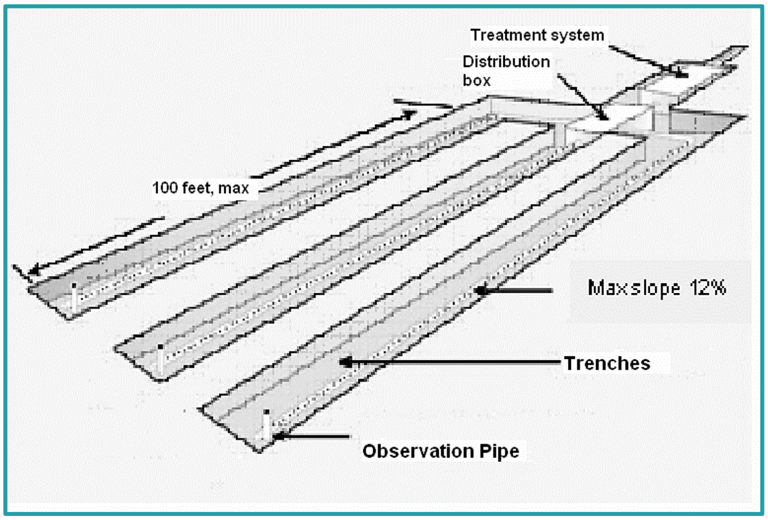


Figure 1.16: Absorption Trenches Diagram (Water Resources Research Center, & Engineering Solutions, Inc. (2008).

The DOH design requirements state absorption trenches should have a width ranging from eighteen to thirty-six inches. Additionally, there should be a minimum vertical separation of three feet between the bottom of the trenches and the underlying groundwater or bedrock.

Leach fields with absorption trenches are applicable for this project when the site has a grade between 8%-12% and will be discussed further in the report.

Usage Case: Absorption trenches are typically used on steeper terrain.

3.3 Seepage Pits

Seepage pits are a vertical means of achieving the percolation area requirements for a disposal system. These systems typically consist of a 15-30-foot-deep pit lined with stacked precast perforated concrete rings or CMUs, to an internal diameter of 6-8 ft. Seepage pits are both less area intensive and less expensive than absorption beds, if converted from an existing cesspool. A seepage pit must include a cover which extends at least 12 inches beyond the seepage pit excavation or over a provided concrete lining. An access hatch must be provided in the concrete cover to allow inspection and maintenance of the pit. The seepage pit may be designed to be traffic rated by providing the sufficient strength required in the design of the concrete lining and cover.

The effective area of the seepage pit is equal to the vertical wall area corresponding to the effective depth of the pit. Slow percolation rates translate to a larger required absorption area and deeper pit (Figure 1.21).

While seepage pits are an approved means of disposal in Hawaiʻi, they are often only permitted when it can be demonstrated that an alternative means of disposal was not possible, i.e., insufficient land area, steep terrain (>12%) or very slow percolation rates (>60 min/inch). Where slow percolation rates present, seepage pits will need to be dug through the basalt rock layer to reach more porous soils or a variance will be required from HAR 11-62-34 d(1)b:

Seepage pits shall not be constructed in soils having a percolation rate slower than ten minutes per inch (weighted average) or where rapid percolation through such soils may result in contamination of water-bearing formations or surface water.

Seepage pits are applicable for consideration to be used for this project for site conditions mentioned above and will be evaluated further.

Usage Case: Used on highly spatially constrained, slope constrained, or geologically constrained lots where sufficient percolation rates can be achieved.

3.4 Subsurface Drip Irrigation

Subsurface drip irrigation is an extremely water efficient means of wastewater disposal, slowly delivering effluent into the existing vegetation present on site within the disposal area, promoting efficient uptake of nutrients by the microbes and plants in the soil medium (Figure 1.17). Installation of subsurface systems often requires less disruption to the absorption area though in Hawaiʻi, they traditionally cost more. Regular maintenance is required to ensure the continued operation of the irrigation pump and manage biofouling in the distribution lines.

Subsurface drip irrigation may not be suitable for the project, as more O&M is required compared to alternative wastewater disposal systems, in order to flush the drip lines to mitigate clogging from debris. Chlorination to remove biological growth from the drip lines would also be needed, which has a greater environmental impact than other means of disposal. Additionally, each IWS with subsurface drip irrigation would require the incorporation of an ATU to properly treat wastewater for reuse.

Usage Case: Used on lots that prioritize wastewater reuse, are served by an ATU, and have a robust maintenance strategy.

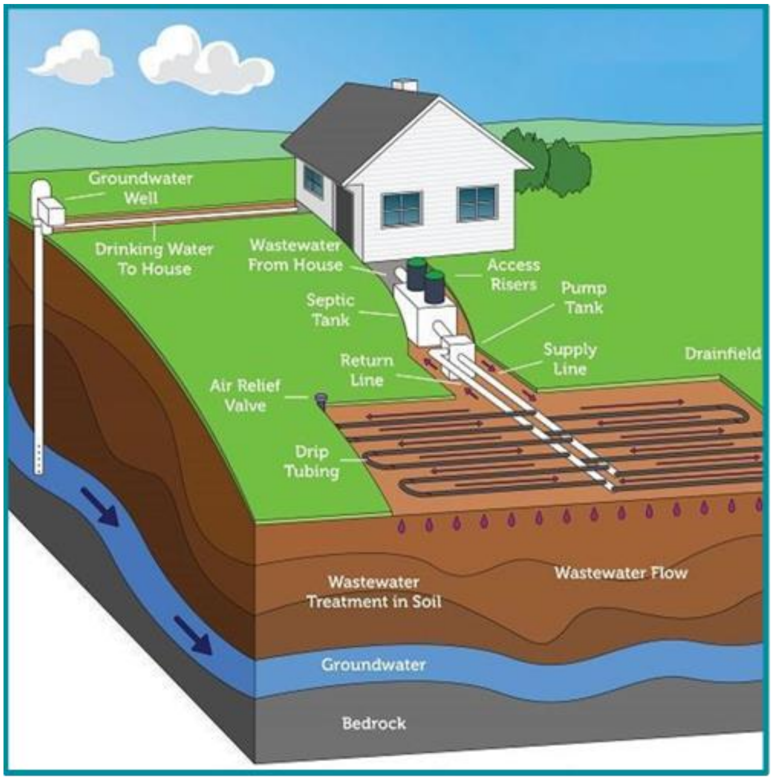


Figure 1.17: Typical subsurface drip irrigation installed following a septic tank (EPA, 2023)

3.5 Evaporation Beds

Evaporation beds, or evapotranspiration (ET) wastewater treatment systems, are utilized to remove effluent by direct evaporation and by plant transpiration. If soil infiltration is desired, evapotranspiration/infiltration (ETI) process can dispose of effluent by employing both evapotranspiration and soil penetration. ET and ETI systems follow primary pretreatment units which filter out settleable and floatable soils.

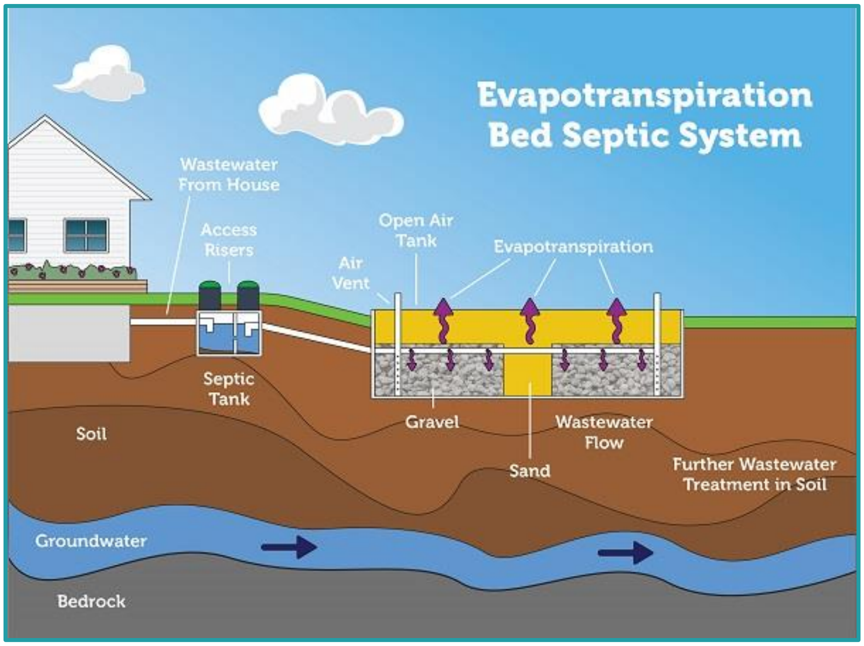


Figure 1.18: Evapotranspiration (ET) bed (EPA, 2018)

The septic tank effluent enters the ET or ETI units through distribution pipes and reaches a porous bed. To prevent water from seeping out, a liner is used below the bed in ET systems. Aquatic plants are grown on the surface of the sand bed to aid in transpiration of the effluent. In ETI systems, the effluent can percolate into the underlying soil with the absence of a liner.

Evaporation beds are sizable disposal systems that may experience overflow if the evapotranspiration rate does not exceed the monthly precipitation by more than 2 inches a month. ET/ETIs are reviewed on a case-by-case basis by the Department of Health.

Evaporation beds may not be appropriate for this project, as Nāʻālehu has a tropical climate with abundant rainfall throughout the year. Additional maintenance is also required to prune and monitor plant health to ensure the system functions effectively.

Usage Case: Arid climates where evaporation exceeds precipitation. Sensitive locations where no subsurface disposal is desired/allowed. These disposal units are most appropriate following ATUs but are occasionally employed after septic tanks.

3.6 Mound Septic System

When greater vertical distance from groundwater is needed, or when the soil quality or slope is not conducive for an absorption bed or trench system, above-ground mounds of sand or soil can be implemented to achieve desired conditions (Water Resources Research Center, & Engineering Solutions, Inc., 2008). The process begins by preparing the land where the mound will be situated through tilling. Afterward, a layer of sand is spread across the tilled area along with a distribution system. Next, the top of the mound is covered with soil from the surrounding area and given an aesthetically pleasing landscape. A pump system is necessary due to the influent disposal point being at a higher elevation than the treatment system. This factor raises the overall cost of the system since electricity is required.

Figure 1.19: Elevated Mound System (EPA, 2018)

The Department of Health (DOH) evaluates the design criteria for mound disposal systems on a case-by-case basis when systems are outside the percolation rate range of trenches and beds. There is a three feet minimum required separation from any existing groundwater.

Mound septic systems may not be suitable for this project as the cost can be more expensive to install and maintain compared to traditional wastewater disposal systems. This is due to the additional design components required, such as the construction of the mound itself and specialized fill material. Mound systems require a larger land area than other disposal systems which can be a limitation on smaller lots.

Usage Case: In areas where limitations exist due to poor soil or proximity to groundwater.

4. Traditional IWS Recommendation

When considering an IWS alternative, existing regulations, environmental concerns, site constraints, economics, and performance shall be taken into consideration. Overall, each household IWS could cost between $30,000-$150,000 to install and roughly $1,000 per year to operate and maintain. However, associated O&M fees for household IWS are dependent on the management model implemented, which is further discussed in Section 11.

Based on the EPA requirements for this project, the installation of traditional septic tanks with standard absorption bed is recommended for Nāʻālehu. Per the revised AOC, IWS is a possible alternative to be considered, however technical and environmental analysis will still need to be conducted for Hawaiʻi County to make an informed decision. Septic tanks coupled with absorption beds offer the lowest cost, least complex solution, with low maintenance requirements in comparison to the other options previously discussed. Where space and grading constraints prevent the installation of an absorption bed, an existing cesspool (if present) can be repurposed as a seepage pit for disposal, or a new seepage pit can be installed. This alternative is being considered for three reasons:

- **Passive Operation:** The traditional septic IWS offers a passive operation with minimal O&M cost. While the ATUs offer the promise of improved performance, in practice in Hawaiʻi, the added mechanical and electrical complexity and the operator requirement results in higher-than-average O&M costs or facing the risk of falling into similar treatment performance comparable to septic tanks.
- **Adaptive to Small Lots:** The option of disposing to seepage pits allows septic systems to be installed particularly on spatially and geographically constrained lots.
- **Familiarity:** Hawaiʻi's engineers, regulators, contractors, and septic pumpers are familiar with septic tanks, absorption beds, and seepage pits. The ATU and CTDS, on the other hand, are less known. Furthermore, the lack of certified wastewater operators throughout the State will result in higher costs and reduced performance for ATU and CTDS options.
- **Cost:** Traditional septic tanks don't have moving parts or require significant mechanical components, which can reduce maintenance and repair costs over time. Energy cost savings are also significant since septic tanks rely on natural processes for wastewater treatment, compared to some other IWS that involve mechanical or electrical components.
- **O&M:** The absence of complex components in septic tanks simplifies maintenance. There are no pumps, blowers, or electronic controls to manage or repair, making maintenance tasks more straightforward.
- **Environmental:** Septic tanks are more energy efficient than other IWS alternatives such as ATUs, which require energy-intensive processes (aeration) for effective treatment. Additionally, other IWS alternatives may require the use of chemicals for treatment processes. Septic tanks generally involve fewer chemical inputs, contributing to a lower environmental impact.

5. Traditional IWS Components

5.1 Types of IWS Tanks

There are several septic tank providers commonly used in Hawaiʻi offering tanks of a variety of price points and materials. Septic tanks can be made from concrete, plastic, and fiberglass (Figure 1.20), each of which having its own set of pros and cons (Table 1.2). Where a septic tank is located beneath a vehicular traffic area, a traffic rated concrete septic tank can be used or a structural concrete slab designed for H-20 loading spanning a non-traffic tank may be used.

The yellow plastic tank displayed in Figure 1.20 is manufactured locally by Chemtainer in Keaau, Hawaiʻi. This HDPE tank is the most economical type of septic tank. However, because the material is flimsy, it is prone to be installed improperly, and has more restrictions on the location to which it can be installed.

Septic tank manufacturers and distributers in Hawaiʻi include Jensen Precast, Ferguson (Infiltrator Chambers), Chemtainer, and Orenco.

Figure 1.20: Common septic tank materials and shapes in Hawaiʻi (Carollo, 2021).

Rectangular, Concrete Tank Tank

Oval, Concrete Tank

Cylindrical, Concrete

Rectangular, Plastic Tank

Fiberglass, Oval Tank

Steel, Horizontal, Cylindrical Tank

Table 1.2: Advantages and Disadvantages of Septic Tank Materials (Carollo, 2021).

Septic Tank Material	Advantages	Disadvantages
Concrete	• Durable • Suitable for installation in traffic area • Less susceptible to collapse and floatation • May be cast-in-place for custom shape	• Precast tanks can be more expensive than plastic or FRP due to shipping and installation costs • Typically requires use of a crane for installation • Concrete may corrode over time due to acidic sewer gases
Plastic (polyethylene)	• Less expensive than precast concrete tanks (lower shipping and installation costs) • Manufactured locally on the island • Plastics are typically resistant to corrosion • May not require a crane for installation	• Plastic tanks may deform depending upon quality of the plastic and potential structural weaknesses of the material • If not installed properly, plastic tanks can float if flooded
Plastic (polypropylene)	• Less expensive than precast concrete tanks (lower shipping and installation costs) • Plastics are typically resistant to corrosion • Locally stocked • Higher tank rigidity. More tolerant impact and backfill loads • May not require a crane for installation	• Not a traffic rated tank without additional structural slab • A two half-clam shell construction. Requires proper factory assembly to achieve watertightness
Fiberglass-reinforced polyester (FRP)	• Less expensive than precast concrete tanks (lower shipping and installation costs) • Variety of manufacturers and sizes for desired footprint • Fiberglass is typically resistant to corrosion • May not require a crane for installation • More rigid and sturdy than plastic tanks	• Less structurally strong than concrete tanks • If not installed properly, fiberglass tanks can float if flooded

Ultimately, the choice of septic tank material will depend on availability, budget, and site constraints (Table 1.3). At a minimum, septic tanks in Hawaiʻi must comply with International Association of Plumbing and Mechanical Officials (IAPMO) material and property standards for septic tanks. Further, sizing and installation criteria are regulated by HAR 11-62-33. The minimum septic tank capacity is 1,000 gallons for a household of 4 bedrooms or less and 1250 gallons minimum for households of 5 bedrooms. Septic tanks serving households greater than 5 bedrooms will require a variance from the DOH.

Table 1.3: Common single family home septic tank products in Hawaiʻi.

Product	Material	Traffic Rated	Capacity (Gal)	Length (in)	Width (in)	Height (in)	Weight (lbs)	Local List Price[1]
Chem-tainer	HDPE	No	1250	96	58	62	400	$3,189
Infiltrator (via Ferguson)	PP	No	1287	127	62.2	54.7	320	$2,863
Orenco (via Custom Concrete & Septic)	DCPD	No	1500	168	72	64.5	620	$6,850
Jensen Precast	Concrete	Yes	1250	138	70	57	16,700	$6,850

[1] Local list price does not include tax, contractor markup, transportation, or installation.

6. Design of IWS: DOH Chapter 11-62 Guidelines

Per the EPA Revised Administrative Order on Consent (AOC), SDWA-UIC-AOC-2017-002, dated August 22, 2022, the County must provide wastewater services for 194 properties, and close the Nāʻālehu Community Cesspools. The Department of Health's Hawaiʻi Administrative Rules, HAR 11-62-31.1, outlines general requirements for individual wastewater systems used in lieu of wastewater treatment works. Lots to be served in the community vary in size from 0.12 to 1.94 acres with a median size of 0.16 acres. The current Hawaiʻi Administrative Rules for IWS installation require a minimum of 10,000 ft^2 (0.23 acres) of usable land for each system for lots constructed and documented after August 30, 1991. The Nāʻālehu subdivision was approved for recordation in April 1966, and is exempt from this rule and considered grandfathered in.

The maximum permitted wastewater volume entering a single wastewater system must not surpass one thousand gallons, and this system is not allowed to cater to more than five bedrooms, regardless of whether these bedrooms are in one dwelling unit or distributed across two.

Table 1.4: Septic sizing guide per DOH

DOH Septic Tank Sizing Guide	
No. of Bedrooms	**Minimum Capacity (Gallons)**
4 or less	1000
5	1250

If a single household has more than five bedrooms and requires an IWS to handle the wastewater, then an Aerobic Treatment Units (ATUs) needs to be installed, which can handle a larger wastewater load.

6.1 Percolation Testing

Prior to installing the IWS, a percolation test is performed to assess the soil's ability to accept and treat the effluent. The results of this test help determine the appropriate size and design of the system. Per HAR 11-62, soil percolation tests must be carried out at a depth of at least three feet. If the soil composition varies between the three-foot and five-foot depths during construction, an additional percolation test should be conducted at the level corresponding to the bottom of the absorption system.

6.2 Sizing of Absorption Beds and Disposal Systems

The sizing for absorption beds and disposal systems is outlined in Figure 1.21 from HAR 11-62. Absorption beds cannot be built in soil with a percolation rate slower than sixty minutes per inch.

Appendix A contains preliminary absorption bed sizing calculations for the properties in Nāʻālehu using the HAR11-62 Absorption Bed Sizing Table. Percolation rates ranging from 10-15 min/in per site were used for the basis of this calculation, which is reflective of the higher end of the percolation test results.

Percolation Rate (min/inch) Less than or equal to	Required Absorption Area (ft2/bedroom or 200 gallons)	Percolation Rate (min/inch) Less than or equal to	Required Absorption Area (ft2/bedroom or 200 gallons)
1	70	31	253
2	85	32	257
3	100	33	260
4	115	34	263
5	125	35	267
6	133	36	270
7	141	37	273
8	149	38	277
9	157	39	280
10	165	40	283
11	170	41	287
12	175	42	290
13	180	43	293
14	185	44	297
15	190	45	300
16	194	46	302
17	198	47	304
18	202	48	306
19	206	49	308
20	210	50	310
21	214	51	312
22	218	52	314
23	222	53	316
24	226	54	318
25	230	55	320
26	234	56	322
27	238	57	324
28	242	58	326
29	246	59	328
30	250	60	330

Figure 1.21: HAR 11-62 Table III Absorption bed sizing table.

6.3 DOH Required Setbacks

The actual location of treatment and disposal infrastructure is limited by setback requirements. DOH-required setbacks are presented below (Table 1.5). From a system design perspective, it is recommended that systems should also be a minimum of 20 feet from any cut-face slopes present on a site to avoid surfacing of treated effluent. This is a particular constriction to heavily sloped sites.

Table 1.5: DOH required setbacks for wastewater systems: Table II, HAR 11-62.

Minimum Horizontal Distance From	Cesspool (ft)	Treatment Unit (ft)	Seepage Pit (ft)	Soil Absorption System (ft)
Wall line of any structure or building	5	5	5	5
Property line	9	5	9	5
Stream, the ocean at the shoreline certification, pond, lake, or other surface water body	50	50	50	50
Large trees	10	5	10	10
Treatment unit	5	5	5	5
Seepage pit	18	5	12	5
Cesspool	18	5	18	5
Soil absorption system	5	5	5	5
Potable water sources serving public water systems	1000	500	1000	1000

6.4 DOH Variance

The Nāʻālehu LCC Replacement project will likely encounter the following situations where a DOH Variance request would be needed:

- Absorption field placement due to percolation rates and property size.
- Seepage pit installation when space is limited.
- Septic tanks serving households with more than 5 bedrooms.

Variance application and review usually adds approximately two months to the permitting process following design package review but the DOH has expressed willingness to allow the request for variance to be included with the initial design package.

7. Typical IWS Permitting and Construction Process

The permitting and construction process for IWS begins with the Engineer of Record (EoR) preparing and submitting a design package for DOH approval. Once DOH reviews and approves the design package, DOH issues an "Approval for Construction" letter. While during DOH review process, the plumbing contractor submits the Plumbing Permit application with County DPW Building Division for building sewer line modification. Once both permits are issued, the contractor can start excavation of the septic tank, drainfield and pipe trenches, while the existing sewer lines are protected in place, or a temporary bypass is installed to keep the existing system in service. The EoR will perform a site-specific percolation test at the bottom of the drainfield excavation to further verify the actual percolation rate and adjust drainfield sizing as necessary. Per HAR 11-62, retesting is only required however when the soil profile at 5 feet depth is different than the soil profile that was performed during the percolation tests. After the installation of drain rocks, drainfield piping and the septic tank, the engineer performs a final inspection and document as-built of the installed system before final backfilling and files a final inspection report to DOH for review. If everything is in order, the DOH returns an "Approval for Use" letter and the building waste line can be changed over to the newly installed IWS.

The Nā'ālehu LCC Replacement project will use a project specific approach in three principal ways:

- **Properties to be Permitted in Bulk:** The DOH has expressed the capacity to receive packages in groups of 10 or more properties. This will serve to expedite the DOH review process.
- **Variance Requests:** Due to preliminary data on percolation rates and property sizes, it's expected that DOH variances to setback constraints will be required to accommodate the IWS installation. Where space is still overly constrained, DOH variances will be required to allow for seepage pit installation. During the design phase, the specific lots requiring variances will be identified, which the variance application can be submitted prior to submitting the design application to construct, which will enable the permitting process to go more efficiently.
- **Optioned Design:** The geological conditions present at a given site will not be known until construction on that site commences. These conditions will have a significant impact on the disposal system design. To accommodate this uncertainty in the design and facilitate permitting, an optioned design package shall be submitted to the DOH to allow for a field determination according to the actual percolation rates and soil composition encountered. We would expect the in-situ soil stratum to change from 4-5 feet deep of volcanic ash soil at the surface to solid or fractured basaltic rock below 6 feet deep. The percolation rates between the volcanic ash soil and the basalt rock are great, so as comparing fractured and unfractured rock. Since the absorption bed sizing greatly depends on the percolation rate, location and the depth of required excavation, it is best to perform the percolation test at the bottom of the excavation to represent actual conditions of design.

The timeline and deliverables for this procedure is outlined on the following page (Table 1.6).

Table 1.6: Hawaiʻi Permitting and Construction Process[1]

Step	Timeline	Party	Deliverables
Design Package + Variance Application Preparation	4-6 months	Engineer	• Site Evaluation/Percolation Test Result • IWS Calculation • Parcel map • Plot Plan • Simple Building Floor Plan for number of bedroom determination • IWS Layout • IWS Profile • IWS Details • Owner Certification form • $100 IWS Application Fee
Design Package Review	2-4 weeks	DOH	Letter of Approval
Variance Application Review	2 months	DOH	Letter of Approval for Construction
Construction	1-2 weeks/property	Contractor	Completed IWS
Final Inspection and Final Inspection Report (FIR)	1 day/property 1 month/report	Engineer	• Final Inspection Report (FIR) • As-Builts
FIR Review	1 month	DOH	Approval for Use

[1] Timelines are estimates, as it needs to be clarified if the design/DOH review is for the group of 10 lots or all 194 lots.

7.1 Project Schedule

Phasing for this project should reflect deadlines as well as workforce and product availability. The AOC requires the County to implement the selected wastewater treatment alternative for the 194 properties and to close the Nāʻālehu Community Cesspools no later than December 31, 2027. The County still needs to make a decision if the project will be bid out in groups of IWS (multiple bid packages) or if the project will be bid out in its entirety. Two different options for the project schedule are proposed as follows:

- Scheduling of the sites could be in accordance with the three currently open LCCs, which are displayed in Figure 1.22 below. The smaller two LCCs, LCC 4 and 5, which serve 16 and 8 lots respectively, could be grouped together to streamline the bidding process and timeframe. The construction of the smaller LCCs could also be treated as a pilot stage, with lessons learned to be integrated into the design of LCC 3, which serves approximately 138 lots. This option would streamline the amount of coordination required to inspect each of the three groups of connected properties and is displayed in Table 1.7.

During construction, lots should be grouped and approached with a top-down approach, starting with lots on the north end of Nāʻālehu and working down, keeping the previous sewer mains in mind and keeping the existing collective system active. This will enable the households downstream to not be affected by the construction. Multiple contractors could be awarded construction contracts for each bidding cycle which overlaps for increased time efficiency. The County could also choose to do the procurement and construction process separately for each of the three LCCs, starting with the smallest, LCC 5 with 8 lots as a pilot phase, and assign multiple contractors to each LCC. However, this would increase the total time of the project due to the additional round of bidding that would occur from ungrouping the construction of LCC 4 and 5.

- A pilot consisting of 10 sites consisting of LCC connected properties could be first permitted and constructed prior to the remaining parcels. Lessons learned from the pilot would be integrated into the design and permitting of the next phase of 154 LCC connected properties. Multiple contractors would be awarded construction contracts in 10-lot bundles that may be completed simultaneously.

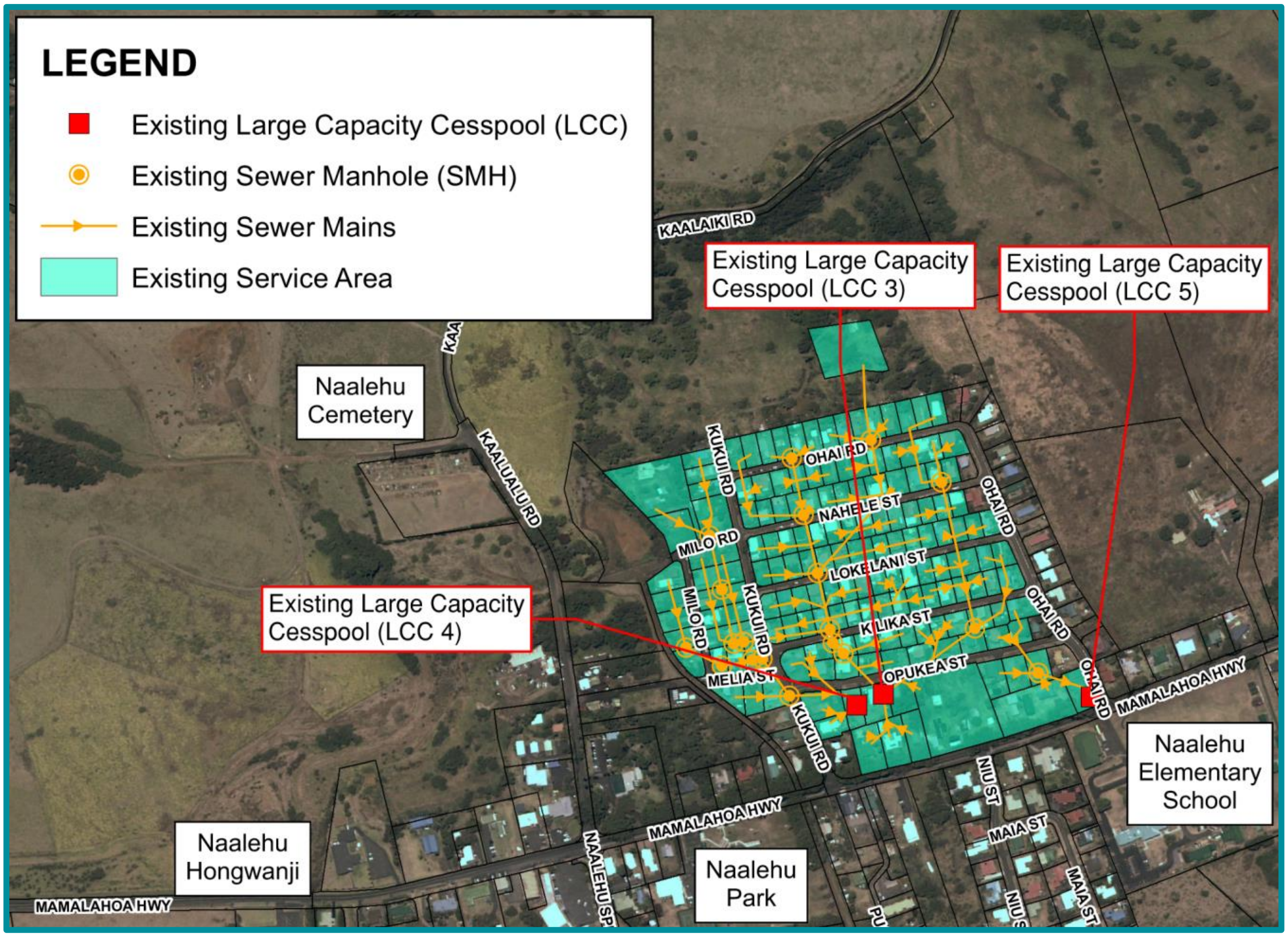
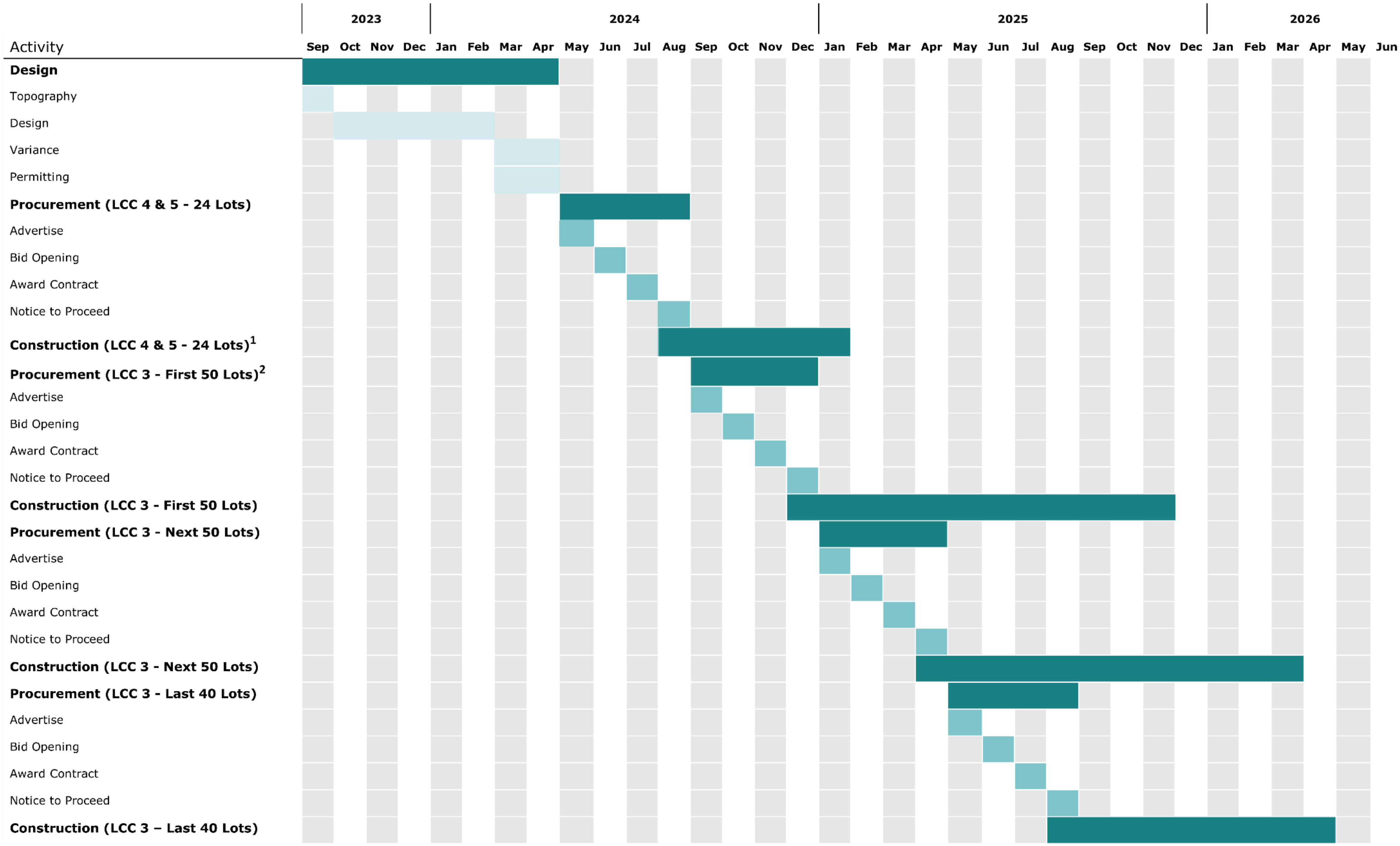


Figure 1.22: Existing Large Capacity Cesspools: LCC 4, LCC 3, and LCC 5 (Brown & Caldwell, 2023).

Table 1.7: Nāʻālehu LCC Replacement Schedule



[1] IWS construction time is an estimate. For a single contractor, it is estimated an IWS can be installed at a rate of one site per week for simple sites, and one site every two weeks where soil conditions and accessibility are less favorable. However contractors could have multiple crews which would allow them to complete more lots per week.

[2] Grouping of lots for LCC 3 can be modified to best keep the existing collection system active.

8. IWS Constructability Challenges & Solutions

8.1 Lack of Yard Space

Lots to be served in the Nāʻālehu community vary in size from 0.12 to 1.94 acres with a median size of 0.16 acres. Space available for IWS installation on these properties is further limited by the presence of existing structures. As a solution, seepage pits can be installed in lieu of absorption beds in space constrained lots, and IWS installations could be installed encroaching into County Right-of-Way, if permitted by Hawaiʻi County.

8.2 Landscaping

Wastewater infiltration and dispersion can be affected by landform position, and thus the selection of the IWS location needs to account for each site's unique landscape during the design and construction phase.

Additionally, disturbance of existing landscape can occur during the installation of an IWS, which can be a relatively invasive process requiring large equipment like excavators, dump trucks and cranes. Front-yard IWS installations can be utilized as an alternative to back-yard installations if this helps to minimize the disturbance of landscape. Thorough site assessments can help identify existing landscape features such as vegetation, trees, retaining walls, fence walls and natural drainage patterns which can help preserve key elements in the design. Trees may be temporarily removed and properly stored, and then later reinstalled to restore vegetation back to the original location. Speed and care in the restoration of any displaced and disturbed landscapes should be taken to increase homeowner satisfaction and to restore the site to pre-construction conditions or better.

8.3 Ground slope

When analyzing the topography of the site, key features offer more design flexibility than others. Long, planar slopes or plateaus should be prioritized when constructing an IWS over ridges, knolls, or other mounded or steeply sloping sites. Considering the ground slope is crucial in the design of an absorption bed to be downstream of the dwelling and septic tank. According to the EPA Manual, subsurface flows can accumulate in the presence of swales, depressions, or floodplains, and optimal landscape positions consist of convex slopes and flat areas with deep, permeable soils (EPA, 2002).

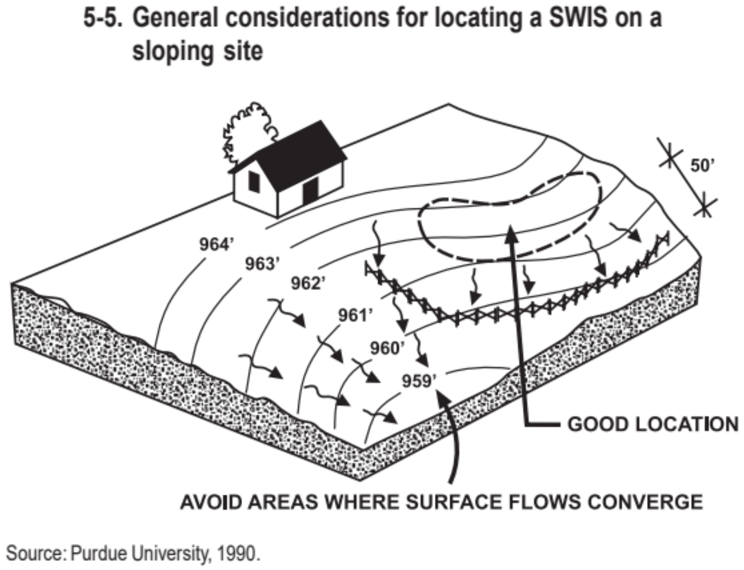


Figure 1.23: EPA Manual recommendation for placement of subsurface wastewater infiltration system (SWIS)

Sites in Nāʻālehu have slopes that vary from 6-10% (Appendix C). This is likely to affect the constructability of absorption beds as a method of wastewater disposal. Per HAR 11-62-34, absorption beds shall not be installed on land with a slope gradient greater than 8%, while absorption trenches are permitted on a slope of up to 12%. In case this slope requirement cannot be fulfilled, the DOH would allow a seepage pit to be installed instead of an absorption bed.

8.4 Cut Slope

Constructing an individual wastewater system (IWS) near a cut face can present challenges, particularly in relation to potential side seepage. The excavation or cut face may pose a risk of allowing wastewater to permeate the surrounding soil or potentially contaminate nearby groundwater sources.

To mitigate potential side seepage into adjacent cut face slopes, the IWS should be designed to consider the proximity to the cut face by adjusting the location and depth of the components, such as septic tanks or absorption beds. Installing the bottom of the absorption bed 20 feet away from the cut face slope ensures adequate distance for potential side seepage not to occur.

8.5 Site Geology

The National Resources Conservation Service Soil Survey was consulted for site soils information (Appendix D). The site is principally composed of Nāʻālehu medial silty clay loam and Puueo-Nāʻālehu complex. The surface of both compositions features silty loam, but the Puueo-Nāʻālehu complex gives way to lithic bedrock at a depth of approximately 20-40 inches.

This data was reinforced by the boring logs from a 2007 Geotechnical Investigation Report conducted by Masa Fujioka & Associates (Appendix E):

> ***"The Naalehu site has been mapped by the SCS as consisting of NaC and NaD, Naalehu series soils.***
>
> ***NaC and NaD are Naalehu series soils, which consists of well-drained silty clay loams that formed in volcanic ash. This soil normally overlies pahoehoe or a'a lava flows at depths of more than 40 inches.***
>
> ***During our site reconnaissance, we noted that the Naalehu site had been graded to form terraces with a road and adjacent homes on each terrace. Between terraces, we noted that the between terrace slopes consist of relatively thin near surface volcanic ash soils overlying basaltic rock.***
>
> ***The boring at the Septic Tank site area was drilled to a depth of 25 feet. This boring encountered topsoil (one foot) overlying ash soil to a depth of 4.5 feet. Broken and highly vesicular basaltic rock was encountered for the first 15 feet. The deeper rock appeared to become moderately fractured and vesicular, with higher recovery and RQD."***

Figure 1.24: Boring hole locations in Nāʻālehu, adapted from Masa Fujioka & Associates.

Figure 1.24 displays the boring hole locations from the analysis conducted by Masa Fujioka & Associates in Nāʻālehu. The detailed boring logs of these seven locations is located in Appendix E.

The underlying basalt found at the site will significantly increase the size and installation cost of IWS. As a solution, the engineer could show in the bid document/construction drawing that basalt rock layer is likely to be encountered, where the depth of the rock layer varies. The Contractor should price in dealing with rock layer in the bid. Further, it will be important to exercise caution during excavation due to the potential to encounter underground cavities and lava tubes. Appendix G outlines the proposed solution if lava tubes are encountered.

8.6 Soil Permeability: Percolation Test Results

IWS sizing is based on the percolation rate of the receiving soil. Percolation rates were conducted on July 31st and August 7th, 2023, at the Nāʻālehu site and ranged from 7-15 minutes/inch. The tests were conducted to a depth of 4-4.5 feet at four sites distributed across the project (Figure 1.25). Mainly soft dirt with occasional rocks with varying sizes was encountered during the tests with the full percolation test reports included in Appendix H.

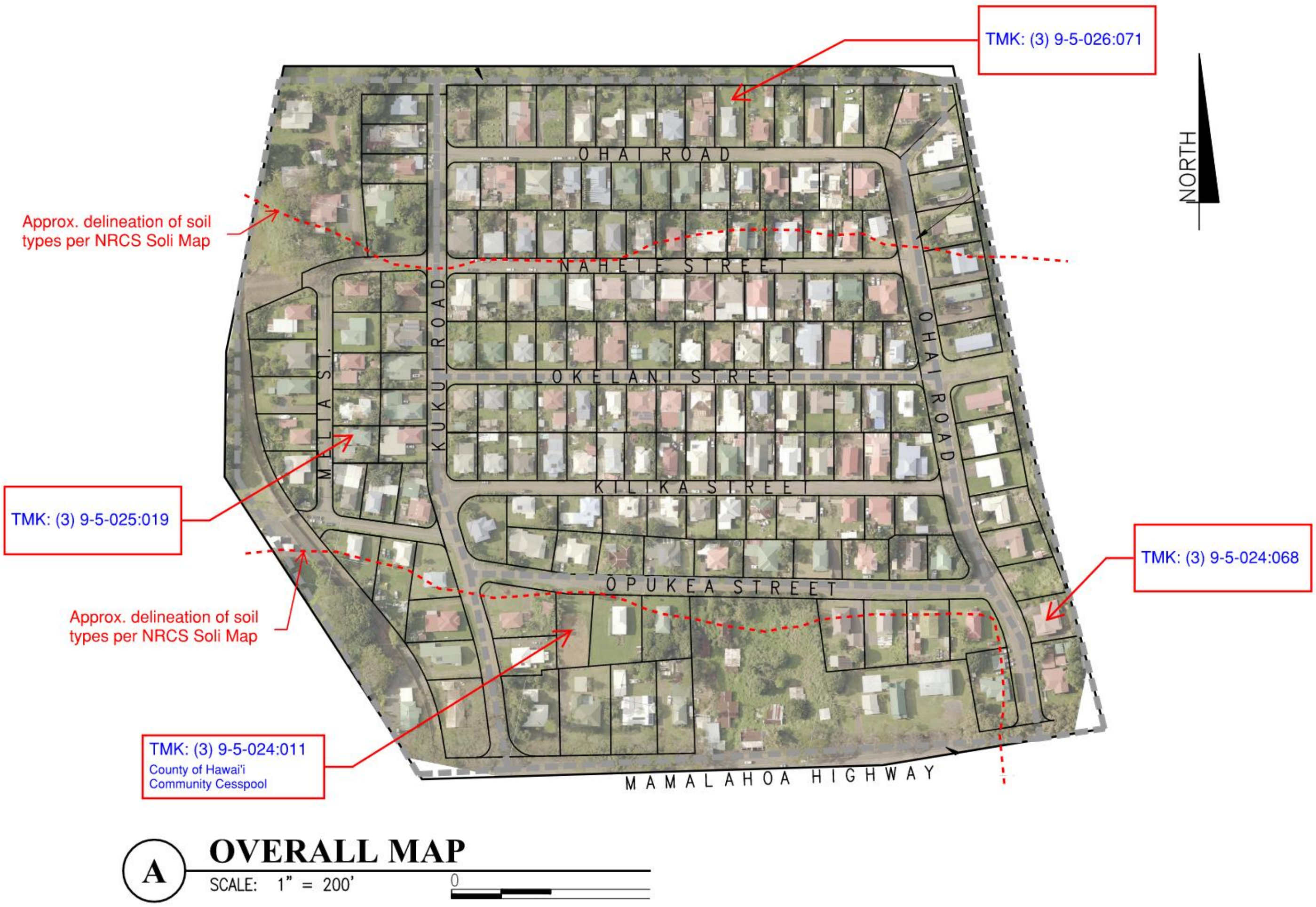


Figure 1.25: Percolation test locations in Nā'ālehu.

Table 1.8: Percolation test results

Measure	TMK Number	2023 EPI Nāʻālehu LCC Replacement PER (min/in)	Test Date
Test 1	(3) 9-5-025:019	10 @ 4 ft	07/31/23 8:00am
Test 2	(3) 9-5-024:011	7 @ 4.5 ft	07/31/23 1:30pm
Test 3	(3) 9-5-024:068	8 @ 4 ft	08/07/23 12:00pm
Test 4	(3) 9-5-026:071	15 @ 4ft	08/07/23 8:00pm

8.7 Traffic Area

It is generally not good practice to install an IWS under a trafficked or otherwise concreted area. The presence of concrete or traffic compresses the soil in distribution systems and affects the accessibility of the system for maintenance. However, it is sometimes unavoidable on particularly spatially constrained properties. In this event, a system may be installed underneath a driveway or vehicle path of travel provided the system is designed to that end and traffic rated treatment components are used. These may include products such as concrete septic tanks and/or H-20 traffic related chambered disposal beds.

Furthermore, for front-yard IWS installations, it would be necessary to incorporate traffic-rated IWS components in potential parking areas. One alternative could be constructing barriers (wall, bollards, etc.) to prevent vehicle traffic over the system, allowing the need for traffic-rated IWS components to be alleviated.

8.8 Existing Building Structures

Existing structures limit available space for IWS installation. There is a possibility that some of the existing structures are unpermitted, but the amount/extent of unpermitted structures present is unknown. Not only can structures obstruct access points to the IWS components, making maintenance and servicing more challenging, unpermitted building additions can increase the wastewater flow to the IWS beyond its original design capacity. Previous conversations with the State of Hawaiʻi Department of Health have concluded that the design of each in Nāʻālehu should only account for the number of permitted bedrooms for each TMK parcel. Unpermitted buildings will not contribute to the design of the IWS capacity. Unpermitted structures will require further coordination and discussions between the County and homeowner on addressing unpermitted structures and including existing permitted obstructing structures that may need to be temporarily removed and reconstructed.

8.9 Access of Large Construction Equipment

Excavators, loaders, and cranes are examples of some of the large construction equipment needed to install an IWS. Access for this equipment is often obstructed by low roof overhang, blockage by permanent structures, overhead utilities, vegetation, or from insufficient path widths. Accommodating such equipment on especially small lots often requires temporarily removing and reconstructing of fencing, walls, existing landscaping, and in some cases small structures.

Access should be considered for installation as well as future maintenance activities. Opportunities to resolve access issues include:

- **IWS Placement:** Front-yard installations are recommended for homes without sufficient path widths to accommodate equipment access into the backyard.
- **Lightweight or Cast-in-Place Technologies:** The use of a crane can be avoided by specifying cast-in-place concrete septic tanks instead of precast varieties for particularly inaccessible locations. Alternatively, plastic and fiberglass offer lightweight alternatives for simplified installation.
- **Alternate Access Routes:** Access to a property's backyard may be accessed from an adjoining neighbor's property. Such access will need to obtain permission of the neighboring homeowner.

8.10 Access of Maintenance Equipment

IWS maintenance typically includes regular inspections, septage pumping, and periodic cleaning when a traditional septic tank is used. If systems are not inspected, septic tanks should be pumped every 3 to 5 years depending on the size of the tank, the number of building occupants, and household appliances and habits (EPA, 2002). In order to remove wastewater and/or sludge from the tank, the lid must be removed, and the tank is pumped through an access port. Debris surrounding the system must be removed, and access must be available for the pump hose which will pump wastewater to the pump truck parked in close proximity. Pump hoses are typically 150 feet long and can usually access the backyard of a property from the County Right-of-Way where the pump truck is parked.

8.11 Coordination with Landowners or Tenants

The successful completion of this project necessitates that homeowners/tenants cooperate in the design, permitting process, and during the construction phase of the project. It is recommended for the County to continuously notify landowners or tenants of project milestones, such as the selection of which IWS system will be installed. The County will obtain individual Right-of-Entry (ROE) forms to access properties during the design site assessment and topographic surveys. Separate ROE forms will be obtained for construction activities during the construction phase. The County will need to maintain a continuous dialogue and coordination with homeowners/tenants through the duration of the project for the project to be successful.

Potential challenges can occur if the homeowner has a preference on the location of the IWS. If the IWS is not in a favorable location for the homeowner, the IWS layout can be moved to a location with less disturbance to the homeowner to try to accommodate this request. Solutions may include altering the system layout by adding pumps, excavating deeper, etc.

8.12 Absorption Beds vs. Seepage Pits

When setback requirements and absorption bed sizing requirements cannot be reached due to lack of horizontal yard space, seepage pits can be employed to dispose of wastewater in a vertical manner.

Based on a visual walk through within the ROW roadway of the site on July 20, 2023 (Section 9), a preliminary estimate of 83 seepage pits will need to be installed in lieu of absorption beds. This estimate was made by visual inspection and would need to be confirmed by further surveys, as backyard space was not always visual from the public roads. Sites lacking the necessary space in the backyard for an absorption bed were assumed to require a seepage pit, unless significant space was present in the side or front yard. Appendix A estimates the yard area required for an absorption bed, which ranged from 330 sq. ft. to 1,330 sq. ft., based on the upper range of the percolation rate of the soil found (10 and 15 minutes/inch) and the number of permitted bedrooms in each dwelling.

8.13 Availability of Resources and Contractors

As an island state, Hawaiʻi faces unique challenges when it comes to the availability of IWS. The entire IWS market in Hawaiʻi grew from 1192 units per year in 2018 to 1414 units per year in 2021. Based on permits issued, sourcing the 194 treatment units required for this project will require an increase in statewide treatment unit supply and workforce size by 14% (DOH, 2022). This will require advanced planning to overcome this logistic hurdle.

Locally based septic manufacturers include Jensen Precast on Oʻahu and Chemtainer on Hawaiʻi Island. At present, Jensen produces approximately 40 concrete septic tanks per year. With that said, the company has stated that they have the capacity to build one septic tank per day to keep up with the needs of the project. The bottleneck to their current production rate has been cited as a shortage of inspectors, contractors, and engineers in the local market. Chemtainer manufactures polyethylene septic systems on Hawaiʻi Island but was unwilling to share their annual production rates. Mainland treatment system manufacturers like Orenco and Infiltrator have significantly higher production rates but will also face increased shipping costs in transporting the units to Hawaiʻi. Material availability and delivery will be the critical path item during construction, and the design of the IWS needs to ensure the possibility of different manufacturers being utilized is accommodated.

9. Technically Challenging Sites

A visual walk-through within the ROW was conducted at Nāʻālehu by EPI on July 20th, 2023. The aim of this site visit was to identify common conditions that are present in the community that would render an IWS installation technically difficult, due to the challenges of IWS construction. Five properties were initially flagged during the walk-through to have such challenging conditions, however the total number of properties with potential technical challenges is not limited to following households identified. Each dwelling may or may not have its own unique challenges and this section serves as an example of which challenges may be encountered during IWS design and construction.

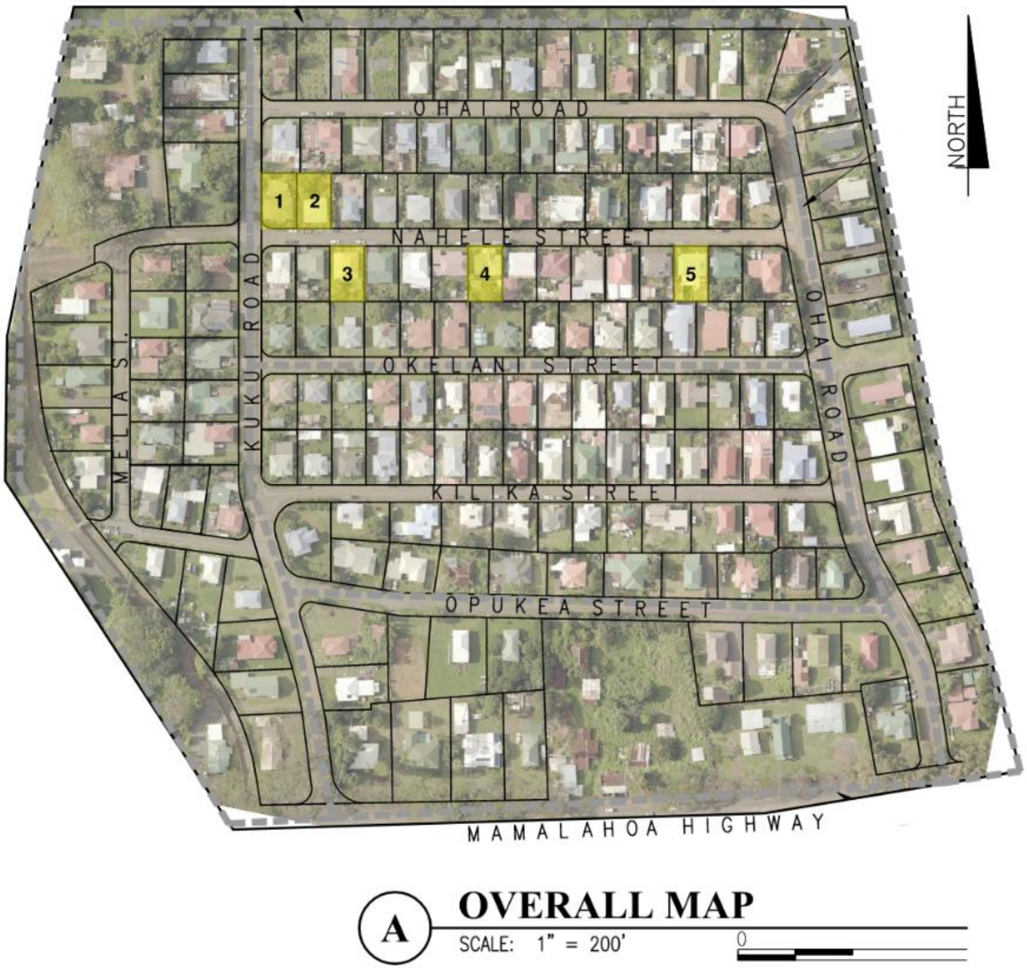


Figure 1.26: Examples of sites with technically challenging conditions are marked in yellow (but are not limited to these five properties).

Table 1.9: Examples of Nāʻālehu Problematic Sites

Site	Problem Description	Photo
1	Site 1 has physical barriers such as concrete retaining walls and stairs that limit available space for IWS installation and will be costly to remove and reinstall. The IWS components will likely need to be installed under the driveway, which is at a higher elevation than the house, which may require a pump to be included in the system design. The IWS will also need to be traffic-rated. The IWS components could also be installed 3-4 feet deeper than usual under the driveway to ensure proper wastewater drainage from the household, and to negate the need for a pump. The driveway will not allow sufficient space for an absorption bed, and thus a seepage pit will need to be installed in addition to the septic tank. Installing a seepage pit in lieu of an absorption bed will require a variance application to the DOH. The seepage pit will also not meet the defined setback requirements to the property line and existing structures and will thus require an additional variance application.	 Address: 95-1204 Kukui Rd, Nāʻālehu, HI 96772
2	Site 2 has limited available accessible space for IWS installation. Communication with the homeowner will need to be made to see which household items can be relocated or removed to ensure space for the IWS components. The front and side yards do not allow sufficient space for an absorption bed, and thus a seepage pit will need to be installed in addition to the septic tank. Installing a seepage pit in lieu of an absorption bed will require a variance application to the DOH. The seepage pit will also not meet the defined setback requirements to the property line and existing structures and will thus require an additional variance application.	 Address: 95-5586 Nahele St, Nāʻālehu, HI 96772
3	Site 3 is situated on a slope, which may be difficult to maneuver with installation equipment. During installation, a temporary ramp may need to be constructed with material such as gravel for the equipment to be able to reach the lower level of the yard. Physical barriers such as the estimated 6-7 feet high retaining wall may need to be removed to access the available yard space. In addition, this site contains a basement that is estimated to 6 feet below the street grade, which will require deeper installation of IWS components or a pump.	 Address: 95-5581 Nahele St, Nāʻālehu, HI 96772

	The available yard space and driveway does not allow sufficient room for an absorption bed, and thus a seepage pit will need to be installed in addition to the septic tank. Installing a seepage pit in lieu of an absorption bed will require a variance application to the DOH. The seepage pit will also not meet the defined setback requirements to the property line and existing structures and will thus require an additional variance application.	
4	Site 4 has limited yard space for installation in the front and sides, and the front yard is difficult to access due to physical barriers and the sloped nature of the site. The driveway/parking area is located up higher than the house and the backyard drops in elevation, which would require the septic system be installed deep into the ground, if the backyard is not accessible by neighboring sites. The property does not have sufficient space for an absorption bed, and thus a seepage pit will need to be installed in addition to the septic tank. Installing a seepage pit in lieu of an absorption bed will require a variance application to the DOH. The seepage pit will also not meet the defined setback requirements to the property line and existing structures and will thus require an additional variance application.	 Address: 95-5573 Nahele St, Nāʻālehu, HI 96772
5	Site 5 has limited yard space, is situated on a slope, and contains physical barriers that will need to be removed. The basement level is lower than the street, which may require a pump to be installed in the IWS system if a front-yard installation is chosen, or a deeper installation. This site also appears to be abandoned. The driveway and front yard do not have sufficient space for an absorption bed, and thus a seepage pit will need to be installed in addition to the septic tank. Installing a seepage pit in lieu of an absorption bed will require a variance application to the DOH. The seepage pit will also not meet the defined setback requirements to the property line and existing structures and will thus require an additional variance application.	 Address: 95-5557 Nahele St, Nāʻālehu, HI 96772

10. Operational Considerations

An effective Individual Wastewater System (IWS) management strategy is crucial to ensuring distributed treatment systems are maintained and operated in a way that ensures they are functioning properly and effectively treating wastewater. This strategy may include but is not limited to:

- **Monitoring:** Regular inspections of system components, such as septic tanks and drain fields, ensure they are functioning properly and identify and address any issues that may arise.
- **Maintenance:** Proper maintenance of the system is also crucial, including regular pumping of septic tanks, cleaning and maintenance of the distribution systems, and proper maintenance of the treatment components. Regular maintenance can help prevent issues such as clogs and backups, which can lead to costly repairs and potential health hazards.
- **Regulatory Compliance:** Necessary permits and licenses are obtained for the system, and required inspection and reporting schedules are met with the local regulator.
- **Community Education:** Information and training on proper usage and maintenance of the systems are provided to homeowners, and any concerns or questions that may arise are addressed.

In Hawaiʻi, centralized wastewater treatment plants and cluster systems are regulated and inspected by the Department of Health (DOH) Wastewater Branch (Hawaiʻi Administrative Rules 11-62).

State-licensed WWTP operators are required for oversight of Wastewater Treatment plants to ensure that systems are inspected, operated, and maintained as required. A similar regulatory requirement does not exist for IWS in Hawaiʻi. The State DOH Wastewater Branch is responsible for regulating IWS while operation and maintenance are currently the responsibility of the individual homeowner. If IWS were selected to serve Nāʻālehu Community, maintenance responsibilities could be distributed in a number of ways. Per Voluntary National Guidelines for Management of Onsite and Clustered (Decentralized) Wastewater Treatment System, the EPA outlined five management models that can be used for the operation and maintenance of IWS (Table 1.10).

When selecting an appropriate management model for a network of IWS it is important to take into account the regulatory and cultural framework within which the IWS is situated. As it stands in Hawaiʻi, IWS are currently managed similar to a combination of management Models 1 and 2 (DOH, 2016):

- **Model 1: Homeowner Awareness.** The DOH allows septic systems to be managed by the homeowner under this model. Homeowners own and operate their own IWS and are responsible for keeping the system in good working order.

- **Model 2: Maintenance Contracts.** The DOH requires that Aerobic Treatment Units

(ATUs) are managed by a state licensed wastewater operator using this model. Homeowners are required to have an active service contract with a certified operator or factory certified representative, and a copy of that active service contract must be submitted annually to the DOH (DOH, 2016). Elevated regulation around the operation of ATUs is a reflection of their increased mechanical complexity and associated maintenance demands.

Table 1.10: The five management models for IWS maintenance (EPA, 2003)

The Five Management Models				
Model 1	**Model 2**	**Model 3**	**Model 4**	**Model 5**
Homeowner Awareness:	**Maintenance Contracts:**	**Operating Permits:**	**Responsible Management Entity (R:ME) Operation and Maintenance:**	**RME Ownership:**
specifies appropriate program elements and activities where treatment systems are owned and operated by individual property owners in areas of low environmental sensitivity. This program is adequate where treatment technologies are limited to conventional systems that require little owner attention. To help ensure that timely maintenance is performed, the regulatory authority mails maintenance reminders to owners at appropriate intervals.	specifies program elements and activities where more complex designs are employed to enhance the capacity of conventional systems to accept and treat wastewater. Because of treatment complexity, contracts with qualified technicians are needed to ensure proper and timely maintenance.	specifies program elements and activities where sustained performance of treatment systems is critical to protect public health and water quality. Limited-term operating permits are issued to the owner and are renewable for another term if the owner demonstrates that the system is in compliance with the terms and conditions of the permit. Performance--based designs may be incorporated into programs with management controls at this level.	specifies program elements and activities where frequent and highly reliable operation and maintenance of decentralized systems is required to ensure water resource protection in sensitive environments. Under this model, the operating permit is issued to an RME instead of the property owner to provide the needed assurance that the appropriate maintenance is performed.	specifies that program elements and activities for treatment systems are owned, operated, and maintained by the RME, which removes the property owner from responsibility for the system. This program is analogous to central sewerage and provides the greatest assurance of system performance in the most sensitive of environments.

The AOC stipulates that the County of Hawaiʻi must administer a more active management strategy than is typical in Hawaiʻi, either a Model 2 (Maintenance Contract) or Model 3 (Operating Permit) management strategy for a network of IWS at Nāʻālehu. These models reflect varying degrees of responsibility to the County and homeowner (Table 1.11). Four potential variations of these models are outlined here for implementation on this project:

- **Management Model 2A: Maintenance Contract with County In-House Staff**
  The County employs and trains an in-house IWS management team; purchases and maintains its own pumping/hauling equipment; and administers the management program. The homeowner pays a monthly sewer fee that covers a portion of the costs.
- **Management Model 2B: Maintenance Contract with Third-Party Service**
  The County administers the management program, keeps an operations and maintenance (O&M) schedule, and contracts out O&M activities to a third-party service provider. The homeowner pays a monthly sewer fee that covers a portion of the costs.
- **Management Model 3A: Operating Permits with O&M by Users**
  The County issues an operating permit to the homeowner; keeps an O&M schedule; and sends out maintenance reminders to homeowners. The homeowner is responsible for contracting a third-party service provider to conduct maintenance.
- **Management Model 3B: Operating Permits with O&M Voucher by County**
  The County issues an operating permit to the homeowner; keeps an O&M schedule; and sends out maintenance reminders with service vouchers to homeowners. The homeowner pays a sewer fee and is responsible for contracting a third-party service provider to conduct annual maintenance using the voucher.

These management strategies presented here are required by the AOC but are also unique to Hawaiʻi and will present a number of barriers for implementation at the legislative, regulatory, and public levels. The Nāʻālehu Community and Hawaiʻi's stakeholders at large are accustomed to Management Model 1, which is the standard practice across the State of Hawaiʻi.

Table 1.11: County and homeowner responsibilities under variations of the EPA Management Models 2 and 3

Management Model	Brief Description of Management Model	County's Responsibility	Homeowner / User's Responsibility	Pros	Cons
2A	Maintenance Contract w/ County in-house staff	• Funds design and construction of IWS • Purchase equipment & train IWS operator • O&M of IWS including trouble calls • Keeping record of O&M log • Send out notices and reminders to homeowners • Submit IWS inspection reports and variance renewals to State DOH	• Report IWS problem to County • Cooperates and allows County staff to enter private property and provide maintenance of IWS • Maintain clearance to IWS for easy access	1. Best control on O&M schedule 2. Ensure best IWS performance	1. Highest cost 2. May not receive cooperation from some homeowners/users 3. Homeowner may have more trouble calls 4. Potential dispute between homeowner & County on plumbing repair cost & IWS repair cost
2B	Maintenance Contract w/ 3rd Party Service	• Funds design and construction of IWS • Select/prequalify certain 3rd party service provider (Pumper) • Issue PO to Pumper & plumber for annual inspection and trouble calls • Keeping record of O&M log • Send out notices and reminders to homeowners • Submit IWS inspection reports and variance renewals to State DOH	• Report IWS problem to County • Cooperates and allows service providers to enter private property and provide maintenance of IWS • Maintain clearance to IWS for easy access	1. Better control on O&M schedule 2. Ensure better IWS performance 3. Less County staff to train 4. No pumping/hauling equipment to purchase & maintain	1. Higher cost 2. May not receive cooperation from some homeowners/users 3. Homeowner may have more trouble calls 4. Potential dispute between homeowner & County on plumbing repair cost & IWS repair cost
3A	Operating Permits w/ O&M by Users	• Funds design and construction of IWS • Keeping record of O&M log • Send out notices and reminders to homeowners • Enforce rules and regulations • Issues permit to homeowner to use, operate & maintain the IWS	• Contracts with preferred pumper / plumber to maintain the IWS • Pay for the O&M service • Submit O&M record to County • Submit IWS inspection reports and variance renewals to State DOH	1. Least cost to County 2. No O&M staff or equipment 3. No trouble calls	1. Least control for IWS compliance & performance 2. Conflict with non-compliant homeowners 3. Highest cost to homeowner
3B	Operating Permits w/ O&M Voucher by County	• Funds design and construction of IWS • Keeping record of O&M log • Send out notices and reminders to homeowners • Enforce rules and regulations • Pre-select qualifying service providers • Issue vouchers to homeowners for annual inspections and pumping • Issues permit to homeowner to use, operate & maintain the IWS	• Contracts with preferred pre-qualified pumper / plumber to maintain the IWS • Pay for the annual O&M service with voucher • Submit O&M record to County by pumper • Submit IWS inspection reports and variance renewals to State DOH	1. Reasonable control on O&M 2. Reasonable IWS performance 3. Less County staff to train 4. No pumping/hauling equipment to purchase & maintain 5. Trouble calls to be paid for by homeowner	1. High cost to County 2. Less control of all IWS

11. Economic Considerations

When assessing the overall cost of a given system, it is important to consider the net present value lifecycle cost taking system lifetime and installation, maintenance, and operation costs into account. These costs can be affected by a variety of factors, including:

- **Type of treatment and disposal system:** The selection of different types of treatment systems such as traffic rated tanks or aerobic treatment significantly affects overall installed cost. For disposal, seepage pits are significantly lower cost than absorption fields when it is possible to convert an existing cesspool. Site specific conditions will control which options are required. For residential IWS installations subject to State procurement regulations, capital costs per household are typically in the range of $30,000-$100,000 (Table 1.12). Due to the potential need for repair and reconstruction of property site elements (fencing, walls, structures, etc.) that may need to be removed during IWS installation, the cost per household in Nāʻālehu can be up to $150,000. At this stage of the project, it is fully unknown the extent of reconstruction of existing site elements, and this would need to be further determined during the design phase of the project, which can impact the stated estimated cost.

Table 1.12: Installation cost estimates for a standard septic tank installed in conjunction with an absorption bed (left) and seepage pit (right). Figures are based on a 3-bedroom house and a percolation rate no slower than 5 min/inch.

	Standard Absorption Bed		Seepage Pit	
	Low (non-traffic)	High (Traffic Rated)	Low (non-traffic)	High (Traffic Rated)
Septic Tank	3,000.00	7,000.00	3,000.00	7,000.00
D-Box	750.00	2000 .00	–	–
Sewer pipe	250.00	250.00	250.00	250.00
Leach field-pipe/ chamber	500.00	3,000.00	–	–
Leach field-gravel	1,000.00	500.00	–	–
Cone. Ring	–	–	3,000.00	3,000.00
Cone. Cover	–	–	2,500.00	4,000.00
Soil replacement	1,500.00	1,500.00	–	–
Inspection ports	500.00	500.00	–	–
Misc. material	2,000.00	2,000.00	2,000.00	2,000.00
Material Total	**$ 9,500.00**	**$ 16,750.00**	**$ 10,750.00**	**$ 16,250.00**
Labor / Equipment	7,500.00	15,000.00	7,500.00	15,000.00
Remoteness	5,000.00	5,000.00	5,000.00	5,000.00
Trucking for spoils	3,000.00	3,000.00	3,000.00	3,000.00
Tight working space	3,000.00	10,000.00	3,000.00	10,000.00
Relocate/reinstall/ repair	5,000.00	100,000.00	5,000.00	100,000.00
TOTAL	**$ 33,000.00**	**$ 149,750.00**	**$ 34,250.00**	**$ 149,250.00**

- **Operations and Maintenance cost:** Operations and maintenance cost also play a big role in the overall cost of IWS. Annual maintenance costs to the County and homeowner vary depending on the management strategy. It is estimated that bringing maintenance in-house is the most affordable option (Table 1.13). Annual costs over a 20-year service lifetime are further expounded in Appendix B.

Table 1.13: The costs associated with four IWS management models, assuming septic systems with leach fields (Appendix B).

Management Model	Average Annual Cost to County		Average Annual Cost to Homeowner		Net Annual Cost to County	Total Annual Dollars Spent
	Third-Party Service Provider	In-House	Third-Party Service Provider	County Sewer Bill[1]		
2A: Maintenance Contract w/ County in-house staff	–	($956)	-	$600	($356)	($956)
2B: Maintenance Contract w/ 3rd Party Service	($783)	($572)	-	$600	($755)	($1,355)
3A: Operating Permits w/ O&M by Users	–	($572)	($733)	-	($572)	($1,305)
3B: Operating Permits w/ O&M Voucher by County	($533)	($572)	-	$600	($505)	($1,105)

[1] The annual average County sewer bill is based on an average of $50/month sewer bill for households in Hawaiʻi.

References

Babcock, R., Barnes, M., Fung, A., Goodell, W., & Oleson, K. (2019). ***Investigation of Cesspool Upgrade Alternatives in Upcountry Maui Final Report***. University of Hawaii Manoa.

Babcock, R., Lamichhane, K. M., Cummings, M. J., & Cheong, G. H. (2014). Condition assessment survey of onsite sewage disposal systems (OSDSs) in Hawaii. ***Water Science and Technology***, 70(6), 1083–1089. https://doi.org/10.2166/wst.2014.336

Carollo Engineers (2021). ***Cesspool Conversion Technologies Research Summary Report***. Hawaii State Department of Health.

Hawaii County Code, Chapter 21 Sewers (2016). https://www.hawaiicounty.gov/home/showpublisheddocument/46/637032750278570000

Hawaii State Department of Business, Economic Development and Tourism. (2021). ***The State of Hawaii Data Book: A Statistical Abstract***. State of Hawaii Department of Business, Economic Development, and Tourism. https://files.hawaii.gov/dbedt/economic/databook/db2021/DB2021_final.pdf

Environmental Protection Agency. (1996, June). ***Wastewater Treatment: Alternatives to Septic Systems***. Environmental Protection Agency. https://nepis.epa.gov/Exe/ZyPURL.cgi?Dockey=20013JJ8.txt

Environmental Protection Agency. (2002). ***USEPA Onsite Wastewater Treatment Systems Manual***. In *United States Environmental Protection Agency* (pp. 1–367). https://www.epa.gov/sites/default/files/2015-06/documents/2004_07_07_septics_septic_2002_osdm_all.pdf

Environmental Protection Agency (2003). ***Voluntary National Guidelines for Management of Onsite and Clustered (Decentralized) Wastewater Treatment Systems***. Environmental Protection Agency. https://www.epa.gov/sites/default/files/2015-06/documents/septic_guidelines.pdf

Environmental Protection Agency. (2018) ***Types of Septic Systems*** [Overviews and Factsheets]. https://www.epa.gov/septic/types-septic-systems

Environmental Protection Agency (2010). Code of Federal Regulations, 40 C.F.R §144.80 Title 40—Protection of Environment. https://www.govinfo.gov/content/pkg/CFR-2014-title40-vol23/xml/CFR-2014-title40-vol23-part144.xml

Evergreen. (2023). ***Separett Villa urine separating toilet***. Evergreen. https://evergreen.eco/en-us/products/urine-separating-toilet-villa-9020-12v-230v

Hawaii State Department of Health. (2009). ***Guidelines for the Reuse of Gray Water***. Hawaii Department of Health Wastewater Branch. https://health.hawaii.gov/wastewater/

files/2016/03/14_Gray_Water_GL.pdf

Hawaii State Department of Health. (2016) HAR 11-62 Wastewater Systems. https://health.hawaii.gov/opppd/files/2015/06/11-62-Wastewater-Systems.pdf

Hawaii State Department of Health. (2017). ***Relating to Cesspools and Prioritization for Replacement***. https://health.hawaii.gov/opppd/files/2017/12/Act-125-HB1244-HD1-SD3-CD1-29th-Legislature-Cesspool-Report.pdf

Hawaii State Department of Health. (2022). ***IWS Final Approval Date Report (04/01/2017-04/01/2022)***. Hawaii State Department of Health Wastewater Branch.

Incinerating Toilets Inc. (2023). ***Cinderella Comfort Incinerating Toilets***. Incinerating Toilets Inc. https://incineratingtoilets.com/us/product/cinderella-comfort/

Infiltrator. (2023). ***Advanced Enviro-Septic***. Infiltrator Water Technologies. https://www.infiltratorwater.com/products/presby-environmental/advanced-enviro-septic/

Lekven, C. (2019). ***Pahala Wastewater Treatment Plant Preliminary Engineering Report***. Brown and Caldwell. https://www.epa.gov/sites/default/files/2020-03/documents/pahala_final_ea_vol_2_508_2020-02-24.pdf

Mezzacapo, M., & Shuler, C. (2021). 2021 ***Hawaii Cesspool Hazard Assessment and Prioritization Tool***. University of Hawaii. https://health.hawaii.gov/wastewater/files/2022/01/priortizationtoolreport.pdf

Mohamed, R. (2009). Why households in the United States do not maintain their septic systems and why state-led regulations are necessary: Explanations from public goods theory. ***International Journal of Sustainable Development and Planning***, 4, 143–157. https://doi.org/10.2495/SDP-V4-N2-143-157

New Zealand Distributors. (2018). AES Homeowners Manual AES (Advanced Enviro-Septic TM) Owners Manual (pp. 1–12).

Nuflow Wide Bay. (2015, September 15). *Blocked Drains Bundaberg*. Nuflow Wide Bay. https://www.nuflowwidebay.com.au/understanding-the-mysteries-of-septic-tanks/

Peterson, A. (2010, May 2). Plantation Era Is Gone, and Pahala Lives On: Historic Preservationist Julia Neal Uses Hammer and Wood to Pull the Community Together. ***Ke Ola Magazine***. https://keolamagazine.com/community/plantation-era-gone-pahala-lives-on/

Pine Creek Structures. (2023). ***Excel NE Sun-Mar Composting Toilet***. https://www.storageshedspa.com/sheds/other-products/17759/excel-ne-sun-mar-composting-toilet

Presby Environmental, Inc. (2017). *The Presby Wastewater Treatment System Design and Installation Specifications for Advanced Enviro-Septic ®, Enviro-Septic ® and Simple-Septic ® Wastewater Treatment Systems Minimizes the Expense*

Protects the Environment Preserves the Site. wwww.PresbyEnvironmental.com

Presby Environmental, Inc. (n.d.). *The Next Generation of Wastewater Treatment Technology Brochure*.

ShopTinyHouses. (2023). ***Sun-Mar Centrex 3000 Composting Toilet System***. ShopTinyHouses.Com. https://www.shoptinyhouses.com/products/sun-mar-centrex-3000-central-composting-toilet-system

Water Resources Research Center, & Engineering Solutions, Inc. (2008). ***ONSITE WASTEWATER TREATMENT SURVEY AND ASSESSMENT***. In *State of Hawaii, Department of Health*. https://health.hawaii.gov/wastewater/files/2013/06/onsitesurvey.pdf

A

Appendix A - LCC Closure Properties

Naalehu LLC Closure							Absorption Field		
							DOH HAR Chapter 11-62		
					Est. WW		Absorption Field Size (sqr ft)		
TMK#	Address	Land Area (acres)	# of Bldgs	# of Bedrms	Gen. (gpd)	Septic Tank (gal)	for Perc Rate: 10min/in.	for Perc Rate: 15min/in.	Remarks
9-5-008-022	95-1214 KUKUI ROAD	1.0540	1	4	800	1000	660	760	
9-5-025-032	95-1199 A MILO ROAD	1.1810	2	2	400	1000	330	380	
9-5-025-031	95-1211 KUKUI ROAD	0.1818	1	3	600	1000	495	570	
9-5-025-030	95-1209 KUKUI ROAD	0.1818	1	3	600	1000	495	570	
9-5-025-029	95-1207 KUKUI ROAD	0.1818	1	3	600	1000	495	570	
9-5-025-028	95-1201 KUKUI ROAD	0.4198	1	4	800	1000	660	760	
9-5-025-014	95-1199 MILO ROAD	0.4037	1	4	800	1000	660	760	
9-5-025-013	95-1195 MILO ROAD	0.2673	1	3	600	1000	495	570	
9-5-025-012	95-1193 MILO ROAD	0.2335	1	3	600	1000	495	570	
9-5-025-011	95-1189 MILO ROAD	0.2210	1	3	600	1000	495	570	
9-5-025-010	95-1187 MILO ROAD	0.2017	1	3	600	1000	495	570	
9-5-025-009	95-1181 MILO ROAD	0.2182	1	2	400	1000	330	380	
9-5-025-015	95-1202 MILO ROAD	0.1744	1	3	600	1000	495	570	
9-5-025-016	95-1194 MILO ROAD	0.1826	1	3	600	1000	495	570	
9-5-025-017	95-1192 MILO ROAD	0.1637	1	3	600	1000	495	570	
9-5-025-018	95-1188 MILO ROAD	0.1616	1	3	600	1000	495	570	
9-5-025-019	95-1186 MILO ROAD	0.1606	1	3	600	1000	495	570	
9-5-025-020	95-1184 MILO ROAD	0.1533	1	3	600	1000	495	570	
9-5-025-021	95-5598 MELIA STREET	0.1896	1	3	600	1000	495	570	
9-5-025-022	95-5596 MELIA STREET	0.1900	1	2	400	1000	330	380	
9-5-025-023	95-1185 KUKUI ROAD	0.1666	1	3	600	1000	495	570	
9-5-025-024	95-1187 KUKUI ROAD	0.1590	1	3	600	1000	495	570	
9-5-025-025	95-1191 KUKUI ROAD	0.1569	1	3	600	1000	495	570	
9-5-025-026	95-1193 KUKUI ROAD	0.1751	1	5	1000	**1250**	825	950	**Note: 1250 gal Septic Tank**
9-5-025-027	95-1206 MILO ROAD	0.1765	1	3	600	1000	495	570	
9-5-025-008	95-5599 MELIA STREET	0.2129	1	3	600	1000	495	570	
9-5-025-007	95-5595 MELIA STREET	0.2112	1	2	400	1000	330	380	
9-5-025-006	95-1173 KUKUI ROAD	0.2013	1	1	200	1000	165	190	
9-5-025-005	95-1171 KUKUI ROAD	0.3108	1	3	600	1000	495	570	
9-5-025-004	95-1165 KUKUI ROAD	0.2190	1	3	600	1000	495	570	
9-5-025-003	95-1163 KUKUI ROAD	0.3522	1	3	600	1000	495	570	
9-5-026-080	95-1212 KUKUI ROAD	0.1731			0	1000	0	0	abandoned
9-5-026-079	95-1240 OHAI ROAD	0.1722			0	1000	0	0	no structure
9-5-026-078	95-1238 OHAI ROAD	0.1722	1	3	600	1000	495	570	
9-5-026-077	OHAI ROAD	0.1722			0	1000	0	0	no structure
9-5-026-076	95-1234 OHAI ROAD	0.1722	1	3	600	1000	495	570	
9-5-026-075	95-1232 OHAI ROAD	0.1722	1	2	400	1000	330	380	
9-5-026-074	95-1228 OHAI ROAD	0.1722	1	3	600	1000	495	570	
9-5-026-073	95-1226 OHAI ROAD	0.1722	1	3	600	1000	495	570	
9-5-026-072	95-1224 OHAI ROAD	0.1722	1	3	600	1000	495	570	
9-5-026-071	95-1222 OHAI ROAD	0.1722	1	3	600	1000	495	570	
9-5-026-070	95-1220 OHAI ROAD	0.1722	1	3	600	1000	495	570	
9-5-026-069	95-1216 OHAI ROAD	0.1722	1	3	600	1000	495	570	
9-5-026-068	95-1214 OHAI ROAD	0.1722	1	2	400	1000	330	380	
9-5-026-067	95-1212 OHAI ROAD	0.1722	1	3	600	1000	495	570	
9-5-026-066	OHAI ROAD	0.1835	0	0	0	1000	0	0	
9-5-026-065	95-1208 OHAI ROAD	0.2703	1	3	600	1000	495	570	
9-5-026-064	95-1206 OHAI ROAD	0.2399	1	3	600	1000	495	570	
9-5-026-063	95-1204 OHAI ROAD	0.2077	1	3	600	1000	495	570	
9-5-026-062	95-1202 OHAI ROAD	0.2204	1	3	600	1000	495	570	
9-5-026-061	95-1198 OHAI ROAD	0.2146	1	3	600	1000	495	570	
9-5-026-060	95-1196 OHAI ROAD	0.2066	1	3	600	1000	495	570	
9-5-026-059	95-5548 LOKELANI STREET	0.2046	1	3	600	1000	495	570	
9-5-024-075		0.1417	0	0	0	1000	0	0	
9-5-024-074	95-5547 LOKELANI STREET	0.2046	1	3	600	1000	495	570	
9-5-024-073	95-1184 OHAI ROAD	0.2066	1	3	600	1000	495	570	
9-5-024-072	95-1182 OHAI ROAD	0.2066	1	3	600	1000	495	570	
9-5-024-071	95-1178 OHAI ROAD	0.2111	1	3	600	1000	495	570	
9-5-024-070	95-1174 OHAI ROAD	0.2301	1	3	600	1000	495	570	
9-5-024-069	OHAI ROAD	0.1286	0	0	0	1000	0	0	
9-5-024-068	95-1166 OHAI ROAD	0.2537	1	3	600	1000	495	570	
9-5-026-031	95-5554 NAHELE STREET	0.1421	1	2	400	1000	330	380	
9-5-026-032	NAHELE STREET	0.1336	1	3	600	1000	495	570	
9-5-026-033	95-5558 NAHELE STREET	0.1336	1	3	600	1000	495	570	

9-5-026-034	95-5562 NAHELE STREET	0.1703	1	3	600	1000	495	570	
9-5-026-035	95-5564 NAHELE STREET	0.1703	1	3	600	1000	495	570	
9-5-026-036	95-5568 NAHELE STREET	0.1871	1	6	1200	**1500**	990	1140	**Note: 1500 gal Septic Tank with DOH Variance**
9-5-026-037	95-5570 NAHELE STREET	0.1336	1	3	600	1000	495	570	
9-5-026-038	95-5572 NAHELE STREET	0.1336	1	3	600	1000	495	570	
9-5-026-039	95-5576 NAHELE STREET	0.1336	1	3	600	1000	495	570	
9-5-026-040	95-5578 NAHELE STREET	0.1425	1	3	600	1000	495	570	
9-5-026-041	95-5580 NAHELE STREET	0.1247	1	3	600	1000	495	570	
9-5-026-042	95-5582 NAHELE STREET	0.1336	1	3	600	1000	495	570	
9-5-026-043	95-5586 NAHELE STREET	0.1336	1	3	600	1000	495	570	
9-5-026-044	95-1204 KUKUI ROAD	0.1361	1	3	600	1000	495	570	
9-5-026-045	95-1241 OHAI ROAD	0.1494	1	2	400	1000	330	380	
9-5-026-046	95-1239 OHAI ROAD	0.1603	1	3	600	1000	495	570	
9-5-026-047	95-1237 OHAI ROAD	0.1603	1	3	600	1000	495	570	
9-5-026-048	95-1235 OHAI ROAD	0.1336	1	3	600	1000	495	570	
9-5-026-049	95-1231 OHAI ROAD	0.1336	1	3	600	1000	495	570	
9-5-026-050	95-1227 OHAI ROAD	0.1336	1	3	600	1000	495	570	
9-5-026-051	95-1225 OHAI ROAD	0.1336	1	3	600	1000	495	570	
9-5-026-052	95-1223 OHAI ROAD	0.1336	1	3	600	1000	495	570	
9-5-026-053	95-1221 OHAI ROAD	0.1336	1	3	600	1000	495	570	
9-5-026-054	95-1219 OHAI ROAD	0.1336	1	3	600	1000	495	570	
9-5-026-055	95-1217 OHAI ROAD	0.1336	1	3	600	1000	495	570	
9-5-026-056	95-1215 OHAI ROAD	0.2316	1	3	600	1000	495	570	
9-5-026-058	95-1207 OHAI ROAD	0.1990	1	3	600	1000	495	570	
9-5-026-001	95-5554 LOKELANI STREET	0.1442	1	3	600	1000	495	570	
9-5-026-002	95-5556 LOKELANI STREET	0.1336	1	3	600	1000	495	570	
9-5-026-003	95-5558 LOKELANI STREET	0.1336	1	3	600	1000	495	570	
9-5-026-004	95-5560 LOKELANI STREET	0.1336	1	3	600	1000	495	570	
9-5-026-005	95-5562 LOKELANI STREET	0.1336	1	3	600	1000	495	570	
9-5-026-006	95-5566 LOKELANI STREET	0.1336	1	3	600	1000	495	570	
9-5-026-007	95-5568 LOKELANI STREET	0.1336	1	3	600	1000	495	570	
9-5-026-008	95-5570 LOKELANI STREET	0.1336	1	3	600	1000	495	570	
9-5-026-009	95-5574 LOKELANI STREET	0.2120	1	3	600	1000	495	570	
9-5-026-010	95-5576 LOKELANI STREET	0.1386	1	3	600	1000	495	570	
9-5-026-011	95-5580 LOKELANI STREET	0.1336	1	3	600	1000	495	570	
9-5-026-012	95-5582 LOKELANI STREET	0.1394	1	3	600	1000	495	570	
9-5-026-013	95-5584 LOKELANI STREET	0.1336	1	3	600	1000	495	570	
9-5-026-014	95-5586 LOKELANI STREET	0.1336	1	3	600	1000	495	570	
9-5-026-015	95-5588 LOKELANI STREET	0.1316	1	3	600	1000	495	570	
9-5-026-016	95-1198 KUKUI ROAD	0.1316	1	2	400	1000	330	380	
9-5-026-017	95-5583 NAHELE STREET	0.1336	1	3	600	1000	495	570	
9-5-026-018	95-5581 NAHELE STREET	0.1336	1	7	1400	**1750**	1155	1330	**Note: 1750 gal Septic Tank with DOH Variance**
9-5-026-019	95-5579 NAHELE STREET	0.1336	1	3	600	1000	495	570	
9-5-026-020	95-5577 NAHELE STREET	0.1269	1	3	600	1000	495	570	
9-5-026-021	95-5575 NAHELE STREET	0.1425	1	3	600	1000	495	570	
9-5-026-022	95-5573 NAHELE STREET	0.1336	1	3	600	1000	495	570	
9-5-026-023	95-5569 NAHELE STREET	0.1336	1	3	600	1000	495	570	
9-5-026-024	95-5567 NAHELE STREET	0.1336	1	3	600	1000	495	570	
9-5-026-025	95-5565 NAHELE STREET	0.1336	1	3	600	1000	495	570	
9-5-026-026	95-5561 NAHELE STREET	0.1336	1	3	600	1000	495	570	
9-5-026-027	95-5559 NAHELE STREET	0.1336	1	5	1000	**1250**	825	950	**Note: 1250 gal Septic Tank**
9-5-026-028	95-5557 NAHELE STREET	0.1336	1	3	600	1000	495	570	
9-5-026-029	95-5555 NAHELE STREET	0.1336	1	3	600	1000	495	570	
9-5-026-030	95-5551 NAHELE STREET	0.1924	1	3	600	1000	495	570	
9-5-024-052	95-5553 LOKELANI STREET	0.1627	1	3	600	1000	495	570	
9-5-024-053	95-5555 LOKELANI STREET	0.1336	1	3	600	1000	495	570	
9-5-024-054	95-5557 LOKELANI STREET	0.2530	1	3	600	1000	495	570	
9-5-024-056	95-5561 LOKELANI STREET	0.1336	1	3	600	1000	495	570	
9-5-024-057	95-5565 LOKELANI STREET	0.1336	1	3	600	1000	495	570	
9-5-024-058	95-5567 LOKELANI STREET	0.1336	1	3	600	1000	495	570	
9-5-024-059	95-5569 LOKELANI STREET	0.1336	1	3	600	1250	495	570	
9-5-024-060	95-5571 LOKELANI STREET	0.1336	1	3	600	1000	495	570	
9-5-024-061	95-5573 LOKELANI STREET	0.1336	1	6	1200	**1500**	990	1140	**Note: 1500 gal Septic Tank with DOH Variance**
9-5-024-062	95-5577 LOKELANI STREET	0.1336	1	3	600	1000	495	570	
9-5-024-063	95-5579 LOKELANI STREET	0.1336	1	3	600	1000	495	570	
9-5-024-064	95-5581 LOKELANI STREET	0.1336	1	3	600	1000	495	570	
9-5-024-065	95-5583 LOKELANI STREET	0.1336	1	3	600	1000	495	570	
9-5-024-066	95-5585 LOKELANI STREET	0.1336	1	3	600	1000	495	570	

9-5-024-067	95-1188 KUKUI ROAD	0.1379	1	3	600	1000	495	570	
9-5-024-036	95-1184 KUKUI ROAD	0.1379	1	3	600	1000	495	570	
9-5-024-037	95-5588 KILIKA STREET	0.1336	1	3	600	1000	495	570	
9-5-024-038	95-5586 KILIKA STREET	0.1336	1	3	600	1000	495	570	
9-5-024-039	95-5584 KILIKA STREET	0.1336	1	5	1000	**1250**	825	950	**Note: 1250 gal Septic Tank**
9-5-024-040	95-5582 KILIKA STREET	0.1336	1	3	600	1000	495	570	
9-5-024-041	95-5578 KILIKA STREET	0.1247	1	3	600	1000	495	570	
9-5-024-042	95-5576 KILIKA STREET	0.1425	1	3	600	1000	495	570	
9-5-024-043	95-5574 KILIKA STREET	0.1336	1	3	600	1000	495	570	
9-5-024-044	95-5572 KILIKA STREET	0.1336	1	3	600	1000	495	570	
9-5-024-045	95-5570 KILIKA STREET	0.1336	1	3	600	1000	495	570	
9-5-024-046	95-5568 KILIKA STREET	0.1336	1	3	600	1000	495	570	
9-5-024-047	95-5564 KILIKA STREET	0.1336	1	3	600	1000	495	570	
9-5-024-048	95-5562 KILIKA STREET	0.1305	1	3	600	1000	495	570	
9-5-024-049	95-5560 KILIKA STREET	0.1336	1	3	600	1000	495	570	
9-5-024-050	95-5558 KILIKA STREET	0.1225	1	6	1200	**1500**	990	1140	**Note: 1500 gal Septic Tank with DOH Variance**
9-5-024-051	95-5554 KILIKA STREET	0.2015	1	4	800	1000	660	760	
9-5-024-014	95-1178 KUKUI ROAD	0.2650	1	3	600	1000	495	570	
9-5-024-015	95-5590 OPUKEA STREET	0.1778	1	3	600	1000	495	570	
9-5-024-016	95-5586 OPUKEA STREET	0.1768	1	3	600	1000	495	570	
9-5-024-017	95-5582 OPUKEA STREET	0.1688	1	3	600	1000	495	570	
9-5-024-018	95-5578 OPUKEA STREET	0.1800	1	3	600	1000	495	570	
9-5-024-019	95-5574 OPUKEA STREET	0.1725	1	3	600	1000	495	570	
9-5-024-020	95-5570 OPUKEA STREET	0.1795	1	3	600	1000	495	570	
9-5-024-021	95-5564 OPUKEA STREET	0.1779	1	3	600	1000	495	570	
9-5-024-022	95-5560 OPUKEA STREET	0.1771	1	3	600	1000	495	570	
9-5-024-023	95-5558 OPUKEA STREET	0.1605	1	3	600	1000	495	570	
9-5-024-024	95-5556 OPUKEA STREET	0.1433	1	3	600	1000	495	570	
9-5-024-025	95-5553 KILIKA STREET	0.1958	1	3	600	1000	495	570	
9-5-024-026	95-5555 KILIKA STREET	0.1228	1	3	600	1000	495	570	
9-5-024-027	95-5559 KILIKA STREET	0.1871	1	3	600	1000	495	570	
9-5-024-028	95-5563 KILIKA STREET	0.1739	1	3	600	1000	495	570	
9-5-024-029	95-5565 KILIKA STREET	0.1251	1	3	600	1000	495	570	
9-5-024-030	95-5567 KILIKA STREET	0.1220	1	3	600	1000	495	570	
9-5-024-031	95-5571 KILIKA STREET	0.1232	1	3	600	1000	495	570	
9-5-024-032	95-5573 KILIKA STREET	0.1538	1	3	600	1000	495	570	
9-5-024-033	95-5577 KILIKA STREET	0.1633	1	3	600	1000	495	570	
9-5-024-034	95-5581 KILIKA STREET	0.1507	1	4	800	1000	660	760	
9-5-024-035	95-5583 KILIKA STREET	0.1570	1	3	600	1000	495	570	
9-5-024-001	95-1163 OHAI ROAD	0.3013	1	3	600	1000	495	570	
9-5-024-002	95-5558 MAMALAHOA HIGHW	1.0000	1	2	400	1000	330	380	
9-5-024-003	95-1167 OHAI ROAD	0.2502	1	3	600	1000	495	570	
9-5-024-004	95-5555 OPUKEA STREET	0.2503	1	3	600	1000	495	570	
9-5-024-005	95-5559 OPUKEA STREET	0.2655	1	3	600	1000	495	570	
9-5-024-006	95-5561 OPUKEA STREET	0.2842	1	3	600	1000	495	570	
9-5-024-007	95-5572 MAMALAHOA HIGHW	1.9440			0		0	0	
9-5-024-008	59-553 KAALA ROAD	0.3990	1	3	600	1000	495	570	
9-5-024-009	95-5575 OPUKEA STREET	0.2916	1	3	600	1000	495	570	
9-5-024-010	95-5579 OPUKEA STREET	0.3293	1	3	600	1000	495	570	
9-5-024-011	OPUKEO STREET	0.2375	0	0	0		0	0	no structure
9-5-024-012	95-1168 KUKUI ROAD	0.2040	1	3	600	1000	495	570	
9-5-024-013	95-1172 KUKUI ROAD	0.3329	1	2	400	1000	330	380	
9-5-024-076	95-5586 MAMALAHOA HIGHW	0.4231	1	4	800	1000	660	760	
9-5-024-077	95-5582 MAMALAHOA HIGHW	0.3950	1	3	600	1000	495	570	
9-5-024-078	95-5580 MAMALAHOA HIGHW	0.3217	1	3	600	1000	495	570	

B

Appendix B - Cost Calculations

B

IWS Management Model 2A County In-House Maintenance Cost[1]

Year	Tasks		County O&M Staff Cost
0	IWS Installation		Capital Cost/Household
0	Pumping & Hauling equipment	$250,000.00	$ 1,470.59
0	Personnel Training	$ 50,000.00	$ 294.12
1	Annual Inspection by & Trouble Calls by County staff – two IWS Operators / Plumbers		$ 822.00
2	Annual Inspection by & Trouble Calls by County staff - two IWS Operators / Plumbers		$ 822.00
3	Septic sludge pumping & disposal by County staff		$ 1,250.00
4	Annual Inspection by & Trouble Calls by County staff - two IWS Operators / Plumbers		$ 822.00
5	Annual Inspection by & Trouble Calls by County staff - two IWS Operators / Plumbers		$ 822.00
6	Septic sludge pumping & disposal by County staff		$ 1,250.00
7	Annual Inspection by & Trouble Calls by County staff - two IWS Operators / Plumbers		$ 822.00
8	Annual Inspection by & Trouble Calls by County staff - two IWS Operators / Plumbers		$ 822.00
9	Septic sludge pumping & disposal by County staff		$ 1,250.00
10	Annual Inspection by & Trouble Calls by County staff - two IWS Operators / Plumbers		$ 822.00
11	Annual Inspection by & Trouble Calls by County staff - two IWS Operators / Plumbers		$ 822.00
12	Septic sludge pumping & disposal by County staff		$ 1,250.00
13	Annual Inspection by & Trouble Calls by County staff - two IWS Operators / Plumbers		$ 822.00
14	Annual Inspection by & Trouble Calls by County staff - two IWS Operators / Plumbers		$ 822.00
15	Septic sludge pumping & disposal by County staff		$ 1,250.00
16	Annual Inspection by & Trouble Calls by County staff - two IWS Operators / Plumbers		$ 822.00
17	Annual Inspection by & Trouble Calls by County staff - two IWS Operators / Plumbers		$ 822.00
18	Septic sludge pumping & disposal by County staff		$ 1,250.00
19	Annual Inspection by & Trouble Calls by County staff - two IWS Operators / Plumbers		$ 822.00
20	Absorption bed replacement		$ 30,000.00
20	Pumping & Hauling equipment Replacement	$250,000.00	$ 1,470.59
21	Annual Inspection by & Trouble Calls by County staff - two IWS Operators / Plumbers		$ 822.00
	Average Annual Maintenance Cost		$ 956.44

[1] Cost of services are estimated from 2023 USD rates and would increase due to inflation.

IWS Management Model 2B Outsource Maintenance Cost

Year	Tasks	Outsource O&M Cost		County Admin Staff	
0	IWS Installation	Capital Cost			
1	IWS annual inspection & Trouble Calls	$	550.00	$	572.00
2	IWS annual inspection & Trouble Calls	$	550.00	$	572.00
3	Septic sludge pumping & disposal	$	1,250.00	$	572.00
4	IWS annual inspection & Trouble Calls	$	550.00	$	572.00
5	IWS annual inspection & Trouble Calls	$	550.00	$	572.00
6	Septic sludge pumping & disposal	$	1,250.00	$	572.00
7	IWS annual inspection & Trouble Calls	$	550.00	$	572.00
8	IWS annual inspection & Trouble Calls	$	550.00	$	572.00
9	Septic sludge pumping & disposal	$	1,250.00	$	572.00
10	IWS annual inspection & Trouble Calls	$	550.00	$	572.00
11	IWS annual inspection & Trouble Calls	$	550.00	$	572.00
12	Septic sludge pumping & disposal	$	1,250.00	$	572.00
13	IWS annual inspection & Trouble Calls	$	550.00	$	572.00
14	IWS annual inspection & Trouble Calls	$	550.00	$	572.00
15	Septic sludge pumping & disposal	$	1,250.00	$	572.00
16	IWS annual inspection & Trouble Calls	$	550.00	$	572.00
17	IWS annual inspection & Trouble Calls	$	550.00	$	572.00
18	Septic sludge pumping & disposal	$	1,250.00	$	572.00
19	IWS annual inspection & Trouble Calls	$	550.00	$	572.00
20	Absorption bed replacement	$	30,000.00	$	572.00
21	IWS annual inspection (repeat as Year 1)	$	550.00	$	572.00
	Average Annual Maintenance Cost	$	783.33	$	572.00

County WWD Admin Personnel Cost for Record Keeping and administering — Based on $100000/175 = $572

Trouble calls, emergency repairs per IWS per year — $ 500.00

IWS Management Model 3A Operating Permit User O&M Cost

Year	Tasks	O&M Cost to Homeowner/User	County Admin Cost
0	IWS Installation	0	Capital Cost
1	IWS annual inspection & Trouble calls	$ 500.00	$ 572.00
2	IWS annual inspection	$ 500.00	$ 572.00
3	Septic sludge pumping & disposal	$ 1,200.00	$ 572.00
4	IWS annual inspection	$ 500.00	$ 572.00
5	IWS annual inspection	$ 500.00	$ 572.00
6	Septic sludge pumping & disposal	$ 1,200.00	$ 572.00
7	IWS annual inspection	$ 500.00	$ 572.00
8	IWS annual inspection	$ 500.00	$ 572.00
9	Septic sludge pumping & disposal	$ 1,200.00	$ 572.00
10	IWS annual inspection	$ 500.00	$ 572.00
11	IWS annual inspection	$ 500.00	$ 572.00
12	Septic sludge pumping & disposal	$ 1,200.00	$ 572.00
13	IWS annual inspection	$ 500.00	$ 572.00
14	IWS annual inspection	$ 500.00	$ 572.00
15	Septic sludge pumping & disposal	$ 1,200.00	$ 572.00
16	IWS annual inspection	$ 500.00	$ 572.00
17	IWS annual inspection	$ 500.00	$ 572.00
18	Septic sludge pumping & disposal	$ 1,200.00	$ 572.00
19	IWS annual inspection	$ 500.00	$ 300.00
20	Absorption bed replacement	$ -	$ 30,000.00
21	IWS annual inspection (repeat as Year 1)	$ 500.00	$ 572.00
	Annual Average Cost	$ 733.33	$ 572.00

O&M Cost to Homeowner / User includes trouble calls $300 + $200 = $500

County admin staff to provide recording keeping, regulation and enforcing Based on $100000/175

B

IWS Management Model 3B County Voucher O&M Cost

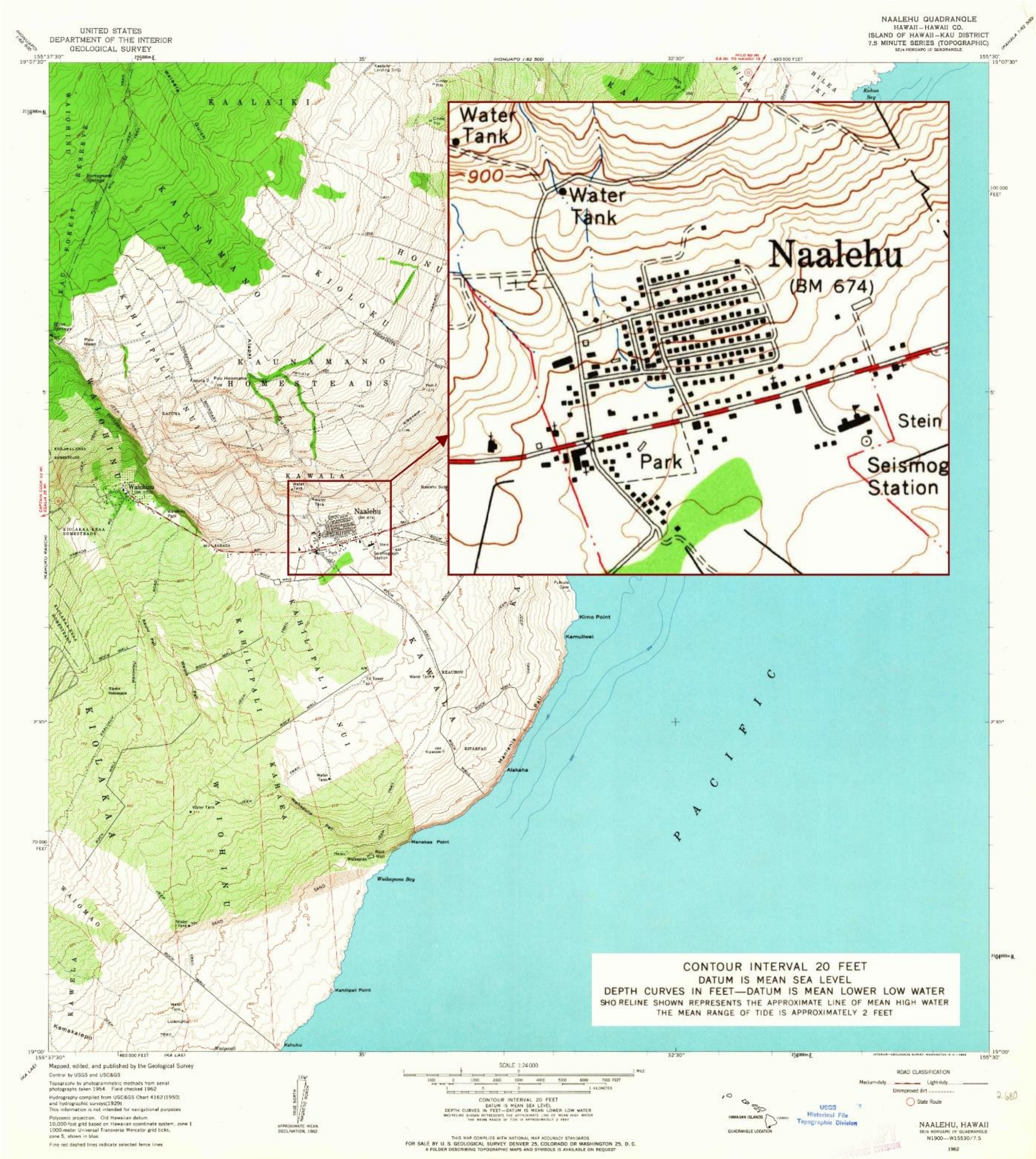
Year	Tasks	Trouble call Cost to Homeowner/User	County Voucher Cost (present value)
0	IWS Installation	0	Capital Cost
1	IWS annual inspection & Trouble calls	$ 600.00	$ 872.00
2	IWS annual inspection	$ 600.00	$ 872.00
3	Septic sludge pumping & disposal	$ 600.00	$ 1,572.00
4	IWS annual inspection	$ 600.00	$ 872.00
5	IWS annual inspection	$ 600.00	$ 872.00
6	Septic sludge pumping & disposal	$ 600.00	$ 1,572.00
7	IWS annual inspection	$ 600.00	$ 872.00
8	IWS annual inspection	$ 600.00	$ 872.00
9	Septic sludge pumping & disposal	$ 600.00	$ 1,572.00
10	IWS annual inspection	$ 600.00	$ 872.00
11	IWS annual inspection	$ 600.00	$ 872.00
12	Septic sludge pumping & disposal	$ 600.00	$ 1,572.00
13	IWS annual inspection	$ 600.00	$ 872.00
14	IWS annual inspection	$ 600.00	$ 872.00
15	Septic sludge pumping & disposal	$ 600.00	$ 1,572.00
16	IWS annual inspection	$ 600.00	$ 872.00
17	IWS annual inspection	$ 600.00	$ 872.00
18	Septic sludge pumping & disposal	$ 600.00	$ 1,572.00
19	IWS annual inspection	$ 600.00	$ 872.00
20	Absorption bed replacement	$ -	$ 30,000.00
21	IWS annual inspection (repeat as Year 1)	$ 600.00	$ 872.00
	Annual Average Cost	$ 600.00	$ 1,105.33

County voucher cost: $572 + $300

Based on Admin staff cost $100000/175

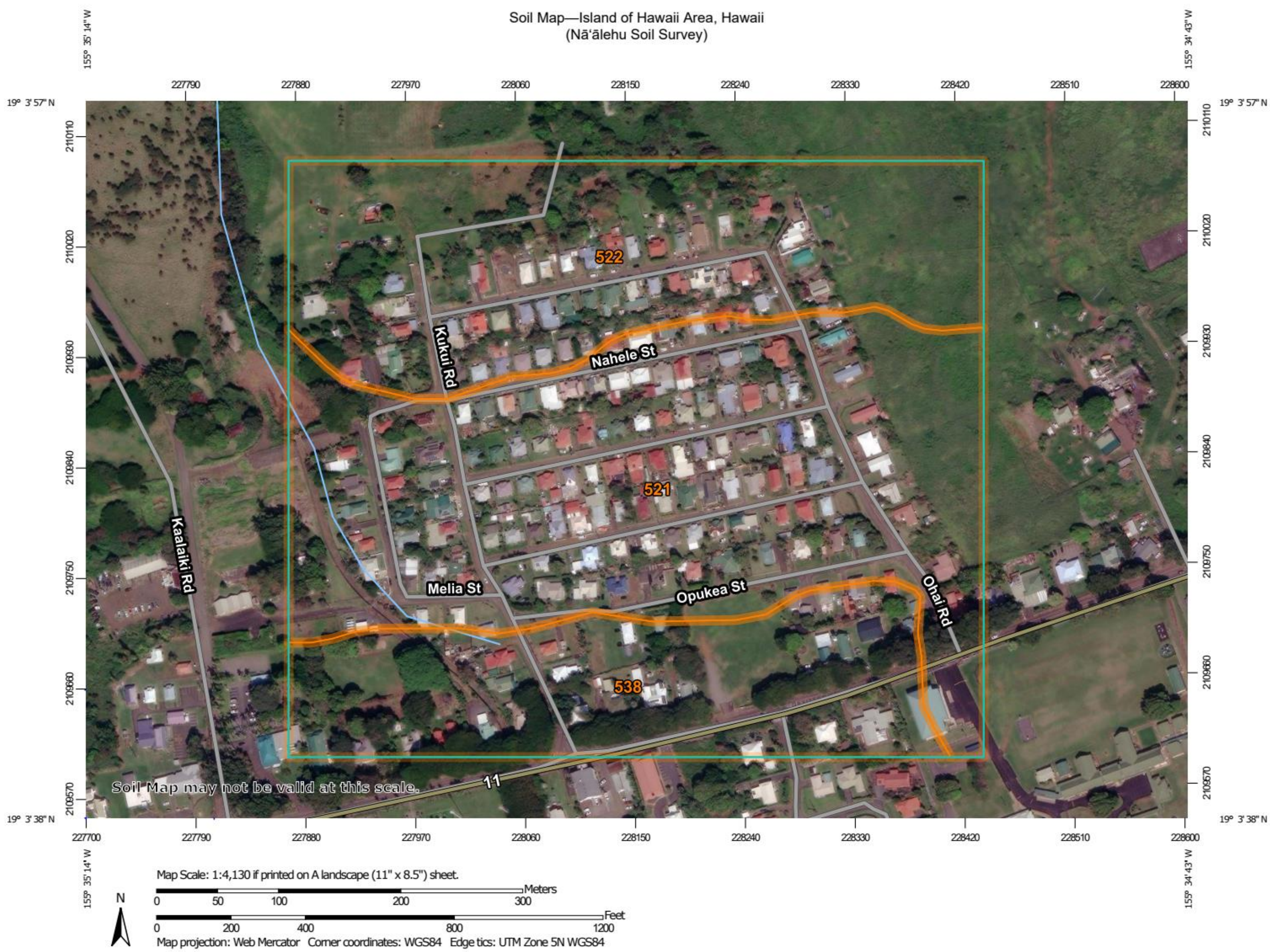
C

Appendix C - Topography


UNITED STATES
DEPARTMENT OF THE INTERIOR
GEOLOGICAL SURVEY
NAALEHU QUADRANGLE
HAWAII – HAWAII CO.
ISLAND OF HAWAII – KAU DISTRICT
7.5 MINUTE SERIES (TOPOGRAPHIC)
Water Tank
900
Water Tank
Naalehu
(BM 674)
Stein
Park
Seismog
Station
Kimo Point
Kamilo Point
Manakaa Point
Waikapuna Bay
Kahilipali Point
Kahuku
P A C I F I C
CONTOUR INTERVAL 20 FEET
DATUM IS MEAN SEA LEVEL
DEPTH CURVES IN FEET—DATUM IS MEAN LOWER LOW WATER
SHORELINE SHOWN REPRESENTS THE APPROXIMATE LINE OF MEAN HIGH WATER
THE MEAN RANGE OF TIDE IS APPROXIMATELY 2 FEET
Mapped, edited, and published by the Geological Survey
SCALE 1:24000
ROAD CLASSIFICATION
NAALEHU, HAWAII
1962
FOR SALE BY U. S. GEOLOGICAL SURVEY, DENVER 25, COLORADO OR WASHINGTON 25, D. C.

D

Appendix D – USGS Soil Survey



USDA Natural Resources Conservation Service

Web Soil Survey
National Cooperative Soil Survey

7/6/2023
Page 1 of 3

MAP LEGEND

Area of Interest (AOI)

Area of Interest (AOI)

Soils

Soil Survey Areas

Soil Map Unit Polygons

Soil Map Unit Lines

Soil Map Unit Points

Special Point Features

Blowout

Borrow Pit

Clay Spot

Closed Depression

Gravel Pit

Gravelly Spot

Landfill

Lava Flow

Marsh or swamp

Mine or Quarry

Miscellaneous Water

Perennial Water

Rock Outcrop

Saline Spot

Sandy Spot

Severely Eroded Spot

Sinkhole

Slide or Slip

Sodic Spot

Spoil Area

Stony Spot

Very Stony Spot

Wet Spot

Other

Special Line Features

Water Features

Streams and Canals

Transportation

Rails

Interstate Highways

US Routes

Major Roads

Local Roads

Background

Aerial Photography

MAP INFORMATION

The soil surveys that comprise your AOI were mapped at 1:24,000.

Warning: Soil Map may not be valid at this scale.

Enlargement of maps beyond the scale of mapping can cause misunderstanding of the detail of mapping and accuracy of soil line placement. The maps do not show the small areas of contrasting soils that could have been shown at a more detailed scale.

Please rely on the bar scale on each map sheet for map measurements.

Source of Map: Natural Resources Conservation Service
Web Soil Survey URL:
Coordinate System: Web Mercator (EPSG:3857)

Maps from the Web Soil Survey are based on the Web Mercator projection, which preserves direction and shape but distorts distance and area. A projection that preserves area, such as the Albers equal-area conic projection, should be used if more accurate calculations of distance or area are required.

This product is generated from the USDA-NRCS certified data as of the version date(s) listed below.

Soil Survey Area: Island of Hawaii Area, Hawaii
Survey Area Data: Version 15, Aug 30, 2022

Soil map units are labeled (as space allows) for map scales 1:50,000 or larger.

Date(s) aerial images were photographed: Jan 3, 2019—Jun 28, 2022

The orthophoto or other base map on which the soil lines were compiled and digitized probably differs from the background imagery displayed on these maps. As a result, some minor shifting of map unit boundaries may be evident.

Map Unit Legend

Map Unit Symbol	Map Unit Name	Acres in AOI	Percent of AOI
521	Naalehu medial silty clay loam, 3 to 10 percent slopes	32.4	47.2%
522	Naalehu medial silty clay loam, 10 to 20 percent slopes	21.4	31.2%
538	Naalehu medial silt loam, 0 to 3 percent slopes	14.8	21.6%
Totals for Area of Interest		**68.5**	**100.0%**

E

Appendix E - Geotech Report

Masa Fujioka & Associates 98-021 Kamehameha Highway #337 Aiea, Hawaii ph (808)484-5366, FAX 484-0007	**BORING LOG: N-1** Na'alehu Cesspool Conversion Na'alehu, Island of Hawaii, Hawaii	Page 1 Job Number: 05086-091 Elevation:

Driller: Geolabs		Drilling	Date:
Drill Method: Core Barrel		Started:	11/13/06
Sample Method: 2.5" Core		Finished:	11/13/06
Boring Diameter: 2.5"	Water Level : n/a	Logged By: JR	Checked By: MF

Recovery	RQD	Sample Type	Depth (Ft)	USCS	Material Description	Test Results
			0	AS	Asphaltic-concrete pavement	
				GM	Basecourse	
				MH	Reddish-brown clayey SILT, med. stiff	
				BR	Black BASALT, highly vesicular & well fractured, mod. stiff	
			2			
20%	0%	Core 1				
			4			
			6			
					becomes moderately fractured	
100%	50%	Core 2				
			8			
			10		Boring terminated @ 10 ft bgs, no groundwater encountered	
			12			
			14			

Masa Fujioka & Associates 98-021 Kamehameha Highway #337 Aiea, Hawaii ph (808)484-5366, FAX 484-0007	**BORING LOG: N-2** Na'alehu Cesspool Conversion Na'alehu, Island of Hawaii, Hawaii	Page 2 Job Number: 05086-091 Elevation:

Driller: Geolabs		Drilling	Date:
Drill Method: Core Barrel		Started:	11/13/06
Sample Method: 2.5" Core		Finished:	11/13/06
Boring Diameter: 2.5"	Water Level : n/a	Logged By: JR	Checked By: MF

Recovery	RQD	Sample Type	Depth (Ft)	USCS	Material Description	Test Results
			0	AS	Asphaltic-concrete pavement	
				GM	Basecourse	
				GM	Silty GRAVEL (Fill)	
				BR	Black BASALT, highly vesicular, fractured and broken	
			2			
20%	0%	Core 1				
			4			
					becomes moderately vesicular	
			6			
100%	87%	Core 2				
			8			
					colors brownish-black, becomes highly vesicular	
			10		Boring terminated @ 10 ft bgs, no groundwater encountered	
			12			
			14			

Masa Fujioka & Associates 98-021 Kamehameha Highway #337 Aiea, Hawaii ph (808)484-5366, FAX 484-0007	**BORING LOG: N-3** Na'alehu Cesspool Conversion Na'alehu, Island of Hawaii, Hawaii	Page 3 Job Number: 05086-091 Elevation:

Driller: Geolabs		Drilling	Date:
Drill Method: Core Barrel		Started:	11/13/06
Sample Method: 2.5" Core		Finished:	11/13/06
Boring Diameter: 2.5"	Water Level : n/a	Logged By: JR	Checked By: MF

Recovery	RQD	Sample Type	Depth (Ft)	USCS	Material Description	Test Results
			0	AS	Asphaltic-concrete pavement	
				GM	Basecourse	
				GM	Silty GRAVEL (Fill)	
				MH	Reddish-brown clayey SILT	
		Bulk	2		Sample collected from adjacent test pit	MC = 29 % CBR = 18 % Exp = 0.8 %
18%	7%	Core 1		BR	Black BASALT, fragmented	
			4		Decreases in vesicles w/ depth to 8'	
57%	17%	Core 2	6			
			8	BR	Brownish-black BASALT, moderately vesicular	
100%	46%	Core 3				
			10		Boring terminated @ 10 ft bgs, no groundwater encountered	
			12			
			14			

Masa Fujioka & Associates 98-021 Kamehameha Highway #337 Aiea, Hawaii ph (808)484-5366, FAX 484-0007	BORING LOG: N-4 Na'alehu Cesspool Conversion Na'alehu, Island of Hawaii, Hawaii	Page 4 Job Number: 05086-091 Elevation:

Driller: Geolabs		Drilling	Date:
Drill Method: Core Barrel		Started:	11/13/06
Sample Method: 2.5" Core		Finished:	11/13/06
Boring Diameter: 2.5"	Water Level : n/a	Logged By: JR	Checked By: MF

Recovery	RQD	Depth (Ft)	Sample Type	USCS	Material Description	Test Results
		0		AS	Asphaltic-concrete pavement	
				GM	Basecourse	
				MH	Brown SILT, med. stiff	
				BR	Black BASALT, highly vesicular	
		2			Bulk Sample collected from adjacent test pit	MC = 29 % CBR = 2 % Exp = 0.1 %
32%	10%	Core 1				
		4				
		6				
38%	24%	Core 2				
		8			becomes broken and vesicular	
		10			Boring terminated @ 10 ft bgs, no groundwater encountered	
		12				
		14				

Masa Fujioka & Associates 98-021 Kamehameha Highway #337 Aiea, Hawaii ph (808)484-5366, FAX 484-0007	BORING LOG: N-5 Na'alehu Cesspool Conversion Na'alehu, Island of Hawaii, Hawaii	Page 5 Job Number: 05086-091 Elevation:

Driller: Geolabs		Drilling	Date:
Drill Method: Core Barrel		Started:	11/13/06
Sample Method: 2.5" Core		Finished:	11/13/06
Boring Diameter: 2.5"	Water Level : n/a	Logged By: JR	Checked By: MF

Recovery	RQD	Sample Type	Depth (Ft)	USCS	Material Description	Test Results
			0	AS	Asphaltic-concrete pavement	
				GM	Basecourse	
				GM	Brownish-black silty basaltic GRAVEL (Fill?)	
			2			
20%	0%	Core 1				
				MH	Reddish-brown clayey SILT	
			4			
				BR	Black BASALT, vesicular, well fractured	
			6			
14%	14%	Core 2				
			8			
			10		Boring terminated @ 10 ft bgs, no groundwater encountered	
			12			
			14			

Masa Fujioka & Associates 98-021 Kamehameha Highway #337 Aiea, Hawaii ph (808)484-5366, FAX 484-0007	**BORING LOG: N-6** Na'alehu Cesspool Conversion Na'alehu, Island of Hawaii, Hawaii	Page 6 Job Number: 05086-091 Elevation:

Driller: Geolabs		Drilling	Date:
Drill Method: Core Barrel		Started:	11/13/06
Sample Method: 2.5" Core		Finished:	11/13/06
Boring Diameter: 2.5"	Water Level : n/a	Logged By: JR	Checked By: MF

Recovery	RQD	Sample Type	Depth (Ft)	USCS	Material Description	Test Results
			0	AS	Asphaltic-concrete pavement	
				GM	Basecourse	
				BR	Olivine-rich BASALT, moderately vesicular, well fractured and broken	
			2			
10%	0%	Core 1				
			4			
			6			
38%	15%	Core 2				
			8	BR	Grey BASALT, slightly vesicular, 3% plagioclase (a'a)	
			10		Boring terminated @ 10 ft bgs, no groundwater encountered	
			12			
			14			

Masa Fujioka & Associates
98-021 Kamehameha Highway #337
Aiea, Hawaii
ph (808)484-5366, FAX 484-0007

BORING LOG: N-7

Na'alehu Cesspool Conversion
Na'alehu, Island of Hawaii, Hawaii

Job Number: 05086-091

Elevation:

Driller: Geolabs		Drilling	Date:
Drill Method: Core Barrel		Started:	11/13/06
Sample Method: 2.5" Core		Finished:	11/13/06
Boring Diameter: 2.5"	Water Level : n/a	Logged By: JR	Checked By: MF

Recovery	RQD	Depth (Ft) / Sample Type	USCS	Material Description	Test Results
		0	MH	Clayey SILT (top soil), with grass and roots	
			MH	Brown clayey SILT	
30%	10%	Core 1 (0–5); 2		colors reddish-brown Bulk Sample collected from adjacent test pit	MC = 40 % CBR = 11 % Exp = 1.4 %
		4	BR	Black BASALT, broken and highly vesicular	
		6	BR	Blackish-brown BASALT, highly vesicular clinker	
16%	0%	Core 2 (5–10); 8			
		10	BR	Black BASALT, moderately vesicular	
10%	0%	Core 3 (10–14); 12			
		14			

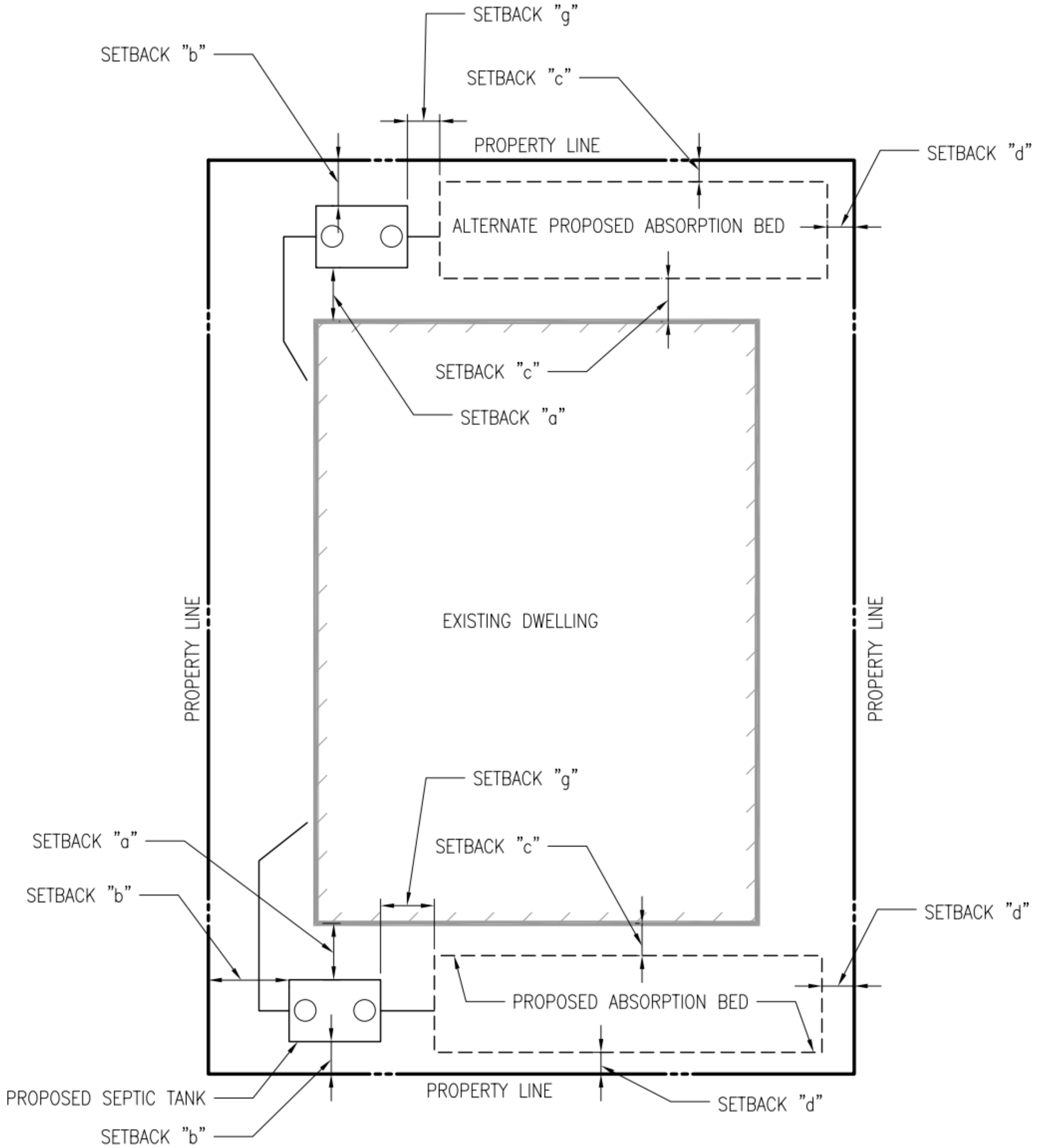
Masa Fujioka & Associates 98-021 Kamehameha Highway #337 Aiea, Hawaii ph (808)484-5366, FAX 484-0007	BORING LOG: N-7 Na'alehu Cesspool Conversion Na'alehu, Island of Hawaii, Hawaii	Page 8 Job Number: 05086-091 Elevation:

Driller: Geolabs		Drilling	Date:
Drill Method: Core Barrel		Started:	11/13/06
Sample Method: 2.5" Core		Finished:	11/13/06
Boring Diameter: 2.5"	Water Level : n/a	Logged By: JR	Checked By: MF

Recovery	RQD	Depth (Ft)	Sample Type	USCS	Material Description	Test Results
		14	Core 3	BR	Black BASALT, moderately vesicular	
		16				
62%	52%	18	Core 4			
		20		BR	Black BASALT, highly vesicular and fractured	
22%	12%	22	Core 5			
		24				
		25			Boring terminated @ 25 ft bgs, no groundwater encountered	
		26				
		28				

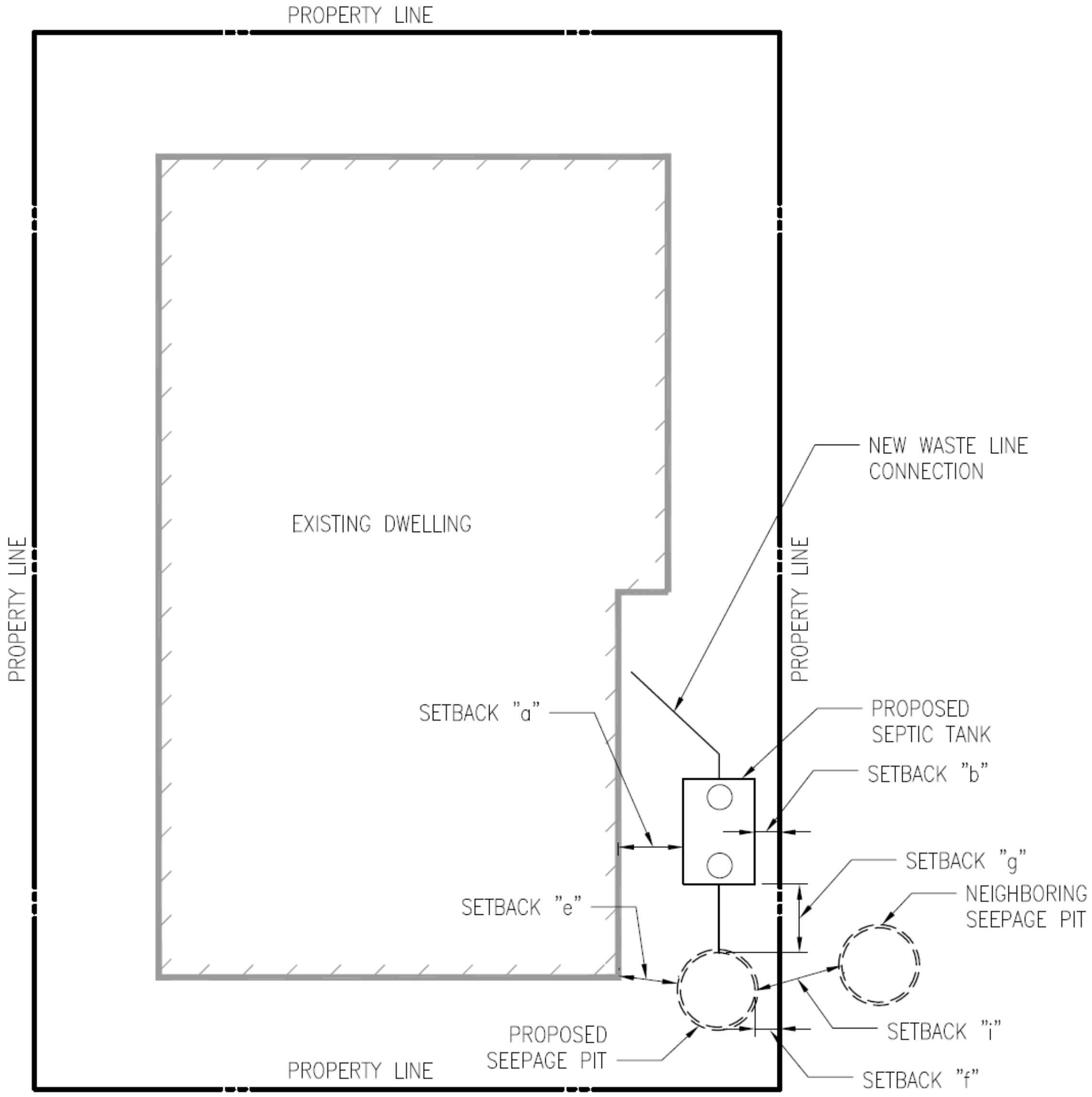
F

Appendix F - Typical IWS Layout & Components



TYPICAL IWS LAYOUT W/ ABSORPTION BED

See Schedule A for Required Setback Dimensions and Variance Request



TYPICAL IWS LAYOUT W/ SEEPAGE PIT

See Schedule A for Required Setback Dimensions and Variance Request

SCHEDULE A

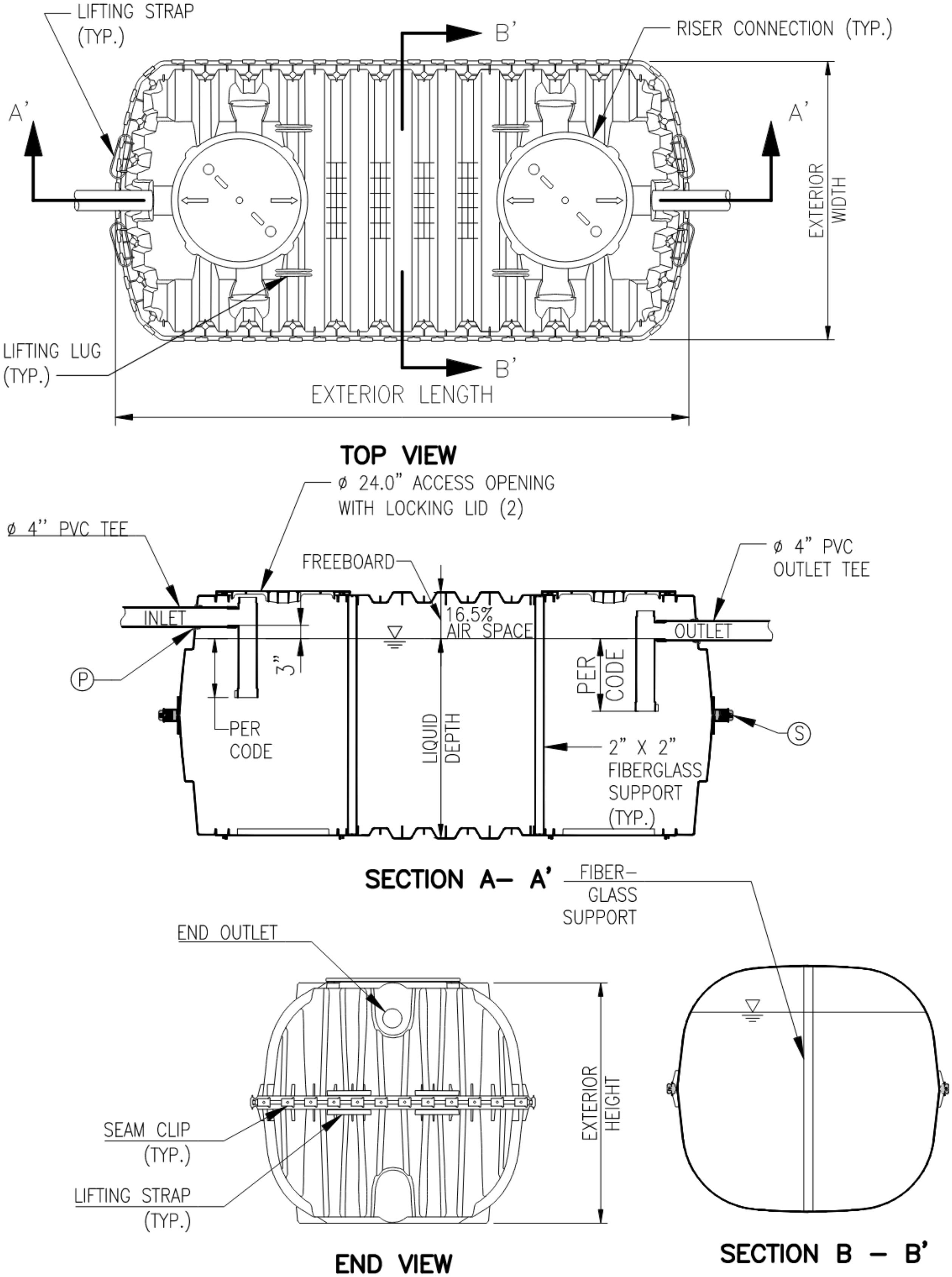
SETBACK TYPE	DESCRIPTION/MIN. PER DOH, TABLE (SEE NOTE 1)	VARIANCE REQUEST (SEE NOTE 1)
a	DIST. BTW BLDG & SEPTIC TANK / 5' MIN.	2' < "a" < 5'
	DIST. BTW PROPERTY LINE & SEPTIC TANK / 5' MIN.	1' < "b" < 5'
c	DIST. BTW BLDG & ABSORPTION BED / 5' MIN.	2' < c < 5'
"d"	DIST. BTW PROPERTY LINE & ABSORPTION BED / 5' MIN.	0 < "d" < 5'
"e"	DIST. BTW BLDG & SEEPAGE PIT / 5' MIN.	2' < "e" < 5'
"f"	DIST. BTW PROPERTY LINE & SEEPAGE PIT / 9' MIN.	1' < "f" < 5'
"g"	DIST. BTW SEPTIC TANK & ABSORPTION BED / 5' MIN.	2' < "g" < 5'
	DIST. BTW NEIGHBORING ABSORPTION BEDS / 5' MIN.	1' < "h" < 5'
	DIST. BTW NEIGHBORING SEEPAGE PITS / 12' MIN.	6' < "i" < 12'

ABSORPTION BED & SEEPAGE PIT SIZING

# OF BEDROMS SEPTIC TANK SIZE	NON-TRAFFIC RATED ABSORPTION BED SIZE (SEE NOTE 3)	TRAFFIC RATED ABSORPTION BED SIZE (SEE NOTE 3)	SEEPAGE PIT SIZE (SEE NOTE 4)
3 1000 GAL.	10' X 24' 12' X 20' 15' X 16'	9' 24' 12' 18' 15' 16'	6'r/J X 12' DEEP
4 1000 OR 1250 GAL.	10' X 32' 12' X 27' 15' X 22'	9' 30' 12' 24' 15' 18'	6'r/J X 15' DEEP 8'r/! X 12' DEEP
5 1250 GAL.	10' X 40' 12' X 34' 15' X 27'	9' 40' 12' 28' 15' 24'	8'r/! X 14' DEEP
6 1250 GAL. PER DWS (SEE NOTE 2)	10' X 48' 12' X 40' 15' X 32'	g' 48' 12' 36' 15' 28'	8'r/! X 17' DEEP
7 1250 GAL. PER DWS (SEE NOTE 2)	10' X 56' 12' X 47' 15' X 38'	g' 52' 12' 40' 15' 32'	8'r/! X 20' DEEP

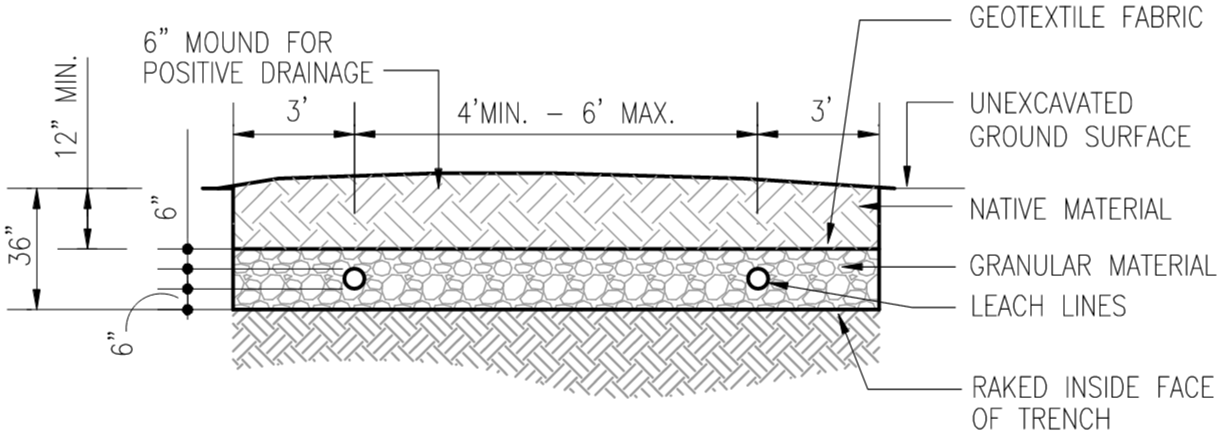
NOTES:

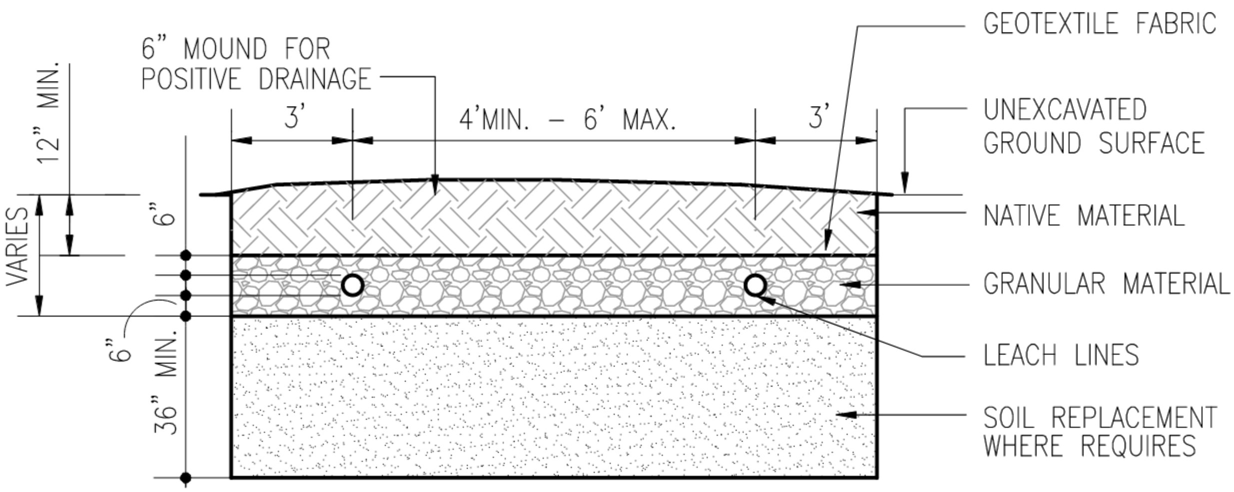
1. The request is for Variance from Section 11-62-22 Spacing of Individual Wastewater Systems, Table II in Appendix D and Section 11-62-31.1(1)(D) where states that one IWS cannot serve more than 5 bedrooms.
2. For dwellings with more than 5 bedrooms, we request a Variance to base the IWS design on the DWS water consumption record rather than based on number of bedrooms.
3. Absorption Bed for standard perforated pipe with gravel bed installation (non-traffic rated), a percolation rate of 2 min./inch is assumed. For gravel-less installation (Infiltrator Chambers) or traffic rated chambers, 17% reduction is taken for the required area of absorption bed.
4. For sizing of seepage pit, a percolation rate of 1 min/inch is assumed because the soil condition is likely to be granular or rocky type at that depth.



SEPTIC TANK DETAIL

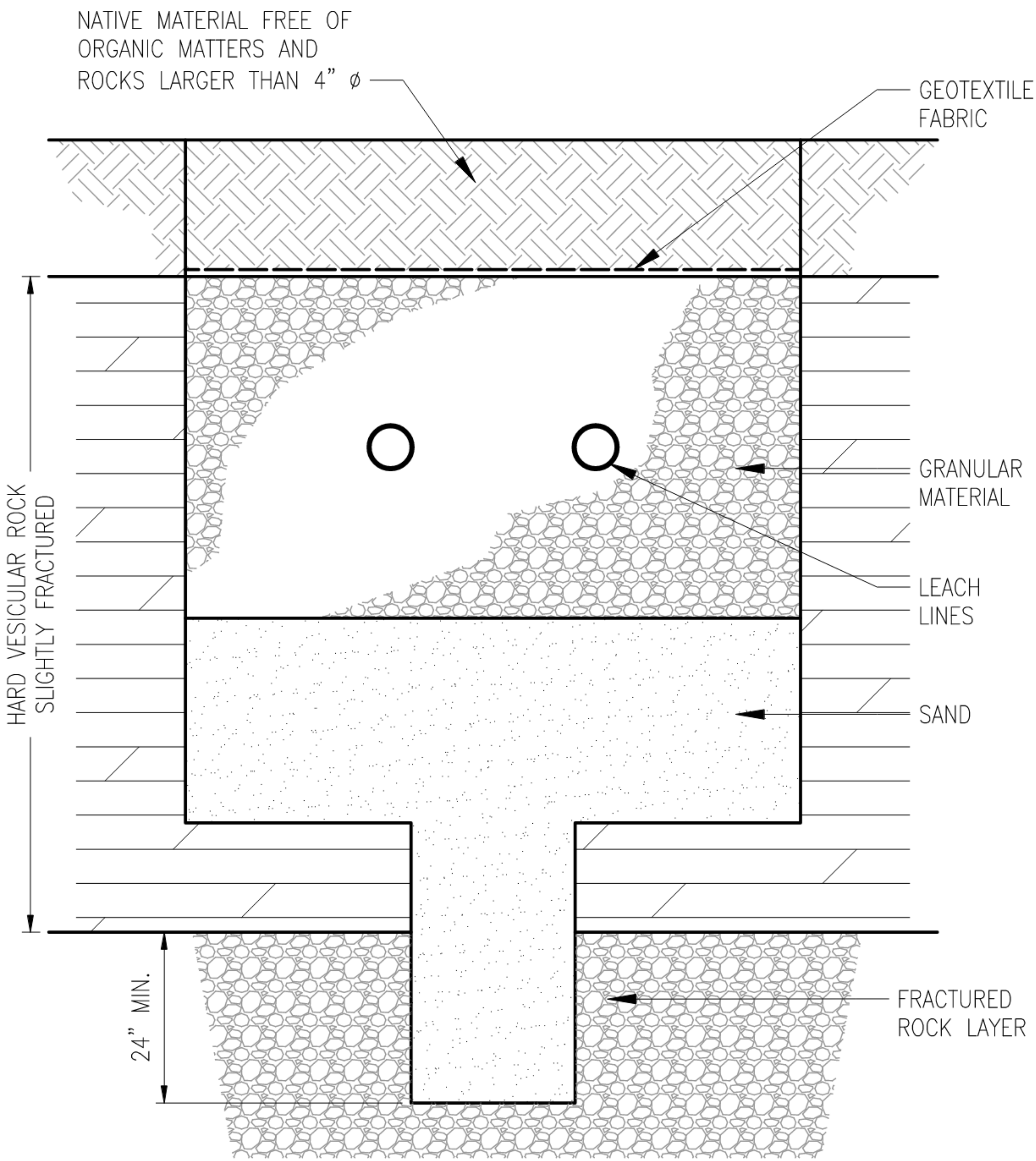
SCALE: NOT TO SCALE





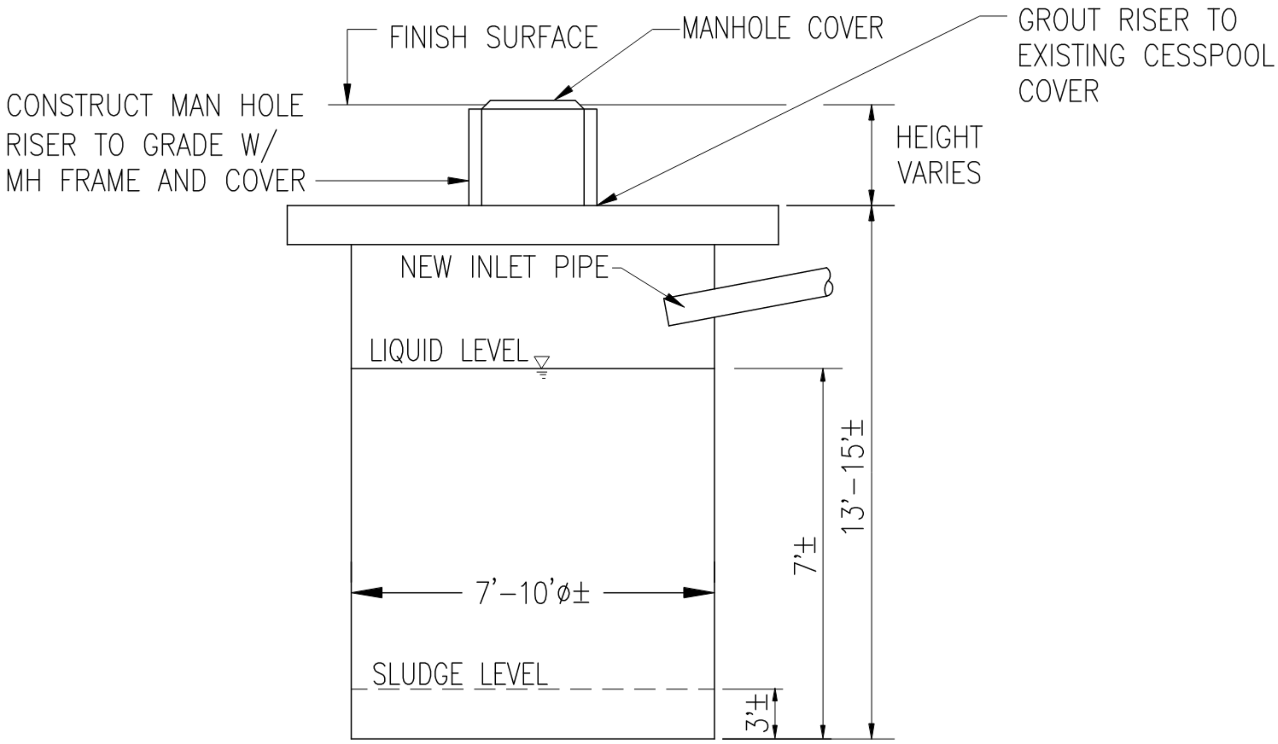
ABSORPTION BED TYP. DETAILS

SCALE: NOT TO SCALE



ABSORPTION BED TYP. DETAILS
FOR HARD ROCK SOIL CONDITION

SCALE: NOT TO SCALE

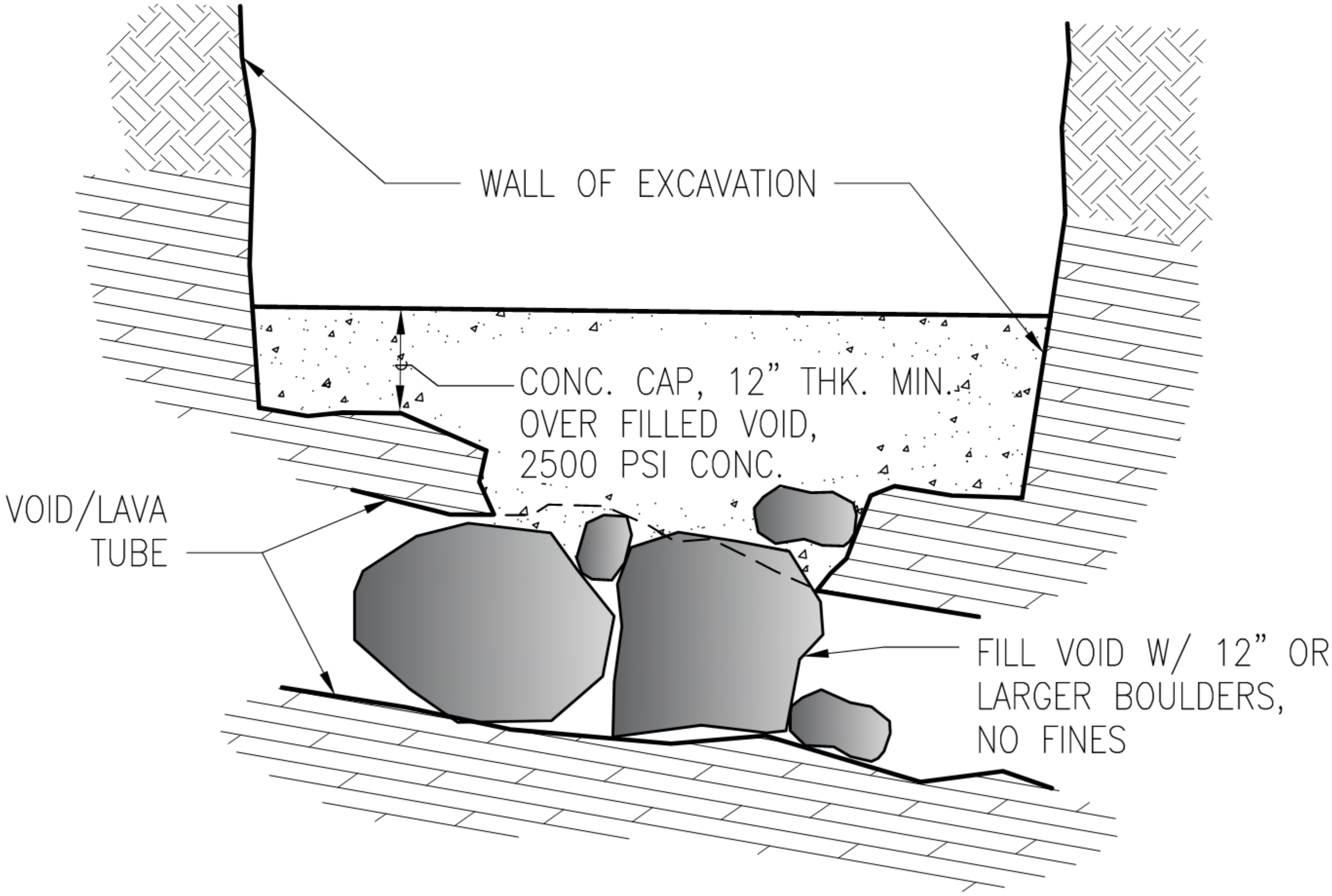


TYPICAL SEEPAGE PIT DETAIL

SCALE: NOT TO SCALE

G

Appendix G - Lava Tube Backfill Detail



LAVA TUBE BACKFILL DET.

SCALE: NOT TO SCALE

H

Appendix H – Percolation Test Results

DEPARTMENT OF HEALTH - WASTEWATER BRANCH
INDIVIDUAL WASTEWATER SYSTEM (IWS) - SITE EVALUATION / PERCOLATION TEST

Date / Time: 7/31/23 8:00 am Test Performed by: Austin Ah Hee

Owner: Karl Stewart Ragan TMK: (3) 9 - 5 - 025 : 019

Elevation: 700 feet
Depth to Groundwater Table: N/A feet below grade
Depth to Bedrock (if observed): N/A feet below grade
Diameter of Hole: 12 inches
Depth to Hole Bottom: 4 feet below grade

Depth, inches below grade	Soil Profile (color, texture, other)
0-48	Soft brown dirt

PERCOLATION READINGS:
Time 12 inches of water to seep away: 93 minutes
Time 12 inches of water to seep away: 84 minutes

Check one:

____ Percolation tests in sandy soils, recorded time intervals and water drops at least every 10 minutes for at least 1 hour.

X Percolation tests in no-sandy soils, presoaked the test hole for at least 4 hours. Recorded time intervals and water drops at least every 10 minutes for 1 hour of time for the first 6 inches to seep away in greater than 30 minutes record time intervals and water drops at lest every 30 minutes for 4 hours or until 2 successive drops do not vary by more than 1/16 inch.

Time Interval	Drop in Inches	Time Interval	Drop in Inches
30	3-1/4"		
30	3-1/4"		

Percolation Rate (time/final water level drop): ~10 minutes/inches

As the engineer responsible for gathering and providing site information and percolation test results, I attest to the fact that above site information is accurate and that the site evaluation was conducted in accordance with the provisions of Chapter 11-62, "Wastewater Systems" and the results were acceptable. I also attest that three feet of suitable soil exist between the bottom of the soil absorption system and the groundwater table or any other limiting layer.

Engineer's Signature/Stamp Date

DEPARTMENT OF HEALTH - WASTEWATER BRANCH
INDIVIDUAL WASTEWATER SYSTEM (IWS) - SITE EVALUATION / PERCOLATION TEST

Date / Time: 7/31/23 1:30 pm Test Performed by: Austin Ah Hee

Owner: County of Hawai'i TMK: (3) 9 - 5 - 024 : 011

Elevation: 670 feet
Depth to Groundwater Table: N/A feet below grade
Depth to Bedrock (if observed): N/A feet below grade
Diameter of Hole: 12 inches
Depth to Hole Bottom: 4.5 feet below grade

Depth, inches below grade	Soil Profile (color, texture, other)
0-54	Soft brown dirt

PERCOLATION READINGS:
Time 12 inches of water to seep away: 83 minutes
Time 12 inches of water to seep away: 81 minutes

Check one:

____ Percolation tests in sandy soils, recorded time intervals and water drops at least every 10 minutes for at least 1 hour.

X Percolation tests in no-sandy soils, presoaked the test hole for at least 4 hours. Recorded time intervals and water drops at least every 10 minutes for 1 hour of time for the first 6 inches to seep away in greater than 30 minutes record time intervals and water drops at lest every 30 minutes for 4 hours or until 2 successive drops do not vary by more than 1/16 inch.

Time Interval	Drop in Inches	Time Interval	Drop in Inches
30	4-1/2"		
30	4-1/2"		

Percolation Rate (time/final water level drop): ~7 minutes/inches

As the engineer responsible for gathering and providing site information and percolation test results, I attest to the fact that above site information is accurate and that the site evaluation was conducted in accordance with the provisions of Chapter 11-62, "Wastewater Systems" and the results were acceptable. I also attest that three feet of suitable soil exist between the bottom of the soil absorption system and the groundwater table or any other limiting layer.

Engineer's Signature/Stamp Date

DEPARTMENT OF HEALTH - WASTEWATER BRANCH
INDIVIDUAL WASTEWATER SYSTEM (IWS) - SITE EVALUATION / PERCOLATION TEST

Date / Time: 8/7/23 8:00 pm Test Performed by: Austin Ah Hee

Owner: Sophia Rhonda Lorenzo TMK: (3) 9 - 5 - 026 : 071

Elevation: 760 feet

Depth to Groundwater Table: N/A feet below grade

Depth to Bedrock (if observed): N/A feet below grade

Diameter of Hole: 12 inches

Depth to Hole Bottom: 4 feet below grade

Depth, inches below grade	Soil Profile (color, texture, other)
0-48	Hard brown dirt, some rocks

PERCOLATION READINGS:

Time 12 inches of water to seep away: 180 minutes

Time 12 inches of water to seep away: 180 minutes

Check one:

____ Percolation tests in sandy soils, recorded time intervals and water drops at least every 10 minutes for at least 1 hour.

X Percolation tests in no-sandy soils, presoaked the test hole for at least 4 hours. Recorded time intervals and water drops at least every 10 minutes for 1 hour of time for the first 6 inches to seep away in greater than 30 minutes record time intervals and water drops at lest every 30 minutes for 4 hours or until 2 successive drops do not vary by more than 1/16 inch.

Time Interval	Drop in Inches	Time Interval	Drop in Inches
30	2"		
30	2"		

Percolation Rate (time/final water level drop): 15 minutes/inches

As the engineer responsible for gathering and providing site information and percolation test results, I attest to the fact that above site information is accurate and that the site evaluation was conducted in accordance with the provisions of Chapter 11-62, "Wastewater Systems" and the results were acceptable. I also attest that three feet of suitable soil exist between the bottom of the soil absorption system and the groundwater table or any other limiting layer.

Engineer's Signature/Stamp Date

DEPARTMENT OF HEALTH - WASTEWATER BRANCH
INDIVIDUAL WASTEWATER SYSTEM (IWS) - SITE EVALUATION / PERCOLATION TEST

Date / Time: 8/7/23 12:00 pm Test Performed by: Austin Ah Hee

Owner: Baron Yamamoto Trust TMK: (3) 9 - 5 - 024 : 068

Elevation: 665 feet

Depth to Groundwater Table: N/A feet below grade

Depth to Bedrock (if observed): N/A feet below grade

Diameter of Hole: 12 inches

Depth to Hole Bottom: 4 feet below grade

Depth, inches below grade	Soil Profile (color, texture, other)
0-48	Soft brown dirt, some small rocks

PERCOLATION READINGS:

Time 12 inches of water to seep away: 110 minutes

Time 12 inches of water to seep away: 94 minutes

Check one:

____ Percolation tests in sandy soils, recorded time intervals and water drops at least every 10 minutes for at least 1 hour.

X Percolation tests in no-sandy soils, presoaked the test hole for at least 4 hours. Recorded time intervals and water drops at least every 10 minutes for 1 hour of time for the first 6 inches to seep away in greater than 30 minutes record time intervals and water drops at lest every 30 minutes for 4 hours or until 2 successive drops do not vary by more than 1/16 inch.

Time Interval	Drop in Inches	Time Interval	Drop in Inches
30	4-1/4"		
30	4-1/4"		

Percolation Rate (time/final water level drop): 8 minutes/inches

As the engineer responsible for gathering and providing site information and percolation test results, I attest to the fact that above site information is accurate and that the site evaluation was conducted in accordance with the provisions of Chapter 11-62, "Wastewater Systems" and the results were acceptable. I also attest that three feet of suitable soil exist between the bottom of the soil absorption system and the groundwater table or any other limiting layer.

Engineer's Signature/Stamp Date

DEPARTMENT OF HEALTH - WASTEWATER BRANCH
INDIVIDUAL WASTEWATER SYSTEM

FALLING HEAD TEST PROCEDURE

1. Preparing Percolation Test Hole(s)
   1. Dig or bore a hole, four to twelve inches in diameter with vertical walls to the approximate depth of the soil absorption system (bottom of trench or bed).
   2. Scratch the side wall and bottom to remove any smeared soil and remove loose material.
   3. Place one inch of coarse sand or gravel on bottom.

B. Determine Percolation Rate
   1. Place twelve inches of water in hole and determine time to seep away. Record this time on the site evaluation form.
   2. Repeat step B.1. above. Also record this time on the site evaluation form.
   3. If the time of the second test is less than 10 minutes go to Step C, if not skip to Step D.

C. Sandy (granular) Soils
   1. Establish a fixed reference point, add water to six inches above gravel and measure water level drops every ten minutes for 1 hour.
   2. Use a shorter time interval if first six inches seeps away in ten minutes or less.
   3. Refill when necessary, do not exceed six inches of water.
   4. Record time intervals and water drops on site evaluation form.
   5. Use final water level drop interval to calculate percolation rate. (Step E)

D. Other Soils (non-granular, e.g. silt, loams & clays)
   1. Maintain at least twelve inches of water in the hole for at least four hours to presoak soil.
   2. Do not remove water remaining after four hours.
   3. Permit soil to swell at least 12 hours. (Dry clayey soils should be soaked and permitted to swell for longer periods to obtain stabilized percolation rates).
   4. After swelling, remove loose material on top of gravel.
   5. Use fixed referenced point, adjust water level to six inches above gravel and measure water level drop.
   6. If the first six inches of water seeps away in less than 30 minutes, measure water level drops every ten-minutes and run for one hour.
   7. If the first six inches of water takes longer than 20 minutes to seep away, use 30 minute time intervals for four hours or until two successive drops do not vary by more than one-sixteenth inch (stabilized rate).
   8. Refill with water only when necessary, but no adjustment during last three readings except to the limit of the last drop. Do no exceed six inches of water.

E. Use final drop interval to calculate percolation rate and record on site evaluation form:

$$\frac{\text{Time Interval}}{\text{Water Level Drop}} = \text{Percolation Rate}$$