Naalehu Wastewater Treatment Plant

Preliminary Engineering Report

Prepared for

County of Hawaii, Department of Environmental Management

October 2018

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

THIS WORK WAS PREPARED BY ME OR UNDER MY SUPERVISION.

April 30, 2020

Signature | Expiration Date of the License

Brown AND Caldwell
1955 Main Street, Suite 200
Wailuku, Hawaii 96793

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List of Abbreviations

AB	aggregate base
AC	asphalt concrete
BMP	Best Management Practices
BOD_5	5-day biochemical oxygen demand
CCH	City and County of Honolulu
cfs	cubic feet per second
COH	County of Hawaii
CFR	Code of Federal Regulations
DNA	deoxyribonucleic acid
DEM	Department of Environmental Management
DOH	Department of Health
ELLF	end-of-lamp-life
FIRM	Flood Insurance Rate Map
FOG	fats, oils, and grease
ft^3	cubic feet
FTE	full-time equivalent
GAC	granular activated carbon
gpm	gallons per minute
H_2S	hydrogen sulfide
HAR	Hawaii Administrative Rules
HDPE	high density polyethylene
HELCO	Hawaii Electric Light Company
hp	horsepower
hp/Mgal	horsepower per million gallons
hr	hour
hp-hr	horsepower-hour
IWS	individual wastewater system
L	liter
lbs	pounds
LCC	large capacity cesspools
LPHO	low pressure high output
MBR	membrane bioreactor
Mg	milligrams
Mgal	million gallons
mm	millimeter
MSL	mean sea level
N	nitrogen
NPV	net present value
O&M	Operation and Maintenance
P	Phosphorus
Psi	pounds per square inch
RNA	ribonucleic acid
ROW	right-of-way
SR	slow rate
TSS	total suspended solids
UIC	Underground Injection Control
USEPA	United States Environmental Protection Agency
UV	ultraviolet
WQV	Water Quality Volume
WWTP	Wastewater Treatment Plant

Section 1

Introduction

1.1 Background

Naalehu is located in the Kau district of the Island of Hawaii. According to the 2010 United States Census, the total population for the Naalehu census designated place (CDP) was approximately 866 people.

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. 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 (COH) Department of Environmental Management (DEM) after a vote by the community.

In 1998, the U.S. Environmental Protection Agency (USEPA) promulgated regulations – 40 Code of Federal Regulations (CFR) 144.14 – which require the elimination of LCCs. As a result, the County intends to construct a new sewer collection system located primarily within public right-of-way (ROW) and replace the existing LCCs with a wastewater treatment plant (WWTP) to address the wastewater treatment and disposal needs of the Naalehu community.

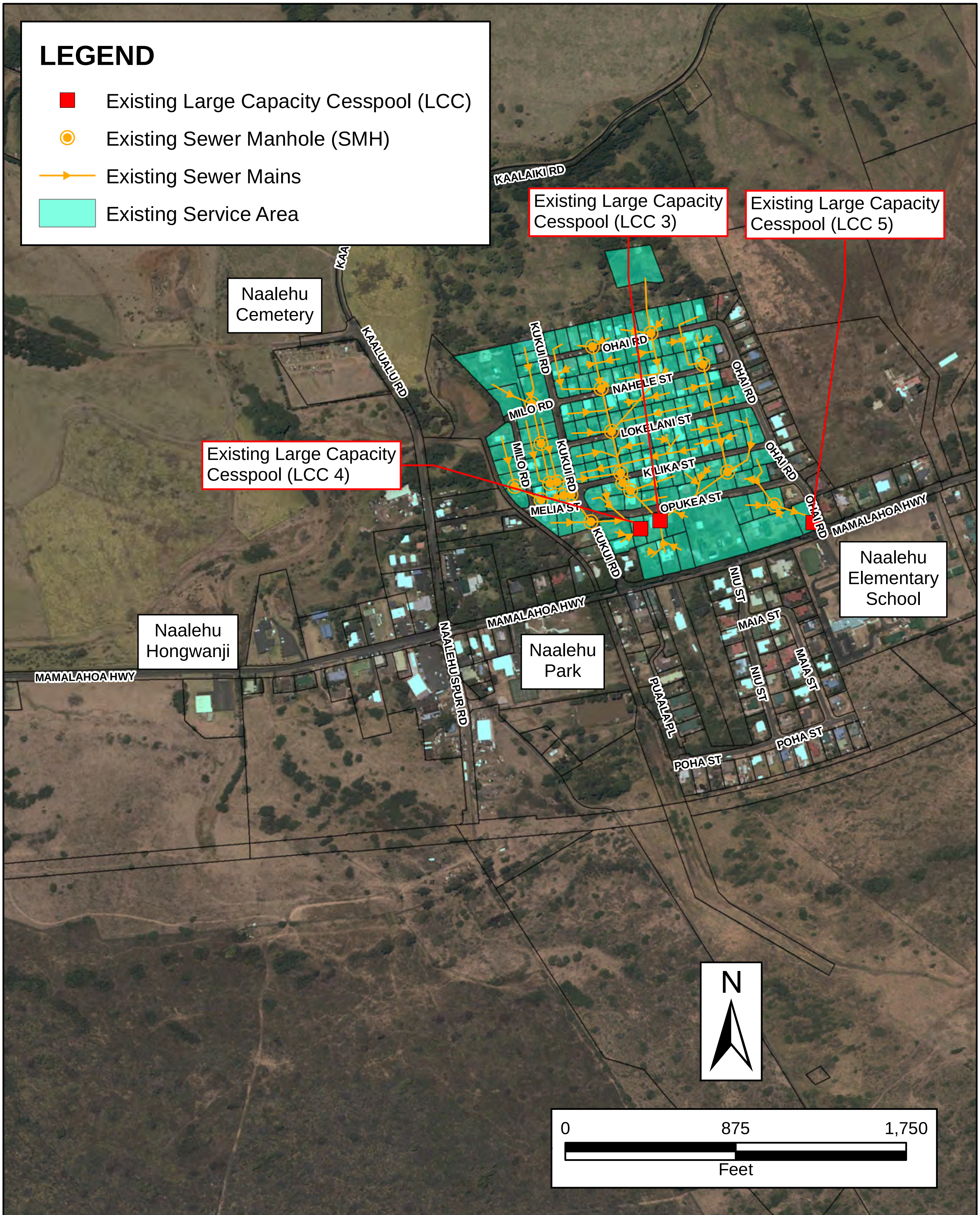
This report summarizes a proposed WWTP needed in order to treat and dispose of the wastewater flow that is currently discharged to the LCCs, plus additional sewer connections. While the initial plan was to construct a community septic tank and convert one of the existing LCCs to a seepage pit, subsequent evaluation determined this was not feasible, as discussed further in Section 7.5. The report presents the existing and estimated future flows and loads to the WWTP, the proposed treatment processes, recommendation for the WWTP upgrades needed to meet the future treatment needs, and an initial estimate of the cost to construct the improvements project.

1.2 Existing System

Figure 1-1 shows the collection system network and service areas for the LCCs. The LCCs in Naalehu are numbered 3, 4, and 5; LCCs 1 and 2 are located in Pahala. The collection system is a network of gravity sewers that discharge to three existing LCCs. 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 about 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 private property, which transmit wastewater to the LCCs. There are approximately 13 manholes in the sewer system (M&E Pacific, December 2004). 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 because the existing sewer mains run under homes and in easements on private property. The County intends to dissolve the majority of the existing easements when the new collection system is constructed and the easements are no longer needed. The sewer system is not designed to collect storm water.

1.3 Report Contents

Section 2 presents flow and load projections for the new WWTP. Section 3 evaluates effluent management options, and the treatment requirements for the preferred option. Section 4 presents evaluations conducted to develop the preliminary design of the proposed WWTP, which is presented in Section 5. An implementation plan is briefly presented in Section 6, followed by discussion of other treatment options that were considered and evaluated in Section 7. The report concludes with a WWTP site selection evaluation summary in Section 8.



Brown AND Caldwell	SCALE AS SHOWN JOB NO.: 151494	NAALEHU WASTEWATER TREATMENT PLANT Existing Naalehu Wastewater System	FIGURE 1-1

Section 2

Flow and Load Projections

This section summarizes the flow and load projections for the new WWTP.

2.1 Service Area

Figure 2-1 shows the initial service area for the new WWTP. The Kau Community Development Plan indicates that the sewer system may eventually be expanded to service the entire community, as shown in Figure 2-2; however, the initial collection system and WWTP presented in this report will service the properties currently connected to the LCCs or newly accessible to the new collection system. Although this report does not include design for the full buildout service area, the proposed WWTP has been designed to accommodate modifications within the proposed site for additional expansion of the service area in the future.

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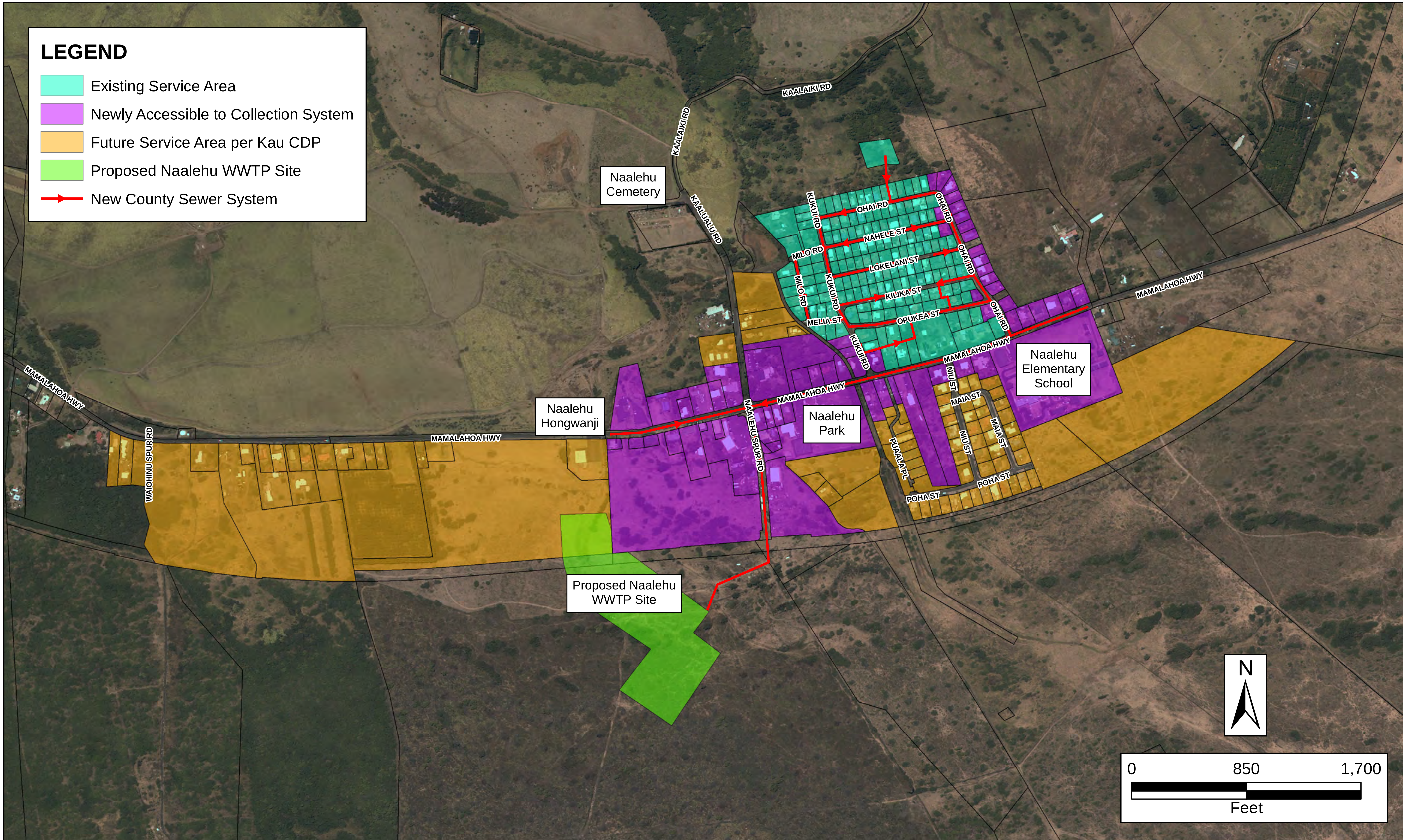
NAALEHU WASTEWATER TREATMENT PLANT

Naalehu LCC Conversion Project

FIGURE 2-1



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NAALEHU WASTEWATER TREATMENT PLANT

Naalehu WWTP Full Buildout Condition

FIGURE

2-2

2.2 Flow Projections for LCC Conversion Project

Hawaii Administrative Rules (HAR) chapter 11-62 requires proposed county WWTPs be designed in accordance with their respective county standards. If a county does not have design standards in place then the design standards of the City and County of Honolulu (CCH) shall be used. The County of Hawaii has not developed its own standards, so wastewater flow projections were developed using the CCH current (2017) wastewater standards. Table 2-1 summarizes the flow projections for the LCC Conversion Project. Details are provided in Appendix A.

Table 2-1. Naalehu LCC Conversion Project Flow Projections

Description	Value	Peaking Factor
Average dry weather flow	225,000 gallons per day	1.0
Peak day wet weather flow	565,000 gallons per day	2.5 [a]
Peak hour wet weather flow	490 gallons per minute	3.1

**Derived from Crites and Tchobanoglous, 1998*

The WWTP will be designed to accommodate the flow projections shown in the table.

2.3 Influent Characteristics

The properties within the existing service area are primarily residential, but do include several commercial, apartment, and industrial zoned parcels. The wastewater characteristics of the WWTP influent are assumed to be similar to typical domestic wastewater. Table 2-2 provides a summary of the assumed influent characteristics.

Table 2-2. Summary of Assumed Influent Characteristics

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

2.4 Influent Mass Loads

Table 2-3 summarizes the projected loads to the WWTP, based on the proposed average dry weather capacity of 225,000 gallons per day and the influent characteristics presented in Table 2-2.

Table 2-3. Projected Influent Mass Loads

Description	Value
BOD_5	565 lbs./day
TSS	565 lbs./day
Total nitrogen	75 lbs./day
Total phosphorus	13 lbs./day

2.5 Mass Loads to the Environment via Existing LCCs

Currently, the connected properties discharge without treatment to three LCCs, as shown in Figure 1-1. These types of cesspools are a public health and environmental concern because of their likelihood to release disease causing pathogens and other contaminants, such as nitrate, to groundwater. In addition, properties that will be newly accessible to the new collection system currently discharge contaminants to the environment via individual wastewater systems (IWS). The current annual mass loads to the environment via the existing LCCs based on the flow projections and assumed wastewater characteristics presented above are summarized in Table 2-4.

Table 2-4. Mass Loads to the Environment via Existing LCCs and Newly Accessible Property IWS

Parameter	Annual Load
BOD_5	206,000 lbs./year
TSS	206,000 lbs./year
Total N	27,000 lbs./year
Total P	4,700 lbs./year

Section 3

Effluent Management Options and Regulatory Requirements

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

3.1 Effluent Management Options

There are few effluent management options available for the community, as discussed below.

3.1.1 Ocean Discharge

The coastal waters in the Naalehu area are classified as “AA” marine waters by DOH. HAR 11-54 does not allow zones of mixing in waters up to a distance of 300 meters (one thousand feet) off shore if there is no defined reef area and if the depth is greater than 18 meters (ten fathoms). The water quality criteria for nutrients for Class AA embayments are listed in Table 3-1. 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.

Table 3-1. Nutrient Water Quality Standards for Class AA Embayments

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

3.1.2 Subsurface Disposal via Injection Wells

Per Hawaii Administrative Rules (HAR), Title 11, Chapter 23, disposal to groundwater via an injection well is not allowed mauka of the State of Hawaii Department of Health (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 option. 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.

3.1.3 Water Recycling

An irrigation assessment was prepared to assess 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 3-1 is a summary of the assessment that shows there is typically no irrigation demand for three months of the year 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 alone is not a viable primary effluent management strategy for the community. See Section 7 for additional discussion on potential water recycling.

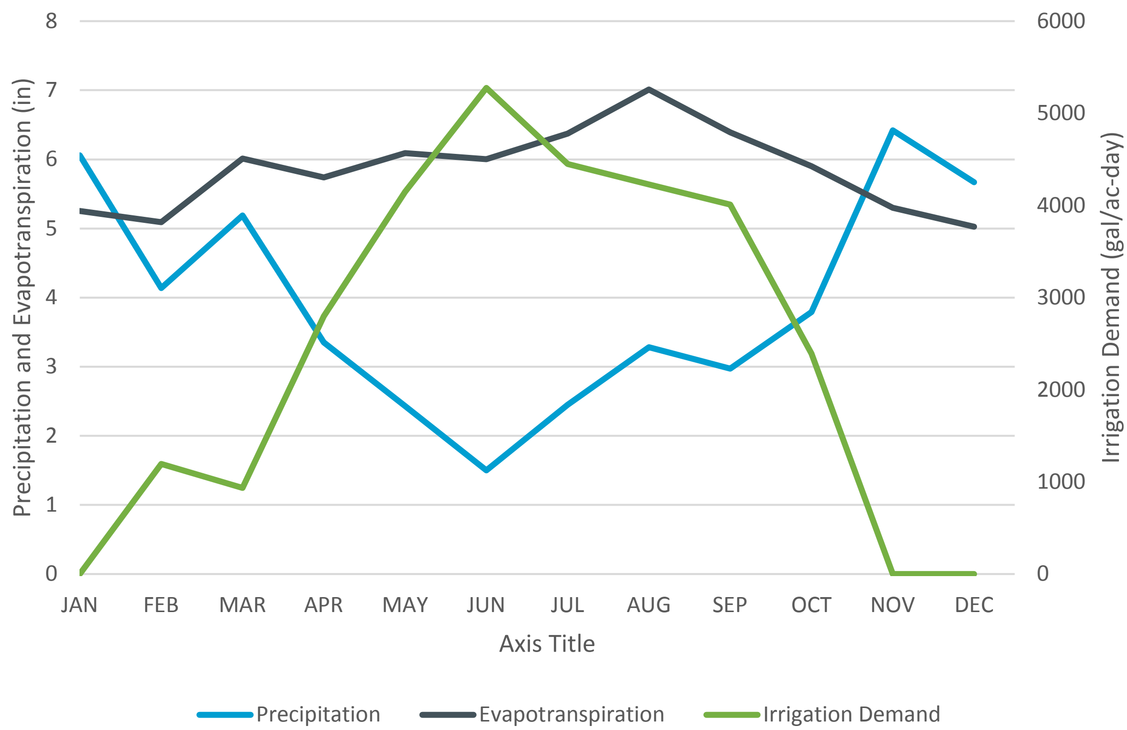


Figure 3-1. Irrigation Demand Assessment

3.1.4 Land Treatment

The USEPA defines land treatment as "the application of appropriately pre-treated municipal and industrial wastewater to the land at a controlled rate in a designed and engineered setting. The purpose of the activity is to obtain beneficial use of these materials, to improve environmental quality, and to achieve treatment goals in a cost-effective and environmentally sound manner" (USEPA, September 2006).

Land treatment systems rely on soil and vegetation to achieve treatment objectives, rather than energy-intensive mechanical equipment. As such, they are considered to be a form of "natural" treatment (Crites, et. al., 2014).

Land treatment is not a new concept. "Land application of wastewater was the first 'natural' technology to be rediscovered (after passage of the Clean Water Act of 1972). In the 1840s in England, it was recognized as avoiding water pollution as well as returning nutrients in wastewater back to the land. In the 19th century it was the only acceptable method for waste treatment, but it gradually slipped from use with the invention of modern devices" (Crites, et. al., 2014).

The soil at the proposed WWTP location are suitable for slow rate (SR) land treatment. The proposed WWTP effluent management system will make use of an area containing Naalehu medial silty clay loam soil (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. Additional discussion is provided in Section 5.4.

3.1.5 Drain Field

A drain field (i.e., 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 standards indicate approximately 25 acres of leach fields would be required to accommodate the anticipated flow and provide a 100-percent redundant drain field per the requirements. There is insufficient soil area available at the proposed WWTP site to construct a drain field of this size. Therefore, this option is considered to be not feasible.

3.1.6 Recommendation

A slow rate land treatment system is recommended for effluent management, as it is the only feasible effluent management system available to the community.

3.2 Treatment Requirements

The DOH regulates land treatment as “land disposal” per HAR 11-62. Table 3-2 lists the effluent requirements for land disposal applicable to the project that were in effect at the time this report was prepared.

Table 3-2. Applicable HAR 11-62 Land Disposal Requirements

Description	Value	HAR Reference
BOD_5	30 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 4

Wastewater Treatment Evaluations

This section presents the evaluations conducted in the conceptual development of the proposed WWTP.

4.1 Preliminary Treatment

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

4.1.1 Screening

Screening is recommended to protect the downstream system operations from large objects, debris, and rags that can be present in wastewater. Aerated lagoon treatment systems require a minimum of coarse screens to protect the aeration equipment. The industry trend is towards finer screening systems that remove greater amounts of debris from the waste stream; screens with 6-millimeter (mm) (¼-inch) openings are frequently used for activated sludge treatment systems. An aerated lagoon treatment system can benefit from ¼-inch screening to reduce the amount of floatable debris on the lagoon shoreline, creating a cleaner facility that is less attractive to birds. Since the Naalehu WWTP will not be continuously staffed, a screening process requiring minimal attention is desirable. Furthermore, the screenings volume 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.

4.1.1.1 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 4-1 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 two in-channel cylindrical screens, one will be on-line when the other is redundant, plus a bypass channel with manually cleaned bar rack.

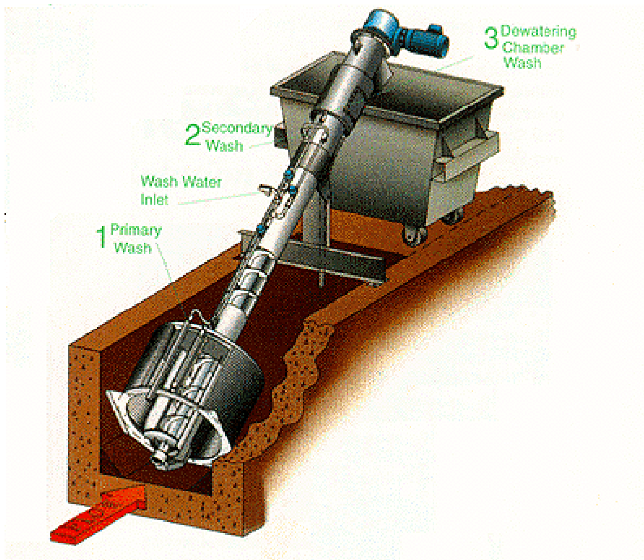


Figure 4-1. In-Channel Cylindrical Screen

4.1.2 Influent Flow Measurement

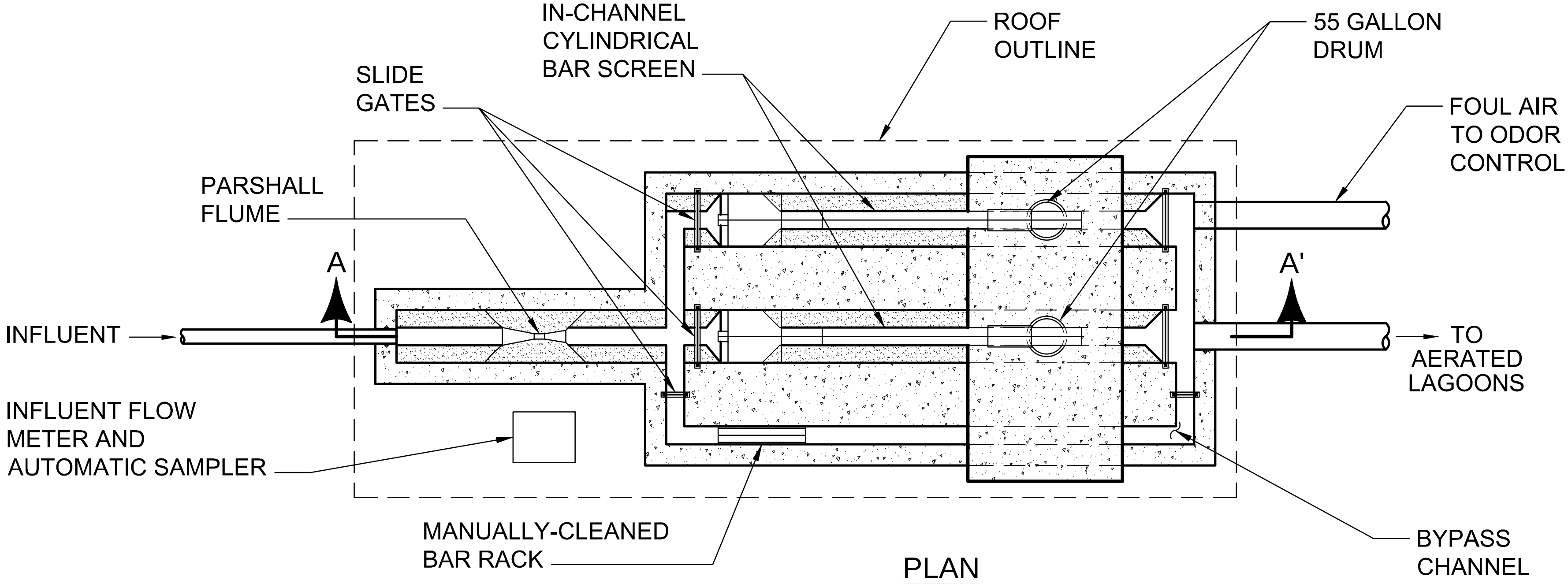
Influent flow measurement is recommended to allow assessment of flows and loads to the biological treatment process, and to assess the biological treatment process performance. A Parshall flume will be provided upstream of the screening system to continuously record influent flow rates. Parshall flumes 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.

4.1.3 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.

4.1.4 Preliminary Design of Headworks

Figure 4-2 shows a plan and section of the proposed headworks. Influent wastewater will enter the upstream end of the headworks channel. Stop plates will be used to divert the flow to one of the two the in-channel cylindrical screens, or to the manually-cleaned bar rack. The slide gates will be designed to allow automatic overflow to the other channels in the event of mechanical screen failure. The washed and compacted screenings will be deposited in a bag or 55-gallon drum for periodic disposal. The Parshall flume and automatic refrigerated composite sampler will be located upstream of the screens. The channels will be covered with fiberglass or aluminum plate to facilitate foul air collection, which will be conveyed to an odor control unit. In addition, a free-standing roof structure will be constructed over the headworks to protect the operators and equipment from rain and sun.



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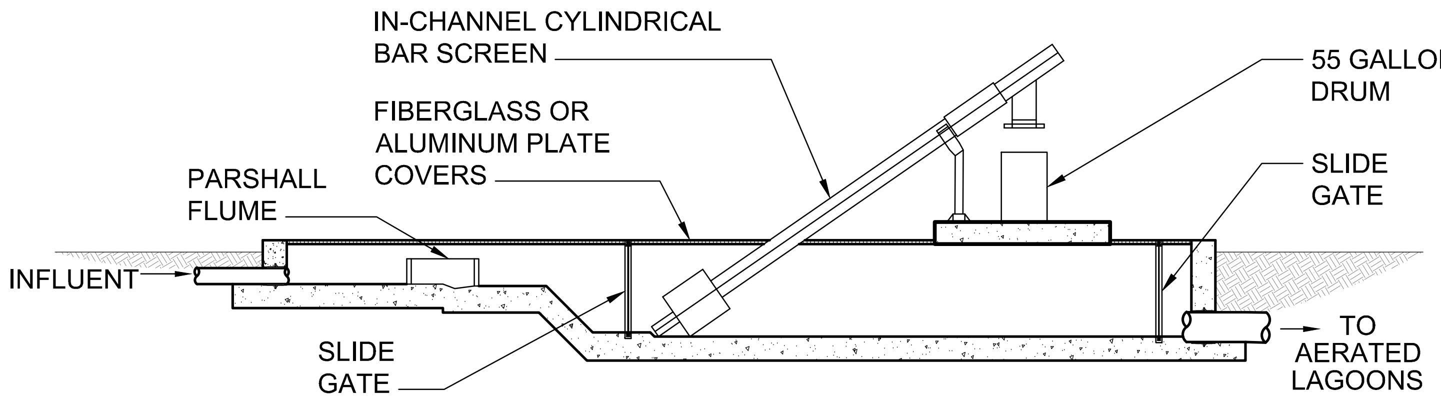


SECTION A - A'

Brown AND Caldwell

SCALE:

JOB NO: 150440

NAALEHU WASTEWATER TREATMENT PLANT

HEADWORKS

FIGURE

4-2

4.1.5 Odor Control

A prime 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.

4.1.5.1 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. Package 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 4-3 illustrates the process. The County currently operates GAC scrubbers at other facilities and purchases the GAC media in bulk to reduce costs.

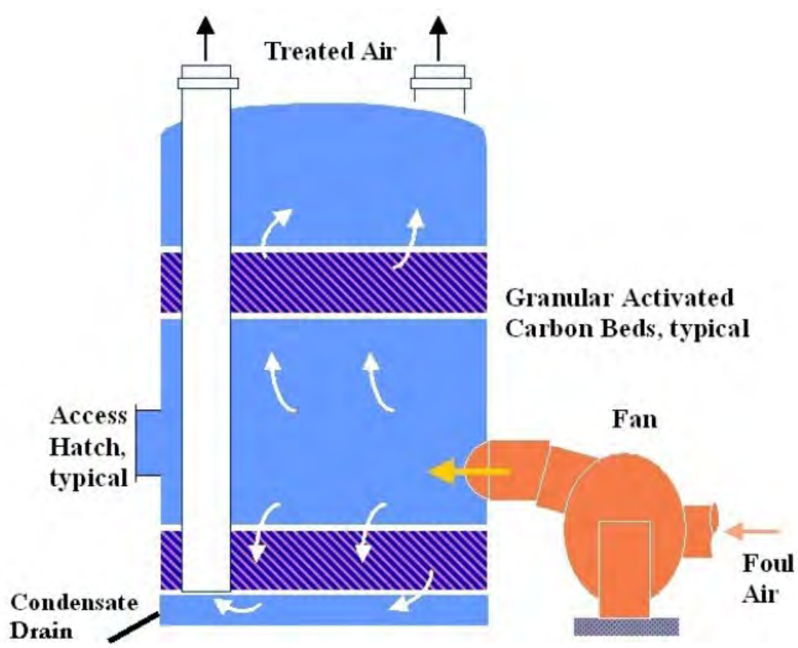


Figure 4-3. Activated Carbon Scrubber (GAC)

4.2 Aerated Lagoon Treatment System

The biological wastewater treatment needs at the Naalehu WWTP will be met by a series of aerated lagoons. A floating cover will be installed on the last cell to reduce algae in the effluent. The preliminary design of the aerated lagoon treatment system is developed in this section.

4.2.1 Aerated Lagoon Kinetics

The Naalehu WWTP design is reliant on partial mix aerated lagoon environments to provide the community's wastewater treatment needs for the initial buildout condition. Partial mix aerated lagoon kinetics are described below.

4.2.1.1 Partial Mix Model

Partial mix aerated lagoons are based on the concept of allowing solids to settle in lagoons while providing only enough aeration and mixing to meet the oxygen requirements of the naturally occurring micro-organisms in the system. The solids tend to settle in areas of the lagoon that are subject to less mixing energy, where they anaerobically decompose. Infrequent sludge removal is required to maintain sufficient lagoon treatment volume.

Removal of BOD_5 in partial-mix aerated lagoons depends on the hydraulic detention time. The design model for partial mixed ponds of equal size in series is (Crites, et. al., 2006):

$$\frac{Cn}{Co} = \frac{1}{[1+(kt/n)^n}$$

Where Cn = effluent BOD_5 concentration in cell n, mg/L

Co = influent BOD_5 concentration, mg/L

k = partial-mix first-order reaction rate constant, day^{-1}

t = total hydraulic residence time in the lagoon system, day

n = number of cells in the series

If the lagoons in a system are of unequal size, then the equation must be applied to each lagoon in the series. The Ten-States Standards recommends using a value of 0.276 day^{-1} at 20 ºC for the reaction rate constant (Great Lakes – Upper Mississippi River Board, 1997).

4.2.1.2 Mixing in Lagoon Systems

The energy required for mixing in aerated lagoon systems is generally provided by the aeration system. For partial mix systems the aeration system is sized to provide enough oxygen to maintain aerobic conditions and no more. For mechanical aeration systems energy input of at least 30 horsepower per million gallons (hp/Mgal) of lagoon volume is required to keep solids in suspension (Rich, 1999).

4.2.2 Aeration in Lagoon Systems

Oxygen requirements in aerated lagoon systems are based on the organic loading entering the cell. Supplying oxygen at a rate of 1.5 times the BOD_5 mass entering the cell has been found to be sufficient to treat the wastewater. The following equation is used to estimate the oxygen transfer rate (Crites, et. al., 2006):

$$N = \frac{N_a}{\alpha\left[\frac{(C_{sw} - C_L)}{C_S}\right](1.025)^{(Tw-20)}}$$

Where N = Equivalent oxygen transfer to tap water at standard conditions (lbs/hr)

N_a = Oxygen required to treat the wastewater (lbs/hr)

α = (oxygen transfer in wastewater)/(oxygen transfer in tap water)

C_{sw} = $\beta(C_{ss})P$ = oxygen saturation value of the waste, mg/L

β = wastewater saturation value/tap water oxygen saturation value = 0.9

C_{ss} = tap water oxygen saturation value at temperature Tw

P = ratio of barometric pressure at the site to barometric pressure at sea level

C_L = minimum dissolved oxygen concentration to be maintained

C_S = oxygen saturation value of tap water at 20°C and 1 atm pressure

T_w = wastewater temperature, °C

Oxygen can be supplied to aerated lagoon systems using mechanical aerators or diffused aeration systems. Mechanical aerators are commonly rated by the number of pounds of oxygen the units will supply under standard conditions per horsepower-hour (lbs. O_2/hp-hr). Diffused air requirements are calculated using the following equation (Crites and Tchobanoglous, 1998):

$$Q_{air} = \frac{W_{oxygen}}{(AOTE)(O_2)(\gamma_{air})(1440)}$$

Where Q_{air} = Required air flow (ft^3/min)

W_{oxygen} = Oxygen requirements (lbs/day)

$AOTE$ = Actual oxygen transfer efficiency, expressed as a fraction

O_2 = Fractional percent of oxygen in air by weight (0.2315)

γ_{air} = Specific weight of air (0.075 lbs/ft^3 at 1 atmosphere and 20°C

The oxygen transfer efficiency of a diffused air system is a function of the air bubble size and the depth of the water column. Smaller air bubbles result in higher oxygen transfer efficiencies than larger bubbles, as do diffusers that are set at deeper depths within the water column.

4.2.2.1 High Speed Floating Aerators

High-speed floating aerators are commonly used for aerated lagoon systems. The units consist of a motor and impeller attached to a float. The units are typically anchored to the lagoon shore using cables. High-speed floating aerators are designed to pump water from the lagoon and spray it into the air, allowing oxygen to diffuse into the water droplets. The high-speed floating aerators can be outfitted with draft tubes to enhance deep water lagoon mixing or anti-erosion plates to ensure water is drawn from the surface. Figure 4-4 shows a typical high-speed floating aerator, and a photo of a unit in operation.

Figure 4-4. High Speed Floating Aerator

Advantages of this system include low capital costs, relatively high oxygen transfer efficiency, good mixing efficiency, and simple operation and maintenance. The chief disadvantage of the system is the creation of aerosols as the lagoon water is sprayed into the air.

Manufacturers of this type of aerator include Aqua-Aerobics, Aerator Products and Europlec/Aeromix Systems Inc.

High-speed floating aerators are recommended for the Naalehu WWTP due to their relatively high oxygen transfer efficiency, low capital cost, and simple operation and maintenance. High-speed floating aerators are easy to remove from service, and can be easily moved between lagoons or cells, if needed.

4.2.3 Aerated Lagoon Configuration

The normal operating condition for the Naalehu WWTP will be to operate the four lagoon cells in series as partial mix environments. Figure 4-5 is a schematic representation of the normal operating mode. The fourth cell will be outfitted with a floating cover to preclude algae growth. Having four lagoons will allow the County to take a lagoon out of service for maintenance.

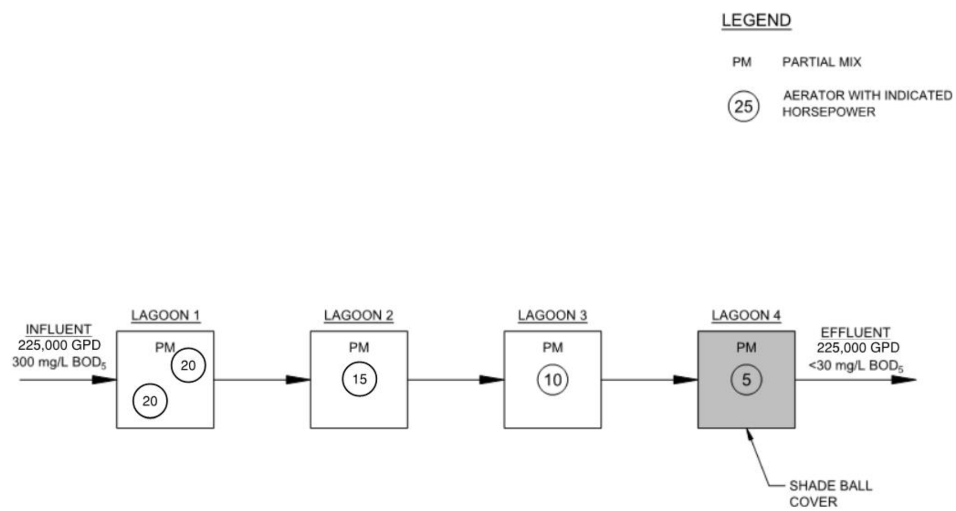


Figure 4-5. Normal Lagoon Configuration Schematic

Table 4-1 summarizes the results of the aeration and mixing calculations for the normal operational configuration treating the design average dry weather flow rate of 225,000 gallons per day. Comparison of the minimum aerator requirements shown in Table 4-1 with the proposed aerator layout shown in Figure 4-5 reveals that the aerator power supplied exceeds the minimum requirements. An aerator control system will be provided that will intermittently turn the aerators on and off in accordance with the operator settings to supply sufficient oxygen to the system.

Table 4-1. Normal Configuration Aeration and Mixing Requirements

Cell	Volume (gal)	Influent BOD_5 (mg/L)	Effluent BOD_5 (mg/L)	Minimum Aerator Requirement (hp)	Mixing Density (hp/Mgal)
1	145,000	300	129	32	30
2	145,000	129	55	14	13
3	145,000	55	24	6	6
4	145,000	Redundant for maintenance purposes			

4.2.4 Lagoon Liner

Lagoon liners are required by DOH to prevent wastewater seepage into the ground. The liner will be exposed to sunlight, so resistance to ultraviolet light (UV) degradation is a key factor in the selection of the liner material, as is the compatibility of the material with typical domestic wastewater characteristics and ease of liner maintenance. An 80-mil textured high density polyethylene (HDPE) geomembrane is recommend for this application.

Textured HDPE is known to have excellent UV resistance, good chemical resistance, and generally is not affected by fats, oils, and grease (FOG). Maintenance of HDPE requires a specialty contractor who can complete fusion weld repairs. Unlike smooth HDPE, textured HDPE presents minimal slipping hazard to operations personnel. Furthermore, the anticipated useful service of an HDPE liner in typical Hawaii municipal wastewater treatment conditions is 25 to 30 years.

4.2.5 Lagoon Cover

In the normal operating mode, the final cell in the lagoon series will be covered in order to deprive algae of sunlight. This will reduce the algae concentration, which can increase total suspended solids (TSS) levels in the system effluent. The cover should float on the surface of the water, be UV resistant, suitable for windy environments, and allow for rainwater to pass through the cover to prevent ponding. A floating shade ball cover is proposed for this installation.

Floating shade ball covers have been used for decades in in the mining, water and wastewater treatment industries. Figure 4-6 shows the design elements of a typical shade ball, and Figure 4-7 shows how shade balls provide cover on a reservoir. In addition to reducing algae growth, shade ball covers deter waterfowl from storage ponds. The black, UV-stable HDPE resin has known to withstand a range of challenging chemical and environmental conditions. Table 4-2 summarizes technical data for the balls.

Table 4-2. Lagoon Shade Ball Cover Application Parameters

Requirement	Description
Algae Control	Balls – 90% shade coverage
Temperature	50^{0}C to 95^{0}C
Wind Resistance	Balls ballasted with potable water tested in winds of 120 mph (category 3 hurricane)
Waterfowl Safety	Waterfowl do not recognize ball-covered pond as a water body and will not nest on the unstable surface
Lifecycle/Warranty	The shade balls are warrantied for 10 years, with an expected resin life of 25+years
Operations and Maintenance	Self-cleaning, self-levelling and require little to no maintenance Balls will move out of the way of maintenance barge, and can be restrained with booms Little installation effort required Precipitation does not affect the cover
Sustainability	Resin is recyclable, paraben free and suitable for drinking water applications Ballast is potable water Resin can be made from recycled plastic
Environment	Balls have been installed in chemically harsh environments (mining industry), in drinking water reservoirs, and in tropical locations Balls reduce algae formation and corresponding disinfectant byproducts in chlorination applications

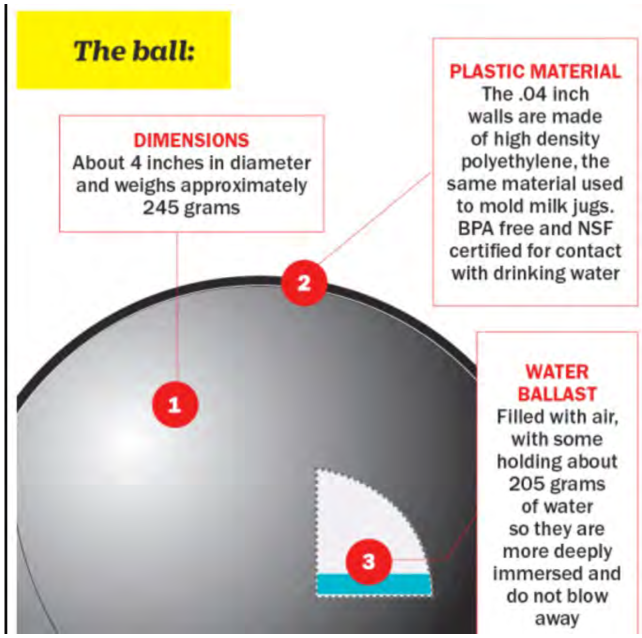


Figure 4-6. Floating HDPE Shade Balls

Figure 4-7. Floating shade balls with current and turbulence in reservoir.

4.2.6 Lagoon Sludge Management

Partial-mix aerated lagoons are designed to allow solids to settle to the bottom of the lagoon, forming a sludge layer. The sludge slowly anaerobically digests in the bottom of the lagoon. The mechanical aerators in the lagoon maintain an aerobic water cap at the surface of the lagoon that oxidizes any odors that are released from the anaerobic sludge layer at the bottom of the lagoon. Sludge is removed infrequently, typically every 20 to 30 years, when the sludge blanket thickness begins to affect treatment performance or in conjunction with lagoon liner replacement. Aerated lagoon operators typically monitor sludge blanket thicknesses semi-annually to assess sludge accumulation.

Sludge removal contractors are typically employed to dredge the solids, dewater, and haul to a landfill for disposal. Sludge from aerated lagoons is typically does not create nuisance odors when dewatered due to the long residence time in the bottom of the lagoon.

Alternatively, the sludge can be recycled if a permitted land application site is available and the sludge meets State and Federal requirements for land application or composted with green waste at a permitted composting facility. However, at the time this report was written there were no permitted land application sites or composting facilities permitted to take WWTP sludge on the island.

4.3 Subsurface Flow Constructed Wetland

A subsurface flow constructed wetland is recommended to provide additional treatment and polishing of the aerated lagoon effluent. It is anticipated that the aerated lagoon system will convert ammonia that is present in the wastewater influent into nitrate via a process called nitrification. A subsurface flow constructed wetland will remove this nitrogen from the wastewater via a process called denitrification. Reduction of nitrogen loading through the constructed wetland will decrease the area required for overland flow effluent management.

Subsurface flow wetlands consist of shallow lined basins that are filled with gravel media and planted with emergent wetland vegetation. Water is introduced to the gravel media layer and flows horizontally through the basin. The water level in the wetland is maintained below the gravel surface at all times. Treatment occurs through physical, chemical, and biological mechanisms as the water flows horizontally through the gravel media bed. Figure 4-8 is an illustration of the concept.

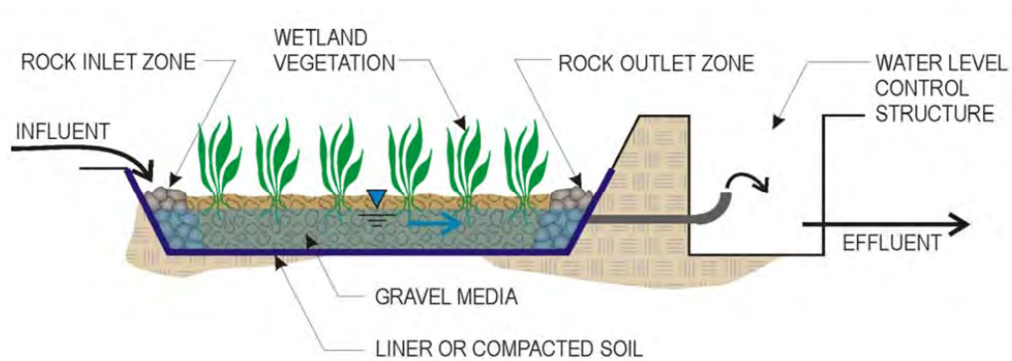


Figure 4-8. Subsurface Flow Constructed Wetland Concept

4.3.1 Denitrification in Subsurface Flow Constructed Wetlands

Denitrification is a biological process whereby nitrate molecules are transformed into nitrogen gas molecules by naturally-occurring bacteria. The denitrifying bacteria require five conditions for the process to occur:

- A place to grow.
- A source of nitrate.
- An anoxic (low-oxygen) environment.
- A source of carbon.
- Adequate water temperature.

The equation used to predict denitrification in subsurface flow constructed wetlands is shown below (Crites, et.al., 2014).

$$\frac{C_e}{C_o} = \exp(-K_T t)$$

where:

C_e = effluent nitrate-nitrogen concentration (mg/L)

C_o = influent nitrate-nitrogen concentration (mg/L)

K_T = temperature-dependent rate constant = $1.00(1.15)^{(T-20)}$ days^{-1} when T>1°C

t = hydraulic residence time (days)

Subsurface flow constructed wetlands are capable of providing additional treatment benefits beyond nitrogen reduction, such as removal of organic carbon, suspended solids, phosphorus, metals, trace organics, and pathogens. The additional treatment benefits are not primary design parameters but should be considered as additional polishing treatment benefits that may be realized for the Naalehu WWTP.

4.4 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 a calcium hypochlorite system and a UV system were evaluated.

4.4.1 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 sodium hypochlorite (bleach). Calcium hypochlorite decomposes in an exothermic reaction if exposed to moisture.

The solid can be directly applied to wastewater at very small WWTPs. Figure 4-9 shows a typical calcium hypochlorite feed system.

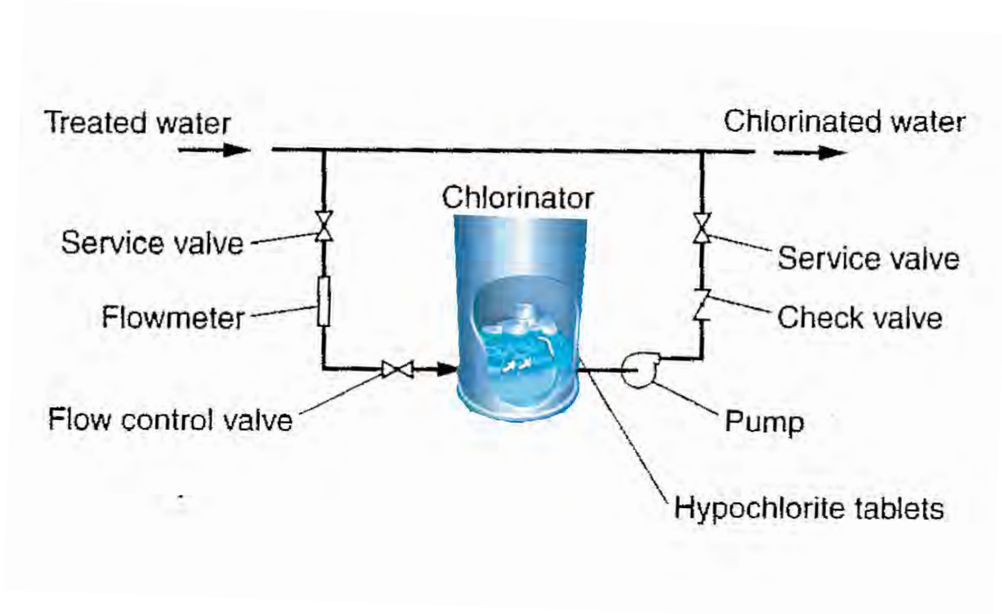


Figure 4-9. Typical Calcium Hypochlorite Feed System

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 4-3 summaries calcium hypochlorite characteristics.

Table 4-3. Calcium Hypochlorite Summary

Description	Characteristic
Transported form	Solid
Typical transported concentration	70%
Largest transported volume available	55 lb. pails
Decay Rate	Decays 3-5% per year
pH	N/A
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

4.4.1.1 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 4-4 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.

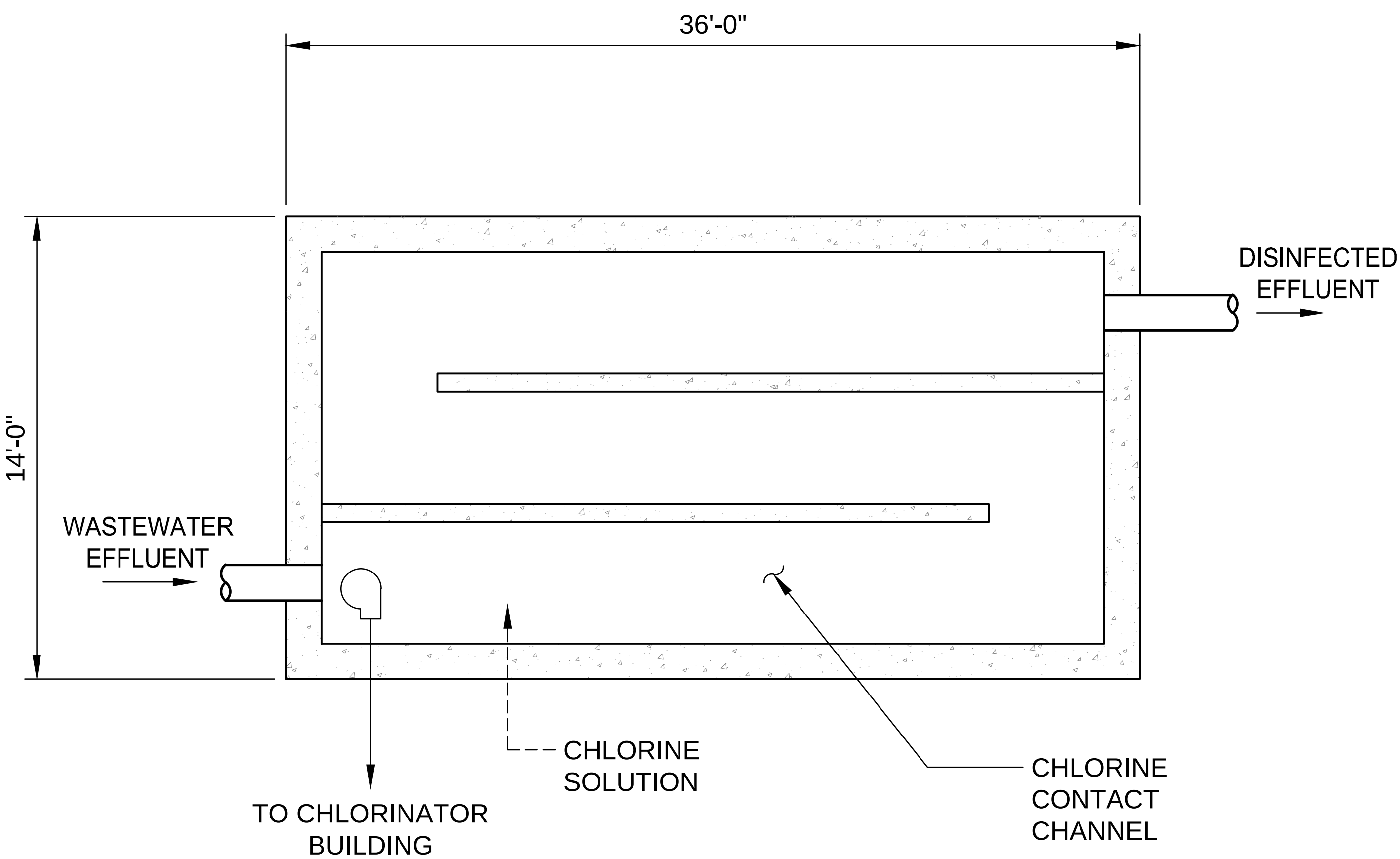
Table 4-4. Chlorine Demand

Description	Flow	Chlorine Demand
Average dry weather flow	0.225 mgd	8 - 15 lbs./day
Peak day wet weather flow	0.690 mgd	20 - 38 lbs./day
Peak hour wet weather flow	500 gpm	25 – 47 lbs./day

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). The size of the chlorine contact tank will need to accommodate a 15-minute contact time for the peak discharge rate. Disinfection is usually sized for the design peak day wet weather flow. However, for this application, the peak discharge rate will need to be equal to the peak distribution rate into the native tree grove slow rate disposal system. To be conservative, one and a half times the design peak hour flow will be used instead of the peak day wet weather flow. Table 4-5 summarizes the contact tank dimensions, while Figure 4-10 shows a conceptual contact tank configuration.

Table 4-5. Chlorine Contact Tank

Description	Value
Peak discharge rate	750 gpm
Minimum chlorine contact tank	15 minutes
Tank volume required	1,500 cubic feet
Channel water depth	5 feet
Channel width	3 feet
Tank channel total length	100 feet
Tank dimensions including channel walls	14 feet x 36 feet



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JOB NUMBER: 151494

NAALEHU WASTEWATER TREATMENT PLANT

CHLORINE CONTACT TANK CONFIGURATION

FIGURE 4-10

4.4.2 Ultraviolet Light (UV) 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.

4.4.3 UV System Design Summary

A UV disinfection system requires about the same size footprint as chlorine. 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 US, 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 UV300+ 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 4-6 summarizes the size and design criteria for the UV system required to treat the WWTP discharge.

Table 4-6. UV Disinfection Design Summary

Description	Value
Peak Hour Wet Weather Discharge	750 gpm
Minimum UV transmittance	65 percent
No. of UV channels	1
Design dose	35,000 μWs/cm2
Disinfection limit	30 e-coli per 100mL
Validation factors	0.98 end of lamp factor

4.4.4 Cost Evaluation

A summary of capital and life-cycle estimated costs for both chlorination and UV disinfection is presented in Table 4-7 for comparison.

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 78 percent of the UV option.

Table 4-7. Estimated Disinfection Costs

Description	Chlorination	UV System
Capital Cost	$240,000	$950,000
Annual Operations and Maintenance*	$18,000	$7,500
Life-cycle Cost (30-Year Net Present Value)	$700,000	$1,300,000

**Does not include annual labor.*

4.4.4.1 Non-Economic Evaluation

Table 4-8 presents a summary of advantages and disadvantages of using an ultraviolet light for disinfection.

Table 4-8. Ultraviolet Disinfection – Advantages and Disadvantages

Advantages	Disadvantages
Effective at inactivating most viruses, spores, and cysts	Low dosage may not be effective on some pathogens and some organisms can repair and reverse the destructive effects of UV
It's a physical process, instead of chemical – it eliminated the need to transport, handle, store toxic or corrosive chemicals	Turbidity and TSS in the wastewater can reduce UV disinfection effectiveness
No harmful residual compounds created that are toxic to humans or aquatic life	Will likely require more call-outs by operators due to alarms caused by "dirty power".
Shorter contact time (less than a minute)	The relative intensity of equipment maintenance requirements, including staffing training and on-island availability.

4.4.5 Disinfection Recommendation

A tablet chlorination system is the recommended disinfection option over the UV system for the WWTP because it incurs lower capital and lifecycle costs. In addition, tablet chlorination will be more-reliable than UV due to frequent "dirty power" conditions experience on the island.

4.5 Effluent Management

For effluent management, a slow-rate land application system is proposed. The concept is to intermittently apply wastewater to crops growing in permeable soils. As the applied water percolates through the soil matrix or is taken up by the crop, it is treated by physical filtration and by biological mechanisms. After an application period or wetting period, the surface dries and oxygen can enter the soil matrix, which aids aerobic biological treatment. The frequent wetting and drying also maintains the infiltration rate through the soil surface and minimizes soil clogging. This method of land application is an effective treatment process for BOD_5, TSS, trace organics, phosphorus, metals and pathogen removal. Furthermore, removal of nitrogen can be significant when the system is designed and managed for that objective.

4.5.1 Design

The proposed slow-rate system site consists of a net area of approximately 8.5 acres. The 8.5 acres will be divided into 4 groves of native trees, so that water application will be rotated to a different grove each day. By using one groove per day the wet/dry cycle will be 1-day wetting and 3-days drying. The system will be designed to allow a grove to be temporarily removed from service for maintenance purposes.

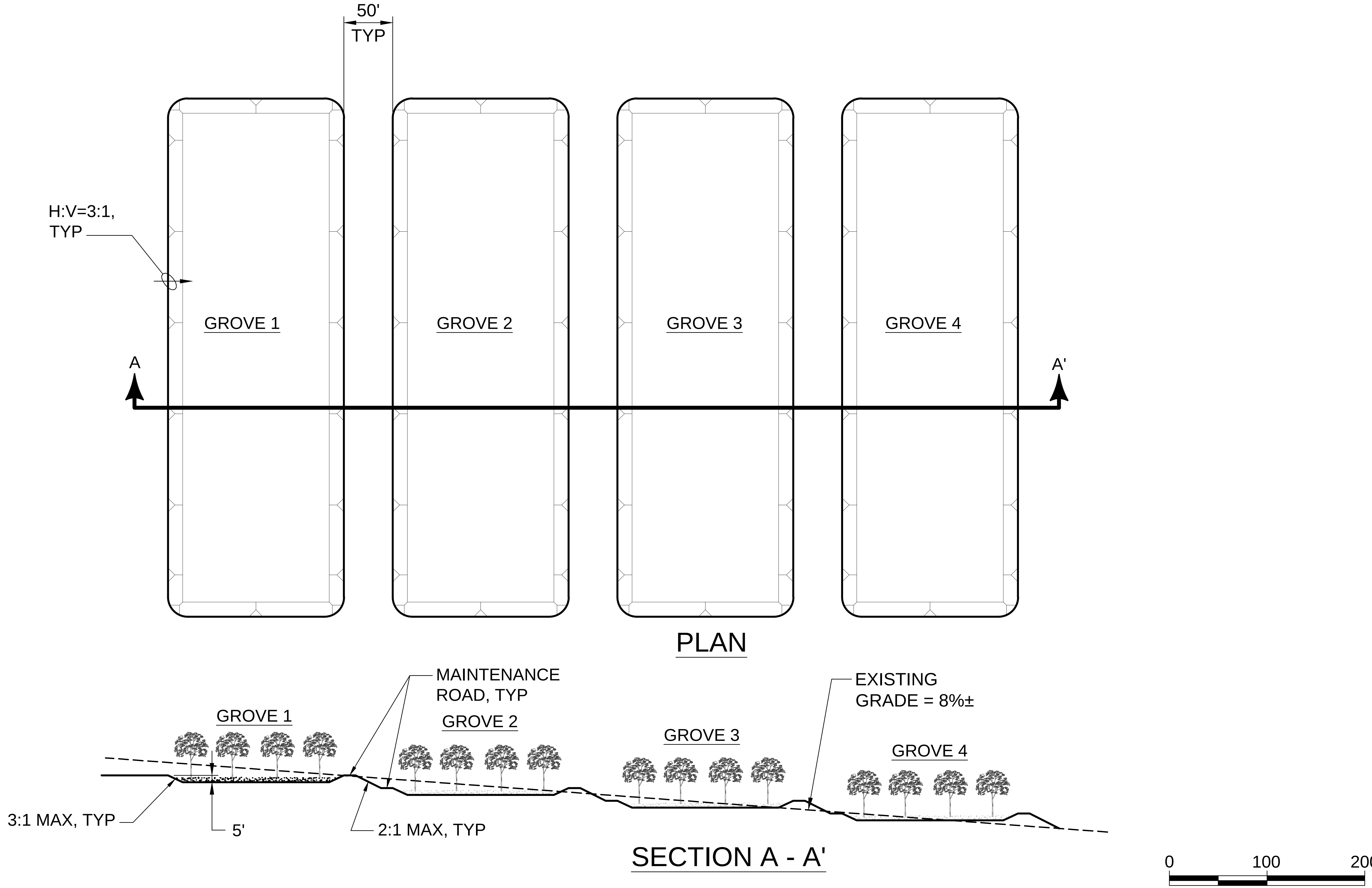
The groves will be planted with native Hawaiian trees. Trees grown within the land application area will need to be water tolerant. Table 4-9 lists a few potential native tree species. Local experts will be consulted to develop the final list of species.

Table 4-9. Potential Land Application System Tree Species

Common Name	Genus Species	Salt Tolerance	Water Requirements	Rubbish and Maintenance	Preferred Elevation
Milo	Thespesia populnea	Very	Dry to Wet	Moderate	Low to Medium
Loulu	Pritchardia hillebrandii	Very	Dry to Wet	Low	Low
Aalii	Dodonaea viscosa	Very	Dry to Medium	Low	Low to High
Kou	Cordia subcordata	Very	Dry to Wet	Moderate	Low
Golden Loulu	Pritchardia arecina	Moderate	Dry to Wet	Low	Low to Medium
Wiliwili	Erythrina sandwicensis	Moderate	Dry to Medium	Moderate	Low

The distribution system will consist of gated pipe located on the surface. A photo of gated pipe in operation is shown in Figure 4-11. The gated pipe has small adjustable slots to allow the applied wastewater to uniformly be distributed over the grove surface. A perimeter fence will be installed to limit access. Access roads will surround each grove. Figure 4-12 reflects the proposed land application schematic.

Figure 4-11. Gated Pipe in Use



Path: P:\Projects\Hawaii, County Of (HI)\151494 COH Naalehu WWTP PER_CAD\2-FIGURES\PER Figures File Name: 151494-FIG-LandAppSystem Plot Date: October 9, 2018 2:14 PM Cadd User: Richard Sellona

Brown AND Caldwell	SCALE: NONE JOB NO: 151494	NAALEHU WASTEWATER TREATMENT PLANT LAND APPLICATION SYSTEM SCHEMATIC	FIGURE 4-12

4.6 Ancillary Systems

4.6.1 Water

Potable water is not currently available at the site. The nearest potable water system is located uphill in town. Table 4-10 provides an initial assessment of the potential water demands at the WWTP. The water demands are either for process or potable uses. As shown in the table, the process water demands are significantly greater than the potable demands.

Table 4-10. Potential Water Demands

Description	Flow Rate	Type	Priority
Screenings washer	20 gpm for 10 min/hour 4,800 gpd	Process	Mandatory with screen
Hose bibs	10 gpm for 20 min/day 200 gpd	Process	Desirable to maintain facility
Emergency eye wash / shower	20 gal per use	Potable	Mandatory
Restroom	20 gpd	Potable	Recommended

To supply water to the WWTP, pipe will be installed from the nearest location in town to supply a 1-inch water meter with 1 ½-inch backflow preventer.

The on-site water system will be split into two branches, one for process water and one for potable water. The potable water will service the restroom and emergency eye wash/shower. A second backflow preventer will separate the process water uses from the potable uses.

4.6.2 Access Road

All weather access will be provided to the WWTP. Access to the site will be provided by connection to Spur Road. A paved extension to Spur Road is proposed as shown in Figure 5-2. The road will cross the new drainage channel via a culvert and all-weather maintenance roads will extend into the site to provide access to and around the various WWTP infrastructure. Additionally, a turn-around area large enough to accommodate a fire truck will be provided.

Access road pavement options include aggregate base (AB) gravel, asphalt concrete (AC), or concrete. AB is the lowest cost option but requires the most maintenance. AC pavement is not recommended for steep (greater than 12 percent) grades. Concrete is the highest cost option but is the most durable and requires the least maintenance.

The recommended driveway pavement section is 2-inches of AC over 6-inches of aggregate base course. If any portions of the driveway exceed 12 percent slope, a concrete pavement section is recommended.

4.6.3 Stormwater Management

The overall goal of stormwater management is to mitigate the adverse impact of new construction on the environment. Stormwater management can generally be separated into two areas:

1. Stormwater Quantity: management of the quantity of stormwater runoff to prevent increased flows and volumes from leaving the site and adversely impacting downstream watercourses.
2. Stormwater Quality: management of the quality of stormwater runoff to prevent contaminants such as silt, trash, hydrocarbons, heavy metals, and pesticides from leaving the site through stormwater runoff.

Per the Hawaii County Code, Chapter 27, Section 20, the site drainage plan shall accommodate the additional runoff caused by the proposed development, within the site boundaries. A preliminary evaluation of the pre and post development stormwater conditions is described in the following sections, but a complete analysis will be completed during the design phase to ensure that requirements of the county code are met or exceeded and that no adverse impact to downstream or adjacent properties occurs.

4.6.4 Pre-development Stormwater Conditions

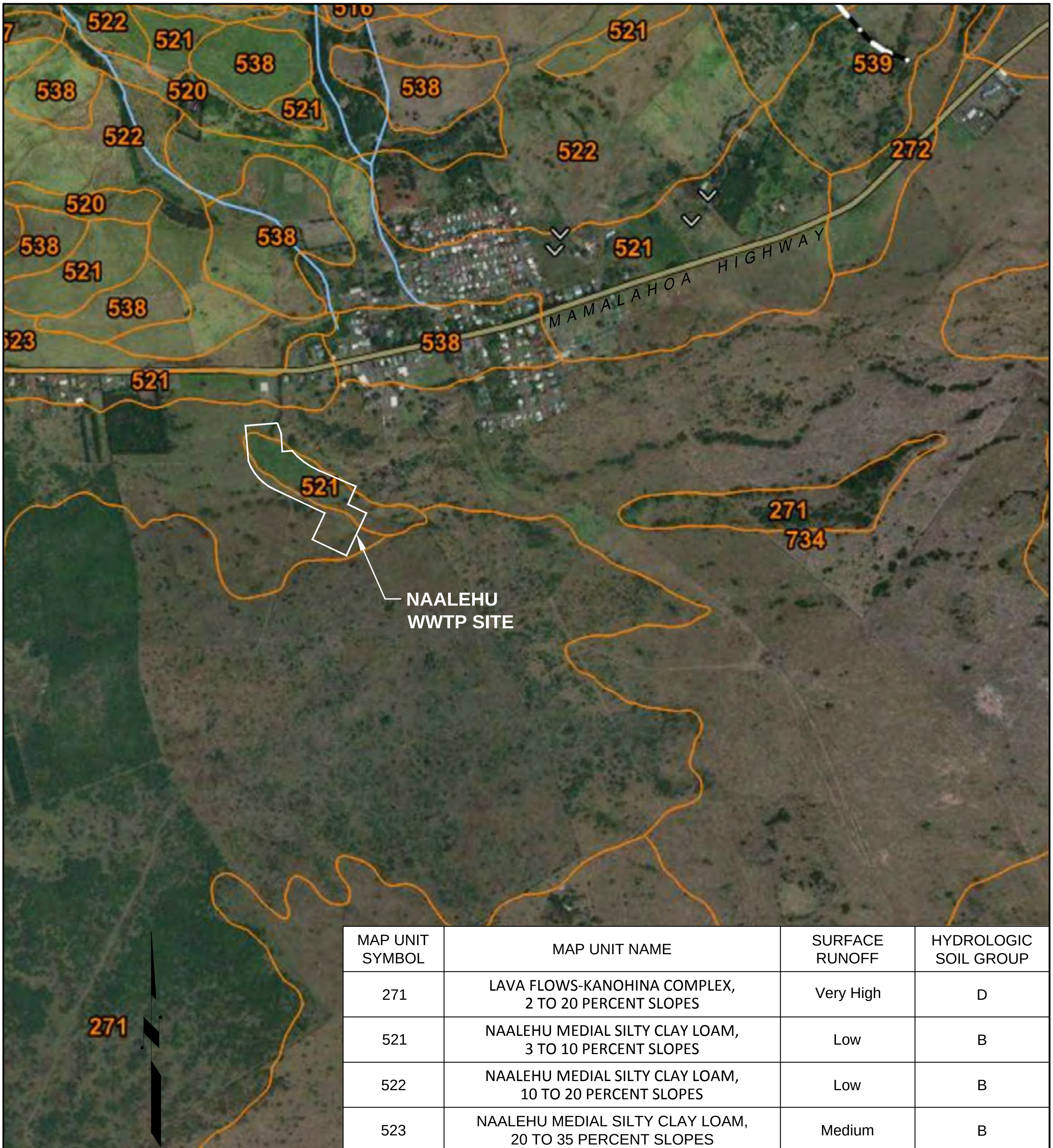
The site stormwater can be divided into two categories: 1) on-site flows that are generated at or in close proximity to the WWTP site and will be directly affected by the plant construction and 2) off-site flows that are generated at higher elevations and are captured and conveyed by an existing vegetated diversion channel, which currently outlets at the proposed WWTP site. Though flows from both sources have the same ultimate discharge point to the southeast of the site, the distinction is made because a proposed diversion channel will be constructed that will extend and relocate the outlet of the existing channel. The new outlet will re-join the natural drainage patterns while allowing for construction of the WWTP.

On-Site

The watershed area that contributes to on-site stormwater flows is located makai of Mamalahoa Highway. The proposed WWTP site will occupy approximately 25 acres on what is primarily undeveloped agricultural land, consisting of mostly barren ranch land with trees and brush or cane fields. The total watershed area contributing to on-site runoff is approximately 123 acres. The areas contributing to on-site flows are either currently undeveloped or are utilized for agricultural purposes, and the parcels comprising this area each have a land use zoning classification of either agricultural or low density residential.

The existing elevations range between 680 to 820 feet above mean sea level (MSL) and slope in a southeasterly direction at a mild slope of 5 percent or less. The soils in this area are described as primarily Lava Flows-Kanohina Complex (271), Kanohina-Lava Flows Complex (734) or Medial Silty Clay Loam (521) by the Natural Resources Conservation Service (NRCS) Soil Survey (Figure 4-13). These soils are considered to have high surface runoff, however flows reaching the lava area fan out and are known to percolate through tubes within the lava rock.

The majority of runoff within the on-site watershed areas are overland sheet flow or shallow concentrated flows that move in a southeasterly direction towards the ocean.



MAP UNIT SYMBOL	MAP UNIT NAME	SURFACE RUNOFF	HYDROLOGIC SOIL GROUP
271	LAVA FLOWS-KANOHINA COMPLEX, 2 TO 20 PERCENT SLOPES	Very High	D
521	NAALEHU MEDIAL SILTY CLAY LOAM, 3 TO 10 PERCENT SLOPES	Low	B
522	NAALEHU MEDIAL SILTY CLAY LOAM, 10 TO 20 PERCENT SLOPES	Low	B
523	NAALEHU MEDIAL SILTY CLAY LOAM, 20 TO 35 PERCENT SLOPES	Medium	B
538	NAALEHU MEDIAL SILT LOAM, 0 TO 3 PERCENT SLOPES	Very Low	B
734	KANOHINA-LAVA FLOWS COMPLEX, 2 TO 10 PERCENT SLOPES	Very High	C or D

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SCALE: 1"=1500'

JOB NUMBER: 151494

DATE: October 23, 2018

NAALEHU WASTEWATER TREATMENT PLANT

NRCS SOIL MAP

FIGURE

4-13

Plot Date: October 23, 2018 9:39 AM

151494-FIG-4-12-NRCS Soil Map

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Off-Site

The off-site watershed area is located mauka of Mamalahoa Highway and has an agricultural land use zoning classification. The total watershed area contributing to off-site runoff is approximately 780 acres. This area is well-vegetated, with varying topographical characteristics including a range of both steep and mild slopes.

Currently, the runoff from the off-site area is collected and conveyed through an existing vegetated diversion channel. The existing channel starts just below the Naalehu cemetery and collects and conveys off-site runoff to the south of Mamalahoa Highway, where it discharges flow at an outlet within the on-site watershed area. In the predevelopment condition, flow from the outlet fans and dissipates into shallow concentrated flow that runs through the ranch lava-rock land in an easterly direction. Figure 4-14 conceptualizes the existing drainage system.

4.6.4.1 Flood and Tsunami Hazards

The subject property is designated Zone X, area of minimal flood hazard corresponding to areas outside of the five-hundred-year flood plain, as indicated on the current September 29, 2017 Flood Insurance Rate Map (FIRM), Community Panel No. 155166 1925 F. Zone X designations are not subject to the requirements of the Standards of Floodways, Chapter 27, Section 22 of the Hawaii County Code. See Figure 4-15 for the Flood Insurance Rate Map.

4.6.4.2 Stormwater Quantity

The on-site and off-site peak stormwater discharges were respectively approximated using the methods outlined in the current County of Hawaii, Department of Public Works (DPW) Storm Drainage Standards (Department of Public Works, 1970).

On-Site

The total watershed area contributing to on-site runoff is approximately 123 acres and the estimated pre-development 50-year 1-hour peak stormwater runoff is 157 cubic feet per second (cfs).

Off-Site

The total watershed area contributing to off-site runoff is approximately 780 acres. The total estimated pre-development 100-year 1-hour peak stormwater runoff is 3,000 cfs.

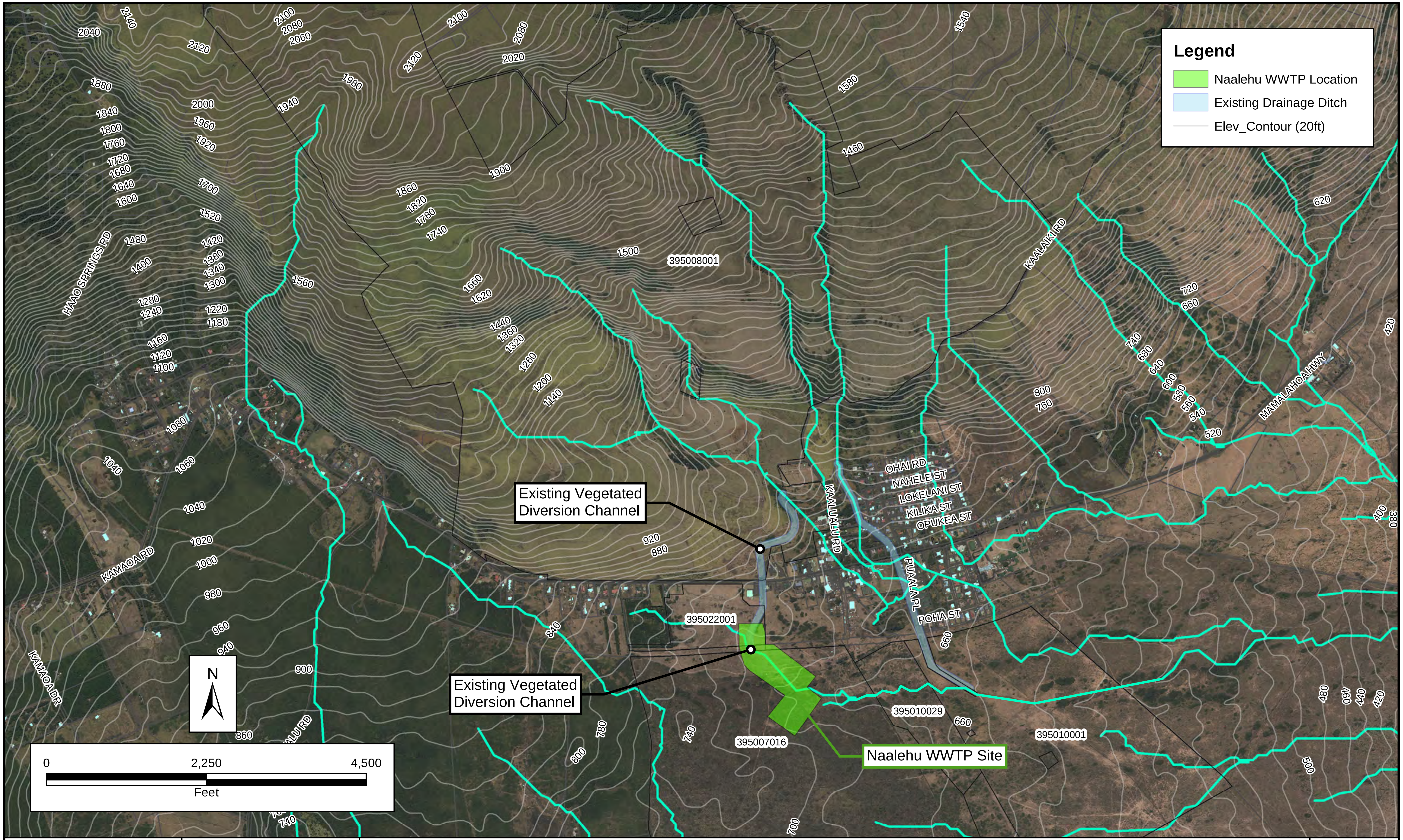
4.6.5 Post-Development Conditions

The overall on-site post-development drainage patterns are anticipated to be consistent with the pre-development condition. The WWTP site is anticipated to bisect runoff into swales that will run around the site and follow a similar drainage direction to that of the surrounding area. The site improvements for the WWTP will include grading for the facility, including but not limited to buildings, lagoon basins, roadways, and parking.

In addition, improvements will include relocation of the existing vegetated diversion channel outlet makai of the WWTP. The channel alignment will be extended and run along the perimeter of the WWTP, to divert off-site flows around the WWTP. Figure 5-1 displays the conceptual site plan.

4.6.5.1 Stormwater Quantity

Post-development stormwater is evaluated in terms of on-site and off-site flows.



SCALE AS SHOWN

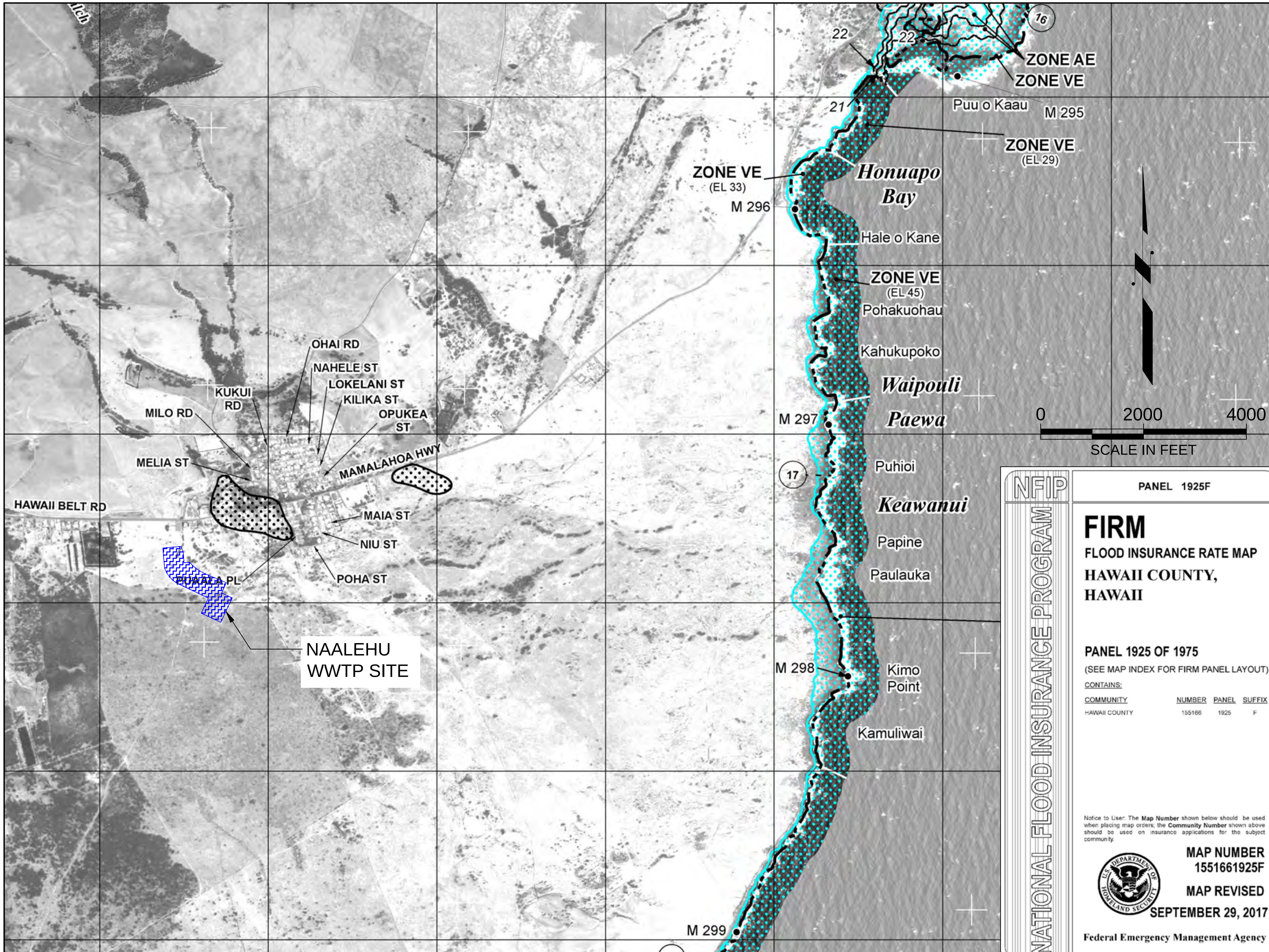
JOB NO.: 151494

NAALEHU WASTEWATER TREATMENT PLANT

NAALEHU EXISTING WATERCOURSES

FIGURE

4-14



LEGEND

SPECIAL FLOOD HAZARD AREAS SUBJECT TO INUNDATION BY THE 1% ANNUAL CHANCE FLOOD

The 1% annual chance flood (100-year flood), also known as the base flood, is the flood that has a 1% chance of being equaled or exceeded in any given year. The Special Flood Hazard Area is the area subject to flooding by the 1% annual chance flood. Areas of Special Flood Hazard include Zones A, AE, AH, AO, AR, A99, V, and VE. The Base Flood Elevation is the water-surface elevation of the 1% annual chance flood.

ZONE A	No Base Flood Elevations determined.
ZONE AE	Base Flood Elevations determined.
ZONE AH	Flood depths of 1 to 3 feet (usually areas of ponding); Base Flood Elevations determined.
ZONE AO	Flood depths of 1 to 3 feet (usually sheet flow on sloping terrain); average depths determined. For areas of alluvial fan flooding, velocities also determined.
ZONE AR	Special Flood Hazard Area formerly protected from the 1% annual chance flood by a flood control system that was subsequently decertified. Zone AR indicates that the former flood control system is being restored to provide protection from the 1% annual chance or greater flood.
ZONE A99	Areas to be protected from 1% annual chance flood event by a Federal flood protection system under construction; no Base Flood Elevations determined.
ZONE V	Coastal flood zone with velocity hazard (wave action); no Base Flood Elevations determined.
ZONE VE	Coastal flood zone with velocity hazard (wave action); Base Flood Elevations determined.

FLOODWAY AREAS IN ZONE AE

The floodway is the channel of a stream plus any adjacent floodplain areas that must be kept free of encroachment so that the 1% annual chance flood can be carried without substantial increases in flood heights.

OTHER FLOOD AREAS

ZONE X	Areas of 0.2% annual chance flood; areas of 1% annual chance flood with average depths of less than 1 foot or with drainage areas less than 1 square mile; and areas protected by levees from 1% annual chance flood.

OTHER AREAS

ZONE X	Areas determined to be outside the 0.2% annual chance floodplain.
ZONE D	Areas in which flood hazards are undetermined, but possible.

COASTAL BARRIER RESOURCES SYSTEM (CBRS) AREAS

OTHERWISE PROTECTED AREAS (OPAs)

CBRS areas and OPAs are normally located within or adjacent to Special Flood Hazard Areas.

1% annual chance floodplain boundary
0.2% annual chance floodplain boundary
Floodway boundary
Zone D boundary
CBRS and OPA boundary
Boundary dividing Special Flood Hazard Area Zones and boundary dividing Special Flood Hazard Areas of different Base Flood Elevations, flood depths, or flood velocities
513 — Base Flood Elevation line and value; elevation in feet*
(EL 987) — Base Flood Elevation value where uniform within zone; elevation in feet*
* Referenced to the LOCAL TIDAL DATUM
A — A Cross section line
23 — 23 Transect line
97°07'30", 32°22'30" Geographic coordinates referenced to the North American Datum of 1983 (NAD 83), Western Hemisphere
$^{42}75^{000m}$E 1000-meter Universal Transverse Mercator grid values, zone 5
6000000 FT 5000-foot grid ticks: Hawaii State Plane coordinate system, Zone 1 (FIPSZONE = 5101), Transverse Mercator projection
DX5510 Bench mark (see explanation in Notes to Users section of this FIRM panel)
• M1.5 River Mile

THIS IS AN OFFICIAL COPY OF A PORTION OF THE ABOVE REFERENCED FLOOD MAP. IT WAS EXTRACTED USING F-MIT ON-LINE. THIS MAP DOES NO REFLECT CHANGES OR AMENDMENTS WHICH MAY HAVE BEEN MADE SUBSEQUENT TO THE DATE ON THE TITLE BLOCK. FOR THE LATEST PRODUCT INFORMATION ABOUT NATIONAL FLOOD INSURANCE PROGRAM FLOOD MAPS CHECK THE FEMA FLOOD MAP STORE AT WWW.MSC.FEMA.GOV.

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SCALE: 1"=2000'

JOB NO: 151494

NAALEHU WASTEWATER TREATMENT PLANT

FLOOD INSURANCE RATE MAP

FIGURE 4-15

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File Name: 151494-FIG-4-14-FIRM Plot Date: October 23, 2018 9:41 AM Cadd User: Irina Constantinescu

On-Site

The total watershed area contributing to on-site runoff remains the same as the pre-development condition at approximately 123 acres. However, because the increase in peak flow is a function of the increase in impervious area associated with improvements, the estimated post-development 50-year 1-hour peak stormwater runoff is 198 cfs. The WWTP site is anticipated to increase runoff by roughly 41 cfs.

To ensure that there is no adverse impact on adjacent or downstream properties due to post-development flows, an on-site drainage system will collect runoff via grated inlets or swales. These flows will be conveyed to on-site drainage detention systems, such as subsurface linear infiltration or depressed detention basins, to detain flows and volumes to their pre-development condition. Additionally, all exposed (not enclosed) treatment processes will be sized to include free-board depth to accommodate the 24-hour, 100-year storm event. Thus, no stormwater runoff from these areas is anticipated.

Off-Site

The estimated stormwater runoff flow rate will be the same as the pre-development runoff, which is approximately 3,000 cfs.

The proposed watercourse diversion will circumvent the WWTP site and will rejoin the natural flow path to the east, without any additional on-site flow contribution. In addition to not increasing flows above the pre-development condition, the channel will be designed to comply with the COH Drainage Standards such that flow conditions at the proposed outlet do not exceed 5 feet per second and to ensure no negative impact to downstream or adjacent properties (Department of Public Works, 1970).

4.6.5.2 Stormwater Quality

The quality of stormwater leaving the site is also a concern. Stormwater quality degrades with development and increased impervious surfaces, because various pollutants are introduced into the stormwater runoff.

The first half-inch of runoff during a storm is referred to as the Water Quality Volume (WQV) or the “first-flush” volume. This portion of the runoff from a storm contains measurably more suspended solids plus other contaminants per cubic foot than would be expected in runoff occurring later in the storm.

Because the anticipated total disturbed area for this project is greater than 1 acre, this project will trigger compliance with the National Pollutant Discharge Elimination System (NPDES) construction stormwater permit from the Department of Health (DOH) Clean Water Branch (CWB). To mitigate the quality of runoff, several best management practices (BMPs) will be considered to satisfy requirements of the NPDES permit and to ensure that construction activities do not adversely affect downstream waterways during site development and construction of the diversion channel. BMPs that will be considered for use during construction include thoughtful project scheduling, flow routing, and the use of perimeter controls and sediment traps.

Additionally, permanent BMPs will be employed, which include scheduled good-housekeeping, which will reduce litter and other constituents from being washed into the storm drain system, and detention basins and underground infiltration facilities that prevent the release of sediment and other pollutants to downstream waterways or adjacent properties. A full assessment of all available BMP’s to optimize water quality will be provided during design of the project.

4.6.6 Electrical Systems

It will be necessary to bring electrical power to the WWTP site. It is anticipated that Hawaii Electric Light Company (HELCO) will bring overhead power lines to the site and supply 480-volt, 3 phase power to the WWTP via a pole-mounted transformer to a service panel with a meter.

The floating surface aerators will consume the majority of the electricity supplied to the site. An electrical room will house the electrical gear, plant control equipment and the chlorination system. Exterior lighting at the site will be limited to manually switched lights at the entrance to the electrical building and at the headworks area.

A standby power system will be provided in the form of a pad-mounted diesel generator and above-ground fuel tank with capacity to support three consecutive days of operation. In addition, the electrical service panel will be equipped with a manual transfer switch and generator receptacle to allow connection of a trailer-mounted generator in the event of emergency generator failure during an extended power outage.

4.6.7 Telemetry Systems

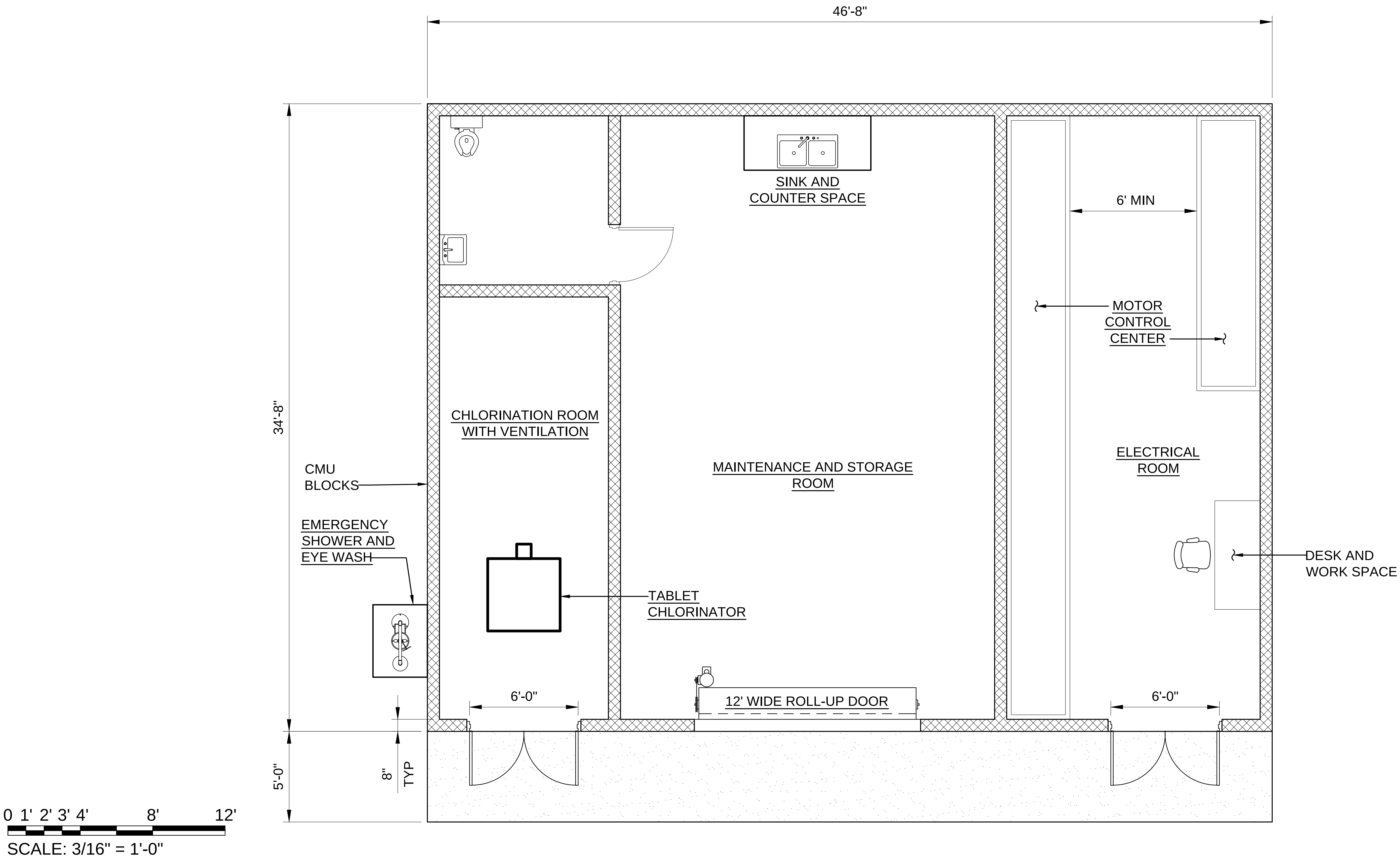
A land-line telephone telemetry system with auto-dialer will provide Hilo-based operation staff of alarm conditions and key operational parameters at the WWTP.

4.6.8 Operations Building

An operations building will be constructed to include the electrical room, chlorinator room, restroom, and maintenance/storage room, as shown in Figure 4-16.

4.6.9 Site Fencing

The entire WWTP site, including the treatment systems and the land application system, will be fenced (6-foot high chain link) and posted to prevent unauthorized public access.



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SCALE: 3/16" = 1'-0"

JOB NO: 151494

NAALEHU WASTEWATER TREATMENT PLANT

OPERATIONS BUILDING PRELIMINARY FLOOR PLAN

FIGURE 4-16

Section 5

Preliminary Design of Improvements

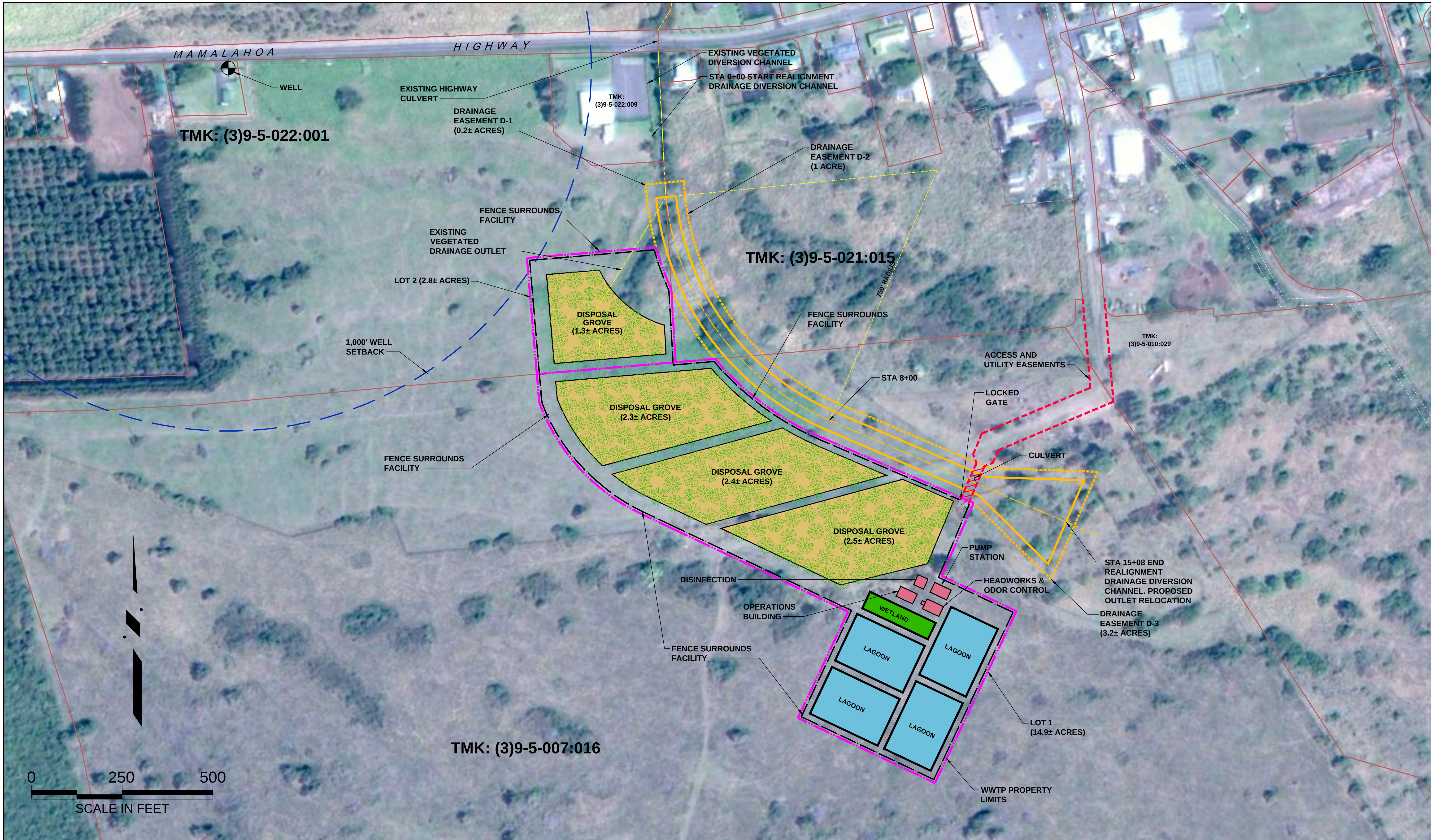
The following is a summary of the preliminary design for the proposed Naalehu WWTP.

5.1 Site Plan

The existing parcels are ranch lands. The prevailing grade is in the north-west to south-east direction at 2 to 3 percent slope. Approximately 17.7 acres of land will be cleared for the construction of the proposed facility. Figure 5-1 presents a preliminary site plan for the WWTP.

5.2 Process Schematic

Figure 5-2 presents the recommended facilities process schematic.



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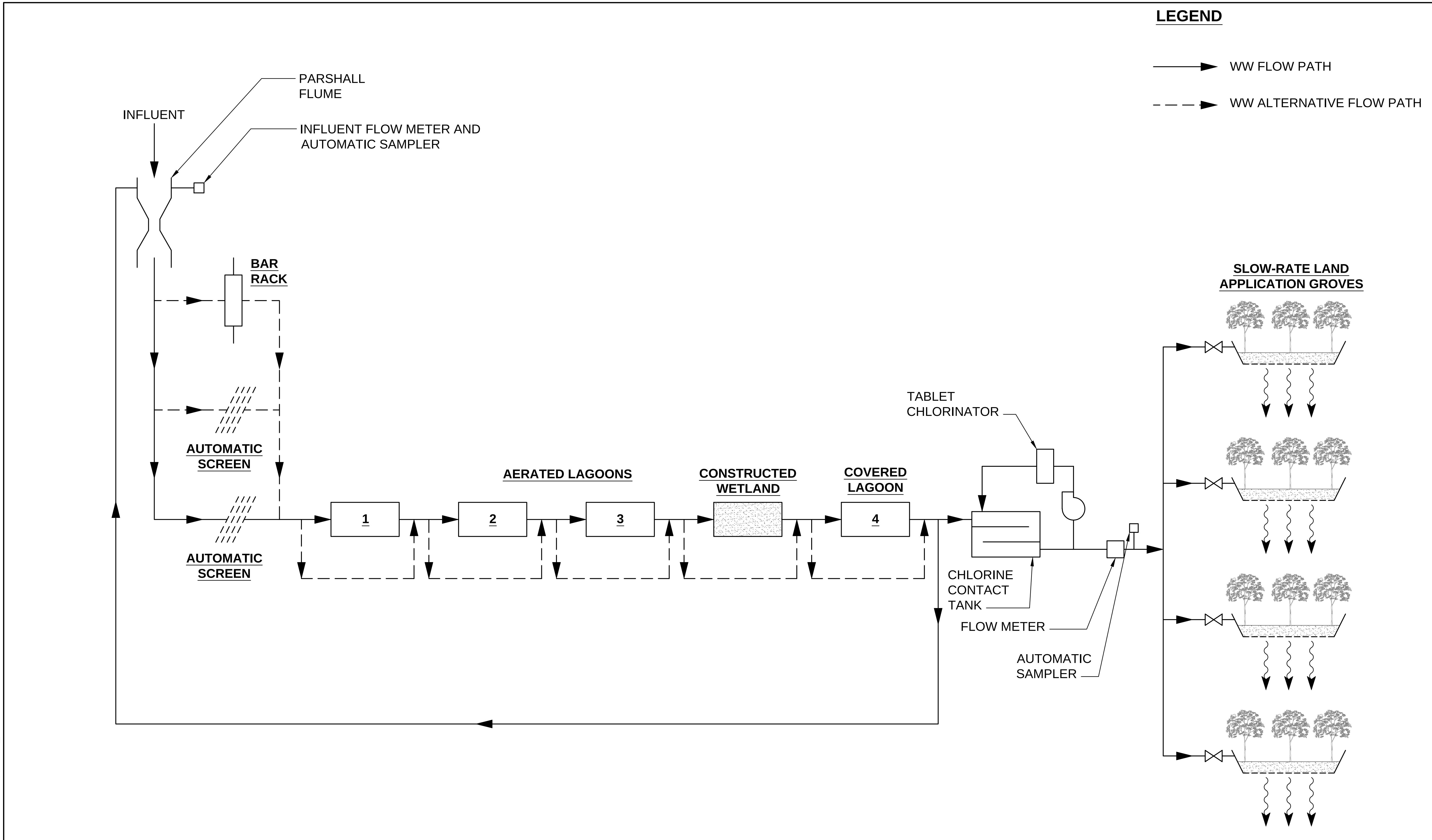
File Name: 151494-FIG-5-1-WWTP-PreliminarySitePlan Plot Date: October 22, 2018 4:06 PM

Brown AND Caldwell	SCALE: 1" = 200' 151494 DATE: October 22, 2018	NAALEHU WASTEWATER TREATMENT PLANT PRELIMINARY SITE PLAN	FIGURE 5-1



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NAALEHU WASTEWATER TREATMENT PLANT

RECOMMENDED FACILITIES PROCESS SCHEMATIC

FIGURE 5-2

5.3 Design Criteria

Table 5-1 provides preliminary design criteria for the facility.

Table 5-1. Preliminary Design Criteria	
Description	**Value**
Influent flows:	
• Average dry weather	225,000 gpd
• Peak day wet weather	690,000 gpd
• Peak hour wet weather	500 gpm
Influent characteristics	
• BOD_5	300 mg/L
• TSS	300 mg/L
Odor control – granular activated carbon	
• Airflow rate	500 cfm
• H_2S Inlet concentration	1-10 ppm
• H_2S removal efficiency	99%
• Media type	High-capacity carbon
• Vessel diameter	3 feet
• Vessel height	7 feet
• Minimum carbon quantity	680 lbs
• Minimum bed depth	3.5 feet
• Fan motor	2 hp
• Nominal inlet size	8 inches
Mechanical screens	
• Number of units	2
• Type	In-channel cylindrical
• Screen opening size	0.25 inch (6 mm)
• Maximum flow rate capacity	Greater than 750 gpm each
• Screening washing	Integral
• Screening compaction	Integral
• Screening wash water flow	20 gpm
• Screening wash water pressure	50 psi
Bypass screen	
• Type	Manually-cleaned bar rack
• Bar spacing	1 inch
• Rake	Interlocking with bars
Screenings receptacle	

Table 5-1. Preliminary Design Criteria continued	
• Type	55-gallon drum or bags
• Screenings volume per million gallons treated	5 ft^3/Mgal
• Estimated screenings quantity	1 ft^3/day
• Disposal frequency	1/week
Influent flow metering	
• Type	Parshall flume
• Maximum flow capacity	Greater than 1,250 gpm
• Minimum straight upstream channel section	20 times the throat width
Influent flow sampling	Refrigerated automatic composite sampler
Lagoon cells	
• Number of cells	4
• Maximum lagoon temperature	25°C
• Minimum lagoon temperature	20°C
• Freeboard	3 feet
• Working water depth	13 feet
• Allowance for sludge	3 feet
• Total lagoon depth	16 feet
• Side slope	3(H) : 1(V)
• Working volume of lagoon 1 to 3	1.45 Mgal
• Working volume of lagoon 4	1.45 Mgal
Aerators	
• Type	Floating mechanical surface aerators
• Cell 1 aerators	40 hp (2 at 20 hp)
• Cell 2 aerator	15 hp
• Cell 3 aerator	10 hp
• Cell 4 aerator	5 hp aspirator style, floating ball cover for algae control
Constructed Wetland	
• Water temperature	25 degrees C
• Aerated lagoon effluent nitrate-N concentration	19 mg/l
• Aerated lagoon effluent ammonia-N concentration	1 mg/l
• Constructed wetland effluent total N concentration	15.3 mg/l
• Total constructed wetland surface area	0.3 acres
• Flow path length	60 feet
• Hydraulic application width	220 feet
• Media depth	24 inches
• Media type	Medium gravel, D_{10} = ¾ inch

Table 5-1. Preliminary Design Criteria continued	
• Media porosity	38 percent
• Percolation prevention system	60 mil high density polyethylene (HDPE) liner
• Vegetation	Native Hawaiian reeds and/or rushes, species to be determined
Disinfection system	
• Type	Chlorine
• Form	Calcium hypochlorite tablets
• Design chlorine dose	4-8 mg/L
• Chlorine contact time	15 minutes minimum
Effluent flow metering	
• Type	Magnetic
Effluent sampler	
• Type	Refrigerated automatic composite
Effluent quality	
• BOD_5	Less than 30 mg/L monthly average Less than 60 mg/L peak
• TSS	Less than 30 mg/L monthly average Less than 60 mg/L peak
Effluent management system	
• Type	Slow-rate land application groves
• Number	4
• Minimum depth	5 feet
• Design percolation rate	0.0095 inches per minute
• Design application rate	8 percent of percolation rate
• Distribution system	Gated pipe
• Stormwater containment	100-year, 24-hour storm event
• Vegetation	Native Hawaiian trees
Stormwater site management	10-year, 1-hour storm

5.4 Environmental Benefits

A well-designed and managed land treatment system limits wastewater application to rates that minimize adverse impact to groundwater quality. The percolate from the SR land treatment system is expected to contain less than 1 mg/L of BOD_5 and TSS. While the State of Hawaii has not adopted formal groundwater quality standards, the drinking water standard for nitrate (10 mg/L as N) in the annual average percolate was used as a performance target to design the land treatment site. Phosphorus adsorption is excellent in SR land treatment systems, and 99 percent or greater phosphorus removal is anticipated. Table 5-2 compares the current loads to the environment via the LCCs and the loads to the environment after the proposed project is implemented via the percolate from the land treatment system. Figure 5-3 provides a graphical representation of the environmental benefits of the proposed project compared to the status quo.

Table 5-2. Environmental Benefits of Proposed Project

Parameter	Current Annual Load to Environment via LCCs	Annual Load to Environment via Proposed Land Treatment System Deep Percolate	Reduction
BOD_5	206,000 lbs./year	750 lbs./year	>99%
TSS	206,000 lbs./year	750 lbs./year	>99%
Nitrogen	27,000 lbs./year	4,900 lbs./year	83%
Phosphorus	4,700 lbs./year	48 lbs./year	>99%

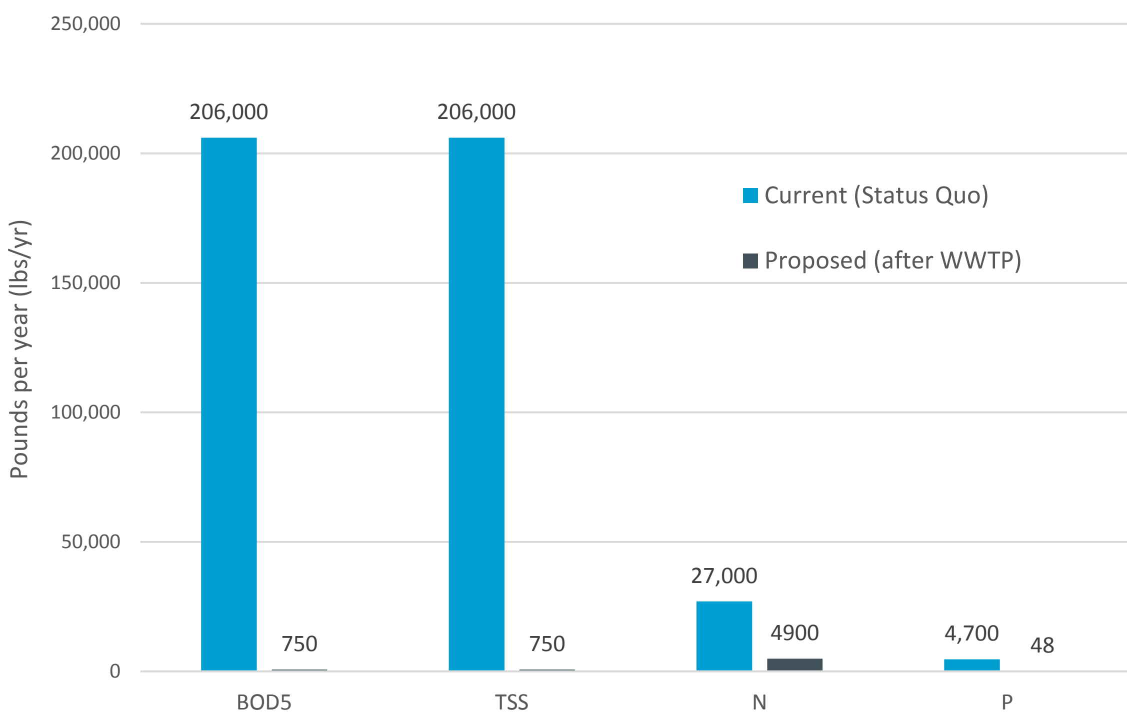


Figure 5-3. Environmental Benefits of Proposed Project

5.5 Cost Estimates

An order of magnitude probable capital cost is summarized in Table 5-3. The estimate includes a 20 percent estimating contingency. The detailed cost estimate is included in Appendix B.

Table 5-3. Naalehu WWTP Order of Magnitude Capital Cost Estimate

Description	Estimated Construction Cost
Wastewater treatment plant and utilities	$14,600,000
Land application system	$6,400,000
Drainage improvements	$11,400,000
Total construction cost	$32,400,000
Engineering, administration, and legal at 25% of construction cost	$8,100,000
Total capital cost	$40,500,000

5.6 Future Expansion

5.6.1 Full Buildout Flows

Full buildout wastewater flow projections were developed using the Draft Ka'u Community Development Plan (March 2015) and the CCH's current (2017) wastewater standards. Table 5-4 summarizes the projected full buildout flows for the community, and Figure 2-2 shows the WWTP full buildout service area.

Table 5-4. Naalehu WWTP Full Buildout Flow Projections

Description	Value	Peaking Factor
Average dry weather flow	390,000 gallons per day	1.0
Peak day wet weather flow	1,200,000 gallons per day	2.5*
Peak hour wet weather flow	1,250 gallons per minute	4.6

**Derived from Crites and Tchobanoglous, 1998*

5.6.2 Improvements

To accommodate treatment of the increased flow anticipated from the full buildout of the Naalehu wastewater collection system, the WWTP will require facility upgrades. The recommended upgrades include headworks and odor control expansion within the existing WWTP site.

Additionally, the lagoon system will require modifications. Lagoon 1 will be converted to a complete mix aerated lagoon environment to accommodate wastewater treatment needs. In a complete mix aerated lagoon, sufficient mixing energy is provided to maintain the lagoon solids in suspension always. A completely mixed aerated lagoon system performs as an activated sludge process without solids recycle. The higher mixing energy, as compared to a partial mix lagoon, creates greater opportunity for contact between the naturally-occurring micro-organisms in the lagoon and dissolved organic matter. As a result, complete mix lagoons provide greater levels of treatment within a smaller volume than partial mix lagoons. However, facilities must be provided downstream of complete mixed lagoons to allow removal of settleable solids from the water column. To provide a place for solids settling, lagoons 2 through 4 will continue to act as partial mix aerated lagoons downstream of the complete mix lagoon 1. Lagoon 4 will require no aeration and will continue to be covered to deprive algae of sunlight and allow suspended solids to settle out of the system effluent.

Based on published soil information, the proposed slow rate groves can accommodate at least a 50 percent flow increase from estimated LCC conversion project flows. Dual-ring infiltrometer testing will be conducted during design to confirm the actual percolation rate of the site soils. However, in order to dispose of the full buildout anticipated flow, additional slow rate basins at another location may have to be identified.

Section 6

Implementation

Table 6-1 provides the implementation schedule for the WWTP. The LCCs will be closed following connection of the existing sewer system to the WWTP.

Table 6-1. Implementation Schedule	
Description	**Milestone**
Complete design of WWTP	June 28, 2020
Complete construction of WWTP	February 28, 2022
Connect existing collection system to WWTP	April 17, 2022

Section 7

Alternative Treatment Options Evaluation

Several other treatment alternatives were considered for the Naalehu WWTP, as summarized below.

7.1 Option Descriptions

7.1.1 Option 1: Aerated Lagoons/Constructed Wetland/Land Application

Option 1 consists of an aerated lagoon treatment system with a constructed wetland and disinfection, followed by land application for effluent management, as described previously throughout this report. Figure 7-1 is a schematic diagram for Option 1.

Figure 7-1. Option 1 Schematic Diagram

7.1.2 Option 2: R-1 Treatment/Land Application

Option 2 consists of constructing a membrane bioreactor (MBR) or an activated sludge treatment process followed by cloth media filtration, followed by UV disinfection, to produce recycled water that meets DOH R-1 recycled water criteria. R-1 recycled water is effluent that has undergone oxidation, filtration, and disinfection. R-1 is considered the highest grade of recycled water and can be used for irrigation of golf courses, parks, schools, and all types of agricultural crops. The R-1 treatment system would be followed by land application as per Option 1. Figure 7-2 is a schematic diagram for Option 2.

Figure 7-2. Option 2 Schematic Diagram

7.1.3 Option 3: R-1 Treatment/Seasonal Water Recycling

Option 3 consists of a treatment system similar to Option 2 to produce R-1 recycled water. The recycled water would be used to irrigate nearby coffee farms and other agricultural uses. In addition, the recycled water could be used to irrigate parks and school fields in Naalehu. Figure 7-3 provides a schematic diagram of Option 3.

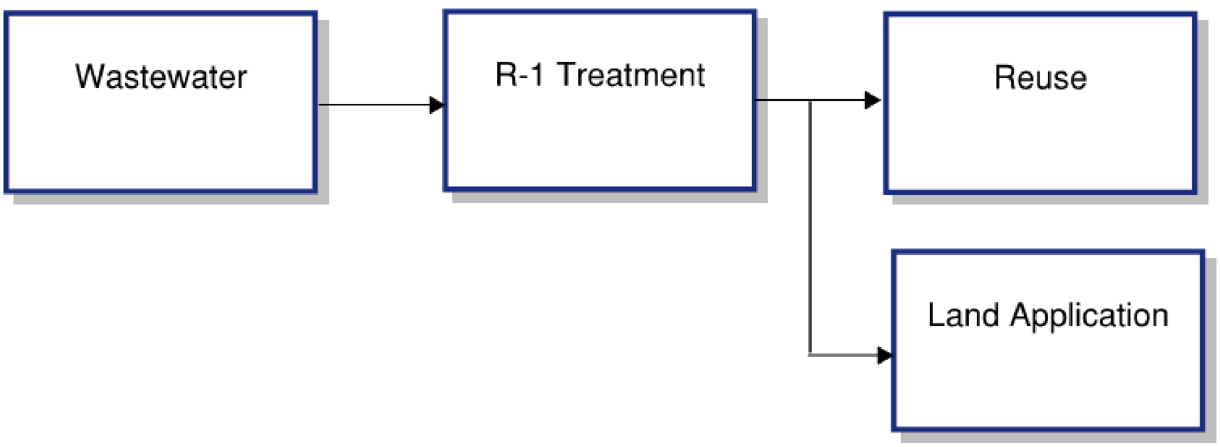


Figure 7-3. Option 3 Schematic Diagram

A water recycling analysis was prepared to assess the potential seasonal demand for recycled water produced by the WWTP. Figure 7-4 is an irrigation demand assessment for the Naalehu area based on published climate data. The graph shows precipitation, estimated evapotranspiration, and the irrigation demand for each month of the year. As shown in the figure, irrigation is typically needed from February through October, reaching a peak demand in June. The graph shows that no irrigation is typically needed between November and January, because precipitation exceeds evapotranspiration during those months.

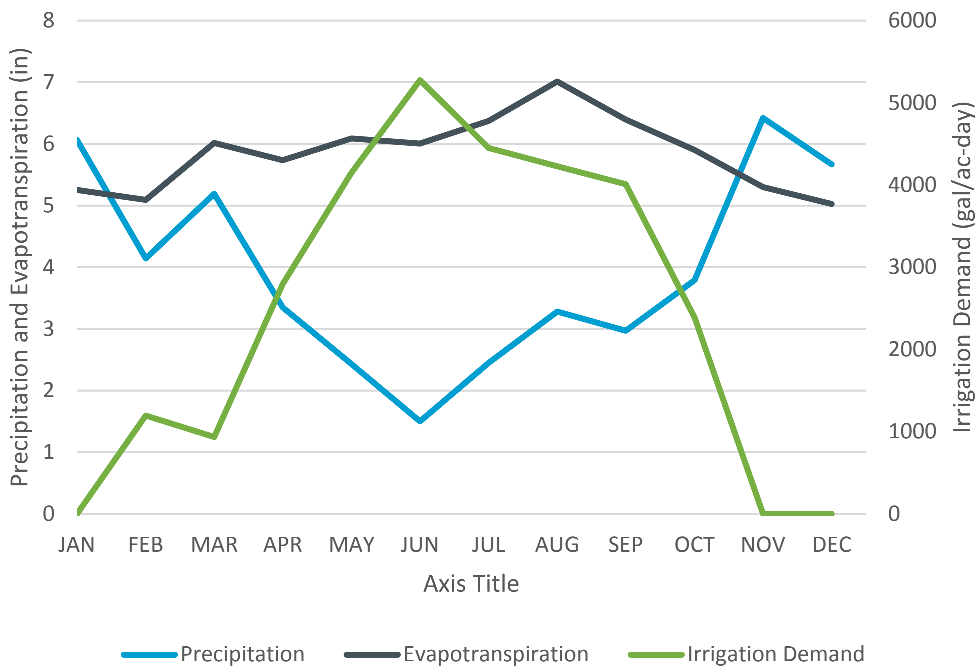


Figure 7-4. Irrigation Demand Assessment

The potential demand for recycled water produced by the Naalehu WWTP was assessed, as shown in Figure 7-5. The WWTP could potentially provide irrigation water for approximately 43 acres, based on the peak month irrigation demand in June. During June, all the recycled water produced by the WWTP would be used on the 43 acres. During all other months the supply of recycled water will typically exceed the demand, and the excess water would be land applied on the WWTP property as per the previous alternatives.

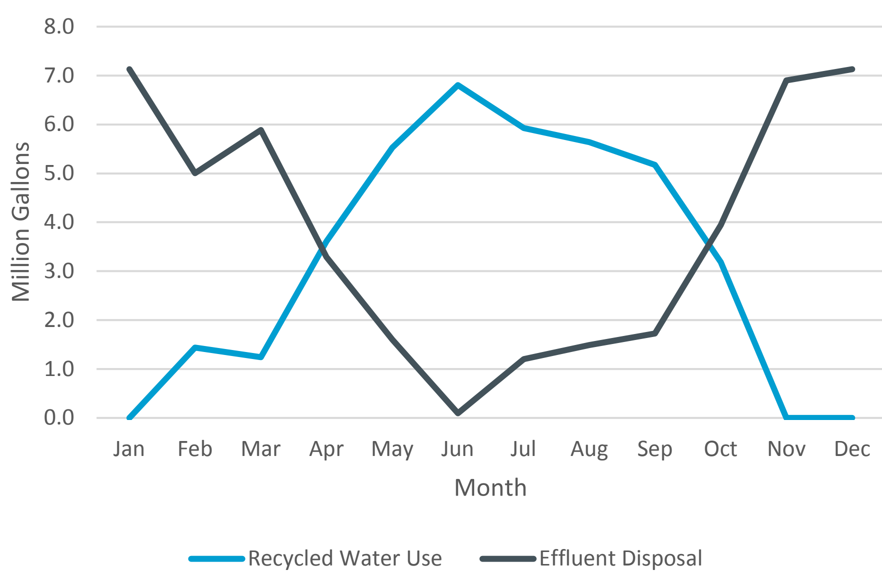


Figure 7-5. Option 3 Recycled Water Demand Assessment

The Naalehu climate makes it possible to recycle only about 46 percent of the annual flow in this scenario, due to the long wet season and relatively low evapotranspiration rate during the dry season. This is in stark contrast to the Kailua-Kona area on the leeward side of the island, where the climate will allow approximately 88 percent of the recycled water produced at the Kealakehe WWTP throughout the year to be recycled. Figure 7-6 provides a comparison of the irrigation demand in Naalehu with the irrigation demand at Kealakehe.

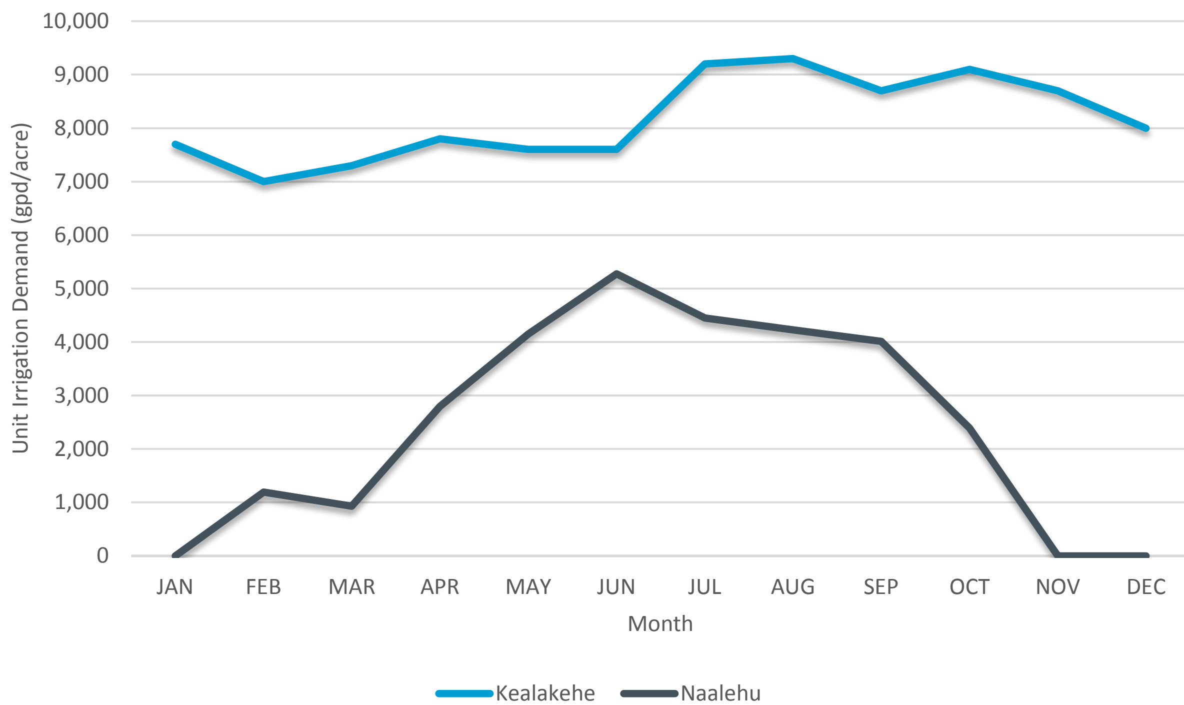


Figure 7-6. Comparison of Irrigation Demands at Naalehu and Kealakehe

7.1.4 Option 4: R-1 Treatment and Storage for 100% Water Recycling

Option 4 adds a seasonal storage reservoir, as shown schematically in Figure 7-7.

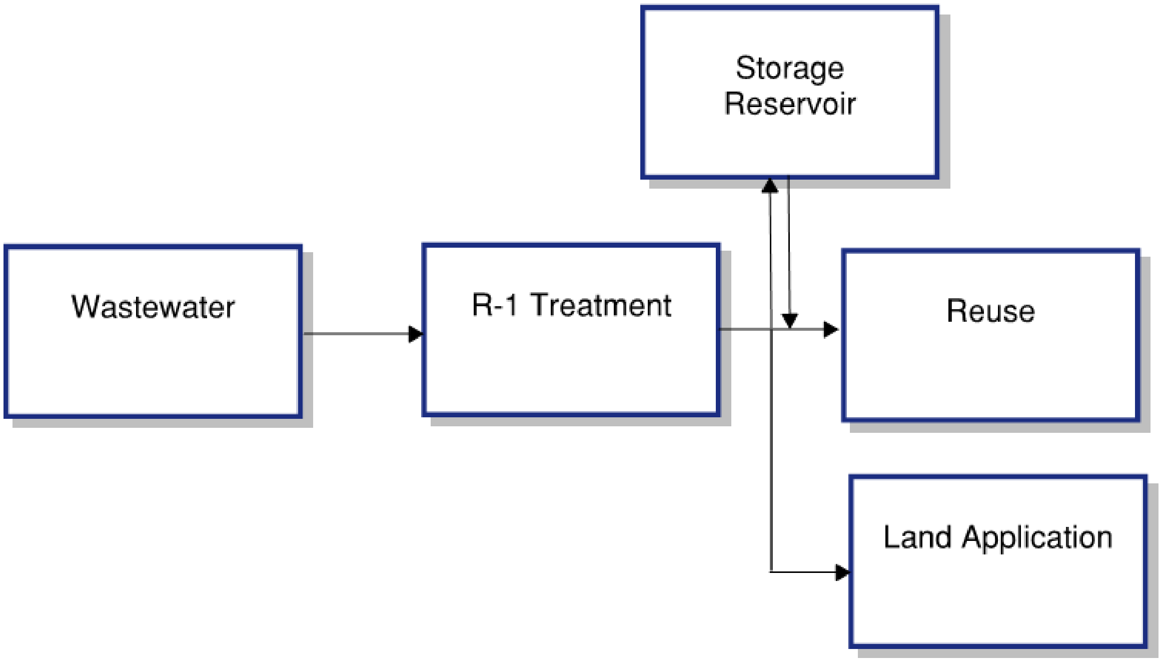


Figure 7-7. Option 4 Schematic Diagram

Implementation of a seasonal storage reservoir would make it possible to recycle 100 percent of the R-1 water produced by the Naalehu WWTP in a typical year. The seasonal storage reservoir would make it possible to save recycled water produced during the wet season for use during the dry season. An annual water balance was prepared to assess the seasonal storage reservoir needs for the Naalehu WWTP. Figure 7-8 provides a summary of the evaluation, and shows recycled water supply, use, and storage throughout a typical year. As shown in the graph, peak storage of approximately 30 million gallons (Mgal) would occur during March, and by September the storage reservoir would be dry and ready for another wet season. Under this scenario it would be possible to irrigate approximately 95 acres land. The lined, 20-foot-deep storage reservoir would have a water surface area of approximately 4.7 acres.

Storage of recycled water is not without its challenges. Recycled water contains nutrients that allow algae to grow. The algae can cause odors if stagnant water conditions are allowed to develop. Recycled water that is stored in open reservoirs must often be re-treated to improve the water quality characteristics. Recycled water reservoirs can be equipped with mixers to prevent stagnant water conditions, and/or be equipped with floating covers to block the sunlight that fosters algal growth.

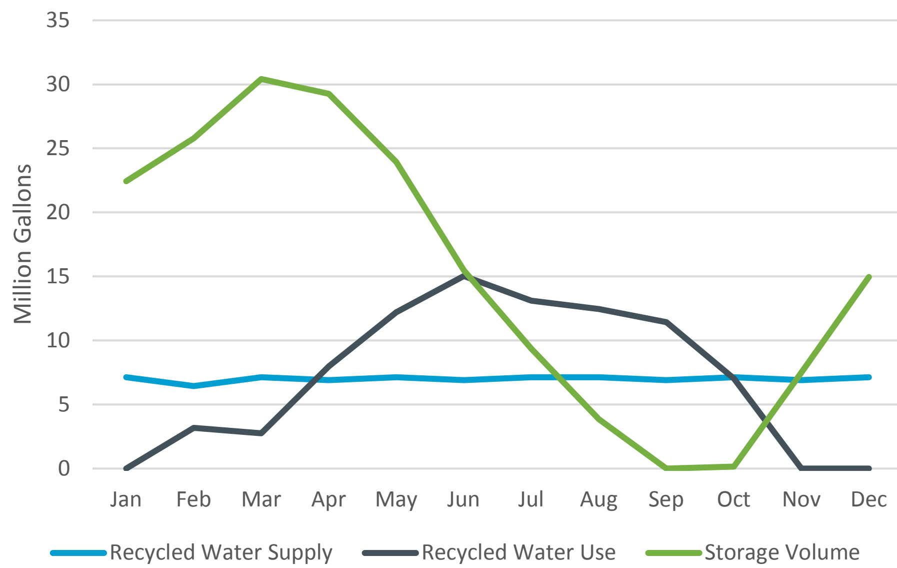


Figure 7-8. Seasonal Storage Reservoir Analysis

Implementation of a seasonal storage reservoir and recycling program would not eliminate the need for a land application system at the WWTP, as described previously. HAR 11-62 requires a disposal system for all recycled water system, to provide a means for disposal of water that does not meet R-1 standards or disposal of excess water should the seasonal storage reservoir capacity be exceeded during an exceptionally wet year.

7.1.5 Option 5: Maximum Practical Treatment

Option 5 consist of implementing advanced wastewater treatment processes that represent maximum practical treatment. The option is illustrated schematically in Figure 7-9. The process treatment train consists of a 5-stage Bardenpho activated sludge treatment process, followed by chemical addition and denitrifying filters to reliably reduce total nitrogen to less than 4 mg/L and total phosphorus to less than 0.1 mg/L. The treatment processes would be followed by a disinfection process to create R-1 recycled water. The recycled water produced would be used to irrigate macadamia nut trees as per Option 3. A seasonal storage reservoir could also be implemented at additional cost. A land application system would be required as per the previous Options.

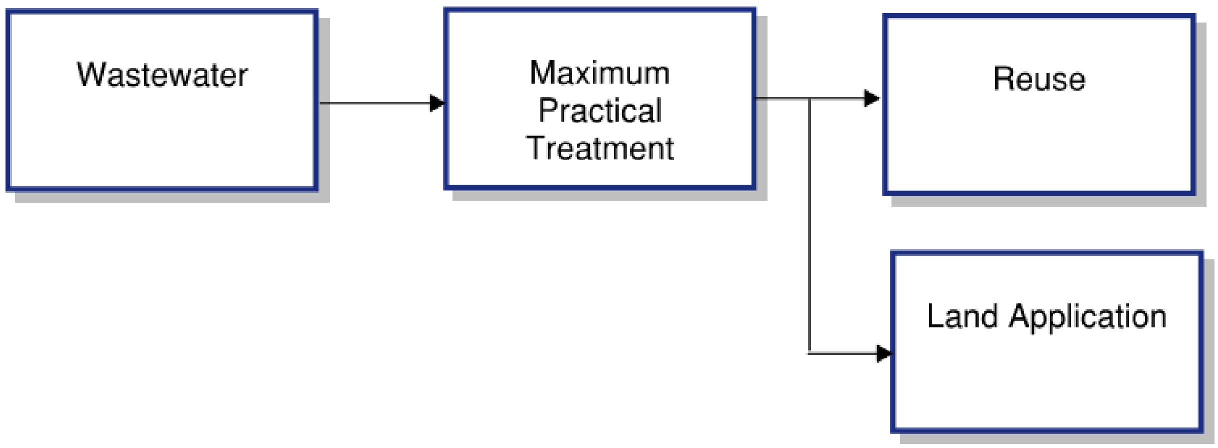


Figure 7-9. Option 5 Schematic Diagram

7.2 Cost Comparisons

Planning-level cost estimates were prepared for the five options, as described below.

7.2.1 Capital Costs

Table 7-1 summarizes the capital costs associated with the options described above. Additional detail can be found in Appendix B. The capital costs shown in the table do not include costs associated with collection system improvements or closure of the existing LCCs.

Table 7-1. Summary of Capital Cost Estimates

Option	Name	Estimated Capital Cost
1	Aerated lagoons/constructed wetland/land application	$35.1 million
2	R-1 treatment/land application	$45.0 million
3	R-1 treatment/seasonal water recycling	$47.1 million
4	R-1 treatment and storage for 100% water recycling	$54.4 million
5	Maximum practical treatment	$55.0 million

Comparison of options 1 and 2 shows that providing R-1 treatment instead of the aerated lagoon and wetland natural treatment system will increase the capital cost by approximately $4.5 million. Option 3 shows that addition of water recycling to reuse approximately 46 percent of the annual flow would add an additional $2.1 million in capital costs. Option 4 shows that constructing a seasonal storage reservoir to recycle 100 percent of the flow would add an additional $7.3 million in capital costs. Comparison of options 3 and 5 shows that providing maximum practical treatment instead of normal R-1 treatment would add $7.9 million in capital costs.

7.2.2 Operation and Maintenance Costs

Operation and maintenance (O&M) costs include labor, electricity, chemicals, spare parts, sludge management, and other costs required to operate and maintain the facility. Table 7-2 provides a summary of the O&M cost estimates developed for the options. Additional details can be found in Appendix B.

Table 7-2. Summary of O&M Cost Estimates		
Option	Name	Estimated Annual O&M Cost
1	Aerated lagoons/constructed wetland/land application	$328,000
2	R-1 treatment/land application	$1,100,000
3	R-1 treatment/seasonal water recycling	$1,106,000
4	R-1 treatment and storage for 100% water recycling	$1,122,000
5	Maximum practical treatment	$1,493,000

As shown in the table above, Option 1 incurs significantly lower O&M costs than the other options. The significant cost differential is due to the simple aerated lagoon natural treatment system that requires less labor, electricity, chemical, and maintenance than the other options.

7.2.3 Recycled Water Sale Proceeds

Options 3, 4, and 5 will produce a marketable product in the form of R-1 recycled water that could be sold to users for irrigation purposes. The value of recycled water is a function of the value of the water that it replaces. In general, recycled water is sold to users at a fraction of the price of the water that is being replaced to provide a financial incentive to use the product. The typical recycled water price is 25 percent to 90 percent of the cost of the water it replaces.

The Naalehu WWTP will be located at elevation 690 feet MSL. The cost to pump groundwater from the basal lens to the ground surface at the WWTP is approximately $992 per million gallons. Table 7-3 provides a summary of a recycled water sales assessment of each option, assuming the recycled water is sold for 90 percent of the cost of the irrigation water it would replace. Additional detail is provided in Appendix B.

Table 7-3. Summary of Annual Recycled Water Sale Proceeds			
Option	Name	Annual Volume Recycled (Mgal)	Maximum Annual Sales Proceeds
1	Aerated lagoons/constructed wetland/land application	0	$0
2	R-1 treatment/land application	0	$0
3	R-1 treatment/seasonal water recycling	39	$34,000
4	R-1 treatment and storage for 100% water recycling	85	$76,000
5	Maximum practical treatment	39	$34,000

7.2.4 Life-Cycle Costs

Life-cycle costs represent the total costs to the community to construct and operate the wastewater treatment system over a 30-year period. The life-cycle cost evaluation includes capital and O&M costs, and recycled water sales proceeds as described above. In addition, equipment replacement allowances are included after 20-years of operation. The life-cycle cost evaluation includes an inflationary factor to account for long-term changes in the value of money. The life-cycle costs are expressed as the Net Present Value (NPV). The NPV represents the amount of money that the County would need to set aside now in an interest-bearing account to cover all of the costs over the defined life-cycle. Table 7-4 provide a summary of the life-cycle cost evaluation. Additional detail can be found in Appendix B.

Table 7-4. Summary of Life-Cycle Cost Estimates

Option	Name	Estimated Life-Cycle Cost
1	Aerated lagoons/constructed wetland/land application	$50.3 million
2	R-1 treatment/land application	$71.9 million
3	R-1 treatment/seasonal water recycling	$72.7 million
4	R-1 treatment and storage for 100% water recycling	$80.1 million
5	Maximum practical treatment	$91.1 million

As shown in the table, Option 1 incurs the lowest life-cycle costs, and the other options would all incur substantially higher costs over the 30-year life-cycle. The life-cycle cost estimates are shown graphically in Figure 7-10. The operating costs shown in the figure include benefits (i.e., cost reductions) from recycled water sales where applicable.

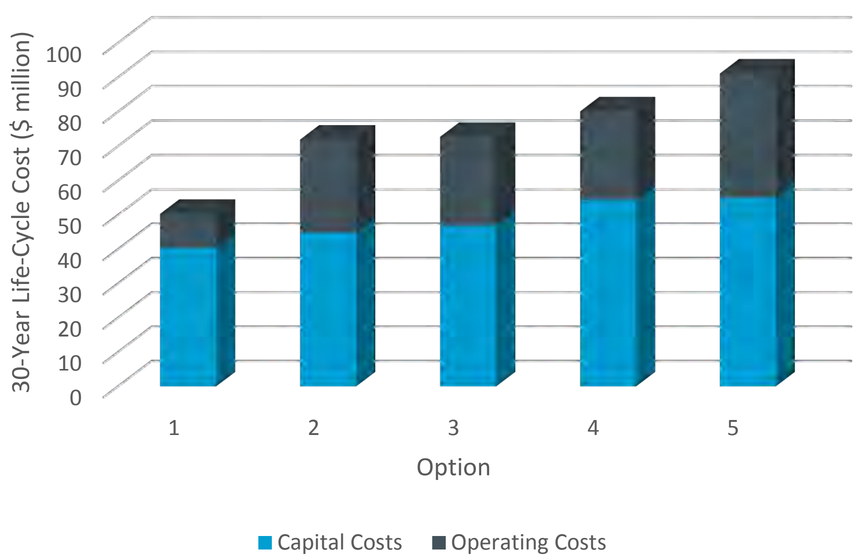


Figure 7-10. Life-Cycle Costs of Options

As shown in the graph, the operating cost differential between Option 1 and the other options is the leading contributor to the lower life-cycle cost of Option 1. The major operating cost differences are discussed below.

7.3 Non-Economic Discussion

The options are discussed on a non-economic basis below.

7.3.1 Labor Requirements

The Naalehu WWTP will be operated by the COH DEM, Wastewater Division that is based in Hilo. The Hilo-based WWTP operators will regularly visit to facility to check the system status, make operational adjustments, and draw samples for required laboratory testing. In addition, maintenance personnel will visit the WWTP as needed to conduct equipment and electrical system repairs.

A major difference between Option 1 and the other options is the frequency of routine operator visits required, and the number of personnel routinely required. Option 1 will require a single operator to normally visit the site once per week. The other options will require daily operator visits to conduct sampling that is required for R-1 compliance. In addition, Options 2 through 5 consist of mechanical treatment technology that required more operator attention than option 1. Table 7-5 compares the operational labor differences for the options, as expressed as full-time equivalents (FTEs).

Table 7-5. Comparison of Operational Labor Requirements

Option	Name	Estimated Operational Labor Requirement (FTEs)
1	Aerated lagoons/constructed wetland/land application	0.3
2	R-1 treatment/land application	3.7
3	R-1 treatment/seasonal water recycling	3.7
4	R-1 treatment and storage for 100% water recycling	3.7
5	Maximum practical treatment	5.6

7.3.2 Operational Complexity

HAR 11-61 establishes operator certification requirements for WWTPs. The DOH requires that certified operators operate municipal WWTPs. The larger and/or more complex the wastewater treatment process, the higher grade of operator required at the facility. Options 1 through 5 were evaluated for operator certification requirements based on the criteria established in HAR 11-61. Table 7-6 summarizes the results of the evaluation. As shown in the table, Option 1 would require a Grade I operator, while the other options would require a Grade IV operator (the highest grade). The higher requirements for Options 2 through 5 are due to the complexity of the treatment processes compared to Option 1. In general, the County has difficulty attracting and retaining Grade IV operators.

Table 7-6. Comparison of Operator Certification Requirements per HAR 11-61

Option	Name	Operator Certification Level Requirement
1	Aerated lagoons/constructed wetland/land application	I
2	R-1 treatment/land application	IV
3	R-1 treatment/seasonal water recycling	IV
4	R-1 treatment and storage for 100% water recycling	IV
5	Maximum practical treatment	IV

7.3.3 Energy Consumption

Figure 7-11 provides a comparison of the electrical energy requirements of the five options. As shown in the graph, Option 1 will require significantly less electrical energy to operate, due to the use of natural treatment systems (aerated lagoons) instead of mechanical treatment processes that require more aeration and process pumping.

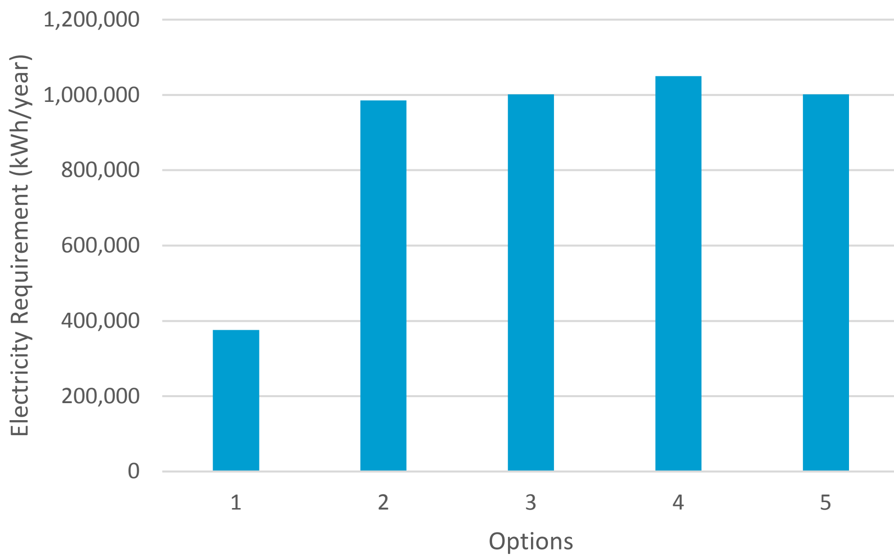


Figure 7-11. Comparison of Electrical Energy Requirements

7.3.4 Sludge Management

Sludge management for Option 1 is significantly different than the other options. The partial-mix aerated lagoon treatment system allows wastewater solids to accumulate at the bottom of the lagoon, forming a sludge blanket that slowly anaerobically digests. Sludge removal is infrequent, typically on the order once every 20 years. The resulting solids are well-digested and inoffensive due to the long retention time in the lagoons.

Options 2 through 5 would require an aerobic digester to stabilize and store waste solids from the activated sludge treatment process. The solids would need to be dewatered and trucked to a landfill on a weekly basis.

7.4 Living Machine®

Living Machine® technology was suggested during community outreach meetings. Living Machine® is a proprietary technology by Worrell Water Technologies that incorporates aerated tanks planted with vegetation to provide an attractive wastewater treatment process. In colder climates the aerated tanks are housed in a greenhouse for protection. In addition, subsurface flow wetlands with continuous and/or batch flow can be included in the process to provide desired treatment.

The Living Machine® technology has been implemented in “green” buildings like the San Francisco Public Utilities Commission building, the Port of Portland Headquarters, and others. Review of the company’s website did not reveal any municipal projects completed on the scale of what would be needed for Naalehu. Therefore, the technology is considered to be not feasible.

It should be noted that the proposed non-proprietary treatment system (aerated lagoons and subsurface flow wetland) uses essentially the same natural treatment processes as the Living Machine®, but on a municipal scale.

7.5 Septic Tank Alternatives

A previous assessment recommended installation of a community septic tank and repurposing one of the existing LCCs to serve as a seepage pit (SSFM, July 2007), in accordance with Alternative 1 proposed to the community by the County in 2004 (County of Hawaii, November 5, 2004). This and other options that have been raised during the community outreach process that incorporate septic tank technology are discussed below.

7.5.1 Community Septic Tank

The effectiveness of a septic tank is directly related to the amount of hydraulic detention time provided by the tank volume. The previous study (SSFM, July 2007) suggested a 24-hour detention time would be adequate. Applying the current flow projections for the project indicate a 230,000-gallon tank would be appropriate if this criterion is used. However, for large community septic tanks it has been found that longer detention times are needed to optimize treatment performance, avoid the need for frequent septage pumping, and to account for peak flow rates that are developed by community wastewater collection systems. Applying appropriate design criteria (Crites and Tchobanoglous, 1998), to the project results in the need for an 966,000-gallon tank, which would require pumping on a 3-year interval. The area required for an appropriately-sized community septic tank would be approximately ¼ acre.

The use of a community septic tank would require the DOH to issue a variance to HAR 11-62-23.1, which requires WWTPs with design capacities greater than 100,000 gallons per day to produce effluent containing less than 30 mg/L of both BOD_5 and TSS; septic tanks are not able to produce effluent of this quality. A secondary treatment process is needed to comply with the effluent quality requirements contained in the DOH regulations. The County would need to reapply for the variance every 5-years, and if not renewed then secondary treatment would need to be provided.

Additionally, odors from a community septic tank present a significant concern. A septic tank is an anaerobic treatment process that produces hydrogen sulfide, reduced sulfur compounds, and other odorous gases. Odors emanating from septic tanks at individual residences are typically dispersed to the atmosphere throughout the community via the household plumbing roof vents. A community septic tank would concentrate the community's emissions to a single point source that would require foul air collection and treatment to avoid nuisance odor conditions. A dual-stage scrubber capable of treating approximately 4,300 cubic feet per minute of foul air would be required to avoid nuisance odor conditions. The dual-stage scrubber would consist of a biotrickling filter, followed by a granular activated scrubber.

7.5.2 Converting LCC to Seepage Pit

A previous study (SSFM, July 2007) suggested that the existing LCC located on the County-owned parcel TMK 9-5-024:011 could be converted to a seepage pit that would be regulated by DOH as an injection well. HAR 11-23-07 allows injection wells located mauka of the UIC line that were in existence prior to July 6, 1984 to continue to operate. However, the flow to the wells cannot increase, nor can a new well be constructed. Therefore, the earlier plan to convert the existing LCC to a seepage pit is not feasible for the following reasons:

- Closing the other two LCCs in the community that are located on private property would increase the flow to the LCC (converted to a seepage pit that is regulated as an injection well) that is located on County property.
- Percolation testing conducted on the existing cesspool on County-owned parcel TMK 9-5-024:011 revealed a disposal capacity of 3 gpm, (Masa Fujioka & Associates, February 9, 2009) or about 4,320 gpd. Cleaning the cesspool would likely increase the capacity somewhat, but the resulting capacity would be far below what the community needs.
- HAR 11-62-25 requires new and proposed effluent disposal systems to have a backup disposal system capable of handling the peak flow. A second seepage pit cannot be constructed to comply with the regulatory requirement because the site is located mauka of the UIC line. If the existing seepage pit were to fail then a replacement cannot be constructed.
- The Kau Community Development Plan requires the County to provide for eventual construction of sewers throughout the community. Providing sewers for the entire community will increase wastewater flows considerably, as presented in Section 5. Increasing flow to the existing LCC (converted to a seepage pit) would not be allowed. Therefore, the use of the existing LCC as a disposal system could prevent the County from providing the community's desired future wastewater needs.
- Act 131 (18), signed into law on July 5, 2018, prohibits DOH from issuing permits "for the construction of sewage wastewater injection wells unless alternative wastewater disposal options are not available, feasible, or practical."

For these reasons, converting the existing LCC to a seepage pit is considered to be not feasible.

7.5.3 Leachfield Disposal

Leachfields are effluent disposal systems consisting of buried gravel-filled absorption trenches. Significant treatment occurs as septic tank effluent percolates through the soil surrounding the leachfield trenches. Leachfields are an integral part of residential septic systems, and DOH has established trench design criteria applicable to both residential and municipal-scale leachfields. In particular, HAR 11-62-34 requires trenches to be sized based on bottom area only. Application of the DOH criteria to the project yields a need for at least 25 acres of land to satisfy DOH hydraulic loading rate and redundancy requirements. Achieving even distribution of effluent over a leachfield of this size would be challenging at best. Therefore, leachfield disposal for the project is considered to be not feasible.

7.5.4 Conversion to Individual Wastewater Systems

The concept of a community wastewater system could be abandoned, and all houses be required to construct individual wastewater systems comprised of a septic tank and leachfield. However, many of the lots in the community are small (less than 10,000 square feet) and significantly improved, making the feasibility of constructing individual wastewater systems on every lot uncertain. HAR 11-62-34 allows construction of seepage pits where there is insufficient land area to install absorption trenches (i.e., a leachfield), but prohibits construction in soils having percolation rates slower than 10 minutes per inch or where rapid percolation through such soils may result in contamination of water-bearing formations. The soils in the community are classified as Naalehu medial silty clay loams in the National Resource Conservation Service soil survey. Borings at County-owned parcel TMK 9-5-024:011 revealed soil depths varying from 2 feet to 27 feet deep over hard basaltic rock or clinker. Lots with inadequate soil depth for conventional soil absorption trenches would be required to import fill soil to create elevated mound systems in accordance with HAR 11-62-34 to achieve adequate soil depth. Residents without sufficient space could potentially install seepage pits if suitable subsurface geology could be located. Conversion to individual wastewater systems is considered to be not feasible due to the high level of uncertainty associated with site conditions on the small lots.

Section 8

Alternative Site Evaluation

Thirty-two sites were evaluated as potential locations for the Naalehu WWTP. Each site was assessed for twenty-one criteria, in four broad categories: environmental, social and cultural; location and site; land use and availability; and collection system and service area.

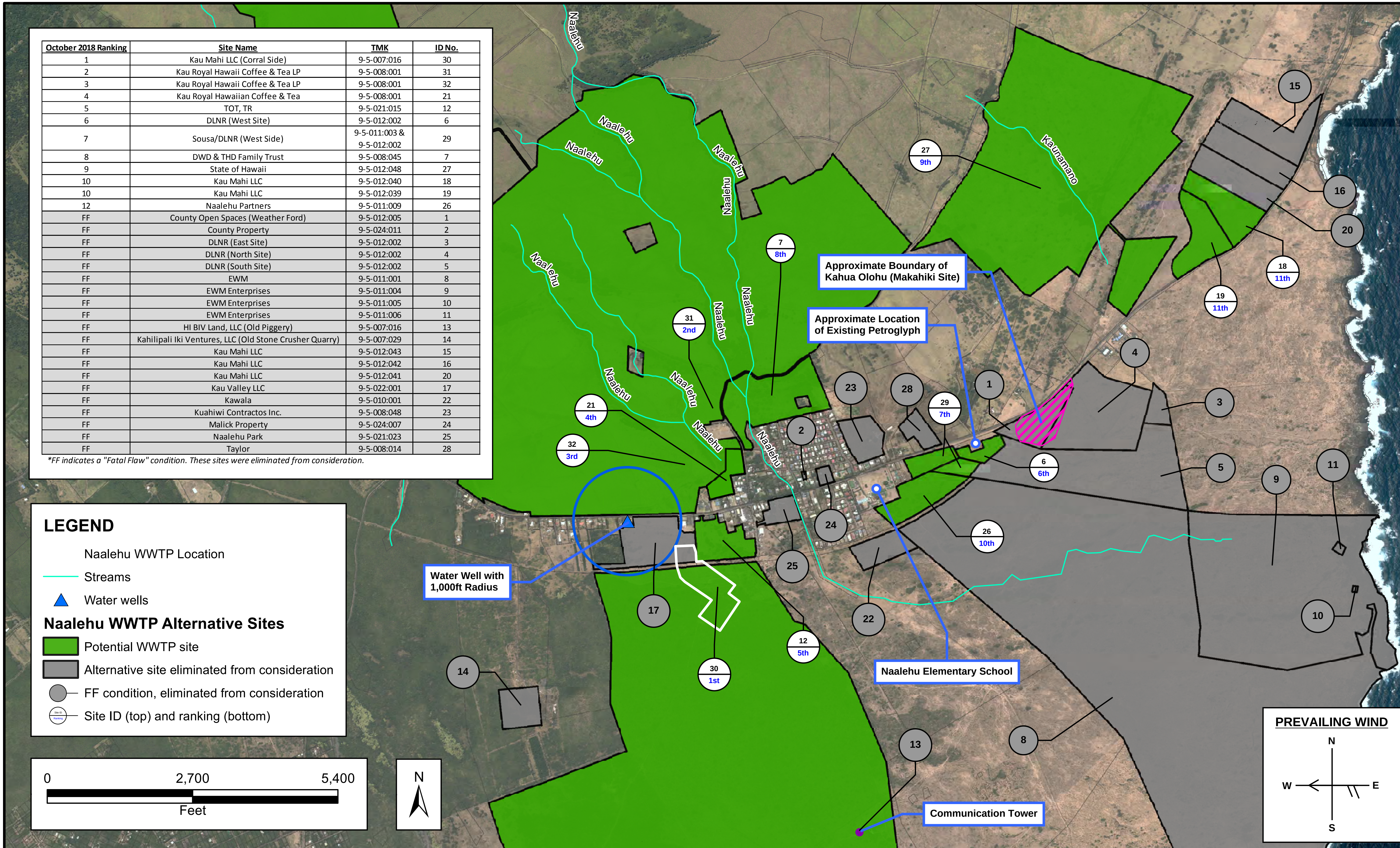
8.1 Methodology

The site evaluation was performed according to the following process:

1. Potential sites for the Naalehu WWTP were initially identified by the Department of Environmental Management. Additional sites were identified based on feedback from the Naalehu community obtained during Community Outreach meetings that took place in April 2018.
2. Four general categories and twenty-one criteria were established and defined for the analysis.
3. Six "fatal flaw" conditions were identified. Sites with one or more fatal flaw were eliminated from further consideration.
4. Relative weighting factors were established for each category and criteria.
5. Sites were mapped using GIS. Data such as soil type, location of subsurface and surface water, topography, zoning and prevailing wind direction were determined.
6. Each site was evaluated and scored for the twenty-one criteria.
7. A weighted ranking was determined for each site, based on the weighting factors established in Step 4.
8. A preferred site was identified, based on the weighted high score.

8.2 Site Locations

Ownership, location, and proximity to the existing LCCs for all siting alternatives considered is illustrated in Figure 8-1.



October 2018 Ranking	Site Name	TMK	ID No.
1	Kau Mahi LLC (Corral Side)	9-5-007:016	30
2	Kau Royal Hawaii Coffee & Tea LP	9-5-008:001	31
3	Kau Royal Hawaii Coffee & Tea LP	9-5-008:001	32
4	Kau Royal Hawaiian Coffee & Tea	9-5-008:001	21
5	TOT, TR	9-5-021:015	12
6	DLNR (West Site)	9-5-012:002	6
7	Sousa/DLNR (West Side)	9-5-011:003 & 9-5-012:002	29
8	DWD & THD Family Trust	9-5-008:045	7
9	State of Hawaii	9-5-012:048	27
10	Kau Mahi LLC	9-5-012:040	18
10	Kau Mahi LLC	9-5-012:039	19
12	Naalehu Partners	9-5-011:009	26
FF	County Open Spaces (Weather Ford)	9-5-012:005	1
FF	County Property	9-5-024:011	2
FF	DLNR (East Site)	9-5-012:002	3
FF	DLNR (North Site)	9-5-012:002	4
FF	DLNR (South Site)	9-5-012:002	5
FF	EWM	9-5-011:001	8
FF	EWM Enterprises	9-5-011:004	9
FF	EWM Enterprises	9-5-011:005	10
FF	EWM Enterprises	9-5-011:006	11
FF	HI BIV Land, LLC (Old Piggery)	9-5-007:016	13
FF	Kahilipali Iki Ventures, LLC (Old Stone Crusher Quarry)	9-5-007:029	14
FF	Kau Mahi LLC	9-5-012:043	15
FF	Kau Mahi LLC	9-5-012:042	16
FF	Kau Mahi LLC	9-5-012:041	20
FF	Kau Valley LLC	9-5-022:001	17
FF	Kawala	9-5-010:001	22
FF	Kuahiwi Contractos Inc.	9-5-008:048	23
FF	Malick Property	9-5-024:007	24
FF	Naalehu Park	9-5-021:023	25
FF	Taylor	9-5-008:014	28

**FF indicates a "Fatal Flaw" condition. These sites were eliminated from consideration.*

Brown AND Caldwell

SCALE AS SHOWN

JOB NO.: 151494

NAALEHU WASTEWATER TREATMENT PLANT

Naalehu WWTP Alternative Sites

FIGURE

8-1

8.3 Criteria

The criteria used for the analysis are presented for each of four categories in Tables 8-1, 8-2, 8-3 and 8-4. A score was assigned to each criterion based on definitions included in the tables. A score of five represents a preferred or positive condition, and a score of one a less preferred or negative condition. A score of zero indicates a fatal flaw; six fatal flaw conditions were identified during the analysis are identified in the corresponding table.

Table 8-1 outlines the environmental, social, and cultural criteria considered in the analysis.

Table 8-1. Environmental, Social and Cultural Criteria

Criteria	Scoring and Definitions					
	5	4	3	2	1	0 = Fatal Flaw
Presence of or proximity to archaeological/cultural sites	No known or suspected sites	Confirmed or suspected sites and mitigable	No information available	Suspected sites and mitigation ability unknown	Confirmed sites and mitigation ability unknown	Confirmed sites and unmitigable
Proximity of treatment units to existing occupied buildings	More than 1000 ft. from any occupied building		Between 50 and 1000 ft. from non-school building	Between 50 and 1000 ft. of school	Less than 50 ft from any occupied building	
Prevailing wind direction	Site is downwind of most of the community		Site is central		Site is upwind of most of the community	
Biology	Endangered or threatened species not present		Presence of endangered or threatened species unknown		Endangered or threatened species known to be present	Endangered or threatened species known to be present and unmitigable
Visual impact	Natural visual mitigation (hill, berm, vegetation, remoteness) exists		Visible location, mitigable with trees or other engineered buffers		Visible location, unmitigable	
Contamination from prior land use	No suspected industry-related contamination issues		Presence of contamination unknown		Suspected or confirmed contamination issues	
Previously disturbed or developed	Yes		Partial		No previous development or disturbance	

The circumstance where a cultural or historical site is known to exist within the treatment facility footprint and mitigation to relocate, protect, or preserve that site is not possible, was identified as a fatal flaw condition.

From an environmental perspective, the presence of endangered or threatened species was considered negative. A site previously disturbed or developed was viewed as positive, unless contamination from a previous land use was suspected.

Considerations specific to social impact include proximity to occupied buildings (including residences, school, commercial establishments and others), prevailing wind direction, and visual impact. Based on community feedback received in April 2018, locations near Naalehu Elementary School were considered less favorable than other locations.

Table 8-2 outlines the location and site characteristics considered in the analysis.

Table 8-2. Location and Site Characteristics

Criteria	Scoring and Definitions					
	5	4	3	2	1	0 = Fatal Flaw
Parcel size	More than 14.9 acres					Less than 14.9 acres
Subsurface Geology	Good soil and in sufficient amounts in area of parcel useable for disposal				Good soil but over limited area and disposal modification required or marginal soil in area of parcel useable for disposal	No soil in area of parcel useable for disposal or no clinker layer for drainage
Topography	Gentle slopes (less than 8%)		Moderate slopes (8% - 18%) or localized high/low points		Steep slopes (18% - 20%)	Extreme slopes (greater than 20%)
Proximity to water well	Outside of both 1000 ft. radius and upgradient influence zone of any well		Outside of 1000 ft. but suspected within upgradient influence zone of non-potable well		Within 1000 ft. or within upgradient influence zone of non-potable well	Within 1000 ft. or within upgradient influence zone of potable well
Presence of lava tubes	None		Possible or unknown		Known	
Proximity to surface water, intermittent stream or coast line	Treatment and disposal more than 500 ft. away		Treatment and disposal between 50 to 500 ft.		Treatment and disposal less than 50 ft. away	
Flood control / drainage	No risk of flooding	Flood risk known and mitigatable	Flood risk unknown		Prone to flooding or within flood zone	
Vehicle access	Vehicle access currently exists		Existing easement, but new road or significant road upgrades required in or via county/private right if way	Existing easement, but new road or significant road upgrades required in or via state right-of-way	No current vehicle access or easement, access legally restricted, or significant obstruction to access	

Table 8-2. Location and Site Characteristics						
Criteria	Scoring and Definitions					
	5	4	3	2	1	0 = Fatal Flaw
Power and potable water availability	Utilities currently available at property line and within 400 ft. of site, no new easement required, no known significant obstructions (i.e. - culverts, streams, cultural sites)		Utilities available within 400 yds. of property or unknown		Potable water and/or power not currently available within 400 yds. of property and/or significant obstruction to utility construction	

Three fatal flaw conditions were identified for the location and site characteristics category in Table 8-2:

- Sites less than 14.9 acres in size, which is the least amount of land needed for treatment, disposal, and future growth.
- Average slopes greater than 20 percent, which significantly increase the cost of construction and limit design options.
- Location within a 1000-foot radius surrounding a potable water well, which is prohibited by HAR 11-62 for the protection of drinking water in the State of Hawaii.

Table 8-3 outlines the collection system and service area characteristics considered in the analysis.

Table 8-3. Collection System and Service Area Criteria					
Criteria	Scoring and Definitions				
	5	4	3	2	1
Distance from LCC collection area	Parcel is adjacent to existing LCC or less than 0.25 miles away	Parcel is 0.25-0.5 mile away from existing LCC	Parcel is 0.5-1.0 miles away from existing LCC	Parcel is 1.0 – 1.5 miles away from existing LCC	Parcel is more than 1.5 miles away from existing LCC
Gravity flow possible or pumping required	Gravity flow possible				Pumping required for wastewater transmission from collection area to site
Number of properties newly accessible	Central village commercial area becomes accessible				Additional individual residential properties become accessible outside of LCC service area

A site location requiring large transmission distances of more than two miles are less preferable due to both initial capital cost and future operations and maintenance requirements. Similarly, sites where wastewater can flow via gravity from the collection area are preferable to those requiring a pump station.

Newly accessible refers to properties within the service area that are not currently connected to the LCC, but will become accessible to the County-owned sewer system when the collection lines are relocated into the roadways fronting the property. Hawaii County Code requires connection of these properties once the new collection system is constructed, and their individual wastewater systems (cesspools or septic tanks) properly removed from service. All individual cesspools in the State of Hawaii must be converted or closed by the year 2050.

In accordance with the Kau CDP, locations that enhanced the ability to serve commercial facilities were considered favorable in the analysis.

No fatal flaws were identified for the Collection System and Service Area category.

Table 8-4 outlines the land use and availability characteristics considered in the analysis.

Table 8-4. Land Use and Availability Criteria

Criteria	Scoring and Definitions				
	5	4	3	2	1
Current zoning and land use	WWTP currently permitted in zoning without Special Permit		WWTP possible onsite Special Permit required		WWTP not recommended on site
Land availability	Owner willing and able to sell or land currently government (state, county) owned	Subdivision required or friendly condemnation required	Difficult or lengthy approval process expected or owner willingness to sell unknown	Owner unwilling to sell or unfriendly condemnation of land required (private corporate owner)	Owner unwilling to sell or unfriendly condemnation required (private family owner)

Although public facilities are permitted in any zoning in the County of Hawaii, construction of a wastewater treatment facility requires a Special Permit within some zones. Based on community feedback received in April 2018, locations necessitating condemnation of private, family-owned properties were considered much less favorable than locations with a willing seller or currently government-owned. No fatal flaws were identified for the land use and availability category.

8.4 Criteria Weighting Factors

To consider the relative importance to the categories and criteria, each was assigned a weighting factor for the analysis. Weighting allows for appropriate consideration of all factors - both the technical and non-technical - associated with siting. Relative weighting is summarized in Table 8-5.

Table 8-5. Relative Weighting Factors

Category	Category Weight	Criteria	Criteria Weight
Environmental, social and cultural	35%	Presence of and/or proximity to archaeological/cultural sites	25%
		Proximity of treatment units to existing occupied buildings	25%
		Prevailing wind direction	25%
		Biology	10%
		Visual impact	5%
		Contamination from prior land use	5%
		Previously disturbed or developed	5%
			100%
Location and site characteristics	35%	Parcel size	25%
		Soils type	25%
		Topography	15%
		Proximity to water well	10%
		Presence of lava tubes	8%
		Proximity to surface water, intermittent stream or coast line	6%
		Flood control / drainage	5%
		Existing vehicle access	3%
		Power and potable water availability	3%
			100%
Collection system and service area	15%	Distance from LCC collection area	50%
		Gravity flow possible or pumping required	30%
		Number of properties newly accessible	20%
			100%
Land use and availability	15%	Current ownership	55%
		Current zoning and land use	45%
			100%

8.5 Raw Scores

For the thirty-two sites identified in Figure 8-1, raw scores were assigned for each of the twenty-one criteria according to the definitions in Section 8.3. The results are presented in Table 8-6.

Table 8-6. Alternatives Analysis – Raw Scores

Category	Criteria	Site Raw Score																															
		1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30	31	32
Environmental, social and cultural	Presence of and/or proximity to archaeological/cultural sites [a]	FF																															
	Proximity of treatment units to existing occupied buildings																																
	Prevailing wind direction																																
	Biology																																
	Visual impact																																
	Contamination from prior land use																																
	Previously disturbed or developed																																
Location and site characteristics	Parcel size [b]		FF								FF	FF			FF								FF	FF	FF	FF			FF				
	Soils type [c]	FF		FF	FF	FF			FF	FF				FF	FF	FF	FF						FF										
	Topography [d]																				FF												
	Proximity to water well [e]																	FF															
	Presence of lava tubes																																
	Proximity to surface water, intermittent stream or coast line																																
	Flood control / drainage																																
	Existing vehicle access																																
	Power and potable water availability																																
Collection system and service area	Distance from LCC collection area																																
	Gravity flow possible or pumping required																																
	Number of properties newly accessible																																
Land use and availability	Current zoning and land use																																
	Current ownership																																
	Raw score totals (maximum possible = 105)	FF	FF	FF	FF	FF	*75*	*72*	FF	FF	FF	FF	*78*	FF	FF	FF	FF	FF	*65*	*65*	FF	*79*	FF	FF	FF	FF	***68***	***68***	FF	*77*	*81*	*77*	*75*

[a] *Fatal flaw condition for Site 1.*

[b] *Fatal flaw condition for Sites 2,10,11,14,22,23,24,25,and 28.*

[c] *Fatal flaw condition for Sites 1,4,5,8,9,13, 14,15,16, and 22.*

[d] *Fatal flaw condition for Site 20.*

[e] *Fatal flaw condition for Site 17.*

The scoring was completed based on the best information available at the time of writing. Changing circumstances can and do affect the scoring over time. Circumstances that have affected the ranking of sites in Naalehu include:

- The identification of an unmitigable cultural site at Site 1 during archaeologic investigation.
- Community preference for a larger than regulated buffer between the elementary school and treatment facilities during community outreach meetings.
- Willingness of Site 29 owners to sell.
- Elimination of injection wells as a disposal method for consideration resulting in larger area requirements for disposal.
- Inadequate soil permeability conditions for disposal identified during investigations of Sites 2, and 24.
- Subsurface conditions not conducive to disposal identified during exploratory geotechnical drilling on Site 13.

As indicated in Table 8-6, fatal flaw conditions were identified for the following 19 sites:

- Sites 2,10,11,14,22,23,24,25, and 28 (due to site size).
- Sites 1,4,5,8,9,13, 14,15,16, and 22. (due to subsurface or soil conditions).
- Site 1 (due to unmitigable archaeologic/cultural site).
- Site 20 (due to extreme slopes).
- Sites 17 (area within 1000 ft of a potable water well).

These affected areas were removed from further analysis.

8.6 Weighted Analysis

The weighted analysis is presented in Table 8-7.

Table 8-7. Alternatives Analysis – Weighted Scores

Category	Criteria	Site Raw Score																															
		1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30	31	32
Environmental, social and cultural	Presence of and/or proximity to archaeological/cultural sites [a]	FF					0.25	0.75					0.75	1					0.75	0.75		0.75					0.75	0.75		1	1	0.75	0.75
	Proximity of treatment units to existing occupied buildings						1.25	0.75					0.75	1.25					1.25	1.25		0.75					0.5	1.25		0.5	1.25	1.25	1.25
	Prevailing wind direction						0.25	0.75					1.25	1.25					1.25	1.25		0.75					1.25	1.25		0.25	1.25	1.25	1.25
	Biology						0.3	0.3					0.3	0.3					0.3	0.3		0.3					0.3	0.3		0.3	0.3	0.3	0.3
	Visual impact						0.15	0.15					0.15	0.25					0.15	0.15		0.15					0.15	0.25		0.15	0.25	0.15	0.25
	Contamination from prior land use						0.15	0.15					0.15	0.25					0.15	0.15		0.15					0.15	0.15		0.15	0.15	0.15	0.15
	Previously disturbed or developed						0.15	0.15					0.25	0.15					0.15	0.15		0.25					0.15	0.15		0.25	0.25	0.25	0.25
Location and site characteristics	Parcel size [b]		FF				1.25	1.25			FF	FF	1.25	1.25	FF				1.25	1.25		1.25	FF	FF	FF	FF	1.25	1.25	FF	1.25	1.25	1.25	1.25
	Soils type [c]	FF		FF	FF	FF	1.25	0.75	FF	FF			0.75	0.25	FF	FF	FF		0.25	0.25		1.25	FF				1.25	0.25		1.25	1.25	1.25	1.25
	Topography						0.75	0.45					0.75	0.75					0.15	0.15	FF	0.45					0.45	0.45		0.75	0.75	0.45	0.45
	Proximity to water well [d]						0.5	0.5					0.5	0.5				FF	0.5	0.5		0.5					0.5	0.5		0.5	0.5	0.5	0.5
	Presence of lava tubes						0.08	0.24					0.24	0.24					0.24	0.24		0.24					0.24	0.24		0.24	0.24	0.24	0.24
	Proximity to surface water, intermittent stream or coast line						0.3	0.3					0.3	0.3					0.3	0.3		0.18					0.3	0.3		0.3	0.3	0.18	0.18
	Flood control / drainage						0.15	0.15					0.15	0.15					0.15	0.15		0.15					0.15	0.15		0.15	0.15	0.15	0.15
	Existing vehicle access						0.15	0.15					0.15	0.09					0.15	0.15		0.15					0.09	0.09		0.15	0.09	0.15	0.09
	Power and potable water availability						0.15	0.15					0.15	0.03					0.03	0.03		0.15					0.09	0.03		0.15	0.09	0.09	0.03
Collection system and service area	Distance from LCC collection area						1.5	2					2	1					0.5	0.5		2					2	1		2.5	2	1.5	1.5
	Gravity flow possible or pumping required						1.5	0.3					0.3	0.3					0.3	0.3		0.3					1.5	0.3		1.5	0.3	0.3	0.3
	Number of properties newly accessible						0.6	0.6					1	1					0.6	0.6		1					0.6	0.6		0.6	1	1	1
Land use and availability	Current zoning and land use						1.35	1.35					2.25	1.35					1.35	1.35		2.25					1.35	1.35		1.35	1.35	1.35	1.35
	Current ownership						2.75	1.65					1.65	2.2					2.75	2.75		2.2					1.65	2.75		0.55	2.2	1.65	1.65
	Weighted score totals (maximum possible = 5	FF	FF	FF	FF	FF	3.61	3.31	FF	FF	FF	FF	3.79	3.62	FF	FF	FF	FF	3.15	3.15	FF	3.70	FF	FF	FF	FF	3.77	3.37	FF	3.68	4.19	3.79	3.78

8.7 Results

The results of the analysis are presented in Table 8-8. Nineteen sites were identified as having fatal flaws and the remaining thirteen were ranked in accordance with the overall weighted score.

Table 8-8. Naalehu Alternative Site Ranking

Rank	Site
1	30
2	31
3	32
4	21
5	12
6	6
7	29
8	7
9	27
10 (tie)	16
10 (tie)	18
12	26

The top three sites for the Naalehu WWTP are:

1. Site 30 (TMK 9-5-007:016)
2. Site 31 (TMK 9-5-008:001)
3. Site 32 (TMK 9-5-008:001)

Site 30 is preferred to the second and third ranked sites for the following reasons:

- Preliminary Archaeological investigations for Site 30, indicate no unmitigable cultural sites in the vicinity of the proposed facility.
- Existing land use for Site 31 and 32 include coffee and tea production and an established greenhouse. Site 30 is currently range land.
- The current landowners of Site 30 are amenable to subdivision and sale.
- Site 31 and 32 require uphill forcemain pumping transmission
- The topography of Site 30 is less steep than Sites 31 and 32
- Site 30 is closer to the collection area than site 31 and 32

8.8 Conclusion

Based on the analysis, Site 30 (TMK 9-5-007:016) was selected as the preferred location for the Naalehu WWTP.

Section 9

References

County of Hawaii. Letter to Community Homeowners, signed by Mayor Harry Kim. November 5, 2004.

Crites, Ron, and George Tchobanoglous. Small and Decentralized Wastewater Management Systems. WCB McGraw-Hill, 1998.

Crites, Ronald W., E. Joe Middlebrooks, Robert K. Bastian, and Sherwood C. Reed. "Natural Wastewater Treatment Systems, Second Edition". CRC Press, 2014.

Crites, Ronald W., E. Joe Middlebrooks, Sherwood C. Reed. Natural Wastewater Treatment Systems. CRC Taylor & Francis, 2006.

Crites, Ronald W., Sherwood C. Reed, and Robert K. Bastian. "Land Treatment Systems for Municipal and Industrial Wastes". McGraw-Hill, 2000.

Department of Planning, County of Hawaii. Kau Community Development Plan. October 2017.

Department of Public Works. Storm Drainage Standards, County of Hawaii, 1970.

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.

Fukunaga and Associates, Inc. Revised Preliminary Engineering Report – Naalehu Sewage Transmission, Wastewater Treatment and Disposal System. June 2013.

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

Hawaii Administrative Rules (HAR), Title 11, Department of Health Administrative Rules.

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

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

NRCS (National Resources Conservation Service). Web Soil Survey, located on the Internet at address https://websoilsurvey.nrcs.usda.gov/app/HomePage.htm . 2018.

Reed, Sherwood C., Ronald W. Crites, E. Joe Middlebrooks. Natural Systems for Waste Management and Treatment. McGraw-Hill, Inc. 1995.

Rich, Linvil G. High Performance Aerated Lagoon Systems. American Academy of Environmental Engineers, 1999.

SSFM International, Inc. Final Preliminary Engineering Report for Naalehu and Naalehu 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.

Water Pollution Control Federation. Aeration, Manual of Practice FD-13. 1988.

White, George Clifford, *Handbook of Chlorination and Alternative Disinfectants*, John Wiley & Sons Inc., New York, 1999.

Appendix A: Flow Projections Summary

Naalehu WWTP - Flow Projections Summary

Naalehu LCC Conversion Project

	BSF	*GWI*	*ADWF (BSF+GWI)*	*PBSF (BSFx2.5)*	*PDWF (PBSF+GWI)*	*I/I*	*Design Flow (PDWF+I/I)*	*Equivalent Population*
Existing Service Area	52,430	26,215	*78,645*	131,075	157,290	96,162	*253,452*	749
Newly Accessible to Collection System	91,620	54,180	*145,800*	229,050	283,230	152,593	*435,823*	1,548
Grand Total	144,050	80,395	***224,445***	360,125	440,520	248,755	***689,275***	2,297

Naalehu Full Buildout Condition

	BSF	*GWI*	*ADWF (BSF+GWI)*	*PBSF (BSFx2.5)*	*PDWF (PBSF+GWI)*	*I/I*	*Design Flow (PDWF+I/I)*	*Equivalent Population*
Existing Service Area	52,430	26,215	*78,645*	131,075	157,290	96,162	*253,452*	749
Newly Accessible to Collection System	91,620	54,180	*145,800*	229,050	283,230	152,593	*435,823*	1,548
Future Service Area (per Kau CDP)	109,340	54,670	*164,010*	273,350	328,020	755,733	*1,083,753*	1,562
Grand Total	*253,390*	*135,065*	***388,455***	*633,475*	*768,540*	*1,004,488*	***1,773,028***	*3,859*

NOTE: Design flow calculations based on City and County of Honolulu Wastewater System Design Standards, July 2017 (Chapter 2).

Abbreviations:

BSF	Base Sanitary Flow
PBSF	Peak Base Sanitary Flow
ADWF	Average Dry Weather Flow
GWI	Groundwater Infiltration Rate
PDWF	Peak Dry Weather Flow
I/I	Wet Weather Infiltration / Inflow
Design Flow	Peak Hourly Flow

Appendix B: Cost Estimates

County of Hawaii Department of Environmental Management

Naalehu WWTP
Options Assessment Cost Summary

Capital Costs

Option No.	Treatment	Disposal	Recycling	Capital Cost ($M)									**Total ($M)**
				Lagoons	R-1	Limit of TT	Drainage	Disposal	Reservoir	Diurnal Tank	R-1 Pumps	R-1 Pipelines	
1	Aerated lagoons/wetland/disinfection	Land application	None	18.2			14.3	8.0					**40.5**
2	MBR (R-1)	Land application	None		22.8		14.3	8.0					**45.0**
3	MBR (R-1)	Land application	Seasonal (25% of total annual flow)		22.8		14.3	8.0		1.0	0.5	0.6	**47.1**
4	MBR (R-1)	Land application	Annual storage reservoir (100% of flow)		22.8		14.3	8.0	4.4	2.3	1.0	1.7	**54.4**
5	Limit of treatment technology	Land application	Seasonal (25% of total annual flow)			30.6	14.3	8.0		1.0	0.5	0.6	**55.0**

Annual O&M Costs

No.	Treatment	Disposal	Recycling	Annual O&M Costs ($)					
				Labor	Electricity	Chemicals	Maintenance	Sludge Mgmt	Total
1	Aerated lagoons/wetland/disinfection	Land application	None	$42,000	$132,000	$14,000	$91,000	$49,000	$328,000
2	MBR (R-1)	Land application	None	$582,000	$345,000	$10,000	$114,000	$49,000	$1,100,000
3	MBR (R-1)	Land application	Seasonal (46% of total annual flow)	$582,000	$351,000	$10,000	$114,000	$49,000	$1,106,000
4	MBR (R-1)	Land application	Annual storage reservoir (100% of flow)	$582,000	$367,000	$10,000	$114,000	$49,000	$1,122,000
5	Limit of treatment technology	Land application	Seasonal (46% of total annual flow)	$874,000	$351,000	$41,000	$153,000	$74,000	$1,493,000

Annual Recycled Water Sales

No.	Treatment	Disposal	Recycling	Annual R-1 Water Sales	
				High Price	Low Price
1	Aerated lagoons/wetland/disinfection	Land application	None	$0	$0
2	MBR (R-1)	Land application	None	$0	$0
3	MBR (R-1)	Land application	Seasonal (46% of total annual flow)	$34,000	$19,000
4	MBR (R-1)	Land application	Annual storage reservoir (100% of flow)	$76,000	$42,000
5	Limit of treatment technology	Land application	Seasonal (46% of total annual flow)	$34,000	$19,000

Equipment Replacement at 20-Years

No.	Treatment	Disposal	Recycling	Equipment Replacement
1	Aerated lagoons/wetland/disinfection	Land application	None	$4,554,000
2	MBR (R-1)	Land application	None	$5,688,000
3	MBR (R-1)	Land application	Seasonal (46% of total annual flow)	$5,688,000
4	MBR (R-1)	Land application	Annual storage reservoir (100% of flow)	$5,688,000
5	Limit of treatment technology	Land application	Seasonal (46% of total annual flow)	$7,662,000

County of Hawaii Department of Environmental Management
Naalehu WWTP
Preliminary Options Assessment - Capital Costs

Common Capital Inputs

Current ENRCCI:	11170	**Sep-18**
Area markup factor:	30%	
Contingency factor:	20%	
Project soft costs factor:	25%	

Lagoon-Wetland Treatment

Description	Quantity	Units	Unit Cost	Extension
Clear and grub	7	AC	$5,000	$35,000
BMPs	7	AC	$13,000	$91,000
Earthwork	27,000	CY	$100	$2,700,000
Sewer extension	1,200	LF	$160	$192,000
Headworks	1	EA	$500,000	$500,000
Lagoons	1	LS	$2,500,000	$2,500,000
Wetlands	1	LS	$350,000	$350,000
Chlorine contact tank	1	LS	$125,000	$125,000
Chlorine feed system	1	LS	$30,000	$30,000
Operations building	1,500	SF	$500	$750,000
Generator and tank	1	LS	$250,000	$250,000
Fencing	1,500	LF	$100	$150,000
Paving	25,000	SY	$55	$1,375,000
Water line extension	1,200	LF	$160	$192,000
Yard piping	1	LS	$600,000	$600,000
Miscellaneous site work	1	LS	200,000	$200,000
HELCO power	1	LS	50,000	$50,000
Hawaiian Telcom	1	LS	20,000	$20,000
Archeological monitoring	4	AC	2,500	$10,000

Subtotal		$10,120,000	
Electrical and instrumentation	20%	$2,024,000	
Total construction		$12,144,000	
Contingency		$2,428,800	
Total construction		$14,572,800	
Project soft costs		$3,643,200	
Total project cost:		**$18.2**	**million**

Land Application

Description	Quantity	Units	Unit Cost	Extension
Clear and grub	11	AC	$5,000	$55,000
BMPs	11	AC	$13,000	$143,000
Earthwork	33,500	CY	$100	$3,350,000
Fencing	1,700	LF	$100	$170,000
Gravel access roads	26,000	SY	$30	$780,000
Yard piping	4,000	LF	$160	$640,000
Planting	9	AC	10,000	$90,000
Effluent flow meter and sampler	1	LS	50,000	$50,000
Archeological monitoring	11	AC	2,500	$27,500

Subtotal		$5,305,500	
Electrical and instrumentation	0%	$0	
Total construction		$5,305,500	
Contingency		$1,061,100	
Total construction		$6,366,600	
Project soft costs		$1,591,650	
Total project cost:		**$8.0**	**million**

Drainage Channel

Description	Quantity	Units	Unit Cost	Extension
Clear and grub	1	AC	$5,000	$5,000
BMPs	1	AC	$13,000	$13,000
Earthwork (1,500' channel, 50' wide)	49,500	CY	$100	$4,950,000
Concrete Culvert (3 to cover 24' x 50')	1	LS	$750,000	$750,000
Archeological monitoring	1	AC	2,500	$2,500

Subtotal		$5,720,500	
Electrical and instrumentation	0%	$0	
Total construction		$5,720,500	
Contingency		$5,720,500	
Total construction		$11,441,000	
Project soft costs		$2,860,250	
Total project cost:		**$14.3**	**million**

R-1 Treatment

Capacity:	0.225	mgd	
Mainland cost at current ENRCCI:	$51.85	/gpd	from R-1 WWRF capital regression. y=28.639*(x^-0.398)
Local construction cost:	$67.41	/gpd	
Construction estimate:	$15.2	million	
Contingency:	$3.0	million	
Total construction cost:	$18.2	million	
Project soft costs:	$4.6	million	
Total project cost:	**$22.8**	**million**	

Limit of Treatment Technology

ENRCCI of estimate:	8952		
10 mgd WWTP cost:	$13.80	/gpd	
10 mgd WWTP cost at current ENRCCI:	$17.22	/gpd	
Small flow escalation:	$69.85	/gpd	y=44.65x^-0.3 Per WERF analysis. BNR + advanced nutrient removal
Local cost:	$90.81	/gpd	
Construction estimate:	$20.4	million	
Contingency:	$4.1	million	
Total construction cost:	$24.5	million	
Project soft costs:	$6.1	million	
Total project cost:	**$30.6**	**million**	

Seasonal Storage Reservoir

Volume:	91	ac-ft
Mainland construction cost:	$25,000	/ac-ft
Subtotal:	$2.3	million
Local construction cost:	$2.9	million
Contingency:	$0.6	million
Total construction cost:	$3.5	million
Project soft costs:	$0.9	million
Total project cost:	**$4.4**	**million**

Diurnal R-1 Tank - Seasonal Program

Volume:	0.225	mgal	1 peak day
Local construction cost:	$3.00	/gallon	
Subtotal:	$0.7	million	
Contingency:	$0.1	million	
Total construction cost:	$0.8	million	
Project soft costs:	$0.2	million	
Total project cost:	$1.0	million	

Diurnal R-1 Tank - Reservoir Program

Volume:	0.50	mgal	1 peak day
Local construction cost:	$3.00	/gallon	
Subtotal:	$1.5	million	
Contingency:	$0.3	million	
Total construction cost:	$1.8	million	
Project soft costs:	$0.45	million	
Total project cost:	$2.3	million	

R-1 Delivery Pumps - Seasonal Program

Peak day flow	0.225	mgal
Delivery time:	8	hours
Pumping capacity:	469	gpm
Mainland construction cost @ ENRCCI 4500:	$100,000	
Current mainland construction cost:	$248,000	
Local construction cost:	$322,000	
Contingency:	$64,400	
Total construction cost:	$386,400	
Project soft costs:	$96,600	
Total project cost:	$0.5	million

R-1 Delivery Pumps - Reservoir Storage

Peak day flow	0.50	mgal
Delivery time:	8	hours
Pumping capacity:	1044	gpm
Mainland construction cost @ ENRCCI 4500:	$200,000	
Current mainland construction cost:	$496,000	
Local construction cost:	$645,000	
Contingency:	$129,000	
Total construction cost:	$774,000	
Project soft costs:	$193,500	
Total project cost:	$1.0	million

R-1 Pipelines - Seasonal Program

Peak delivery rate:	469	gpm
Pipeline diameter:	8	inches
Hawaii construction cost:	$25	/in-ft
Estimated length:	2000	feet
Local construction cost:	$400,000	
Contingency:	$80,000	
Total construction cost:	$480,000	
Project soft costs:	$120,000	
Total project cost:	$0.6	million

R-1 Pipelines - Reservoir Storage

Peak delivery rate:	1044	gpm
Pipeline diameter:	10	inches
Hawaii construction cost:	$25	/in-ft
Estimated length:	4500	feet
Local construction cost:	$1,125,000	
Contingency:	$225,000	
Total construction cost:	$1,350,000	
Project soft costs:	$337,500	
Total project cost:	$1.7	million

County of Hawaii Department of Environmental Management

Naalehu WWTP
Preliminary Options Assessment
O&M Costs

Common O&M Inputs

Labor cost:	$100	/hr (loaded)
FTE effective labor:	1,560	hours/year
Chlorine tab cost:	$4	/lb
Alum cost:	$2	/lb
Electricity cost:	$0.35	/kWh
Maintenance cost:	2%	/year of equipment capital
Sludge management cost:	$1,500	/dry ton, dewatering, hauling, tip fee
Average flow:	0.225	mgd

Lagoon Treatment/Wetlands/Disinfection

Labor

Normal requirement:	1	visit/week
Operators/visit:	1	
Time per visit:	8	hours/visit
Weekly labor hours:	8	hours/week
Annual labor hours:	416	hours/year
FTEs:	0.3	FTEs
Annual labor cost:	**$41,600**	/yr

Electricity

Load	Equiv hp	Percent	kWhr/mo	$/month
Aerators	55	100%	29,530	$10,335
Screens	2	10%	107	$38
Chlorine pumps	0.5	30%	81	$28
Effluent pumps	3	100%	1,611	$564
Totals				$10,965
Annual power cost:	**$131,579**			
Annual power consumption:			375940	kWh/yr

Chemicals

Chlorine dose:	5	mg/L
Daily use:	9	lbs/d
Annual use:	3425	lbs/d
Annual cost:	**$13,698**	/yr

Maintenance

Equipment cost:	$4,554,000	(assume 25% of capital cost)
Annual maintenance:	**$91,080**	/yr

Sludge Management

Production rate:	0.4	dry tons/mgal
Annual production:	32.85	/dry tons
Sludge management cost:	$49,275	/year (deferred for 20 years)

R-1 Treatment

Labor

Normal requirement:	7	visits/week
Operators/visit:	2	
Time per visit:	8	hours/visit
Weekly labor hours:	112	hours/week
Annual labor hours:	5824	hours/year
FTEs:	3.7	FTEs
Annual labor cost:	**$582,400**	

Electricity

Daily power use:	2,700	kWh/d
Annual power use:	985,500	kWh/yr
Annual power cost:	**$344,925**	/yr

Chemicals

Annual chemical cost:	$10,000

Maintenance

Equipment cost:	$5,687,827	(assume 25% of capital cost)
Annual maintenance:	**$113,757**	/yr

Sludge Management

Sludge production:	0.4	dry tons/mgal
Annual production:	33	/dry tons
Sludge management cost:	**$49,275**	/year

Limit of Treatment Technology

Labor

Normal requirement:	7	visits/week
Operators/visit:	3	
Time per visit:	8	hours/visit
Weekly labor hours:	168	hours/week
Annual labor hours:	8736	hours/year
FTEs:	5.6	FTEs
Annual labor cost:	**$873,600**	

Electricity

Daily power use:	2,700	kWh/d
Annual power use:	985,500	kWh/yr
Annual power cost:	**$344,925**	/yr

Chemicals

Alum dose	30	mg/L
Alum use:	56	lbs/d
Alum cost:	**$41,095**	/yr

Maintenance

Equipment cost:	$7,661,677	(assume 25% of capital cost)
Annual maintenance:	**$153,234**	/yr

Sludge Management

Sludge production:	0.6	dry tons/mgal
Annual production:	49	/dry tons
Sludge management cost:	**$73,913**	/year

Seasonal Water Recycling (25%)

Load	Equiv hp	Percent	kWhr/mo	$/month
R-1 delivery pumps	10	25%	1,342	$470
Totals				$470
Annual power cost:	**$5,637**			
Annual power consumption:			16107	kWh/yr

Annual Water Recycling (100%)

Load	Equiv hp	Percent	kWhr/mo	$/month
R-1 delivery pumps	10	100%	5,369	$1,879
Totals				$1,879
Annual power cost:	**$22,550**			
Annual power consumption:			64428	kWh/yr

County of Hawaii Department of Environmental Management
Naalehu WWTP

R-1 Sales Assessment

Avoided Cost of Pumping Irrigation Water

Assume pumping from basal lens

Elevation at WWTP:	690	feet MSL
Flow rate:	1000	gpm
	2.2	cfs
Pump efficiency:	85%	
Motor efficiency:	90%	
Power cost:	$0.35	/kWh
BHP:	205	hp
Motor draw:	170	kW
Unit volume:	1	mgal
Time to pump unit vol:	16.7	hours
Power to pump unit vol:	2833	kWh
Cost to pump unit vol:	$992	

Recycled Water Pricing

High price:	90%	of avoided cost
Low price:	50%	of avoided cost

Recycled Water Sales

High price:	$893	/mgal
Low price:	$496	/mgal

Seasonal Recycling Sales

Annual reuse volume:	39	mgal
High price sales:	$34,408	/year
Low price sales:	$19,116	/year

100% Recycling Sales

Annual reuse volume:	85	mgal
High price sales:	$76,018	/year
Low price sales:	$42,232	/year

County of Hawaii Department of Environmental Management

Naalehu WWTP
Preliminary Options Assessment
Operator Requirement Evaluation

No.	Treatment	Disposal	Recycling
1	Aerated lagoons/disinfection	Land application	None
2	MBR (R-1)	Land application	None
3	MBR (R-1)	Land application	Seasonal (25% of total annual flow)
4	MBR (R-1)	Land application	Annual storage reservoir (100% of flow)
5	Limit of treatment technology	Land application	Seasonal (25% of total annual flow)

	Option				
Criteria per HAR 11-61	1	2	3	4	5
Population served	1	1	1	1	1
Design average flow	1	1	1	1	1
Effluent discharge	2	2	6	6	6
Variation on raw wastes	0	0	0	0	0
Pretreatment	5	10	10	10	10
Primary treatment	0	0	0	0	0
Secondary treatment	8	15	15	15	20
Advanced waste treatment	0	12	12	12	22
Additional treatment processes	7	7	7	7	7
Solids handling	0	19	19	19	19
Disinfection	5	10	10	10	10
Laboratory control bacteriological	0	0	0	0	0
Laboratory control chemical/physical	0	0	0	0	0
Total points	29	77	81	81	96
WWTP Classification per 11-61	I	IV	IV	IV	IV

County of Hawaii Department of Environmental Management
Naalehu WWTP
Water Recycling Assessments

Seasonal Recycling with Disposal

Average flow: 0.225 mgd
Irrigated acreage: 43 acres

Month	Days	WW Flow (mgal)	Irrig Demand (gpd/ac)	Irrig Demand (mgal)	Disposal (mgal)
Jan	31	7.0	0	0.0	7.0
Feb	28	6.3	1195	1.4	4.9
Mar	31	7.0	933	1.2	5.7
Apr	30	6.8	2797	3.6	3.1
May	31	7.0	4148	5.5	1.4
Jun	30	6.8	5275	6.8	-0.1
Jul	31	7.0	4448	5.9	1.0
Aug	31	7.0	4229	5.6	1.3
Sep	30	6.8	4010	5.2	1.6
Oct	31	7.0	2391	3.2	3.8
Nov	30	6.8	0	0.0	6.8
Dec	31	7.0	0	0.0	7.0
Totals	365	82.125		39	44

Recycling efficiency: **47%**

Recycling with Annual Storage Reservoir

Average flow: 0.225 mgd
Irrigated acreage: 95 acres
Reservoir surface area: 4.7 acres
Reservoir pan coefficient: 0.7

					Reservoir Storage									
Month	Days	WW Flow (mgal)	Irrig Demand (gpd/ac)	Irrig Demand (mgal)	WW in (mgal)	Precipitation in (inches)	Precipitation in (mgal)	Pan Evap (inches)	Evap out Inches	Evap out (mgal)	Delta Storage (mgal)	Cumulative Storage (mgal)	Cumulative Storage (ac-ft)	Water Depth (feet)
Jan	31	7.0	0	0.0	7.0	5.98	0.8	4.55	3.2	0.4	7.3	21.8	67.0	14.3
Feb	28	6.3	1195	3.2	3.1	3.77	0.5	4.54	3.2	0.4	3.2	25.0	76.8	16.3
Mar	31	7.0	933	2.7	4.2	5.45	0.7	4.97	3.5	0.4	4.5	29.5	90.6	19.3
Apr	30	6.8	2797	8.0	-1.2	3.23	0.4	5.4	3.8	0.5	-1.3	28.2	86.6	18.4
May	31	7.0	4148	12.2	-5.2	1.94	0.2	5.6	3.9	0.5	-5.5	22.7	69.8	14.8
Jun	30	6.8	5275	15.0	-8.3	1.56	0.2	5.94	4.2	0.5	-8.6	14.1	43.3	9.2
Jul	31	7.0	4448	13.1	-6.1	3.27	0.4	6.37	4.5	0.6	-6.3	7.8	24.1	5.1
Aug	31	7.0	4229	12.5	-5.5	3.08	0.4	6.23	4.4	0.6	-5.6	2.2	6.7	1.4
Sep	30	6.8	4010	11.4	-4.7	3.6	0.5	5.55	3.9	0.5	-4.7	0.0	0.0	0.0
Oct	31	7.0	2391	7.0	-0.1	3.98	0.5	5.05	3.5	0.5	0.0	0.0	0.0	0.0
Nov	30	6.8	0	0.0	6.8	6.7	0.9	4.49	3.1	0.4	7.2	7.2	22.1	4.7
Dec	31	7.0	0	0.0	7.0	5.82	0.7	4.62	3.2	0.4	7.3	14.5	44.5	9.5
Totals	365	82.125		85		48.4	6.2	63.3		5.7	-2.5			

Recycling efficiency: **104%**

Max Volume: 30 Mgal
91 ac ft

Peak demand: 15.0 mgal/mo
0.50 mgd

County of Hawaii, DEM
Naalehu WWTP Options Assessment
Alternatives Net Present Value Analysis

Agency: County of Hawaii, DEM

Project/Problem: Naalehu WWTP Options Assessment

	Description	Risk Premium	Sensitivity Adjustments (%) Benefits	Capital Costs	Other Costs	Results: Capital Cost	30-year NPV	Benefit over Status Quo
Alternative 1	Lagoons / wetlands/ disinfection / land application					$40,500,000	($50,317,478)	
Alternative 2	R-1 treatment / land application					$45,000,000	($71,891,318)	($21,573,840)
Alternative 3	R-1 treatment / seasonal recycling (25%)					$47,100,000	($72,719,813)	($22,402,335)
Alternative 4	R-1 treatment / annual storage res (100%)					$54,400,000	($80,146,964)	($29,829,486)
Alternative 5	Limit of treatment technology / 25% recycle					$55,000,000	($91,059,595)	($40,742,117)
Alternative 6								
Alternative 7								
Alternative 8								
Alternative 9								
Alternative 10								
Alternative 11								
Alternative 12								

Year of analysis: 2018
Escalation rate: 3.20%
Discount rate: 5.50%

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