Final Report

Hilo Landfill Feasibility Study

Hawai‘i County, Department of Environmental Management

March 28, 2012

R·W·BECK

An SAIC Company

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Letter of Transmittal

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LIST OF ACRONYMS

Acronym	Definition
AUF	Airspace Utilization Factor
BLNR	Hawai'i State Board of Land and Natural Resources
BOD	biological oxygen demand
CAA	Clean Air Act
CFR	Code of Federal Regulations
cm/sec	centimeters per second
COD	chemical oxygen demand
County	County of Hawai'i
cy	cubic yards
DBEDT	Department of Business, Economic Development, and Tourism
DEM	County of Hawai'i Department of Environmental Management
DHHL	Department of Hawaiian Homelands
DOH	Hawai'i Department of Health
DOTA	Department of Transpiration Airports Division
EA	Environmental Assessment
EIS	Environmental Impact Statement
EO	Governor's Executive Order
EPA	United States Environmental Protection Agency
FAA	Federal Aviation Administration
FEMA	Federal Emergency Management Agency
FEIS	Final Environmental Impact Statement
FTE	full-time equivalent, or full-time employee
GCCS	landfill gas collection and control system
GCL	geosynthetic clay liner
gpd	gallons per day
HAR	Hawai'i Administrative Rules
HDD	Hawai'i State Department of Defense
HDPE	high density polyethylene
HELP	Hydrologic Evaluation of Landfill Performance
HHL	Hawaiian Homelands
HWA	HWA Geosciences, Inc.
IRSWMP	County of Hawai'i Integrated Resources and Solid Waste Management Plan

Acronym	Definition
KMR	Keaukaha Military Reservation
KS	Kamehameha Schools
lb/cy	pounds per cubic yard
LFG	landfill gas
M	million
Mg	megagrams
MGD	million gallons per day
msl	mean sea level
MSW	Municipal Solid Waste
NMOC	Non-methane organic compounds
NSPS	New Source Performance Standards
P&R	County of Hawai'i Parks and Recreation
RCRA	Resource Conservation and Recovery Act
RD&D	Research, Development and Demonstration
ROW	Right of Way
RP	Revocable Permit
SHSL	South Hilo Sanitary Landfill
SSR	side slope riser
TMK	Tax Map Key
UIC	Underground Injection Control
VOC	volatile organic compounds
WETT	Whole Effluent Toxicity Testing
WHSL	West Hawai'i Sanitary Landfill
WWD	Wastewater District
WWTP	Wastewater Treatment Plant

R. W. Beck, Inc. evaluated the technical, economic, environmental, socioeconomic and regulatory/permitting feasibility of continued waste disposal through landfilling adjacent to the existing County of Hawai‘i (County) South Hilo Sanitary Landfill (SHSL) using wetlands as the primary method of leachate treatment with treated effluent disposed via ground discharge. In addition, we compared the planning-level economics of that waste disposal method with disposal of east County waste at the West Hawai`i Sanitary Landfill (WHSL).

Technical Feasibility

A landfill adjacent to the existing SHSL is technically feasible, including the use of onsite constructed subsurface wetlands for treatment of the expected volume of leachate that would be produced from a lined landfill in an area of high rainfall. However, it is likely that the Hawai`i Department of Health (DOH) would require a fully redundant backup system (100 percent backup) of proven technology for leachate treatment because the use of constructed wetlands is not a DOH recognized treatment system. While the County's existing Hilo wastewater treatment plant (WWTP) would be considered a proven technology for leachate treatment, it presently lacks the capacity to provide the necessary backup without a major and very expensive expansion. In addition, the WWTP has, since August 2010, been experiencing apparent effluent toxicity issues as measured by whole effluent toxicity testing (WETT). Even if the WWTP was expanded, the addition of leachate to the plant would further complicate the resolution of the plant's failure to pass the WETT standard. Finally, the addition of leachate to the WWTP, if allowed, would in all likelihood trigger a requirement to institute and administer a costly wastewater pretreatment program. To date, such a program has been avoided given the plant's throughput being at or below the industrial wastewater discharge program threshold level of 5 million gallons per day (MGD) because there are currently no Significant Industrial Users.

Permitting Feasibility

The probability of obtaining a DOH permit for siting and operating a landfill immediately adjacent to the existing landfill, or an expansion of the existing landfill, is very low (high uncertainty) for the following reasons:

1. Unlike the current unlined landfill, the new landfill area would be required to meet United States Environmental Protection Agency (USEPA) Subtitle D rules including a liner system. A strong argument might be made by the DOH that such a landfill would be classified as a new landfill rather than a lateral expansion of the existing landfill. The new landfill, if so classified, would be located within six

miles of a public airport, and as such is not permitted under Federal Aviation Administration (FAA) rules unless a variance was granted. A lateral expansion of the existing landfill, if so classified, may be grandfathered in under FAA rules. However, the hazard it presents to aviation from potential bird strikes would be the same as a new landfill.

2. The DOH has indicated that they cannot provide an initial assessment of the permittability of a lateral expansion of the existing landfill until a permit application is submitted. The process would therefore require a significant investment in design development and environmental review to develop a permit application and obtain a permit ruling.
3. Presently, the DOH does not have standards for discharging treated leachate into the ground using injection or infiltration galleries. Therefore, if onsite treatment of leachate were included, new discharge standards would need to be developed by DOH, which could be problematic not the least from a schedule standpoint.

Land use approval and permitting are discussed on the follow page.

Economic Feasibility

Assuming the permitting and leachate treatment challenges discussed above could be overcome, we estimate the cost in 2011 dollars per ton of waste disposed for a new landfill, including capital and operating costs using onsite constructed subsurface wetland treatment of leachate with 100 percent WWTP backup, to range from approximately $70/ton to $107/ton (Sections 2 and 6). Alternatively, our estimated cost per ton of waste for a new landfill that relies solely on an expanded WWTP for leachate treatment (no onsite wetland treatment) ranges from $85/ton to $130/ton, which reflects the added cost of delivery and processing leachate at the WWTP.

One feasible alternative in the near term to continued landfilling in the South Hilo area would be to haul waste from the sort station near Hilo to the WHSL. We estimate the cost for this alternative disposal method, including hauling costs and the savings on disposal costs for west Hawai`i waste due to the decreasing unit rate for disposal at WHSL as tonnage increases per the existing agreement, to range from approximately $53 to $57/ton in 2011 dollars, or $67 to $71/ton if the savings on disposal of west county waste is not included (see Section 5). Therefore, the estimated cost differential between waste disposal adjacent to the SHSL and at the WHSL could range between $17/ton up to $77/ton ($70-$53 and $130-$53 respectively). In our opinion, taking into account the volatility of fuel prices and other pricing uncertainties, as well as uncertainties about the stability of the long term contracted cost to operate the WHSL, a reasonable estimate for the possible range of savings for hauling and landfilling eastside waste at WHSL, based on projected eastside tonnage generation of around 1,748,000 tons for the 20 year period 2013-2033, could be on the order of $20,000,000 to $30,000,000 in 2011 dollars, or between $1,000,000 and $1,500,000 per year.

Environmental and Socioeconomic Feasibility

Land Use Compatibility Feasibility

Areas that directly adjoin the properties identified for the landfill expansion include the Keaukaha Military Reservation (KMR), a County Parks and Recreation skeet range, unencumbered State land, active and former rock quarries, and Department of Hawaiian Homes (DHHL) land. Provided below is a discussion of the issues that affect the feasibility of the expansion.

The existing Skeet Range on 15 acres of State land within tax map key (TMK) 2-1-13:01 proposed for the leachate treatment area needs to be relocated at DEM expense. No costs estimates are available, but assuming there is State land that can be transferred to the County under the Governor's Executive Order (EO), the costs may exceed $750,000.

The County must fulfill or renegotiate the conditions for the cancellation of OE Nos. 3975, 2432 and 2841, and establishment of a Set Aside of State Lands to the County of Hawaiʻi of TMKs 2-1-13: 11, 1423, 150, 156, 162, 167, 168 and portions of road right-of-ways for solid waste, road and utility purposes. The County will need to acquire right-of-way (ROW) within property under its control either through existing property or future consolidation resubdivision. The proposed Set Aside is vital to rationalize land use and eliminate paper road ROWs that traverse the existing landfill, ensure legal access to properties beyond the landfill, and acquire ROW that avoids unauthorized use of DHHL land. Key to this are two 60-foot road ROWs, one of which is located with Manā Quarry and is not currently buildable because of topography. Although the solutions to these problems are not infeasible, they may be expensive and will require considerable investment in time and engineering.

The Hawaiʻi State Department of Defense (HDD) plans an expansion of the Keaukaha Military Reservation (KMR) for the Hawaiʻi Army National Guard, and the landfill expansion area is located down-range of the National Guard's pistol and rifle ranges. The associated safety zones for live-training at these ranges extend beyond KMR's southern boundary. HDD has stated that the leachate treatment area will impose undue impacts on its safety zone and asserts that although it has no easements, EOs, revocable permits, leases or other claims on the State land in this area, nor plans to obtain them, other uses should be prohibited. Although not infeasible, expansion into the area across KMR will require negotiation and probably conflict with HDD, which has expressed opposition to a new landfill for environmental reasons as well.

DHHL owns two parcels of land that directly border County solid waste operations and the Department, its East Hawaiʻi Commissioner and the beneficiaries have expressed severe concerns with impacts to these properties from a new landfill. In meetings of community associations, Hawaiian Homeland (HHL) beneficiaries have often raised the idea of income-generating options for this land, with some proposing a prison, value-added recycling/re-se facilities and even a private waste-to-energy plant for these or nearby DHHL properties. Although this does not make the landfill infeasible, the County Department of Environmental Management will need to invest considerable time in discussions with DHHL on effects to these properties.

Land Use Permitting Feasibility

The proposed expansion area and leachate treatment area lack proper land use designation, and the existing landfill is not in compliance with its Special Permit conditions. A new landfill will require a State Land Use Commission, State Land Use District Boundary Amendment (or County of Hawaiʻi Special Use Permit), a County General Plan Interpretation and/or Amendment, and a County Change of Zone. The County has attempted to address this situation several times and has determined that the cost is on the order of $500,000 and requires specialized legal services that can only be provided by County employees, but which cannot be accommodated within existing employee workloads.

Nuisance Issues and Environmental Justice Feasibility

Nuisances from the expanded landfill including noise, visual impacts, traffic, odors, landfill gases, trash, pests, are present but not significant for most of Hilo except for Panaewa because of distance. Panaewa has a very high proportion of Native Hawaiians because of the requirement for lessees to have “not less than one-half part of the blood of the races inhabiting the Hawaiian Islands previous to 1778,” and many Panaewa residents are also low-income. Landfill nuisance impacts are seen by many in the context of a long history of appropriation of Hawaiian land or use of lands adjacent to Hawaiian communities for airports, industrial areas, quarries, ports and solid waste uses. The pattern of such use in which a low-income and minority community disproportionately bears the adverse impacts of public infrastructure is a matter of environmental justice. This situation affects the feasibility of a landfill expansion because it is unknown whether this land use can be continued and expanded in a manner that is equitable for the affected community and avoids significant community opposition. In order to optimize the process, the County may consider convening a landfill planning committee composed of agency personnel, neighboring land managers, and community members to determine whether and how the SHSL can be expanded in an environmentally and socially acceptable way. Early and meaningful involvement in project planning is a cornerstone of genuine environmental justice efforts. Mitigation for nuisance impacts and community benefits would need to be considered. If a formal community benefit package is developed, the affected community should have a strong voice in its components.

Other Environmental Issues and Feasibility

No streams, wetlands or designated floodplains exist near the proposed expansion area. Endangered plant species are very unlikely to be present based on reconnaissance and surveys from nearby areas, and impacts to endangered animals can likely be avoided through seasonal landclearing restrictions and surface vegetation treatments. If treatment wetlands are used, they can be designed so as not to attract wetlands or other birds and to keep wild pigs out. The scavenger birds commonly involved in bird/aircraft collisions are not present at the SHSL, and therefore birds may not present a significant hazard to the Hilo International Airport operations. Because of the extensive history of surface disturbance in the expansion area, cultural resources including archaeological sites and gathering resources may not be present. Hydrological, biological and cultural constraints are not expected to have a substantial impact on the feasibility of an expanded landfill.

Conclusions

In our opinion, while it is technically feasible, it is neither practical nor economically sound to proceed with design and permitting a landfill expansion in Hilo. Permitting constraints, land use constraints, and leachate management issues all present significant and, perhaps, insurmountable obstacles. Furthermore, based on our planning level cost estimates, trucking and disposal of waste at the existing West Hawaii Sanitary Landfill provides a potentially feasible and more cost effective disposal alternative. It is recommended that the traffic impact assessment prepared for the FEIS for the East Hawai`i Regional Sort Station (February 2004) be updated with respect to trucking of waste to the WHSL.

Section 1
INTRODUCTION

1.1 Purpose and Scope

The County of Hawai'i (County) is interested in the feasibility of continuing to landfill municipal solid waste (MSW) on the eastern side of the island of Hawai'i adjacent to the South Hilo Sanitary Landfill (SHSL). The County retained R. W. Beck, Inc., an SAIC Company to conduct a feasibility study on expansion of the SHSL from the existing waste limits and progressively constructing waste cells within an existing quarry located adjacent to the unlined landfill.

The feasibility study (Study) evaluated the technological and economic feasibility of constructing a landfill expansion, expanding under the current United States Environmental Protection Agency (EPA) Subtitle D and Hawai'i Department of Health (DOH) regulations with landfill liner and leachate collection system and taking into consideration:

- The significant amount of rainfall in the Hilo, Hawai'i region;
- Lack of low permeability soils at or near the SHSL;
- Mitigation and management of landfill leachate; and
- Leachate treatment and disposal options.

The economic feasibility of expanding and continuing to operate the SHSL was compared to the option of transporting waste from the east side of the island and disposal in the West Hawai'i Sanitary Landfill (WHSL) located approximately 78 road miles west in the Kailua-Kona region of the island.

This Study included considerations of the environmental and socioeconomic feasibility and impacts that expanding the Landfill may have. Involvement and discussions with local environmental groups and agencies, community organizers, and other affected organizations were conducted to provide an opportunity for public input into the environmental and socioeconomic effects the expansion may have.

The Study is not intended to provide permit level documents or final design information for landfill expansion. Rather it will provide planning level information that will be useful for decision makers to assess the feasibility of expanding the SHSL versus hauling and disposal of MSW on the west side of the County at the WHSL. The Study is broken into seven sections:

Section 1 – Provides details of the purpose and scope of the Study, and current site historical information.

Section 2 – Provides a discussion on the technical aspects of constructing and operational considerations of the SHSL expansion.

Section 3 – Provides a discussion on leachate generation, characteristics and treatment options.

Section 4 – Provides a discussion on land use and development, permitting, environmental impacts, and socioeconomic considerations.

Section 5 – Provides an updated evaluation of the cost of hauling waste from the Hilo area to the WHSL and a planning level estimate of the total cost to landfill east county waste at the WHSL.

Section 6 – Provides a summarized assessment of the technical and financial feasibility of expanding the SHSL.

Section 7 – Provides a summary of the study and conclusions

1.2 Landfill Background and History

The SHSL is owned and operated by the County of Hawai‘i Department of Environmental Management (DEM). The current Landfill site is approximately 40 acres in size, the majority of which is used for landfilling waste (Figure 1 in Appendix A). Based on information provided by DEM, the Landfill has been in operation since at least the 1960s.

1.2.1 Site Location

The SHSL is located on the east side of the City of Hilo in an area of mixed industrial, agriculture and airport use. The landfill is approximately one mile east of Kanoelehua Avenue (State Highway 11) and approximately 1.6 miles south of the Hilo International Airport. Access to the landfill is via Leilani Street and an unnamed access road.

1.2.2 Adjacent Land Use

Land uses adjacent to the SHSL are as follows:

- Located east and northeast of the landfill is the Keaukaha Military Reservation (KMR) of the Air National Guard. The Hilo International Airport is immediately beyond the KMR, approximately 7,750 feet to the end of the nearest runway.
- North of the landfill is vacant land;
- Immediately northwest of the landfill is the green waste mulching site, scrap metal collection facility and the County’s Hilo Convenience Center.

- Adjacent to the landfill on the south are quarries, vacant land and further south approximately 1.3 miles is the Panaewa Drag Strip.
- West of the landfill are the Department of Hawaiian Home Lands (DHHL) Panaewa Farm Lots.

Section 4 of this Study includes more details on current and future land use status at the landfill and surrounding areas.

1.2.3 Historical and Projected Waste Volumes

The landfill accepts MSW from commercial and residential haulers as well as waste transported from nine of the County's 21 convenience centers. On average, 82 commercial vehicles enter the SHSL each day. The County also hauls more than 10 convenience center trailers to SHSL each day. In fiscal year 2007-2008 it was reported that 81,487 tons of waste were disposed of at SHSL. This represents nearly 40 percent of the total MSW disposed of within the County's two landfills for the year.

According to the most recent annual information, the County generated approximately 208,650 tons of waste in 2010. The total tonnage of waste disposed of at the SHSL and the WHSL in 2010 was approximately 166,450 of which 63,450 tons were disposed of at the SHSL. The current economic downturn has likely led to a large decrease in the generated waste amounts and is reflected in the lower tons disposed of at the SHSL.

As of May 2008, the landfill had approximately 910,000 cubic yards of airspace remaining for the current active area at the SHSL. (SWT Engineering, November 2008) At the time of developing this Study, an accurate airspace volume was unavailable; therefore, the 2008 estimate will be used and projected to 2011. The projected capacity numbers are calculated using the airspace utilization factor (AUF). The AUF is calculated using annual tonnage information for waste placed in the disposal area and corresponding annual survey data. It is a value used to identify the efficiency of waste placing operation and is affected by the moisture content of the waste, equipment used to place the waste, daily cover material type, ratio of waste to cover, and type of waste placed.

According to discussions with County staff, the AUF is likely around 1,200 pounds per cubic yard (lb/cy) and the estimated earliest year to reach current capacity is 2015. However, the remaining life of the current disposal area is likely to extend due to the potential for increased waste diversion. The County is exploring potential projects that will provide additional recycling or reuse programs that will divert waste from disposal in the Landfill, including diversion of organics from the waste stream. Therefore, it is difficult to accurately estimate the year capacity is reached and how any diversion program will lengthen the life of the Landfill.[1]

[1] On site operations show that there is more remaining airspace than originally estimated. There are also areas of settlement that allow for recovered airspace within the next 2 to 3 years.

In addition to the diversion programs, the County could see an increase in life expectancy due to additional variables. According to the EPA, the current economic downturn has resulted in decreased waste generation rates across the United States. This in turn reduces projected waste disposal rates for the immediate future; therefore, lowering the overall long term projections. In addition to lower waste disposal rates, the wet climate of Hilo is likely increasing the moisture content of the waste which may potentially result in an AUF that is higher than previously calculated.[2] It has been demonstrated that when landfills have increased moisture content either through elevated precipitation or recirculation of leachate, the AUF rates and the rate of waste settlement may increase over the next 2 to 3 years. This allows a landfill to recapture previously used airspace within 2 to 3 years of the last waste placement in that area.[3]

A waste generation model was created to project future SHSL capacities. The assumptions used for the model were derived from the County of Hawai‘i Integrated Resources and Solid Waste Management Plan Update, December 2009 (IRSWMP) and the Department of Business, Economic Development, and Tourism (DBEDT) Population and Economic Projections for the State of Hawai‘i to 2035, July 2009 (Revised). The forecasted waste generation tonnage rates have been updated to reflect current waste disposal information. The 2009 IRSWMP provided data showing the percent of the total waste disposed of at the SHSL and the WHSL. According to the information provided for 2000 through 2008, approximately 40 percent of the total disposed waste for the County is taken to the SHSL. The 2010 tonnage information reflects that this trend continues.[4]

The County has a relatively high tourist activity which results in generation of waste per population rates that are higher than average (U.S. Environmental Protection Agency, 2009). The waste projection model considers several different factors for forecasting waste disposal at the SHSL. The assumptions used from the 2009 IRSWMP have been updated to reflect current trends in the County and as experienced nationwide. Since publication of the IRSWMP report, key values have been updated including the 2010 population. The projected 2010 population in the IRSWMP was 176,750, and according to the 2010 United States Census results, the population of the County in 2010 was 185,079. In addition to the population number, the employment estimates have been updated. The projected employment for 2010 in IRSWMP was

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[2] AUF rates for landfills in the Midwestern United States with annual precipitation between 25 to 30 inches can typically achieve rates between 950 lbs/cy to 1,250 lbs/cy at Lyon County Sanitary Landfill in Minnesota and the Sioux Falls Sanitary Landfill in South Dakota, respectively.

[3] Crow Wing County Landfill, Minnesota has demonstrated an increase in AUF of 1,000 lbs/cy prior to increasing moisture content through recirculation to over 1,500 lbs/cy within 3 years after initiating recirculation. The landfill has also demonstrated accelerated settlement by increasing the moisture content of the waste through recirculation. Although, the SHSL does not recirculate, the high precipitation likely increases the moisture content to levels that lead to accelerated waste settlement.

[4] The total tons disposed of at the SHSL and the WHSL for fiscal year (FY) 2010 is 166,450 tons, of which 63,450 were disposed of at SHSL, or approximately 38.1 percent.

approximately 103,400[5] while the projected employment in the 2009 revised DBEDT was 101,149. This shows the employment numbers have been slightly reduced, which are likely the result of the current economic downturn.

Using the revised tonnage, population, and employment numbers the new waste generation forecast was modeled. The assumptions and results of the projection model are provided in Table 1 of Appendix B. The waste disposal projections are for years 2010 through 2035 are provided in Table 1-1.

Table 1-1

Year	Tons	Year	Tons
2010	63,445	2023	88,086
2011	64,088	2024	90,459
2012	65,620	2025	92,896
2013	67,189	2026	95,181
2014	68,795	2027	97,523
2015	70,440	2028	99,923
2016	72,495	2029	102,391
2017	74,609	2030	104,900
2018	76,786	2031	107,326
2019	79,027	2032	109,809
2020	81,334	2033	112,349
2021	83,525	2034	114,947
2022	85,775	2035	117,606

1.2.4 Daily Operations

Daily landfill operations consist of heavy equipment operators spreading incoming MSW in layers or lifts, up the slope of the working face. A daily working face of approximately 50 to 100 feet is maintained and a general slope of 5:1 or less to achieve waste compaction. State and federal solid waste operations regulations require daily cover of refuse to mitigate odors, vectors and wind-blown debris. Exposed waste is covered daily using processed quarry rock ("red rock") from the adjacent quarry activity.

1.3 Climate

Hawai‘i boasts 11 of the 13 climate zones in the world, each with unique ecosystems and weather characteristics. Factors such as elevation, pressure variations, rainfall, wind and topography combine to create distinctive locations throughout the islands.

[5] The IRSWMP notes an employment number of 97,738 for 2007 and a projected value of 107,100 for 2015. This equals a 1.15 percent annual growth which equates to approximately 103,400 employees for 2010.

Hilo is within the humid tropical climate zone. Average annual rainfall at the Hilo International Airport near the Landfill is approximately 126 inches (National Oceanic and Atmospheric Administration Weather Database, 2011). The average temperature ranges from 71 degrees Fahrenheit in January to 76 degrees Fahrenheit in August. Northeast tradewinds are generally steady in the summer months and a bit weaker and inconsistent in the winter. The prevailing winds in the Hilo area are generally from the northeast during daylight hours and southwesterly during the night.

1.4 Geology and Hydrogeology

1.4.1 Geology

The Island of Hawai‘i was formed by the coalescence of five shield volcanoes (from north to south: Kohala, Mauna Kea, Hualalai, Mauna Loa and Kilauea) that accreted into a broad dome extending from the Hawaiian Ridge along the floor of the Pacific Ocean. The Hawaiian Ridge formed as the result of the Pacific Plate passing over a “hot spot” that is fixed in the underlying mantle. As the plate moved progressively from southeast to northwest over the hot spot, volcanic activity has waned in the north and central portions of the island over time. During recent geologic time, only the southern portion of the island experiences active volcanism.

At the SHSL site, the near surface is underlain by recent (Holocene) lava flows belonging to the Kau Basalt of the Mauna Loa Volcano. The Kau Basalt is predominantly tholeitic and includes flows of a’a’ (clinker) and pahoehoe (ropy) interlayered with relatively unconsolidated cinders. Flows are typically five to 20 feet thick. As a result of its relatively young age, the Kau Basalt in the area of SHSL is relatively unweathered and has developed only a thin soil layer. These basalt layers are typically highly fractured resulting in high permeability.

1.4.2 Hydrogeology

Fresh groundwater originates as rainfall that infiltrates the ground surface, percolates downward to the water table and then flows seaward to discharge into the ocean in diffuse near-shore springs. Groundwater beneath the SHSL occurs in the fractures, vesicles and lava tubes in the flow basalts. Historical water level data from nine monitoring wells constructed at the SHSL indicate the potentiometric surface beneath the site ranges from six to seven feet above mean sea level (msl). Groundwater flow direction is from southwest to northeast at a gradient of approximately 0.00045 feet per foot (SWT Engineering, November 2008).

The SHSL, including the potential landfill expansion area is makai or seaward of the Underground Injection Control (UIC) line. This location makes it unlikely that municipal supply wells will be developed in the vicinity.

Section 2
PROPOSED LANDFILL EXPANSION

2.1 Lateral Expansion

The County is evaluating the feasibility of expanding the current Landfill in the area of the quarry south of the Landfill (See Figure 2 in Appendix A). It is assumed that the County would apply to permit the expansion as a lateral expansion according to the definitions in the Hawai‘i Administrative Rules (HAR) Title 11 DOH Chapter 58.1 (HAR-11-58.1). The area depicted shows an expansion of approximately 60 acres of quarry footprint, of which, approximately 45 could be obtained for waste disposal use. The waste disposal estimates generated as part of Section 1 of this Study were used to estimate the airspace required per cell construction. The proposed area would be located adjacent to the current disposal area and separated by the scale house and access road. This would allow the County to continue to use the scale facilities in its current location and to eliminate additional excavation to create a contiguous disposal area.[6] At the time of permitting, the ultimate expansion area of the Landfill may include the area between the current disposal area and the proposed expansion location. Additionally, if the expansion area is constructed, the sort station facility and associated scale would likely be utilized to weigh incoming trucks and transfer the waste to trailers for disposal. However, for purposes of this study, the area as proposed in Figure 2 will be evaluated.

It may be difficult at the time of construction of the first cells of the expansion area to include the area between the proposed expansion and current disposal area because of the Subtitle D requirements and the installation of a bottom liner system. Generally, landfill expansions include a “piggy-back” over the existing waste in which the new disposal area includes placing waste over the existing, adjacent slope. However, these scenarios usually have the existing waste over a lined cell. Since the existing waste at SHSL does not include a liner, this would require a liner to be installed on the existing southerly slope. This may not be practical due to the nature of differential settlement of the waste in the existing disposal area. Differential settlement may create stresses on the liner system and could ultimately lead to tears and leaks in the liner system. The existing south slope may need to have a surcharge placed on it to reduce the likelihood of future differential settlement. At some point in the future, waste could be filled between the existing disposal area and the expansion area.

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[6] Non-contiguous disposal areas have been constructed at other landfills in the United States while still being considered a lateral expansion under the same permit. The Sioux Falls Sanitary Landfill in Sioux Falls, South Dakota has two separate disposal areas separated by an intermediate access road. Although these areas require separation of collected leachate, they operate under one permit.

One of the key factors for establishing landfill development is to minimize the amount of leachate generation. The size of the landfill footprint is an important consideration for evaluating the amount of leachate generation because of the high precipitation in Hilo. There are design and construction considerations and operational practices that can be implemented to minimize precipitation infiltrating the waste and thereby minimizing leachate generation. However, any methods to reduce infiltration could result in an increase in surface water generation. Therefore, consideration should also be given to managing the storm water runoff from the landfill surface. Further information is presented in Section 2.3.

Presented in Figure 3 in Appendix A is a preliminary layout of the Landfill expansion cell development. With the exception of Cell 1 at seven acres, Cells 2 through 8 would be less than five acres. The total developed footprint for waste disposal is approximately 40 acres, with five acres retained for staging of operations and infiltration basins.

The size of each conceptual landfill cell provides approximately four to five years of airspace capacity. This allows for the cell to accept waste for three to four years before the next cell design would begin. The life of each cell was calculated by using the projected waste disposal numbers estimated in Section 1 of this Study. As proposed, Cell 1 is slightly larger than the remaining seven cells, and is estimated to provide at least six years of capacity. The estimates assume 20 percent use of capacity is for cover soils. The overall airspace capacity for the 40 acre footprint is approximately 5.4 million cubic yards (cy) using the general design described above.

The cost of design and construction versus the cost and requirements to operate and manage the landfill and leachate are two scenarios that should to be considered. Because construction costs are generally higher in Hawaiʻi relative to the mainland United States, constructing a larger footprint or two cells at once would potentially provide savings on construction over the long term. Annual operational costs will depend on the equipment used at the Landfill and the depth and level of sophistication of management facilities.[7] General cost estimates for design and construction are provided as part of this Study (Section 2.5) based on the conceptual layouts provided in Figure 3 and described herein. Annual operation cost estimates are also provided in Section 2.5.

Currently, in the location of the potential expansion cells are active quarries. The quarry area provides a relatively good starting point for the base of the landfill. However, the nature of the volcanic rock in the proposed expansion area would require significant effort and cost to excavate and create slope embankments. Specialized heavy equipment would be required to complete much of the earth moving work. In order to shape the side walls and side slopes, the equipment required include; large excavators with hydraulic hammers and bull dozers with ripper attachments. The lowest points in the quarried area are approximately 20 feet above mean sea level

[7] Management facilities include equipment for managing leachate, such as the wetlands evaluated as part of this Study, the day to day requirements for minimizing storm water infiltration in the waste, and potential landfill gas control equipment.

(msl). However, portions of the base are at 30 to 40 feet above msl. As a way to minimize additional excavation, filling, and to keep a separation from the ground water table, the base of the landfill would be approximately 25 feet above msl. The perimeter elevation (natural topography) is approximately 90 feet above msl. This equates to a 65-foot deep landfill.

The outline of the expansion area is generally located along the outer edges of the quarried area. Some of the quarry side walls are vertical, and to make it more suitable for waste placement, the perimeter will be reshaped to include internal sideslope embankments at a slope of 3 to 1. This would require the placement of fill material in some areas to reshape the internal side slopes. On-site material would be utilized to obtain the side slope requirements. However, the upper three to five feet of the side slope would require a small aggregate well graded material to ensure a nicely graded side slope. In addition, soil material stability calculations would be required to determine the material specifications.

The current disposal area would have a peak final elevation of equal to that of the existing landfill, nearly 100 feet higher than the surrounding topography. As is typically standard for landfill closure, the side slope covers were evaluated using a 4 to 1 ratio with the crown having a maximum seven-percent slope.

Using the assumption that waste diversion rates will continue as they currently do, Table 2-1 provides an estimated airspace breakdown for Cells 1 through 8 with an airspace utilization factor (AUF) at 1,200 pounds per cubic yard (lb/cy) for 2016 to 2046. The table below assumes that the current disposal area reaches capacity in 2016. As noted above, Cell 1 would provide at least six years of capacity. Any waste diversion program or increase in AUF would likely extend the life of the current disposal area and any future disposal area development.

Table 2-1

Year	Annual Airspace Consumed (cy)	Airspace Remaining	Cell Opens
2016	120,824	5,400,000	Cell 1
2017	124,349	5,279,176	
2018	127,977	5,154,827	
2019	131,712	5,026,849	
2020	135,557	4,895,137	
2021	139,208	4,759,579	
2022	142,958	4,620,371	Cell 2
2023	146,809	4,477,413	
2024	150,764	4,330,604	
2025	154,827	4,179,839	
2026	158,636	4,025,013	Cell 3
2027	162,539	3,866,377	
2028	166,538	3,703,838	
2029	170,635	3,537,300	
2030	174,833	3,366,665	
2031	178,877	3,191,832	Cell 4
2032	183,015	3,012,954	
2033	187,248	2,829,940	
2034	191,579	2,642,692	
2035	196,010	2,451,113	Cell 5
2036	197,970	2,255,102	
2037	199,950	2,057,132	
2038	201,950	1,857,182	
2039	203,969	1,655,232	Cell 6
2040	206,009	1,451,263	
2041	208,069	1,245,255	
2042	210,149	1,037,186	
2043	212,251	827,036	Cell 7
2044	214,373	614,785	
2045	216,517	400,412	Cell 8
2046	218,682	183,895	
2047		-34,788	

Site Upgrades

Regardless of the expansion of the disposal area, the Landfill would require site and facility upgrades. The incoming access road from the sort station location and the scale house should be upgraded to accommodate traffic to the Landfill and provide a through way for mining traffic and traffic to the drag strip located south of the Landfill. Most of the Landfill's perimeter access roads near the quarry area would need to be further evaluated to determine the level of upgrade required to accommodate mining and waste traffic. Section 4 provides additional information regarding current road uses and potential road improvements.

Facility upgrades for leachate treatment would depend on the leachate treatment system to be implemented. At a minimum, the expansion of the Landfill would require the installation of a leachate collection and transfer system. This system removes the leachate from the expansion area and transfers it to the treatment facility. Regardless of the treatment option that is implemented, the Landfill may need to install a storage system, usually consisting of storage ponds or large storage tanks. As part of the storage system, the Landfill may also consider providing a load out facility that would be utilized in the event the treatment facilities are under extended maintenance or otherwise unavailable for an extended period of time.

The upgrades to the leachate management system could potentially require the Landfill's electrical supply to be upgraded from a single-phase power supply to a three-phase power supply. Currently, there is a three-phase power supply installed up to the scrap metal recycling area. In order to accommodate the equipment (pumps, lighting, control equipment) that would be installed during the first cell construction, the three-phase power lines would need to be extended approximately two-thirds of a mile. It may be possible to utilize the existing single-phase power supply near the sort station and extend that line in the first few years. However, as the Landfill expands and more equipment is installed, the demand may be such that the power lines need to be upgraded.

2.2 Landfill Design

The current waste disposal area at SHSL began operation before Subtitle D requirements were implemented in 1991. Therefore, any new horizontal expansion of the Landfill would require the County to submit a permit application in which the proposed expansion meets the design requirements of the Subtitle D Rules. The major components of a Subtitle D landfill include the landfill's liner system, the leachate collection system, and the final cover system.

The liner system for the expansion would be required to conform to the composite liner system prescribed in the HAR-11-58.1. This consists of a compacted sub-base of onsite soils, a two-foot layer of compacted low permeable soil (hydraulic conductivity less than $1x10^{-7}$ centimeters/second or cm/sec), and flexible membrane liner (hydraulic conductivity less than $1x10^{-7}$ cm/sec).

In general, a landfill's cover system should have a permeability less than or equal to that of the bottom liner system. This typically consists of a 12 inch final layer of soil placed directly on top of the waste, a flexible membrane liner (typically a 40 mil low linear density polyethylene), and 24 inches of soil with the top six inches capable of supporting vegetation. Alternative cover systems can be utilized, including the use of soils without flexible membrane liners or evapotranspiration systems. However, for this Study, the typical cover system noted above was evaluated.

As noted in the scope for the Study, leachate management is a critical issue for the SHSL. The controlling requirement for leachate management is that a landfill must maintain a depth of leachate over the liner of less than 12 inches.

2.3 Landfill Liner, Cover and Leachate Collection Requirements

The design requirements prescribed from Subtitle D are the basis for the evaluation of the Landfill expansion. Complete technical analysis of the materials used in the expansion design would be required at the time of permitting.

Due to the uneven nature of the quarry base and the vertical quarry sidewalls, some onsite processed material would be required to make a more suitable landfill base. The compacted low permeable soil is generally constructed using a clay material. However, the native soils onsite do not include this low permeable type of material. There is a potential local low permeable soil source located northwest of Hilo. However, this material would require additives to make is suitable for use as a low permeable soil. Additional information is presented in Section 2.4.2.

An alternative to using a two-foot thick low permeable soil liner would be the use of a geo-synthetic clay liner (GCL). A GCL consists of a layer of sodium bentonite encased between two layers of geotextile material. The two layers are generally stitched together and manufactured to provide shear strength support for placement along sideslopes. The WHSL was constructed using a GCL, likely due to the lack of low permeable soils on the island. However, for use at the SHSL, testing would be required to demonstrate that the use of only the GCL is equivalent to the low permeable soil layer would be required during the permitting process and final approval for use would be determined by the DOH.

The second part of the composite liner system is the flexible membrane liner. Typically, the flexible membrane liner consists of a textured 60 mil High Density Polyethylene (HDPE) membrane.

Typical leachate collection systems consist of a network of perforated collection pipes that allow leachate to flow to a collection sump within the landfill or to a wet-well outside the landfill. Surrounding the collection piping are two layers of gravel filters that minimize sediment infiltration into the piping that may lead to restricted liquid flow. The piping and pump sizes would be designed according to the anticipated leachate generation volumes at the maximum rate as determined by computer modeling. To enhance leachate flow through the base drainage layers to the collection piping, a geocomposite drain net would be installed. The drain net consists of a HDPE grid encased between two layers of geotextiles with a void space to allow liquid movement. As part of the collection system, a 12 inch layer of permeable soil (greater than $1x10^{-3}$ cm/sec) would be installed above the geocomposite drain net. This layer allows leachate to flow to the collection trenches. This layer would also serve as a protective layer over the membrane liner during the placement of the first lift of waste.

This layer is typically constructed from sand. Since there is a lack of sand sources immediately in the area, processed aggregate from the quarry could be utilized.

The conceptual layout in Figure 3 shows leachate collected along the eastern perimeter of the Landfill expansion area. Leachate would be collected in sumps located within the landfill and pumped up to the perimeter of the landfill. It would then be piped to a gravity system that drains to the treatment facilities located to the northeast of the expansion area.

The composite liner system evaluated uses processed quarry material to bring the subgrade to the required elevation, a GCL, 60 mil HDPE membrane liner, a geocomposite drain net, and a one-foot layer of processed quarry material for drainage.

2.4 Geotechnical Assessment

As part of the initial data collection, HWA Geosciences, Inc. (HWA) completed a preliminary evaluation of the onsite soils at the Landfill and a potential borrow source for construction materials. The findings from the geotechnical assessment of the local soil are provided in Appendix C. Additionally, HWA reviewed a potential source of low permeable borrow soil located northwest of Hilo.

The location of the Landfill is part of a seismically active region due to volcanism (SWT Engineering, November 2008). Because of this, the expansion permitting would require additional seismic analysis and consideration to meet the seismic requirements of Subtitle D and the requirements of HAR-11-58.1-13.

Any proposed expansion that requires an environmental assessment as part of the permitting process would need to include an investigation on the stability of the existing land and soil (HAR-11-58.1-13). Much of the geologic conditions can be seen from the existing quarry area and there are no man-made features that could potentially result in landfill instability (SWT Engineering, November 2008).

An evaluation of the potential for tsunami impacts note the current and proposed expansion area are well beyond the tsunami inundation zone (SWT Engineering, November 2008).

2.4.1 Landfill Expansion Site Preparation

The expansion area will require some subgrade preparation work. Because the area has been quarried and a large portion is 60 to 70 feet deeper than the surrounding topography, the expansion will not require significant excavation. However, depending on how deep the base of the landfill is, some soil material would need to be placed and compacted. For the evaluation of this Study, the lowest portion of the landfill base is approximately 25 feet above msl, with the lowest portions of the quarry at approximately 20 feet above msl. Material from the quarry would be used to obtain the required elevations. Ultimately, the base grades of the landfill would be determined at the time of permitting.

The placement of soil embankments along the quarry side walls to create the interior landfill side slopes would likely be the most practical option for landfill construction. These areas would require fill soil placement and significant compaction requirements to obtain the stable 3 to 1 side slopes. Alternatives to this method exist including excavating a portion of the quarry sidewall or installing a vertical liner system in portions of the quarry.

Excavating the quarry side walls to obtain the 3 to 1 side slopes would require a much larger landfill footprint and additional consideration be given to the level of effort for excavating in the native rock. Installation of the composite liner system vertically on a portion of the quarry side walls requires the low permeable layer. This may be difficult because of the lack of low permeable soils on site that could be used to create this layer, and as noted above, the use of a GCL would likely be required. Installation of a GCL vertically would not be likely because the material is not specifically design to be installed along a vertical face, and it would be likely that the bentonite material will would accumulate at the base because of gravity. A combination of the options could also be considered. For purposes of this study, it was assumed that the internal area of the quarry would require fill material to bring up to 3 to 1 side slopes and that the amount of excavation and filling would be equal.

2.4.2 Local Soil Borrow

The potential borrow source located northwest of Hilo was evaluated for its use as the low permeable layer of the composite liner system and a low permeable cover option. The detailed assessment is provided in Appendix C. The results from the analysis indicate the soil is less than ideal for use in construction. This material consists of a regional volcanic ash deposit, the Pahala Ash. It can become very difficult to place when the soil becomes wet. However, it may be possible to use the soil as the low permeable liner layer if the soil is amended using cement, lime, or some other agent. Further testing would be required to determine if using soil amendments would provide a usable product. Other on Island sources may be available, but at a greater distance for hauling to the Landfill, which may add to costs associated with permitting, excavation, hauling and potential mitigation of local environmental impacts.

2.5 Landfill Operations

The Subtitle D expansion would include new site operations management that does not currently exist. The expansion would require new equipment be installed, including pumps and piping, and vault and manhole structures for management leachate. In addition, new monitoring plans would need to be developed for ensuring an optimal and efficient system. Additional care and effort would be needed during waste placement to minimize leachate generation. The implementation of the lined landfill, leachate management system, leachate treatment system, and additional systems (landfill gas) would likely require additional training to existing and new Landfill staff.

Waste placing methods would likely not be much different from the current methods used for the landfill, with the exception of the trailers from the Sort Station delivering the waste. However, a specific phasing plan may need to be implemented to ensure coordination of cover material placement and working face activities. This effort would be implemented to limit precipitation into the waste and to control the surface water runoff.

2.5.1 Leachate Management

The phasing and method of placing the waste may impact leachate generation. One of the main operational concerns would be to limit precipitation infiltration into the waste. This may be accomplished through design and construction of the lined disposal cell, operational methods such as sheeting or the use of a rain coat, or a combination as described below.

Typically, landfill cells are designed to drain all precipitation and other liquids that fall within the lined footprint to the sump located at one end of the cell. The high rate of precipitation in the Hilo area would result in significant leachate generation from a footprint of five to seven acres. A practical method to minimize precipitation from contacting the waste would be to deploy a lightweight geosynthetic liner (raincoat) above the buffer layer after the completion of the cell construction. The raincoat would serve as a barrier layer to minimize precipitation infiltration and mixing with leachate that is collected through the leachate collection underdrain system. Typically, the raincoat is “shingled” so each sheet may be easily removed.

Initial waste placing would begin at the end opposite of the low point or sump. This would allow surface water to flow away from the working face and collect at the sump location above the raincoat. As waste is placed and the working face moves toward the low point, the raincoat would be “peeled” back. The areas of waste that reach a set height for each lift will be capped with a temporary geomembrane similar to that of the raincoat to again minimize infiltration. Careful planning during the design and construction will be needed to ensure that the storm water infiltration is minimized. In addition, the day to day operations would be critical to maintaining the integrity of the cover and diversion systems.

Any leachate generated will need to be properly managed to maintain the head level below 12 inches. Estimates for leachate generation are approximately 5,300 gallons per day per acre. More discussion regarding leachate generation is provided in Section 3.2. As noted above, typical cell designs direct leachate collection to a low point sump through the leachate collection underdrain system of the collection piping and geocomposite drain net. Generally the pipe size can range from six-inch diameter to 14-inch diameter and would be designed and sized at the time of permitting. There are generally two options to remove leachate from a landfill in order to maintain the required less than 12-inch leachate level.

The first option would be to install a leachate pipe that drains by gravity to a wet well pump station located outside of the landfill boundary. This set up allows for easier maintenance as the equipment is located outside of the landfill. However, this option would require a penetration through the liner and side slope of the landfill and may potentially increase the likelihood of leachate leaks. Also, in order for the system to drain by gravity, the wet well would need to be deeper than the base of the landfill which would likely increase the cost of construction because of the depth required and the nature of the underlying rock material on site. Generally, favorable site topography is needed for this option.

The second method would be to install a dewatering pump within the landfill located at the low point. The pump would be placed in a side slope riser (SSR) pipe at the sump location above the liner. The SSR pipe is a large diameter (24-inches or larger depending on the size of the pump) that contains the pump and the discharge piping. This SSR pipe brings the leachate up to the top of the slope where it would then gravity drain or be pumped via forcemain to the leachate treatment facility. The maintenance for this type of dewatering system is more labor intensive and would be impacted by the size of the pump as it requires the pump to be pulled up the slope. In this arrangement, the pump would be contained within the liner system and likely minimize the chance for leaks because there are no penetrations required.

For purposes of this Study, the pump would be contained within the landfill and would use the SSR pipe. Leachate would gravity drain to lined manholes located along the eastern perimeter as shown in Figure 3. The leachate collection and transfer piping would be cleaned on an annual basis as part of the Landfill's operations program.

As noted above, the raincoat system or other surface water diversion methods would be implemented to minimize infiltration of precipitation into the waste mass. This would create a scenario where additional surface water controls may be required to handle the increase volumes of surface water runoff. Therefore, additional considerations would need to be given to the management of the surface water. Table 2 in Appendix D presents a scenario in which only 25 percent of the waste mass is covered with a raincoat at any given time. Surface water runoff estimates from a 25-year storm event show a peak flow of nearly 1300 gallons per minute per acre.

Management methods of the surface water would likely vary depending on the location and height of the waste mass. The topography of the proposed expansion area would allow for the potential of infiltration basins to be constructed adjacent to the disposal area. The infiltration basins would be sized to handle the surface water runoff from the disposal area, assuming the runoff has not come into contact with the waste. The infiltration basins would be installed on a temporary basis and would move locations to accommodate specific needs. Once the waste mass breaches the surrounding topography elevation, diversion of surface water would likely be to infiltration basins outside of the quarry.

To control the surface water diversion, berms or other temporary control structures would likely need to be installed within and around the disposal area. The temporary control structures would likely move as the waste mass moves. If a raincoat system is used, there would likely be locations of ponding water. A portable, high capacity pump or series of pumps could be utilized to transfer storm water from pond areas to infiltration basins.

2.5.2 Additional Considerations

Landfill Gas

The large volumes of rainfall would likely increase the moisture content of the waste even if infiltration is limited to the working face area. The increased moisture content would have an impact on compaction rates, settlement, and landfill gas (LFG) generation. Increased moisture content has been shown to accelerate LFG generation, whether the increased moisture content is natural through precipitation or through recirculation of leachate (U.S. Environmental Protection Agency, 2011)

There are several concerns regarding LFG generation, including migration to structures on or offsite, impacts to surrounding vegetation, odors and perceived health risks, and greenhouse gas emissions. The current disposal area provides adequate monitoring probes to demonstrate there has been no offsite migration or to structures on the property (SWT Engineering, November 2008). It is likely that with the proposed landfill expansion, additional gas monitoring probes would be required.

The lined expansion area would act as a barrier for LFG, thereby reducing the potential for offsite migration. The use of the SSR would also help to minimize potential migration as there will be no penetrations through the liner as noted previously. With a liner in place, LFG would tend to migrate through the side slopes and top of the waste. This may have a potential impact on localized odors and efforts to minimize these odors may be required. If the waste cover methods used are insufficient and LFG or waste odors were to increase to the point where mitigation is needed, landfills have implemented programs to neutralize odors. Some of these methods include an active LFG collection and control system (GCCS) or installing odor neutralizing equipment which may include deodorizing misters (Benzaco Scientific, 2011). Because of the potential environmental and health concerns that may arise due to odor or LFG issues, the County would likely need to be proactive in its efforts to mitigate any potential odor or LFG migration.

Landfill Gas Collection and Control System

Under the New Source Performance Standards (NSPS) of the Clean Air Act (CAA), landfills meeting certain criteria must evaluate their non-methane organic compounds (NMOC) emissions (U.S. Environmental Protection Agency, 2011). If all the criteria are exceeded, the landfill will be required to install an active LFG collection and control system (GCCS).

The first criterion (Tier 1) requires landfills to demonstrate they have a design capacity below 2.5 million cubic meters (or 3.27 million cy) and have less than 2.5 million megagrams (Mg) (or 2.76 million tons) of waste in place. The design capacity is typically referenced as the allowed permit capacity issued by the state regulatory agency. A permitted design capacity was not available for the SHSL, so an estimate of the waste in place for SHSL was performed, resulting in a 2008 waste-in-place of approximately 3.5 million cy. This estimate is above the 3.27 million cy noted above. A landfill has the option to calculate the design capacity on either a mass or volume basis. This may be accomplished by calculating the amount of tons in place by tonnage records or by converting the volume to tons with a calculated density. Using an estimated density of 1,200 lb/cy (SWT Engineering, November 2008) the waste in place is approximately 2.1 million tons (or 1.9 million Mg) which is below the Tier 1 calculation threshold. The County may want to consult its permit to evaluate the design airspace capacity of the current disposal area relative to the threshold limits noted above.

If the design capacity or tons exceed the threshold, the USEPA Landfill Gas Emission Model (LandGEM) may be used to estimate NMOC emissions (detailed in the Tier 2 requirements). The NMOC emission threshold is 50 Mg per year. The LandGEM model estimates the year this value will be exceeded. The year in which exceedance occurs according to the LandGEM, the landfill must complete field testing of NMOC emissions. NMOC emissions vary based on a number of variables including the type of waste and the moisture content of the waste. Tier 2 testing provides “real” data to re-calculate the model.

The results from the updated model will determine the frequency of field testing and may result in a different year in which 50 Mg NMOC emissions are exceeded. The year the Tier 2 revised model indicates exceedance of the 50 Mg threshold (Tier 3) additional field testing may be performed. This includes obtaining additional field data to more accurately calculate the model. The Tier 3 revised model will generate an updated estimate of if or when NMOC emissions will exceed 50 Mg. If the Tier 3 models indicates an exceedance of 50 Mg per year, then installation of a GCCS would be required within 30 months of the discovery of NMOC threshold exceedence. It should be note that Tier 3 testing requirements are very cumbersome and often times may be cost prohibitive. Once Tier 2 testing indicates the 50 Mg NMOC threshold is exceeded, the facility typically initiate the GCCS installation requirements rather than pursue Tier 3 testing.

For purposes of this study, it will be assumed that current approximate waste in place as of 2008 is 1.9 million Mg (SWT Engineering, November 2008).[8] The estimated tons generated per year indicate the landfill will reach 2.5 Mg of waste in place in approximately 2016. The LandGEM modeling results would likely require the landfill to complete actual field testing of NMOC emissions. It is a reasonable assumption that prior to the landfill reaching final closure of the entire expansion area, a GCCS will be required. This is because the current disposal area is at or near the Tier 1 threshold, and the entire expansion area build-out will more than double the amount of waste in place. This Study does not include a detailed evaluation. A general design, construction, and operating cost estimate is provided in Section 2.6.

The landfill permit application process will require a discussion on the landfill’s applicability to a Title V air permit under the CAA. If the design capacity exceeds the 2.5 million cubic meters, the landfill will be required to apply for a Title V air emissions permit through the State of Hawai‘i. The landfill would be required to submit an application for the Title V permit within the required timeframe upon

[8] There were 3.5 million cubic yards of waste in place as of May 2008. If this is converted to Mg there are approximately 1.9 million Mg or 2.1 million tons of waste in place.

construction commencement of the cell that will exceed the 2.5 million cubic yard threshold.

A GCCS is a proven method for minimizing potential odors, LFG emissions, and potential LFG migration. Installation of a GCCS either voluntarily or due to regulatory requirements can also provide the potential for a reuse project.

2.6 Cost Estimate – Design, Construction, Operation, and Closure

Appendix G provides a general cost estimate in 2011 dollars for the design, construction, and operation of the SHSL expansion. The estimate provided is intended to offer a general idea of the level of cost and considerations that will need to be addressed for the expansion of the landfill. The cost to construct the initial expansion will be more expensive than the successive construction because of the general infrastructure upgrade requirements. To design and construct the entire 7-acre Cell 1, the estimated cost is approximately $9.97 million (M). Subsequent cell construction will vary in cost depending on the specific needs at the time. In general, the future cell constructions will cost approximately $1.3M per acre.

The expansion will require upgrades to the monitoring and management plans. Currently, there is monitoring of the groundwater and LFG. However, the monitoring plans will likely require adjustments and updates which may increase the annual cost associated with these two monitoring programs. In addition to these costs, additional annual operating costs will need to include equipment (heavy equipment, pumps, etc.) maintenance, cover material (raincoat), leachate collection system cleaning, leachate treatment facility, electrical costs, and a contingency for miscellaneous site work. In addition there will be the cost to operate and maintain a GCCS should one be installed. The estimated annual operating cost including a GCCS is approximately $2.86 M.

The cost for capping and closing the landfill is estimated to be $18M and the cost to construct the GCCS over the entire 40-acre disposal area is approximately $4.7M. The annual operating costs for the post closure period is approximately $240,000.

3.1 Leachate Management

Leachate is any liquid, which in passing through matter, extracts solutes, suspended solids or any other component of the material through which it has passed.[9] Within a solid waste landfill, leachate is produced from liquid, generally precipitation, percolating through the deposited waste. Once in contact with the decomposing waste mass, chemical and biological reactions occur. Besides chemical and biological reactions, physical processes such as sorption and dissolution also occur as water passes through the waste mass. The sum total of all these processes and reactions is a liquid termed leachate. Subtitle D regulations regarding the construction and operations of MSW landfills require the collection, treatment and proper disposal of landfill leachate. Modern landfills are constructed with liner and leachate collection systems to mitigate the infiltration of leachate into the subsurface and groundwater.

3.2 Estimated Leachate Generation Volumes and Characteristics

3.2.1 Leachate Volumes

This Study performed a preliminary evaluation of leachate generation using the Hydrologic Evaluation Landfill Performance (HELP) Model Version 3.07. The analysis performed for this Study is a cursory evaluation to estimate leachate generation for developing a leachate treatment system. A more thorough analysis would be required for permitting. The HELP model was developed by the U.S. Army Engineer Waterways Experiment Station for the EPA and has been in use since 1984.

The HELP Model is a quasi-two-dimensional hydrogeologic water balance model developed specifically to perform hazardous and municipal waste landfill evaluations. The model requires weather, soil, and design data that are representative of the landfill location and design. It utilizes solution techniques that account for the effects of surface storage, snow melt, runoff, infiltration, evapotranspiration, vegetative growth, soil moisture storage, lateral subsurface drainage, leachate recirculation, unsaturated vertical drainage, and leakage through soil, geomembrane or composite liners.

[9] HAR-11-58.1-2: "Leachate" means water or other liquid that has percolated or passed through or emerged from solid waste and contains dissolved, soluble, suspended, or miscible materials removed from the waste or due to contact with solid waste or gases there from.

The leachate generation is at its highest during the first one to two years of a new landfill cell operation. Prior to waste placement, precipitation that falls within the lined footprint may be treated as stormwater. Once waste is placed in the landfill, precipitation that falls within the lined footprint and comes into contact with the waste must be treated as leachate. Additional lifts of waste generally result in lower volumes of leachate generation because the waste mass will absorb a portion of the liquid. The HELP model demonstrates the different stages of waste placement and the resulting leachate volumes. The results for the HELP model are discussed in Section 2.5.1 and provided in Appendix D.

The HELP model is a tool that is used for estimating the water balance within a landfill system. The HELP model was originally designed to provide an estimate of the leakage through a landfill's liner system. As a part of the overall water balance calculation, an additional output of estimated leachate generation is provided. The accuracy of the outputs depends upon the inputs used including the soil and waste properties, climatological data, and waste filling operations. The modeling for this Study assumed site data and the installation of a GCL. Site specific conditions that would be variable through the life of the landfill such as the initial moisture content and depth of the waste can impact the leachate generation quantities. The results for leachate generation from this Study provide an estimate only, and further and more detailed analysis for sizing of the leachate system would be needed at the time of permitting.

Generally, waste cells are constructed to provide enough volume for five to ten years of waste disposal life. This is typically based on the cost to construct the waste cell, the volume of airspace available, the ability to properly manage the open area and the average annual airspace volume used for disposal. Using these guidelines, an estimated waste cell area for the SHSL would be seven (7) acres (Cell 1). The amount of precipitation that can be diverted and managed as surface water runoff will impact the amount of leachate generation.

The SHSL is located in a region of relatively high annual rainfall. Precipitation in the Hilo, Hawaiʻi area averages 126 inches of rainfall per year. Since a leachate collection system would collect all liquid falling within the lined 7-acre area, the estimated volume of annual leachate generation would be approximately 24.0 million gallons per year or approximately 66,000 gallons per day (gpd). However, some moisture would be lost to evaporation and held within the waste mass. The average moisture content of MSW delivered to the landfill is 20 to 25 percent with field capacity for MSW up to 60 percent; therefore, the waste mass will retain some liquid. However, for this feasibility study, a leachate generation volume of 66,000 gpd is a conservative estimate and results from the HELP Model will be utilized. A leachate generation value used for analysis of the treatment options is approximately 5,300 gpd per acre. The actual volume will depend on the area exposed to infiltration. Over the seven-acre Cell 1 this could be as high as 40,000 gpd.

The Hilo area receives rain on an almost daily basis (average 278 days per year) and average monthly rainfall ranges from 7.36 inches in June to 15.58 inches in November (National Oceanic and Atmospheric Administration Weather Database, 2011). Therefore, there will be some variations in the volume of leachate produced dependent on: intensity of rainfall events; volume of waste mass; open area; and operation practices including daily and temporary cover and cover type.

3.2.2 Leachate Characteristics

Leachate will collect a variety of dissolved organic and inorganic contaminants resulting from the dissolution or degradation of MSW. The characteristics of leachate will vary over time and characteristics will change with the composition of the waste, age and degree of compaction. In generally, leachate from MSW landfills has similar signatures with respect to organic and inorganic compounds detected in the liquid. Chemical composition can be skewed dependent on the waste stream and the volume and type of industrial waste disposed of within a specific landfill, but in general, similar compounds will consistently be detected within leachate from MSW landfills. The concentrations of the chemicals detected will vary dependent on age of the landfill, amount of annual precipitation and if the landfill recirculates leachate or is a bioreactor landfill.

Raw leachate from MSW landfills contains varying concentrations of organic and inorganic dissolved constituents. This may include petroleum related volatile organic chemicals (VOCs) such as benzene, toluene, ethylbenzene and xylene and chlorinated VOCs such chloroethanes, chlorethenes, tetrahydrofuran and vinyl chloride. Inorganic chemicals found in MSW landfill leachate are typically dissolved metals including, arsenic, barium, boron, cadmium, copper, chromium, lead, mercury, and zinc to name a few of those more commonly detected. Biological oxygen demand (BOD) and chemical oxygen demand (COD) along with ammonia nitrogen are also common chemical concerns with leachate and leachate treatment.

Leachate characteristics for this Study were developed based on a review of the SHSL leachate monitoring results and CH2MHill data collected from Unalaska, Alaska Landfill presented in the SWT report and leachate data reviewed for Oregon landfills located in wet climates and leachate data collected from Minnesota landfills. A summary of leachate parameters from this review is presented in Appendix E and incorporated into the wetland treatment design presented in Section 3.3.6. This summary focuses primarily on inorganic parameters within leachate. Organic compounds, although of concern within landfill leachate, can be more easily treated and removed.

Typically, collected leachate is stored in aerated ponds and/or tanks. Introduction of air (oxygen) into the leachate volatizes the organic compounds into the atmosphere and reduces the concentrations within the leachate. Exposure to UV light will also assist in the degradation and reduction of VOCs; therefore, storage of leachate in aerated ponds or treatment via cascading waterfall greatly reduces the concentrations. In addition, aeration and biological conversion can reduce BOD and COD, assist in the precipitation of dissolved metals and convert ammonia nitrogen to nitrite and nitrate nitrogen.

3.3 Review of Leachate Treatment Options

There are several options available for the treatment and disposal of landfill leachate. Each option has certain advantages and disadvantages related to operations, maintenance and/or costs. This Study provides a review of six of the more commonly employed leachate treatment options associated with MSW landfills, although other leachate treatment options are available.

3.3.1 Wastewater Treatment Plant

Leachate treatment and disposal through a wastewater treatment plant (WWTP) is generally the simplest option for landfill operations. A leachate collection and storage system is designed based on expected leachate generation for the landfill. Leachate pretreatment may occur, typically aeration to decrease BOD and COD and to control odors. A loadout system is designed and constructed to transfer stored leachate to tanker trucks for transport to the WWTP or if a treatment plant or major sewer line is located nearby, a direct pipeline connection can be constructed.

The landfill must receive a permit from the WWTP to discharge leachate for treatment and disposal. These permits typically include limitation on the volume of leachate that can be disposed within a given period of time and restrictions on certain chemical constituents. A price per gallon is charged for treatment and disposal of leachate and additional charges may be incurred if the strength of the leachate is outside the operating conditions of the WWTP.

A technical memorandum was completed by Brown and Caldwell, Inc., in 2003 evaluating leachate generated at the SHSL going to the Hilo WWTP for treatment and disposal (Technical Memorandum on the Impact of Leachate Treatment on the Hilo Wastewater Treatment Plant).

"Year 2005 loadings were used due to the conservative assumption that 2005 is the first year that leachate could reach the WWTP, based on a preliminary landfill construction schedule of 2004. This assessment utilized findings of the 2002 influent loading summary and capacity assessment performed by Brown and Caldwell, during which several wastewater process modeling tools were employed."

The work conducted by Brown and Caldwell on the feasibility of leachate from SHSL going to the Hilo WWTP for treatment and disposal indicated a significant impact to the treatment plant and a large cost to the County.

"Under this scenario, the overall treatment capacity is estimated to be significantly impacted. Leachate addition has a two-fold impact. In the short-term, several plant modifications/additions would be required to treat the leachate waste stream and meet the discharge permit conditions. Secondly, contributions of leachate results in a loss of WWTP capacity which influences the overall plant expansion timeline."

"The overall connection fee is comprised of costs for required modifications and upgrades as well as cost to compensate for lost capacity. Based on the assumptions presented above, a connection fee of approximately $13.3M is estimated for the addition of untreated leachate to the WWTP."

The County discharge rates are based on waste volume as opposed to waste strength (concentration or load). Depending on the actual leachate strength the County may want to consider a strength-based rate structure in this case. The estimated leachate waste stream is approximately 20-times stronger than typical domestic wastewater based on BOD concentrations. The difference in strength justifies having a substantially higher rate in comparison to residential customers. Current (2003) County industrial dischargers rates are $0.10/gallon. Based on a leachate flow of 40,000 gpd, a preliminary estimated of the use fee is $4,000 per day, which translates into approximately $1.46M annually. In addition to the use fee, the additional operation and maintenance cost needs to be developed."

The Hilo WWTP is located near the SHSL. It is likely a direct connection could be made to the WWTP, thus bypassing the need to load and transport leachate resulting in cost savings and a reduction in the carbon footprint versus hauling. Given the estimated daily leachate generation volumes, a direct connection is an advantage. However, based on the Brown and Caldwell evaluation, the Hilo WWTP would be significantly impacted by the volume of leachate from the SHSL and major modifications would be needed to accommodate treatment and disposal of the landfill leachate. These improvement costs along with user fees and operations and maintenance assessments may be significant for the landfill operating budget.

3.3.2 Bioreactor Landfill

The Solid Waste Association of North America defines a bioreactor landfill as "a controlled landfill or landfill cell where liquid and gas conditions are actively managed in order to accelerate or enhance biostabilization of the waste." Some solid waste professionals use the term "bioreactor" and "leachate recirculation" interchangeably. Typically, a bioreactor landfill is a landfill that injects liquids in a controlled manner, into its waste mass in addition to leachate and gas condensate produced on-site. The USEPA defines bioreactor landfill as liquids being added to the waste mass to achieve a minimum of 40 percent by weight moisture in anaerobic conditions.

The goals of bioreactor landfills are to:

- Speed waste stabilization;
- Enhance landfill gas production;
- Increase available landfill airspace through waste settlement;
- Improve leachate storage and treatment; thereby reducing leachate management costs; and
- Reduce the length and cost of post-closure activities.

In a bioreactor landfill, degradable organic wastes are converted to soil-like humus through the addition of liquids to the waste mass. The degradation of waste causes settlement to occur more rapidly versus dry entombment and the available airspace can be reused. Airspace recaptured through settlement may be up to 20 percent or more of the waste depth at bioreactor landfills compared to the eight to 12 percent for traditional landfills. The rapid degradation of organic waste also produces landfill gas of which a major component is methane. This increase in gas production may make a landfill gas to energy project economically feasible.

The primary advantages are more rapid waste stabilization resulting in reduction of long-term risk to ground water contamination and reduction in leachate toxicity (U.S. Environmental Protection Agency, 2011). This reduces long-term potential liability associated with dry tomb landfill technology. The ability to reuse airspace gained from waste settlement may also reduce the total land area required for new or expanded landfills.

Bioreactors have capital costs associated with infrastructure to re-introduce liquids into the waste mass and increased operation and maintenance costs. These costs may be offset in the future through recaptured airspace and reduction of long-term risk.

At this time, bioreactor landfills are only permitted using site-specific rule variances with the EPA and state regulatory agency. The EPA issued the final rule on Research, Development and Demonstration (RD&D) Permits for Municipal Solid Waste Landfills (FR Vol. 65, NO. 55, March 22, 2004) that gives approved states authority to issue permits for bioreactor projects.

3.3.3 Leachate Recirculation

Leachate recirculation is the process of reintroducing collected leachate back into the landfill and is an option for on-site leachate management. Benefits of leachate recirculation include: improvements in leachate quality, faster stabilization of the landfill, and enhanced landfill gas production. There are different methods and designs to recirculate leachate including spraying onto the work face, digging ponds or trenches into the landfill waste and filling them with leachate or installing subsurface leach fields or injection wells.

Leachate recirculation is similar to a bioreactor landfill in that liquid is introduced into the waste mass to promote organic waste degradation and waste stabilization. However, where a bioreactor may accept other liquids in addition to landfill leachate generated on-site to achieve a minimum waste moisture content of 40 percent, leachate recirculation is a leachate management method and the goal is to achieve field capacity of the waste mass, up to 60 percent moisture by weight.

The advantages and disadvantages of leachate recirculation are very similar to bioreactor landfills listed above. Typically leachate recirculation operations are permitted by variance through the state regulatory agency.

3.3.4 Filtration

Filtration technologies include many different forms from simple sand bed filters to complex membrane filtration systems. The drinking water industry and ground water remediation use physio-chemical membrane filtrations systems such as microfiltration, nanofiltration, ultrafiltration, and reverse osmosis. Filtration using granulated activated carbon has been used in groundwater remediation and drinking water filtration for decades. Biological filtration may work well in the treatment of domestic wastewater but may be problematic in handling MSW leachate with high ammonia, BOD, COD and certain inorganic compounds. Physico-chemical filtration may be able to reduce the contaminant load by 60 to 100 percent dependent on the specific chemical; however, filtration technology is susceptible to biological fouling and chemical scaling. Therefore, pretreatment or a series of different filtration technologies may be necessary to achieve the goal of contaminant reduction for disposal.

Biological filtration systems generally require multiple steps to treat wastewater and a larger amount of space for operation. Infrastructure and operations costs can be significant and operation and maintenance may be high and labor intensive. Effectiveness as a treatment process for landfill leachate is low as removal of contaminants may be limited.

Physico-chemical filtration may be more applicable to landfill leachate as its contaminant removal may provide better results. The footprint for a physico-chemical filtration system is smaller than a biological filtration system but multiple systems may need to be constructed in series to handle high flow volumes. Pretreatment of leachate may be necessary to remove suspended particles to reduce scaling of the primary filtration unit. Biological fouling could be problematic with the high organic compound loads found in MSW leachate. Filtration systems will produce concentrated brine, typically 10 to 30 percent of the total volume filter that must be managed. Operation and maintenance of filtration systems can be labor intensive and replacement of membrane filters expensive.

Reverse osmosis filtrations systems have met with the most success for MSW leachate. Estimated costs for a reverse osmosis filtration system that can handle 10,000 gpd are $300,000 to $500,000. Multiple units or a custom system design would likely be required to handle the 40,000 gpd volume of leachate anticipated with the SHSL. The estimated cost for a filtration system to handle a flow of 40,000 gpd is $1.2M to $2.0M. Additional trained personnel would be necessary to operate and maintain the system.

3.3.5 Evaporation

The use of evaporation for treatment of landfill leachate may be effective at removing contaminants. A common type of leachate treatment by evaporation process is single stage flash evaporation. In this process the liquid mixture is heated and enters a flash chamber at a reduced pressure. The mixture partially vaporizes and the vapor comes to equilibrium with the residual liquid at the new lower temperature and pressure. The resulting liquid product is referred to as the concentrate while the resulting vapor product that becomes a liquid upon condensation is referred to as evaporate. Evaporate will be mostly water and may be high quality and easy to dispose compared to effluent streams produced by other treatment methods. The resulting concentrate will contain most of the contaminants and will be approximately 10 to 30 percent of the total volume treated. Depending on its final composition, the concentrate may be classified as a hazardous material, but generally the composition is such that it can be placed back into the waste mass of the landfill. Depending on the chemical makeup of the leachate, a two-step process may be necessary to separate and treat various components.

An advantage of the evaporation process is the ability to reduce large volumes of leachate to more manageable quantities. Typically the footprint for an evaporation treatment system is small in comparison to other treatment systems.

Disadvantages in the process include:

- Concentrate produced may be difficult to dispose of if it is classified as a hazardous waste;
- The volume of concentrate may be up to 30 percent of the original volume treated;
- It is a highly technical process that would be labor intensive and require significant operation and maintenance; and,
- Unless LFG is available as a power source, power requirements for operation are significant.

Estimated cost for a standard evaporation treatment system to process 10,000 gallons per day is approximately $500,000 to $750,000. Multiple units or a custom system design would likely be required to handle the volume of leachate anticipated with the SHSL. Therefore, the estimated cost for an evaporation system to handle 40,000 gpd is $2M to $3M. In addition, operations and maintenance costs would be a significant portion of the landfill operating budget.

3.3.6 Wetlands

Wetland systems exist in North America and Europe and treat a variety of wastewaters, including municipal, mine drainage, urban and agricultural storm water, sludges, leachates, and various industrial effluents. Wetland treatment can take place in natural or (much more often) constructed wetland sites. Principal categories of constructed wetland systems include densely vegetated overland flows, subsurface systems, ponds and island systems, and channels with floating plants.

Several component wetland processes combine to provide the observed overall treatments. Sedimentation and filtration remove solids. Chemical precipitation, ion exchange, and plant uptake remove metals. Nutrients are utilized by plant and algae and cycled to newly formed sediments. Volatile substances volatilize. Many materials undergo microbial transformations. These processes all lead to transformation and transfer of a “removed” pollutant either to the atmosphere or to the wetland sediments, soils and plants.

This technology requires land instead of mechanical devices to accomplish treatment. If necessary land is available, it typically offers modest capital savings over competitive processes. However, it typically offers a very large advantage on operating costs, because operation is simple and maintenance is very low

3.4 Evaluation of Leachate Treatment Options

3.4.1 Wastewater Treatment Plant

Provided the WWTP would be willing to accept the SHSL landfill leachate for treatment and disposal, sending leachate to the WWTP would be the simplest option from an operations and maintenance aspect of the landfill operations. The WWTP is well known for the ability and technology to handle domestic and industrial wastewater. The WWTP holds the appropriate permits to treat and dispose of wastewater. The landfill would in-turn be a permitted user.

Since the WWTP appears to be undersized to handle the anticipated volumes, the County would need to negotiate its financial share in upgrades and improvement to the facility to accommodate the additional disposal. Given the potential daily volumes to be sent for treatment and disposal, and the location of the landfill in relationship to the WWTP, a direct pipeline connection to the WWTP would be preferable to transporting via tanker truck. Therefore, infrastructure for the landfill would include storage ponds or tanks, pumping systems and a forcemain connection to the WWTP or nearby sewer. Cost estimates by Brown and Caldwell (2003) were based on an average daily flow rate of 70,000 gpd. Leachate generation predicted by the HELP model run for this Study estimated average daily leachate generation for Cell 1 at approximately 40,000 gpd. If the landfill employs operation methods to prevent rainfall from entering the waste mass and handling it as stormwater, and landfill leachate recirculation as part of its operations, average daily flow volumes could be significantly less than 70,000 gpd estimated by Brown and Caldwell, and less than 40,000 gpd. Discussion with the WWTP should also be pursued as a possible backup disposal option for any primary leachate treatment and disposal method selected as the DOH, Solid and Hazardous Waste Branch may require WWTP as an alternative or backup plan while implementing a new leachate treatment technology.

Discussions with the Waste Water District (WWD) staff for the Hilo WWTP conducted for this Study indicates the WWD has been in Whole Effluent Toxicity Testing (WETT) failure since August 2010 and has been performing accelerated testing since that time. Since the plant is failing the WETT any additional sources of potential toxicity, like landfill leachate, would not be accepted.

Additionally, leachate disposal to the WWTP, if allowed, would likely trigger a requirement to institute and administer a costly wastewater pretreatment program which here to fore has been avoided given the plant's throughput being at or below the industrial wastewater discharge program threshold level of 5 million gallons per day as there are currently no Significant Industrial Users.

3.4.2 Bioreactor Landfill

Bioreactor landfill technology may be used at the Landfill if it can be permitted through the DOH. This type of landfill operation is granted under a variance to the current EPA Subtitle D landfill rules. The combination of the amount of precipitation and the volume of MSW received by the SHSL are likely not ample enough to operate a bioreactor landfill and thus handle the entire volume of leachate generated. Thus, a secondary leachate management option would be necessary for treatment and disposal of the excess leachate.

A bioreactor operation will require a rigorous permitting process that must be completed pursuant to the RD&D permitting requirements. This means the SHSL would need to complete a demonstration phase of the project before implementing on a larger scale. Bioreactors generally require additional liquid sources (e.g. municipal wastewater), and the additional liquid will need to be stored on site prior to injection into the waste mass. This will increase the facility infrastructure costs. The costs of permitting will vary, but generally can be estimated at $125,000, with another $25,000 per year during the demonstration phase. Infrastructure upgrades and additional liquid procurement is estimated at $250,000

Additional infrastructure and operations and maintenance are required with the operation of a bioreactor landfill. Piping and pumping equipment will be necessary to distribute collected leachate back into the waste mass. Materials and construction costs are estimated at $500,000-$650,000 per waste cell. Additional landfill staff time would be needed on a daily basis to operate and monitor the liquid injection system for the bioreactor. Initial additional labor is estimated at 0.1 to 0.25 full time employee (FTE) but could be as high as 0.5 FTE dependent on operation and maintenance of the system. As the landfill waste mass increases and accelerated degradation takes place, LFG will be produced which will require an active GCCS. The initial phase of a GCCS is estimated at $2M to $2.5M. With the installation of a GCCS, additional labor is necessary to balance and monitor the system for proper operations. In conjunction with the bioreactor, 1 FTE will likely be necessary.

A bioreactor landfill will also require additional engineering and operations monitoring. Since liquid is being pumping into the landfill to achieve moisture content greater than 40 percent by weight, this is generally referred to as a "wet landfill" operation. Operations as a bioreactor can result in additional landfill design and operational considerations. Some of these include, managing increased LFG generation (installation of a GCCS), sideslope design to guard against side slope failure, and proper liquids management to avoid leachate seeps through the side slopes.

Discussions with the Hawaii DOH Solid Waste Branch indicate that the state of Hawaii never formally accepted the EPA RD&D rules for Municipal Solid Waste Landfills. Therefore, a permit application would need to request a variance from the solid waste rules and demonstrate to the DOH that a bioreactor landfill operation is feasible at this location.

3.4.3 Leachate Recirculation

Leachate recirculation is basically the same technology as a bioreactor landfill, except with leachate recirculation, only leachate and condensate generated by the landfill are recirculated into the waste mass to increase the moisture content. The volume or capacity for storage of leachate within the waste mass is dependent of the volume of waste received and placed in the disposal cell, and the precipitation received.

Therefore, leachate recirculation could be used in conjunction with another leachate treatment option as part of the overall leachate management system, but it is likely there will be excess leachate that will need to be treated.

Leachate recirculation operations are typically viewed by the regulatory agency as part of leachate management in conjunction with a primary treatment and disposal option. Therefore, permitting of this type of operation may encounter less permitting restrictions.

Material and constructions cost are similar to bioreactor landfill costs, just less liquid is recirculated back into the waste mass. The same is true with operations, maintenance and monitoring costs. Because leachate recirculation will increase degradation of waste and promote accelerated production of LFG, the County would need to consider a proactive method of managing LFG. This could include the installation of a GCCS.

An additional permit variance may be required to allow leachate recirculation at the landfill expansion. Given the lack of low permeable soil in the area for a prescribed composite liner system, an alternative liner system is proposed using a GCL. However, since DOH has not accepted the EPA RD&D rules, leachate recirculation is only allowed under the prescribed composite liner construction. A permit variance would be needed to allow leachate recirculation with an alternate liner system.

3.4.4 Filtration

Filtration systems have been used for centuries in the drinking water industry. Use of filtration systems with landfill leachate have met with mixed results. The chemical and biological makeup of landfill leachate makes it difficult for one type of filtration to work efficiently and cost effectively to produce a high quality liquid that may be disposed of in surface water bodies. Suspended solids, biofouling and chemical precipitates are trapped within or on the filtration media resulting in high operations and maintenance, and replacement costs associated with the process. Additional cost is incurred with handling the concentrate generated as part of this process. Dependent on the characteristics of concentrate, the material may need to be handled as hazardous waste. Concentrate volumes may be 10 to 30 percent of the volume filtered.

Cost for filtration units are approximately \$300,000 and \$500,000 per 10,000 gpd. Therefore, the initial cost for a filtration system for SHSL may range from \$1.2M to \$2.0M to filter 40,000 gpd. Because of the operation and maintenance involved, the system should be oversized approximately 25 percent to allow for portions of the system to be out of operation for maintenance and repair. Filters may also need to be replaced every three to five years at a cost of up to 50 percent of the original unit cost.

Filtrations systems can be labor intensive and may require 0.5 to 1.0 FTE for monitoring and maintenance of the system or a service contract with a qualified vendor to keep the system operational. Also, an alternative treatment option may be necessary, likely WWTP, to divert excess leachate or handle leachate if the system goes off line.

3.4.5 Evaporation

Evaporation systems have been used over the past decade or so for leachate treatment. Like filtrations systems, they can be labor intensive to operate and maintain and they produce a concentrate that will require additional handling and disposal. Evaporation systems require a significant amount of energy to operate and unless the landfill is producing LFG to be used as an energy source, the energy requirement may be a large portion of a small landfill's operating budget. An alternate leachate treatment and disposal option is likely required during periods of excessive leachate generation and system down time for maintenance.

Cost for evaporation units are approximately $500,000 and $750,000 per 10,000 gpd. Therefore, the initial cost for an evaporation system for SHSL may range from $2.0M to $3.0M to filter 40,000 gpd. Because of the operation and maintenance involved, an alternative option may be necessary when the system is out of operation for maintenance and repair.

Evaporation systems can be labor intensive and may require 0.5 to 1.0 FTE for monitoring and maintenance of the system or a service contract with a qualified vendor to keep the system operational.

3.4.6 Wetlands

Thousands of wetland treatment systems are in operation in North America and Europe treating a variety of wastewater. Wetland treatment of landfill leachate has been successful in several locations throughout the United States.

Constructed wetlands have the advantage of offering long-term, sustainable treatment with very low operation and maintenance costs of passive systems. Passive constructed wetlands offer very long lifetimes, with little or no equipment replacement. Active constructed wetlands offer the same advantages for the online period, but do require periodic refurbishment and disposal of spent substrate.

Wetlands offer a passive, sustainable remedy of a large number of chemical constituents. They are a low cost alternative, and since leachate flow volumes are relatively modest, the land-intensive attribute of treatment wetlands does not cause the severe siting limitations.

On the negative side, removal of certain chemicals may be limited and it may be necessary to thoroughly understand the fate of leachate contaminants to determine treatment. The extent of treatment is dependent on the regulatory requirements for the receiving body of water or infiltration area in which treated water would be discharged. As part of the scope of work for this Study, a conceptual wetland treatment system was designed to further evaluate wetland type, land needs and treated water discharge. A cursory evaluation of leachate management and treatment options was conducted and given the cost of construction, operation and maintenance; wetland treatment was selected for a detailed evaluation. A copy of the detailed evaluation of the wetland treatment system is provided in Appendix E.

Table 3-1

Landfill Leachate Management and Treatment Options
South Hilo Sanitary Landfill

Method	Advantages	Disadvantages	Costs
Wastewater Treatment Plant (WWTP)	Ease of wastewater disposal. Treatment and disposal handled under permit to the WWTP	Hilo WWTP is currently not capable of handling the estimate flow volumes of leachate generation.	Upgrades to WWTP could be up to $13 million.
Bioreactor Landfill	Management of landfill leachate on-site. Potential for accelerated waste stabilization, LFG generation. Accelerated waste settlement can lead to recovery of airspace and delay construction.	Additional costs for infrastructure and equipment to distribute leachate into the waste mass of the landfill. Bioreactor landfill will require additional staff (0.1-0.25 FTE) for operations, maintenance and monitoring. Incoming waste volumes and waste mass may be too low with high precipitation and leachate generation at SHSL site to allow for bioreactor operations. Extra engineering and permitting efforts will be required to obtain DOH approval	Estimated cost for infrastructure is $500,000 to $750,000/waste cell. Additional personnel 0.1 to 0.25 FTE Likely result in need for landfill gas (LFG) gas collection and control system (GCCS). Initial phase of GCCS up to $2.5 million. Permitting costs for permit application up to $125,000 and $25,000 per year during demonstration. Additional infrastructure and liquid procurement up to $250,000
Leachate Recirculation Landfill	Management of landfill leachate on-site. Potential for accelerated waste stabilization, LFG generation. Accelerated waste settlement can lead to recovery of airspace and delay construction.	Additional costs for infrastructure and equipment to distribute leachate into the waste mass of the landfill. Leachate recirculation will require additional staff (0.1-0.25 FTE) for operations, maintenance and monitoring. Extra engineering and permitting efforts will be required to obtain DOH approval	Estimated cost for infrastructure is $500,000 to $750,000/waste cell. Additional personnel 0.1 to 0.25 FTE Likely result in need for landfill gas (LFG) gas collection and control system (GCCS). Initial phase of GCCS up to $2.5 million.

Table 3-1

Landfill Leachate Management and Treatment Options
South Hilo Sanitary Landfill

Method	Advantages	Disadvantages	Costs
Leachate Filtration	Removal of contaminants within leachate allowing for easy disposal of treated water. Newer advanced systems can remove between 60% and 100% of contaminant.	Filtrations systems are labor intensive requiring maintenance and monitoring to keep operation running efficiently. A concentrated brine is produced as a byproduct of the filtration system that may be between 10% to 30% by volume of the wastewater filter. Disposal is difficult if concentrate is determined to be a hazardous waste.	Estimated cost for filtration operation to handle 40,000 gpd is $1.2 to $2.0 million design and construction. Additional labor for maintenance and monitoring is 0.5 to 1.0 FTE
Leachate Evaporation	Removal of contaminants within leachate allowing for easy disposal of treated water. Newer advanced systems can remove between 60% and 100% of contaminant.	Evaporation systems are labor intensive requiring maintenance and monitoring to keep operation running efficiently. A concentrated brine is produced as a byproduct of the filtration system that may be between 10% to 30% by volume of the wastewater filter.	Estimated cost for filtration operation to handle 40,000 gpd is $2.0 to $3.0 million for design and construction. Additional labor for maintenance and monitoring is 0.5 to 1.0 FTE
Wetland Treatment	Passive wetlands are low cost and low maintenance. Contaminant removal is via plant uptake, adsorption, mineralization and volatilization processes. Operation and maintenance is requirements are low once vegetation is established.	Wetlands and the associated storage ponds and infiltration galleries can require large open areas of land. Passive systems require time to treat waste water and may be limited in the contaminants that can be removed.	Estimated cost to construct a wetland system to handle 40,000 gpd is $450,000 for design and construction. Additional labor for upkeep and monitoring is 0.1 to 0.25 FTE

3.5 Wetland Treatment System

The wetland treatment system design evaluated for landfill leachate management associated with the SHSL is an active treatment wetland utilizing subsurface flow and aerated beds. A constructed subsurface flow treatment wetland is a lined, shallow pond (approximately one-foot deep), below the ground, filled with pea-gravel or other substrate and contains emergent wetland plants. Water is introduced at one end and proceeds across the wetland to a discharge point. In a subsurface flow treatment wetland, flow is in and around the roots of the wetland plants.

Treatment begins by pumping collected leachate form the storage pond/tank to an elevated passive cascade aerator with sediment pond/tank. This is to provide a means to aerate, precipitate and settle out soluble iron and other metals and remove VOCs. An aerated wetland will follow the sedimentation pond and provide removal of BOD, ammonia, suspended solids and other filterable components. The wetland will be designed to remove constituents by sorption, aerobic and anaerobic degradation and plant uptake. Aeration of the wetland provides a stable oxic environment for the growth of aerobic bacteria responsible for the oxidation of complex hydrocarbons and ammonia. The gravel bed of the wetland will be underlain with a network of aeration tubing that provides uniform air flow across the floor of the bed. Upon completion of the treatment process, the treated liquid is generally discharged into the ground either through an infiltration gallery or through direct injection wells.

The Hawaii DOH has not permitted a facility using constructed wetland technology to treat landfill leachate for discharge. As such, as with any type of treatment technology never approved by the DOH, an acceptable backup treatment system would need to be in place until the alternative technology demonstrates it is feasible and is approved. Also, since treated leachate would need to discharge via infiltration to groundwater or overland discharge to surface water, discharge standards would need to be established. At this time the DOH has not accepted the EPA discharge standards for treated leachate via wetlands and the DOH does not feel it has the expertise in-house to establish such standards. The DOH will not be able to provide an answer regarding the permitability of this process until a permit application is submitted for review.

Based on the conceptual wetland design, the estimated cost to construct a treatment wetland for 40,000 gpd for the SHSL is approximately \$590,000. Operation and maintenance of the system should be relatively low and may require 0.1 to 0.25 FTE for monitoring and operation of the system.

Because of the uncertainty of approval for leachate treatment via constructed wetlands, two scenarios were evaluated for cost comparison. Both scenarios require the WWTP to complete facility expansion and upgrades for leachate treatment. The first scenario assumes the DOH approves the wetlands with the WWTP as the contingent back-up. Although, not used as the primary treatment option, the landfill would be assessed a standby fee to reserve the necessary capacity. The second scenario assumes the landfill forgoes permitting and design of the wetlands and treats leachate only with the WWTP. It would be assumed that there would be as cost for installation of a direct pipeline and there is a processing and treatment fee. The cost details are provided in Section 6.2

Section 4
LAND, PERMITTING, ENVIRONMENT, AND SOCIOECONOMIC CONSIDERATIONS

This Study included many conversations with various stakeholders affected or potentially involved with the development of the landfill expansion. The proposed expansion area is part of government land that is in operation as a quarry. Potential issues arising from the reclassification of the land use, proposed developments in and near the area, and road right-of-ways are presented in Section 4.1. Conversations with various regulatory authorities are presented in Section 4.2. The DOH would be the main party involved with the permitting process. Lastly, conversations were held with other entities potentially impacted by the continued operation of the landfill and the potential expansion. These are summarized in Section 4.3. A table summarizing the meetings is presented in Appendix F. This Study was not intended to provide a comprehensive set of solutions, but instead to bring awareness to the concerns of the local residents.

4.1 Land Use, Land Use Designation, and Land Use Permits

Figure 1 in Appendix F depicts properties in the general vicinity of the proposed landfill expansion and illustrates zoning, ownership and land use. Table 4-1, below, provides, detailed property information for immediately adjacent properties.

Table 4-1

Tax Map Key (TMK)	Owner	Current Use	Tax Acres	Zoning
2-1-012-003	State	Keaukaha Mil. Res.	442.486	A20-a[B]
2-1-012-029	State DHHL	None	184.820	A20-a
2-1-013-002	State	Various, Mostly Vacant	2407.756	A20-a
2-1-013-010	State	Keaukaha Mil. Res.	61.174	A20-a
*2-1-013-011	State	Various Solid Waste[A]	6.500	MG1-a[C]
*2-1-013-142	State	Quarry	40.000	MG1-a
2-1-013-148	County EO	Flood Detention Basin	40.000	MG1-a
*2-1-013-150	County EO	Various Solid Waste	35.400	MG1-a
*2-1-013-152	County EO	Landfill	19.482	A20-a
*2-1-013-156	County EO	Landfill	20.000	MG1-a/A20-a
2-1-013-158	State DHHL	Various Solid Waste	95.392	A20-a
2-1-013-160	State	Quarry	13.333	A20-a

Table 4-1

Tax Map Key (TMK)	Owner	Current Use	Tax Acres	Zoning
2-1-013-161	State	Quarry	13.333	A20-a
*2-1-013-162	State	Landfill? Quarry?[A]	6.000	A20-a
2-1-013-163	State	Quarry	13.333	MG1-a
*2-1-013-167	State	Various SW?[A]	13.860	A20-a
*2-1-013-168	State	Various SW[A]	10.940	A20-a

*Proposed along with various road ROW for DLNR Set Aside to County of Hawai‘i for Solid Waste and Road and Utility Purposes, per BLNR Action 04HD-258 &259, approved by BLNR 9/24/04, with conditions (pending fulfillment).

[A] Items Require Discussion With County.

[B] A20-a: Agriculture, minimum lot size 20 acres. Per Chapter 25 (Zoning) of the Hawaii County Code, a “public dump” may be permitted in the A district, provided that a special permit is obtained for suchuse if the building site is located within the State land use agricultural district. §25-5-72(c)(12).

[C] MG1-a: General Industrial, 1 acre minimum lot size. A “public dump” is an outright permitted use. §25-5-152(a)(46).

As illustrated in Figure 1 of Appendix F, areas that directly adjoin the properties identified for the landfill expansion (including the potential leachate treatment area) include the Keaukaha Military Reservation (KMR), a County Parks and Recreation (P&R) skeet range, unencumbered State land, active and former rock quarries, and Department of Hawaiian Homes (DHHL) land. Discussions with former and current land managers indicated awareness of the long history of use for solid waste purposes and no insurmountable obstacles to permitting, or interference with existing or planned uses per se. However, there were various concerns, some of them potentially substantial, as outlined below. To summarize:

State Land Division

- The proposed project area itself (which includes unencumbered State land for the leachate treatment area) does not appear to conflict with any existing or proposed State uses, including Revocable Permits (RPs), if County P&R concerns regarding the Skeet Range are successfully addressed.
- In September 2004, the Hawai‘i State Board of Land and Natural Resources (BLNR) approved an item dealing with the cancellation of Governor’s Executive Order (EO) Nos. 3975, 2432 and 2841, and establishment of a Set Aside of State Lands to the County of Hawai‘i of tax map keys (TMK) 2-1-13: 11, 142, 150, 156, 162, 167, 168 and portions of road right-of-ways (ROW) for solid waste, road and utility purposes (PSF: 04HD-258 and 259). The Set Aside was vital to rationalize land use relative to existing property boundaries. For example, there are paper road right-of-ways that traverse over the top of the existing landfill that were excluded from the previous EOs that authorized County use of this State land. It would be unlikely that a public road would be constructed over the top of a closed landfill. Furthermore, the existing road to

the landfill crosses DHHL land with no authority and is not an official public right-of-way. It is thus critical that such road right-of-ways be included in the land under County control. The cancellation of existing EOs and establishment of a new Set Aside will only come into effect when the consolidation/subdivision, which was supposed to include two 60 foot road ROWs located to the satisfaction of the BLNR, is completed and registered to the satisfaction of the State. Without this, there are many unresolved problems related to the location of facilities on parcels and serious access and safety issues. It is important to note, however, that a portion of access depended upon the State completing its Mana Quarry industrial lands project, which included extensions of Leilani and Lanikaula Streets. This project has been postponed, and likely cancelled. Without this project, it will be difficult to comply with the conditions related to access and right-of-ways and to obtain acceptable legal access to the SHSL site, although the County may be able to meet the other conditions.

- In the 1970s, the Department of Transportation, Airports Division (DOTA), extended the Hilo runway and landlocked the Kamehameha Schools (KS) quarry area (see Figure 1, Appendix F). In order to remedy this, the BLNR approved a revocable permit to DOTA for an access easement over the road east of the SHSL that accesses the Skeet Range. However, this road was formerly subject to closure during certain operations at the KMR. Although these operations no longer occur, future plans at KMR include the potential for at least brief closures of this road. DOTA remains under legal obligation to provide a permanent, full-time legal access to these KS lands. When the BLNR approved the 2004 Set Aside referenced above, it was subject to a revocable permit easement and a possible long term easement to KS and its quarry licensees.
- BLNR is interested in ensuring and facilitating access to 2,553 acres of State lands east of the SHSL. Given existing plans, the County must carefully coordinate its access roads with BLNR to fulfill long-term State needs.
- The Puainako Extension is still shown as a “Future Collector” on the current (2005) Hawai‘i County General Plan. The road is shown following a corridor between the existing SHSL and the proposed expansion site, then traversing the KMR to intersect with the Hilo International Airport access road. Although even without considering the landfill expansion this route no longer appears practical, the fact that it has been legally adopted in the Hawai‘i County General Plan will require consideration.

Keaukaha Military Reservation

- According to the Hawai‘i State Department of Defense (HDD) (see letter in Appendix F), which operates the KMR for the Hawai‘i Army National Guard, the area for the proposed expansion is located down-range of the National Guard’s pistol and known-distance (KD1) rifle ranges. The associated safety zones for the types of weapon and munitions authorized for live-training at these ranges extend beyond KMR’s southern boundary and will probably “impact” the proposed expansion area. In the view of the HDD, the proposed landfill expansion will unduly "raise the safety bar" for live-fire training, as the expansion would increase down-range traffic and industrial activities. The HDD has stated that although it has no easements, EOs, revocable permits, leases or other claims on the State land in this area, other uses should be prohibited.

- The HDD is also concerned that with increased vehicular traffic, movement of loose dirt and solid wastes, and waste decomposition, this ambient air quality will substantially deteriorate. Soldiers doing their training and daily physical exercises will be most affected by prolonged exposure to unhealthy air. The HDD also has concerns regarding native ecosystems and endangered species within the KMR boundary. Flammable gas, landfill fires, groundwater contamination, and storm water pollution are also mentioned. These concerns do not account for the fact that the landfill would be moving further away, not closer, to KMR, but the points remain items of concern for the HDD.

County Department of Parks and Recreation (P&R)

- The County maintains a Skeet Range on about 15 acres of State land within TMK 2-1-13:01. The Skeet Range is marginally within the area that could be used for leachate treatment. The County operates this facility under Revocable Permit S-4171, dated April 26, 1968, which covers 113 acres of land. P&R has been aware of the potential for Landfill expansion for at least five years and has expressed the unofficial policy that it would cooperate with a relocation, so long as a reasonable area for a new skeet range was identified and the County paid for the relocation without an impact on the P&R budget. The cost of relocation has not been studied in detail but could exceed $750,000.
- Maintenance of adequate access the County drag strip, located about a mile south of existing solid waste facilities, is also essential to P&R interests.

Department of Hawaiian Home Lands (DHHL)

- Long-term plans for parcels adjacent to the SHSL are still under development, and DHHL will keep the County apprised of its decisions.

Potential Impacts and Feasible Mitigation Measures

- The open issue of the DLNR Set Aside to the County of Hawaiʻi needs to be resolved in order to provide appropriate property boundaries and access.
- The question of the Puainako Extension route should be resolved.
- DEM should work with KMR to evaluate whether, given the location and design, their concerns are realistic, and to determine studies that will be necessary for a landfill expansion Environmental Impact Statement (EIS).
- DEM should work with P&R on design, schedule and cost estimates for removing existing skeet range and building a new facility.
- DEM should continue to monitor DHHL plans.

The following land use and related environmental permits and approvals would be required to expand the landfill at the present site.

- Completion of an EIS conformant with Chapter 343, Hawai‘i Revised Statutes
- Chapter 6e (Historic Sites) Determination of No Effect
- State Land Use Commission, State Land Use District Boundary Amendment (or County of Hawai‘i Special Use Permit)
- County General Plan Interpretation and/or Amendment
- County Change of Zone
- County Grubbing, Grading, Stockpiling Permits
- County Building Permits (for various miscellaneous improvements including electrical, pump, scale, and related facilities)

4.2 Landfill Permitting

Permitting of an expansion at a landfill site can be a multistep, multiyear process. The expansion will require further in depth evaluations of the composite liner system, leachate treatment facilities, and geotechnical aspects. The County must demonstrate that any alternatives to those prescribed in HAR-11-58.1, meet or exceed the requirements.

The permitting process will require considerations for other nearby landowners, business, industries, and activities to ensure that impacts to them are minimal. Many of these potential concerns are noted below in Section 4.3 and should be addressed either through communication with the respective party or through inclusion in the permit. Additionally, the permitting process will require public comments and possibly hearings to address concerns. Through the planning of the expansion development, many of the concerns identified in Section 4.3 can be addressed.

4.2.1 Hawai‘i DOH

The Hawai‘i Department of Health (DOH) will be the local regulatory body to review and provide permits for the operation of the SHSL. The DOH was contacted to discuss the potential of expanding the SHSL and requirements related to operation of an on-site leachate treatment system and discharge of treated water.

Solid Waste

DOH Office of Solid Waste Management is the regulatory body that will provide review of a permit application to expand the SHSL. Landfill design, construction and operation must conform to Subtitle D and DOH Solid Waste Rules. In discussion with DOH personnel for this Study, Solid Waste Management stated no decisions can be made concerning the expansion of the landfill or the treatment of leachate on-site until a detailed permit application is submitted.

Concerns raised by DOH staff in there preliminary discussions include:

- The location of the landfill in relationship to the Hilo International Airport;
- Leachate management given the average annual rainfall in the Hilo area; and
- A backup landfill leachate treatment option, such as an agreement with the local WWTP for any leachate management plan that is not a generally accepted management practice.

Since on-site landfill leachate management through wetland treatment has not been used at another facility in the State of Hawaiʻi, DOH staff may require DEM to have an agreement in place with the WWTP as an alternative leachate treatment option.

Wastewater

Leachate management via treatment wetland with treated discharge water going to an infiltration gallery would fall under the jurisdiction of the DOH Clean Water Branch. Discussions with DOH staff in this program indicate a treatment of this type would be the first of its kind in the state. Staff has stated that the DOH does not have established discharge limits or criteria for such situation and does not have the time, expertise or budget to establish such limits. If this discharge option is pursued, DOH would need to be willing to accept the established EPA criteria for such treatment options or DEM may need to develop discharge standards as part of the permit application.

The landfill is located seaward (*makai*) of the UIC line. However, since discharge of treated leachate is not proposed via injection wells, the UIC Program would not have jurisdiction regarding an infiltration gallery.

4.2.2 Environmental Resources and Permits

The Mauna Loa lava substrate is geologically recent, and streams have not yet had time to form in the eastern, less steeply sloped parts of Hilo, including the SHSL. The nearest streams are Waiakea and Palai Streams located at one and a half and two miles to the west and southwest, respectively. In the National Wetlands Inventory, the U.S. Fish and Wildlife Service has mapped no wetlands in the vicinity of the SHSL (http://www.fws.gov/wetlands/Data/Mapper.html). Site reconnaissance near the SHSL indicates a very low likelihood of encountering wetlands in either the existing quarry where the landfill expansion is proposed or in the adjacent land which may be used for leachate treatment. At this point, it does not appear that a Clean Water Act Department of the Army Permit for dredge or fill in waters of the U.S. will be necessary.

Floodplain status for the project area has been determined by the Federal Emergency Management Agency (FEMA), which has mapped the area as part of the National Flood Insurance Program's Flood Insurance Rate Maps. The SHSL area is within Zone X, defined as areas identified in the community flood insurance study as areas of moderate or minimal hazard from the principal source of flood in the area. Although local drainage must be considered, no mapped flood hazard exists in the area, and no floodplain approvals from FEMA or the Department of Public Works appear to be required. As defined by the State of Hawaiʻi, groundwater beneath the site is part of the North East Mauna Loa aquifer sector of the Hilo aquifer system. The SHSL lies above the segment of this aquifer that is *makai* of the UIC line, which is found a minimum of one mile inland (*mauka*) of the SHSL. According to the DOH, groundwater that is *makai* of the UIC is not considered a drinking water source, while groundwater that is *mauka* of the UIC is considered a drinking water source. Thus, the portion of the North East Mauna Loa aquifer below the SHSL extending downgradient towards the Pacific Ocean is not considered a suitable drinking water source, and a landfill would be an allowed activity.

The vegetation near the SHSL has been highly disturbed and is a mixture of post-agricultural fallow vegetation (mixed alien shrubland and grassland and alien-dominated forest with relatively few native plants) with remnant lowland *‘ohi‘a* forest (Gagne & Guddihy, 1990). Areas directly adjacent to the SHSL are generally highly disturbed. Much of the area is early successional weed communities in areas that have recently been or are periodically disturbed, such as the quarry and access roads. There is also late successional forest, which is dominated by alien trees, including *Albizia moluccana*, gunpowder tree (*Trema orientalis*), trumpet tree (*Cecropia obtusifolia*), strawberry guava (*Psidium cattleianum*), *Melastoma candidum*, and bingabing tree (*Macaranga mappa*). Even in such disturbed areas there are occasional natives, including mini-groves of low-stature hala (*Pandanus tectorius*) resting on old bulldozer pushpiles, and a few scattered *‘ohi‘a* (*Metrosideros polymorpha*). As with the first vegetation type, this secondary forest has little conservation value for either the plant species it contains or as animal habitat, although Hawaiian Hawks may be able to forage there for rats and Hawaiian hoary bats may forage there for insects. No endangered plant species are likely to be found in this low elevation, highly disturbed area. Consultation of U.S. Fish and Wildlife Service maps indicates that no critical habitat for endangered plant (or animal) species is present on or near the proposed expansion area. Although wetlands treatment of leachate could potentially provide habitat for endangered species, discussions with resource agencies indicated that the existence of abundant alternative habitat and strict regulations for dealing with endangered species would be overly burdensome and of only meager benefit.

It should be noted that directly across the KS Quarry Access Road from the SHSL, on a portion of the 500 acres of the Hawai‘i Army National Guard’s KMR, is a somewhat intact remnant of moderately invaded lowland *‘ohi‘a* forest. This site contains a number of native plants that are relatively uncommon in the lowlands of Hilo and not present in the expansion area. These include *kolea* (*Myrsine* spp.), *kopiko* (*Psychotria* spp.), *mamaki* (*Pipturus albidus*), and *‘ie‘ie* (*Freycinetia arborea*) (Whistler, 2003). A joint University of Hawai‘i at Hilo-U.S. Forest Service-National Guard project is experimenting with restoration of the native forest at the site (Cordell, 2011). The HDD expressed concern for indirect impacts to this forest in their July 24, 2011 letter.

The South Hilo area supports a variety of common alien mammals, birds, reptiles and amphibians. More importantly, several species of native birds forage or fly over the site, including the Hawaiian Hawk or *‘Io* (*Buteo solitarius*), an endangered species. The endangered Hawaiian Petrel (*Pterodroma sandwichensis*), and the threatened Newell’s Shearwater (*Puffinus auricularis newelli*), which can be adversely affected by poles, wires and particularly unshielded lighting, may occasionally overfly the area.

The endangered *Nene* (*Branta sandvicensis*) is growing in population and is increasingly being found in grassy areas, where it may have adverse encounters with people or nest in areas of longer grass. Foraging habitat for Hawai‘i’s only land mammal, the endangered *Lasiurus cinereus semotus* (the *‘ope‘ape‘a* or Hawaiian hoary bat), may also be present in the forested areas surrounding the SHSL.

There has been concern that some species of birds attracted to the SHSL may impact aircraft using Hilo International Airport. If this concern is valid, expanded landfill operations have at least some potential to attract greater quantities of birds and thus pose a greater potential hazard to airport operations. Landfill hazards to airports are addressed as part of the Subtitle D, Location Restrictions, which states that “*Owners or operators of new MSWLF [municipal solid waste landfill] units, existing MSWLF units, and lateral expansions that are located within 10,000 feet (3,048 meters) of any airport runway end used by turbojet aircraft…must demonstrate that the units are designed and operated so that the MSWLF unit does not pose a bird hazard to aircraft.*” The SHSL fits these criteria because it lies within the regulated distance to the Hilo International Airport runway ends.

At the request of the Hawai‘i County Department of Public Works, A-Mehr, Inc. performed a study of the SHSL to determine whether the landfill posed a bird hazard to aircraft in 1998. The results of this study were detailed in the *South Hilo Sanitary Landfill Airport Safety Demonstration Report* (Report on file at DEM). A-Mehr concluded that, because of waste management techniques observed being practiced at the SHSL, scavenger birds commonly involved in bird/aircraft collisions were not present at the SHSL, and therefore did not present a significant hazard to the Hilo International Airport operations.

The current SHSL is viewed as a significant attractant to wild pigs, and various wildlife agencies and adjacent land managers expressed concern about controlling pig damage both at the SHSL and on adjacent lands.

Cultural resources include archaeological sites, cultural sites used for ceremonial purposes or honored for associations with historical or legendary beings, and traditional cultural practices, such as gathering. The general area has been used for quarrying and solid waste collection operations from at least the 1950s, and the potential for historic or cultural resources is fairly low. Archaeologists conducted surveys in an approximately 60-acre area just north of the SHSL as part of the EIS for the Sort Station and found no archaeological or cultural resources (Hawai‘i DEM, 2004). Discussions with community members and organizations as part of this research did not reveal any such sites or practices. An expansion of the landfill would require compliance with Chapter 343, HRS, including the obligation to prepare a Cultural Impact Assessment.

Necessary Studies, Permits and Approvals

- A complete botanical survey of all areas to be affected would need to be conducted as part of landfill planning.

- The landfill planning team would need to coordinate with the HDD, the University of Hawai‘i at Hilo, and the U.S. Forest Service concerning avoiding impacts to the *‘ohi‘a* forest at the KMR.
- Early in landfill planning, DEM would need liaison with the Big Island Invasive Species Council to begin monitoring for the presence of any new and potentially virulent alien species that might appear.
- As policy regarding impact analysis and mitigation measures relative to endangered seabirds and the *Nene* is rapidly evolving, it is advisable that the County would engage an ornithologist with experience in endangered species consultations to analyze impacts and devise appropriate avoidance and mitigation in consultation with regulatory agencies as part of landfill planning.
- Expansion of the landfill would entail the removal of trees taller than 15 feet, which could provide habitat for roosting of the endangered Hawaiian hoary bat. In conformance with standard impact avoidance protocols, the County of Hawai‘i and its contractors would need to schedule landclearing to refrain from activities that trim or remove trees larger than 15 feet in height during the critical pupping months for the Hawaiian hoary bat, from May 15 to August 15 of each year. Furthermore, if heavy equipment is to be used between April 1 and August 15, the area should be surveyed for Hawaiian Hawk nests by a qualified ornithologist. If they are found, construction in that area will be avoided during the remainder of the nesting period.
- Analysis in 1998 determined that bird species responsible for aircraft/bird collision are not present in the area. This conclusion would need to be revisited based on current bird species distributions.
- Landfill and leachate treatment facility design would need to exclude pigs to greatest degree feasible.
- As part of preparation for an EIS and early landfill planning, the County would need to conduct an archaeological survey and cultural impact assessment that includes consultation with knowledgeable community members, to determine if the affected area contains historic or cultural resources, and plan for mitigation as appropriate.

4.2.3 Federal Aviation Administration

The Federal Aviation Administration (FAA) imposes statutory requirements on owners and operators of landfills located in proximity to public airports. In general, landfills must be operated so as not to create a bird hazard to approaching and departing aircraft. Because the existing SHSL is located within a 10,000 feet radius of the Hilo International Airport, 40 Code of Federal Regulations (CFR) Section 258.10 requires demonstration of these mitigation measures as a component of the operating permit.

Advisory Circular 150/5200-34A creates a statutory restriction preventing construction of new municipal solid waste landfills within a six-mile proximity of a public airport. If the proposed SHSL project was considered to be an *expansion* of an existing facility as opposed to the construction of a *new* facility, this Advisory Circular would not apply. However, as discussed elsewhere in this study, it is not clear how the proposed landfill would be classified (new or expansion).

Any proposed expansion of the SHSL from its existing waste limits would be subject to FAA review per the Environmental Assessment (EA)/EIS process to ensure the expansion includes the proper considerations for ongoing mitigation of bird hazards to aircraft at the Hilo International Airport.

4.3 Socioeconomic Considerations

Environmental justice is a term that refers to social inequity in bearing the burdens of adverse environmental impacts. Certain socioeconomic groups in the United States, including ethnic minorities, the elderly, rural residents and others, have historically experienced a disproportionate share of undesirable side effects from locally undesirable land uses such as toxic waste dumps, landfills, and freeway projects. The policy of the State of Hawaiʻi is to address whether any minority or low-income group is disproportionately impacted by a proposed project and identify mitigation measures to avoid or minimize any adverse social impacts. This policy is in keeping with federal Executive Order 12898, *Federal Actions to Address Environmental Justice in Minority Populations and Low Income Populations*.

Expansion of the SHSL would affect the Panaewa Hawaiian Home Lands area and, more generally, other parts of Hilo, including Keaukaha Hawaiian Home Lands, which is intimately related to Panaewa. Hilo and Hawaiʻi County in general have a diverse population that for the last thirty years has been among the 100 fastest-growing counties in the U.S. The proportion of the Hawaiian population in Hilo is more than one-third greater than that of the County as a whole. Several segments of the population that typically exhibit disadvantaged measures of social welfare are disproportionately represented in the population of Hilo as compared to the County or State of Hawaiʻi. Median family income is less than 65 percent that of the County as a whole. More than 15 percent of individuals have income below the poverty level, double the statewide rate. The Panaewa area has a very high proportion of Native Hawaiians because of the requirement for lessees to have "not less than one-half part of the blood of the races inhabiting the Hawaiian Islands previous to 1778," and many Panaewa residents are low-income.

Activities conducted for this Study included extensive outreach with community organizations and individuals from the Panaewa and Keaukaha areas. Most important were discussions with the HHLs Community Associations, Community Development Corporations, and Farmer's Associations from these communities, as well as officials with the DHHL. Table 4-2 summarizes these contacts.

These discussions indicated that local residents feel that the area experiences odors, dust, noise, pests, pathogens, traffic and litter from the presence of the landfill. Residents also wonder whether there are other, unseen air quality effects related to gases. Keaukaha residents expressed concern about impacts to water quality from uncontrolled release of leachate into the groundwater, which could affect inland fishponds and the shoreline, which is heavily used for recreation, fishing and cultural purposes. Such issues are potentially magnified by existing community health problems. These impacts are seen by many in the context of a history of appropriation of Hawaiian land or use of lands adjacent to Hawaiian communities for airports, industrial areas, quarries, ports and solid waste uses, with little or no benefits supplied to the host community. The pattern of such use is a textbook example of a low-income and minority community disproportionately bearing the adverse impacts of public infrastructure.

Table 4-2

Group/Individual	Attendees/Contact Info	Date/place	Concerns/Outcome
Keaukaha Hawaiian Home Lands Community Association	Patrick Kahawaiola'a 959-5080	8/17/11: Presented to Board and 25 attendees	Why County keeping existing landfill open despite water quality effects; alternative sites need examination; revolving door of waste solutions; host community benefits.
Panaewa Hawaiian Home Lands Community Association	Kelly Lincoln 987-9266	7/19/11: Presented to 20 attendees at general membership meeting	Concerned with air quality. Waste to energy should be considered. Confused by County's shifting plans.
Panaewa Community Development Corporation	Donnalyn Johns johnsd@hawaii.edu Attended by members	6/15/11: Presented to 6 board members	Continuing dialogue wanted; community benefits; commitment to reduced waste by County.
Keaukaha-Panaewa Farmers Association	Mele Spencer muspencer@hawaii.rr.com	7/9/11 and 11/7/11: Presented to 8-10 board members	Air quality (odor and other issues); water quality are issues. Community has many health problems.
Hawai'i DHHL	Chair Alapaki Nahale-a 769-2012	7/29/11: Interviewed by phone	Keep community and Department informed.
Hawai'i DHHL	East Hawai'i Commissioner Ian Lee Loy ianleeloy@hotmail.com	7/25/11: Interviewed by phone 10/13/11: Meeting with 40 beneficiaries	Concerns about local impacts; wants community benefits and involvement. Community concerns: waste to energy wanted; effects of farm and residential lots; use of DHHL lands; pests and disease; advisory group should be picked by beneficiaries.

Benefits and costs are often borne differently by “host communities” (those adjacent to an existing or proposed landfill) and the general population that is served by a landfill. Overall, benefits may outweigh costs, but benefits are more diffuse and rarely lead the general community to advocate for new landfills, whereas host communities almost always oppose them (O’Hare, et. al., 1983).

The traditional “top-down” approach of siting solid waste facilities is no longer acceptable to local residents. Local opposition to the construction of solid waste landfills has become the rule and the siting of landfills in the U.S. has become progressively more time consuming and expensive (Jenkins, et. al., 2002). Host community compensation, consisting of cash payments or in-kind gifts that are paid to a community by the developer or local government unit, are common in negotiations between landfill developers and communities (Ibid). This occurrence is most common with privately owned and operated landfills negotiating with local government units such as cities or counties, but it can occur between local governments and affected communities as well. Host community compensation is seen by many economists as introducing a market-like decision-making processes that promotes local decision-making in facility siting. The practice internalizes costs by compensating communities for harms and losses, real and perceived, associated with a landfill. The negotiation process often helps focus attention on scientifically identifying and quantifying impacts, which also benefits the process. Many states find the process so key to landfill siting that they quantify minimum benefits for communities, such as $1 per ton of landfilled waste.

The State of Hawai‘i does not have specific host community compensation requirements, but a review of landfills in other counties indicates that community benefits are strongly considered. The management of Waimanalo Gulch Landfill advertises on its website a $2 million community benefits package focused on Leeward communities. It has provided $1 million in grants to 19 nonprofit organizations and an additional $1 million in supplemental park improvements. A key component was the establishment in 2006 by the City and County of Honolulu of an Oversight Advisory Committee to provide a forum for community members to share concerns about the operation of Waimanalo Gulch Landfill with the commercial operator, Waste Management of Hawaii Inc. The goal of this committee, which meets quarterly, is to promote cooperation between the landfill and the community to ensure that the landfill is operated in a manner sensitive to mutual concerns. (http://www.keepinghawaiiclean.com/about.htm).

On Kaua‘i, the Kekaha Landfill has a Host Community Benefit Program that attempts to “balance the need for safe disposal of solid waste with the sacrifices borne by the host community.” The program involves both mitigation, such as landscaping and revegetation to alleviate visual impacts, and compensation to offset the impacts of residing near the landfill.

(http://www.kauai.gov/Government/Departments/PublicWorks/SolidWaste/KekahaHostCommunityBenefits/tabid/246/Default.aspx)

The County of Kauai reports that at least $810,000 has been appropriated to the Kekaha community as a form of compensation for the expansion of the Kekaha

Landfill. Similar to Honolulu, there is a nine-member citizen's advisory committee appointed by the Mayor and advised by County and Waste Management of Hawaii Inc. staff. Their mission is both to help recommend projects and administer the disbursement of community benefit funds, in accordance with the general consensus of the Kekaha community.

The issue of host community benefits is complicated by the fact that not only Panaewa but much of the rest of the island of Hawaiʻi is affected in some way by whether or not there is an expansion of the SHSL. If it is not built, parts of North Hilo, Hamakua, South Kohala, and the extreme north of North Kona will experience long-haul traffic associated with East Hawaiʻi solid waste. The issue of transporting solid waste from East Hawaiʻi to the Puʻuanahulu Landfill via the towns of Waimea and Waikoloa is controversial, particularly for West Hawaiʻi residents. For some, a principal issue is that waste generated on the east side of the island should be disposed of on the east side. There is resentment that because of its dry climate, West Hawaiʻi may be used as a "dumping ground" for East Hawaiʻi waste and may impact the ability of West Hawaiʻi resorts to attract visitors.

Feasible Mitigation Measures

- If the County chooses to proceed with new landfill planning, it should consider convening a landfill planning committee composed of agency personnel, neighboring land managers, and Panaewa/Keaukaha community members to determine whether and how the SHSL can be expanded in an environmentally and socially acceptable way. Early and meaningful involvement in project planning is a cornerstone of genuine environmental justice efforts. Mitigation for nuisance impacts and community benefits need to be considered.
- Existing community groups can provide recommendations regarding the committee makeup to help assemble a legitimate and accountable membership.
- A useful starting point for a committee would be to come to a consensus on the nature and severity of various impacts.
- Based on impacts that are identified by this process, mitigation for odor, gases, pests litter and other pertinent impacts that have been successfully employed in other Hawaiʻi and mainland landfills can be investigated by the committee for technical feasibility. This might include pointing trucks into the wind while dumping, smaller working faces, redundant moveable fences to catch windblown litter, dedicated litter patrol crews, early and ongoing soil cover for active areas on windy days, and odor misters.

5.1 Background

The County owns, operates, and maintains 21 transfer stations. Of the 21, nine deliver the residential solid waste to the SHSL, with the remainder hauling to WHSL. The 2009 IRSWMP provided an evaluation of the use of a transfer station at the SHSL and hauling waste to WHSL. The IRSWMP estimated a cost of $1,852,478 annually to haul from SHSL, with an additional $106,000 to haul directly from Pahoa TS to WHSL.

5.2 Updated Cost Estimate

A review and update to the planning level truck hauling cost estimate contained in Appendix F of the IRSWMP was undertaken. The details for the revised estimate can be found in Appendix H.

This effort involved discussions with County staff to validate and/or update the underlying assumptions and inputs used in the IRSWMP estimate including the following:

- Travel distance for the Saddle Road route
- Estimated average travel speed for the round trip
- Average payloads
- Fuel cost
- Fuel consumption rate
- Lease cost for tractors and trailers
- Number of tractors and trailers that are leased versus owned
- Labor rates
- Tire life and cost
- Applicability of federal, state and local taxes
- Non-productive labor time
- Operating days and hours
- Insurance costs
- Contingency

The review of the above input areas with County staff resulted in several changes from the assumptions and input used for the IRSWMP estimate including the following:

Table 5-1

Item	IRSWMP Estimate	Updated Estimate
Average Payload	18 tons	20 tons
Fuel Consumption Rate	5 mpg	4.5 mpg
Fuel Cost	$4.00/gallon	$3.10/gallon
Replacement Tire Cost	NA	$450/each
Tire Life	NA	30,000 miles
Work Days per Week	7	6
Insurance	$1000/vehicle	$0/vehicle
License and Fees	$0/vehicle	$500/vehicle
Labor Rates	$62,206/year/driver	$63,000/year/drive
Non-Productive Time	0%	21.1%
Contingency	0%	10%
Miscellaneous Cost Allowance	0%	2%

The original model worksheet used for the cost estimate provided in the IRSWMP was unavailable. A revised model was created to include more recent data, additional inputs, and other pertinent information. The output for this model is provided in Appendix H and includes the following principal spreadsheets:

Table 1 – This spreadsheet generates the number of roundtrips per trailer per day which in this case is two.

Table 2 – This spreadsheet generates the number of drivers, tractors and trailers required and the total hauling miles and miles per tractor and trailer traveled based on the number of tons per year of waste to be hauled, the roundtrip travel distance, and the days of hauling operations per week. This spreadsheet is run using the 2011 waste generation forecast of 64,088 tons.

Table 3 – This spreadsheet is a summary of the hauling cost estimate contained in Table 4 and breaks the cost between the major cost categories, applies the contingency, adds for miscellaneous costs and displays the hauling cost as a gross annual cost as well as a cost per mile and a cost per ton-mile.

Table 4 – This spreadsheet uses input data from Table 2 and is the detailed hauling cost estimate by cost category including labor, fuel, tires, equipment maintenance and repairs, equipment lease cost and insurance, license and taxes.

The results of the updated planning level estimate indicate that the cost of hauling waste to the WSLF in 2011 would be around $20.34 per ton at 2011 rates for labor and fuel, which is considerably lower than the estimated cost of $24.03/ton in the IRSWMP. This calculation is based on 100 percent leased equipment. As the number of County owned trailers and tractors increase from zero, the cost of hauling would drop. If all the trailers and tractors were County-owned, the cost would drop to approximately $16.52 per ton. These reduced costs do not include provisions for a sinking fund for replacement equipment. On the other hand, if the cost of diesel fuel rises to $5.00/gallon, or approximately 61 percent above the assumed 2011 price of $3.10, the hauling cost per ton in 2011 would rise approximately $3.60 per ton to around $23.95 per ton for the scenario of 100 percent leased equipment. So, as expected, the hauling cost is very sensitive to fuel pricing.

There are uncertainties regarding fuel and labor costs, fuel consumption rate, average payload tonnage and transport speeds that can be achieved, the mix of owned and leased equipment, and other underlying assumptions that are generally inherent in any forecast modeling. However, the estimates of hauling costs presented as part of this Study and the IRSWMP suggest that it would be reasonable for the County to expect that a hauling operation would have a unit cost rate in the range of from $20 to $24 per ton in 2011. This rate does not include the cost to load the trailers.

5.3 Estimated Cost to Landfill East Side Waste in WHSL

To determine the estimated planning level cost of landfilling eastside waste in the WHSL, it is necessary to add the cost of loading trailers, hauling and landfilling. Assuming, based on experience from other projects, that the added cost to load trailers by compactor for the haul to WHSL is approximately $1/ton more than the cost of toploading waste in trailers for hauling to the SHSL, the estimated cost in 2011 dollars for waste disposal at the WHSL is estimated to be in the range of $21/ton to $25/ton (load plus haul costs) plus the cost of landfilling at the WHSL. The county’s contract with Waste Management, Inc. has a stepped rate scale where the unit cost for landfilling decreases as tonnage increases up to 350 tons per day. The addition of more than 200 tons per day of waste from the east side of the island at WHSL would result in a conservatively estimated average rate decrease for all the tons of waste disposed at WSHL of around $10/ton. If in 2011 it is assumed that the average waste tonnage generated on the west side of the island is around 280 tons per day, and an average of around 205 tons per day were hauled from the east side of the island (485 tons total disposal per day), the total disposal cost for the 205 tons could be evaluated as falling in the following range, assuming the cost of loading plus hauling across island ranges from $21/ton to $25/ton:

Loading/Hauling cost + Landfilling cost – Saving in Landfilling Cost for Westside Waste

Low end of range:

$21/ton + $45.54/ton - 280 tons x $10/ton savings/205 tons = $52.88/ton

Say $53/ton

High end of range:

$25/ton + $45.54/ton - 280 tons x $10/ton savings/205 tons = $56.88/ton

Say $57/ton

The cost range of $53 to $57 per ton is a net cost range and reflects the estimated savings derived from a lower rate for landfilling of the westside tonnage. If that savings is excluded from the estimate the eastside tonnage landfilling rate, the cost range for landfilling eastside tonnage at the WHSL would increase to $67 to $71 per ton.

It must be noted that a review of the contract terms of the County's contract with WM was not in the scope of work of this study. The durability of the current operational cost structure over an extended period has not been evaluated.

Section 6
LANDFILL FEASIBILITY ASSESSMENT

6.1 Technical Feasibility

Several steps would need to be evaluated and completed in order to begin disposal in the expansion area of the landfill. Generally, the technical aspects of the landfill expansion (e.g. soils, liner installation, leachate collection and management, etc.) can be met through engineering practices. Additionally, consideration would need to be given to other aspects that would be affected by the expansion. These are discussed in more detail in Section 4. The landfill expansion would require rezoning of the current surrounding land, permitting and EIS work, design and construction, and an operations and monitoring plan.

Land in or near the expansion area would need to be rezoned in order to comply with local requirements to allow for waste disposal. A majority of the land required for the expansion is government owned and would need to be acquired through the steps outlined in Section 4. Assuming the County could resolve any issues regarding land use, an EIS and permit application would be required for DOH review. The EIS would need to address many of the concerns identified in Section 4. A permit application would need to demonstrate through calculation that all technical requirements per HAR-11-58.1 for waste disposal and leachate management are satisfied. If the County pursued leachate treatment via wetlands, an extensive demonstration would likely be required since these would be the first leachate treatment wetlands in Hawai‘i.

The area planned for the expansion is currently an active quarry. Modifications to the bottom topography would be required to create an adequate subgrade for the bottom of the landfill. This would be completed using existing quarry material to meet the required grades. Due to the nature of the rocky material on site, processing of the materials would be required to crush and screen the rock to meet specified gradations for use as construction material. The existing quarry activities demonstrate that this is feasible. The quarry currently produces material that is used construction around the area. The level of effort required to excavate and process the material would depend on the product specifications. The HWA report in Appendix C identifies additional construction material located within the area.

For purposes of this Study, constructing 3 to 1 side slope embankments using quarried material was reviewed as the option for landfill construction. As noted in Section 2, alternatives to this might include excavating to create 3 to 1 in situ rock side slopes or leaving the existing more or less vertical side walls of the quarry. Regardless of the shape of the sides of the landfill, soil embankments will be required. Based on previous construction in the area, processed quarry material shows the capability to maintain 3 to 1 side slopes. Areas where fill material is placed either at the base or side slopes would have to meet compaction requirements. A complete analysis would be required to determine the soil requirements and placement to meet stability needs, but we believe that these requirements can be met.

It does not appear that a readily available low permeable soil is available for use in the composite liner system. Further evaluation would be needed on the ability to amend local borrow soils to meet regulatory requirements (Appendix C). Therefore, it is likely that the composite liner system will require the use of a GCL as the low permeable layer. This would require approval from the DOH as an equivalent to a 2-foot layer of low permeable soil. The composite liner system in use at the WHSL includes a GCL. The HDPE geomembrane liner and geocomposite drain net would be installed on the 3 to 1 side slope with an anchor trench at the top. An evaluation of the size of the anchor trench would need to demonstrate the stability of the geosynthetic materials. If aggregate drainage material is utilized, puncture analysis of the geocomposite would be required to verify that rocks and stone aggregates would not damage the geomembrane liner.

Minimizing leachate generation would depend on the use of some form of low permeable temporary cover material. The leachate dewatering pump would be sized according to the expected leachate generation volumes. To manage leachate generation, this Study selected subsurface wetlands as the leachate treatment options for further evaluation. Leachate treatment via wetlands has been utilized at other landfills in the United States. The calculations provided in this Study (Appendix E) demonstrate wetland treatment for the SHSL provides an adequate treatment option that would allow the County to safely discharge its treated leachate if approved by the DOH.

Alternatives to leachate treatment via wetlands could be further explored or there may be a potential to combine different treatment options. Potential combinations include wetland treatment with recirculation or bioreactor. Treatment using recirculation or a bioreactor would likely require the installation of a GCCS to capture the accelerated LFG generated. Another potential combination is a wetland treatment system that utilizes the relatively nearby WWTP for discharge instead of an onsite infiltration gallery. The WWTP may be more willing to accept treated leachate versus raw leachate. In addition, the location of the WWTP near the SHSL would allow for the installation of a direct piping system versus trucking leachate to the WWTP.

Methods to ensure that the environmental requirements for controlling LFG and protecting groundwater are being maintained could be completed through the use of monitoring wells or probes. The monitoring network currently in place would likely need to be increased due to the location of the expansion. The extent to which the monitoring network would need to expand would be evaluated during permitting.

The SHSL expansion would include new equipment and site operations that do not currently exist. New equipment and structures that would be installed as part of the expansion include pumps, piping, underground vaults, and manhole structures for managing leachate. In addition, new monitoring plans would need to be developed for ensuring an optimal and efficient waste and leachate management system. The implementation of the lined landfill, leachate management systems, leachate treatment system, and any additional systems (e.g. GCCS) would require operational and safety training for existing and new landfill staff.

Our overall assessment is that there are no insurmountable technical challenges with a new landfill.

6.2 Financial Assessment

The financial assessment for the expansion takes into account costs associated with land issues, permitting, cell and leachate treatment construction, operation, and closure and post closure care.

The costs associated with land use issues and permitting are based on the level of effort that is normally required to complete formal documents for submittal and any follow up that may be necessary. However, because of the additional level of effort that may be required to demonstrate wetland as a treatment option, the estimated cost provided may need to increase accordingly.

Landfill cell construction costs are based on information provided from Engineering Partners of Hilo, Hawai‘i and a geosynthetics contractor in State of Washington that does extensive work in Hawai‘i. Equipment costs such as pumps and other controls are based on previous construction projects around the United States and include an increase in costs due to shipping and installation in Hawai‘i. Cell 1 would be expected to have a high cost because of the necessary infrastructure improvements for piping and leachate treatment. Not included in the cell construction costs are site upgrades that would include power lines and site access roads. Some of site upgrades would be required regardless of the expansion.

Costs that are associated with current site operations such as equipment upkeep and site monitoring would continue. However, annual operational costs would likely increase because of the leachate management system, landfill cover maintenance, and new equipment (pumps, meters, etc.). Electrical costs would likely be limited at first, but as additional electrical equipment are installed, the landfill's electrical costs would increase. The leachate management and treatment system would require additional monitoring and documentation which would add to the cost of the landfill's monitoring program. The increase in the landfill management systems including the leachate system and future GCCS would also require additional staff time.

The method and level of technical sophistication of a leachate treatment system would determine the cost for leachate treatment. This Study evaluated a constructed wetland for leachate treatment. This method has minimal construction costs and relatively low operating costs because of the limited equipment involved.

At the end of the life of the expansion area, a final cover system will need to be installed. Generally, this consists of installing permanent cover materials, storm water controls, and additional collection points for the GCCS. The County would also be required to perform annual monitoring and site maintenance for the duration of the post closure period which is at least 30 years beyond final cover placement.

Appendix G provides a more detailed cost estimate for landfill permitting, design and construction, closure and wetland construction.

Tables 6-1 and 6-2 show estimated costs in 2011 dollars for the various work required to site, permit, complete additional site upgrades, construct the landfill expansion and to construct upgrades for the WWTP for two scenarios. Table 6-1 provides estimated costs where the DOH has approved construction of the wetland treatment contingent upon the WWTP as the back-up treatment option. Table 6-2 assumes the DOH does not approve, or the County forgoes permitting and construction of the wetland and pipes leachate to the WWTP for treatment and disposal.

Table 6-1

Item	Cost
Land Uses, Environmental Assessment, Zoning[1]	$1,000,000
Road/Site/Power Upgrades	$250,000
Permitting[2]	$730,000
Cell 1 Design and Construction	$9,970,000
Subsequent Design and Construction	$42,900,000
GCCS Construction	$4,700,000
Landfill Closure	$18,000,000
WWTP Cost[3]	$17,000,000
Wetland Construction	$590,000
Capital Cost Subtotal	$95,140,000
Landfill Operations – w/out Post Closure Care	$2,860,000
WWTP annual stand-by fee[4]	$325,000
Post-Closure Care (annual)	$240,000
Annual Cost Subtotal	$3,425,000
Total Cost ($/ton)	$82

1. Planning estimate only. Additional costs may occur during Environmental Assessment work and other unforeseen circumstances.
2. The permitting process may take 2 to 3 years for approving the alternative liner design and leachate treatment via wetlands and discharge. Additional costs may occur for additional work and time that may be required.
3. Cost from 2003 Brown and Caldwell study of $13.3 million and escalated at 3%.
4. The landfill may need to reserve capacity at the WWTP in the event leachate must be transferred for treatment and disposal. The cost is estimated at $5/ton for 65,000 tons in 2011.

Table 6-2

Item	Cost
Land Uses, Environmental Assessment, Zoning[1]	$1,000,000
Road/Site/Power Upgrades	$250,000
Permitting[2]	$730,000
Cell 1 Design and Construction	$9,970,000
Subsequent Design and Construction	$42,900,000
GCCS Construction	$4,700,000
Landfill Closure	$18,000,000
WWTP Cost[3]	$17,000,000
Pipeline from Landfill to WWTP[4]	$3,250,000
Capital Cost Subtotal	$97,800,000
Landfill Operations – w/out Post Closure Care	$2,860,000
WWTP annual treatment fee[5]	$1,460,000
Post-Closure Care (annual)	$240,000
Annual Cost Subtotal	$4,560,000
Total Cost ($/ton)	$100

1. Planning estimate only. Additional costs may occur during Environmental Assessment work and other unforeseen circumstances.
2. The permitting process may take 2 to 3 years for approving the alternative liner design and leachate treatment via wetlands and discharge. Additional costs may occur for additional work and time that may be required.
3. Cost from 2003 Brown and Caldwell study of $13.3 million and escalated at 3%.
4. Design and construction costs for a direct pipeline to the WWTP is estimated at $250 per foot at 13,000 feet.
5. Based on $0.10 per gallon for treatment of 40,000 gpd of raw leachate (Section 3).

The cost estimate for landfilling if the County received approval for wetland treatment and treated leachate discharge through infiltration galleries is approximately $82/ton. Assuming an estimating accuracy range of -15 percent to +30 percent, the estimated costs could range from $70/ton to $107/ton.

If the County were to forgo permitting of the wetland treatment option and directly pipe leachate to the upgraded WWTP, the estimated cost of landfilling is approximately $100/ton. Assuming an accuracy range of -15 percent to +30 percent, the estimated costs could range from $85/ton to $130/ton.

7.1 Summary

The Department of Environmental Management is evaluating the feasibility of expanding landfilling of MSW beyond the footprint of the current SHSL versus transporting waste to the WHSL for disposal. The current SHSL was constructed prior to the requirement of Subtitle D. The expansion of the SHSL would need to conform to the current RCRA regulations and the landfill would have to be constructed with a base liner and leachate collection system. In addition to the construction requirements, landfill leachate collection, treatment and disposal are necessary; and as the landfill would continue to grow, a landfill gas collection and control system would likely be required.

The SHSL is located on the east side of the City of Hilo in an area of mixed industrial, agriculture and airport use. Adjacent land includes State property that is or was leased for quarrying. Vacated quarries offer sufficient area for expansion of the landfill and geotechnical aspects of the site can be engineered to accommodate landfill construction. Although expansion of the landfill would be away from the existing airport, there is substantial potential for conflict with surrounding land uses. These include DHHL farm lots, the KMR, and especially the County Skeet Range, particularly if large areas are required for leachate treatment. The County would need to work with the State to obtain ownership of the properties and have the area appropriately rezoned to accommodate the intended use.

The SHSL has operated near the Hilo International Airport without incident for many years. Although the landfill expansion could be considered a continuation of the existing facility (a lateral expansion), and new waste cells would be constructed further away from the airport than the current operational area, it is possible that the DOH and the FAA would deem the expansion area to be a new landfill. FAA rules prohibit construction of new municipal solid waste landfills within six miles of a public airport, and this adds considerable uncertainty regarding the County's ability to permit a new landfill area outside the current SHLF footprint.

In our opinion, engineering issues associated with the site such as appropriate side slope for liner placement and unavailability of low permeable soil for sub-base grades can be handled through engineering design and engineered products. And it seems entirely feasible to construct a landfill expansion in conformance with Subtitle D regulation. However, because of the high annual rainfall of the region potentially large amounts of leachate would be generated. Landfill operations could be managed to significantly decrease the volume of leachate that would need to be collected, treated and disposed. These management options would include:

- The use of temporary daily cover materials that reduce rainfall infiltration thus shedding precipitation as clean stormwater instead of generating additional leachate
- Incorporating leachate recirculation into the landfill design and operations to manage and store leachate within the waste mass
- Constructing man-made wetlands at the landfill site to treat leachate on-site with potential ultimate disposal through infiltration galleries on the SHSL property.

While the study suggests that constructed wetlands would provide a technically feasible and relatively inexpensive treatment system for leachate, this form of treatment is a new concept to the DOH. Discussions with staff of the DOH Solid Waste Branch indicate that any alternative leachate management options would need to be approved through the DOH and most likely would require the SHSL to have a recognized treatment technology, such as a WWTP, as a backup to the constructed wetland treatment system until the new system is proven to be viable. Discussions with DOH staff concerning expansion of the SHSL were conceptual for this feasibility study, and DOH staff has stated that they cannot comment on the possible approval of any alternative treatment technology until a landfill permit application is submitted and reviewed by DOH.

The County's existing Hilo WWTP does not have the capacity to take the quantity of leachate that would be generated at the landfill without significant and costly upgrades that would also take considerable time to design and construct. In addition, discussions with staff responsible for the County's wastewater treatment program indicate that the Hilo WWTP has since August 2010 been experiencing apparent effluent toxicity issues as measured by WETT. Even if the WWTP was expanded, the addition of leachate to the plant would further complicate the resolution of the plant's failure to pass the WETT standard. Finally, the addition of leachate to the WWTP, if allowed, would very likely trigger a requirement to institute and administer a costly wastewater pretreatment program which here to fore has been avoided given the plant's throughput being at or below the industrial wastewater discharge program threshold level of 5 million gallons per day (MGD) as there are currently no Significant Industrial Users.

The DOH does not have standards for discharging treated leachate into the ground using injection or infiltration galleries. Therefore, if onsite treatment of leachate were included, new discharge standards would need to be developed and adopted by the DOH for this, which could be problematic not the least from a schedule standpoint.

The expansion of the SHSL involves a number of issues that would need to be addressed through the environmental review and permitting process. Landfill expansion would require the completion of an EIS along with a landfill permit application. These two processes are typically lengthy and very likely would require at least two to three years to complete.

The cost to develop and operate a new landfill over 30 years was evaluated as part of the study for two different leachate treatment options. For the option using an onsite constructed wetland with a WWTP backup, we estimate the cost per ton of waste landfilled in 2011 to range from $70/ton to $107/ton. For the option only using a WWTP for leachate treatment we estimate the cost per ton of waste landfilled in 2011 to range from $85/ton to $130/ton.

As part of this Study, the cost to transport and dispose of solid waste currently going to the SHSL at the WHSL was examined. The cost of transporting the waste was previously completed under the IRSWMP and this information was reviewed and the cost updated as part of this Study. In addition, a planning level cost estimate range was prepared that includes the cost to load, haul and dispose of waste at the WHSL. This cost is estimated to range from $53/ton to $57/ton landfilled at WHSL where the cost includes the savings derived from achieving a lower disposal rate at the WHSL for waste generated on the west side of the County.

The cost estimates for landfilling east County waste at SHLF and at WHSL indicate that the County would realize a substantial cost savings (between $29,000,000 to $134,000,000 for 1,748,000 tons landfilled over twenty years) with disposal of east Hawai'i waste at the WHSL. However, in our opinion, taking into account the volatility of fuel prices and other pricing uncertainties, as well as uncertainties about the stability of the long term contracted cost to operate the WHSL, a reasonable expected range for the savings would be on the order of $20,000,000 to $30,000,000 over the twenty years 2013-2033 in 2011 dollars, or an average of $1,000,000 to $1,500,000 per year. The reader is reminded that these are planning level estimates.

The Final Environmental Impact Statement (FEIS) for Construction and Operation of the East Hawai`i Regional Sort Station, prepared by the County of Hawai`i Department of Environmental Management in February 2004, included a traffic impact assessment (Appendix E) that addressed the impact of truck traffic hauling waste from the Sort Station in Hilo to the WHSL. It would be prudent to update this study to verify its conclusions regarding the impact of this truck traffic.

7.2 Conclusions

Based on the findings of this study, we conclude that while it is technically feasible to operate a landfill adjacent to the existing SHSL, it is neither practical nor economically sound to proceed with design and permitting a landfill expansion in Hilo. Permitting constraints, land use constraints, and leachate management issues all present significant and, perhaps, insurmountable obstacles. Furthermore, based on our cost estimates, trucking and disposal of waste at the existing West Hawaii Sanitary Landfill provides a potentially feasible and more cost effective disposal alternative. It is recommended that the traffic impact assessment prepared for the FEIS for the East Hawai`i Regional Sort Station (February 2004) be updated with respect to trucking of waste to the WHSL.

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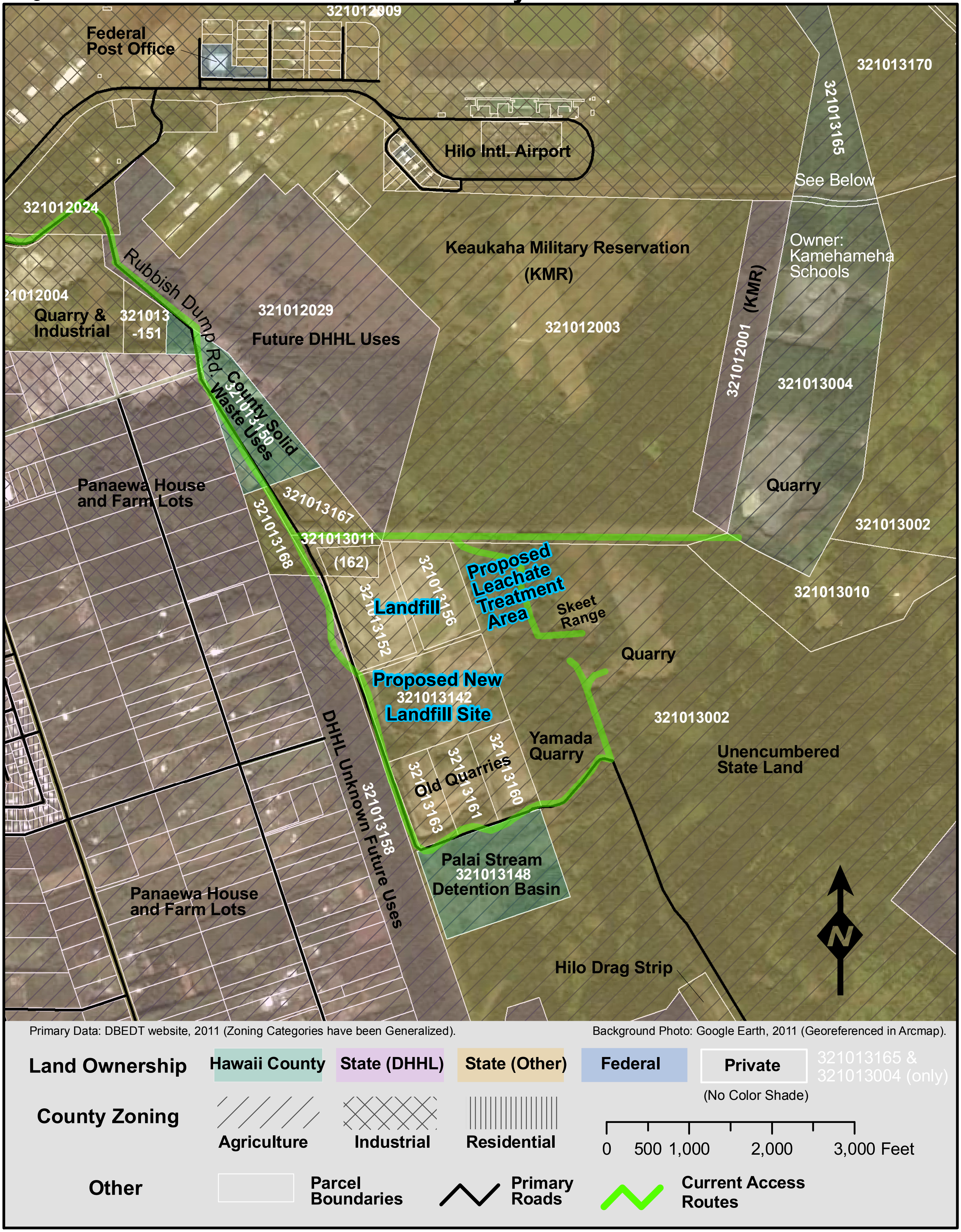
Appendix A
SITE DRAWINGS

Figure 1.

South Hilo Sanitary Landfill Area



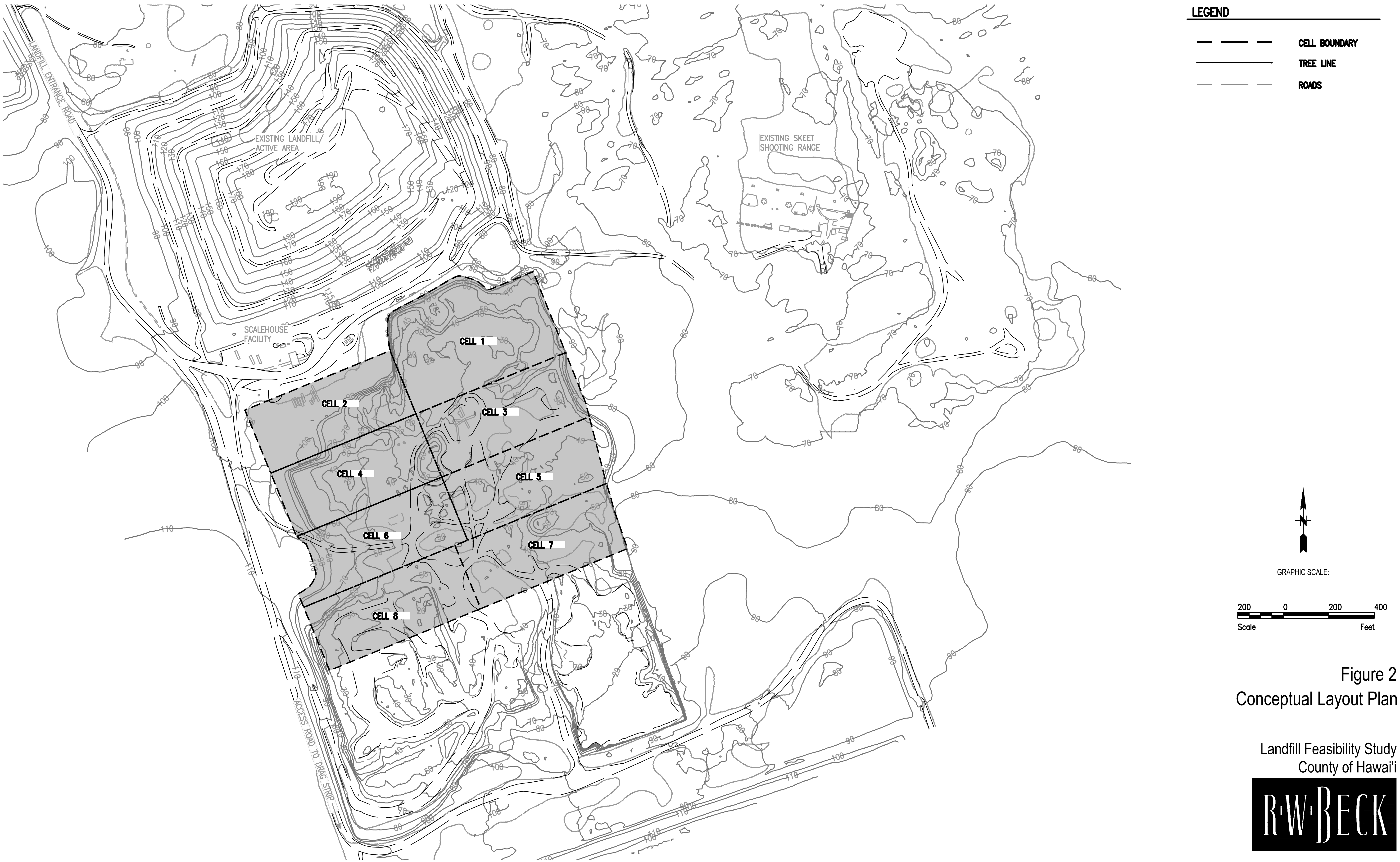


Figure 2
Conceptual Layout Plan

Landfill Feasibility Study
County of Hawai'i

R·W·BECK
An SAIC Company

COOK, MICHAEL J. – 9/27/2011 11:18:01 AM – R:\Seattle\011304 HAWAII CNTY, DEPT ENV MGMT\Hilo Landfill Feasibility Study\Data-Analytical\CAD\Figure 2_Conceptual Plan.dwg

Figure 3
Conceptual Leachate Management Plan

Landfill Feasibility Study
County of Hawai'i

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Appendix B
WASTE PROJECTIONS AND CELL DEVELOPMENT

Table Appendix B.1

South Hilo Sanitary Landfill Waste Projections

Assumptions			2010		2007						
			208,641 tons*		97,738 **						
Year		Population[1]	Defacto Pop.[2]	Employment[3]	Ton/P-E[4]	Diversion Rate*,[5]	WHSL*,[6]	SHSL*,[6]	SHSL - TS[6]	SHSL - Comm.[6]	
2010		185,079	1.12	101,149	0.729	20%	62%	38%	48.5%	51.5%	
2011	2015	1.5%	1.12	1.15%	1.0%	25%	60%	40%	48.5%	51.5%	
2015	2020	2.1%	1.12	1.52%	1.0%	25%	60%	40%	48.5%	51.5%	
2020	2025	1.8%	1.12	1.44%	1.0%	25%	60%	40%	48.5%	51.5%	
2025	2030	1.5%	1.12	1.34%	1.0%	25%	60%	40%	48.5%	51.5%	
2030	2035	1.3%	1.12	1.30%	1.0%	25%	60%	40%	48.5%	51.5%	

Year	Population	Defacto Pop.	Employment	Ton/P-E	Waste Generation[&]	Waste Diverted	Waste Disposed	SHSL Disposal	SHSL - TS	SHSL - Comm.
2010	185,079	207,288	101,149	0.73	208,641	42,188	166,453	63,455	30,776	32,679
2011	187,855	210,398	102,312	0.74	213,628	53,407	160,221	64,088	31,083	33,005
2012	190,673	213,554	103,489	0.74	218,734	54,684	164,051	65,620	31,826	33,794
2013	193,533	216,757	104,679	0.75	223,963	55,991	167,972	67,189	32,587	34,602
2014	196,436	220,008	105,883	0.76	229,318	57,330	171,989	68,795	33,366	35,430
2015	199,383	223,309	107,100	0.77	234,801	58,700	176,101	70,440	34,164	36,277
2016	203,570	227,998	108,728	0.77	241,649	60,412	181,237	72,495	35,160	37,335
2017	207,845	232,786	110,381	0.78	248,698	62,175	186,524	74,609	36,186	38,424
2018	212,209	237,675	112,059	0.79	255,955	63,989	191,966	76,786	37,241	39,545
2019	216,666	242,666	113,762	0.80	263,425	65,856	197,569	79,027	38,328	40,699
2020	221,216	247,762	115,491	0.81	271,115	67,779	203,336	81,334	39,447	41,887
2021	225,198	252,221	117,154	0.81	278,417	69,604	208,813	83,525	40,510	43,015
2022	229,251	256,761	118,841	0.82	285,916	71,479	214,437	85,775	41,601	44,174
2023	233,378	261,383	120,553	0.83	293,618	73,405	220,214	88,086	42,721	45,364
2024	237,579	266,088	122,288	0.84	301,529	75,382	226,147	90,459	43,872	46,586
2025	241,855	270,878	124,049	0.85	309,653	77,413	232,240	92,896	45,055	47,841
2026	245,483	274,941	125,712	0.85	317,272	79,318	237,954	95,181	46,163	49,018
2027	249,165	279,065	127,396	0.86	325,077	81,269	243,808	97,523	47,299	50,224
2028	252,902	283,251	129,103	0.87	333,075	83,269	249,806	99,923	48,462	51,460
2029	256,696	287,500	130,833	0.88	341,270	85,318	255,953	102,381	49,655	52,726
2030	260,546	291,812	132,586	0.89	349,667	87,417	262,250	104,900	50,877	54,024
2031	263,934	295,606	134,310	0.90	357,755	89,439	268,316	107,326	52,053	55,273
2032	267,365	299,448	136,056	0.91	366,030	91,507	274,522	109,809	53,257	56,552
2033	270,840	303,341	137,825	0.92	374,496	93,624	280,872	112,349	54,489	57,860
2034	274,361	307,285	139,617	0.93	383,158	95,790	287,369	114,947	55,749	59,198
2035	277,928	311,279	141,432	0.93	392,020	98,005	294,015	117,606	57,039	60,567

* Model has been calibrated using FY 2010 tonnage data. The 2010 values have been updated to reflect the updated tonnage data. Forecasts projected from this value

** Last known value was in 2007 - Forecasts projected from this value

[1] 2010 Value from 2010 U.S. Census Data and population projections are from the 2009 Update (August - online) of the Population and Economic Projections for the State of Hawaii to 2035 - DBEDT 2035 Series

[2] Defacto Ratio obtained from 2009 County of Hawaii IRSWMP - average over all years in study

[3] Employment for 2007 and Employment projections are from the 2009 July Update (August - online) of the Population and Economic Projections for the State of Hawaii to 2035 - DBEDT 2035 Series

[4] Value for 2010 is obtained from 2009 County of Hawaii IRSWMP - Value differs based on parameters used to calculate and is the average from 2000 to 2007

[5] Diversion rates from the 2009 County of Hawaii IRSWMP - updated using current diversion rates

[6] Percent of wastes break down provided in the 2009 County of Hawaii IRSWMP

[&] Waste Generation is calculated by (Ton/P-E) * (Population plus Employment)

Table Appendix B.2

South Hilo Sanitary Landfill Waste Filling Projections

Assumptions

Daily Cover Material:	Daily Cover % total volume:	20%			
Intermediate Cover Material:	Intermediate Cover % total volume:	20%			
Compaction Equipment:	Compaction Rate:	1,500	lbs/cy	0.75	tons/cy
	Airspace Utilization Factor (AUF):	1,200	lbs/cy	0.60	tons/cy

Cell Volumes (CY):							
Cell 1	848,803	Cell 3	846,435	Cell 5	799,994	Cell 7	420,831
Cell 2	477,933	Cell 4	796,256	Cell 6	784,532	Cell 8	416,675

Year	SHSL Disposal	Waste Volume (cy)	Airspace Consumed (cy)	Total Consumed (cy)	Airspace Remaining				
2008					910,000	*as noted in the 2008 SWT report based on current survey at that time.			
2009		137,233	176,433	176,433	733,567				
2010	63,455	84,607	105,758	282,191	627,809				
2011	64,088	85,451	106,814	389,005	520,995				
2012	65,620	87,494	109,367	498,372	411,628				
2013	67,189	89,585	111,982	610,354	299,646				
2014	68,795	91,727	114,659	725,013	184,987				
2015	70,440	93,921	117,401	842,414	67,586				
2016	72,495	96,660	120,824	963,238	795,565	Current Fill Area Consumed - begin:			Cell 1
2017	74,609	99,479	124,349	1,087,587	671,216				
2018	76,786	102,382	127,977	1,215,564	543,239				
2019	79,027	105,370	131,712	1,347,277	411,526				
2020	81,334	108,446	135,557	1,482,834	275,969				
2021	83,525	111,367	139,208	1,622,043	136,760				
2022	85,775	114,366	142,958	1,765,001	471,735	Cell 2	Total tons Cell 1	509,282	
2023	88,086	117,447	146,809	1,911,810	324,926				
2024	90,459	120,612	150,764	2,062,574	174,162				
2025	92,896	123,861	154,827	2,217,401	19,335				
2026	95,181	126,909	158,636	2,376,037	707,134	Cell 3	Total tons Cell 2	286,760	
2027	97,523	130,031	162,539	2,538,576	544,595				
2028	99,923	133,230	166,538	2,705,113	378,058				
2029	102,381	136,508	170,635	2,875,748	207,423				
2030	104,900	139,867	174,833	3,050,582	32,589				
2031	107,326	143,102	178,877	3,229,459	649,968	Cell 4	Total tons Cell 3	507,861	
2032	109,809	146,412	183,015	3,412,474	466,953				
2033	112,349	149,798	187,248	3,599,722	279,705				
2034	114,947	153,263	191,579	3,791,301	88,126				
2035	117,606	156,808	196,010	3,987,311	692,110	Cell 5	Total tons Cell 4	477,754	
2036	118,782	158,376	197,970	4,185,282	494,139				
2037	119,970	159,960	199,950	4,385,232	294,189				
2038	121,170	161,560	201,950	4,587,181	92,240				
2039	122,381	163,175	203,969	4,791,150	672,803	Cell 6	Total tons Cell 5	479,996	
2040	123,605	164,807	206,009	4,997,159	466,794				
2041	124,841	166,455	208,069	5,205,228	258,725				
2042	126,090	168,120	210,149	5,415,377	48,576				
2043	127,351	169,801	212,251	5,627,628	257,155	Cell 7	Total tons Cell 6	470,719	
2044	128,624	171,499	214,373	5,842,002	42,782				
2045	129,910	173,214	216,517	6,058,519	242,940	Cell 8	Total tons Cell 7	252,498	
2046	131,209	174,946	218,682	6,277,201	24,257				
2047	132,522	176,695	220,869	6,498,071	(196,612)		Total tons Cell 8	250,005	
							Total tons Cells 1-8	3,234,875	

Appendix C
GEOTECHNICAL EVALUATION

HWA GEOSCIENCES INC.
Geotechnical & Pavement Engineering • Hydrogeology • Geoenvironmental • Planning & Permitting • Inspection & Testing

August 31, 2011
HWA Project No. 2011-047-21

SAIC Energy, Environment and Infrastructure LLC
1001 Fourth Avenue, Suite 2500
Seattle, Washington 98154-1004

Attention: Mr. Kyle Rhorer, P.E.

Subject: **GEOTECHNICAL ASSESSMENT OF LOCAL BORROW SOIL**
South Hilo Sanitary Landfill Expansion
Hilo, Hawaii

Dear Mr. Rohrer:

HWA GeoSciences Inc., (HWA) conducted a limited field and laboratory assessment of local borrow soils with regard to their potential use as daily cover, or low permeability landfill liner soil, during the construction of a new cell at the South Hilo Sanitary Landfill (SHSL) in Hilo, Hawaii. The field investigation consisted of a visit to the project site (see Figure 1) and geotechnical reconnaissance within the immediate area surrounding the site; including observations and sampling of selected exposures of potential borrow soils. Samples of potential borrow soil obtained during our reconnaissance were evaluated at our laboratory to determine relevant engineering properties. Additionally, examination and evaluation of existing data related to previous geotechnical studies in the area, information from the published USDA or USGS sources, available groundwater data, and information regarding local material resources, was performed.

PROJECT CONDITIONS

SITE DESCRIPTION

The South Hilo Sanitary Landfill is located east of urban Hilo in an area of mixed industrial, agricultural and airport use (see Figures 1 and 2). The SHSL is owned by the County of Hawai'i and has been in operation since the 1960's. It is expected to reach capacity at current utilization rates by the end of 2013.

A site adjacent to the existing SHSL that currently accommodates an operating quarry has been identified as a potentially viable site for a landfill or landfill expansion (Geometrican Associates Inc., 2008). The quarry excavation is approximately 60 feet deep and has near vertical walls on three sides with

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room to expand to the south. The quarry excavation, after some modification, appears to be the most likely site for a new landfill in east Hawaii (Geometrician Associates Inc., 2008).

ISLAND AND LOCAL GEOLOGY

The Island of Hawaii is the largest in the Hawaiian Archipelago with an approximate area of 4,030 square miles. The island formed by the coalescence of five shield volcanoes (from north to south: Kohala, Mauna Kea, Hualalai, Mauna Loa and Kilauea) that accreted into a broad dome extending from the Hawaiian Ridge along the floor of the Pacific Ocean. The Hawaiian Ridge formed as the result of the Pacific Plate passing over a “hot spot” that is fixed in the underlying mantle (Williams, et al, 1979). As the plate moved progressively from southeast to northwest over the “hot spot” volcanic activity has waned in the north and central portions of the island over time. During recent geologic time, only the southern portion of the island experiences active volcanism. Offshore of the southeast coastline of Hawaii an active submarine volcano is building and may emerge above sea level a few hundred thousand years in the future.

At the SHSL site, the near surface is underlain by lava flows belonging to the Kau Basalt. Northwest of the SHSL project site, the surface slopes of Mauna Kea Volcano are covered by a veneer of volcanic ash (Pahala Ash) that overlies lava flows of the Hamakua Volcanic Series (Wolfe, et al, 1996). The Hamakua Volcanic Series consists of basaltic lava flows and some interbedded volcanic tephra (ash and cinder) deposits from eruption during the shield forming stage of Mauna Kea Volcano. Locally, the lava flows are mantled in varying extent by aeolian (wind blown), tephra (air-fall cinder and ash), and colluvial (erosional) deposits.

LOCAL SURFACE CONDITIONS

SHSL & Quarries

On April 21, 2011, Mr. Sa Hong, P.E., a principal of HWA, conducted a reconnaissance of the existing SHSL and adjacent quarry sites. At that time, the following observations were noted:

- The bedrock units exposed at the landfill site consist of strong, bluish gray, columnar basalt that has a relatively thin upper flow top consisting of weak, reddish brown, basalt. The upper flow top is highly weathered and is capped by a thin organic rich soil layer (see Plate 1).

<u>Plate 1</u>. Photograph of an Exposure of Kau Basalt at the South Hilo Sanitary Landfill site.

- The near surface reddish brown flow top rock is light weight and possesses low strength relative to the more competent gray rock. It does not appear to be suitable for use as road base course. However, if crushed, the red top rock can be used as daily cover or common borrow when properly moisture conditioned and adequately compacted. The upper organic soils are unsuitable for use as fill except for use as topsoil within landscaping areas.

- The strong gray basalt is being actively quarried for daily cover used at the SHSL, and is also crushed to produce concrete and road base aggregates for County and commercial use. Review of several material testing summaries provided by JAS. W. Glover LTD indicates that crushed material made from the local bedrock has relatively good durability (L.A Abrasion values ranging from 18 to 26.6), moderate absorption (2.8% for fine aggregate), and high resistance to freeze and thaw or heating and cooling (i.e., magnesium sulfate soundness: 0.3 to 2.5%). Copies of the test results reviewed are duplicated in Appendix A.

- The existing perimeter walls at the quarry are nearly vertical and will likely need to be flattened to at least a 2H: 1V slope in order to construct a liner and sub-drainage system for a future landfill cell. The final interior slope angle may vary depending upon the liner system selected. The slopes can be laid back by blasting or alternatively the interior of the pit can be filled with crushed granular fill to attain the desired geometry.

Potential Borrow Soils

According to the *Geologic Map of the Island of Hawaii*, by Edward W. Wolfe and Jean Morris, 1996, a regional volcanic ash deposit, the Pahala Ash, has been mapped over a wide area north of the Wailuku River, northwest of Hilo, as shown on Figure 3. For the purposes of this assessment, the Pahala Ash was considered the most locally abundant borrow material that potentially could be used as a landfill liner component during construction, or for daily cover during operation, of a new landfill facility. The extent and thickness of this deposit is depicted on Figure 4 (from: Stearns et al, 1946). Indications are this material ranges from 2 to 5 meters thick in deposits mantling local basalt bedrock northwest of Hilo.

The Pahala Ash is a mixture of altered (palagonite) and unaltered volcanic glass, pumice, olivine, and plagioclase crystals (Hassan et al, 1975). It is derived from ash-fall deposits, weathered and reworked ash, and sediments. The ash is comprised predominately of silt and clay sized particles with some sand sizes. The appearance of the ash is greatly influenced by local climate (precipitation). In relatively dry areas, the ash appears granular, sandy, and dry. In high rainfall areas, such as those around Hilo, the ash appears to consist of a plastic, silty, clay-like material with some apparent cohesion. Locally, it appears that areas underlain by the Pahala Ash were predominately utilized in sugar cane production. Now, the area is sparsely occupied with rural dwellings.

Field Observations

On April 22, 2011, Mr. Hong conducted a reconnaissance northwest of the SHSL to observe local soil conditions exposed within road cuts and agricultural areas depicted on the local geologic map as being underlain by Pahala Ash. At that time, the following observations were noted:

- The soil can stand vertically if it is not disturbed, but it becomes unstable and can slide if it is disturbed (See Plate 2). Typically road cuts stand at steep angles when the soils have not been disturbed (See Plate 3).

Plate 2. Photograph of Minor Slide Developing in Pahala Ash.

Plate 3. Photograph of a Steep Road Cut Through Weathered Bedrock and Pahala Ash on Kaiwiki Road.

- The surface geomorphology of local ash covered terrain is shown in Plate 4. We expect that the undulating surface to some extent mimics the top of the underlying basalt bedrock, but may also express to a certain extent modification by wind and runoff in addition to shrink and swell caused by alternating periods of wet and dry weather. Local building practices in this area use piles extending to bedrock for heavy structures to avoid damage from shrinkage or lateral spreading induced settlement.

.

<u>Plate 4.</u> Photograph depicting local undulating topography underlain by Pahala Ash.

- North of the county cemetery (see Plate 5), farming activities appear to dry and granularize the near surface soils. This soil appears more clay-like where moist at depth.

<u>Plate 5.</u> Photograph of Pahala Ash Soil in Cultivation North of the County Cemetery.

FIELD SAMPLING

Sampling of potential liner soils was conducted during the reconnaissance on April 22, 2011. Locations were selected after reviewing the USGS geologic map for the island (Wolfe and Morris, 1996). Samples were obtained in the vicinity of the County Cemetery, and in the area between Amauulu Road and Alae Cemetery located west of Highway 19. Three disturbed soil samples were taken at the locations described below and sealed in plastic jars and shipped to our laboratory in Bothell, Washington. The approximate sample locations are noted on Figure 3.

<u>Sample S-1 (see Plate 6):</u>

Location: One mile west of Highway 19 on Amauulu Road.
Description: Reddish brown, medium stiff, clayey SILT, highly plastic.

<u>Plate 6</u>. Photograph of Soil exposure where Sample S-1 was obtained. The surface soil is somewhat desiccated and exhibited cohesion and moderate shear strength due to capillary tension between soil particles.

<u>Sample S-2</u>:

Location: Amauulu, near the end of the road, 1000 feet south of reservoir.
Description: Reddish, medium stiff, saturated clayey SILT, organic, light weight, plastic.

<u>Sample S-3</u> (see Plate 7):

Location: Kaiwiki, Creek crossing after (west of) tree tunnel, on right side of road bank, where fresh slough occurred.
Description: Brick red, soft to medium stiff, saturated, clayey SILT, sticky plastic, 5 feet high road cut, sample taken four feet from the top.

<u>Plate 7.</u> Sample S-3 was taken from this road cut. At the time when the soil was sampled the exposed slope was standing vertical.

LABORATORY TESTING

Representative soils samples taken from roadside cuts were returned to the HWA's laboratory for further examination and testing to characterize certain properties of the on-site soils. The laboratory testing program was performed in general accordance with appropriate ASTM Standards as outlined below.

PH TEST RESULTS: Testing was carried out on selected specimens using WSDOT Test Method No. 417. The measured pH of the soil sample is summarized in Table 1:

Table 1. Soil pH	
Sample	**pH**
S-2	5.8

MOISTURE CONTENT, ASH, AND ORGANIC MATTER: Selected specimens were tested in general accordance with method ASTM D 2974, using moisture content method 'A' (oven dried at 105^{0} C) and ash content method 'C' (burned at 440^{0} C). The test results are summarized in Table 2. The results are percent by weight of dry soil.

Table 2. Moisture Content, Ash, and Organic Matter			
Sample	Moisture Content (%)	Ash Content (%)	Organic Content (%)
S-3	360	78.1	21.9

LIQUID LIMIT, PLASTIC LIMIT, AND PLASTICITY INDEX OF SOILS (ATTERBERG LIMITS) AND SHRINKAGE LIMIT: Selected specimens were tested using method ASTM D 4318, multi-point method. The results are reported in Table 3 and on the attached Liquid Limit, Plastic Limit, and Plasticity Index report, Figure B-1 in Appendix B.

Table 3. Summary of Atterberg Limit Determinations				
Sample	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	Soil Classification
S-1	117	85	32	MH
S-2	353	115	238	MH
S-3	340	192	148	MH

DRYING SHRINKAGE: The amount of volume reduction (shrinkage) due to drying of a remolded soil specimen, confined within a metal ring, was determined by replacing the lost soil volume with wax after drying. Moist soil was remolded into a 1-inch high by 2.4-inch I.D. brass ring, weighed, then placed into an 110^{o} C oven and dried. After drying, wax with a known density was poured into the ring to replace the void created by shrinkage and soil volume loss was calculated. The results are summarized in Table 4.

Table 4. Dried shrinkage (%) measured, compared with initial wet volume			
Sample	Initial Soil Moisture Content (%)	Dried Volume (%)	Total Shrinkage (%)
S-3	345.4	11.7	88.3

SPECIFIC GRAVITY OF SOILS: The specific gravity of selected soil specimens were determined using method ASTM D 854. The test results are summarized in Table 5 below:

Table 5. Specific Gravity of Soil	
Sample	**Measured Specific Gravity**
S-2	2.775
S-3	2.704

SHEAR STRENGTH PARAMETERS OF SOIL: One point direct shear tests were conducted on specimens from sample S-2, in general accordance with ASTM D-3080. Specimens were prepared using virgin soil, soil with 30% by dry weight Portland Cement admixture (7.2% by wet weight), and soil with 20% by dry weight quick lime admixture (4.8% by wet weight). Soil specimens amended with Portland Cement (PC) or quick lime (QL) were mixed thoroughly, allowed to react for ½-hour prior to compaction, and then cured for an additional ½-hour prior to testing. Testing was constructed at a strain rate of 0.2% per minute. All three specimens were tested dry under a normal load of approximately 1 ksf. The test results are presented on the attached Direct Shear Strength of Soils report, Figure B-2 in Appendix B, and summarized in Table 6.

Table 6. Summary of Shear Strength Parameters of Soil*					
Sample	**MC (%)**	**WD (pcf)**	**DD (pcf)**	**Phi Peak**	**Phi Residual**
S-2	317.9	72.4	17.3	38.7	36.3
S-2 + 30% PC	317.9	77.1	18.5	40.8	39.0
S-2 + 20% QL	317.9	76.2	18.2	48.1	41.3

*Note: Indicated strength parameters assuming no cohesion.

SOIL ENGINEERING PROPERTIES

In summary, the soil materials represented by these samples exhibit the following engineering properties:

- High natural moisture contents; ranging from 111% to 360%.
- Low bulk density and dry unit weight; although soil particles have a specific gravity of about 2.7-2.8 indicating soil porosity of about 80%.
- Soil Classification of MH (elastic silt) based on Atterberg Limits

- Soils shrink continuously with drying; by as much as 80% by volume.
- Measured shear frictional angle is relatively high (38.7 degrees); assuming material has no cohesion.
- When amended with Portland cement or quick lime, the material exhibits an apparent increase in shear strength. Quick lime appears to be more effective than Portland cement in this application.

Review of literature describing similar soils on the Island of Hawaii indicates that soils derived from basaltic ash, formed in high rainfall areas like Hilo (see Figure 5), exhibit a relatively low pH, a high clay and iron oxide percentage, in addition to relatively high carbon and high water content at the permanent wilting point (Hassan, et al, 1975). Mineralogical analysis indicates these soils are composed of palagonite, clay, and iron oxides. Clay minerals in high rainfall areas such as Hilo are typically kaolinite or gibbsite, and the mineraloid allophane. This mineral assemblage is indicative of relatively good drainage and a high degree of leaching.

These soils have a high proportion of pore space and are considered thixotropic, behaving like a fluid when shaken. In addition, field reports indicate that when these soils are subject to long periods of drying after removal of vegetation, an irreversible aggregation process takes place at the surface producing coarse sand/gravel sized particles that are easily eroded by runoff (Aguliar et al., 1991).

DISCUSSION & CONCLUSIONS

Although this potential borrow material appears to be available in suitable quantities relatively close to the landfill site, the high moisture content, high porosity and thixotropic nature of the soil requires that it be amended to stabilize it for handling and use during construction. We do not recommend using this soil for daily cover without amending it with cement, lime or some other agents, because of its extremely high moisture and propensity to release water into the waste pile when it consolidates under self weight or surcharging effects.

While our initial evaluation is encouraging with regard to strength increase upon amendment, additional testing is required to evaluate whether this soil can be economically amended to produce a material that can be handled and placed efficiently with conventional construction equipment to produce a strong low permeability soil layer. Such a future geotechnical engineering and testing program may include but not be limited to;

- A mix design program utilizing Portland cement, quick lime, or other agents to produce a workable material that has properties similar to moderately plastic clay.
- Leachate compatibility tests with prepared material to observe any adverse reactions and/or changes in important engineering properties that may limit long term functionality.

- Field batch testing to assess potential material handling, mixing and constructability issues.

Alternatively, there may be other natural soil resources located on the dry (west) side of the island that occur in quantities sufficient for construction of a soil liner at the SHSL. However, the use of these materials may not be viable due to cost of permitting, excavation, hauling and potential mitigation of local environmental impacts. The additional costs involved with assessing a remote borrow source or soil amendment should be compared with conventional liner construction utilizing a composite HDPE membrane and GCL liner system.

CLOSURE

We have prepared this report for SAIC Energy, Environment and Infrastructure LLC., and the County of Hawai'i for use during the initial feasibility stage of this project. The conclusions and interpretations presented in this report should not be construed as our warranty of field performances.

Sufficient geotechnical monitoring, testing, and consultation should be provided during design and construction to confirm that the actual field conditions are consistent with those indicated by our preliminary feasibility studies.

The scope of our work did not include environmental assessments or evaluations regarding the presence or absence of wetlands or hazardous substances in the soil, surface water, or ground water at this site.

We appreciate this opportunity to be of service.

Sincerely,

HWA GeoSciences Inc.

SA H. HONG
REGISTERED PROFESSIONAL ENGINEER
No. 5545-C
HAWAII, U.S.A.

Steven E. Greene, L.G., L.E.G.
Senior Engineering Geologist/Vice-President

Sa H. Hong, P.E.
Principal Geotechnical Engineer

FIGURES

Figure 1 Project Site and Vicinity Map

Figure 2 Aerial Photograph of Project Site

Figure 3 Geologic and Sample Location Map

Figure 4 Distribution of Pahala Ash

Figure 5 Annual Precipitation Map of Hawaii

APPENDICES

Appendix A Quarry Materials Information

Materials reports provided by Jas. W. Glover, Ltd

Appendix B Laboratory Testing

Figure B-1 Soil Plasticity Chart

Figure B-2 Direct Shear Strength of Soils Report

REFERENCES

Aguilar, R., and M. Waite, 1991, *Soil Depth Characteristics and Erosion Estimates along the Hamakua Coast, Island of Hawaii*. J. Haw. Pac. Agrl., Vol. 3, p 39-51.

CH2M Hill Inc., 2008, *Waste Composition Study*, prepared for the County of Hawai'i.

Geometrician Associates Inc, 2008, *Considerations for Sitting a New Landfill in East Hawai'i*, for CH2M Hill Inc., and the County of Hawai'i DEM.

Hassan, T.S, H. Ikawa, and L.D. Swindale, 1975, *The Properties and Genesis of Four Soils Derived from Basaltic Ash, Mauna Loa, Hawaii*. Pacific Science, Vol. 29, No. 3, p.301-308.

Stearns, H.T., and G.A. McDonald, 1946, *Geology and Ground Water Resources of the Island of Hawaii*. Bulletin 9, Hawaii Division of Hydrology, Advertiser Publications Co. LtD, Honolulu, HI, 363 pp.

Williams, H, and A.R. McBirney, 1979, *Volcanology*, Freeman, Cooper & Co., San Francisco, 397 pp.

NORTH

Map taken from: Geometrician Associates, 2008.

HWA GEOSCIENCES INC.

PROJECT SITE AND VICINTY MAP

GEOTECHNICAL ASSESSMENT OF BORROW SOIL
SOUTH HILO SANITARY LANDFILL
HILO, HAWAII

FIGURE NO.
1

PROJECT NO.
2011-047

NORTH

Photo taken from: CH2M Hill, 2009.

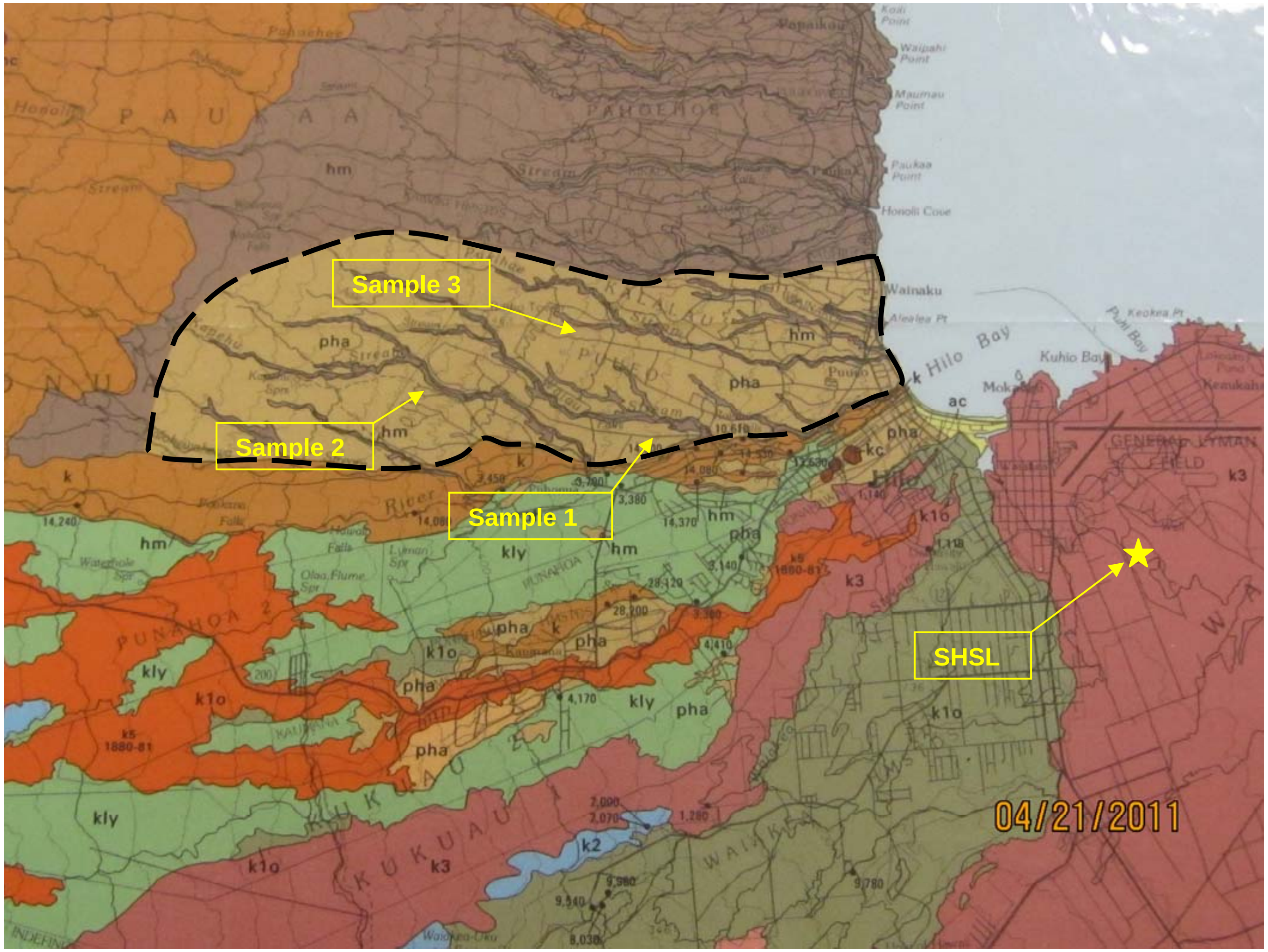
HWA GEOSCIENCES INC.

AERIAL PHOTOGRAPH OF PROJECT SITE

GEOTECHNICAL ASSESSMENT OF BORROW SOIL
SOUTH HILO SANITARY LANDFILL
HILO, HAWAII

FIGURE NO.
2

PROJECT NO.
2011-047



NORTH

Map Symbol Legend	Unit Description
pha	Pahala Ash – Deeply Weathered Ash Deposits
k2, k3, kly, k1o	Kau Basalt – Predominately Tholeiitic Basalt
hm	Lava Flows of the Hamakua Volcanics – Predominately Alkalic and Transitional Basalt
— — —	Mapped Limits of Pahala Ash - Northwest of Hilo.

Map Taken From: Geologic Map of the Island of Hawaii, by Edward W. Wolfe and Jean Morris, 1996, USGS.

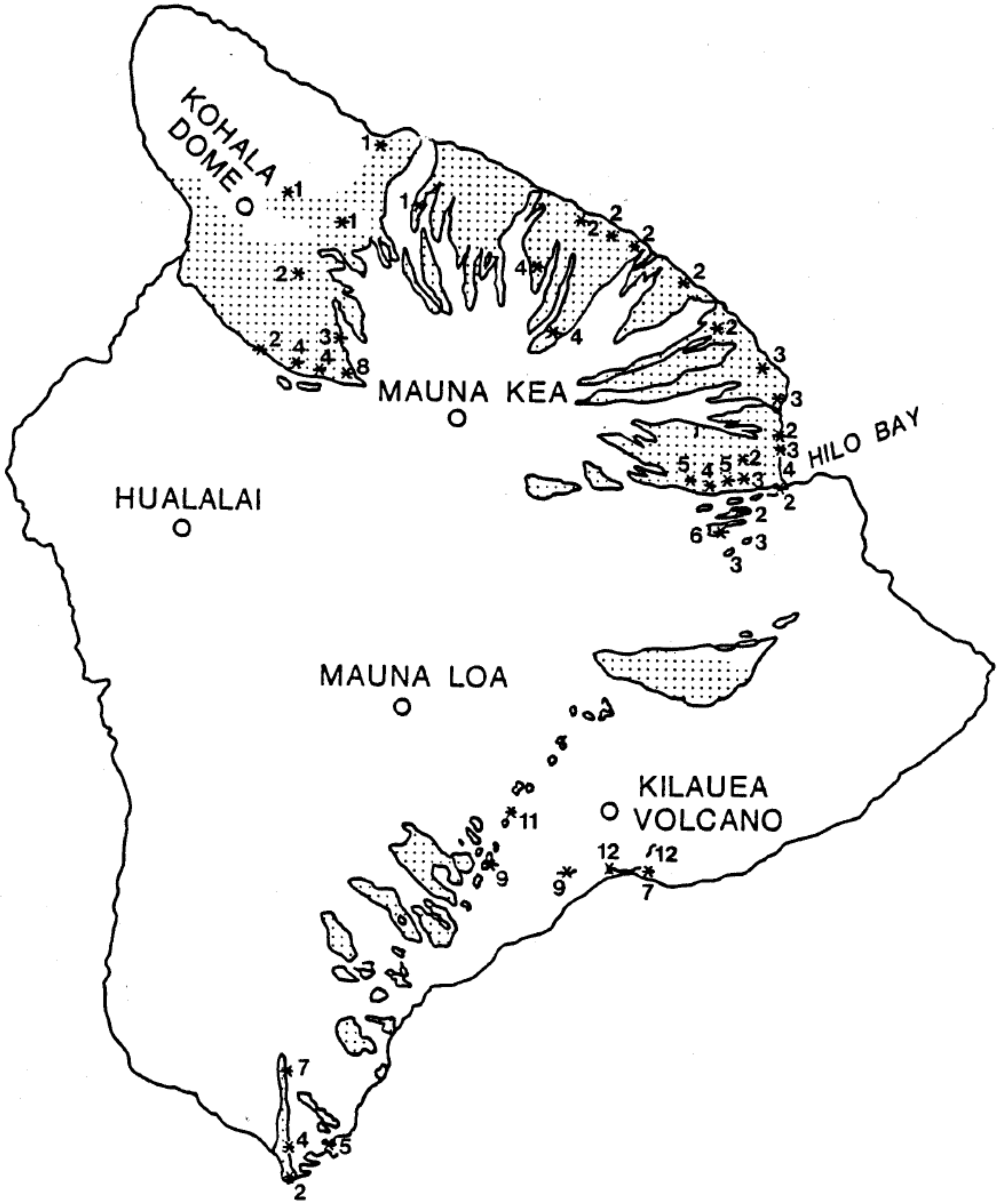
HWA GEOSCIENCES INC.

GEOLOGIC AND SAMPLE LOCATION MAP

GEOTECHNICAL ASSESSMENT OF BORROW SOIL
SOUTH HILO SANITARY LANDFILL
HILO, HAWAII

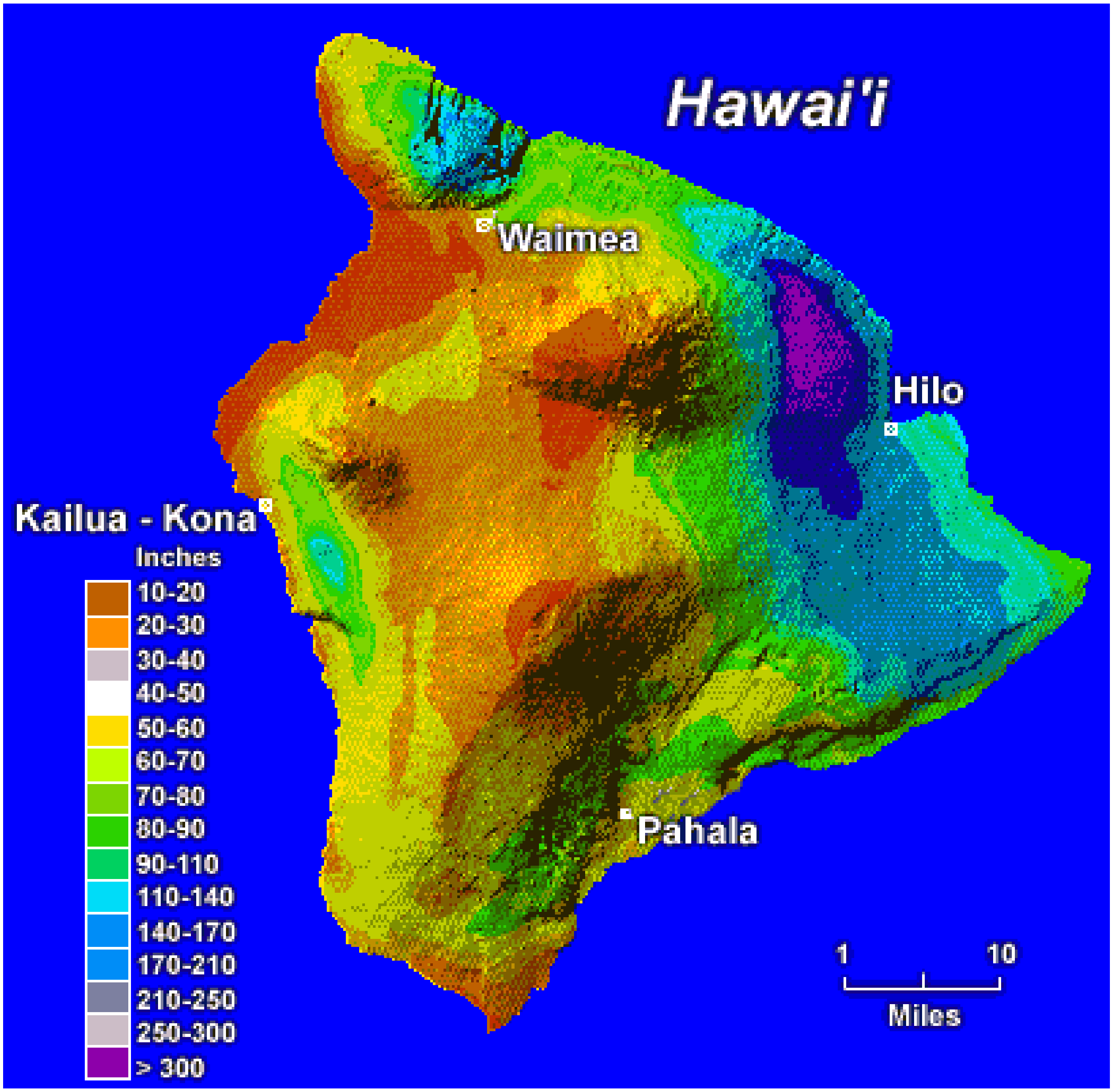
FIGURE NO. **3**

PROJECT NO. 2011-047



Map depicts distribution of Pahala Ash over the Island of Hawaii. Note numbers indicate measured thickness of ash layer in meters at locations shown. Map taken from: Stearns and McDonald, 1946, Geology and Ground Water Resources of the Island of Hawaii.

HWA GEOSCIENCES INC.	**DISTRIBUTION OF PAHALA ASH**	FIGURE NO. **4**
	GEOTECHNICAL ASSESMENT OF BORROW SOIL **SOUTH HILO SANITARY LANDFILL** **HILO, HAWAII**	PROJECT NO. 2011-047



HWA GEOSCIENCES INC.

ANNUAL PRECIPITATION MAP OF HAWAII	FIGURE NO. 5
GEOTECHNICAL ASSESMENT OF BORROW SOIL SOUTH HILO SANITARY LANDFILL HILO, HAWAII	PROJECT NO. 2011-047

APPENDIX A

Materials Test Reports

Jas. W. Glover, Ltd

JAS. W. GLOVER, LTD.

GENERAL CONTRACTORS
License No. ABC-3

PHONE NO. (808) 935-0871 FAX NO. (808) 961-9237
HILO OPERATIONS
890 LEILANI STREET - HILO, HAWAII 96720

JUNE 7, 2010

SIEVE ANALYSIS OF FINE AGGREGATE.
(A.S.T.M. C-136)

SIEVE SIZE	PERCENT PASSING SPECS ASTM(C33)	CONCRETE FINE #4 SAND BASALT HILO QUARRY
3/8"	100	100
#4	95-100	100
#8	80-100	95
#16	50-85	65
#30	25-50	44
#50	15-30	30
#100	2-10	21
#200	0-5	14

Dry Rodded Weight:	163.5 lbs/ft3
Fineness Modulus:	2.45 +/-.1
Sand Equivalent:	75
Absorption:	2.8%
Specific Gravity:	2.84

PROJECT: PAHOA RECYCLING AND TRANSFER STATION

PROJECT #:

CONTRACTOR: REEF DEVELOPMENT OF HAWAII, INC.

Remarks: Non-compliance with ASTM (C33) paragraph 5.1; Note Section 5, paragraph 6.4 in lieu of gradation requirements, based on acceptable performance of aggregate in State of Hawaii (DOT) Concrete Mixes.

BOBBY-VAN KRUEGER
QUALITY CONTROL TECHNICIAN

JAS. W. GLOVER, LTD.

GENERAL CONTRACTORS

License No. ABC-3

PHONE NO. (808) 935-0871 FAX NO. (808) 961-9237

HILO OPERATIONS

890 LEILANI STREET - HILO, HAWAII 96720

JUNE 7, 2010

SIEVE ANALYSIS OF COARSE AGGREGATE.
(A.S.T.M. C-136)

SIEVE SIZE	PERCENT PASSING SPECS ASTM C33	#67 BASALT HILO QUARRY
1"	100	100
3/4"	90-100	100
1/2"	NONE	73
3/8"	20-55	33
#4	0-10	1
#8	0-5	0.4
#200	1.5	0.1

Dry Rodded Weight (AASHTO T-19):	103.25 lbs/ ft3
Fineness Modulus (AASHTO T-27):	6.65
Absorption (AASHTO T-85):	2.1%
Specific Gravity (SSD) (AASHTO T-85):	2.83
L.A. Abrasion:	22.5
Fractured Faces:	100
Flat and Elongated:	0.2
Soundness:	0.7

PROJECT: PAHOA RECYCLING AND TRANSFER STATION

PROJECT #:

CONTRACTOR: REEF DEVELOPMENT OF HAWAII, INC.

Remarks: Compliance with ASTM C33 #67.

BOBBY-VAN KRUEGER
QUALITY CONTROL TECHNICIAN

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P.O. Box 579 • Honolulu, HI 96809
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P.O. Box 1929 • Lihue, HI 96766
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JAS. W. GLOVER, LTD.

GENERAL CONTRACTORS
License No. ABC-3

Date: FEBRUARY 10, 2010

To: **WILLIAM C. LOEFFLER CONSTRUCTION, INC**
Attention: TINA MARIE
Facsimile: (808) 935-4422
Telephone: (808) 961-5588

Project: **UNIVERSITY of HAWAI'I – HILO OFFSITE IMPROVEMENTS; LANIKAULA STREET; HILO UHH JOB # 2003-609C ISLAND of HAWAI'I**

Material: **1½"; ¾"; Basalt Base Course Materials**

At your request; we are pleased to submit the following sieve analysis; based on our current and historical data for our Base Course sized basaltic materials listed for your consideration. The basalt products are produced at our Island of Hawai'i operations @ Hilo Quarry.:

Sieve size □ opening;	1½" BC:	¾" BC:
3"		
2½"		
2"	100%	
1½"	97%	
1"	n/a	100%
¾"	87%	94%
½"	n/a	n/a
⅜"	n/a	n/a
#4	45%	47%
#8	n/a	n/a
#16	n/a	n/a
#100	n/a	n/a
#200	8%	8%

Density: 139.5 lbs/ft^3
Optimum Moisture: 6.6%
LA Abrasion: 26.6%
Sand Equivalent 78
Liquid Limit CNBD
Plasticity Index Non-Plastic
Soundness Loss: 0.4

Respectfully submitted by:

Richard S. Gribbin
Statewide Manager – QC/QA
Jas. W. Glover, Ltd.

YAMADA AND SONS, INC.
P.O BOX 4699 • 733 KANOELEHUA AVENUE • HILO, HAWAII 96720-4700
TELEPHONE: (808) 933-8434 • FAX NO. (808) 933-8415

AGGREGATE QUALIFICATION SUMMARY

			Aggregate Type				
			AC #4 Sand	#8 - Chip Stone	#67	#56	Combined
			55%	45%	0%	0%	100%
Test	Method	Requirement					
Sand Equivalent	AASHTO T-176	45% Min.	71	N/A	N/A	N/A	N/A
Los Angeles Abrasion	AASHTO T-96	30% Max.	N/A	18.2%	25.8%	25.4%	8.2%
Stripping	AASHTO T-182	Above 95%	N/A	>95%	>95%	>95%	>95%
Flat and Elongated Pieces	ASTM D 4791 (By Weight)	25% Max.	N/A	14.0%	8%	15%	6.3%
Soundness	AASHTO T-104	9% Max.	0.3%	0.3%	0.3%	2.5%	0.3%
Absorption	AASHTO T 84 AASHTO T 85	5% Max.					#VALUE!
Fractured Faces	90% Min		100%	100%	100%	100%	100%

Note: All of the above aggregate is 100% crushed material.

Date 12/28/2010

Leslie C Pedersen

Leslie C. Pedersen
Yamada and Sons, Inc.

Construction Engineering Labs

Yamada & Sons, Inc.
733 Kanoelehua Ave.
Hilo, Hawaii 96720

Date: 8/25/10
Report #: 19807

TEST REPORT

Project: Aggregate Qualifications	W.O. No. 19807
Client: Yamada & Sons	Received: 7/12/10
Description of material: #56, #57, #8, AC Sand Basalt Aggregate	Tech: WP
Source: Hilo Quarry	Sample #: 19807

ASTM C 88-Soundness of Aggregate by Use of Magnesium Sulfate

Sieve Size	#56	#57	#8	AC Sand
1 ½" to ¾" Loss	0.3	NA	NA	NA
3/4" to 3/8" Loss	0.2	0.3	0.4	NA
3/8" to #4 Loss	NA	0.3	0.2	NA
#4 to #8 Loss	NA	NA	NA	0.2
#8 to #16 Loss	NA	NA	NA	0.3
#16 to #30 Loss	NA	NA	NA	0.3
#30 to #50 Loss	NA	NA	NA	0.2

ASTM C131- LA Abrasion

TEST	#56	#57	#8	AC Sand
L A Abrasion	25.4%	25.8	18.2	NA

Please contact our office if you have any questions or need additional information.

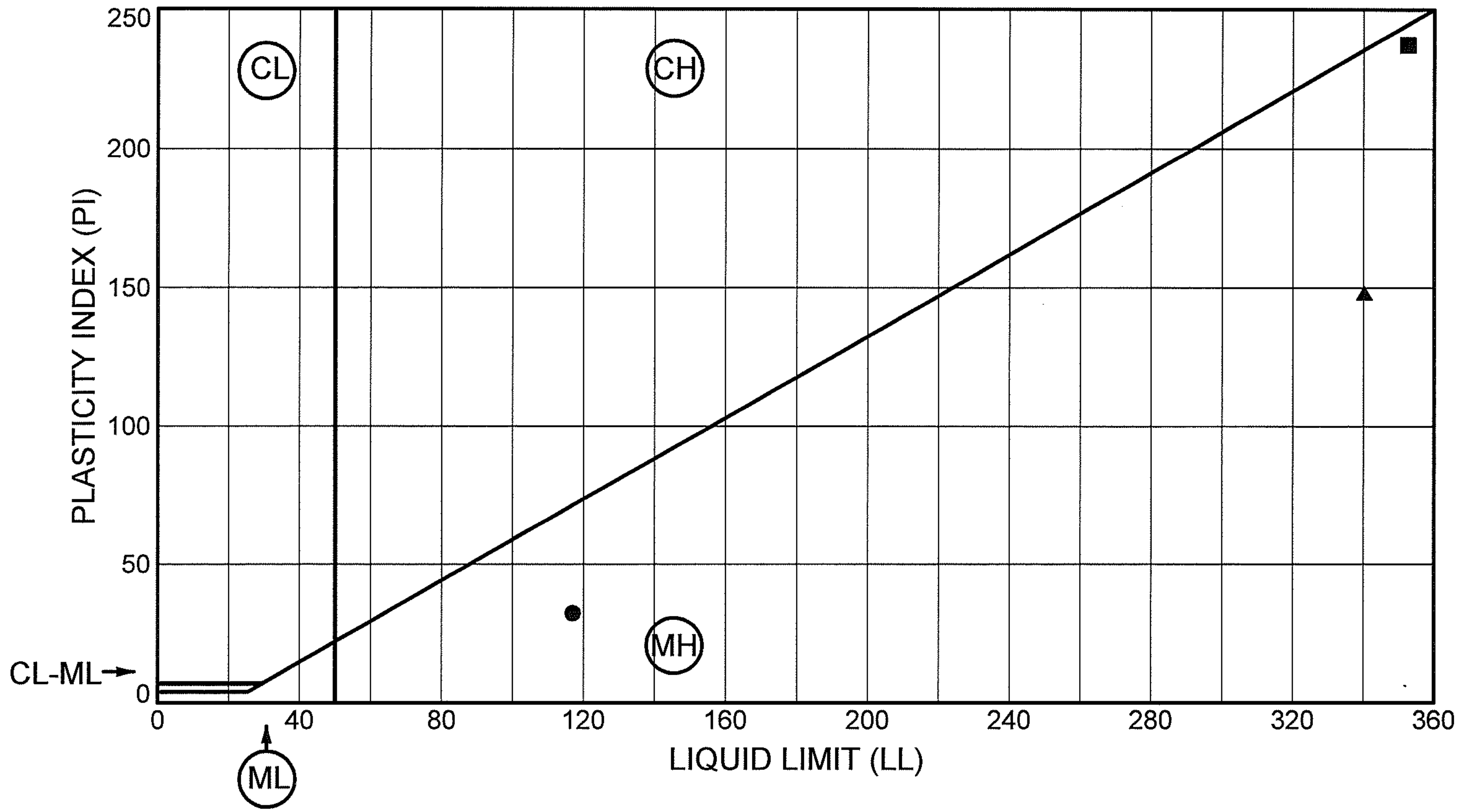
Respectfully,
CONSTRUCTION ENGINEERING LABS, INC.

By: Ronald A. Pickering II
Its: President

96-1173 Waihona St., Unit B-7, Pearl City, Hawaii 96782
Phone: 808-455-1522, Fax: 808-455-1384, Email cel@hawaii.rr.com

APPENDIX B

HWA Laboratory Testing Results



SYMBOL	SAMPLE		DEPTH (ft)	CLASSIFICATION	% MC	LL	PL	PI	% Fines
●	S-1			(MH) Yellowish red, elastic SILT	111	117	85	32	
■	S-2			(MH) Yellowish red, elastic SILT	359	353	115	238	
▲	S-3			(MH) Yellowish red, elastic SILT (organic content 21.9%)	360	340	192	148	

HWA GEOSCIENCES INC.

Geotechnical Assessment of Borrow Soil
South Hilo Sanitary Landfill
Hilo, County of Hawaii, HI

LIQUID LIMIT, PLASTIC LIMIT AND PLASTICITY INDEX OF SOILS METHOD ASTM D4318

PROJECT NO.: 2011-047-21 FIGURE: B-1

HWA GeoSciences Inc. Materials Testing Laboratory

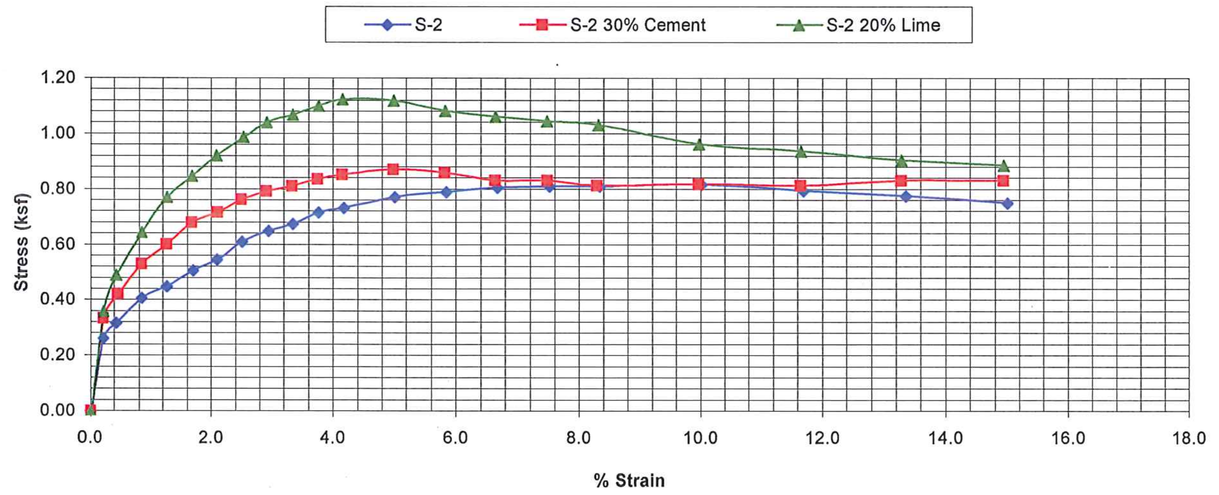
Direct Shear Test of Soils Under Consolidated Drained Conditions (ASTM D 3080)

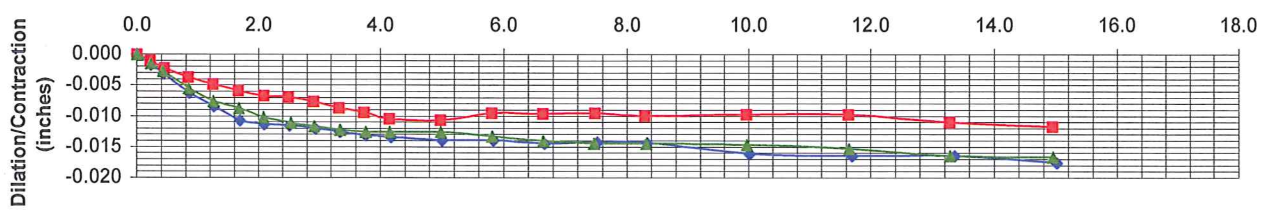
Project Name:	South Hilo Sanitary Landfill				**Project Number:**	2011-047-21		
Sample Point:	N/A	**Sample No.:** S-2			**Sample Depth:**	0		
Soil Description:	elastic SILT (MH)							
Soil Color:	Reddish Brown				Strain rate:	0.2	% per min.	
Soil Group Symbol:	MH	Virgin	30% Cement	20% Lime	Soil Specific Gravity:	3.00	(assumed)	

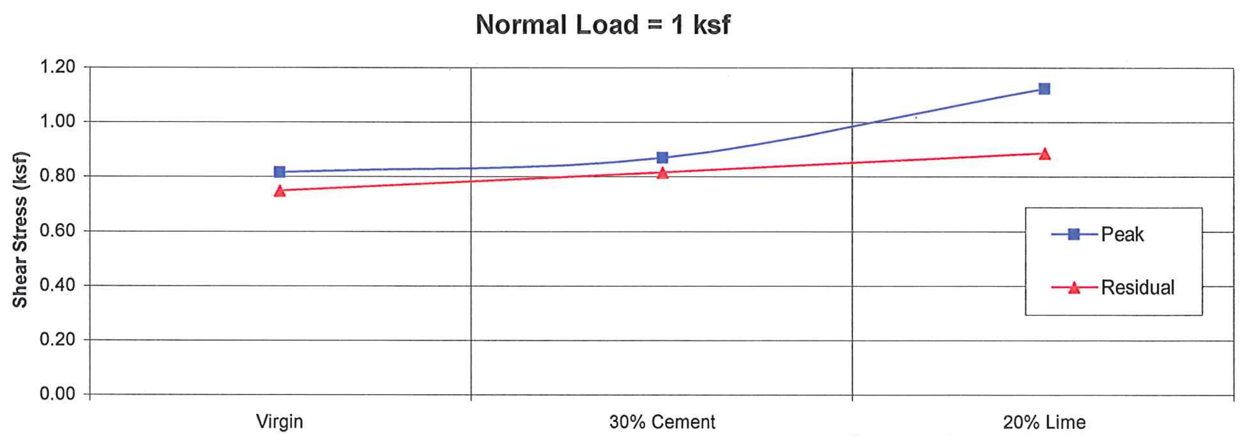
	Virgin	30% Cement	20% Lime	Average
Normal Stress (ksf)	1.02	1.01	1.01	
Peak Stress (ksf)	0.817	0.872	1.124	
Residual Stress (ksf)	0.750	0.818	0.887	
Initial Moisture Content (%):	317.9	317.9	317.9	317.9
Wet Unit Weight (pcf):	72.4	77.1	76.2	75.2
Dry Unit Weight (pcf):	17.3	18.5	18.2	18.0
Calculated Void Ratio	9.801	9.145	9.271	9.406
Calculated Porosity	0.907	0.901	0.903	0.904
Calculated Saturation (%)	97.3	104.3	102.9	101.5

Indicated Strength Parameters Assuming no Cohesion

phi Angle	Peak (degrees)	Residual (degrees)
Virgin	38.7	36.3
30% Cement	40.8	39.0
20% Lime	48.1	41.3







Checked By: [signature]

Figure B-2

Appendix D
HYDROLOGIC EVALUATION LANDFILL PERFORMANCE (HELP) MODEL

Introduction

The purpose of this memorandum is to document modeling results for the South Hilo Sanitary Landfill located in Hilo, Hawaii. The analysis was performed to support the leachate management system design and size the proposed leachate wetlands.

Overview

The primary analysis method used for the landfill water balance analysis is the Hydrologic Evaluation Landfill Performance (HELP) Model, Version 3.07. The model was developed by the U.S. Army Engineer Waterways Experiment Station for the U.S. Environmental Protection Agency (EPA) and has been in use since 1984.

The HELP Model is a quasi-two-dimensional hydrogeologic water balance model developed specifically to perform hazardous and municipal waste landfill evaluations. The model requires weather, soil, and design data that are representative of the landfill location and design. It utilizes solution techniques that account for the effects of surface storage, snow melt, runoff, infiltration, evapotranspiration, vegetative growth, soil moisture storage, lateral subsurface drainage, leachate recirculation, unsaturated vertical drainage, and leakage through soil, geomembrane or composite liners.

The model can be used to evaluate various combinations of vegetation, cover soils, waste cells, lateral drainage layers, low permeability barrier soils, and synthetic geomembrane liners. The model provides estimates of the amount of runoff, evapotranspiration, drainage, leachate collection, and barrier soil and liner leakage that may be expected based on the design model case. The primary purpose of the model is for comparison of landfill design alternatives based on their performance.

It is also important to note that the results from this HELP analysis will be used to estimate leachate generation as part of the feasibility study. A revised model will need to be completed as part of the permitting process.

Design Criteria

The HELP Model requires climatological, vegetative, soil, and design data specific to the landfill site and design. The required climatological data for the HELP Model includes monthly precipitation averages, mean monthly temperatures, evapotranspiration and solar radiation representative of the landfill location.

For the purposes of this analysis, default mean monthly temperatures, evapotranspiration and solar radiation data available within the model for were utilized for Honolulu, Hawaii. Default monthly precipitation values were not available for Honolulu, Hawaii and were instead manually generated using historical monthly precipitation records for the Hilo International Airport.

The following Table 1.1 describes the basis for data selection, including assumptions, and the layer profiles used in the HELP Model analysis.

Table 1.1
HELP Modeling Methodology/Assumptions

Scenario	1 Open Condition (Daily Cover)	2 Intermediate Cover Condition	3 Final Cover Condition	4 Open Condition (Daily Cover)
Subgrade	6-Inch Compacted Native Soil[1]	6-Inch Compacted Native Soil[1]	6-Inch Compacted Native Soil[1]	None
Low Perm Layer	GCL	GCL	GCL	24-Inch Low Perm Soil
Base Geomembrane	60-mil HDPE	60-mil HDPE	60-mil HDPE	60-mil HDPE
Drainage Layer	Drainage Net (0.6 cm)	Drainage Net (0.6 cm)	Drainage Net (0.6 cm)	Drainage Net (0.6 cm)
Buffer Layer	12-Inches Gravel	12-Inches Gravel	12-Inches Gravel	12-Inches Gravel
Waste Depth	10-Feet	140-Feet	140-Feet	10-Feet
Cover Soil	6-Inches Daily[1]	12-Inches Intermediate[1]	12-Inches Intermediate[1]	6-Inches Daily[1]
Buffer Layer	None	None	6-Inch Low Perm Layer	None
Final Cover Geomembrane	None	None	40-mil LLDPE	None
Drainage Layer	None	None	18-Inches Drainage Sand	None
Topsoil	None	None	6-Inches	None
Ground Cover	Bare	Bare	Good Grass	Bare
Recirculation	No – 0%	No – 0%	No – 0%	No – 0%
Liner Defects	1.0 Pinhole/ Acre, 4 Defects/Acre, “Good” Placement	1.0 Pinhole/ Acre, 4 Defects/Acre, “Good” Placement	1.0 Pinhole/ Acre, 4 Defects/Acre, “Good” Placement	1.0 Pinhole/ Acre, 4 Defects/Acre, “Good” Placement
SCS Runoff Curve Number	94.6	94.6	81.3	94.6
Evaporative Zone Depth	24-Inches	24-Inches	24-Inches	24-Inches
Allowable Runoff	0%	25%	100%	0%
Simulation Period	4-Years	20-Years	30-Years	4-Years
Simulation Area	1 Acre	1 Acre	1 Acre	1 Acre

1. Native soil utilized for daily and intermediate cover is assumed to be silty clay loam as described in the United States Department of Agriculture Natural Resources Conservation Service listing for the Hilo series.

As shown in Table 1.1, Scenarios 1 through 3 have been designed to utilize a geosynthetic clay liner (GCL). Scenario 4 has been included to show the comparison between this design and a traditional design using low-permeability soils. The cover use of soil was used to demonstrate typical cover material. Use of a raincoat will further minimize precipitation infiltration over the portion it is used.

Results

Detailed HELP model outputs for each scenario are included as attachments to this submittal. Peak Daily Hydraulic Head Over Liner values are listed in Table 1.2.

Table 1.2
Peak Daily Hydraulic Head Over Liner

Scenario		Average Head (in/acre)	Maximum Head (in/acre)
1	Open Condition	0.14	0.27
2	Intermediate Cover Condition	0.04	0.08
3	Final Cover Condition	0.00	0.00
4	Open Condition (Low Perm Soils)	0.14	0.27

The average and maximum hydraulic head for each scenario described in Table 1.2 are below the maximum allowable 12 inches of head.

To determine the size of the leachate treatment system, the volumes of leachate collected from the drainage layer must be analyzed. Table 1.3 summarizes the peak daily and average annual drainage collected from the drainage layer in each scenario.

Table 1.3
Drainage Collected from Drainage Layer

Scenario		Peak Daily		Average Annual	
		in/acre	ft^3	in/acre	ft^3
1	Open Condition	1.78	6,450.97	70.35	255,367.56
2	Intermediate Cover Condition	0.50	1,826.59	54.35	197,304.95
3	Final Cover Condition	0.00	0.03	0.002	6.25
4	Open Condition (Low Perm Soils)	1.78	6,450.97	70.35	255,367.56

As shown in Table 1.3, the peak drainage collected from the drainage layer is 1.78 inches per acre, or 6,450.97 ft^3. Because the HELP modeling was completed over a 1-acre simulation area, this value must be multiplied by the total area of the landfill to estimate the leachate generation over the entire facility. This peak daily value and the average annual value should be used as the design criteria when appropriately sizing the leachate treatment facilities for the landfill. For design of the wetlands, the average annual value is used, and is approximately 5,230 gallons per acre per day.

Table 2

Surface Water Runoff Estimates for South Hilo Sanitary Landfill for 25-year, 1-hour duration storm event.

Year	Cell size	Cover (%)	open area (acre)	Leachate/acre (gpd) HELP[a]	Leachate/cell (gpd) HELP	Leachate/cell (gpm) HELP	precip intensity[b] (in/hr) 25-yr, 1-hr	Potential runoff[c] (gpm/acre)	Potential runoff (gpm)
1	7	25%	5.25	5,300	27,825	19	3.0	1,279	2,238
2	7	25%	5.25	5,300	27,825	19	3.0	1,279	2,238
3	7	25%	5.25	5,300	27,825	19	3.0	1,279	2,238
4	7	25%	5.25	5,300	27,825	19	3.0	1,279	2,238
5	7	25%	5.25	5,300	27,825	19	3.0	1,279	2,238
6	7	25%	5.25	5,300	27,825	19	3.0	1,279	2,238
7	11.7	25%	8.78	5,300	46,508	32	3.0	1,279	3,741
8	11.7	25%	8.78	5,300	46,508	32	3.0	1,279	3,741
9	11.7	25%	8.78	5,300	46,508	32	3.0	1,279	3,741
10	11.7	25%	8.78	5,300	46,508	32	3.0	1,279	3,741
11	16.4	25%	12.30	5,300	65,190	45	3.0	1,279	5,244
12	16.4	25%	12.30	5,300	65,190	45	3.0	1,279	5,244
13	16.4	25%	12.30	5,300	65,190	45	3.0	1,279	5,244
14	16.4	25%	12.30	5,300	65,190	45	3.0	1,279	5,244
15	16.4	25%	12.30	5,300	65,190	45	3.0	1,279	5,244
16	21.1	25%	15.83	5,300	83,873	58	3.0	1,279	6,747
17	21.1	25%	15.83	5,300	83,873	58	3.0	1,279	6,747
18	21.1	25%	15.83	5,300	83,873	58	3.0	1,279	6,747
19	21.1	25%	15.83	5,300	83,873	58	3.0	1,279	6,747
20	25.8	25%	19.35	5,300	102,555	71	3.0	1,279	8,250
21	25.8	25%	19.35	5,300	102,555	71	3.0	1,279	8,250
22	25.8	25%	19.35	5,300	102,555	71	3.0	1,279	8,250
23	25.8	25%	19.35	5,300	102,555	71	3.0	1,279	8,250
24	30.5	25%	22.88	5,300	121,238	84	3.0	1,279	9,753
25	30.5	25%	22.88	5,300	121,238	84	3.0	1,279	9,753
26	30.6	25%	22.95	5,300	121,635	84	3.0	1,279	9,785
27	30.6	25%	22.95	5,300	121,635	84	3.0	1,279	9,785
28	35.3	25%	26.48	5,300	140,318	97	3.0	1,279	11,288
29	35.3	25%	26.48	5,300	140,318	97	3.0	1,279	11,288
30	40	25%	30.00	5,300	159,000	110	3.0	1,279	12,791
31	40	25%	30.00	5,300	159,000	110	3.0	1,279	12,791

[a] Estimates provided by the HELP Model assume a continuous generation of leachate from the landfill. The estimate provided is for a waste depth of 10 feet, and this value will vary as the depth of waste changes.

[b] Intensity values provided by the National Oceanic and Atmospheric Administration, Atlas 14, Volume 4, Version 2 Hawaiʻi

[c] Peak runoff estimate from storm event. The value is not a constant flow, but a peak value. Diversion systems and pumping equipment would be designed to handle this type of scenario or event.

Appendix E
WETLAND TREATMENT OF LEACHATE EVALUATION

To: Randall Sippel, SAIC

From: Mark Liner, NWC

Subject: South Hilo Sanitary Landfill Leachate Treatment – Design Brief

Date: August 23, 2011

CC: Scott Wallace, NWC

BACKGROUND

Hawaii County is in the process of evaluating the expansion of the existing South Hilo Landfill to adjacent rock quarries. If expanded, the new landfill must be lined so that local groundwater is protected from the landfill's leachate. The purpose of this Design Brief is to present a conceptual design for an wetland-based treatment system that will achieve applicable discharge standards for the quality of leachate predicted for the lined landfill.

Over the past decade, improvements in the engineering of wetlands have increased their performance and reliability. Engineers have borrowed the biology, chemistry, and hydraulics of the wastewater industry and are employing it successfully to create treatment systems that perform like conventional mechanical plants. Inclusion of aeration in subsurface wetlands has greatly advanced the ability of these systems to aerobically degrade hydrocarbons and ammonia and for this reason "engineered" wetlands are currently being widely used for treatment of contaminated groundwater, airport deicing fluid, and landfill leachate. Wetland-based systems are currently in operation at Buffalo Niagara International Airport (see photo below) and at the BP remediation site at Wellsville, NY(see http://www.arwellsville.com/site/ and attached presentation).

Figure 1: Aerial Photo of Buffalo Airport Treatment System (located in foreground)

Treatment wetlands have been used across the world for the treatment of landfill leachate. A comprehensive summary of experience is compiled in Constructed Wetlands for the Treatment of Landfill Leachates (1999) by George Mulamoottil et al. In recent years, the use of aerated wetlands has proven particularly effective in providing reliable treatment for ammonia, which is often the controlling design parameter for the treatment of landfill leachate. Nivala et al. (2007) document case studies in which the extent of ammonia removal is quantified. Based on these and similar studies, a conceptual design for a wetland-based treatment system can be developed with a high degree of confidence with respect to predicted performance.

BASIS OF DESIGN

EXISTING FACILITIES

An aerial view of the existing facility is provided below. Land for a leachate treatment system is available to the east (topside of the image) of the existing landfill and quarry.

Figure 2: Aerial Photo of South Hilo Landfill

DESIGN FLOW

Initial estimates of leachate volumes were developed by CH_2M Hill and are presented in a Technical Memorandum as an appendix to SWT's 2008 Proposed Expansion Feasibility and Capital Cost Estimate Report (SWT Report). A worst case annual average design flow of 8.9 gallons per minute (12,816 gallons per day) is listed as the design flow for development of the treatment system.

SAIC conducted a separate analysis using USEPA's HELP (Hydrologic Evaluation of Landfill Performance) model. Results indicate that on average 39,193 gallons per day of leachate will be generated for the 7.5 acre expansion or roughly 5,300 gallons/acre/day. Uncovered, the same landfill area could generate up to 361,928 gallons per day of leachate (as a maximum day peak). Noting that the model provides a more conservative estimate of leachate production, a treatment system flow rate of 40,000 gpd will be assumed for the purpose of preliminary sizing.

INFLUENT CHARACTERIZATION

To develop the size of a treatment system, an assumption of the influent quality is required. For typical domestic wastewater treatment systems, the quality of influent does not change substantially over the life of the system. However, this is not the case with landfill leachate; the concentrations of various parameters change as the landfill ages. It is commonly understood that the concentrations will decrease over time (see Mulamoottil, et al, 1999) - new landfills will have high concentrations and older, closed landfills will have much lower concentrations. Since the proposed landfill expansion will involve the lining and leachate collection of new cells, it is prudent to use higher concentrations that reflect the quality of leachate for a young landfill.

Four characterizations were reviewed to develop a "basis of design" that would be representative of a leachate from new cells at the South Hilo Landfill (SHL). A summary of each characterization is provided below:

1. Leachate monitoring results from samples taken at the existing South Hilo Landfill are presented in the SWT report. The results provide an indication of the leachate quality for the existing *unlined* landfill but are likely not representative of leachate that would be collected in a *lined* landfill.
2. CH2MHill utilized the SHL data along with results from a landfill in Unalaksa, Alaska to develop a predicted leachate concentration for the lined expansion. The Biochemical Oxygen Demand (BOD) value was increased from 72 to 700 mg/L. A value of 500 mg/L was included for Total Suspended Solids.
3. In 1999, the United States EPA proposed Effluent Guidelines for landfills that are designed to provide permit writers with a technical basis for establishing treatment standards. A comprehensive sampling effort was undertaken to support the effort and a nationwide leachate characterization was developed for lined, non-hazardous, municipal landfills.
4. Available leachate data from a sampling of lined landfills in Oregon was compiled by SAIC.

A summary table of the characterizations is provided below:

Table 1: Summary of Leachate Characterizations

	SHL	CH2MHill	EPA	Oregon Landfills
Parameter	(mg/L)	(mg/L)	(mg/L)	(mg/L)
Ammonia	80.4	85	81.7	78
Alkalinity	704	704	N/A	1102
BOD_5	72	700	240	49
TSS	N/A	500	137	53
Arsenic	0.0282	0.0282	N/A	0.013
Chromium	0.025	0.025	0.028	<0.03

Copper	0.0852	0.0852	N/A	0.00063
Lead	0.1437	0.1437	N/A	<0.1
Nickel	0.0212	0.0212	N/A	<0.04
Zinc	0.306	0.306	0.1	0.036

Based on a review of the various characterizations, it appears that the characterization utilized by CH_2MHill in the November 25, 2008 Technical Memorandum is an appropriate estimate of leachate quality for conceptual design of the treatment system. It will be used as the "basis of design" characterization for treatment system sizing.

EFFLUENT REQUIREMENTS

SAIC consulted with local regulatory authorities in Hawaii County in regard to applicable treatment standards for leachate disposal. Hawaii Department of Health will lead the review for any treatment system that will discharge via an infiltration gallery. The default standards for discharge under this scenario are 30 mg/L BOD_5 and 30 mg/L TSS.

The EPA's effluent guidelines for RCRA Subtitle D Non-Hazardous Waste Landfills are provided in the table below. The values provided for each "regulated parameter" are ceiling values; the DOH can set the discharge limits lower but not higher. With exception of the DOH established value of 30 mg/L for BOD, it is reasonable to assume that EPA's effluent limitations will dictate the quality of effluent required from the treatment system.

Table 2: EPA Effluent Guidelines for Lined Municipal Landfills

Effluent Limitations

Regulated parameter	Maximum daily \1\	Maximum monthly avg. \1\
BOD....................................	140	37
TSS....................................	88	27
Ammonia (as N).........................	10	4.9
[alpha]-Terpineol......................	0.033	0.016
Benzoic acid...........................	0.12	0.071
p-Cresol...............................	0.025	0.014
Phenol.................................	0.026	0.015
Zinc...................................	0.20	0.11
pH.....................................	(\2\)	(\2\)

\1\ Milligrams per liter (mg/L, ppm)
\2\ Within the range 6 to 9.

[65 FR 3048, Jan. 19, 2000; 65 FR 14344, Mar. 16, 2000]

PROCESS SELECTION AND SIZING

PROCESS SELECTION

Leachate typically has high concentrations of iron that will readily oxidize when exposed to air. For this reason, a passive cascade aerator with sedimentation pond/tank is suggested to provide a means to aerate, precipitate, and settle out soluble iron. Although primarily designed for iron removal, the cascade aerator and pond will promote the removal of other compounds that are removed by similar mechanisms.

An aerated wetland will follow the sedimentation pond and provide removal of BOD, ammonia, suspended solids and other filterable components of the leachate. The wetland will be designed to remove constituents by sorption, aerobic and anaerobic degradation, and plant uptake. Aeration of the wetland provides a stable oxic environment for the growth of aerobic bacteria responsible for the oxidation of complex hydrocarbons and ammonia. The gravel bed of the wetland will be underlain with a network of aeration tubing that provides uniform air flow across the floor of the bed.

Removal of Zinc is required but the magnitude of removal is small (0.306 mg/L to 0.110 mg/L). It is reasonable to expect that this level of removal will be achieved through various precipitation and sorption mechanisms in the system and that a dedicated treatment process is not warranted. However, Zinc removal can be achieved by the use of limestone beds or beds with similarly high carbonate content. It is recommended that candidate aggregate for the wetland be evaluated for carbonate content and that an aggregate with high carbonate content be utilized for 25% of the second stage wetland.

A schematic of the process train is provided below:

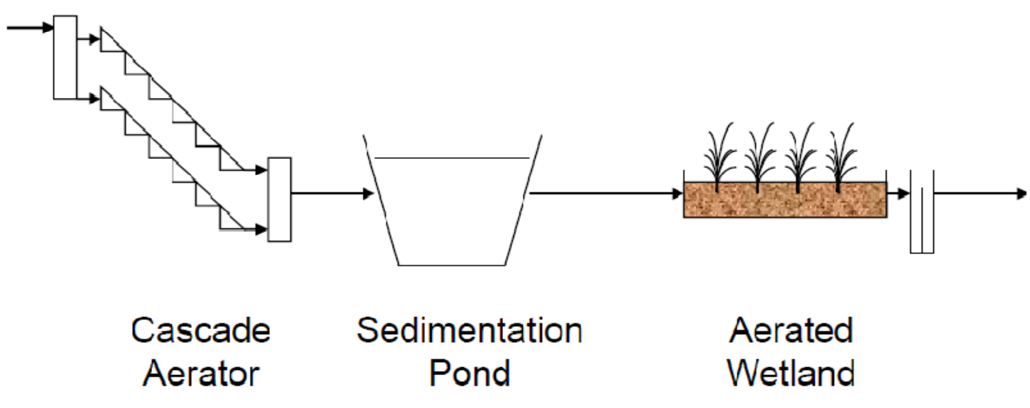


Figure 3: Process Schematic

UNIT SIZING

CASCADE AERATOR

The purpose of a cascade aerator is to increase dissolved oxygen levels in the water. The unit is designed to induce turbulent flow, which increases the mixing of air with the influent water. Typically, the units include a series of hydraulic steps, however inclined corrugated pipes or flow over rip rap can also be used. Cascade aerators are commonly used to elevate dissolved oxygen from the discharges of sewage plants and for removing iron and manganese from mine tailing pond waters.

Empirical formulas have been derived by Barret (1960) that relate the height of a cascade aerator to the oxygen deficit ratio, wastewater type, cascade geometry, and water temperature. The equation is provided below:

Equation 1: Cascade Aerator Sizing

$$H = \frac{R-1}{0.11ab(1+0.046T)}$$

where $R = \frac{C_s - C_o}{C_s - C}$

C_s = Dissolved-oxygen saturation concentration of the wastewater at temperature T, mg/l
C_o = Dissolved-oxygen concentration of the post-aeration influent, mg/l
C = Required final dissolved-oxygen level after post-aeration, mg/l
a = Water quality parameter equal to 0.8 for a wastewater treatment plant
b = Weir geometry parameter
T = Water temperature, degrees Celsius
H = Height through which water falls, ft

The following input values were used with the Barret equation to determine a cascade aerator height of 3.9 feet:

1. C_s: 10.2 mg/l (saturation at 15 deg. C)
2. C_o: 0 mg/l (assume anaerobic groundwater)
3. C: 4 mg/l (to achieve iron precipitation)
4. a: 0.8 (conservative assumption)
5. b: 1.1 (assume steps)
6. T: 15 deg. C (assumed leachate temperature)

Due to the relatively low flow, the cascade aerator will be constructed out of corrugated pipe.

A recent study was conducted by BB & E (see attached) to support a similar design and it is expected that the results from the study would be an excellent reference for quantifying performance of a similarly designed cascade aerator.

SEDIMENTATION POND/TANK

Two days of detention time is recommended for sizing sedimentation ponds (Laine and Jarvis, 2003). Using a 40,000 gpd design flow for sedimentation, a settling volume of roughly 80,000 gallons is recommended. The sedimentation volume could be provided by two subsurface 40,000 gallon fiberglass tanks. This option is costly and would be recommended primarily to eliminate free water surfaces at the site; bird attraction is a concern of the nearby Hilo Airport. A more cost effective approach would involve the construction of an earthen sedimentation pond with bird prevention wires (see photo below). The wires have proven effective at Edmonton International Airport (EIA) in Alberta, Canada.

Figure 4: Bird Prevention Wires (EIA, Alberta, Canada)

Suggested dimensions of the pond are provided in the table below:

Table 3: Sedimentation Pond Dimensions

Flow	0.040	MGD
Radius (at toe of slope)	1	feet
Length (waterline)	75	feet
Length (bottom)	39	feet
Width (waterline)	50	feet
Width (bottom)	14	feet
Depth (waterline to bottom)	6.0	feet
Slope	3.0	to 1
Freeboard	3.0	feet
Length (at top of berm)	93	feet
Width (at top of berm)	68	feet
Water Surface Area	3,436	sq.ft.
Volume	10,694	cu. ft.
Detention Time	2.0	days

AERATED WETLAND

Contaminant removal in aerated wetlands commonly follows first order removal kinetics that utilizes the following equation:

Equation 2: First Order Removal

$$\left(\frac{C - C^*}{C_i - C^*}\right) = \frac{1}{(1 + k/Pq)^P} = \frac{1}{(1 + k_V \tau / P)^P}$$

Where:

C	=	Effluent Concentration
C*	=	Background Concentration
C_i	=	Influent Concentration
k	=	Contaminant specific removal rate
P	=	Apparent number of tanks in series
q	=	Hydraulic loading rate
k_v	=	Modified first-order volumetric rate constant
t	=	Residence time

Using the above equation and estimated values for rate constants, aerated wetland areas were calculated based on the design influent characterization and discharge standards. The calculations assume an aerated bed of 1 meter water depth.

Table 4: Aerated Wetland Sizing

Flow	0.04	0.04	MGD
Compound	BOD	Ammonia	
Influent, C_i	700	85	mg/L
Effluent, C	30	4.9	mg/L
Residual, C*	5.0	0.0	mg/L
Tanks in	2	2	TIS
kv	8.69	5.7	d^{-1}
Porosity	0.3	0.3	
Detention	0.98	1.11	days
Bed Volume	648	732	CY
Bed Depth	3.28	3.28	ft
Bed Area	5,339	6,028	sf

The aerated wetland would be configured as a two stage process. The first stage will be designed for removal of organics. Due to the relatively high initial concentrations of BOD, the first stage bed will be engineered in a vertical flow configuration that maximizes the influent flow "window" and reduces the potential for clogging related to overloading. The second stage of the wetland will be designed for ammonia removal and have a horizontal flow configuration.

Based on a total bed volume of 1,380 CY, the wetland will be laid out as two side by side, 107 feet x 53 feet beds. The beds will be engineered such that the hydraulics of the beds can be modified based on the changes in flow and characterization associated with the aging landfill.

The aerated wetland will require two regenerative blowers sized at 380 SCFM and 3 psig each. Each blower will be approximately 7.5 HP. Two wetland influent pumps (one operating and one standby) will be required to deliver flow from the sedimentation pond/tank to the aerated wetland. Each pump will be sized for 55 gpm at 15 feet total dynamic head. Each pump will be approximately 0.75 HP.

LAYOUT

A conceptual layout of the proposed design is provided below:

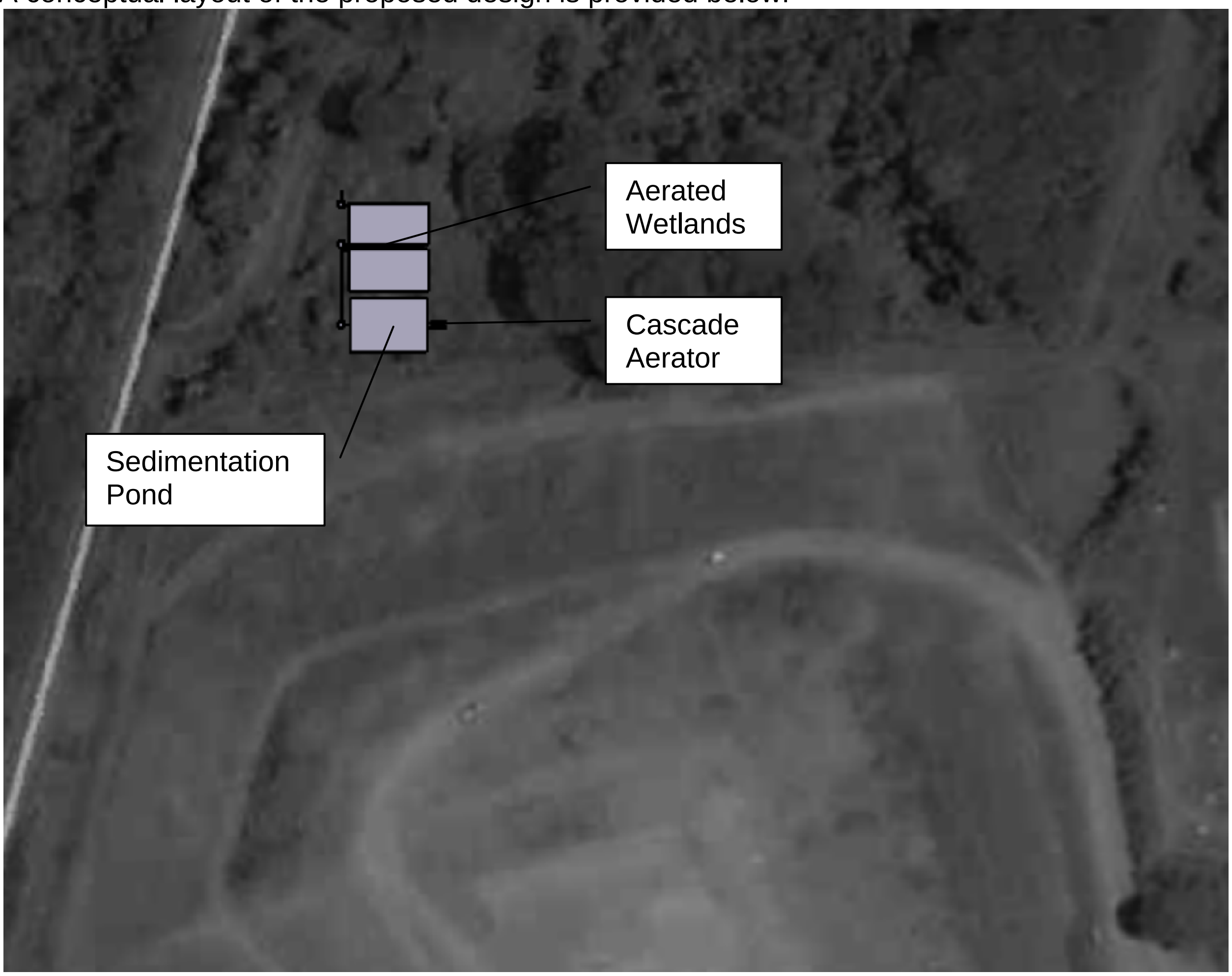


Figure 5: Conceptual Layout of Treatment System

COST ESTIMATE

A preliminary engineer's opinion of cost is provided below:

Engineer's Opinion of Preliminary Design Project Costs					
South Hilo Landfill - Treatment Wetland					
ITEM		**QUANTITY**	**UNIT**	**UNIT COST**	**TOTAL COST**
Cascade Aerator					
	Earthwork	1	LS	$ 5,000	$ 5,000
	Manhole, Gaskets, Penetrations, Grating	1	each	$ 5,000	$ 5,000
	Culvert Pipe	30	LF	$ 50	$ 1,500
Sedimentation Ponds					
	Earthwork	400	CY	$ 10.00	$ 4,000
	Liner 60 mil HDPE (with geotextile beneath	8500	SF	$ 2.00	$ 17,000
	Yard Piping	1	LS	$ 10,000	$ 10,000
	Hydraulic Control Structure and Lift Statior	1	LS	$ 75,000	$ 75,000
Aerated Wetland					
	Earthwork	1200	CY	$ 10	$ 12,000
	Sand	200	CY	$ 30	$ 6,000
	Liner 60 mil HDPE (with geotextile beneath	15000	SF	$ 2.0	$ 30,000
	Walls	700	LF	$ 10	$ 7,000
	Aggregate	1000	CY	$ 50	$ 50,000
	In-Bed Pipe	1000	LF	$ 5	$ 5,000
	Hyraulic Control Structure	2	each	$ 10,000	$ 20,000
	Plants	2000	each	$ 5	$ 10,000
	Water Balance Test	2	each	$ 3,000	$ 6,000
	Blowers and Enclosures	2	each	$ 10,000	$ 20,000
	Aeration Manifold	200	LF	$ 15	$ 3,000
	Aeration Tubing	50000	LF	$ 1.50	$ 75,000
	Panel	1	each	$ 5,000	$ 5,000
	Electrical supply (Panel to Blower)	1	LS	$ 3,000	$ 3,000
CONSTRUCTION TOTAL					**$ 369,500**
Notes:					
1) Units costs are for supply and install					
2) Electricity to panel not included					

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Treatment of landfill leachate using an aerated, horizontal subsurface-flow constructed wetland

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Abstract

A pilot-scale subsurface-flow constructed wetland was installed at the Jones County Municipal Landfill, near Anamosa, Iowa, in August 1999 to demonstrate the use of constructed wetlands as a viable low-cost treatment option for leachate generated at small landfills. The system was equipped with a patented wetland aeration process to aid in removal of organic matter and ammonia nitrogen. The high iron content of the leachate caused the aeration system to cease 2 years into operation. Upon the installation of a pretreatment chamber for iron removal and a new aeration system, treatment efficiencies dramatically improved. Seasonal performance with and without aeration is reported for 5-day biochemical oxygen demand (BOD_5), chemical oxygen demand (COD), ammonia nitrogen (NH_4–N), and nitrate nitrogen (NO_3–N).

Since winter air temperatures in Iowa can be very cold, a layer of mulch insulation was installed on top of the wetland bed to keep the system from freezing. When the insulation layer was properly maintained (either through sufficient litterfall or replenishing the mulch layer), the wetland sustained air temperatures of as low as −26 °C without freezing problems.

Keywords: Aeration; Ammonia; Clogging; Cold climate; Constructed wetland; Horizontal subsurface flow (HSSF); Iron; Landfill leachate; Nitrogen

1. Introduction

Over 14,000 landfills were closed in the United States between 1978 and 1988 (Mulamoottil et al., 1998). The number of active landfills in the United States is steadily decreasing; 8000 active sites were reported in 1988, but only 1858 landfills remained open in 2001 (U.S.EPA, 2003). Landfills that have been closed (which, by rough estimate, number over 20,000 in the United States alone) still require leachate collection and management.

Both surface-flow and subsurface-flow constructed wetlands have been identified as promising technologies for the treatment of landfill leachate (Kadlec and Knight, 1996). Constructed wetlands have a small ecological footprint, utilize "low-tech" technology, and have an aesthetic value similar to that of natural wetlands. The application of wetland technology for treating landfill leachate is still developing. There has been a call by academics and professionals alike for a better understanding of the movement, transformation, and removal

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of contaminants in these treatment systems through extensive and long-term studies (Mulamoottil et al., 1998).

In cold climates, ambient air temperatures during winter months are well below freezing and can reach lows of −30 °C or colder. As a result, wetland systems in cold climates must be designed and operated to sustain extended periods of sub-freezing air temperatures. Cold-climate constructed wetlands are typically insulated with a plant detritus layer (Brix, 1994; Smith et al., 1997) or a layer of mulch (Henneck et al., 2001; Mæhlum et al., 1995; Wallace et al., 2001), and the water surface is kept below the insulation/gravel interface. Keeping the water below the surface of the mulch effectively creates a layer of dry gravel, which provides additional thermal insulation to the system. The insulation layers minimize energy loss from the system, making subsurface-flow constructed wetlands a viable onsite treatment alternative in cold climates despite harsh climatic conditions.

Two of the main concerns in treating leachate are the high concentrations of organic matter and NH_4–N present in the waste stream. The main treatment mechanisms for nitrification and aerobic removal of organic material are oxygen-limited processes. Plants were originally thought to be the dominant oxygen-transfer mechanism in wetland treatment systems, but studies have shown that plant-mediated oxygen-transfer rates are very small relative to the oxygen demand exerted by the wastewater under common loading conditions (Brix and Schierup, 1990; Wu et al., 2001). As a result, many current wetland designs neglect plant-mediated oxygen transfer altogether.

In free water surface wetlands, oxygen is mainly supplied to the wetland through algal photosynthesis and atmospheric diffusion. However, in a cold-climate subsurface-flow system, algal photosynthesis is not an oxygen-transfer mechanism and the insulation layer limits atmospheric diffusion. The limited oxygen-transfer capability of standard subsurface-flow wetlands has led to the development of enhanced treatment systems. These enhanced systems are capable of providing sufficient oxygen transfer for nitrification and removal of organic material, introducing oxygen to the system through frequent water level fluctuation (tidal-flow) (Austin et al., 2003; Behrends et al., 1996; Zoeller and Byers, 1999), passive air pumps (vertical-flow) (Green et al., 1998) or direct mechanical aeration of the water in the gravel bed (horizontal-flow) (Dufay, 2000; Flowers, 2002; Wallace, 2001). The following study investigated the ability of an aerated horizontal subsurface-flow constructed wetland to treat landfill leachate in a cold temperate climate. This paper focuses particularly on cold-weather removal of organic matter and NH_4–N from landfill leachate.

2. Materials and methods

2.1. Site description

A pilot-scale subsurface-flow constructed wetland was installed at the Jones County Municipal Landfill near Anamosa, Iowa in August 1999 to demonstrate the use of constructed wetlands as a viable low-cost treatment option for leachate generated at small, rural landfills. The system was equipped with a patented aeration process (Wallace, 2001) to aid in the removal of the high concentrations of organic matter and NH_4–N present in the landfill leachate.

The treatment wetland consisted of one 93 m^2 (15.5-m long by 6-m wide) cell and was lined with an impermeable 30-mil high-density polyethylene liner. The impermeable liner ensured that groundwater did not interfere with the hydrology of the treatment system. The wetland bed was comprised of a 30-cm layer of pea gravel (d_{10} = 5.0 mm) underneath a 15-cm layer of well-decomposed yard waste.

The leachate generated at the Jones County Municipal Landfill was collected and stored in an underground storage tank (capacity of 38 m^3). A portion of the leachate stored in the tank was used to feed the pilot-scale wetland system, which was designed to operate at a hydraulic loading rate of approximately 4 mm/d, (corresponding roughly to 0.4 m^3/d). A ⅓-horsepower submersible sump pump was used to transport the leachate from the settling chamber into the wetland. Two programmable timers in series controlled the pump; one timer dictated the time of pumping events, and the other dictated the time between pumping events. This way, the quantity of flow delivered to the wetland could be adjusted relatively easily, even at low daily flow rates. Leachate was pumped into an above-grade wetwell at the inlet end of the wetland, where it flowed through a Zabel A-300 filter, into a flow meter (tipping bucket) enclosure, and was delivered to the wetland via the distribution header. The filter is specifically designed to remove particles larger than 1.5 mm from septic tank effluent and was installed to minimize the accumulation of solids in the influent distribution pipe. The influent header was a 10-cm diameter PVC pipe that ran along the width of the wetland cell just beneath the 15-cm layer of mulch. The pipe was drilled with evenly spaced holes in order to promote even distribution of the leachate to the wetland bed. Since the wetland was only used as a demonstration study, treated leachate was pumped back into the underground storage tank and was trucked to a wastewater treatment plant in Cedar Rapids, Iowa, as needed.

Accumulation of iron precipitate on the surface of the gravel media and aeration tubing in the wetland bed in 2002 resulted in a decreased treatment performance (Hoos, 2003), so a new aeration system and pretreatment chamber was installed in September 2002. The purpose of the pretreatment system was to oxidize the iron present in the leachate, and to settle the precipitate out of solution before pumping the leachate into the wetland system. Prior to the renovation there was no pretreatment system for the raw leachate.

Following the renovation, leachate was pumped from the underground storage tank into a pretreatment chamber (capacity of 2.6 m^3), where it was aerated in 15-minute intervals 24 h a day. Air was provided to the pretreatment chamber by an air blower. After the pretreatment chamber, the leachate flowed over a baffle wall into settling tank (capacity of 1.1 m^3) to allow particulate iron (mainly ferric hydroxide precipitates) to settle out of solution before the leachate was pumped into the wetland. The settling chamber was approximately 0.5 m (L) by 0.5 m (W) by 2 m (D). Influent total iron concentrations ($Fe^{+2}+Fe^{+3}$) to the wetland decreased from 21 mg/L to 9 mg/L upon installation of the pretreatment chamber (Hoos, 2003).

2.2. Aeration system

The wetland bed was designed with a patented integral aeration system (Forced Bed Aeration™) developed by Wallace (2001). The original aeration system consisted of an air blower, a 5-cm PVC air distribution header, and four 15-meter loops of 1.25-cm diameter perforated flexible HDPE tubing. The tubing was placed on top of the impermeable liner during construction before the pea gravel was added. Air was delivered to the wetland bed 12 h a day (6 a.m. to 6 p.m.) to enhance oxygen-limited microbial processes but still allow for anaerobic processes to occur at night.

The gravel media and aeration tubing became clogged with ferric hydroxide precipitates during late 2001 and early 2002. In April 2002 the aeration system was taken offline completely, and the system operated without supplemental air until the aeration tubing was replaced during the September 2002 renovation. The September 2002 renovation included replacement of the air blower (capable of delivering 1.8 m^3 of air per minute to the system) and installation of new, replaceable aeration lines. The new aeration lines were made of perforated 2.5-cm diameter steel pipe and were placed perpendicular to the direction of flow (the drilled holes were 3 mm in diameter, and were drilled evenly over the surface area of the pipe). Infiltration chambers were placed over the aeration lines to protect the lines from future clogging problems, and to facilitate the replacement procedure should it be necessary to do so in the future. Renovation work was done “in the wet” because it is very difficult to drain subsurface-flow wetland beds without extensive pumping equipment (which was not available at the research site). As a result, in some instances it was difficult to place the new aeration pipes directly on the wetland bottom.

The steel aeration piping was placed as close to the liner as possible, which in most cases was within a few centimeters at the bottom of the wetland cell (Hoos, 2003). The new aeration lines were placed at transects across the width of the wetland, at fractional distances of approximately 20%, 40%, 60%, and 80% through the cell. The resulting configuration of the aeration system was significantly different than the original layout. Fig. 1 is a schematic of the original aeration system (a) and the replacement aeration system (b).

2.3. Instrumentation

Flow was measured using tipping buckets equipped with a DigiKey magnetic sensor and HOBO event logger (Onset Computer Corporation, Massachusetts). Three HOBO TMC5-HA temperature sensors were connected to a four-channel HOBO H8 data-logging recorder to record influent water temperature, effluent water temperature, and air temperature at 6-hour intervals. Data

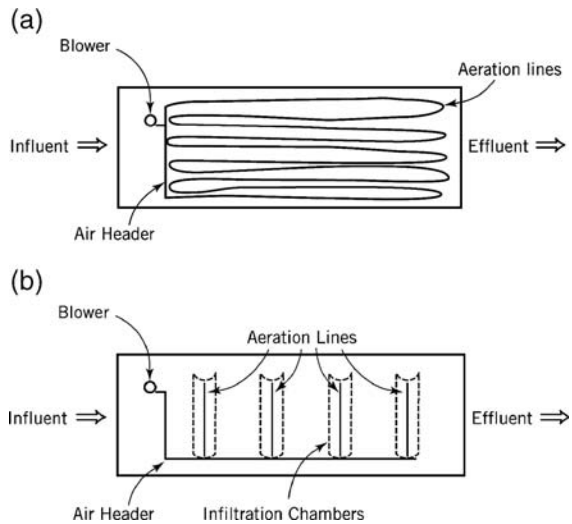

Fig. 1. Layout of original (a) and replacement (b) aeration system.

was downloaded from the logger onto a laptop computer using BoxCar Pro 4.2.10 software.

2.4. Sampling

In addition to influent and effluent sampling locations, the wetland was equipped with a set of sampling ports that provide access to flow at specified depths (15, 30, and 45 cm) at four locations along the centerline of the wetland. Sampling locations are located at the influent, and at 25%, 50%, and 75%, and 100% of the bed length. Effluent samples (100% of bed length) were collected from the effluent standpipe. Fig. 2 shows the layout of the effluent water level control structure. Samples were generally taken during midday, when the aeration system was in mid-cycle.

2.5. Plants

The wetland system was originally planted with 100 potted stiff goldenrod (*Solidago rigida*) plants in August 1999 (Cross, 2001). Plants were spaced evenly in the wetland bed (one plant per m^2). Goldenrod is a facultative wetland plant, and is known for its ability to root deeply in soil (Boon and Groe, 1990). The goldenrod proved to be hearty and capable of thriving in the wetland system despite the high NH_4–N concentrations in the leachate (Cross, 2001).

The September 2002 renovations were very intrusive, and most of the plant community was disrupted due to the trench work associated with replacing the aeration lines. It was thought that the goldenrod plants would rejuvenate in the next growing season, but they did not. As a result, local plant species, such as curly dock (*Rumex crispus*), bull thistle (*Cirsium vulgare*), stinging nettle (*Urtica dioica*), willow (*Salix* spp.), and cannabis (*Cannabis sativa*) dominated the plant community in the wetland, with self-established patches of cattail (*Typha latifolia*) near the effluent end of the bed. The system was seeded in Fall 2003 with goldenrod in an attempt to re-establish the original plant community, but the seeds did not produce plants in Spring 2004. *R. crispus*, a short plant with a massive tuberous and shallow root structure, was the dominant plant species in the wetland between 2003 and 2005 and did not provide any substantial detritus layer during the winters of 2003 and 2004.

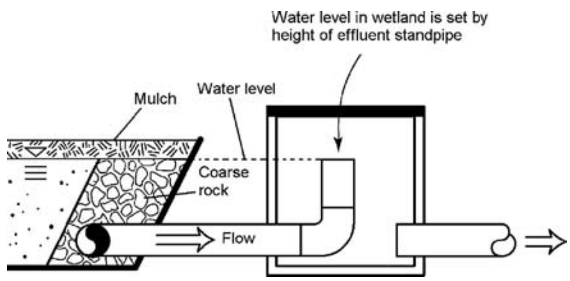

Fig. 2. Schematic of water level control structure.

2.6. Water quality analyses

The following parameters were determined in accordance to Standards Methods (American Public Health Association et al., 1998): Ammonia Nitrogen (Method 4500-NH_3 F, using an Orion 95–12 ion selective electrode which was calibrated with standard solutions before each use), 5-day biochemical oxygen demand (BOD_5) (Method 5210 A, using a Hach BODTrak instrument), chemical oxygen demand (COD) (Method 5220 D, using the Hach COD Digestion Solution), total iron (1,10-Phenathroline method), and anions (nitrate, nitrite) (Method 4110, using a Dionex ICS 2000 Ion Chromatograph, IonPac AS-18 column, an eluant solution of 39 mM KOH, and a Dionex AS50 Autosampler). For COD measurements, the leachate and standards were diluted 1:20 with deionized water so that potential interference due to the color of the leachate was minimized.

3. Results and discussion

3.1. Leachate characteristics

The leachate at the Jones County Landfill is similar to that of the leachate from other landfills in the Midwestern United States. The leachate is characterized by relatively high concentrations of COD and NH_4–N. The Jones County wetland system was designed using the results of a 1994 laboratory analysis. The data in Table 1 show the 1994 and 1999–2004 average characteristics of the Jones County Landfill leachate compared against that of other aging landfills in the Midwest (Kadlec, 1999; Nivala, 2005). Most of the contaminant concentrations in the Jones County leachate have decreased significantly since 1994, indicating that the landfill is in the later phases of leachate generation.

3.2. Temperature and insulation

During the 5 years of operation, air temperatures at the Jones County wetland site ranged from 37 °C to −26 °C. Mean air temperature during the coldest month of the year (February) was ranged from −7.5 °C to −0.3 °C. Mean water temperature during this time

Table 1
Selected constituents in leachate from landfills in the Midwestern United States

Constituent (mg/L)	Fulton County, Indiana [a]	Sarnia, Ontario, Canada [a]	City Sand, Michigan [a]	Saginaw, Michigan [a]	Jones County, Iowa [b]	
					1994	1999–2004
BOD	390	407	312	729	110	116
COD	1540	1036	3203	–	2210	781
TSS	7840	–	241	–	5790	186
NH_4^+–N	284	254	2074	322	423	212
NO_3^-–N	3	<0.3	0	0	<10	2
Iron	178	17.6	–	22	470	21

[a] (Kadlec, 1999).
[b] This Study.

ranged from −0.4 °C to 1.8 °C. A stock tank heater was installed in the influent holding tank during February 2004 to ensure that the leachate in the aboveground influent wetwell would not freeze. Installation of the heater helped to keep influent water temperatures above freezing through the remainder of the study. During the winter of 1999–2000, the minimum effluent water temperature was approximately −0.2 °C, although the effluent water did not freeze. It is speculated that the high concentration of total dissolved solids (TDS) in the leachate depressed the freezing temperature. This phenomenon was observed again during the winter of 2003–2004.

As mentioned previously, an insulating layer of mulch was installed over the surface of the wetland bed to keep the system from freezing. When the insulation layer was properly maintained, (either through sufficient litterfall or replenishing the mulch layer), the wetland sustained air temperatures as low as −26 °C without freezing.

3.3. Performance

BOD_5 removal efficiencies for the wetland system were high (greater than 90%) from startup in Fall 1999 until Fall 2001, at which time the orifices in the aeration tubing began to clog with iron precipitates (Fig. 3). Decreased performance was observed from Fall 2001 through Spring 2002. In April 2002, it was clear that the aeration system was delivering only minimal air to the

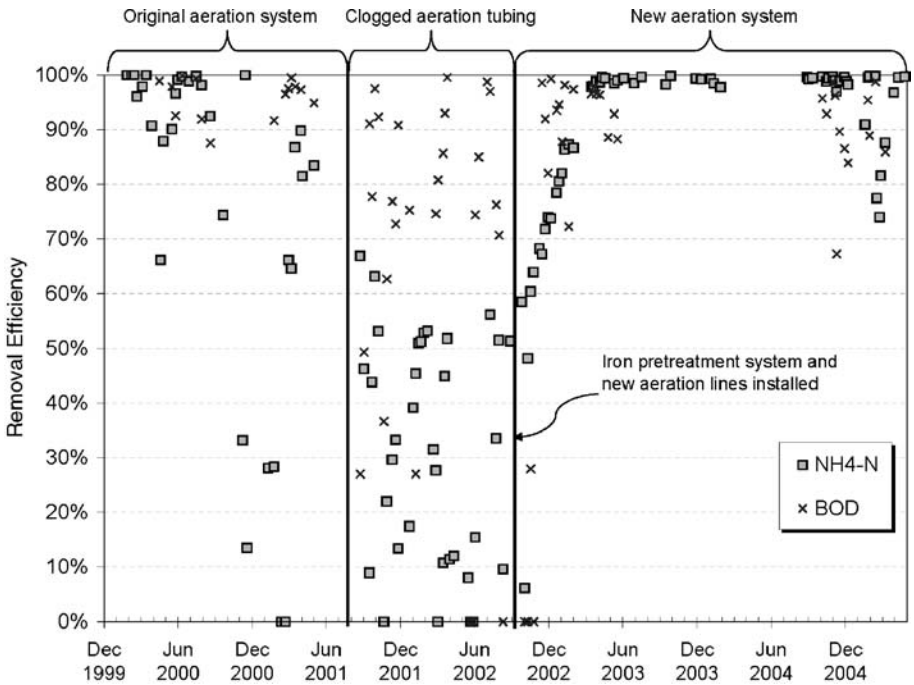

Fig. 3. NH_4–N and BOD removal efficiencies, 1999–2005.

subsurface environment in the wetland, so it was taken completely offline. The wetland operated without supplemental aeration from April 2002 until September 2002. During this time, BOD removal efficiencies were sporadic, ranging from 0% to 100%. The huge variations likely occurred because air was not uniformly distributed in the wetland cell due to iron fouling. BOD_5 Removal efficiencies following the renovation improved immediately and remained high (in excess of 90%) throughout the remainder of the study (Fig. 3), even during the winter months. A gap in the data exists between Fall 2003 and Fall 2004 due to problems with laboratory equipment.

Removal of NH_4^+–N was also greater than 90% within 4 months of startup, and remained high until the aeration lines became clogged with ferric hydroxide precipitates. Without supplemental aeration, treatment performance was inconsistent and very poor, oftentimes resulting in zero removal. Performance improved significantly in the 3 months following the renovation, and remained high (greater than 90%) for the remainder of the study (Fig. 3). The decrease in NH_4^+–N removal in early 2005 coincided with a snowmelt and large rainfall event; during this time, the wetland system received a surge of rainwater and melted snow. The surge of precipitation temporarily reduced the hydraulic residence time of the wetland and diluted the NH_4^+–N concentration in the leachate.

Table 2
Mean values of monitored parameters during period of no aeration (by season)

No Aeration	Winter	Spring	Summer	Fall
	Jan–Mar	Apr–Jun	Jul–Sep	Oct–Dec
Q (m^3/d)	0.14	0.17	0.29	0.18
T_{air} (°C)	−0.8	15.1	21.4	7.3
T_{water} (°C)	1.7	12.0	19.6	10.1
$BOD_{5\ in}$ (mg/L)	308	171	85	89
$BOD_{5\ out}$ (mg/L)	59	36	21	22
BOD Efficiency	*81%*	*79%*	*76%*	*75%*
COD_{in} (mg/L)	1108	626	525	606
COD_{out} (mg/L)	525	674	513	525
COD Efficiency	*53%*	*0%*	*2%*	*13%*
NH_4–N_{in} (mg/L)	176	93	139	173
NH_4–N_{in} (mg/L)	105	80	80	118
NH_4–N Efficiency	*40%*	*14%*	*43%*	*32%*
NO_3–N_{in} (mg/L)	0.4	0.4	0.1	0.0
NO_3–N_{out} (mg/L)	0.0	0.0	0.0	0.0
pH_{in}	–	7.9	7.8	7.8
pH_{out}	–	7.9	7.8	7.4

Table 3
Mean values of monitored parameters during periods of aeration (by season)

Aeration	Winter	Spring	Summer	Fall
	Jan–Mar	Apr–Jun	Jul–Sep	Oct–Dec
Q (m^3/d)	0.27	0.20	0.43	0.53
T_{air} (°C)	−2.0	13.6	20.9	5.6
T_{water} (°C)	1.8	11.5	19.5	10.0
$BOD_{5\ in}$ (mg/L)	97	110	177	41
$BOD_{5\ out}$ (mg/L)	12	5.0	5.5	4.7
BOD Efficiency	*88%*	*95%*	*97%*	*89%*
COD_{in} (mg/L)	734	773	873	628
COD_{out} (mg/L)	414	406	350	409
COD Efficiency	*44%*	*48%*	*60%*	*35%*
NH_4–N_{in} (mg/L)	195	175	253	190
NH_4–N_{in} (mg/L)	14	6.8	4.3	13
NH_4–N Efficiency	*93%*	*96%*	*98%*	*93%*
NO_3–N_{in} (mg/L)	6.9	13.7	0.3	2.3
NO_3–N_{out} (mg/L)	75	50	21	51
pH_{in}	8.0	8.0	7.8	8.2
pH_{out}	7.7	7.7	7.7	7.9

Seasonal mean influent and effluent concentrations for COD, BOD_5, NH_4^+–N, NO_3–N, and pH for the period of non-aeration are presented in Table 2. Flow, air temperature, and water temperature are also provided. The same parameters for the period of aeration are presented in Table 3. Pollutant removal efficiencies for BOD_5, COD, and NH_4^+–N were calculated as concentration reduction (Kadlec and Knight, 1996). Data in Tables 2 and 3 show that during the winter, mean water temperature (as measured at the wetland effluent) was warmer than the mean air temperature. There does not appear to be a strong relationship between treatment performance for BOD_5, COD, or NH_4^+–N without aeration. However, with aeration, highest removal efficiencies were observed in the summer for all three parameters. Springtime performance was also good. The pH values were generally neutral, even when significant nitrification was occurring, indicating that the wetland system had a pronounced buffering capacity.

Aeration greatly improved treatment performance, as can be seen by comparing the data in Tables 2 and 3. BOD_5 removal efficiencies for the wetland system were relatively high (between 75% and 81%) even in the absence of supplemental aeration, but improved (up to 97% removal) with aeration. A significant portion of the COD in the leachate at the Jones County Landfill contained organics that were not readily biodegradable, which is likely because the landfill is in the later stages

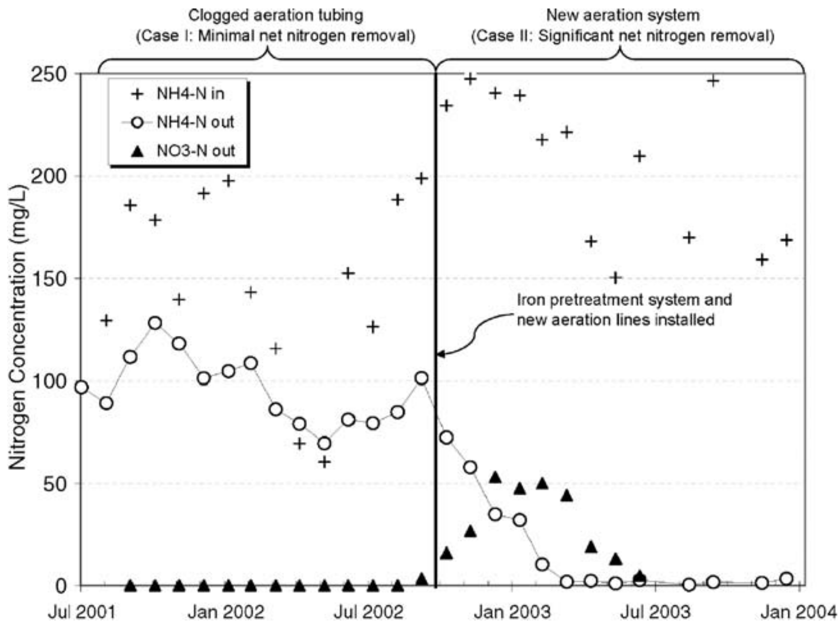

Fig. 4. Monthly average ammonia nitrogen (NO_4–N) and nitrate–nitrogen (NO_3–N) concentrations before and after installation of the replacement aeration system.

of leachate generation. As a result, a sizeable fraction of the COD passed through the wetland, exiting with the effluent flow. COD removal without aeration was intermittent and poor (Table 2), but improved and became consistent (Table 3) when the wetland system was aerated. NH_4^+–N efficiencies ranged between 14% and 40% without aeration. With aeration, NH_4^+–N removal efficiencies were consistently high (93%–98%), indicating that cold-weather nitrification is possible. NO_3–N production did not occur during periods of non-aeration but was significant when the wetland system was aerated (Tables 2 and 3). Overall, effluent NO_3–N was greatest during the winter and lowest during the summer. It is speculated that denitrification was affected by water temperature, but limited by available carbon. Carbon-limited denitrification has been observed in other wetland treatment systems (Hamersley and Howes, 2002; Hume et al., 2002; Kozub and Liehr, 1999).

The presence of low-level influent NO_3–N during the period of aeration is most likely due to the flow scheme described in Section 2.1; although the leachate storage tank was large, some of the effluent nitrogen could have been “recycled” back to the wetland effluent. Another possible explanation for this is the addition of the pretreatment system, where partial nitrification could have occurred before the leachate entered the wetland system.

The generation of NO_3–N in a wetland system is dependent on many factors, including influent total nitrogen loading, dissolved oxygen concentration in the water column, and temperature. NO_3–N production in the Jones County wetland system is presented Fig. 5. The data are presented in conjunction with influent and effluent NH_4^+–N concentration because simply reporting effluent NO_3–N concentration is ambiguous without contextual data. Low concentrations of effluent NO_3–N can mean one of two things; (Case I) that nitrification is not occurring and NO_3–N is not being formed (corresponding to minimal net removal of nitrogen), or (Case II) that both nitrification and denitrification processes are occurring to completion (corresponding to high net removal of nitrogen). Both Case I and Case II are present in Fig. 4.

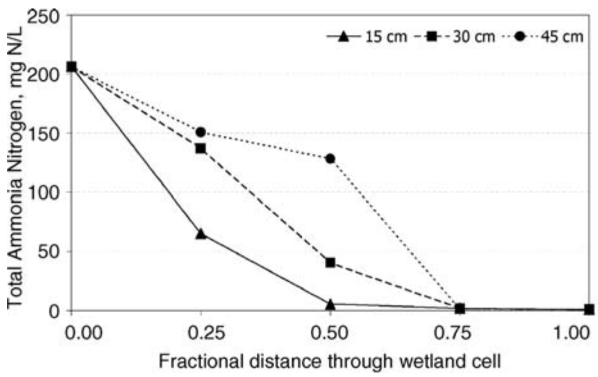

Fig. 5. Vertical stratification of ammonia nitrogen in the wetland cell at 15, 30, and 45-cm depths (October 2004).

Case I (little or no net removal of total nitrogen) was observed in the period from Fall 2001 through Fall 2002. There was a little net reduction of NH_4^+–N because there was not enough oxygen to support nitrification. As a result, NO_3–N was not produced in the wetland during this time and high effluent concentrations of NH_4^+–N were observed (Fig. 4).

Case II (substantial net removal of nitrogen) occurred in the time following the September 2002 modifications when the replacement aeration system was run in a 12-hour cycle (12 h on and 12 h off). Following the modifications, NO_3–N formation in the wetland increased substantially because the oxygen-rich conditions allowed ample nitrification to occur (Fig. 4). Total nitrogen removal (complete nitrification followed by denitrification) was established by Summer 2003, at which time both effluent NH_4^+–N and NO_3–N concentrations were very low (less than 2.0 mg N/L and 5.0 mg N/L, respectively). The lag time that was observed before NO_3–N removal occurred is somewhat unusual, as denitrifying microorganisms are generally heartier than nitrifiers.

Vertical stratification was considerable through the first 75% of the cell. The deepest sampling locations, which were located along the bottom of the wetland cell (45 cm depth), exhibited a noticeably different NH_4^+–N profile than the shallower (15 cm and 30 cm depth) locations (Fig. 5). Vertical stratification has been observed in other subsurface-flow wetlands treating landfill leachate (Liehr et al., 2000; Sanford, 1999). However, most of the wetland systems reported in the literature are not equipped with supplemental aeration. The stratification could be explained due to the differences in the density of leachate and rainwater. The integral aeration system in the Jones County wetland was believed to provide the system with a better degree of internal mixing than conventional (non-aerated) wetland treatment systems. Fig. 5 shows a typical NH_4–N vertical profile for the Jones County wetland treatment system.

There was a short period after the September 2002 modifications in which vertical stratification was not observed (Hoos, 2003), but within 6 months, the original stratification pattern reappeared. The preferential flow path likely reduced the hydraulic retention time of the wetland and reduced the overall treatment efficiency of the system. Despite this, stratification has not prevented reliable treatment of NH_4^+–N to concentrations below 5 mg/L, even during the winter months.

4. Conclusion

Aerated subsurface-flow constructed wetlands are a viable low-cost treatment alternative for the treatment of landfill leachate. With adequate insulation and aeration, these systems can perform well, even during sub-freezing temperatures. Removal efficiencies during periods of no aeration were inconsistent and poor for BOD_5, COD, and NH_4^+–N. However, with a sufficient oxygen supply and pretreatment system for iron removal, treatment efficiency for these parameters was greatly increased, and was highest during the summertime. Notable NO_3–N formation was only observed when the wetland system was aerated. Net nitrogen removal in this type of wetland system is possible with a cyclic aeration scheme (12 h on/12 h off).

Acknowledgements

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Wellsville Treatment Wetland

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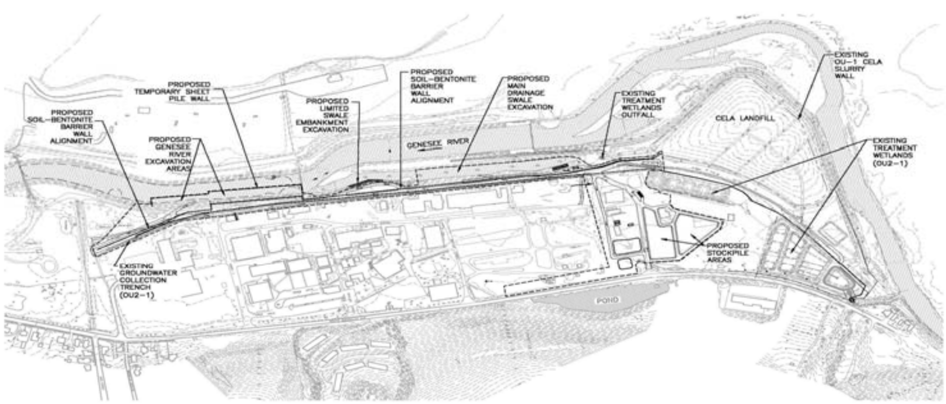
Process design experts in treatment wetlands

Naturally Wallace Consulting

Closure Plan

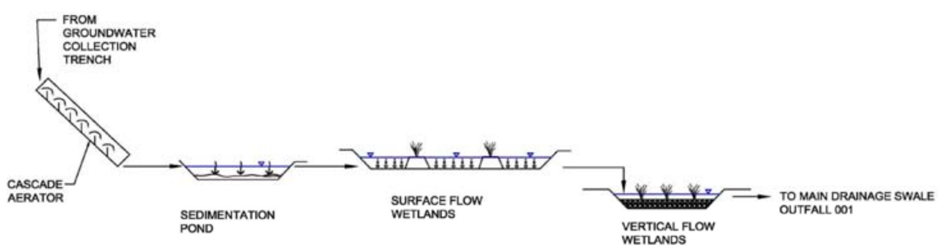


Contaminants of Concern

Parameter	Concentration Range	Proposed Limit
Iron	3- 40 mg/L	2 mg/L
Manganese	4-6 mg/L	0.5 mg/L
Nitrobenzene	1000-5000 ug/L	5 ug/L
Aniline	100-700 ug/L	10 ug/L

Treatment Concept

- Cascade Aerators (iron oxidation)
- Sedimentation Pond (iron precipitation and settling)
- Surface Flow Wetlands (hydrocarbon removal)
- Vertical Flow Wetlands (pH adjustment)



Cascade Aerator Construction

Sedimentation Pond

Naturally Wallace Consulting

Surface Flow Wetlands

Vertical Flow Wetlands

Start Up

- November 2008
- Highly variable flow and load conditions

Summer 2009

- Groundwater collection problems solved
- Vegetation established in the wetlands
- Aeration of deep zones in the surface flow wetlands

Performance Summaries

- Benzene
- Toluene
- Ethylbenzene
- Xylenes
- Aniline
- Nitrobenzene
- 2-Methylnaphthalene
- Iron
- Arsenic
- Manganese
- pH

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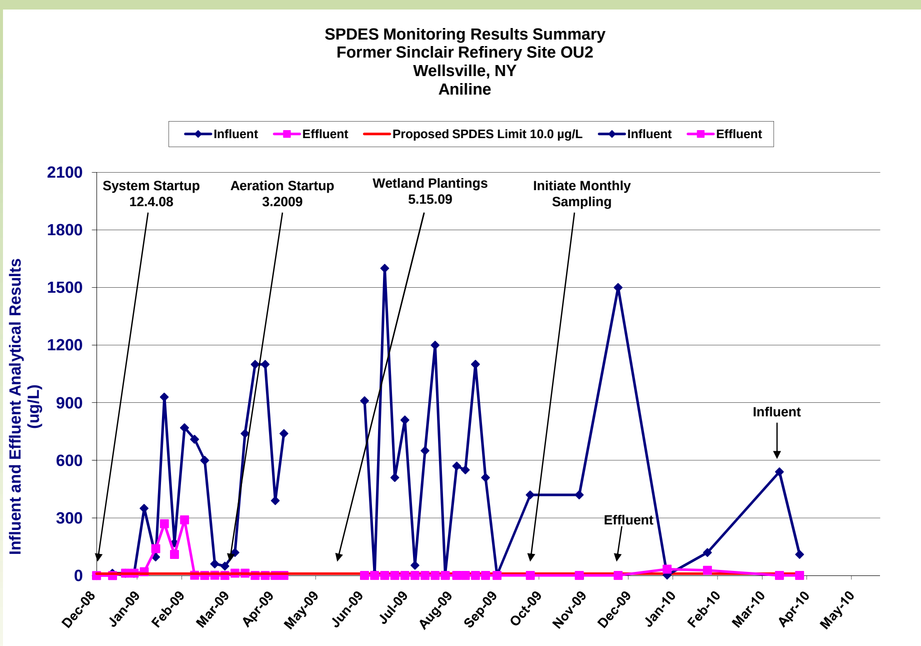

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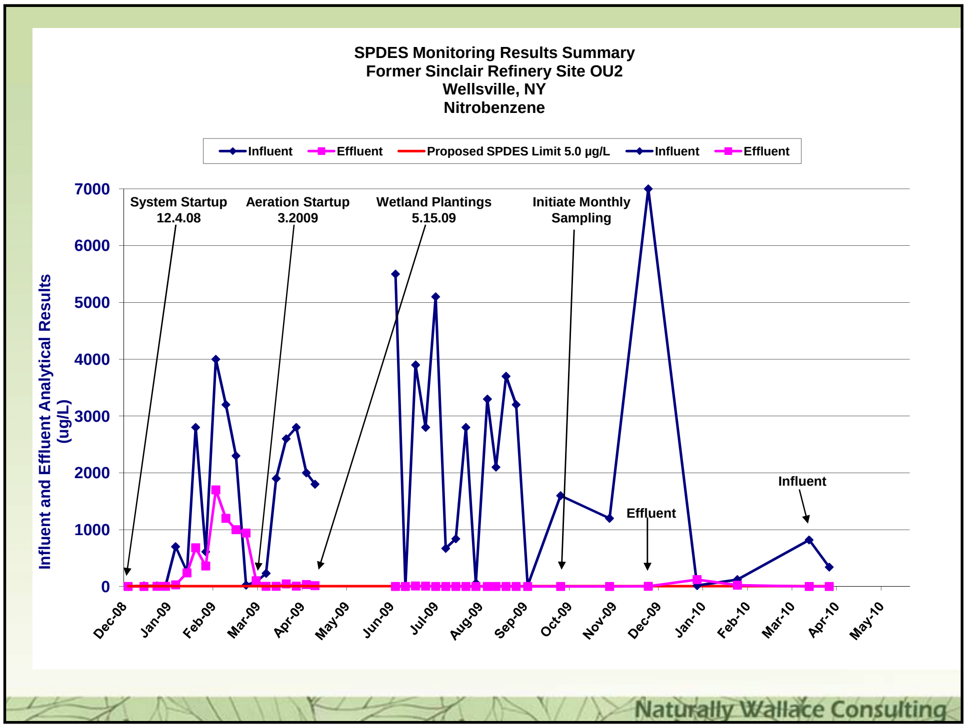
SPDES Monitoring Results Summary
Former Sinclair Refinery Site OU2
Wellsville, NY
Nitrobenzene
Influent
Effluent
Proposed SPDES Limit 5.0 µg/L
Influent
Effluent
Influent and Effluent Analytical Results (ug/L)
7000
6000
5000
4000
3000
2000
1000
0
System Startup 12.4.08
Aeration Startup 3.2009
Wetland Plantings 5.15.09
Initiate Monthly Sampling
Influent
Effluent
Dec-08
Jan-09
Feb-09
Mar-09
Apr-09
May-09
Jun-09
Jul-09
Aug-09
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Oct-09
Nov-09
Dec-09
Jan-10
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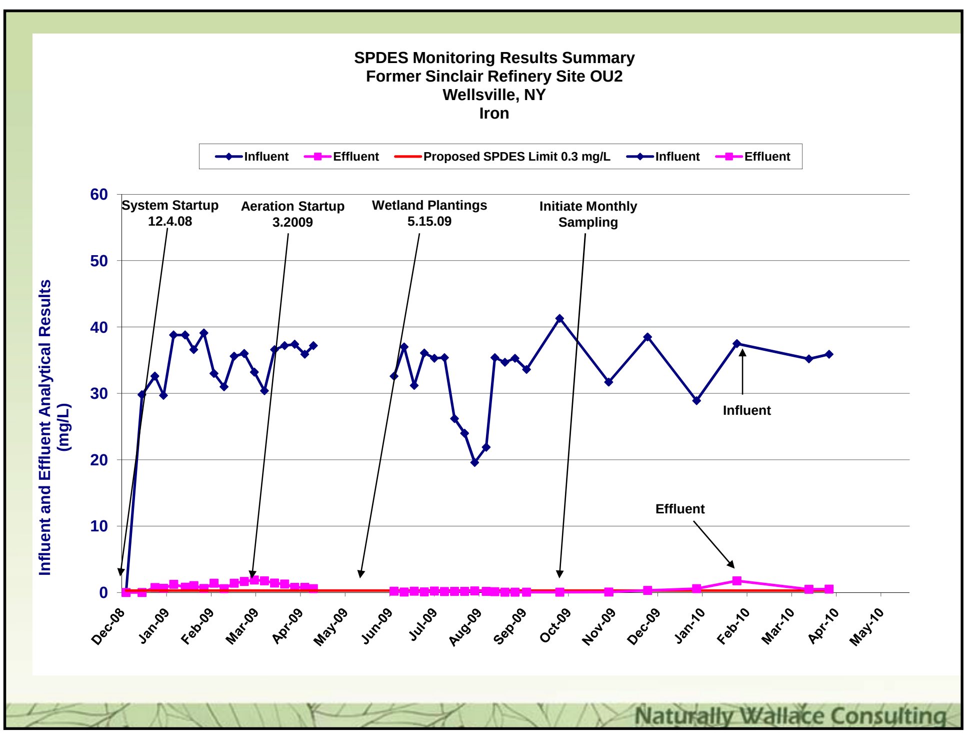
Mar-10
Apr-10
May-10
Naturally Wallace Consulting


SPDES Monitoring Results Summary
Former Sinclair Refinery Site OU2
Wellsville, NY
Iron
Influent
Effluent
Proposed SPDES Limit 0.3 mg/L
Influent
Effluent
Influent and Effluent Analytical Results (mg/L)
60
50
40
30
20
10
0
System Startup 12.4.08
Aeration Startup 3.2009
Wetland Plantings 5.15.09
Initiate Monthly Sampling
Influent
Effluent
Dec-08
Jan-09
Feb-09
Mar-09
Apr-09
May-09
Jun-09
Jul-09
Aug-09
Sep-09
Oct-09
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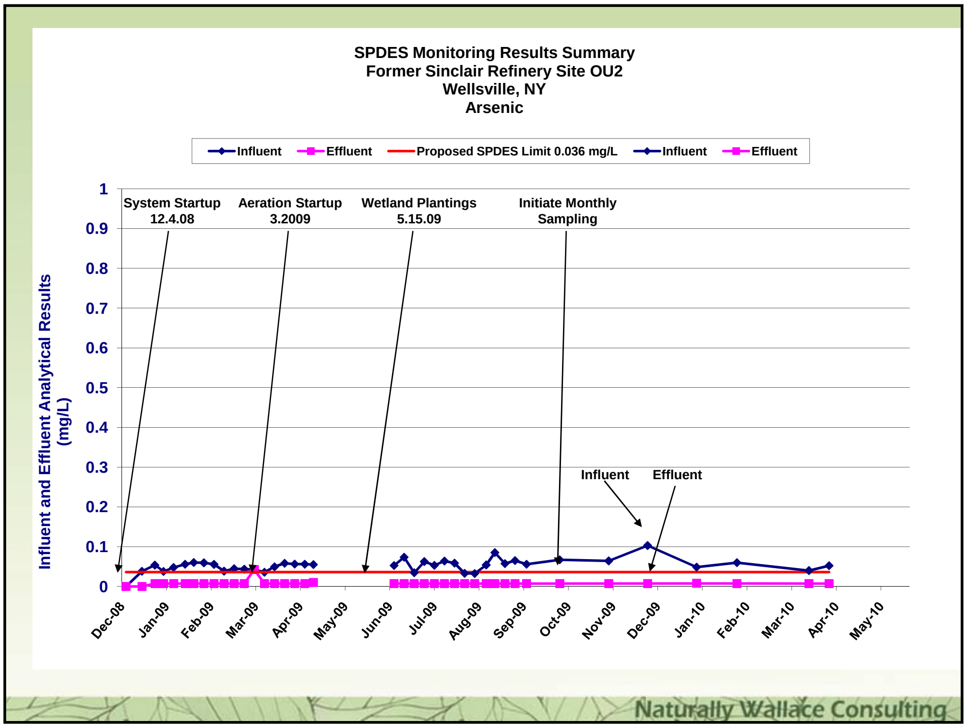
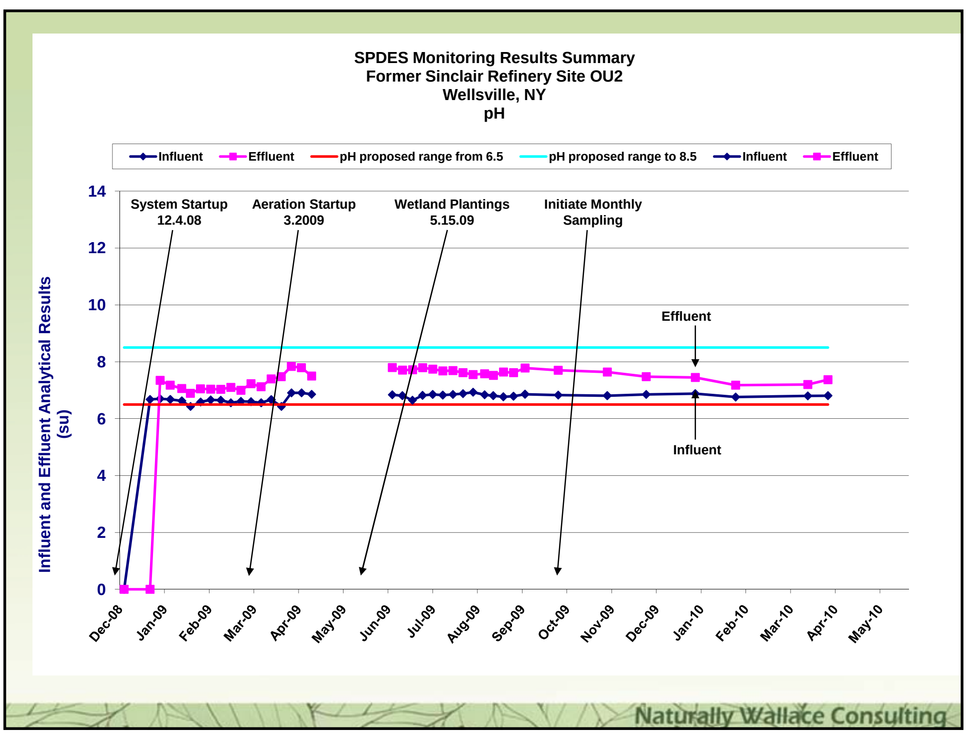
May-10
Naturally Wallace Consulting


SPDES Monitoring Results Summary
Former Sinclair Refinery Site OU2
Wellsville, NY
Arsenic
Influent
Effluent
Proposed SPDES Limit 0.036 mg/L
Influent
Effluent
Influent and Effluent Analytical Results (mg/L)
System Startup 12.4.08
Aeration Startup 3.2009
Wetland Plantings 5.15.09
Initiate Monthly Sampling
Influent
Effluent
1
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
Dec-08
Jan-09
Feb-09
Mar-09
Apr-09
May-09
Jun-09
Jul-09
Aug-09
Sep-09
Oct-09
Nov-09
Dec-09
Jan-10
Feb-10
Mar-10
Apr-10
May-10
Naturally Wallace Consulting


SPDES Monitoring Results Summary
Former Sinclair Refinery Site OU2
Wellsville, NY
pH
Influent
Effluent
pH proposed range from 6.5
pH proposed range to 8.5
Influent
Effluent
Influent and Effluent Analytical Results (su)
System Startup 12.4.08
Aeration Startup 3.2009
Wetland Plantings 5.15.09
Initiate Monthly Sampling
Effluent
Influent
14
12
10
8
6
4
2
0
Dec-08
Jan-09
Feb-09
Mar-09
Apr-09
May-09
Jun-09
Jul-09
Aug-09
Sep-09
Oct-09
Nov-09
Dec-09
Jan-10
Feb-10
Mar-10
Apr-10
May-10
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Conclusions

- Wellsville system has proven very effective at removing:
  - Volatile organic compounds (BTEX)
  - Semi-volatile organic cmpounds (aniline, nitrobenzene, and 2-methylnapthalene)
  - Metals (iron, arsenic) (manganese?)
  - pH adjustment to circumnetural
- Occupies about 10% of the former refinery site area
- Very long term, low O&M solution for site remediation

Thank You!

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Presented at the 10th International Conference on Wetland Systems for Water Pollution Control
23-29 September 2006 (Lisbon, Portugal)

HIGH-RATE AMMONIA REMOVAL IN AERATED ENGINEERED WETLANDS

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ABSTRACT

This paper summarizes results from two studies conducted at an engineered wetland pilot facility at Alfred College (Ontario, Canada). The pilot facility consists of an aerated, vertical downward saturated subsurface flow wetland (1.08 m^2 surface area, 0.83 m bed depth, 1.3 cm gravel media) with supporting feed tanks and equipment. The system can be heated or cooled to control operating temperatures. The first study involved nitrification of domestic wastewater for the Township of North Glengarry, Ontario. The system was operated at a hydraulic loading rate of 33 cm/d and nominal hydraulic retention time of 1.2 days. The observed volumetric 2 TIS rate constant averaged 10.0 day^{-1} at 25°C and 8.4 day^{-1} at 6°C. The calculated θ factor was 1.02, which is comparable to literature values (1.04). The second study involved nitrification of mine process water from the Rosebel Gold Mine (Suriname, South America). The system was operated at 25°C, with a hydraulic loading rate of 12.9 cm/d and a nominal hydraulic retention time of 6.5 days, both with and without aeration. With aeration, the observed volumetric 2 TIS rate constant was averaged 5.7 day^{-1}; without aeration, the rate constant dropped to 0.52 day^{-1}. The results of these two studies indicate aerated wetland systems with a low energy input (approximately 10% of that required by an activated sludge process) can be used for ammonia removal, even at cold water temperatures.

KEYWORDS

Aeration, ammonia removal, cold climate, nitrification, vertical subsurface flow

INTRODUCTION

The standard design approach to constructed wetlands is to accept the wetland as a passive system that is constrained by internal mechanisms. As a result, large wetland areas are typically needed to address design challenges such as high mass loadings or low operating temperatures. In contrast, engineered wetlands are wetland reactors that are designed to allow some degree of process control over the system to improve treatment efficiency.

Engineered wetlands include aerated systems using direct aeration of the wetland bed (Wallace, 2001) or fill-and-drain strategies (Behrends *et al*., 1996; Sun *et al*., 1999). Similarly, wetlands that are designed with a reactive bed media that influences water chemistry can be considered engineered systems. Examples include medias that are designed to supply organic carbon (Kassenga *et al*., 2003), absorb phosphorus (Johansson, 1997; Drizo *et al*., 1997; Arias and Brix, 2004), or provide a specified cation exchange capacity (Johns *et al*., 1998; Gisvold *et al*., 2000).

This work focuses on the relative effects of temperature and oxygen availability in the removal of ammonia nitrogen. It can be demonstrated that relatively small energy inputs (in this case, injection of air from an air pump) can yield large increases in ammonia treatment efficiency. Data from two different pilot studies (Township of North Glengarry and Rosebel Gold Mine) are compared and contrasted.

METHODS

The pilot system at Alfred College consists of an aerated, vertical downward saturated subsurface flow wetland (1.08 m^2 surface area, 0.83 m bed depth, 1.3 cm gravel media) with supporting feed tanks and equipment. The system can be heated or refrigerated to control operating temperatures. The pilot facility is shown in Figure 1. Aeration is provided by a small compressor capable of delivering approximately 2 x 10^{-3} m^3/s per cubic meter of wetland bed. Testing of the aeration system is shown in Figure 2.

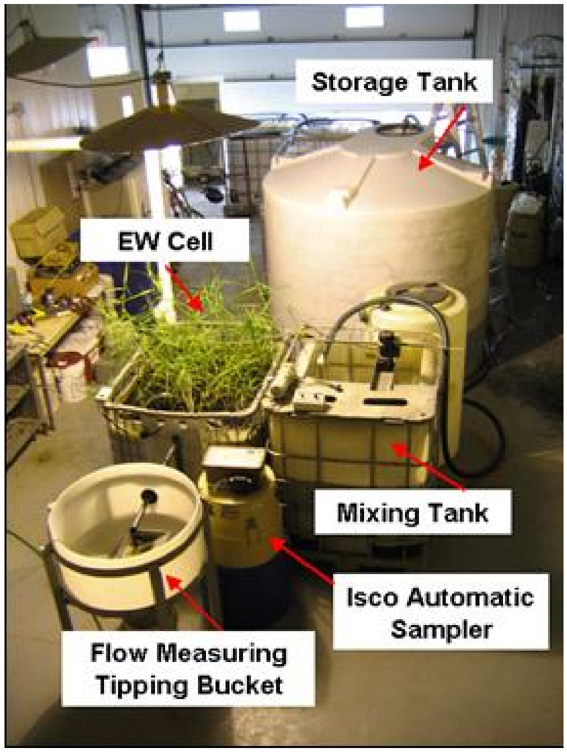

Figure 1. Engineered Wetland Pilot Facility at Alfred College, Canada.

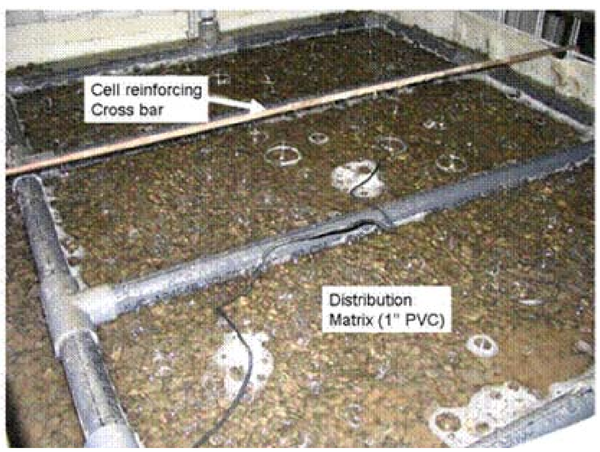

Figure 2. Testing of the Aeration System (note bubbles at water surface).

For each pilot study, ammonia grab samples were collected 5 times per week and analyzed using the colorimetric Phenate Method as outlined in Standard Methods for the Examination of Water and Wastewater (APHA, 1998).

Rosebel Gold Mine

For the Rosebel Gold Mine project, the pilot facility was operated to assess the ability of an engineered wetland to nitrify process water from a gold mining operation in Suriname, South America. For most phases of the pilot, a synthetic feed stock was used, but for one phase of operation (Run C) the pilot was operated with actual wastewater shipped from the job site. The pilot was operated with and without aeration. Operational phases are summarized in Table 1:

Table 1: Operational phases during the Rosebel Gold Mine Study

Run	A	B	C	D
Dates (2005)	15 Feb – 11 Mar	11 Mar – 18 Mar	23 Mar – 29 Mar	29 Mar – 6 Apr
Hydraulic Loading, cm/d	14.7	14.7	12.0	12.0
Feedstock	Synthetic	Synthetic	Actual	Synthetic
Aeration	On	Off	On	On
Temperature, °C	24.0	24.9	24.3	25.3

Township of North Glengarry

For the North Glengarry project, the pilot facility was operated to assess the ability of an engineered wetland to nitrify domestic wastewater from a municipality in Eastern Ontario. The pilot was operated in an aerated mode at two different temperatures using domestic wastewater from the town of North Glengarry. Operational phases are summarized in Table 2:

Table 2: Operational phases during the North Glengarry Study

Run	High Temperature	Low Temperature
Dates (2005)	13 May – 30 Jun	1 July – 16 Sep
Hydraulic Loading, cm/d	33.5	32.8
Feedstock	Actual	Actual
Aeration	On	On
Temperature, °C	25.2	6.0

RESULTS

During the Rosebel Gold Mine study, the wetland was tracer tested using sodium bromide and was observed to operate as one completely stirred tank reactor (CSTR) (as reported in Higgins *et al*., 2006). However, since this degree of internal mixing would likely not occur in a full-scale wetland system, results presented here are based on a two tank-in-series (2 TIS) model.

Township of North Glengarry

Based on observed ammonia nitrogen removal rates, rate constants for a 2 TIS hydraulic model were calculated as summarized in Table 3.

Table 3. Ammonia nitrogen removal rate constants for the North Glengarry pilot study

Run	A	B
Temperature, °C	25.2	6.0
k_t, 2 TIS (d^{-1})	10.0	8.4

Based on this data, the Arrhenius factor (θ) was calculated to be 1.02. This is in close agreement with the θ factor of 1.04 reported elsewhere in the literature (Kadlec and Knight, 1996). The 2 TIS ammonia nitrogen removal rate constant at 20°C (k_{20}) is 9.0 d^{-1}.

Rosebel Gold Mine

Rate constants for observed ammonia nitrogen removal were calculated for each of the four runs. Rate constants were temperature corrected to 20°C using a θ factor of 1.02 as calculated from the North Glengarry data set. Results are summarized in Table 4.

Table 4. Ammonia removal rate constants for the Rosebel Gold Mine pilot study

Run	Feedstock	Aeration	k_{20}, 2 TIS (d^{-1})
A	Synthetic	On	4.56
B	Synthetic	Off	0.52
C	Actual	On	5.54
D	Synthetic	On	7.02

On runs where aeration was employed (Runs A, C, D) the 2 TIS ammonia nitrogen removal rate constant averaged 5.7 d^{-1}. In contrast, without aeration (Run B), the rate constant dropped to 0.52 d^{-1}.

DISCUSSION

Data from the Rosebel Gold Mine study clearly demonstrates the effect of aeration on ammonia nitrogen removal. Without aeration, the 2 TIS rate constant observed in the pilot (0.52 d^{-1}) is similar to standard subsurface flow wetlands. By comparison, a standard area-based PFR rate constant of 34 m/yr (Kadlec and Knight, 1996) yields an equivalent 2 TIS volumetric constant of 0.36 d^{-1} for the pilot system, which is comparable to the non-aerated rate observed in Run B.

When aeration was employed, ammonia nitrogen removal rates were approximately 10-fold greater (5.7 d^{-1}) than without aeration (0.52 d^{-1}). This increase in ammonia nitrogen removal was observed both before (Run A) and after (Runs C and D) the non-aerated run. Enhanced ammonia nitrogen removal as a function of aeration has been previously presented for landfill leachate and manure runoff (Kinsley *et al*., 2002).

Low water temperatures (6°C) of the North Glengarry aerated wetland pilot did not impede treatment performance. Data from the North Glengarry pilot indicate that ammonia nitrogen removal rate constants are still considerably higher than in non-aerated wetlands, even at cold water temperatures. This information is consistent with a growing body of knowledge that cold-climate ammonia nitrogen removal is sustainable in aerated subsurface flow wetland systems (Nivala, 2005).

Since aerated wetlands have substantially higher ammonia nitrogen removal rates, reactor sizes are considerably smaller than for non-aerated wetlands. In the case of North Glengarry, a standard non-aerated subsurface flow wetland would be approximately 30.4 hectares; the engineered wetland alternative is only 1.4 hectares in size.

More efficient treatment through addition of an external energy input (in this case the injection of compressed air), can in some cases be justified due to smaller wetland areas and lower capital costs. While not entirely passive systems, these wetlands use considerably less energy than standard mechanical treatment processes. The full-scale North Glengarry wetlands have an external energy input of only 0.16 kWh/m^3. This energy input is considerably less than activated sludge processes, (2.39 – 0.51 kWh/m^3), and 'Living Machine' systems based on activated sludge principles (39 – 1.51 kWh/m^3) (Brix, 1999).

CONCLUSIONS

Data presented in this study indicate that ammonia nitrogen removal rates in aerated subsurface flow wetland systems are approximately 10 times higher than in non-aerated wetland systems, and that ammonia nitrogen removal can be sustained during cold water temperatures. Aerated wetland systems are both cost effective and energy efficient when compared to other wastewater treatment technologies.

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Water and Nutrient Management in Natural and Constructed Wetlands

Jan Vymazal
Editor

Springer

Jan Vymazal
Editor
Water and Nutrient Management in Natural and Constructed Wetlands

Wetlands provide a wide variety of ecosystem services within the landscape and their importance is commonly accepted. Among the most important are regulating services, i.e., benefits obtained from the regulation of ecosystem processes. For example, wetlands contribute to climate regulation. Land cover can affect local temperature and precipitation, wetland ecosystems may affect greenhouse gas sequestration and emissions, or affect the timing and magnitude of runoff and flooding, for example. Wetlands also improve water quality through mechanical, physical, physico-chemical, biological and biochemical processes. These abilities are also used in constructed wetlands but within a more controlled environment. In addition, wetlands provide the supporting services necessary for the production of all other ecosystem services such as soil formation and retention, nutrient cycling, primary production or water cycling. In short, wetlands are clearly among the most valuable ecosystems on Earth.

In order to provide these services, wetlands need to be properly evaluated, protected and maintained. This book provides results of the latest research in wetland science around the world. Chapters deal with such topics as the use of constructed wetlands for treatment of various types of wastewater, use of constructed wetlands in agroforestry, wetland hydrology and evapotranspiration, the effect of wetlands on landscape temperature, and chemical properties of wetland soils.

This book will be of interest for classes in environmental science, researchers, ecologists, landscape planners and regulators.

in Natural and Constructed Wetlands

Environment

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Chapter 10
Treatment of Landfill Leachate in Aerated Subsurface Flow Wetlands: Two Case Studies

Jaime Nivala and Scott Wallace

Abstract Treatment of landfill leachate is challenging due to high concentrations of oxygen-demanding compounds, such as biochemical oxygen demand (BOD) and ammonia nitrogen. The limited oxygen-transfer capability of conventional subsurface flow treatment wetlands has lead to the development of alternative design configurations that improve subsurface oxygen availability. This chapter compares the treatment performance of two aerated subsurface flow constructed wetlands treating landfill leachate against other, non-aerated systems. Results from these pilot studies indicate that aerated subsurface flow treatment wetlands are a viable technology selection for removal of ammonia-nitrogen of landfill leachate.

Keywords Aeration · Ammonia · Landfill leachate · *P-k-C** model · Subsurface flow wetland

10.1 Introduction

Little thought was originally given to the long-term stability and practical implications of permanently storing solid waste in the ground. Original landfills were unlined, and only some addressed the handling and treatment of the resulting wastewater generated at the site (herein referred to simply as *leachate*). Leachate is produced when rainfall and/or groundwater combines with the waste; when left unmanaged, it can create an imminent threat to nearby groundwater and surface waters. This is no small problem, considering that over 14000 landfills were closed in the United States between 1978 and 1988 (Mulamoottil, McBean, & Rovers, 1998). The number of active landfills in the United States is steadily decreasing; 8000 active sites were reported in 1988, but only 1858 landfills remained open in

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2001 (U.S. EPA, 2003). Landfills that have been closed (which by rough estimate is over 20000 in the United States alone) still require leachate collection and management. Leachate is notorious for containing high concentrations of organic matter and ammonia nitrogen (Table 10.1).

Since many of these sites are located in rural areas, connecting the landfill to the regional sewer is often economically and/or practically infeasible. Pumping and hauling leachate to the closest treatment facility is another option, but with no incoming revenue for closed facilities, who will pay the indefinitely recurring cost of transport and treatment? This is one situation in which onsite wastewater management becomes a favorable alternative. Subsurface-flow (SSF) constructed wetlands have been identified as a promising technologies for the treatment of landfill leachate (Kadlec & Knight, 1996). Treatment wetlands have a small ecological footprint, require a low level of operations and maintenance compared to some conventional wastewater treatment technologies, and have an aesthetic value similar to that of natural wetlands. The application of wetland technology for treating landfill leachate is still developing. There has been a call by academics and professionals alike for a better understanding of the movement, transformation, and removal of contaminants in these treatment systems through extensive and long-term studies (Mulamoottil et al., 1998).

In conventional (passive) subsurface flow treatment wetlands, the oxygen required for nitification and organic matter removal generally exceeds the amount of available oxygen in the system. As a result, passive treatment wetlands systems can discharge high levels of organic matter, and nitrogen in the form of ammonia. The limited oxygen-transfer capability of conventional subsurface flow treatment wetlands has lead to the development of alternative design configurations that improve subsurface oxygen availability, through means of frequent water level fluctuation (Austin, 2006; Behrends, 1999; Zoeller & Byers, 1999), shallow bed depth (García, Morató, Bayona, & Aguirre, 2004), or direct mechanical aeration of the gravel bed (Flowers, 2002; Ouellet-Plamodon, Chazarenc, Comeau, & Brisson, 2006; Wallace, 2001).

Subsurface flow wetlands equipped with mechanical aeration (Fig. 10.1) have been widely used in North America to increase oxygen transfer rates and sustain aerobic conditions in the substratum. This technology variant has been utilized to treat many types of wastewater, including domestic sewage, dairy waste, groundwater contaminated with petroleum hydrocarbons, and landfill leachate (Lockhart, 1999; Nivala, 2005; Kadlec & Wallace, 2005; Muñoz, Drizo, & Hession, 2006).

Treatment performance in constructed wetlands is a combined result of the internal mechanisms that store, transform, and remove organic matter and pollutants. Due to the limited amount of information available, system performance cannot be predicted with a high degree of accuracy. By necessity, current design tools use "lumped" parameters (such as first-order rate coefficients) to represent the aggregate impact of the internal mechanisms in wetland treatment systems. Because the combined effect of internal treatment processes varies from one system to the next, pilot studies that use site-specific wastewater (in this case, landfill leachate) can be used to substantiate parameters for full-scale designs. This paper summarizes results from two aerated subsurface-flow wetland pilot systems treating landfill leachate.

Table 10.1 Leachate characteristics from Landfills in North America

Landfill	BOD (mg L^{-1})	COD (mg L^{-1})	TSS (mg L^{-1})	NH_4–N (mg L^{-1})	NO_3–N (mg L^{-1})	Iron (mg L^{-1})	Reference
Fulton County, Indiana	390	1540	7.840	284	3	178	Kadlec (1999)
Sarnia, Ontario	407	1036	–	254	< 0.3	17.6	Kadlec (1999)
City Sand, Michigan	312	3203	241	2074	0	–	Kadlec (1999)
Saginaw, Michigan	729	–	–	322	0	22	Kadlec (1999)
Maidstone, Ontario	33	–	–	92.5	2.3	–	Kinsley, Crolla, and Higgins (2002)
Jones County, Iowa	116	781	186	212	2	21	This study
Beecher, Illinois	40	–	27	302	0.3	8.1	This study

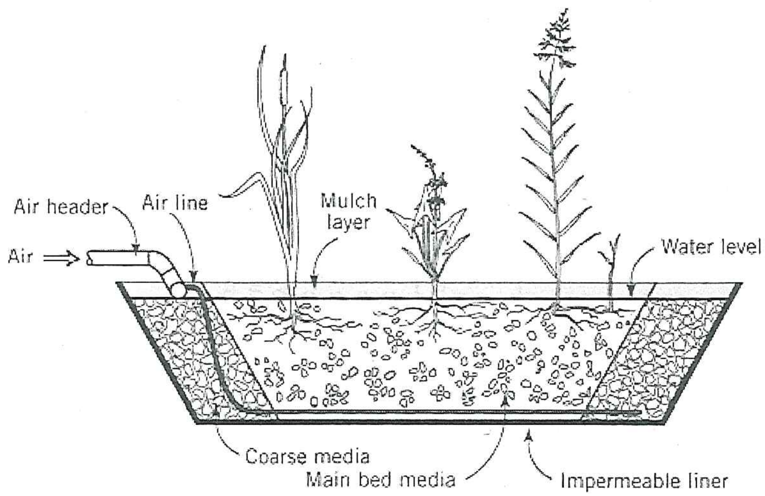

Fig. 10.1 Schematic of an aerated, horizontal subsurface-flow treatment wetland. Reprinted with permission from Kadlec & Wallace (2009)

10.2 Study Sites

10.2.1 Jones County Landfill

The Jones County wetland was installed at the local municipal landfill in 1999 to demonstrate the use of the technology for treatment of landfill leachate at small, rural landfills. The treatment system consists of one 15.5 m long by 6 m wide horizontal flow cell, lined with an impermeable liner. The design flow is 0.4 m^3d^{-1}. The cell is equipped with a patented aeration system (Wallace, 2001) and consists of a 30 cm layer of pea gravel ($d_{10} = 5$ mm) underneath a 15 cm layer of well-decomposed peat. The purpose of the peat layer is to protect the system against freezing in the wintertime. The system received high loads of iron between 1999 and 2002, which clogged the aeration system. The aeration system in the wetland was replaced in September 2002, and a pretreatment step was installed before the wetland cell, which included an aeration chamber and settling tank for iron removal. Additional site details are provided in Nivala, Hoos, Cross, Wallace, and Parkin (2007).

10.2.2 Beecher Landfill

The Beecher Landfill pilot-scale wetland system was constructed in 2008 as a part of an engineering feasibility study to validate a full-scale design. The treatment

components were scaled down by a factor of 2000, which resulted in a design flow of 9.5 L d^{-1} for the pilot system. Based on experience with iron-related clogging at the Jones County pilot (Nivala et al., 2007), the Beecher system included pre-treatment for iron oxidation. Pretreatment was achieved using a 5-L aeration chamber followed by a 15-L sedimentation tank. The design was based on historical leachate data; design concentration for CBOD was 100 mg L^{-1} (treatment target: 15 mg L^{-1}) and ammonia-nitrogen design concentration was 300 mg L^{-1} (treatment target: 0.2 mg L^{-1}). The aeration system was sized to supply enough oxygen to satisfy the sum of the carbonaceous biochemical oxygen demand (CBOD) and nitrogenous oxygen demand (NOD). A two-cell, vertical-flow design was chosen to improve the hydraulic efficiency of the system. Each cell was approximately 55 L in volume and filled with river gravel. Additional components of the pilot system included a limestone cell and peat filter, but those results are not presented here. The pilot system was fed a mixture of water and leachate during start-up of the system. Following the acclimatization period, the pilot system received full-strength leachate. The leachate used in the pilot study was shipped directly from the Beecher Landfill every two weeks.

10.3 Methods

The first-order, tanks-in-series *P-k-C** model set forth by Kadlec & Wallace (2009) can be used to assess and compare performance amongst different wetland treatment systems. The *P-k-C** model is a compromise between accuracy and computational simplicity and can be represented by Eq. (10.1):

$$\frac{C_o - C^*}{C_i - C^*} = \frac{1}{(1 + k/Pq)^P} \tag{10.1}$$

where:

C_o = outlet concentration, mg L^{-1}
C_i = inlet concentration, mg L^{-1}
C^* = background concentration, mg L^{-1}
k = first-order areal rate coefficient, m yr^{-1}
P = apparent number of tanks-in-series
q = hydraulic loading rate, m yr^{-1}

The parameter P is a fitted parameter that accounts for both the hydraulic efficiency of the reactor (number of tanks-in-series, N) and weathering of a pollutant as it undergoes treatment in the wetland (Kadlec, 2003). Thus, P will be always less than or equal to N. The temperature dependence of the rate coefficient k is described by the modified Arrhenius equation (Eq. (10.2)):

$$k_T = k_{20}\,\theta^{(T-20)} \tag{10.2}$$

where:

k_T = first-order areal rate coefficient at temperature T, m yr^{-1}
k_{20} = first-order areal rate coefficient at temperature 20°C, m yr^{-1}
θ = temperature factor, dimensionless
T = operational water temperature, degrees Celsius

The first-order tanks-in-series P-k-C^* model offers a platform for comparing data from different treatment systems. Results for k-rates and θ-factors for both pilot systems were calculated using Solver™ in Microsoft excel. Background ammonia concentrations (C^*) for the Jones County and Beecher treatment systems were selected to be zero, and the P-value was chosen to be six, in order to allow direct comparison to the values reported in Kadlec & Wallace (2009).

10.4 Results and Discussion

10.4.1 Jones County Landfill

In 2002, after replacement of the clogged aeration system, the microbial community in the Jones County wetland took approximately 6 months to adapt to the ammonia loading. Figure 10.2 shows the ammonia and nitrate concentrations during the recommissioning period. Effluent ammonia concentrations decreased throughout the winter months despite water temperatures as cold as 2°C. It is interesting to note the corresponding increase in nitrate during this time, and the following decrease in nitrate as the denitrifying bacteria became established in 2003.

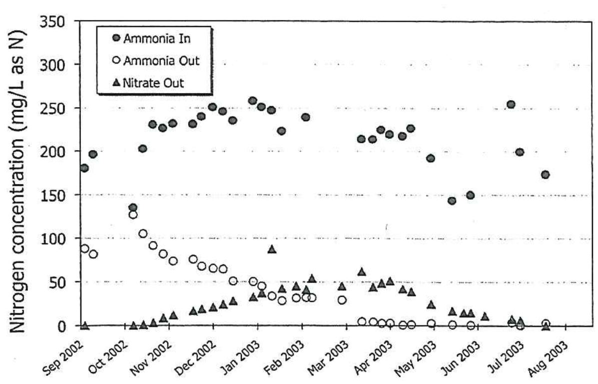

Fig. 10.2 Ammonia removal in the Jones County HSSF wetland

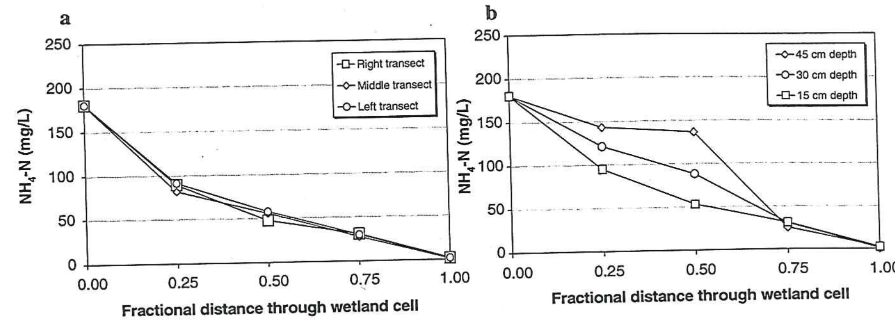

Fig. 10.3 Lateral (a) and vertical (b) ammonia profiles in the Jones County HSSF wetland

The Jones County system was also monitored intensively from August 2004 to May 2005. Internal sampling points located at 25, 50, and 75% through the wetland cell indicated that the wetland was well-mixed in the direction orthogonal to flow (Fig. 10.3a). However, a set of depth-specific sampling points along the centerline of the cell (15, 30, and 45 cm deep) indicated that there was a clear pattern of vertical stratification in the first half of the wetland cell (Fig. 10.3b). These observations were further validated by tracer study results that displayed similar spatial patterns (Nivala, 2005). Ammonia removal efficiency for the wetland system (on a concentration basis) was 98% during this monitoring campaign ($n = 25$).

10.4.2 Beecher Landfill

Analytical data was collected from February until April 2008 (Table 10.2). Samples were collected and analyzed weekly; the average ammonia-nitrogen concentration

Table 10.2 Selected results from the Beecher pilot study

	CBOD		NH_4-N		Iron		pH	
	C_{in} (mg L^{-1})	C_{out} (mg L^{-1})	C_{in} (mg L^{-1})	C_{out} (mg L^{-1})	C_{in} (mg L^{-1})	C_{out} (mg L^{-1})	SU	SU
Week 1	12.8	6.0	175	0.6	5.4	0.1	8.4	8.4
Week 2	9.5	3.4	224	0.1	3.7	0.1	8.6	8.4
Week 3	8.5	2.0	248	0.2	3.6	0.1	8.4	8.4
Week 4	8.4	3.3	201	0.3	3.8	0.2	8.4	8.4
Week 5	7.8	3.1	162	0.3	3.1	0.3	8.7	8.8
Week 6	13.6	2.4	210	0.4	4.1	0.1	8.8	8.9
Week 7	8.9	2.2	249	0.3	4.2	0.2	8.8	8.9
Week 8	7.0	2.0	262	0.5	4.5	0.1	8.8	9.0
Week 9	8.6	3.8	231.0	0.4	3.2	0.2	8.7	9.0
Week 10	9.3	2.9	213.0	0.3	3.3	0.4	8.6	8.9
Week 11	24.0	11.7	160.0	0.3	2.5	0.2	8.5	8.8
Week 12	13.3	2.0	120.0	0.3	2.8	0.1	8.8	9.1
Average	11.0	3.7	204.6	0.3	3.7	0.2	8.6	8.7

Table 10.3 Aerated full-scale treatment wetland *P-k-C** results

Site Name and Location	Flow Direction	Wastewater Type	k_{20} (m/yr)	θ	Reference
Jones County Landfill, Iowa	Horizontal	Landfill Leachate	10.7	0.96	This study
Prinsburg, Minnesota	Horizontal	Municipal Wastewater	9.2	1.08	Wallace and Nivala (2008)
St. Croix Chippewa, Wisconsin	Horizontal	Municipal Wastewater	13.3	1.04	Wallace and Nivala (2008)
			11.4	1.01	Kadlec and Wallace (2009) 50th Percentile

Table 10.4 Aerated pilot-scale treatment wetland *P-k-C** results

Site name and location	Flow direction	Wastewater type	k_{20} (m yr^{-1})	θ	Reference
Beecher Landfill, Illinois	Vertical	Landfill Leachate	137	1.03	This study
North Glengarry, Ontario	Vertical	Municipal Wastewater	818	1.02	Wallace et al. (2006)
Rosebel; Suriname	Vertical	Gold Mine	518	1.02	Wallace et al. (2006)

entering the wetland cells was 204.5 mg L^{-1}; effluent concentration averaged 0.3 mg L^{-1}. Average ammonia removal efficiency (on a concentration basis) was 99.8%. Selected parameters from the Beecher pilot system are shown in Table 10.2 for the duration of the study.

10.4.3 P-k-C Model Results*

The Jones County system, which was operated outdoors under a range of temperatures, displayed an overall k_{20} of 10.7 m yr^{-1}, which is near the 50th percentile for non-aerated HSSF wetlands. The θ-factor of less than 1.0, however, indicates that ammonia removal in the Jones County wetland system was less sensitive to temperature than what has been reported in the literature for non-aerated wetlands. Because the Jones County wetland was often removing ammonia to less than one milligram per liter in the effluent, the reported k-value in Table 10.3 is a low estimate of the system's actual ammonia removal rate coefficient.

Table 10.4 shows that the Beecher system, which was operated indoors and under a small range of temperatures, displayed an overall k_{20} of 137.0 m yr^{-1} (ten-fold higher compared to that of the Jones County system) and a θ-factor of 1.027. In laboratory studies, wetland aeration has been found to increase nitrification rates up to twenty-fold (Kinsley et al., 2002). The results from the laboratory-scale Beecher pilot are consistent with this finding.

10.5 Conclusions

Results from these pilot studies indicate that aerated subsurface flow treatment wetlands are a viable technology selection for effective removal of ammonia-nitrogen of landfill leachate. Furthermore, the Beecher Landfill pilot study indicates that saturated vertical flow wetlands equipped with mechanical aeration are extremely efficient in removal of ammonia-nitrogen. Future research is required to better characterize the rate coefficients in alternative treatment wetland design configurations.

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Wurtsmith Air Force Base, Oscoda, MI
LF-30/31 Expanded Pump and Treat System

Cascade Aeration Pilot Test Results

BB&E, LLC
February 2010

On 4 February 2010, the Cascade Aeration Pilot Test was conducted as per the attached Work Plan dated 15 January 2010 with a 24 inch diameter by 20ft long corrugated HDPE pipe (24" MTS, AASHTO 294-heavy duty). The pipe was tested at two different slopes of 4 to 1 and 3 to 1 (see attached photos). Water samples were collected and tested for Dissolved Oxygen (DO) levels, VOC concentrations, ferrous iron concentrations, and temperature at both the influent and effluent points of the wastewater stream for comparison. DO levels, temperature, and pH were sampled three times for each set-up while ferrous iron concentrations and VOC concentrations were sample two times for each set-up. Of the VOCs tested for, TCE was the primary contaminant of concern. The outside air temperature during testing ranged from 24 to 28°F. The outside air temperature represents the temperature of air entering the cascade pipe at the water effluent end.

The pipe was first constructed at a 4 to 1 slope, the flow rate averaged 51.8 gpm with a water depth of approximately 1 to 1.5 inches in the pipe. Three tests were conducted at this slope and the results are attached. The averages of the tested parameters are show below:

DO Levels (mg/L)		Temperature (°F)		pH		Ferrous Iron (mg/L)		TCE (ug/L)	
Influent	Effluent	Influent	Effluent	Influent	Effluent	Influent	Effluent	Influent	Effluent
2.94	6.32	46.20	45.98	7.64	7.84	0.09	0.15	42.30	30.15

The pipe was then constructed to a 3 to 1 slope, the flow rate averaged 49.4 gpm with a water depth of approximately 1 inch in the pipe. Three tests were conducted at this slope and the results are attached. The averages of the three tests and the tested parameters are show below:

DO Levels (mg/L)		Temperature (°F)		pH		Ferrous Iron (mg/L)		TCE (ug/L)	
Influent	Effluent	Influent	Effluent	Influent	Effluent	Influent	Effluent	Influent	Effluent
2.76	7.77	46.13	46.27	7.93	8.06	0.15	0.14	35.10	24.45

The increase in turbulence created from the corrugated pipe readily increased the DO levels significantly from the influent to effluent samples. The TCE concentrations decreased in both test conditions indicating relatively successful air stripping conditions. Overall, the results were a favorable "proof of concept" of the proposed use of a cascade aerator to pretreat the incoming groundwater stream as part of the engineered wetland system. The proposed design will include 4-24 inch dia. by 40ft long pipes of a similar construction placed on a 3 to 1 slope.

Wurtsmith Air Force Base, Oscoda, MI
LF-30/31 Expanded Pump and Treat System
Photos

Photo 1: Typical Pipe Flow, approx. 50 gpm, 24 inch HDPE corregated pipe

Photo 2: Pipe Slope 3 to 1

LF30/31 Cascade Aerator Test

Volatile Organic Compounds

			Influent #1	Effluent #1	Influent #2	Effluent #2	Influent #3	Effluent #3	Influent #4	Effluent #4
Lab:			2/4/2010	2/4/2010	2/4/2010	2/4/2010	2/4/2010	2/4/2010	2/4/2010	2/4/2010
Date:	MDEQ Part	MDEQ Part	1010	1015	1026	1029	1222	1224	1227	1229
Method:	201 Criteria	201 Criteria	8260	8260	8260	8260	8260	8260	8260	8260
Unit:	ug/L (DW)	ug/L (GSI)	ug/L	ug/L	ug/L	ug/L	ug/L	ug/L	ug/L	ug/L
1,1,1,2-Tetrachloroethane	77	ID (X)	<1.00	<1.00	<1.00	<1.00	<1.00	<1.00	<1.00	<1.00
1,1,1-TCA (Trichloroethane)	200 (A)	200	<1.00	<1.00	<1.00	<1.00	<1.00	<1.00	<1.00	<1.00
1,1,2,2-Tetrachloroethane	8.5	78 (X)	<1.00	<1.00	<1.00	<1.00	<1.00	<1.00	<1.00	<1.00
1,1,2-TCA (Trichloroethane)	5.0 (A)	330 (X)	<1.00	<1.00	<1.00	<1.00	<1.00	<1.00	<1.00	<1.00
1,1-DCA (Dichloroethane)	880	740	<1.00	<1.00	<1.00	<1.00	<1.00	<1.00	<1.00	<1.00
1,1-DCE (Dichloroethylene)	7.0 (A)	65 (X)	<1.00	<1.00	<1.00	<1.00	<1.00	<1.00	<1.00	<1.00
1,1-Dichloropropene	NL	NL	<1.00	<1.00	<1.00	<1.00	<1.00	<1.00	<1.00	<1.00
1,2,3-Trichlorobenzene	NL	NL	<1.00	<1.00	<1.00	<1.00	<1.00	<1.00	<1.00	<1.00
1,2,3-Trichloropropane	42	NA	<1.00	<1.00	<1.00	<1.00	<1.00	<1.00	<1.00	<1.00
1,2,4-Trichlorobenzene	70 (A)	30	<1.00	<1.00	<1.00	<1.00	<1.00	<1.00	<1.00	<1.00
1,2,4-Trimethylbenzene	63 (E)	17	<1.00	<1.00	<1.00	<1.00	<1.00	<1.00	<1.00	<1.00
1,2-DCA (Dichloroethane)	5.0 (A)	360 (X)	<0.500	<0.500	<0.500	<0.500	<0.500	<0.500	<0.500	<0.500
1,2-DCB (Dichlorobenzene)	600 (A)	16	<1.00	<1.00	<1.00	<1.00	<1.00	<1.00	<1.00	<1.00
1,2-Dibromo-3-Chloropropane	NL	NL	<2.00	<2.00	<2.00	<2.00	<2.00	<2.00	<2.00	<2.00
1,2-Dichloropropane	5.0 (A)	290 (X)	<1.00	<1.00	<1.00	<1.00	<1.00	<1.00	<1.00	<1.00
1,2-EDB (Dibromoethane)	NL	NL	<1.00	<1.00	<1.00	<1.00	<1.00	<1.00	<1.00	<1.00
1,3,5-Trimethylbenzene	72 (E)	45	<1.00	<1.00	<1.00	<1.00	<1.00	<1.00	<1.00	<1.00
1,3-DCB (Dichlorobenzene)	6.6	38	<1.00	<1.00	<1.00	<1.00	<1.00	<1.00	<1.00	<1.00
1,3-Dichloropropane	NL	NL	<1.00	<1.00	<1.00	<1.00	<1.00	<1.00	<1.00	<1.00
1,4-DCB (Dichlorobenzene)	75 (A)	13	<1.00	<1.00	<1.00	<1.00	<1.00	<1.00	<1.00	<1.00
1-Chlorohexane	NL	NL	<1.00	<1.00	<1.00	<1.00	<1.00	<1.00	<1.00	<1.00
2,2-Dichloropropane	NL	NL	<1.00	<1.00	<1.00	<1.00	<1.00	<1.00	<1.00	<1.00
2-Chloroethyl Vinyl Ether	ID	NA								
2-Chlorotoluene	NL	NL	<1.00	<1.00	<1.00	<1.00	<1.00	<1.00	<1.00	<1.00
2-Hexanone (MBK)	1,000	NA								
4-Chlorotoluene	NL	NL	<1.00	<1.00	<1.00	<1.00 Q	<1.00 Q	<1.00 Q	<1.00 Q	<1.00 Q
4-Methyl-2-pentanone (MIBK)	1,800	ID								
Acetone	730	1,700	<25.0	<25.0	<25.0	<25.0 Q	<25.0 Q	<25.0 Q	<25.0 Q	<25.0 Q
Acrolein	120	NA								
Acrylonitrile	2.6	4.9 (X)								
Benzene	5.0 (A)	200 (X)	<1.00	<1.00	<1.00	<1.00	<1.00	<1.00	<1.00	<1.00
Bromobenzene	18	NA	<1.00	<1.00	<1.00	<1.00	<1.00	<1.00	<1.00	<1.00
Bromochloromethane	NL	NL	<1.00	<1.00	<1.00	<1.00	<1.00	<1.00	<1.00	<1.00
Bromodichloromethane	80 (A,W)	ID	<1.00	<1.00	<1.00	<1.00	<1.00	<1.00	<1.00	<1.00
Bromoform	80 (A,W)	ID	<1.00	<1.00	<1.00	<1.00	<1.00	<1.00	<1.00	<1.00
Bromomethane	10	35	<1.00	<1.00	<1.00	<1.00	<1.00	<1.00	<1.00	<1.00
Carbon Disulfide	800	ID								
Carbon Tetrachloride	5.0 (A)	45 (X)	<1.00	<1.00	<1.00	<1.00	<1.00	<1.00	<1.00	<1.00
Chlorobenzene	100 (A)	47	<1.00	<1.00	<1.00	<1.00	<1.00	<1.00	<1.00	<1.00
Chloroethane	430	ID	<1.00	<1.00	<1.00	<1.00	<1.00	<1.00	<1.00	<1.00
Chloroform	80 (A,W)	170 (X)	<0.300	<0.300	<0.300	<0.300	<0.300	<0.300	<0.300	<0.300
Chloromethane	260	ID	<1.00	<1.00	<1.00	<1.00	<1.00	<1.00	<1.00	<1.00
cis-1,2- DCE (Dichloroethene)	70 (A)	620	**22.0**	**18.0**	**21.3**	**17.6**	**22.0**	**15.8**	**21.9**	**15.9**
cis-1,3-Dichloropropene	NL	NL	<1.00	<1.00	<1.00	<1.00	<1.00	<1.00	<1.00	<1.00
Dibromochloromethane	80 (A,W)	ID	<1.00	<1.00	<1.00	<1.00	<1.00	<1.00	<1.00	<1.00
Dibromomethane	80	NA	<5.00	<5.00	<5.00	<5.00	<5.00	<5.00	<5.00	<5.00
Dichlorodifluoromethane	1,700	ID	<1.00	<1.00	<1.00	<1.00 Q	<1.00 Q	<1.00 Q	<1.00 Q	<1.00 Q
Ethylbenzene	74 (E)	18	<1.00	<1.00	<1.00	<1.00	<1.00	<1.00	<1.00	<1.00
Hexachlorobutadiene	15	0.05	<5.00	<5.00	<5.00	<5.00	<5.00	<5.00	<5.00	<5.00
Isopropyl Benzene	800	ID	<5.00	<5.00	<5.00	<5.00	<5.00	<5.00	<5.00	<5.00
m & p Xylene	NL	NL	<2.00	<2.00	<2.00	<2.00	<2.00	<2.00	<2.00	<2.00
MEK (2-Butanone)	13,000	2,200	<25.0	<25.0	<25.0	<25.0	<25.0	<25.0	<25.0	<25.0
Methylene Chloride	5.0 (A)	940 (X)	<5.00	<5.00	<5.00	<5.00	<5.00	<5.00	<5.00	<5.00
Methyl-tert-butyl Ether (MTBE)	40 (E)	730 (X)	<5.00	<5.00	<5.00	<5.00	<5.00	<5.00	<5.00	<5.00
MIBK (methyl isobutyl ketone)	NL	NL	<50.0	<50.0	<50.0	<50.0	<50.0	<50.0	<50.0	<50.0
Naphthalene	520	13	<5.00	<5.00	<5.00	<5.00	<5.00	<5.00	<5.00	<5.00
n-Butylbenzene	80	ID	<1.00	<1.00	<1.00	<1.00	<1.00	<1.00	<1.00	<1.00
n-Propyl Benzene	80	ID	<1.00	<1.00	<1.00	<1.00	<1.00	<1.00	<1.00	<1.00
o-Xylene	NL	NL	<1.00	<1.00	<1.00	<1.00	<1.00	<1.00	<1.00	<1.00
p-Isopropyltoluene	NL	NL	<1.00	<1.00	<1.00	<1.00	<1.00	<1.00	<1.00	<1.00
sec-Butylbenzene	80	ID	<1.00	<1.00	<1.00	<1.00	<1.00	<1.00	<1.00	<1.00
Styrene	100 (A)	80	<1.00	<1.00	<1.00	<1.00	<1.00	<1.00	<1.00	<1.00
TCE (Trichloroethylene)	5.0 (A)	200 (X)	43.9	32.6	40.7	27.7	36.9	24.9	33.3	24.0
Tert-Butylbenzene	80	ID	<1.00	<1.00	<1.00	<1.00	<1.00	<1.00	<1.00	<1.00
Tetrachloroethylene (PCE)	5.0 (A)	45 (X)	<1.00	<1.00	<1.00	<1.00	<1.00	<1.00	<1.00	<1.00
Toluene	790(E)	140	<1.00	<1.00	<1.00	<1.00	<1.00	<1.00	<1.00	<1.00
trans-1,2-Dichloroethylene	100 (A)	1,500	0.385 F	<1.00	0.395 F	<1.00	0.398 F	0.265 F	0.361 F	0.255 F
trans-1,3-Dichloropropene	NL	NL	<1.00	<1.00	<1.00	<1.00	<1.00	<1.00	<1.00	<1.00
Trichlorofluoromethane	2,600	NA	<1.00	<1.00	<1.00	<1.00	<1.00	<1.00	<1.00	<1.00
Vinyl Acetate	640	NA								
Vinyl Chloride	2.0 (A)	15	<1.00	<1.00	<1.00	<1.00	<1.00	<1.00	<1.00	<1.00
Xylene-Total	280 (E)	35	<3.00	<3.00	<3.00	<3.00	<3.00	<3.00	<3.00	<3.00

Xylene-Total represents the addition of the laboratory-reported m&p-xylene plus o-xylene concentrations. Xylene-Total, itself, was not reported by the laboratory.
Samples #1 and #2 are from an Influent height of 5 ft tall.
Samples #3 and #4 are from an Influent height of 7 ft tall.
F = Found; the analyte was positively identified with concentration above MDL but below RL.
Q = One or more quality control criteria failed

LF30/31 Cascade Aeration System Pre-Design Study

Purge Well 2 is the Representative Groundwater for the Cascade Aeration Test

Test Number	Date	Time	Height	Length	Slope (Angle)	Flow Rate gpm	DO (HANNA 9142) mg/L		Temperature (Extech) degrees F		pH (Extech)		Ferrous Iron (DR 890) mg/L		Depth of Water in Pipe	Samplers
							Influent	Effluent	Influent	Effluent	Influent	Effluent	Influent	Effluent		
1	2/4/2010	0815	5.0 ft	20 ft	4:1	52.7	3.7		NA		7.15				~1-1.5 in (top of correlated part)	AG
2	2/4/2010	0819	5.0 ft	20 ft	4:1	53.2	4.0		NA		7.52				~1-1.5 in (top of correlated part)	AG
3	2/4/2010	0821	5.0 ft	20 ft	4:1	53.1	2.7		46.4		7.66				~1-1.5 in (top of correlated part)	AG
4	2/4/2010	0829	5.0 ft	20 ft	4:1	54.3		7.5		45.3		7.84			~1-1.5 in (top of correlated part)	AG/RH
5	2/4/2010	0832	5.0 ft	20 ft	4:1	53.8		6.3		46.0		7.79			~1-1.5 in (top of correlated part)	AG/RH
6	2/4/2010	0837	5.0 ft	20 ft	4:1	53.2		7.0		45.5		7.85			~1-1.5 in (top of correlated part)	AG/RH
7	2/4/2010	0849	5.0 ft	20 ft	4:1	55.6							0.07		~1-1.5 in (top of correlated part)	AG/RH
8	2/4/2010	0855	5.0 ft	20 ft	4:1	53.2							0.03		~1-1.5 in (top of correlated part)	AG/RH
9	2/4/2010	0900	5.0 ft	20 ft	4:1	50							0.02		~1-1.5 in (top of correlated part)	AG/RH
10	2/4/2010	0904	5.0 ft	20 ft	4:1	50.4								0.14	~1-1.5 in (top of correlated part)	AG/RH
11	2/4/2010	0909	5.0 ft	20 ft	4:1	51.1								0.13	~1-1.5 in (top of correlated part)	AG/RH
12	2/4/2010	0914	5.0 ft	20 ft	4:1	51.1								0.21	~1-1.5 in (top of correlated part)	AG/RH
13	2/4/2010	0919	5.0 ft	20 ft	4:1	51.1	3.6		46.2		7.70				~1-1.5 in (top of correlated part)	AG/RH
14	2/4/2010	0924	5.0 ft	20 ft	4:1	50.8	2.2		46.4		7.69				~1-1.5 in (top of correlated part)	AG/RH
15	2/4/2010	0928	5.0 ft	20 ft	4:1	50.4	1.6		46.4		7.68				~1-1.5 in (top of correlated part)	AG/RH
16	2/4/2010	0929	5.0 ft	20 ft	4:1	50.9		6.3		46.2		7.80			~1-1.5 in (top of correlated part)	AG/RH
17	2/4/2010	0933	5.0 ft	20 ft	4:1	51.3		5.6		46.2		7.80			~1-1.5 in (top of correlated part)	AG/RH
18	2/4/2010	0936	5.0 ft	20 ft	4:1	51.1		5.6		46.2		7.79			~1-1.5 in (top of correlated part)	AG/RH
19	2/4/2010	0943	5.0 ft	20 ft	4:1	52.2							0.14		~1-1.5 in (top of correlated part)	AG/RH
20	2/4/2010	0948	5.0 ft	20 ft	4:1	52.2							0.12		~1-1.5 in (top of correlated part)	AG/RH
21	2/4/2010	0952	5.0 ft	20 ft	4:1	52.2							0.13		~1-1.5 in (top of correlated part)	AG/RH
22	2/4/2010	0957	5.0 ft	20 ft	4:1	52.2								0.14	~1-1.5 in (top of correlated part)	AG/RH
23	2/4/2010	1001	5.0 ft	20 ft	4:1	52.3								0.13	~1-1.5 in (top of correlated part)	AG/RH
24	2/4/2010	1006	5.0 ft	20 ft	4:1	52.5								0.13	~1-1.5 in (top of correlated part)	AG/RH
25	2/4/2010	1037	5.0 ft	20 ft	4:1	50.4	4.1		46.0		7.76				~1-1.5 in (top of correlated part)	RH/LD
26	2/4/2010	1040	5.0 ft	20 ft	4:1	50.2	2.4		46.0		7.78				~1-1.5 in (top of correlated part)	RH/LD
27	2/4/2010	1044	5.0 ft	20 ft	4:1	50.4	2.2		46.0		7.80				~1-1.5 in (top of correlated part)	RH/LD
28	2/4/2010	1046	5.0 ft	20 ft	4:1	50.4		6.7		46.0		7.91			~1-1.5 in (top of correlated part)	RH/LD
29	2/4/2010	1049	5.0 ft	20 ft	4:1	50.4		6.1		46.2		7.90			~1-1.5 in (top of correlated part)	RH/AG
30	2/4/2010	1052	5.0 ft	20 ft	4:1	50.4		5.8		46.2		7.92			~1-1.5 in (top of correlated part)	RH/AG
				Average:		51.77	2.94	6.32	46.20	45.98	7.64	7.84	0.09	0.15		

LF30/31 Cascade Aeration System Pre-Design Study

Purge Well 2 is the Representative Groundwater for the Cascade Aeration Test

Test Number	Date	Time	Height	Length	Slope (Angle)	Flow Rate gpm	DO (HANNA 9142) mg/L		Temperature (Extech) degrees F		pH (Extech)		Ferrous Iron (DR8 90) mg/L		Depth of Water in Pipe	Samplers
							Influent	Effluent	Influent	Effluent	Influent	Effluent	Influent	Effluent		
1	2/4/2010	1106	7.0 ft	20 ft	3:1	49.5	4.0		46.2		7.96				~1 in (top of correlated part)	RH/AG
2	2/4/2010	1109	7.0 ft	20 ft	3:1	49.5	2.8		46.2		7.90				~1 in (top of correlated part)	RH/AG
3	2/4/2010	1112	7.0 ft	20 ft	3:1	49.3	2.2		46.2		7.89				~1 in (top of correlated part)	RH/AG
4	2/4/2010	1113	7.0 ft	20 ft	3:1	49.5		8.2		46.2		8.02			~1 in (top of correlated part)	RH/AG
5	2/4/2010	1115	7.0 ft	20 ft	3:1	49.3		7.8		46.4		8.01			~1 in (top of correlated part)	RH/AG
6	2/4/2010	1117	7.0 ft	20 ft	3:1	49.5		7.5		46.4		8.04			~1 in (top of correlated part)	RH/AG
7	2/4/2010	1122	7.0 ft	20 ft	3:1	49.1							0.14		~1 in (top of correlated part)	RH/AG
8	2/4/2010	1126	7.0 ft	20 ft	3:1	49.1							0.14		~1 in (top of correlated part)	RH/AG
9	2/4/2010	1130	7.0 ft	20 ft	3:1	49.3							0.18		~1 in (top of correlated part)	RH/AG
10	2/4/2010	1134	7.0 ft	20 ft	3:1	49.3								0.14	~1 in (top of correlated part)	RH/AG
11	2/4/2010	1138	7.0 ft	20 ft	3:1	49.3								0.15	~1 in (top of correlated part)	RH/AG
12	2/4/2010	1141	7.0 ft	20 ft	3:1	49.3								0.14	~1 in (top of correlated part)	RH/AG
13	2/4/2010	1142	7.0 ft	20 ft	3:1	49.6	3.1		46.2		7.92				~1 in (top of correlated part)	LD/AG/RH
14	2/4/2010	1145	7.0 ft	20 ft	3:1	49.5	1.9		46.2		7.92				~1 in (top of correlated part)	LD/AG/RH
15	2/4/2010	1148	7.0 ft	20 ft	3:1	49.3	1.9		46.2		7.92				~1 in (top of correlated part)	LD/AG/RH
16	2/4/2010	1149	7.0 ft	20 ft	3:1	49.6		8.1		46.2		8.04			~1 in (top of correlated part)	RH/AG
17	2/4/2010	1151	7.0 ft	20 ft	3:1	49.8		7.6		46.4		8.06			~1 in (top of correlated part)	RH/AG
18	2/4/2010	1153	7.0 ft	20 ft	3:1	49.6		7.5		46.2		8.05			~1 in (top of correlated part)	RH/AG
19	2/4/2010	1156	7.0 ft	20 ft	3:1	49.6							0.14		~1 in (top of correlated part)	RH/AG
20	2/4/2010	1159	7.0 ft	20 ft	3:1	49.3							0.15		~1 in (top of correlated part)	RH/AG
21	2/4/2010	1203	7.0 ft	20 ft	3:1	49.5								0.13	~1 in (top of correlated part)	RH/AG
22	2/4/2010	1207	7.0 ft	20 ft	3:1	49.1								0.12	~1 in (top of correlated part)	RH/AG
23	2/4/2010	1210	7.0 ft	20 ft	3:1	49.5								0.15	~1 in (top of correlated part)	RH/AG
24	2/4/2010	1211	7.0 ft	20 ft	3:1	49.5	4.0		46.0		7.94				~1 in (top of correlated part)	LD/AG/RH
25	2/4/2010	1213	7.0 ft	20 ft	3:1	49.3	2.7		46.0		7.96				~1 in (top of correlated part)	LD/AG/RH
26	2/4/2010	1215	7.0 ft	20 ft	3:1	49.5	2.2		46.0		7.94				~1 in (top of correlated part)	LD/AG/RH
27	2/4/2010	1217	7.0 ft	20 ft	3:1	49.3		7.9		46.2		8.10			~1 in (top of correlated part)	RH/AG
28	2/4/2010	1219	7.0 ft	20 ft	3:1	49.3		7.7		46.2		8.09			~1 in (top of correlated part)	RH/AG
29	2/4/2010	1221	7.0 ft	20 ft	3:1	49.5		7.6		46.2		8.09			~1 in (top of correlated part)	RH/AG
					Average:	49.4	2.8	7.8	46.1	46.3	7.9	8.1	0.15	0.14		

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Engineering design aspects of passive *in situ* remediation of mining effluents

David M. Laine and Adam P. Jarvis

Abstract

Passive treatment of contaminated effluents can offer a 'low cost' management opportunity to remediate drainages to the standards required by enforcement agencies. However, the initial cost of construction of passive treatment systems is significant and often in excess of that for active treatment systems. It is therefore important that the engineering design of the passive systems produces an effective and efficient scheme to enable the construction and maintenance costs to be minimised as far as possible. Possible parameters for the design of passive systems are suggested to seek to obtain uniformity in size and layout of treatment elements where this may be possible.

Passive treatment systems include aeration systems, sedimentation ponds, aerobic and anaerobic wetlands, anoxic limestone drains and reducing alkalinity producing systems. Most active treatment systems also include passive elements in the treatment stream. The basic design considerations that should be considered to ensure the construction of efficient systems are discussed.

Key words: construction, design, maintenance, mine water, passive treatment

INTRODUCTION

Mine water generation and impacts

When mining galleries are excavated, atmospheric oxygen and water may come into contact with the exposed rock. Pyrite minerals (FeS_2), ubiquitous in Coal Measures and Coal Measure shales, are readily oxidised by the atmospheric oxygen. The oxidised residue is highly soluble, and therefore easily washed from the surface of the rock by water draining through the workings. The overall reaction can be summarised as follows (from Barnes and Romberger 1968):

$$4\,FeS_2 + 15\,O_2 + 14\,H_2O \Rightarrow 4\,Fe(OH)_3 + 16\,H^+ + 8\,SO_4^{2-}$$

In reality, as many as 15 reactions are involved in the oxidative dissolution of pyrite (Nordstrom and Southam 1997), and the reaction is bacterially catalysed (Singer and Stumm 1970). However, the equation above is sufficient to illustrate that:

(i) iron is released into the water, initially perhaps as ferrous iron (Fe^{2+}), but ultimately as ferric hydroxide in the surface environment; and
(ii) the reaction generates acidity (represented here by the protons, H^+).

Other sulphide minerals, such as sphalerite (ZnS), greenockite (CdS) and arsenopyrite (FeAsS), may also release metals into solution, but these monosulphides do not have the same acid-generating potential as the disulphide, pyrite.

Discharges at the surface may be alkaline if, for example, carbonate strata overlie the Coal Measures. Discharges that are not buffered may emerge acidic. These discharges may contain other metals, such as aluminium and manganese, because of their higher solubility in low pH waters. Such is the extent of pyrite within workings, this oxidative dissolution process may continue for many decades, if not centuries, before the pollution abates without intervention.

Approximately 700 kilometres of rivers in the UK are seriously impacted by drainage from abandoned coal workings (Jarvis and Younger 2000). This includes around 60 significantly contaminated mine water discharges and around 350 less significant discharges. Any discharge with an iron concentration above approximately 2 mg/L is likely to cause visual staining due to

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the deposition of iron hydroxide (often termed 'ochre'). High-volume, poor quality, discharges may cause staining for many kilometres downstream, rendering these waters unsuitable for abstraction for many uses, and often devastating the aquatic fauna and flora of the receiving watercourses (Jarvis and Younger 1997).

PASSIVE TREATMENT SYSTEMS

Passive remediation may be defined as follows:

> Passive *in situ* remediation signifies an engineering intervention which prevents, diminishes and/or treats polluted waters at source, using only naturally available energy sources (such as topographical gradient, microbial metabolic energy, photosynthesis and chemical energy), and which requires only infrequent (albeit regular) maintenance to operate successfully over its design life.
> *Passive In situ Remediation of Acidic Mining/Industrial Discharges Research (PIRAMID). This project was funded under the EU Fifth Framework Programme, website www.piramid.org*

The development of passive systems for minewater treatment arose from the longevity of such discharges (which, in some cases, may remain polluted for centuries). Because of this, the application of 'active' treatment methods (i.e. use of chemicals and energy) is often unappealing due to the high, and long-term, on-going costs. Passive treatment systems therefore endeavour to make use of natural (micro)biological and chemical processes that remove contaminants (predominantly iron and acidity) in an equally effective manner.

There are two principal strategies in passive *in situ* remediation (PIR):

- ***Passive prevention of pollutant release*** is achieved by the installation of physical barriers (requiring little or no long-term maintenance) that inhibit pollution generating chemical reactions (for instance, by permanently altering redox and/or moisture dynamics), and/or directly preventing the migration of existing polluted waters.
- ***Passive treatment*** is achieved using constructed (or appropriated natural) gravity flow systems, in which all treatment processes used meet the criteria of the definition given above.

Passive treatment technologies embrace wetland type systems, sub-surface reactive barriers, and an increasing array of gravity flow geochemical reactors. The design of major passive treatment units is discussed further in this report.

Throughout the discussion it is important to bear in mind that passive treatment is not always a feasible option for mine water treatment. This is because pollutant removal mechanisms in passive systems are slower than the equivalent processes in active treatment. Consequently, passive systems typically have to be far greater in size, and the area of land required is therefore greater. For very high volume, poor quality discharges, designers may have no option other than to resort to chemical treatment.

In practice many systems draw on both active and passive treatment technologies. For example, the Old Meadows mine water treatment scheme, Lancashire, UK, employs sodium hydroxide dosing to raise the pH of the acidic discharge (facilitating more rapid precipitation of iron), which then flows through a sedimentation pond and tertiary treatment wetland to remove the iron hydroxide precipitate. At Vivian (Six Bells), South Wales, a net-alkaline (i.e. alkalinity > acidity) discharge contains circa 50 mg/L iron. This discharge is dosed with hydrogen peroxide to facilitate rapid oxidation of ferrous iron to ferric iron (active treatment), before discharge to sedimentation ponds and a tertiary wetland (passive treatment).

DESIGN DATA COLLECTION

It is vital that representative information on the quality and quantity of the discharge is obtained over as long a period of time as possible before the design commences. Ideally flow measurements should be made over a minimum of a twelve-month period, so that monthly and annual variations may be determined. Unfortunately, it is often the case that a 'design and construct' contract is not long enough to permit the collection of data over such a long period.

Sophisticated flow measurement devices (such as continuous data logging) are often unnecessary for mine waters. Simple structures such as 'V'-notch weirs, H-flumes, and even a bucket and stopwatch, can provide adequate information to give confidence in the design. Weir structures are often simple and easy to install and monitor. For treatment systems, the vital design variable is the maximum flow-rate, and variations within that flow-range are not always vital. Regular 'snapshot' monitoring of the flow, by measurement of the depth over a weir, will often provide sufficient data. The maximum flow-rate of a mine water discharge can often be estimated, since ochreous discharges invariably leave a 'tide mark' on the measuring structure.

Wherever possible, it is advisable to build a contingency into the design to allow for unusually high flow-rates. This is particularly the case where few flow-rate data are available. Particularly in the case of shallow abandoned coalfields, flow measurements should be taken in both dry and wet periods, as the influence on discharge rate can be significant.

In terms of water quality measurement, there is no substitute for field testing of the effluent at the point of discharge. This ensures that the design data are truly representative of the quality of the mine water. All too often, designs have been based on the results of laboratory analysis of water samples that have been transported and stored for many days, during which time the chemistry of the effluent may change significantly.

For discharges arising from abandoned coal mines, or their spoil heaps, the key water quality design variables are *usually* pH, acidity (including determination of permanent/temporary status), alkalinity and ferrous and ferric iron concentration. Therefore the field-testing carried out should include these variables, together with other variables such as temperature, dissolved oxygen concentration, and conductivity.

More detailed analysis should be undertaken, particularly at the outset of a project when laboratory analysis of a range of metals should also be undertaken. It is always easy to identify if iron, and often aluminium, is a problem, because of the very visible staining caused. Other metals are not visible in watercourses unless they are at very high concentrations e.g. zinc, manganese, copper. In determining which elements to analyse for, careful inspection of the local geology and land use should give clues as to what else may be present.

Analysis of calcium, magnesium, sodium and potassium will assist in establishing the source and evolution of waters (often important if the pathway of water through long-abandoned workings is initially unclear). Concentrations of the major anions (sulphate, chloride and bicarbonate) should also be determined. Sulphate is invariably present in high concentrations. Although not the case in the UK, in semi-arid countries such as South Africa, removal of sulphate is the primary objective of treatment, since these waters are subsequently required for potable supply (the processes that remove sulphate will also remove any metal contaminants).

In the case of mine waters emerging from deep workings following rebound, the quality of a mine water discharge will normally change over time. Often there is a 'first flush' of very poor water quality (Younger 1997). The quality normally improves quickly but it takes a considerable period of time to reduce to the level of contamination experienced when the mine was working. For example, it took the Morton discharge in Derbyshire, UK about 30 years to improve from the initial quality of 160 mg/L total iron to 40 mg/L total iron: a level still above that measured during operation of the mine. The following equation has been derived for the estimation of the period of the first flush (from Younger *et al.* 2002):

$$t_f = (3.95 \pm 1.2).t_r$$

where t_f = duration of first flush and t_r = rebound time i.e. time for workings to flood

In the UK there has been an increasing trend towards intercepting rising mine waters during rebound, to prevent an uncontrolled surface discharge (this is often done because the likely location of the uncontrolled discharge is unclear). This can be achieved by sinking a borehole into abandoned workings, or utilising old mineshafts and then pumping water at a rate sufficient to maintain a steady water level in the workings. The quality of water discharged from a borehole should be monitored, particularly at the outset of pumping, as it invariably deteriorates, in a similar fashion to that experienced during the 'first flush' period of uncontrolled discharges. Caution should also be taken in respect of sampling of water quality in shafts where stratification can give rise to unrepresentative results.

Mine water hydrology and quality may also change for the worse, often without warning. The Bullhouse mine water discharge, UK, turned from net-alkaline to net-acidic during construction of the treatment system, giving rise to the need to reconsider the treatment philosophy (Laine 1997). Conversely, the mine water quality at the Woolley treatment system, UK, improved significantly because of abstraction and treatment of mine water at the adjoining Caphouse mine (Laine 1997). At the Pelenna site in south Wales, the system was completed and operating successfully when the discharge dried up, only to reappear some 50 metres up the valley, presumably due to a collapse within the workings (P.L. Younger, University of Newcastle, UK, *personal communication* 2001).

These anecdotes reinforce the need for representative mine water quality. The unpredictable nature of such events makes it advisable to incorporate flexibility into designs.

PASSIVE TREATMENT PROCESS SELECTION

In the following paragraphs specific design guidance for passive treatment units is provided. This should provide potential implementers of passive treatment schemes with an overview of engineering requirements for such systems. However, as a broad guide to the reader, Figure 1 illustrates a 'decision tree' for selection of most appropriate passive treatment technology

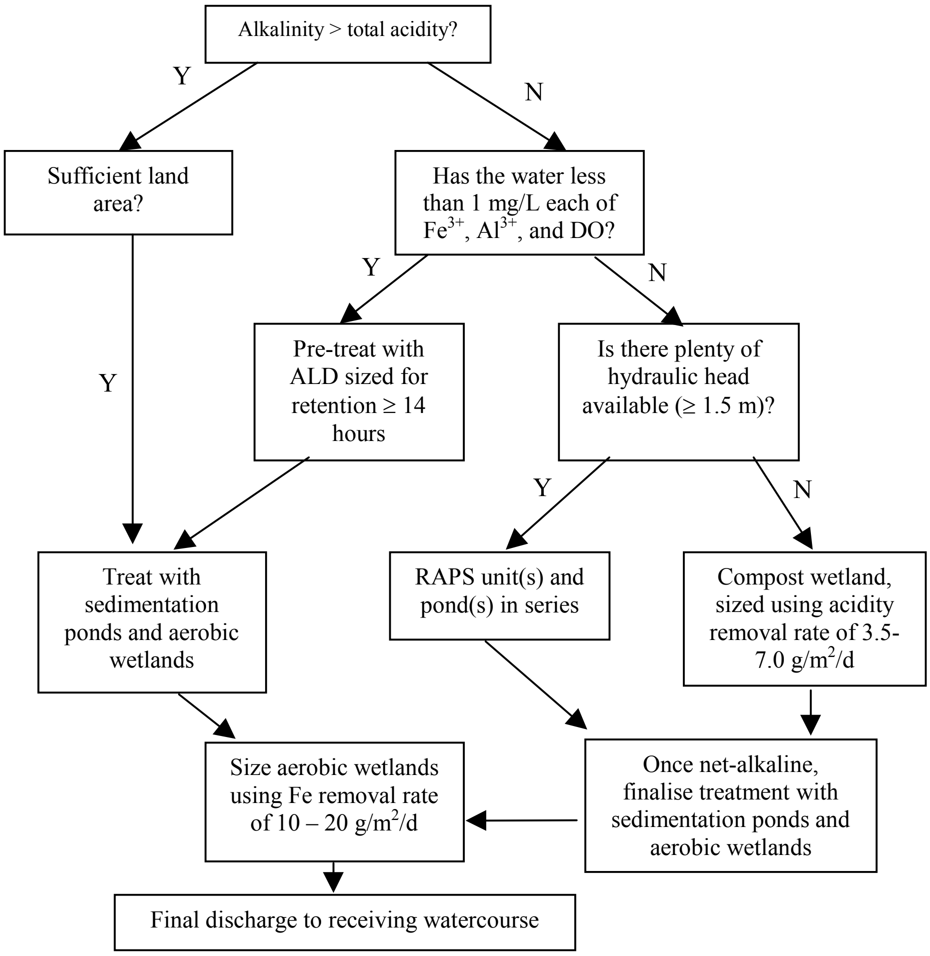


Figure 1. Simplified decision flow chart showing passive treatment unit selection (adapted from Younger et al. *2002) (see text for further details)*

for any given discharge. The flow chart includes only treatment technologies discussed in this article. Other technologies do exist, though they are less widely applied, at least in the UK. For example, if the answer to the 'sufficient land area?' question is 'No', passive treatment may still be possible, by using inorganic media systems (Younger *et al.* 2002). Details can be found in Jarvis and Younger (2001) and Younger *et al.* (2002). Subsurface flows may be intercepted using permeable reactive barriers (PRBs). Benner *et al.* (1997) and Younger *et al.* (2002), give details.

For some discharges (especially those of very poor quality and high volume) passive treatment will be inappropriate. It is not possible to give a specific iron or acidity loading at which passive treatment becomes unfeasible, because these decisions are site specific. Such a decision is typically based on an assessment of the costs and benefits (over the short and long term) of the options available (be they passive or active), in view of the land area available and its topography.

Examples of a variety of passive treatment systems currently operational in the UK are shown in Table 1. The reasons why particular treatment unit(s) were selected at each site should become clear in the following paragraphs.

GUIDELINES FOR THE DESIGN OF SEDIMENTATION PONDS

Introduction

There are significant advantages in having standard guidance for water treatment systems in any industry, based upon experience of design and operation of such structures in compliance with enforcement agency standards. In the past, water treatment facilities for drainages have been considered the 'poor relations' in the development of a site, and were relegated to whatever small area of land remained when the development proposals were completed. With the increasing

Table 1. Examples of different passive treatment systems in the UK

Site name	Water quality/flow			Treatment type	Comments
	Mean flow-rate (L/s)	Net-alkaline/ net-acidic	Mean iron concentration (mg/L)		
Quaking Houses, Co Durham[1]	2	Net-acidic	6	Compost wetland and aerobic wetland	Insufficient hydraulic head for RAPS
Deerplay, Lancashire[2]	2	Net-acidic	22	RAPS and aerobic wetland	Steeply sloping site, limited land availability
Edmondsley, Co Durham[3]	10	Net-alkaline	30	Aerobic wetland	Flat site, Fe load not excessive
Woolley, West Yorkshire[4]	150	Net-alkaline	15	Sedimentation ponds and aerobic wetland	Large flat site; high iron load

1. Jarvis and Younger (1999)
2. Jarvis and England (2002)
3. Younger *et al*. (2002)
4. Laine (1997)

aspirations of the enforcement authorities for higher quality effluent discharges, often driven by European Union legislation, it is increasingly unlikely that facilities allocated in such a manner can comply with the discharge quality constraints imposed. The area required for effective facilities needs to be related to the size of the catchment and/or the quantity and quality of discharge, if effective remediation is to be achieved. Whilst it is accepted that the area allocated must not be excessive, as it may impair the development, it is nevertheless required to be a realistic area that can achieve the water quality improvements necessary.

In the UK coal industry, basic water treatment design guidance for engineers was provided by NCB (1982). To some extent this text was instrumental in achieving a uniformity of effective design throughout the industry. Although not in print, copies remain available in some libraries. However, other than sedimentation ponds, NCB (1982) did not consider the design of passive systems for mine water remediation (i.e. wetlands, biogeochemical reactors). The paragraphs following provide guidance on the design of sedimentation ponds, wetlands and other passive treatment technologies, and provide references to some of the key texts in the field.

Sedimentation pond size

Sedimentation ponds invariably form the main repository for metal precipitates at mine water treatment sites. However, for them to be effective in removing iron solids, the water must be net-alkaline at the influent point, and there must be sufficient dissolved oxygen present to enable the oxidation and precipitation reactions to occur. Therefore sedimentation ponds are invariably preceded by an aeration step, and sometimes passive or active means of alkali generation. Aeration may be chemically induced, or it may achieved by a simple cascade system. Where iron concentrations are high, i.e >30 mg/L, multiple cascades may be required (NCB 1982).

In the UK coal mining industry, sedimentation pond areas were designed on the basis of providing 40 square metres of surface area of pond for every hectare of surface drained. This is effectively equivalent to an overflow rate of 1×10^{-5} metres/second in conventional water industry design parlance, or 100 m^2 of pond surface area for every litre/sec of drainage (NCB 1982). This concept was based upon experience in the USA, and is primarily relevant to inert suspended solids.

Whilst this ‘rule of thumb’ approach is useful for quick estimates of area requirements, it is based on settlement of particle sizes of >4 μm. However, particles of iron hydroxide are typically smaller than this (often less than 2.5 μm, at least initially) (Younger *et al.* 2002). The combination of the time for the formation of $Fe(OH)_3$ precipitates, and the smaller particle size, means that residence time is a crucial design consideration. A retention time of 24 hours is likely to be adequate, but because actual retention time will be less than theoretical retention time (i.e. pond volume divided by flow-rate) as sludge depth increases, total pond volume is often designed on the basis of at least 48 hours retention time.

A further consideration in pond design is the frequency of sludge removal required. As a guide level, sludge should be removed before it is within 1 m of the water surface, but the best guide to when sludge removal is required is the effluent iron concentration. Systems designed by IMC are usually designed in such a way that sludge removal will not have to be undertaken more than once per year.

Chemical dosing in sedimentation ponds

Where inadequate space is available to allow the construction of sedimentation ponds of the desired size, resort may have to be made to chemical dosing. The intention of the dosing is to achieve rapid generation of suspended solids so that settlement takes place immediately over the full area of the pond. There are notable cases of schemes designed without appreciation of the time of reaction required to convert ferrous iron to ferric iron, where mine water had flowed about 50% through the pond system before precipitation of the iron solid allowed sedimentation of generated particles to commence.

IMC Consulting Engineers have recent experience of the construction of active chemical treatment schemes for mine water using hydrogen peroxide at the Vivian (Six Bells) and Acomb mine water schemes. A ‘rule of thumb’ guideline suggests a provision of 50 m^2 of pond surface area for every 1 L/s of influent flow – half the guideline for inert solids, and significantly less than for fine ferrous particles requiring the addition of reaction time.

Rapid precipitation of solids can also be achieved by the use of alkaline materials such as sodium hydroxide, whether the mine water be acidic or alkaline. A provision of pond surface area also of 50 m^2 area for every 1 litres/sec of influent flow is also thought to be appropriate.

The use of chemical dosing involves cost, and the health and safety implications of using aggressive chemicals should be avoided unless land availability constraints dictate active treatment.

Sedimentation pond geometry and layout

Pond geometry and layout are important considerations, which influence both the operational efficiency of treatment, and the amenity value of the completed system. Exact dimensions and layout will clearly be governed to an extent by local site conditions, but there are certain general rules that should be applied in all circumstances.

Where a number of sedimentation ponds are proposed, often to allow for continuation of treatment during desilting, the ponds should be arranged to operate in parallel and not in series. Splitting the flow between two ponds reduces the velocity of flow to 50%, which promotes greater efficiency for sedimentation. Where ponds of equal size are arranged in a series, there is no reason why an inert particle that has not been detained in the first pond should subsequently settle in the second pond, as the sedimentation potential is identical in each.

Practical experience suggests that the best sedimentation performance is obtained when the ratio of the pond length to width is between 3:1 and 5:1. Higher length to width ratios may result in ‘streaming’ across ponds, whilst lower length to width ratios may promote short-circuiting.

Multiple sedimentation ponds have other advantages:

(i) by arranging ponds so that they may be operated in parallel, sludge may be removed from one pond (or maintenance undertaken) whilst the second continues to operate;
(ii) carefully designed, a site may look more attractive with multiple smaller ponds as opposed to a single large pond with an equivalent volume.

Aeration stages

The oxidation of ferrous iron to ferric iron uses up dissolved oxygen in the water. Since there is a maximum concentration of oxygen in water (9.1 mg/L at 1 atmosphere and 20°C, in freshwater) it stands to reason that high iron concentrations will exhaust the supply. For this reason it is sometimes necessary to have multiple aeration cascades, and therefore multiple ponds. Based on the theoretical rate of oxygen consumption, there should be one aeration cascade per 50 mg/L ferrous iron (Younger *et al*. 2002). Practical experience suggests that inefficiencies in oxygen transfer effectively make the requirement one aeration cascade per 30 mg/L ferrous iron (NCB 1982). Thus, for a discharge with 90 mg/L ferrous iron, there should be three aeration cascades, each followed by a sedimentation pond.

Flow inlet and outlet structures

Good flow distribution increases operational efficiency by keeping inlet velocities low, and minimises the potential for short-circuiting. The objective of inlet and outlet structures is to spread the flow evenly across the whole of the pond width. These guidelines apply equally to sedimentation ponds and wetlands.

Conventional means of distribution on sedimentation ponds generally comprise both full-length weir inlet and outlet systems or a multiple pipe inlet arrangement. Full-length concrete weir systems are expensive to install, but operate by minimising velocity over the length of the weir. In reducing the velocity, significant settlement of solids often occurs in the inlet channel. Whilst simple to rectify, safety of operatives working close to deep water must be ensured.

Multiple pipe inlet systems are much cheaper to install and generally comprise a number of pipe outlets over the full width of the pond to achieve even distribution. Whilst this may not be achieved as effectively as a full-length weir, it is nevertheless adequate for most sedimentation pond systems. It is probably slightly more inconvenient to maintain such pipes, as rodding equipment will be necessary, but the locations at which the work will be carried out are generally reasonably convenient and safe. Pipe flow velocity can be designed to minimise accretion of ochre within the pipe network.

Irregular shaped pond systems will provide more natural looking features in the landscape, although it must be recognised that such shapes are not necessarily complementary to effective sedimentation performance. The even passage of water through the system is more difficult to achieve, and features such as shallow ledge areas around the pond perimeter significantly reduce sedimentation capacity. It is vital that the minimum calculated pond requirement is provided with the full depth required for sedimentation purposes and that the aesthetic curved areas to form the landscaping element are *in addition* to the basic requirement.

GUIDELINES FOR WETLAND AND OTHER PASSIVE SYSTEM DESIGN

Introduction

There are two types of wetland that are generally employed for mine water treatment:

(i) aerobic wetlands;
(ii) anaerobic wetlands

Aerobic wetlands are used where treatment requirement is limited to oxidation of dissolved contaminants and the detention of the generated suspended solids. Most commonly they are used for the retention of ferric iron precipitates. Such systems typically have a soil media in which metal-resistant wetland plants are grown.

Anaerobic (compost) wetlands may be used for net-acidic discharges (i.e. acidity > alkalinity). Such systems have a compost medium, the exact nature of which is discussed below. Plants are less of an essential component in compost wetlands, but may still be encouraged for aesthetic reasons. In compost wetlands biogeochemical reactions are encouraged to generate alkalinity through sulphate reduction processes and precipitate metals (especially iron) as insoluble monosulphides.

Aerobic wetlands

The design guidelines of Hedin *et al*. (1994*a*) are still those that are most commonly applied in the design of aerobic wetlands. For a net-alkaline discharge, where removal of iron is the objective, the following formula is used:

Wetland area (m^2) = iron load (g/d) / removal rate ($g/m^2/d$)

It will be evident from this that the knowledge of the flow-rate and iron concentration are required to calculate the numerator of the equation. Hedin *et al*. (1994*a*) derived the removal rate from monitoring wetlands in the USA. For a wetland from which the effluent must meet regulatory standards, a figure of 10 $g/m^2/d$ was derived. A second figure of 20 $g/m^2/d$ was derived for situations where regulatory standards were not so stringent, and a ‘reasonable improvement’ in water quality was sufficient. It should be noted that this equation assumes removal of *all* of the iron. In practice this is both unlikely and unnecessary. The iron load can therefore be calculated by subtracting the target effluent iron concentration (e.g. 2 mg/L) from the influent iron concentration, and then multiplying by the flow-rate.

With the increasing number of wetlands now operational in the UK, it has become possible to design wetlands on a *pro rata* basis – i.e. take the removal rate from an operational wetland and apply it to one under design. However, such an approach must be applied with caution, since mine waters are unique, and therefore removal rates may vary. It is always worth calculating the area according to the removal rates as a check.

Two examples of actual removal rates in aerobic wetlands in the UK are as follows:

(i) the Old Meadows wetland, which has a flow-rate of 40 L/s, and influent total iron concentration of up to 10 mg/L. The surface area of the wetland is 1800 m^2, and effluent iron concentration is (less than) 2 mg/L. Therefore the removal rate is 15.4 $g/m^2/d$;
(ii) the Woolley wetland, which had a flow-rate of 200 L/s, and influent total iron concentration as high as 10 mg/L. The surface area of the wetland is 14 000 m^2, and effluent iron concentration is (less than) 1 mg/L. Therefore the removal rate is 11.1 $g/m^2/d$.

The general dimensions of aerobic wetlands should be as for sedimentation ponds. However, as wetlands may form effective amenity areas, there are significant advantages in avoiding rectilinear shapes. Nevertheless, steps should still be taken to minimise streaming or short-circuiting. Planting will help significantly in spreading water over the full width of the wetland without any engineering intervention.

Vegetation and growing media for aerobic wetlands

Typically the growing medium in aerobic wetlands is a good quality soil, usually placed to a depth of 300–400 mm. Important aspects of the soil quality are as follows:

- it does not contain excessive large stones or other sharp objects, which may puncture pond liners and impede plant growth;
- the soil is not contaminated. The best way to ensure this is to check the source of the soil, and ensure that a reliable analysis is made;
- it contains sufficient nutrients, in appropriate proportions, to support reed growth.

Research is presently taking place into the potential for use of suitable 'waste' materials as a growing medium, as often these can be obtained at minimal costs, thus avoiding the costly import of soil.

Aquatic plants in wetlands serve a number of useful functions:

(i) plants are excellent at ensuring flow is distributed evenly across the wetland, as long as they are planted across the direction of flow rather than parallel to it;
(ii) stems and leaves may provide additional surface area for adsorption of metals;
(iii) they significantly improve the aesthetic appearance of site, and may form a wildlife habitat;
(iv) even in aerobic wetlands, the continuous carbon source provided by plants may encourage sulphate reduction reactions in the subsurface, in turn encouraging the immobilisation of metals (Mitsch and Wise 1998).

Bioaccumulation is not generally recognised as an important removal process in wetlands for mine water treatment, and certainly not in respect of iron. However, recent research at a wetland in the north-east of England suggests that approximately 30% of the iron (influent concentration 3 mg/L) is removed by bioaccumulation (P.L. Younger, University of Newcastle, *personal communication* 2002). This may well be related to the comparatively low influent concentration.

Recent laboratory research (Batty and Younger 2002) has suggested that subjecting wetland plants to water with iron concentrations in excess of 10–20 mg/L will result in limited root development, and potentially poor overall growth and death. Reed die-back at some sites in England has recently been ascribed to the influent iron concentration being too high. However, it is more likely that the depth of water in these wetlands was too great, and the plants were therefore inundated. It is important to limit water depth above the surface to around 100–200 mm to prevent this, although plants such as *Scirpus lacrustus* can tolerate greater depths.

Evidence that reeds are capable of tolerating high iron concentrations comes from the USA. In deriving the design formula discussed above Hedin *et al.* (1994*a*) studied 16 wetlands, nine of which received water with >100 mg/L iron. All of the sites had been operational for several years, but none had suffered significant loss of reeds. At the current time the *laboratory-based* evidence suggesting that reeds will not tolerate iron concentrations greater than 20 mg/L is far outweighed by the *field-based* experiences to the contrary.

Nevertheless, most aerobic wetlands in the UK are used as tertiary treatment systems, following sedimentation lagoons. However, this has nothing to do with reed tolerance to iron but is because:

(i) using the formulae above, a wetland receiving a high iron concentration will require a larger land area than an equivalent sedimentation pond; and
(ii) removing large volumes of ochre from a wetland is far more difficult than removing it from a sedimentation pond (indeed, the plants must be removed as well).

A range of emergent aquatic plants have been used in wetlands for mine water treatment. Most commonly *Typha latifolia* (common name greater reedmace in the UK, or cattails in the USA) have been used. Increasingly monocultures are being avoided, and other types are planted in addition to *Typha*. Other commonly used aquatic plants include:

Phragmites australis (common reed)	Widely used, but may not thrive on exposed sites.
Juncus effusus (soft rush)	A naturally common species in wet upland areas
Scirpus (bulrush)	Is tolerant of deeper water depths where other types may not survive
Iris pseudacorus	An attractive species, improving appearance of wetland.

Typically, reeds are planted at a density of 3–4/m^2, between the months of May and June in the UK. 200 mm pot grown varieties are often favoured, as they are sufficiently advanced in terms of growth to survive conditions in the wetland, although considerable success has been achieved with cheaper 9 cm plugs.

Anaerobic (compost) wetlands

The reactions occurring in compost wetlands are more complex than those of aerobic wetlands. There are two key aspects to the removal of contaminants in compost wetlands. The first is the generation of alkalinity (and therefore neutralisation of acidity). This is accomplished in one or both of the following routes:

(i) generation of alkalinity via microbially mediated sulphate reduction. The media used (see below) must be suitable for the colonisation of sulphate reducing bacteria (SRB);
(ii) dissolution of high calcium carbonate limestone, mixed into the compost media during placement.

For SRB to be active, conditions must be anoxic, the bacteria must have a source of low-carbon number compounds to metabolise, and there must be high sulphate concentration (>100 mg/L). In very simple terms the reaction that occurs (where CH_2O represents the carbon source) is as follows (Younger *et al.* 2002):

$$SO_4^{2-} + 2\,CH_2O \Rightarrow H_2S + 2\,HCO_3^-$$

Metals may subsequently form insoluble precipitates in the following manner (where M^{2+} represents a divalent metal ion) (Hedin *et al.* 1994*a*):

$$M^{2+} + H_2S + 2\,HCO_3^- \Rightarrow MS + 2\,H_2O + 2\,CO_2$$

Since the primary objective of compost wetlands is to generate alkalinity under anoxic conditions, it is usually necessary to follow a compost wetland with another treatment unit, to aerate the water and remove metal contaminants as hydroxide precipitates (a reaction that will happen effectively once alkalinity is raised). In terms of passive treatment, this is ideally achieved using an aerobic wetland.

The design of anaerobic wetlands is usually based on the acidity load of the discharge and the anticipated removal rate of that acidity. As with aerobic wetlands, the design formula of Hedin *et al.* (1994*a*) is most commonly applied:

$$\text{Wetland area (m}^2\text{)} = \text{acidity load (g/d) / removal rate (g/m}^2\text{/d)}$$

For a wetland from which the effluent must meet regulatory standards, a removal rate of 3.5 $g/m^2/d$ was derived. A second figure of 7 $g/m^2/d$ was derived for situations where regulatory standards were not so stringent, and a 'reasonable improvement' in water quality was sufficient.

Anoxic limestone drains (ALDs)

An anoxic limestone drain (ALD) is a buried bed of high calcium carbonate limestone, the objective of which is solely to raise pH. Dissolution of the calcite neutralises proton acidity and generates bicarbonate alkalinity, as shown by the following reactions (Younger *et al.* 2002):

$$CaCO_3 + 2\,H^+ \Leftrightarrow Ca^{2+} + H_2O + CO_2$$

$$CaCO_3 + H_2CO_3 \Leftrightarrow Ca^{2+} + 2\,HCO_3^-$$

The ALD is buried in order to promote anoxic conditions. This prevents oxidation and precipitation of ferrous iron and manganese, which would otherwise coat the limestone surfaces, preventing further dissolution (and therefore alkalinity generation). This coating process is referred to as 'armouring'. In ambient conditions aluminium only occurs as Al^{3+}, and will readily hydrolyse to Al $(OH)_3$ under anoxic conditions.

For these reasons ALDs are only appropriate where Fe^{3+} and Al^{3+} concentrations are less than approximately 2 mg/L, and for other ferruginous waters where dissolved oxygen concentration is ≤1 mg/L (Younger *et al.* 2002).

The limestone should have a calcium carbonate content in the order of 90% to work effectively. To prevent blocking problems this should be single size limestone of size in the range 50–75 mm. Sizing of ALDs is based on residence time required to generate maximum alkalinity (around 300 mg/L as $CaCO_3$). Currently this time is taken to be 14 hours (Hedin *et al.* 1994*b*). For the purposes of design calculations, limestone of the size quoted is assumed to have a porosity of 50%.

Because of the restrictions on the quality of mine water suitable for treatment in an ALD (i.e. the Fe, Al and dissolved oxygen concentration), they are not particularly suited for use in the UK. Acidic discharges in the UK typically contain elevated iron and aluminium concentrations, and therefore for these discharges either a compost wetland or a Reducing and Alkalinity Producing System (RAPS) is adopted.

Reducing and alkalinity producing systems (RAPS)

RAPS were developed in response to the shortcomings of ALDs by Kepler and McCleary (1994). Conceptually a RAPS is an ALD overlain with a layer of compost. Water is driven downwards through the compost substrate, which removes dissolved oxygen, converts Fe^{3+} to Fe^{2+}, and is a sink for Al^{3+} (as aluminium hydroxide). In its reduced state water can pass through the underlying limestone layer without 'armouring' problems. Although not specifically designed to do so, sulphate-reducing bacteria may colonise the compost layer, adding the potential for alkalinity generation and immobilisation of Fe^{2+} as FeS. However, it is unwise to assume this in the design of such systems.

The design of RAPS is along similar lines to ALDs, i.e. the limestone element of the system should be designed to ensure 14 hours residence time (Kepler and

McCleary 1994). Once the volume of limestone is calculated, it is a simple matter to calculate the area required (usually assuming a limestone depth of 0.5–1.0 m. The depth of compost above the limestone should be at least 0.5 m. In addition the compost layer should be submerged to a depth of at least 200 mm.

From the preceding discussion it will be obvious that there must be sufficient hydraulic head available to drive the water through the compost and limestone. Flat sites will not therefore be suitable for RAPS if this is to be accomplished by gravity. However, one of the great advantages of RAPS is that because they are downward flow systems it is much more likely that the entire volume of compost and limestone is utilised, and therefore the land area requirements are far less than equivalent (horizontal flow) compost wetlands.

To maintain the water level above the compost, the effluent discharge channel must be at a similar level to the water surface, which can present some engineering problems. Most simply this is achieved by attaching a 90° elbow joint and suitable length of pipe, to the pipe draining the limestone. However, the permeability of the compost will decrease over time, due to accumulation of solids, and therefore the height of the effluent pipe should be variable. This will enable the height of the effluent pipe to be reduced, such that the effective hydraulic head across the system can be increased, thus compensating for the reduction in compost permeability. The design of a system to allow backwashing of the stone and compost layer would be a distinct advantage.

RAPS systems have been designed and constructed at La Coruña in Spain (Laine 1998) and the Pelenna Valley in South Wales.

As an alternative to a configuration whereby compost overlies limestone, the compost and limestone units may be separated. In this case water flows *downwards* through the compost, and *upwards* through the limestone. Such a system has been installed by IMC at Deerplay, Lancashire, UK (Jarvis and England 2002). The advantages of this system are:

(i) because the limestone bed is an upward flow unit, efficiency of operation is at a maximum;
(ii) the required hydraulic head is less than that of a system with compost overlying limestone.

The only disadvantage of this configuration is that the area of land required is greater (although the limestone is not actually visible).

Because a RAPS is not designed to remove metals, it must be followed by another treatment unit. A sedimentation pond and/or aerobic wetland is the usual option.

Media selection for compost based systems

Selection of media for compost based systems is not (as yet) a precise science. For compost wetlands the primary objective is to select media that will support colonisation by sulphate reducing bacteria. For RAPS an additional requirement is that the media must be suitably permeable/porous. Standardised tests for assessing the potential of a medium to support SRB do not exist, and neither are there recommended values for the permeability and porosity of a compost for a RAPS. Rudimentary laboratory experiments can be used to assess whether compost will support SRB (see for example Younger *et al*. 1997). Tests can be conducted to ascertain permeability and porosity values, but it is not clear what values are acceptable. As a general rule, designers should aim to use compost with the maximum permeability and porosity, whilst still promoting reducing conditions.

Laboratory experiments are clearly of benefit, but because of the uncertainties outlined above, pilot-scale trials are strongly advisable when considering installation of compost based systems. Potential candidates for compost systems include the following:

(i) pressed digested sewage sludge;
(ii) horse manure;
(iii) composted domestic waste;
(iv) waste from paper production;
(v) shredded timber or bark;
(vi) mushroom compost.

Of these, (ii), (iii), (v) and (vi) have been used in successful systems. It will be noted that, with the exception of shredded timber, these media are all waste products. This is preferable because:

(i) using waste products is environmentally sustainable, since otherwise they would present a disposal problem;
(ii) because they are waste products they are usually available at very low cost, if not for free.

CASE STUDIES

Pilot-scale compost wetland to treat acidic colliery leachate at Aspatria, Cumbria

The design of an anaerobic compost system for a site in Cumbria for the UK Environment Agency is being progressed by IMC Consulting Engineers. Surface water run-off from an abandoned colliery spoil heap contains 400 mg/L Fe, 50 mg/L Al, 10 mg/L Mn, and is pH 3. Laboratory testing of effluent from the site mixed with pressed digested domestic sewage showed vigorous reduction of sulphate and generation of alkalinity. Further tests will be completed with paper waste and shredded timber to select the most appropriate compost mix for the site. A pilot testing scheme comprising four cells, each ten metres wide and 20 metres long, will be constructed on the site, and the performance of each in treating the effluent will be monitored, with the assistance of the Hydrogeochemistry Engineering Research and Outreach (HERO) group at Newcastle University. Following successful completion of the field testing, a full-scale treatment system, comprising a 20 000 m^2 surface flow anaerobic wetland and a 20 000 m^2 'volunteer' aerobic wetland is likely to be constructed at the site, to treat the entire volume of run-off from the site.

Design of a RAPS for the Sheephouse Wood mine water discharge, Yorkshire, UK

IMC Consulting Engineers have been commissioned by the UK Coal Authority to design a treatment system for the Sheephouse Wood mine water discharge, located north-west of the city of Sheffield. The discharge is from an abandoned mine adit, at a flow-rate of approximately 12 L/second. Limited field testing results available to date show a pH of 5.6, 50 mg/L total iron, 40 mg/L ferrous iron and acidity >200 mg/L as $CaCO_3$.

Traditional wastewater engineering for such a system would involve chemical injection followed by sedimentation ponds and, possibly, a tertiary treatment wetland. This is a proven approach, and about 20 such installations were constructed in the Yorkshire coalfield in the 1970s and 1980s. Latterly, similar systems have been constructed by IMC at Old Meadows, Lancashire, and Vivian (Six Bells), South Wales (because passive-only treatment was not feasible). Such systems have the advantage of being adaptable to variations in mine water quality. However, this security comes at the cost of provision of power and chemicals for the life of the treatment system, together with the enhanced maintenance costs associated with active treatment systems.

Sufficient land is available at Sheephouse Wood to make a passive-only treatment system a possible option for this acidic drainage. IMC is therefore considering the construction of a large RAPS at the site, in order to reduce the operating costs to the Coal Authority over the life of the site. Construction and long-term cost estimates for the passive RAPS are compared to cost estimates for an active scheme as shown in Table 1. It can be seen that the capital construction costs are in fact slightly higher for the RAPS. The RAPS cost estimate assumes that a man-made liner will be necessary, and includes the cost of high-calcite (>90%) limestone (significant elements of the capital cost). It is always hoped that the site may contain clay below the surface, which can be used as a liner for no cost, and that a cheap source of limestone may be available. However, this is never known until the detailed design of such a system begins. It is something of a misconception that the capital costs of passive treatment schemes will be significantly less than those for active chemical options.

The figures in Table 2 assume that the compost and limestone in the RAPS will require replacement after a period of 15 years (because of the uncertainties associated with the permeability/lifetime of compost – see above).

Table 2. Comparative cost estimates for a passive and active treatment system for the Sheephouse Wood mine water discharge (see text for details)

Item	Passive treatment (£)	Active treatment (£)
Capital cost of construction	600,000	500,000
Total cost over 40 years	1,100,000	1,500,000

40 year life; 6% interest.

Nevertheless, it can be seen that over the long-term, passive treatment systems are significantly cheaper than their active treatment equivalents, and this is the great advantage of passive schemes.

The concerns regarding the construction of such a large RAPS are as follows:

(i) how to ensure that the permeability of the compost remains adequate to pass the design flow. It is possible to raise the hydraulic head to increase flow by a greater depth of supernatant water, should problems occur;

(ii) can a backflushing system to maintain permeability operate successfully?

(iii) ensuring adequate retention time for water in the compost layer;

(iv) for acidity levels >200 mg/L, a conservative design would allow for two RAPS stages, in that Kepler and McCleary (1994) considered that between 150 and 300 mg/L for a single unit could be achieved. A RAPS would not only have to neutralise acidity, but it would also have to impart excess alkalinity to allow sedimentation of dissolved solids from the mine water. A twin RAPS system, with intervening sedimentation ponds and re-aeration systems, would significantly increase construction costs and leave little, if any, financial advantage in a net present value calculation of the whole life of a system.

Summary of Landfill Leachate Data

South Hilo Sanitary Landfill

Feasibility Study 2011

Sample Description	Alkalinity as CaCO3(mg/L)	Ammonia (mg/L)	BOD (mg/L)	COD (mg/L)	Chloride (mg/L)	Antimony (mg/L)	Arsenic (mg/L)	Barium (mg/L)	Beryllium (mg/L)
OREGON									
Agate Beach Landfill									
Average Value	587.4	73.6	77.6	289.4	298	<0.2	0.004	0.15	<0.01
Range of Detected Values	90-1280	9.2-150	11-93	32-1033	110-460	<0.2	0.000007-0.006	0.12-0.18	<0.01
Bandon Landfill									
Range of Detected Values		2.8		170	110		0.009		
Boise Cascade									
Average Value		8.1		800	64		0.009		
Range of Detected Values		5.3-10.9		800	36-92		<0.004-0.009		
GP Wauna Mill Landfill									
Average Value	3233	343		247	237		0.0265		
Range of Detected Values	1700-4000	138-450		170-300	110-300		0.0159-0.0325		
Tillamook Landfill									
Average Value	331.3	28.5	20.2	91.4	83.3	<0.2	<0.149	0.11	<0.1
Range of Detected Values	90-520	6.5-93	9-37	10-175	31-182	<0.2	<0.149	0.09-0.13	<0.1
Reedsport Landfill	255	11		30	45.1		<0.002		
Detected Values	255	11		30	45.1		<0.002		
Waldport Landfill									
Average Value	404	43.5	64	165	170		0.047		
Range of Detected Values	194-980	5.9-150	60-68	48-550	44-410		0.047		
Oregon Range of Detect Values	255-4000	2.8 - 400	9 - >260	10 - 1100	31 - 420	<0.2	0.005 - 0.047	0.09 - 0.18	<0.01
MINNESOTA									
Lyon County Landfill									
Average Value		77.5	513	692				0.723	
Range of Detected Values		64.5-90.4	6-1020	93-1630				0.723	
Crow Wing County Landfill									
Average Value	3373	474	452	2380	1393			0.568	
Range of Detected Values	2530-4640	370-694	264-654	888-6960	1020-2170			0.29-0.964	
Rice County Landfill									
Average Value		548	179	1051	1557			1.39	
Range of Detected Values		427-689	88-351	833-1200	1390-1810			1.01-1.59	
Minnesota Range of Detected Values	2530-4640	64.5-689	6-654	93-6960	1030-2170			0.29-1.59	
HAWAII									
South Hilo Sanitary Landfill									
Average Value	784	85	72	480		0.006		0.195	0.00047
Range of Detected Values	738-830	83-87	72	480		0.0006-0.0029		0.132-0.284	0.000128-0.0011
Other Landfills SWANA									

Summary of Landfill Leachate Data

South Hilo Sanitary Landfill

Feasibility Study 2011

Sample Description	Boron (mg/L)	Cadmium (mg/L)	Calcium (mg/L)	Chromium (mg/L)	Cobalt (mg/L)	Copper (mg/L)	Total Cyanide (mg/L)	Iron (mg/L)	Lead (mg/L)
OREGON									
Agate Beach Landfill									
Average Value	0.864	<0.01	52.2	0.0136	<0.06	<0.02		27.0	<0.1
Range of Detected Values	0.33-1.399		23-160	0.00217-0.044				7.1-52	
Bandon Landfill									
Range of Detected Values	0.7	<0.01	390	<0.03				0.1	
Boise Cascade									
Average Value	0.0455	<0.01	171.5	0.058				1.56	
Range of Detected Values	0.031-0.7	<0.01	92.9-250	0.0067-0.11				0.4-2.73	
GP Wauna Mill Landfill									
Average Value	0.522	0.00025	115	0.01067			0.01	9	
Range of Detected Values	0.31-0.63	0.00025	95.3-127	0.0037-0.0144			0.01	3.84-17.6	
Tillamook Landfill									
Average Value	0.82	<0.01	66.4	0.003	<0.06	<0.02		15.3	<0.1
Range of Detected Values	0.21-1.2	<0.01	18-130	0.00047-0.004	<0.06	<0.02		1.91-42	<0.1
Reedsport Landfill	0.216	<0.0003	51.9	<0.0005				0.103	
Detected Values	0.216	<0.0003	51.9	<0.0005				0.103	
Waldport Landfill									
Average Value	0.276	0.00012	41	0.026				29	
Range of Detected Values	0.246-0.306	0.00012	33.3-51	0.00072-0.1				3-45.2	
Oregon Range of Detect Values	0.031 -1.399	0.00012 - 0.00025	18 - 390	0.00072 - 0.11	<0.06	<0.02	0.01	0.1 - 63	<0.1
MINNESOTA									
Lyon County Landfill									
Average Value						0.007	<0.02		0.0005
Range of Detected Values						0.0049-0.011			0.0001-0.001
Crow Wing County Landfill									
Average Value	8.76					0.026			0.0061
Range of Detected Values	0.724-14.1					0.0162-0.029			0.0061
Rice County Landfill									
Average Value	7.71				0.017	0.0401	<0.02		0.003
Range of Detected Values	7.35-8.95				0.016-1.019	0.0155-0.0881	<0.02		0.002-0.004
Minnesota Range of Detected Values	0.724-14.1				0.016-0.019	0.005-0.029	<.02		0.001-0.004
HAWAII									
South Hilo Sanitary Landfill									
Average Value					0.01	0.25			0.28
Range of Detected Values					0.00912-0.018	0.227-0.27			0.13-0.55
Other Landfills SWANA									133

Summary of Landfill Leachate Data

South Hilo Sanitary Landfill

Feasibility Study 2011

Sample Description	Lead (mg/L)	Magnesium (mg/L)	Manganese (mg/L)	Mercury (mg/L)	Molybdenum (mg/L)	Nickel (mg/L)	Potassium (mg/L)	Selenium (mg/L)	Silver (mg/L)
OREGON									
Agate Beach Landfill									
Average Value	<0.1	27.9	5.25	<0.0005	<0.05	0.00112	63.9	<0.25	0.00062
Range of Detected Values	<0.1	17.3-63	2-6.8	<0.0005	<0.005	0.00109-0.00115	20.3-120	<0.25	0.0059-0.0065
Bandon Landfill									
Range of Detected Values		57	0.8			<0.04	29	<0.005	<0.01
Boise Cascade									
Average Value	<0.0002	180	0.915	<0.00002		0.0293	28.35	<0.005	<0.001
Range of Detected Values	<0.0002	80.1-280	0.76-1.07	<0.0002		0.0086-0.05	17.7-39	<0.005	<0.001
GP Wauna Mill Landfill									
Average Value	<0.0003	74	0.9663	0.00005		0.0115	213	0.0045	0.0003
Range of Detected Values	<0.0003	44.6-89.8	0.933-0.999	0.000022-0.000059		0.0044-0.0151	119-265	0.0041-0.0048	0.0003
Tillamook Landfill									
Average Value	<0.005	17.7	0.98	<0.0005	<0.05	0.00144	29.9	<0.25	<0.01
Range of Detected Values	<0.005	7.02-91	0.31-1.4	<0.0005	<0.05	0.00144	Nov-62	<0.25	<0.01
Reedsport Landfill	<0.0002	15.2	2.49			0.0022	13.2	<0.002	<0.001
Detected Values	<0.0002	152	2.49			0.0022	13.2	<0.002	<0.001
Waldport Landfill									
Average Value	<0.003	9.8	0.66	<0.0005		0.0009	41.1	<0.005	<0.01
Range of Detected Values	<0.003	7.86-14	0.751-1.01	<0.0005		0.00074-0.00113	9.5-130	<0.005	<0.01
Oregon Range of Detect Values	<0.005	4.6 - 280	0.1 - 6.8	0.00022 - 0.00059	<0.05	0.00109 - 0.022	11 - 265	0.0041 - 0.0048	0.00029 - 0.00065
MINNESOTA									
Lyon County Landfill									
Average Value				0.00002	0.02			6.34	
Range of Detected Values				0.00002	0.02			5.85-6.82	
Crow Wing County Landfill									
Average Value				<0.0002			465	0.0169	
Range of Detected Values				<0.0002			329-630	0.0148-0.0175	
Rice County Landfill									
Average Value				<0.00005			372		
Range of Detected Values				<0.00005			326-459		
Minnesota Range of Detected Values				<0.00001	0.02		326-518	0.0148-6.82	
HAWAII									
South Hilo Sanitary Landfill									
Average Value				0.00018	0.0005		38	0.0056	
Range of Detected Values				0.00014-0.000216	0.00012-0.0008		38	0.0003-0.0088	
Other Landfills SWANA									

Summary of Landfill Leachate Data

South Hilo Sanitary Landfill

Feasibility Study 2011

Sample Description	Sodium (mg/L)	Thallium (mg/L)	Vanadium (mg/L)	Zinc (mg/L)	Sulfate (mg/L)	Nitrate/nitrite as N (mg/L)	TKN (mg/L)	TDS (mg/L)	TSS (mg/L)
OREGON									
Agate Beach Landfill									
Average Value	219.2	<1	<0.03	0.0464	48.1	0.09		1042.8	42.6
Range of Detected Values	84.6-290	<1	<0.03	0.0114-0.1	7.6-120	0.03-0.37		84-2100	7-130
Bandon Landfill									
Range of Detected Values	120			<0.02		0.04		1800	110
Boise Cascade									
Average Value	668	<0.001	0.05	<0.02	10.9	0.0579		6200	89
Range of Detected Values	336-1000	<0.001	<0.02-0.05	<0.02	8-13.8	0.03-0.0858		6200	89
GP Wauna Mill Landfill									
Average Value	778	<0.0015	0.0436	0.0030	147	0.017	170	2933	33
Range of Detected Values	443-958	<0.0015	0.0432-0.044	0.00248-0.00361	129-157	0.0137-0.025	170	1800-3500	28-38
Tillamook Landfill									
Average Value	59.9	<1	<0.03	0.06	106.4	0.24	10.3	401	72
Range of Detected Values	34-120	<1	<0.03	0.00865-0.1	8.8-550	0.02-1.9	8.6-12	190-610	5-350
Reedsport Landfill	41.6	<0.0001	<0.004	<0.002	7.94	2.17		360	27
Detected Values	41.6	<0.0001	<0.0004	<0.0002	7.94	2.17		360	27
Waldport Landfill									
Average Value	117	<0.002	<0.03	0.109	3.27	0.0129		290	41
Range of Detected Values	41.6-320	<0.002	<0.03	0.02-0.231	0.2-8.03	0.0129		170-400	11-57
Oregon Range of Detect Values	28.8 - 1000	<1	0.04-0.05	0.00248-0.231	0.2 - 550	0.02 - 2.17	8.6 - 170	170 - 6200	5 - 350
MINNESOTA									
Lyon County Landfill									
Average Value		<0.01		0.68		0.36	96.4	2070	99
Range of Detected Values		<0.01		0.66-1.3		0.36	81.7-111	1570-2570	16-200
Crow Wing County Landfill									
Average Value				0.66	53.6	1.58		5420	162
Range of Detected Values				0.0011-1.88	19-106	0.48-4.05		3740-7210	39-258
Rice County Landfill									
Average Value				0.457	105	0.25		5277	
Range of Detected Values				0.156-1.02	88.4-115	0.25		4910-5990	
Minnesota Range of Detected Values		<0.01		0.0011-1.3	34.4-115	0.21-4.05	81.7-111	1570-7920	16-404
HAWAII									
South Hilo Sanitary Landfill									
Average Value		0.0074	0.0457	0.66				720	
Range of Detected Values		0.000181-0.02	0.0243-0.073	0.132-1.3				720	
Other Landfills SWANA									

Appendix F
STAKEHOLDER INVOLVEMENT PLAN

1. Map and Table of Land Use Data

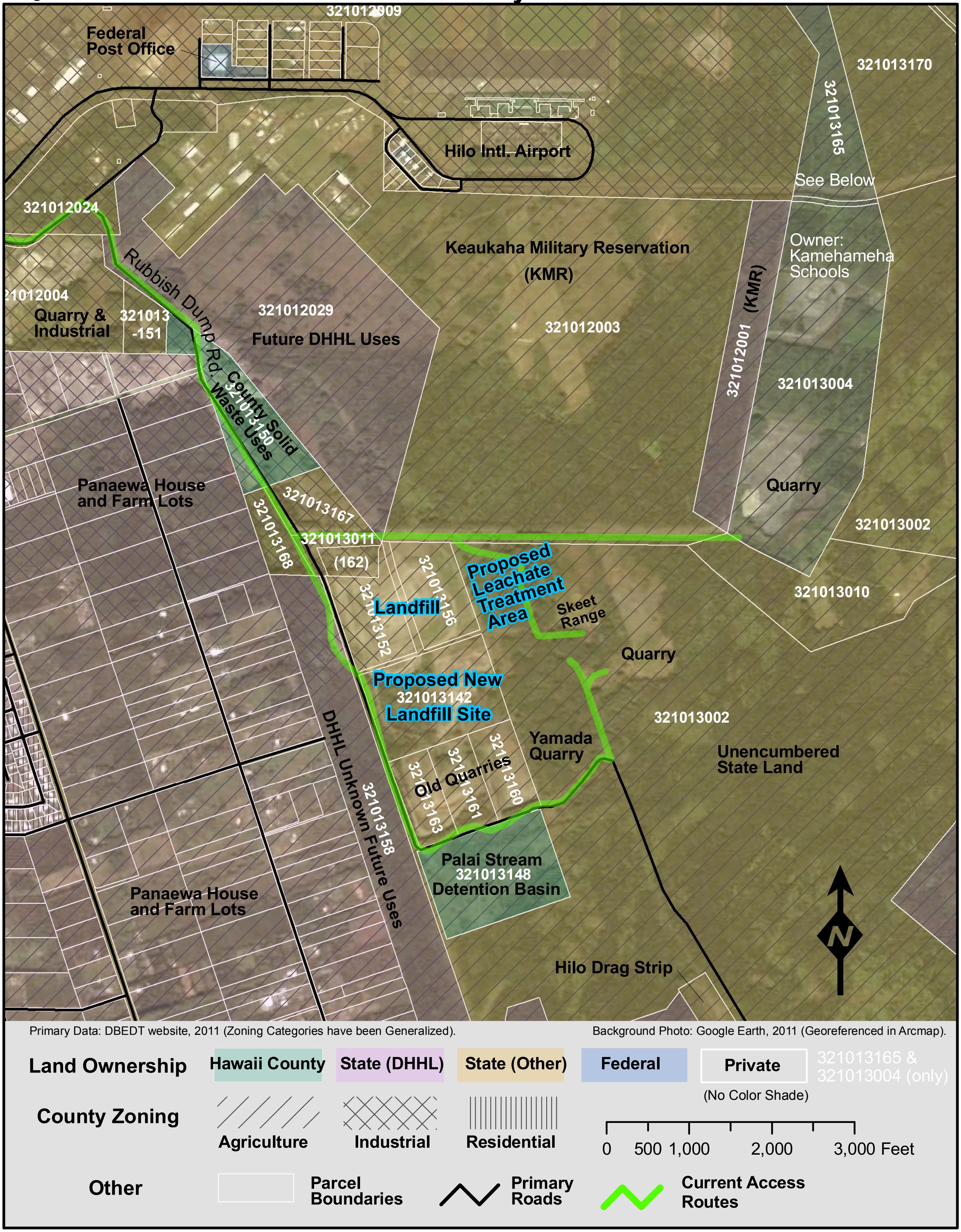
Figure 1 depicts properties in the general vicinity of the proposed landfill expansion and illustrates zoning, ownership and land use. Table 1, below, provides, detailed property information for immediately adjacent properties.

Table 1

TMK	Owner	Current Use	Tax Acres	Zoning
2-1-012-003	State	Keaukaha Mil. Res.	442.486	A20a
2-1-012-029	State DHHL	None	184.820	A20a
2-1-013-002	State	Various, Mostly Vacant	2407.756	A20-a
2-1-013-010	State	Keaukaha Mil. Res.	61.174	A20-a
*2-1-013-011	State	Various Solid Waste[A]	6.500	MG1-a
*2-1-013-142	State	Quarry	40.000	MG1-a
2-1-013-148	County EO	Flood Detention Basin	40.000	MG1-a
*2-1-013-150	County EO	Various Solid Waste	35.400	MG1-a
*2-1-013-152	County EO	Landfill	19.482	A20a
*2-1-013-156	County EO	Landfill	20.000	MG1-a/A20a
2-1-013-158	State DHHL	Various Solid Waste	95.392	A20a
2-1-013-160	State	Quarry	13.333	A20a
2-1-013-161	State	Quarry	13.333	A20a
*2-1-013-162	State	Landfill? Quarry[A]	6.000	A20a
2-1-013-163	State	Quarry	13.333	MG1-a
*2-1-013-167	State	Various SW[A]	13.860	A20a
*2-1-013-168	State	Various SW[A]	10.940	A20a

* Proposed along with various road ROW for DLNR Set Aside to County of Hawai‘i for Solid Waste and Road and Utility Purposes, per BLNR Action 04HD-258 &259, approved by BLNR 9/24/04, with conditions (pending fulfillment).
[A] Additional discussion needed with the County

Figure 1. **South Hilo Sanitary Landfill Area**



Stakeholder Involvement Effort Summary (Vers. 8/18/11)

Group/Individual	Attendees/Contact Info	Date/place	Concerns/Outcome
Hawai'i Island DOH Staff	Aaron Ueno and Newton Inouye Aaron.Ueno@doh.hawaii.gov	Meeting 4/20/11, Hawai'i DOH office, Hilo	Mosquitos, permits for leachate, increasing solid waste diversion, implementation status of IRSWMP
Hawai'i Island DOFAW Staff	Roger Imoto, Joey Mello and Ron Bachman Roger.H.Imoto@Hawaii.gov	Meeting 4/20/11, Hawai'i DOFAW office, Hilo	Wildlife attractant: benefits and problems, wild pigs, inventory of existing vegetation
Hawai'i Island USFWS Staff	Donna Ball Donna_L_Ball@fws.gov	Meeting 4/21/11, USFWS office, Hilo	Invasives, wildlife attractant issue, water quality
Hawai'i Island Division of Aquatic Resources Staff	Dr. Bob Nishimoto Robert.T.Nishimoto@hawaii.gov 974-6201	Interviewed by phone on 5/12	Agreed with need, did not know of pollution but concerned about lack of recycling ease
Keaukaha Hawaiian Home Lands Community Association	Patrick Kahawaiola'a 959-5080 elama@hawaiiantel.net ~ 25 attendees	Presented to Board on 8/17/11	Why is County keeping existing landfill open despite water quality effects; alternative sites need to be examined; revolving door of waste solutions; community benefits
Panaewa Hawaiian Home Lands Community Association	Kelly Lincoln: 987-9266 Donnalyn Johns johnsd@hawaii.edu ~ 20 attendees	Presented at general membership meeting on 7/19/11.	Concerned with air quality. Waste to energy should be considered. Confused by County's shifting plans.
Panaewa Community Development Corporation	Donnalyn Johns johnsd@hawaii.edu Attended by 6 board members	Presented to Board on 6/15/11	Continuing dialogue wanted; community benefits; commitment to reduced waste by County
Keaukaha-Panaewa Farmers Association	Mele Spencer muspencer@hawaii.rr.com Attended by 8 board members	Presented to Board on 7/9/11	Air quality (odor and other issues); water quality are issues. Community has many health problems.
Recycle Hawai'i	Paul Buklarewicz 987-8294 895-0315 961-2676 (RH) pjb01@hawaii.rr.com	Interviewed by phone on 5/12	Concerns about misplaced priorities, loss of focus on recycling, and the waste of resources that a large landfill may promote
Sierra Club, Moku Loa Group	Debbie Ward dward@hawaii.edu Attended by six board members	Presented to Executive Committee on 7/10.	Questioned building a new landfill when efforts to divert reuseables, recyclables and potentially useable organics in IRSWMP are not being implemented
Kanoelehua Industrial Area Association	Caleb Yamanaka lubs2003@gmail.com	Interviewed in-person 5/17/11	Biz opportunities
Hilo Bay Watershed Advisory Council	Susan O'Neill susandan@aloha.net		Group is apparently no longer active; phone calls and emails not returned.

Hawaiʻi County Planning Department	Bobby-Jean Leithead-Todd, Director bjltodd@co.hawaii.hi.us	Interviewed in person 5/18/11	Advisable to amend the State Land Use District boundary for the entire area of landfill operations to the Urban District. Mix of traffic in area is concern.
Hawaiʻi County Department of Public Works	Ben Ishii, Engineering Division Director bishii@co.hawaii.hi.us	Interviewed in person 6/30/11, Hawaiʻi DPW office, Hilo	No real issues foreseen
Hawaiʻi County Parks and Recreation	James Komata: 961-8411 936-0200 Clayton Honma	Interview in person on 6/28/11	Asked about drag racing and skeet range; no concerns that cannot be addressed through coordination
Hawaiʻi DLNR Land Division	Kevin Moore and Wesley Matsunaga kevin.e.moore@hawaii.gov	Interviewed 6/30/11, Hawaiʻi DLNR Land Div. office, Hilo	No major issues, but future access, quarry may be important
Hawaiʻi DHHL	Chairman Alapaki Nahale-a 769-2012, paki@alapaki.com	Interviewed by phone on 7/29/11	DHHL is extremely busy with governance; appreciates the contact and will try for longer interview; please keep Comm. Lee Loy informed; he will update on adjacent parcel use as plans develop
Hawaiʻi DHHL	East Hawaiʻi Commissioner Ian Lee Loy	Interviewed by Phone on 7/25/11	Wants to be kept in loop; has concerns about local impacts; wants community benefits; also serves on KP farmers.
Hawaiʻi Island Chamber of Commerce, Government Affairs Committee	Vaughn Cook, Chair vcook@hawaii.rr.com Attended by 7 board members	Presented to group on 7/6/11	Waste to energy should happen; wetlands can be an attraction and selling point.
U.S. Forest Service	Dr. Susan Cordell, 854-2628 and 987-4115	Interviewed by phone and in person in June and July	No concerns except for preserving forest similar to that just across the road at KMR. Provided map
U.S. Army National Guard, Keaukaha Military Reservation	Major Darren Cox, 844-8605	Interviewed by phone 7/12/11; also emails from Hawaiʻi Dept. of Defense	Pigs, DHHL land, access and use of parcel in back; severe concerns that it will interfere with KMR range safety zone.
DLNR Land Division	Former Land Agent Harry Yada	Interviewed by Phone on 7/6/11	Completion of County EO and Set Aside and solution of property and land use irregularities; Access to KS lands and DLNR land to east; Puainako Extension.

NEIL ABERCROMBIE
GOVERNOR

DARRYLL D. M. WONG
MAJOR GENERAL
ADJUTANT GENERAL

JOSEPH K. KIM
BRIGADIER GENERAL
DEPUTY ADJUTANT GENERAL

STATE OF HAWAII
DEPARTMENT OF DEFENSE
OFFICE OF THE ADJUTANT GENERAL
3949 DIAMOND HEAD ROAD
HONOLULU, HAWAII 96816-4495

July 24, 2011

NGHI-FMO

Mr. Ron Terry
City and County of Hawai'i

SUBJECT: Landfill Expansion

Mr.Terry,

I appreciate the opportunity to provide comments from my facilities staff concerning the Landfill Expansion planned adjacent to Keaukaha Military Reservation, Hilo Hawaii. The comments are as follows:

At the National Guard facility planning office, we view the proper collection and disposal of waste materials as a necessary component of a well-managed community, and we fully support the development of solid waste landfills to minimize the risks of burgeoning waste accumulation to public health. On the other hand, we proffer a guarded voice to this proposal. As the proposed expansion lies in close proximity to the National Guard training areas, our concern should not be construed as a "not-in-our-backyard" position, but an attempt to strike the right balance for compatible use of the landscape and airspace that we both share. Most likely, the National Guard will still be at Keaukaha Military Reservation for many years ahead and may outlive the landfill operation, for which reason we are concerned with the long-term impacts of the landfill expansion to current and future requirements for trainings and facility operations. Some of these concerns are as follows:

1. Limited range use due to increased public presence within designated surface danger zones (SDZs).

The area for the proposed expansion is located down-range of the National Guard's pistol and known-distance (KD1) rifle ranges. The associated SDZs for the types of weapon and munition authorized for live-training at these ranges extend beyond KMR's southern boundary and will probably impact the proposed landfill. In self-imposed restriction, the use of these ranges has been held in abeyance while the Hawaii National Guard is assessing ways to improve the range safety standards using terrain-based mitigation, structural controls, and SOPs for range operations. Even with these mitigation measures in place, the National Guard assumes a non-zero SDZ and endeavors to further enhance range safety by looking into minimizing public access to likely impact areas. In this regard, the proposed landfill expansion will work to the contrary and will unduly raise the safety bar for live-fire training as it consequently increase the traffic and industrial activities down-range. It will also tie-down the National Guard to high capital investment options, such as construction of range baffles, range re-alignment, and land acquisition to ensure that ricochet trajectories are confined within controllable range areas, and to shift the SDZs to areas that pose lesser public risks.

2. Deterioration of air quality due to elevated dust condition, fouling of air odor, and toxic gas emanations from the landfill

National Guard soldiers and reserve components, civilian workers, and range users continuous to experience and enjoy an outdoor air quality of the native Ohia forest in KMR. With increased vehicular traffic, frequency of off-loading and movement of loose dirt and solid wastes, and waste decomposition, this ambient air quality is bound to change and deteriorate. Soldiers doing their training and daily physical exercises will be most affected by prolonged exposure to this air quality.

2.1 What is the proponent's plan for odor neutralization and for mitigation of air quality deterioration and toxic emanation from the landfills?

2.2 Has the proponent conducted a study that shows a year-round air current dispersion model to help devise a strategy for deployment of air quality monitoring stations?

3. Triggering actions for environmental remediation due to airborne litters and dust fallout impact on endangered species and native ecosystems

The National Guard range area will be saddled with environmental remediation issues to clean-up the accumulated airborne litters and mitigate the impact of widespread dust fallout on KMR's population of endangered species, such as the Hawaiian Hawk and Hawaiian Hoary Bat. Over a period of time, this increase in airborne suspensions could jeopardize the survival of sensitive native ecosystems, and tip the balance of floral community towards proliferation of more sturdy invasive species. The impact to endangered species and native ecosystems will trigger remedial actions that require the revision of the Integrated Natural Resources Management Plan (INRMP) for KMR and a new round of Section 7 consultation with U.S. Fish and Wildlife. Essentially, we would like the proponent to address the following concerns:

3.1 What is the mitigation plan for airborne litters and dust control?
3.2 What is the remediation strategy for the removal or airborne litters and dust fallout on KMR?

3.3 What are the impacts of landfill expansion and existing landfill capping on threatened and endangered species (TES) and rare native species and habitats in KMR?

3.4 What is the strategy to combat the spread of invasive species and to mitigate the surges in population of predators, insects, and zoonotic organisms that are attracted to the landfill?

4. Off-site migration of flammable gas, landfill fires, and wildfire hazards

Studies have shown that combination of soil moisture, oxygen concentration, and leachate accumulation can be ignited by the heat generated in biodegradation process, resulting into a spontaneous combustion. This phenomenon is quite commonly associated with many solid waste landfills and poses a critical hazard that requires thorough monitoring. Off-site migration of combustible methane gas and abundance of potential fuel loads in the area heighten the probability for wildfire occurrence in KMR.

4.1 What is the proponent's plan for the prevention and suppression of landfill-triggered wildfires?

4.2 Does decommissioning of old landfill and operation of new landfills involve a system for monitoring of methane gas and its calibrated releases to the atmosphere?

5. Groundwater contamination and storm water pollution

The National Guard is looking into augmentation of its future water requirements by tapping into its own resources. There are two water wells in KMR that are registered with DLNR-CWRM as HIARNG wells, which were pump tested at 900 and 1,000 GPM of potable water. These wells penetrated a freshwater basal aquifer with a calculated thickness of about 160 feet. However, the hydraulic gradient for this aquifer slopes down from the landfill toward KMR, such that the aquifer is prone to contamination of leachates emanating from the landfill. Moreover, the landfill operation will eventually involve the diversion of surface runoffs and draining of storm water toward surrounding low-lying areas. KMR will be at the receiving end of the natural conveyance of surface runoffs and waste-laden storm water, particularly when a positive topography emerges on the landfill. Having the environmental and health impacts of contaminated groundwater and surface flows in mind, we are concerned whether the proponent plans to address and mitigate the following issues:

5.1 Waste pre-treatment and detoxification prior to burial at the landfill

5.2 Monitoring of leachate dispersion and leachate removal plan
5.3 Groundwater quality monitoring and contamination remediation

5.4 Planning and design for landfill storm drain and runoff conveyance system

Please address any questions or points of clarification to Mr. Ronnie Torres, GIS/Master Planning, NGHI-FMO-PP, (808) 844-6554.

Sincerely,

Marjean R. Stubbert
Lieutenant Colonel
Hawaii Army National Guard
Construction & Facilities
Management Officer

Appendix G
COST ESTIMATES

Table 1

Landfill Design and Construction Cost

2011 Dollars for Cell 1, 7-acre development

Item No.	Subitem	Item Description	Approximate Quantity	Unit	Estimated Unit Cost	Estimated Total Cost
1		**Mobilization (10% of)**	1	LS	$ 769,000.00	$ 769,000
2		**Cell Construction**				
	2.1	Grading - Excavation, processing and placement	125,000	CY	$ 40.00	$ 5,000,000
	2.2	Grading - Embankment	100,000	CY	$ -	$ -
	2.3	Processed Material - Subbase	25,000	CY	$ -	$ -
	2.4	Geosynthetic Clay Liner	370,000	SF	$ 0.88	$ 325,600
	2.5	HDPE Textured Geomembrane Liner	370,000	SF	$ 0.65	$ 240,500
	2.6	Geocomposite Drain-net	370,000	SF	$ 0.95	$ 351,500
	2.7	Processed Aggregate Drainage Layer	12,000	CY	$ 50.00	$ 600,000
	2.8	Leachate Collection Piping-Perforated & Non-perforated	900	LF	$ 100.00	$ 90,000
	2.9	1-1/2" Washed Aggregate	650	Ton	$ 50.00	$ 32,500
	2.10	3/8" Washed Pea Gravel	700	Ton	$ 38.00	$ 26,600
	2.11	Collection Sump and Pump	1	LS	$ 30,000.00	$ 30,000
	2.12	Leachate Collection Riser Pipes	330	LF	$ 250.00	$ 82,500
	2.13	Rain Coat	370,000	SF	$ 0.40	$ 148,000
3		**Leachate Management**				
	3.1	Leachate Collection Manhole 1	1	LS	$ 60,000.00	$ 60,000
	3.2	Gravity Piping to Leachate Treatment Facility	1,000	LF	$ 150.00	$ 150,000
	3.3	Leachate Storage Tank/Pond and Auxillaries	1	LS	$ 200,000.00	$ 200,000
	3.4	Leachate Loadout Facility	1	LS	$ 30,000.00	$ 30,000
4		**Miscellaneous Items**				
	4.1	Mobile Stormwater Pump	2	EA	$ 65,000.00	$ 130,000
	4.2	Protection Bollards, Fence and Other Safety Items	1	LS	$ 25,000.00	$ 25,000
	4.3	Surveying	1	LS	$ 35,000.00	$ 35,000
	4.4	Groundwater Monitoring Wells	3	EA	$ 15,000.00	$ 45,000
	4.5	Gas Monitoring Probes	2	EA	$ 15,000.00	$ 30,000
	4.6	Geotechnical Testing				
5		**Electric and Controls**	1	LS	$ 60,000.00	$ 60,000
6		**Site Upgrades - Not included in total costs**				
	6.1	Road Upgrades - Entrance Road, Perimeter Access Roads	1	LS	$ 100,000.00	$ 100,000
	6.2	Electrical Power Supply - Extend 3-Phase	3,520	LF	$ 40.00	$ 140,800
7		**Design - Plans and Specifications (2.5% of Construction Cost)**				**$ 211,530**
8		**Construction Management (5% of Construction Cost)**				**$ 423,060**
9		**Contingency (10% of Total Above Costs)**				**$ 870,200**
Total						**$ 9,970,000**

Table 2

Total Landfill Expansion Capacity Cost

2011 Landfill Expansion Costs per Ton for 40-acre Development

Cell 1 Airspace	848,800	cubic yards
AUF	1,200	lbs/cy
Cell 1 footprint	7	acre
Cell 1 Tons	509,280	tons
Total Landfill footprint	40	acres
Total Landfill Expansion Airspace Capacity (cubic yard)	5,400,000	cubic yards
Total Landfill Expansion Capacity Cost (tons)	3,240,000	tons

Item No.	Subitem	Item Description	Estimated Cost per Acre
1		**Mobilization**	$ 99,589
2		**Cell Construction**	
	2.1	Grading - Excavation	$ 714,286
	2.2	Grading - Fill	$ -
	2.3	Processed Material - Subbase	$ -
	2.4	Geosynthetic Clay Liner	$ 46,514
	2.5	HDPE Textured Geomembrane Liner	$ 34,357
	2.6	Geocomposite Drain-net	$ 50,214
	2.7	Processed Aggregate Drainage Layer	$ 84,507
	2.8	Leachate Collection Piping-Perforated & Non-perforated	
	2.9	1-1/2" Washed Aggregate	$ 4,643
	2.10	3/8" Washed Pea Gravel	$ 3,800
	2.11	Collection Sump and Pump	$ 2,143
	2.12	Leachate Collection Discharge Riser Pipes	$ 5,893
	2.13	Rain Coat	$ 21,143
3		**Leachate Management**	
	3.1	Leachate Collection Wetwell/Manhole 1	$ 4,286
	3.2	Gravity Piping to Leachate Treatment Facility (500 ft/2 Cells)	$ 5,357
4		**Miscellaneous Items**	
	4.1	Mobile Stormwater Pump (one additional)	$ 2,321
	4.2	Protection Bollards, Fence and Other Safety Items	$ 3,571
	4.3	Surveying	$ 5,000
	4.4	Groundwater Monitoring Wells	$ 2,143
	4.5	Gas Monitoring Probes	$ 1,429
5		**Electric and Controls**	$ 4,286
7		**Design (2.5% of Construction Cost)**	$ 27,387
8		**Construction Management (5% of Construction Cost)**	$ 54,774
9		**Contingency (10% of Construction Cost)**	$ 109,548
Total/ton			**$ 1,300,000**
Total - Construction Costs for Cells 2 through 8			**$ 42,900,000**

Table 3

Landfill Leachate Wetland Design and Construction Cost

2011 Dollars for Wetland Design and Construction

Item No.	Subitem	Item Description	Unit	Approximate Quantity	Estimated Unit Cost	Estimated Total Cost
1		**Cascade Aerator**				
	1.1	Earthwork	LS	1	$ 5,000.00	$ 5,000
	1.2	Manhole, Gaskets, Penetrations, Grating	EA	1	$ 5,000.00	$ 5,000
	1.3	Culvert Pipe	LF	30	$ 50.00	$ 1,500
2		**Sedimentation Ponds**				
	2.1	Earthwork	CY	400	$ 45.00	$ 18,000
	2.2	Geosynthetic Clay Liner	SF	8500	$ 0.88	$ 7,480
	2.3	Secondary Liner (60 mil HDPE with geotextile)	SF	8500	$ 1.60	$ 13,600
	2.4	Primary Liner (60 mil HDPE with geotextile)	SF	8500	$ 1.60	$ 13,600
	2.5	Yard Piping	LS	1	$ 10,000.00	$ 10,000
	2.6	Hydraulic Control Structure and Lift Station	LS	1	$ 75,000.00	$ 75,000
3		**Aerated Wetland**				
	3.1	Excavation	CY	1200	$ 35.00	$ 42,000
	3.2	Sand	CY	200	$ 40.00	$ 8,000
	3.3	GCL	SF	15000	$ 0.88	$ 13,200
	3.4	Geomembrane (60 mil)	SF	15000	$ 0.65	$ 9,750
	3.5	Geotextile	SF	15000	$ 0.50	$ 7,500
	3.6	Geomembrane (60 mil)	SF	15000	$ 0.65	$ 9,750
	3.7	Walls	LF	700	$ 10.00	$ 7,000
	3.8	Aggregate	CY	1000	$ 50.00	$ 50,000
	3.9	In-Bed Pipe	LF	1000	$ 5.00	$ 5,000
	3.10	Hydraulic Control Structure	EA	2	$ 12,000.00	$ 24,000
	3.11	Plants	EA	2000	$ 5.00	$ 10,000
	3.12	Water Balance Test	EA	2	$ 3,000.00	$ 6,000
	3.13	Blowers and Enclosures	EA	2	$ 10,000.00	$ 20,000
	3.14	Aeration Manifold	LF	200	$ 20.00	$ 4,000
	3.15	Aeration Tubing	LF	50000	$ 2.00	$ 100,000
	3.16	Panel	EA	1	$ 15,000.00	$ 15,000
	3.17	Electrical supply (Panel to Blower)	LS	1	$ 10,000.00	$ 10,000
4		**Design (10% of Total)**				$ 49,038
5		**Contingency (10% of Total)**				$ 49,038
Total						**$ 588,456**

Table 4

Landfill Gas Collection and Control System Design and Construction Cost

2011 Dollars for Full LFG Collection and Control System Build-out

Landfill Gas Management (GCCS)							
1	Vertical Well Installation	40	EA	$	12,000.00	$	480,000
2	Lateral Piping	10,150	LF	$	100.00	$	1,015,000
3	Header Piping	6,000	LF	$	250.00	$	1,500,000
4	Condensate Management	10	EA	$	50,000.00	$	500,000
5	Flare System	1	LS	$	300,000.00	$	300,000
6	Design, Construction, Contingency (25%)	1	LS			$	948,750
Landfill Gas Collection and Control System Total (rounded)						**$**	**4,700,000**

Table 5

Landfill Closure Design and Construction Cost

2011 Dollars for Expansion Area Closure

Item No.	Subitem	Item Description	Approximate Quantity	Unit	Estimated Unit Cost	Estimated Total Cost
1		**Mobilization (10% of Construction Costs)**	1	LS	$ 1,308,000	$ 1,308,000
2		**Cover Construction**				
	2.1	Grading	1	LS	$ 150,000.00	$ 150,000
	2.2	Intermediate Soil Borrow	68,148	CY	$ 40.00	$ 2,725,926
	2.3	Geocomposite Drain-net	1,840,000	SF	$ 0.95	$ 1,748,000
	2.4	40-mil LLDPE Membrane	1,840,000	SF	$ 0.65	$ 1,196,000
	2.5	Geocomposite Drain-net	1,840,000	SF	$ 0.95	$ 1,748,000
	2.6	18-inch Soil Layer	102,222	CY	$ 40.00	$ 4,088,889
	2.7	6" Topsoil	34,074	CY	$ 5.00	$ 170,370
	2.8	Seeding	40	Acre	$ 20,000.00	$ 800,000
	2.9	Erosion Control	30	Acre	$ 8,000.00	$ 240,000
	2.10	Berm Construction	1	LS	$ 50,000.00	$ 50,000
3		**Miscellaneous Items**				
	3.1	Surveying	1	LS	$ 100,000.00	$ 100,000
4		**Electric and Controls**	1	LS	$ 60,000.00	$ 60,000
5		**Closure Design (10% of Construction Cost)**				**$ 1,438,519**
6		**Closure Construction Management (5% of Construction Cost)**				**$ 719,259**
7		**Closure Contingency (10% of Construction Cost)**				**$ 1,438,519**
Landfill Closure and Construction Total (rounded)						**$ 18,000,000**

Table 6

Landfill Annual Operating Cost

2011 Dollars for Annual Operations

Item No.	Item Description	Unit	Quantity	Unit Cost ($)	Estimated Total Cost
1	**Labor**				
1.1	Solid Waste Oper. Superintendent (1 @ 50%)	hour	1,040	$ 33.65	$ 35,000
1.2	Equipment Operator (3 FTE)	hour	6,240	$ 20.87	$ 131,000
1.3	Landfill Attendant (3)	hour	6,240	$ 17.28	$ 108,000
1.4	Landfill Technician (1)	hour	2,080	$ 20.87	$ 44,000
1.5	Overhead Costs	percent	40%		$ 109,600
					$ 427,600
2	**Equipment Operating and Maintenance**				
2.1	Dozer (Cat D7)	hour	2,080	$ 65.21	$ 136,000
2.2	Compactor (Cat 816B)(2)	hour	4,160	$ 65.21	$ 272,000
2.3	45-ton Dump Truck (Deere 400D)	hour	1,800	$ 71.73	$ 130,000
2.4	Front End Loader (Cat 966F)	hour	2,080	$ 40.43	$ 85,000
2.5	Water Truck	mile	2,500	$ 5.22	$ 14,000
2.6	Small Loader (IT 28)	hour	1,200	$ 26.08	$ 32,000
2.7	Utility Trucks	mile	15,000	$ 0.59	$ 9,000
2.8	Maintenance and Operating Costs	percent	5%		$ 33,900
					$ 711,900
3	**Environmental Monitoring Costs**				
3.1	Groundwater Sampling and Analysis	sample	30	$ 3,521	$ 106,000
3.2	Leachate Sampling and Analysis	sample	4	$ 9,129	$ 37,000
3.3	Landfill Gas Sampling	event	4	$ 3,260	$ 14,000
3.4	Annual Reporting	each	1	$ 97,810	$ 98,000
3.5	Air permitting requirement	each	1	$ 60,000	$ 60,000
3.6	Greenhouse Gas reporting	each	1	$ 15,000	$ 15,000
					$ 330,000
4	**Miscellaneous Cost**				
4.1	Miscellaneous Site and Road Upgrades	lump sum			$ 80,000
4.2	Soil Excavation and Processing	cubic yard	38,000	$ 20.00	$ 760,000
4.3	Office Supplies	lump sum			$ 10,000
4.4	Staff Training and PPE	lump sum			$ 15,000
4.5	Engineering and Legal Fees	lump sum			$ 130,000
4.6	Litter Pickup	lump sum			$ 10,000
4.7	Utilities (at full expansion)	lump sum			$ 100,000
4.8	Pipe Cleaning	each			$ 20,000
4.9	Leachate Treatment Facility Maintenance	lump sum			$ 40,000
4.10	GCCS Monitoring/Maintenance (once installed)	lump sum			$ 40,000
4.11	Raincoat/Stormwater Management	lump sum			$ 100,000
4.12	Financial Assurance				$ 240,000
4.13	Contingency	percent	5%		$ 48,775
					$ 1,593,775
Subtotal - Less Financial Assurance (rounded)					**$ 2,860,000**
Landfill Closure					
1	Groundwater Monitoring				$ 35,000
2	Gas Monitoring				$ 10,000
3	GCCS Monitoring and Mainentance				$ 35,000
4	Annual Reporting				$ 25,000
5	Utilities				$ 96,360
6	Miscellaneous Work/Repairs				$ 35,000
Subtotal (rounded)					**$ 240,000**

Table 7A

Cost Summary

Costs for Landfill permitting, design, construction and leachate treatment through constructed wetland

Costs are in 2011 dollars

Item	Cost		Comments
Landfill Siting and Construction - Capital Costs			
Land Uses, EA, Zoning	$	1,000,000	Estimates provided from Geometricians and include cost estimates from previous site studies from the County
Site Upgrades	$	250,000	Upgrades for onsite infrastructure outside of landfill footprint: roads, electrical, storm ponds
Permitting	$	730,000	Estimate from similar permitting work and includes DOH solid waste permit, NPDES permit, Title V air permit
Cell 1 Design and Construction	$	9,970,000	See details in Table 1
Subsequent Construction	$	42,900,000	See details in Table 2
Wastewater Treatment Plant	$	17,000,000	Previous estimate from Brown and Caldwell (2003), escalated 3% per year
Wetland	$	590,000	Table 3 - Design and construction costs provided by EPI and Naturally Wallace (for first phase only)
GCCS - design and construction	$	4,700,000	See details in Table 4
Landfill Closure	$	18,000,000	See details in Table 5
Capital Costs Subtotal	**$**	**95,140,000**	
Annual Costs			
Landfill Operations	$	2,860,000	See details in Table 6 - Annual operating costs during landfill active life (less P-CC financial assurance)
WWTP annual standby fee	$	325,000	Estimate for capacity reservation at WWTP at $5 per ton and 65,000 tons for 2011
Post-closure Care	$	240,000	See details in Table 6 - Annual operating costs during landfill post closure period (included as financial assurance)
Annual Costs Subtotal	**$**	**3,425,000**	
Total Capacity (tons)		3,240,000	Approximate airspace for 40 acres is 5,400,000 @ 1,200 lbs/cy
Total Life Expectancy-8 cells (years)		30	
2011 Capital Costs ($/ton)	$	29.36	Cost per ton of Capital Cost subtotal over the total estimated tonnage capacity
2011 Annual Operating Cost ($/ton)	$	52.69	Cost per ton of the Annual Cost subtotal for 2011 tons of 65,000 tons
Total Cost	**$**	**82.06**	
Total Cost Range -15%/+30% ($/ton)	**$**	**69.75**	
	$	**106.67**	

Table 7B

Cost Summary

Costs for Landfill permitting, design, construction and leachate treatment at WWTP

Costs are in 2011 dollars

Item		Cost	Comments
Landfill Siting and Construction - Capital Costs			
Land Uses, EA, Zoning	$	1,000,000	Estimates provided from Geometricians and include cost estimates from previous site studies from the County
Site Upgrades	$	250,000	Upgrades for onsite infrastructure outside of landfill footprint: roads, electrical, storm ponds
Permitting	$	730,000	Estimate from similar permitting work and includes DOH solid waste permit, NPDES permit, Title V air permit
Cell 1 Design and Construction	$	9,970,000	See details in Table 1
Subsequent Construction	$	42,900,000	See details in Table 2
Wastewater Treatment Plant	$	17,000,000	previous estimate from Brown and Caldwell (2003), escalated 3% per year
Pipeline from landfill to WWTP	$	3,250,000	Estimate for 12-inch water line from landfill to WWTP at $250 per foot and 13,000 feet
GCCS - design and construction	$	4,700,000	See details in Table 4
Landfill Closure	$	18,000,000	See details in Table 5
Capital Costs Subtotal	**$**	**97,800,000**	
Annual Costs			
Landfill Operations	$	2,860,000	See details in Table 6 - Annual operating costs during landfill active life (less P-CC financial assurance)
WWTP annual fee	$	1,460,000	previous estimate from Brown and Caldwell (2003), at $0.10/gallon and 40,000 gallons per day
Post-closure Care	$	240,000	See details in Table 6 - Annual operating costs during landfill post closure period (included as financial assurance)
Annual Costs Subtotal	**$**	**4,560,000**	
Total Capacity (tons)		3,240,000	Approximate airspace for 40 acres is 5,400,000 @ 1,200 lbs/cy
Total Life Expectancy-8 cells (years)		30	
2011 Capital Costs ($/ton)	$	30.19	Cost per ton of Capital Cost subtotal over the total estimated tonnage capacity
2011 Annual Operating Cost ($/ton)	$	70.15	Cost per ton of the Annual Cost subtotal for 2011 tons of 65,000 tons
Total Cost	**$**	**100.34**	
Total Cost Range -15%/+30% ($/ton)	**$**	**85.29**	
	$	**130.44**	

Appendix H
WASTE HAULING ESTIMATE

TABLE 1
TRANSFER TRAILER TRIP CYCLE TIMES AND NUMBER OF POSSIBLE TRAILER TRIPS PER DAY

ACTIVITY	Duration, Minutes	West Sanitary Landfill
Once Per Day Activities		
Morning Preparation Time	15	
Break Periods	30	
Night Return/Refuel	15	
Subtotal		60
Average Per Load Activities		
Load		20
Highway Preparation		2
Transit to/from West Sanitary Landfill (1)		234
Unload		20
Total Duration Per Trip, Minutes		276
Possible Number of Trips Per Day (2)		1.96
Optimize Trips to Best Whole Number (2)		2
Minutes Left Over/-Short		-11
Actual Work Day Length in Hours		10.18

(1) Assume approximately 156 R/T miles at an average speed of 40 MPH
(2) Assume typical work day to be 10 hrs long (600 minutes) including lunch break; punch in to punch out
(2) Manually adjust number of trips so that actual work day length is between 8.5 and 10.5 hours
If greater than 10.5 hours reduce to next lowest number of trips

TABLE 2

TRANSFER TRAILER LOADS REQUIRED; TRUCKS, DRIVERS AND TRAILERS REQUIRED; AND ANNUAL EQUIPMENT MILEAGE

1	2	3	4	5	6	7	8	9	10	11	12	13
	Description	**R/T Distance (in miles)**	**Transfer Station Tons Per Day (6 days/week)**	**Ave. Trailer Payload (tons)**	**No. of Loads Required Per Day**	**No. of Trips per Day per Trailer (1)**	**Minimum No. of Tractors, Drivers, Trailers Required for No. of Loads**	**Estimated Total No. of Tractors Required (2)**	**Estimated No. of Trailers Required (3)**	**Total Equipment Miles per Year**	**Ave. Tractor Miles per Unit per Year (4)**	**Ave. Trailer Miles per Unit per Year (4)**
	205 tpd;20 ton pay load; WSLF	156	205	20	10	2	5	7	7	486720	70000	70000

Notes:
(1) See Table 2, trips & Trailers worksheet for calculation of best possible number of trips per day
(2) Assume 1 spare for every 3 tractors, round to nearest whole number
(3) Assume 1 spare for every 3 trailers, round to nearest whole number
(4) Assume that all units including spares are used equally

TABLE 3

HILO TO WEST SANITARY LANDFILL
HAULING OPERATIONS ESTIMATE SUMMARY
HAUL TO WSLF, 205 TPD, 6 DAYS/WEEK, WITH 20 TON AVERAGE PAYLOAD

			Cost	Percentage
I.	LABOR	$	378,000	29.0%
II.	FUEL AND OIL	$	335,000	25.7%
III.	TIRES	$	132,000	10.1%
IV.	MAINTENANCE AND REPAIRS	$	92,000	7.1%
V.	EQUIPMENT LEASE COST	$	245,000	18.8%
VI.	INSURANCE, LICENSE AND TAXES	$	7,000	0.5%
CONTINGENCY (10% on non-lease costs)			$94,000	7.2%
ACCOUNTING, SUPPLIES, MISC. (2% on non-lease costs)			$19,000	1.5%
TOTAL ANNUAL HAUL COST		$	1,302,000	100.0%
ANNUAL UNIT HAULING COST PER TON FOR 64,000 TONS			$20.34	
COST PER MILE (NIC DISPOSAL)			$2.68	
COST PER TON-MILE			$0.134	

TABLE 4

HILO TO WEST HAWAI`I SANITARY LANDFILL
HAUL OPERATIONS ESTIMATE

I. LABOR

Job Classification	Driver FTEs (1)	/person/year (2)	Annual Cost	Total
Drivers	6	63000	$ 378,000.00	
Dispatcher	0	0	$ -	
Subtotal I				$ 378,000.00

Notes:
(1) Number of driver full time equivalents required per Table 1 inflated for 21.1% non-productive time, round to whole numbe
(2) Includes full fringe benefits

II. FUEL AND OILS

Item	Annual Miles	Rate	Unit	Unit Price (1)	Annual Cost	Total
Fuels and Oils	486,720	4.5	mpg	$3.10	$335,296	
Subtotal II						$335,296

Notes:
(1) Cost of fuel 2011 at Hilo

III. TIRES

Item	No. Tires/Veh	No. Veh	Quantity (1)	Unit	Unit Price	Annual Cost	Total
Tractor Tires	10	7	163	Tire	$450	$73,350	
Trailer Tires	8	7	131	Tire	$450	$58,950	
Subtotal III							$ 132,300

Notes:
(1) Assume new tires used as replacements every 30,000 miles
Each tractor needs 2.33 sets of tires/year; each trailer needs 2.33 sets of tires per year

IV. MAINTENANCE AND REPAIRS

Item	No. Veh	Quantity	Unit	Unit Price	Annual Cost	Total
Tractors	7	12	Mo	$600	$ 50,400	
Trailers	7	12	Mo	$500	$ 42,000	
Subtotal IV						$ 92,400

Notes:

V. EQUIPMENT LEASE

Item	Unit Cost (1)	No. Veh	Total Cost

Tractors	$24,000	7	$168,000	
Trailers	$11,000	7	$77,000	
Subtotal V				$ 245,000

Notes:
(1) Per County

VI. INSURANCE LICENSE AND TAXES

Item	No. Veh	Unit	Unit Price	Annual Cost	Total
Insurance	7	EA	$0	$ -	
License and Taxes	14	EA	$500	$ 7,000.00	
Overweight Permits	0	EA	$0	$ -	
Subtotal VI					$ 7,000.00

Notes:

Subtotal I through VI	$ 1,189,996.00