ATTACHMENT P-3.2
FACILITY DESIGN (ENGINEERING REPORT)

ENGINEERING REPORT
KEKAHA SANITARY LANDFILL PHASE II, CELL 2 LATERAL EXPANSION
KEKAHA, KAUA‘I, HAWAI‘I

County of Kaua‘i
Department of Public Works
4444 Rice Street, Mo‘ikeha Building, Suite 275
Lihu‘e, Hawai‘i 96766

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Prepared for

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Department of Public Works
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EXECUTIVE SUMMARY

The County of Kaua'i (County) currently owns and operates a landfill facility located near the town of Kekaha in the coastal area on the southwest side of the island of Kaua'i. The facility currently has two landfill phases, Phase I and Phase II. Phase II is the active, permitted landfill. The previous expansion of the Phase II landfill, designated Cell 1, consists of a waste disposal area of approximately 6.4 acres located at the west end of Phase II. The latest approved expansion is the Phase II vertical expansion which increased the maximum final cover elevation to 120 feet. Expansion Cell 1 is now full and Phase II is the active portion of the Kekaha landfill. The County is proposing a second horizontal (lateral) expansion of the Phase II landfill to obtain additional airspace while a future landfill is designed, permitted, and constructed.

The proposed expansion, designated Cell 2, will include the construction of an additional engineered waste disposal area. Cell 2 is to be located immediately adjacent to the south side of the existing Phase II, and will overlay both the north face of the existing Phase I and the south face of the existing Phase II. Construction of Cell 2 will provide approximately 6.6 acres of additional landfill area and create a continuous Subtitle D base liner system that underlies all of the Phase II landfill, including Cell 1 and Cell 2. The Cell 2 expansion will include modifications to the drainage features, the leachate collection system, and the landfill gas extraction system.

Cell 2 will provide an estimated 972,500 cubic yards of gross airspace, based on the proposed expansion limits and a final cover elevation of 120 feet. Based on current landfill waste mass density and daily waste disposal rates, expansion of Cell 2 may allow the facility to continue managing the County's waste stream for approximately 7.7 additional years.

Although the proposed expansion provides the needed airspace to operate the facility, current site operating systems, such as leachate management, surface water management, and landfill gas extraction systems will need further enhancement. Currently, the leachate extracted from Phase II and Cell 1 is collected and managed in an evaporation pond north of Phase II (which was relocated and constructed as part of the Cell 1 expansion). Leachate from Phase II and Cell 1 is collected through a passive gravity design and discharged through pumps and piping to the pond. The proposed leachate extraction and collection system for Cell 2 will utilize sumps, pumps and controls, and will discharge via a forcemain to the existing evaporation pond. The evaporation pond sizing was verified and will accommodate additional leachate generated from Cell 2.

Surface water features have been evaluated for new flow characteristics and required design modifications will be incorporated. Presently, collected surface water discharges to an infiltration ditch located between Phases I and II, as well as to infiltration ditches located around the perimeter of Phase II and a surface water infiltration basin located north of Phase II. The location and configuration of Cell 2 will result in an increase in final cover area and a decrease in the area of the infiltration ditches. The remaining infiltration ditches and existing infiltration basin will continue to successfully manage design surface water flows.

In March 1996, the facility became subject to 40 CFR 60 Subpart WWW, the Municipal Solid Waste Landfill, New Source Performance Standards (NSPS) for air emissions. On June 5, 1996, the County submitted a report demonstrating that the facility did not exceed the 2.5 million megagrams (Mg) or (2.5 million cubic meters) design capacity requirement that would establish applicability of the standard. With the Phase II vertical expansion, the design capacity of the facility exceeded the 2.5 million Mg criteria and the 2.5 million cubic meters criteria. A gas extraction system is currently required pursuant to the NSPS. The system design accounted for the addition of the proposed Cell 2 waste mass and was constructed on both the Phase I and Phase II landfills. Extracted gas is burned at a flare, which is permitted under a Covered Source Permit (0802-01-C) by DOH. The existing gas extraction system will be extended into Cell 2 as discussed in Section 6.0.

The proposed Cell 2 design was evaluated for final buildout stability. The analysis calculated factors of safety greater than 1.5 for static conditions, indicating that the proposed design is stable.

Drawings depicting the facility geometry and design features accompany this report. The design proposed and evaluated in this report demonstrates that the Cell 2 lateral expansion of Phase II is feasible and in conformance with regulatory requirements.

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ACRONYMS AND ABBREVIATIONS

§	Section
°	degree
AECOM	AECOM Technical Services, Inc.
CFR	Code of Federal Regulations
cm/sec	centimeters per second
County	County of Kaua'i
CWA	Clean Water Act
CWB	Clean Water Branch, State of Hawaii
cy	cubic yards
DOH	Department of Health, State of Hawai'i
EPA	Environmental Protection Agency, United States
FML	flexible membrane liner
ft	feet/foot
g	gravitational pull
GCCS	gas collection and control system
GCL	geosynthetic clay liner
gpd	gallons per day
HAR	Hawai'i Administrative Rules
HDPE	high-density polyethylene
HELP	Hydraulic Evaluation of Landfill Performance
HLA	Harding Lawson Associates
in/yr	inches per year
kPa	kilopascal
KSLF	Kekaha Sanitary Landfill
LFG	landfill gas
Mg	megagrams
Mg/yr	megagrams per year
msl	mean sea level
MSWLF	municipal solid waste landfill
NMOC	nonmethane organic compound
NPDES	National Pollutant Discharge Elimination System
NSPS	New Source Performance Standards
oz/syd	ounce per square yard
pcy	per cubic yard
PGE	Pacific Geotechnical Engineers, Inc.
POCA	point of compliance assessment
ppm	part per million
psf	pounds per square foot
PVC	polyvinyl chloride
RCRA	Resource Conservation and Recovery Act
SDR	standard dimension ratio
SHPD	State Historic Preservation Division
ZOI	zone of influence

1.0 INTRODUCTION

The County of Kaua'i (County) proposes to laterally expand Phase II at the Kekaha Sanitary Landfill (KSLF) facility to provide additional air space volume for placement of refuse while the siting, designing, and construction phases for a new landfill facility are completed. The proposed lateral expansion, designated Cell 2, would extend the permitted footprint of the existing Phase II to the south by removing the infiltration ditch and overlaying onto the sideslopes of both Phase I and Phase II. The proposed Cell 2 lateral expansion will result in a continuous Resource Conservation and Recovery Act (RCRA) Subtitle D base liner, constructed as an extension of the existing Phase II and Cell 1 base liner systems, with a maximum finished elevation of 120 feet (ft) above mean sea level (msl) as shown in the attached Drawings.

1.1 Site Background

The KSLF facility (consisting of Phases I and II) is located in a coastal area on the southwest side of the island of Kaua'i as illustrated on Figure 1-1. The KSLF is located at mile marker 28 of the Kaumuali'i Highway (Highway 50) about 2 miles west of Kekaha. Phases I and II are currently noncontiguous landfills at the facility. Drawing No. 1 illustrates the general features of the existing landfill facilities, which will remain.

1.2 Phase I

Phase I occupies approximately 33 acres and varies in elevation from 10 to 50 ft msl. The County opened the Phase I landfill in 1953 and accepted solid wastes at the facility until operations ceased on October 8, 1993. The Phase I landfill has no base liner system beneath the refuse and no leachate collection system.

The final cover for the Phase I landfill was designed and constructed in accordance with the following environmental rules and regulations in effect at the time of construction:

- United States Environmental Protection Agency (EPA) *Solid Waste Disposal Facility Criteria*, under the RCRA Subtitle D, published as 40 Code of Federal Regulations (CFR) Part 258 on October 9, 1991.
- State of Hawai'i Department of Health (DOH) *Solid Waste Management Control and Interim Guidelines for Landfill Closure*, Draft, Hawai'i Administrative Rule (HAR) Title 11, Chapter 58 (DOH 1994).

1.3 Phase II

Phase II of the KSLF facility, constructed in 1993 to meet RCRA Subtitle D criteria, is situated immediately to the northeast of Phase I and is currently an active municipal solid waste landfill (MSWLF). Phase II was initially permitted to a maximum elevation of 37 ft msl. To accommodate waste generated by Hurricane Iniki in 1992, a vertical expansion was proposed and approved in 1998 to raise the maximum fill elevation to 60 ft msl. In 2004, the County prepared, submitted, and gained approval of a vertical expansion to 85 ft msl. In 2009, Phase II was expanded laterally to the northwest with the approval of Cell 1. In 2013, the County obtained a permit for a vertical expansion of the Phase II landfill to a maximum elevation of 120 feet msl.

The Phase II landfill is bounded by Kaumuali'i Highway to the northeast, an unpaved access road and agriculture to the southeast, aquaculture to the northwest, and the Phase I landfill to the southwest. The Phase II landfill base liner elevations and leachate collection system vary from approximately 12 ft above msl to approximately 7 ft above msl and drains towards the northeast. The permitted Phase II landfill has maximum sideslopes of 3.5:1 (horizontal:vertical), will have a 3 percent (%) top slope. The Phase II liner area encompasses 32 acres and is subdivided into 14 lined subcells (each about 2.3 acres).

The existing Phase II landfill base liner system consists of (bottom to top):

- Granular subbase.
- Geosynthetic Clay Liner (GCL) consisting of a smooth 20-mil high-density polyethylene (HDPE) geomembrane with an adhered bentonite powder layer on one side (Gundseal). The GCL was installed with the smooth side in direct contact with the granular subbase and the bentonite component side up.
- Single-sided textured 60-mil HDPE geomembrane (textured side down) against the GCL on the sideslope.
- Smooth 60-mil HDPE geomembrane against the GCL on the base.
- Sand drainage layer (base and sideslope).

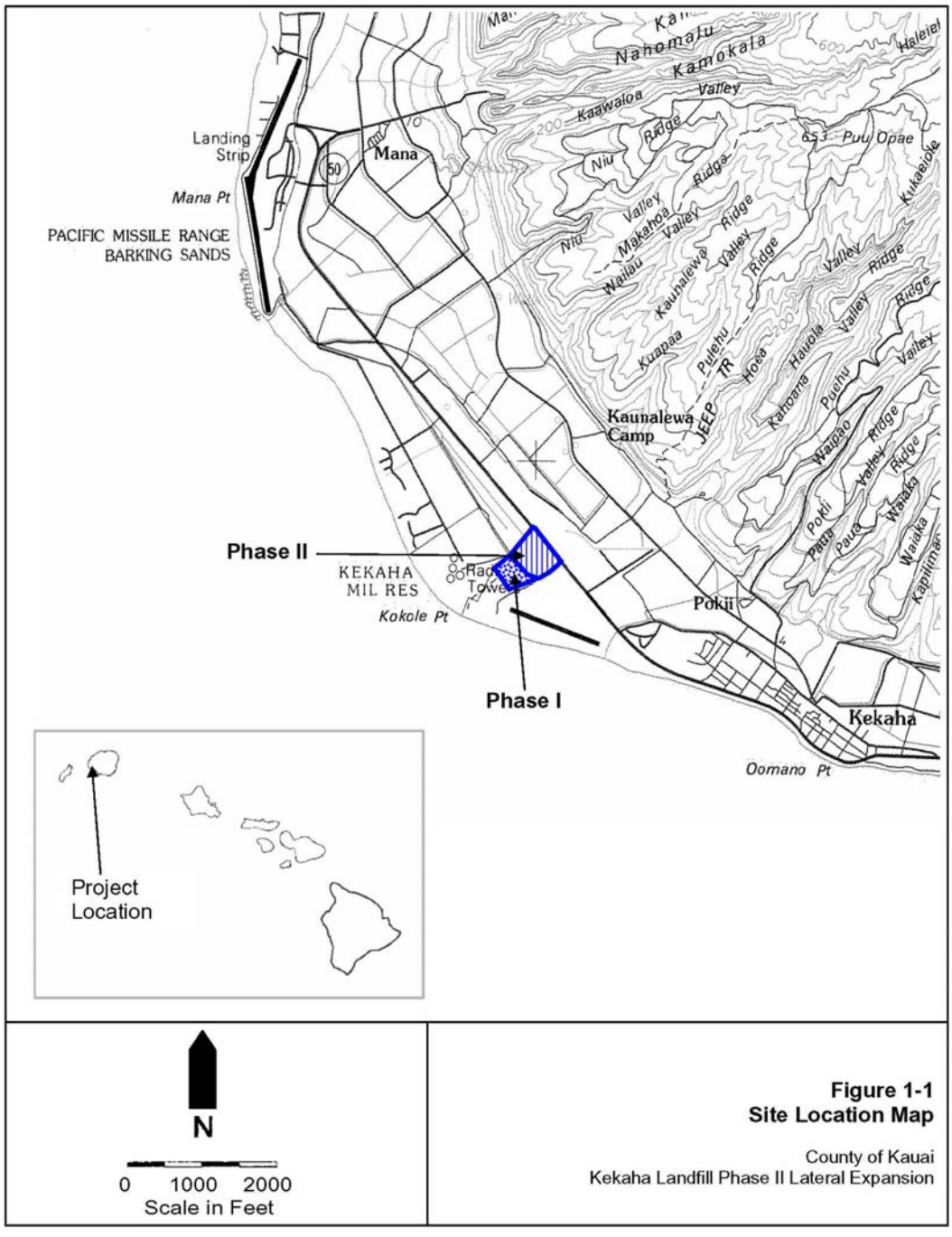
The existing base liner system for Cell 1 of Phase II consists of (bottom to top):

- Prepared subbase grade
- Subbase 60-mil HDPE geomembrane, textured on both sides
- GCL
- 60-mil HDPE geomembrane (upper component), textured on both sides
- Nonwoven cushion geotextile
- 12-inch granular drainage layer (leachate collection)
- Nonwoven separator geotextile
- 24-inch operations layer

Placement of refuse has been completed in Cell 1 of Phase II. Refuse placement is currently occurring in the Vertical Expansion of Phase II.

Phase II is currently operated by Waste Management of Hawaii, and is the only permitted MSWLF on the island of Kaua'i.

Figure 1-1: Site Location Map



2.0 SITE EXPLORATION

As part of the Cell 1 design, AECOM subcontracted Pacific Geotechnical Engineers, Inc. (PGE) to complete a site exploration and investigation for geotechnical properties of the existing soils near Phase II, the (then) existing leachate evaporation pond (where Cell 1 has since been constructed), the current surface water infiltration basin area, and the current leachate evaporation pond location. The site exploration also included a portion of the proposed Cell 2 expansion area.

The site exploration for Cell 1 was completed in late November and early December 2006. PGE completed a total of 11 soil borings and 7 test pits. Based upon the borings, test pits, and laboratory results, the predominant on-site foundation soils are poorly graded sands containing little fines. From a geotechnical standpoint, the project site was considered suitable for construction of the proposed expansion.

Fieldwork for a site exploration program for the Cell 2 design was completed in late November to December 2011 by PGE and included borings, test pits, and geotechnical laboratory testing. The exploration program evaluated surface and subsurface soils beneath the Phase II perimeter berm, the approximate centerline of the existing infiltration ditch between Phases I and II, and the Phase I and II sideslopes. Based on field observations and laboratory analyses as reported in Appendix A, the subsurface materials are similar to those encountered in the Cell 1 geotechnical investigation. Laboratory testing consisted of index and classification tests, direct shear tests, consolidation tests, permeability tests and moisture-density relationships. This data for the area within the footprint of Cell 2 has been applied to the geotechnical analyses in Appendix B.

The Geotechnical Exploration Report (PGE 2012, included as Appendix A) concluded the following regarding subsurface conditions along the perimeter berm of Phase II:

"*Subsurface conditions encountered in Borings B-1 and B-3 and Test Pit TP-8, performed on a gravel road along the west side of the Phase II landfill, generally consisted of fill material to depths of approximately 9 to 10 feet. The fill material is comprised of fat clay, basaltic gravel, and medium dense to very dense poorly graded coralline sand. Fill material was encountered to the bottom of Test Pit TP-8 at 9.5 feet. Below the fill in Borings B-1 and B-3, subsurface conditions generally consisted of beach and/or dune deposits of medium dense to very dense poorly graded coralline sand to the bottom of these borings at a depth of approximately 40 feet."*

The Geotechnical Exploration Report concluded the following regarding subsurface conditions for the infiltration ditch between Phase I and Phase II landfills:

"*Subsurface conditions encountered in Borings B-2, B-4, and B-5 and Test Pits TP-1,TP-3, TP-4, TP-5, TP-6, and TP-7, performed in the area between the Phase I and Phase II landfills, generally included fill material at the surface consisting of about 6 inches to 3 feet of brown, medium stiff to very stiff fat clay, and loose clayey coralline sand. Fill material was not encountered in Test Pit TP-1. Subsurface conditions encountered in Test Pit TP-1 and below the fill material in the other borings and test pits generally consisted of beach and/or dune deposits of medium dense to very dense, poorly graded coralline sand to the bottom of the test pits at depths of 7.5 to 9 feet, to the bottom of Borings B-4 and B-5 at a depth of approximately 40.5 feet, and to a depth of about 47 feet in Boring B-2. The beach and/or dune deposits in Boring B-2 was underlain by coral reef deposits consisting of medium dense to very dense, uncemented to moderately cemented foraminifera sand to the bottom of this boring at a depth of approximately 60.5 feet."*

As recommended in the Geotechnical Exploration Report, preparation of the Cell 2 subbase will require the removal of soils that are fat clays and compaction of poorly graded sand deposits.

3.0 DESIGN AND CAPACITY

Accompanying this report are drawings titled *Engineering Report, Kekaha Landfill Phase II – Cell 2 Lateral Expansion, Kekaha Sanitary Landfill, Kaua'i, Hawai'i*, prepared by AECOM, dated December 2015. These drawings are referenced in the following sections highlighting the components of the proposed lateral expansion.

3.1 Base Liner Design

Hawai'i Administrative Rules (HAR) 11-58.1-14(b)(2) (DOH 1994) require lateral expansions to have a composite base liner with a leachate collection system designed and constructed to maintain leachate at a depth less than 30 centimeters (12 inches) across the liner system. The composite liner must consist of two components, as defined by HAR 11-58.1-14(c):

1. Upper component: 30-mil flexible membrane liner (FML) with a maximum permeability of 1×10^{-7} centimeters per second (cm/sec) or if HDPE is used for the construction of the FML, the minimum thickness shall be 60 mils. The installed FML must be in direct contact with the underlying compacted soil.
2. Lower component: Compacted soil layer with a minimum thickness of 2 ft (ft) and permeability of no more than 1×10^{-7} cm/sec.

The proposed base liner system for the Cell 2 expansion is similar to the permitted base liner for the completed Cell 1 expansion. The proposed Cell 2 base liner system is (from bottom to top):

- Prepared subbase grade
- Geosynthetic reinforcement grid (Phase 1 sideslope overlay for Subcells A and D)
- Nonwoven cushion geotextile
- Secondary 60-mil HDPE geomembrane (lower component) (see note.)
- GCL (proposed as alternative to lower soil component requirement)
- Primary 60-mil HDPE geomembrane (upper component) (see note.)
- Nonwoven cushion geotextile
- 12-inch granular drainage layer (leachate collection)
- Nonwoven separator geotextile
- 24-inch operations layer

Note: Both the primary and secondary geomembranes are textured on both sides of the geomembrane except for the portion of the base liner that overlays directly upon the Phase 1 sideslope. Primary and secondary geomembranes residing on the Phase 1 sideslope will be single-sided textured with the texturing in direct contact with the geotextile layers of the GCL. This system orientation of texturing is to provide a slip surface for the base liner system to relax should a waste depression occur under the Cell 2 base liner on the Phase 1 sideslope.

Detail 1 on Drawing No. 8 displays the liner system. Details 6 and 7 on Drawing No. 8 show the overlay onto Phase I. Detail 1 on Drawing No. 14 shows the base liner tie-in to Phase II, Cell 1.

Unlike the existing base liner system of Phase II, which incorporates a geomembrane/bentonite layer style GCL (Gundseal), the proposed GCL for Cell 2 will be like the one used for Cell 1: a composite product consisting of bentonite powder/granules encapsulated between a carrier geotextile and a cover geotextile. The resulting bentonite and geotextile composite is then needle-punched for additional strength.

While the GCL will be a minimum of 5 ft above groundwater, the subbase 60-mil HDPE geomembrane is incorporated in the design as a separation barrier between groundwater and the GCL. The GCL is comprised of sodium bentonite. When in contact with calcareous soils, sodium bentonite may exhibit a reduction in permeability due to ion exchange. The liner performance modeling completed for the base liner (Appendix) is, therefore, considered conservative because the modeling did not include this extra 60-mil separation geomembrane nor a reduced permeability GCL.

Because the existing base liner system of Phase II consists of a GCL rather than a 2-ft compacted soil layer with a maximum permeability of 1×10^{-7} cm/sec, a point of compliance assessment (POCA) was completed by Harding Lawson Associates (HLA) in September 1993. The POCA used the EPA Hydraulic Evaluation of Landfill Performance (HELP) model, Version 2, to determine a GCL equivalent to a 2-ft compacted soil layer [HLA, 1993]. AECOM Technical Services, Inc. (AECOM) completed an updated POCA for the Cell 1 lateral expansion of Phase II [AECOM, September 2008]. The updated POCA utilized the EPA HELP model, Version 3.07, the Multimedia Exposure Assessment Model program for exposure assessment (which simulates the movement of contaminants leaching from a waste disposal facility), proposed material properties, and recent on-site weather data to determine a proposed GCL equivalent to a 2-ft compacted soil layer. Based upon the updated POCA for Cell 1, no increase in groundwater impact is expected relative to the previous HLA POCA. As described in the Groundwater and Leachate Monitoring Plan (AECOM September 2016) (Operations Manual, Section VI), an extensive review of all of the groundwater contours over the previous 15 years was conducted. Results indicated that, despite a brief mounding observed near the adjacent shrimp farms during a few recent events, groundwater gradients and direction remain relatively stable over time. The POCA has been updated and is included in Appendix G. [AECOM, January 2016] The groundwater data indicates a maximum groundwater elevation at the site of 5 ft msl. To accommodate potential groundwater elevation changes, the lowest elevation of the base liner will not be below 10.0 ft msl.

3.2 CELL GEOMETRY

The proposed Cell 2 expansion will consist of one new containment area that is geometrically subdivided into four subcells for leachate management and extraction. Subbase grades, which represent the surface at which the base liner is constructed for Cell 2, are shown in Drawing No. 2. All discussions relating to elevations are correlated to the site specific vertical datum unless otherwise specified.

Cell 2 will occupy approximately 6.6 acres and will overlay the north face of Phase I and the south face of Phase II. The centerline or leachate collection piping flowline of each Cell 2 subcell is sloped at a minimum of 2% to collection (sump) areas, as shown on Drawing No. 3.

Subbase liner elevations range from approximately 11 to 21 ft. The sideslopes of Cell 2 will have a maximum 3 horizontal (H):1 vertical (V) slope gradient. Leachate from the collection piping and subbase grade slope will be collected in sumps with bottom elevation of 10.5 and pumped to the leachate evaporation pond via a common force main, as shown in Drawing No. 3. Leachate evaporation pond sizing is discussed further in Section 7.4.

As was the case for Cell 1, the base liner of Cell 2 will be tied into Phase II and Cell 1 providing a continuous base liner system throughout the entire (final) Phase II footprint.

3.3 Final Cover Design

HAR 11-58.1-17(a)(1) requires that landfill covers have an 18-inch infiltration layer with a permeability of less than or equal to the permeability of any bottom (base) liner system or 1×10^{-5} cm/sec, whichever is less, and a minimum 6-inch erosion layer that can sustain native plant growth or equivalent alternatives. Both the existing Phase II and Cell 1 landfill base liner system and the proposed Cell 2 base liner system consist of a prepared subgrade, GCL, and a 60-mil HDPE

liner. Calculations and design discussion of the proposed final cover cross section and the permitted bottom liner are in Appendix B, demonstrating that the requirements of HAR 11-58.1-17(a)(1) will be met.

The proposed Cell 2 expansion, as well as the rest of Phase II (including Cell 1), will utilize the following proposed cover system as shown in Detail 4 on Drawing No. 8.

- 18-inch vegetative/protective soil layer
- Geocomposite drainage layer
- 40-mil linear low-density polyethylene liner (double-sided textured)
- GCL
- 6-inch minimum grading layer
- 12 inches of intermediate cover material
- Top of waste

3.4 Landfill Capacity

Airspace from Cell 2 will be comprised of the new horizontal footprint between Phases I and II, and space above the Phase I and 2 sideslopes, up to a final cover height of 120 ft msl. The proposed grading design of the final cover consists of 3.5:1 (H:V) sideslopes with a 15-ft-wide bench and 3% top grade, similar to the design of the permitted Phase II final cover. Drawing No. 7 shows the Cell 2 dimensions and cross sections. As shown in Drawing No. 7, the final cover grade will be modified from those contained in the existing (vertical expansion) permit, to incorporate the new areas. The estimated amount of gross airspace (waste mass and daily cover soils) for the Cell 2 expansion is 972,500 cubic yards (cy). Based on information provided by the site operator, the average in-place density of the waste mass is approximately 1,300 pounds per cubic yard (pcy), which accounts for daily cover soils, and which varies from year to year, as a function of the available working area, compaction equipment, and types of waste deposited. The average tonnage of waste landfilled at the facility from May 2006 to June 2015 is approximately 82,000 tons/year and this waste acceptance rate is projected to continue. As shown in Table 1, the Cell 2 design is expected to provide up to approximately 7.7 years of additional site life.

Table 1: Estimated Additional Landfill Capacity

Expansion Area	Additional Design Volume (cy)	Annual Waste (tons)	Annual In-place Waste (cy)	Additional Years of Capacity With Cell II Expansion
Cell 2	972,500	82,000	124,200	7.7

Assumptions:
(1) Design volume determined to be the volume from the bottom of the 12-inch intermediate cover soil of the final cover system to the top of the 24-inch operation layer of the base liner system of Cell 2.
(2) Waste density of 1,300 pcy is the in-place waste compacted density.

4.0 GEOTECHNICAL DESIGN

4.1 FOUNDATION SETTLEMENT

Foundation settlement analysis evaluates the long-term performance of the landfill's base liner (Cell 2 and the overlay of Cell 2 onto Phase I and onto Cell 1) and the leachate collection system. The foundation settlement analysis evaluates the expected settlement of the foundation soils under the landfill, as well as the tensile strain on the landfill liner with the introduction of additional waste, and determines whether the leachate collection piping system will maintain positive drainage with this settlement.

Because the foundation soils at the site are sandy (HLA 1993; PGE 2007, 2012), total settlement is comprised only of elastic settlement for the leachate collection system and the Cell 2 base liner between the toe of slopes of Phases I and II. Total settlement of Phase I under the Cell 2 overlay is comprised of primary and secondary settlement of the existing waste mass of Phase I plus elastic settlement of the in situ sandy soils. Further discussions are present in Sections 4.2 and 4.3.

The recent geotechnical investigation (Appendix A) indicated that in situ material (once stripped of overlying unsuitable material) will provide a sand foundation of approximately uniform thickness. Therefore, areas of differential settlement are not expected along the base of Cell 2. This design is based on instant loading calculations, which are conservative because the actual overall settlement of the foundation soils will be progressive and occur incrementally as fill is placed.

4.2 Impact on Leachate Collection System

The impact of Cell 2 loading on the leachate collection system was evaluated. Based on the foundation settlement calculation, the base liner for Cell 2 is expected to undergo relatively uniform settlement over the entire area of the cell as demonstrated in Appendix B.1. Therefore, after settlement, the expected slope will maintain positive flow towards the sumps after waste has been placed in the landfill.

In all settlement calculations, loads (waste mass filling, etc.) were applied instantly, resulting in very conservative analysis. Actual filling will occur over several years, which allows soils and waste to adjust to pressures over time.

4.3 Strain on Liner

The strain on, or elongation of, the geomembrane base liner of Cell 2 is estimated to be minimal as settlement across the cell base should be relatively uniform due to the uniform sandy foundation soils. As a result of uniform settlement, the geomembrane base liner will experience minimal elongation.

The sideslope of Phase I was evaluated for settlement as a result of the additional overburden from the waste mass of Cell 2. This analysis accounts for loadings on Phase 1 from the crest to the toe of the slope. The crest of slope of Phase I will see minimal loading as this is where the final cover for Phase II terminates, therefore, settlement at the crest is assumed to be negligible; this results in conservative evaluation of liner strain due to elongation. The midpoint of the Phase I sideslope and the Cell 2 base liner will see the largest settlement. Total settlement at these points accounts for primary and secondary settlement of the existing Phase I waste mass due to the Cell 2 waste mass plus the elastic settlement of the underlying sandy soils. Midpoint total settlement will cause the liner to enter tension between the Phase 1 crest of slope and slope midpoint whereas the liner will be in compression from the midpoint to the toe of slope. The toe of slope will realize only added waste mass of Cell 2 and since underlying soils are sandy, elastic settlement is only calculated at this point.

The resulting tension (strain/elongation) is estimated to range from 1.27% to 1.29% (Appendix B.1B), which is less than the 13% allowable strain for HDPE geomembranes and the 6 to 20% strain

allowed for GCLs. This analysis does not take into account the slip-surface, liner run-out, or geogrid reinforcement.

In all settlement calculations, loads (waste mass filling, etc.) are applied instantly, which results in very conservative analysis. Actual filling will occur over several years which allows soils and waste to adjust to pressures over time.

Supporting calculations are presented in Appendix B.

A geosynthetic reinforcement grid will be installed in Subcells A and D to help resist settlement and strain realized by the Cell 2 base liner overlaying the Phase 1 sideslope. Subcells B and C have sufficient strength in the liner to resist depression settlement without geosynthetic reinforcement grid. Refer to Detail 1 on Drawing No. 8 and Appendix F.8. A depression calculation was performed to determine the amount of liner runout needed for the Cell 2 base liner to resist the effects of a depression forming in Phase 1 under the Cell 2 base liner from a potential subsurface oxidation event in the waste mass. The calculation determined an 11-foot runout for all subcells and that Subcells A and D also include the reinforcement grid. The runout is the geosynthetic which could potentially slip and fill in any depression that may form. It is depicted in Detail 7 on Drawing No. 8 with supporting calculations in Appendix B.7.

To further reduce impacts due to a potential subsurface oxidation event in the waste mass, the added feature of extending the final cover geomembrane of Phase 1 for a minimum of 200 feet past the ends of Subcells A and D is proposed. Much like when the Cell 2 base liner is installed over the Phase 1 sideslope, air intrusion to the waste mass is hampered by the base liner geomembranes. The final cover for Phase 1 only has a geomembrane present on the top portion (or crown) which extends about 10 feet down the sideslope. To create further buffer between potential air intrusion and the base liner of Subcells of A and D, the Phase 1 final cover geomembrane will be extended down the sideslope with 40-mil LLDPE double-sided textured geomembrane. For installation the existing topsoil layer on the Phase 1 sideslope will be removed and stockpiled, the low permeable soil layer rolled, the textured LLDPE geomembrane and cushion geotextile installed, and topsoil pushed back into place. Refer to Detail 8 on Drawing No. 8 and supporting slope stability calculations in Appendix B.9.

4.4 Buildout Slope Stability Evaluation

The final geometry of the proposed Cell 2 lateral expansion, with a maximum elevation of 120 ft msl, was verified for slope stability at final buildout. The final buildout condition represents the facility's final shape after waste placement has ceased and final cover has been installed. Top slopes are designed to be sloped at 3%. Final cover sideslopes are designed to be sloped at a ratio of 3.5H:1V.

The buildout stability analysis included three different failure scenarios based upon the geometry of the facility, and the geotechnical properties of foundation soils and waste mass. The first failure scenario analyzed the stability of the landfill against a block type failure along the base liner critical interface. The second failure scenario involved circular type failures that reside primarily within the waste mass. The third and final scenario was a circular failure that passes through the foundation soils or support structures, such as perimeter berms, Phase I, and also passes through the waste mass.

Based on the assumptions detailed in Appendix B.2, a factor of safety was calculated using the slope stability software, Slope/W, which is part of the GeoStudio 2007 software suite. The scenarios were evaluated in the "worst-case" cross-sections as presented in Appendix B.2. The calculated factor of safety was compared to the minimum factor of safety of 1.5 for each scenario, and the calculated factor of safety was significantly higher than 1.5, in all cases.

Modeled factors of safety are summarized in Table 2.

Table 2: Buildout Slope Stability Factor of Safety Summary

'Worst-case' Cross-Section	Condition	Failure Condition	Calculated Factor of Safety	Minimum Factor of Safety
C-C	Base Liner	Block	2.77	1.50
	Waste Mass	Circular	1.83	1.50
	Foundation	Circular	3.55	1.50
D-D	Base Liner	Block	2.87	1.50
	Waste Mass	Circular	2.70	1.50
	Foundation	Circular	3.42	1.50

As noted in Section 3.0, the geomembrane is either textured on both sides or single-sided textured to promote stability as needed. Even though textured geomembrane provides higher friction angles, this conservative analysis assumed a minimum interface friction angle of only 10 degrees (°) as the governing interface in the block failure analysis which represents smooth geomembrane to a nonwoven geotextile. The installation of textured geomembrane will increase the overall factor safety for buildout stability, beyond that shown here, as well as promote placement of the granular drainage layer and operations layers upon the sideslopes of Cell 2 (Section 4.5). The landfill with the proposed Cell 2 expansion is expected to remain stable based on assumed minimum interface friction angles and soil and waste mass properties presented in Appendix B.2. Verification of interface friction angles will be completed by interface friction angle testing of materials planned for use in construction. Minimum required interface friction angles are defined in Appendix F.

4.4.1 Summary of Assumptions

The waste mass properties were defined based on discussions with the site operators and published articles on waste mass properties. According to the site operator, the site currently maintains a compacted waste (waste only) density of 1,320 pcy and a 3:1 waste to soil ratio. Waste mass properties are dependent upon the degree of compaction, lift thickness, composition of the waste stream, moisture, and the amount of daily cover soil or alternate daily cover used.

A published paper entitled *Shear Strength of Municipal Solid Waste: A Laboratory Study*, authored by Motan, E. Sabri; Benson, Craig H.; Tuncer, B. Edil (Motan et al. ND), proposed waste mass strength values of 35° (35 degrees) and cohesion of 20 kilopascals (kPa) (418 pounds per square ft [psf]) for municipal solid waste. Given the current site waste stream of municipal solid waste and construction and demolition debris, those waste mass strength values are considered representative for the site. In order to be consistent with previous stability analyses performed for Kekaha Phase II expansions, the Cell 2 stability analyses used waste mass properties of 33° and 200 psf cohesion, which is considered conservative compared to the values suggested by Motan et al.

In addition to the waste mass properties, the critical interface of the proposed base liner system requires consideration. The critical interface governs overall site stability. Based upon the proposed base liner system design and angles of friction, the critical interface is between the textured geomembrane to the nonwoven cushion geotextile of the GCL (see Detail 1, Drawing No. 8). This interface exists both between the primary geomembrane bottom and the top of the GCL as well as the bottom of the GCL to the subbase geomembrane top (in other words, both the upper and lower surfaces of the GCL are potential critical interfaces). Based upon typical published and expected literature values for this interface, an interface friction angle of 23° and 8 kPa (167 psf) of adhesion are expected. To provide more conservative design, the calculations were run using a lower critical interface friction angle of 10°. Use of textured geomembrane throughout the facility on the sideslopes and base will further assist in overall stability. Further discussion on the sideslope geomembrane and drainage layer stability is provided in Section 4.5.

4.5 Drainage Layer Stability

It is a challenge to minimize stress added to a base liner system during construction of the granular drainage layer and the operations layer. Therefore, the soil material will be installed by the use of a low-ground pressure dozer pushing the materials to the required thickness up the sideslope. The proposed Cell 2 expansion base liner also utilizes a cushion geotextile under the granular drainage layer and a separation geotextile between the granular drainage layer and the operations layer.

To avoid increasing the tension in the geomembrane during installation of the granular drainage layer and/or the operations layer, the geotextiles must either be placed in tension by anchoring to the top of the sideslope using an anchor trench, or the base liner system must remain stable by use of interface friction angles. Given the variability of geotextiles, the drainage layer stability analysis was conservatively conducted based solely upon interface friction angles. Supporting calculations are provided in Appendix B.3 and indicate a minimum acceptable interface friction of 19.5° in order to place the granular drainage layer the maximum slope length of 85 ft. If laboratory testing at time of construction establishes a lower friction angle component, the length of granular drainage layer must be reduced per Table 2 of Appendix B.3.

Protection of the geomembrane from subbase soils or drainage layer media is provided by the use of cushion geotextiles. Based upon experience of Cell 1 construction and available materials, the cushion geotextile was evaluated for a granular media that include angular particles with an average particle size (d_{50}) of 0.6 to 1.0 inches. The cushion geotextile calculation in Appendix B.4 demonstrates a nonwoven geotextile with a mass per unit weight of 16 ounce per square yard (oz/syd) will protect the geomembrane.

In anticipation of the differing material types between the granular drainage layer and the operations layer, a geotextile separation layer will be installed. Using the calculation in Appendix B.4 and the guidelines defined by the American Association of State Highway and Transportation Officials geotextile specifications, a nonwoven geotextile with a mass per unit area of 6 oz/syd will perform as a separation layer between the granular drainage layer and the operations layer.

4.6 Final Cover Stability Evaluation

The final cover for Cell 2 is expected to be uniform with, and integrated into, the final cover for Phase II, including Cell 1. The Cell 1 final cover equivalency calculation has been updated in Appendix D.7 to incorporate inclusion of the Cell 2 overlay.

The final cover stability of Cell 2 was evaluated to reflect the maximum slope length (maximum distance between toe of slope and any benches or crest of slope). This analysis evaluated stability of the proposed final cover under both static and seepage conditions. Using the Infinite Slope Stability equation, under static conditions the minimum final cover interface friction angle is 23.1°. This equation provides a conservative analysis as the equation is not dependent upon buttressing at the toe of slope and soil thickness, slope length, or cover soil permeability.

Seepage stability accounts for the actual performance of the geocomposite drainage layer under the influence of the estimated runoff quantities and infiltration through the cover soils, based on the design storm event. Using a design 100-year, 24-hour storm event of 12 inches, the maximum slope length of approximately 165 ft, and calculated maximum final cover soil permeability of 5.6×10^{-5} cm/sec, the final cover of the Cell 2 overlay is expected to remain stable. The resulting factor of safety is 1.5 which exceeds the short-term condition factor of safety of 1.2. This scenario is defined as short-term as it only potentially occurs during the design storm event. Supporting calculations are located in Appendix B.5.

4.7 Seismicity and Liquefaction Evaluations

4.7.1 Seismicity Evaluation

HAR 11-58.1-13(e) and the CFR, Section (§)40 (Subtitle D), Part 258.14 require that "new municipal solid waste landfill (MSWLF) units and lateral expansions shall not be located in seismic impact zones unless the owner or operator demonstrates that all containment structures are designed to resist the maximum horizontal acceleration in lithified earth material for the site." HAR 11-58.1-3 defines a "seismic impact zone" as an area with a 10% or greater probability that the maximum horizontal acceleration in lithified earth material (rock) will exceed the earth's gravitational pull (g) by 10% (i.e., 0.1 g) in 250 years.

In the preparation of the *Addendum Operations Manual* (EMCON 1998), EMCON contracted a seismology consultant to determine the maximum horizontal acceleration at Kekaha Landfill. The consultant's report stated that the controlling horizontal peak ground acceleration is 0.06 g on rock with an associated mean magnitude earthquake of 6.2. Because the peak ground acceleration at the site is less than the 0.1 g threshold, the site is not located in a seismic impact zone, and an evaluation of seismic loading effects on the stability of the Cell 2 expansion is not required (HAR 11-58.1-13(e)).

4.7.2 Liquefaction Evaluation

Beneath the landfill is a cohesionless medium-dense sand layer. Increased pore pressures during the design earthquake might cause this layer to have decreased shear strength, potentially causing liquefaction. Liquefaction may cause ground failures such as settlement or lateral spreading.

In the preparation of the *Addendum Operations Manual* (EMCON 1998), EMCON determined whether this layer is liquefiable based on the analysis method developed by Seed and Idriss (1982). Liquefaction settlement was estimated using a relationship between the cyclic stress ratio, corrected penetration resistance, and volumetric strain for saturated, clean sands. The estimated settlement of the medium-dense sand layer during the design earthquake is approximately 1 inch (EMCON 1998) as compared to the 1.5 inches predicted in the Geotechnical Investigation (HLA 1993). Based on the computed factor of safety, there should be no lateral spreading during the design earthquake (EMCON 1998).

PGE completed an empirical review (PGE 2007) of initial soil liquefaction based upon methods by Seed and Idriss (1982) as part of the site exploration. Based upon a Richter Scale magnitude 5.0 earthquake and a peak ground acceleration of 0.075 g, the site generally has low liquefaction potential.

4.8 Geosynthetic Material Properties

Required geosynthetic material properties, such as thickness, strength properties, composition, and unique properties such as geomembrane texturing, are provided in Appendix F.

4.9 Geomembrane Compatibility

Geomembranes are manufactured from HDPE, which has a high UV and chemical resistance while providing flexibility. The geomembrane application for Cell 2 is leachate retention. Geomembranes have a high resistance to chemicals and are the predominant means of liner systems for all modern landfills. Geomembrane compatibility requirements are detailed in Appendix B.6.

5.0 SURFACE WATER MANAGEMENT

The surface water management system is shown on Drawing No. 5. The proposed surface water management system is similar to the permitted Phase II design for storm water conveyance. Surface water will be transferred to existing infiltration ditches and to the infiltration basin on the northwest corner of the facility. The Cell 2 lateral expansion results in an increase in final cover area and decrease in infiltration area when compared to the permitted surface water management system. The surface water management system was analyzed to confirm that the proposed system including the infiltration ditches and basin are capable of handling design flows (Appendix C).

The system includes diversion berms located on the sideslopes approximately 20 to 40 ft below the perimeter of the landfill top deck that direct surface water to rock-lined downdrains. Each downdrain has a trapezoidal cross-section with each downdrain conveying water to an infiltration drainage ditch conveyed via a perimeter channel or a culvert in select locations.

The surface water analyses are included in Appendix C. The analyses were completed using SEDCAD+, a computer program developed by the University of Kentucky, to assist in the design and evaluation of surface water management system components. The program is capable of computing peak flows, runoff volume, sizing channels and culverts, and calculating the hydraulic performance of sedimentation and infiltration basins. The methods utilized in SEDCAD+ include SCS Upland Curve Method for calculating time of concentration, Muskinghum's Method for routing between structures, and SCS Technical Release - 55 (TR-55) parameters for runoff volumes.

The surface water analysis evaluated site conditions under two different storm events, including the following:

- 100-year, 1-hour storm event of 3 inches of rainfall
- 25-year, 24-hour storm event of 8 inches of rainfall

The peak rainfall values are based on the design plates associated with County of Kaua'i Department of Public Works' *Storm Water Runoff System Manual* (DPW 2001).

The infiltration basin has been designed to accommodate both storms and all the surface water from the entire Phase II landfill, including Cell 1 and Cell 2. The infiltration rate from the bottom of the infiltration basin is 1 inch per 6 minutes (PGE 2007). The infiltration basin has a bottom elevation of approximately 7.5 ft site datum, which correlates to the invert elevation of the infiltration ditches. With the additional of expansion Cell 2, the 2.2-acre basin is expected to perform in the same manner and in conjunction with the existing infiltration ditches.

The surface water diversion berms have been conservatively designed to convey runoff from a 100-year, 1-hour storm event and the 25-year, 24-hour storm event with minimal freeboard in place. Each of the four downslope channels has been analyzed. Each of the channels has sideslopes of 2:1 and varies in width from 1 to 2 ft. In all cases, the channels will be lined with riprap for erosion protection.

The surface water management system calculations have been completed for a final cover with a maximum elevation of 120 ft. The proposed surface water management system performs adequately under this scenario. Surface water calculations and feature definitions are detailed in Appendix C.

6.0 LANDFILL GAS MIGRATION AND MONITORING

This section discusses the impacts the proposed Cell 2 expansion will have on the landfill gas collection system for the facility.

6.1 APPLICABLE REQUIREMENTS

KSLF has a design capacity greater than 2.5 million Mg and 2.5 million cubic meters (including all of the Phase I and Phase II landfill areas) as of the permitting of the latest Phase II vertical expansion. Consequently, the facility is currently subject to the New Source Performance Standard for MSW Landfills (NSPS) under 40 CFR 60 Subpart WWW. A gas collection and control system (GCCS) design plan was submitted to the regulatory authorities in May 2015 and the system was subsequently installed in accordance with the provisions of the NSPS in 2016. The system is currently operational and covers all areas of Phase I and Phase II. Once the Cell 2 expansion begins construction, the facility will become subject to a new NSPS for landfills under 40 CFR 60 Subpart XXX[1].

6.1.1 Non-Major Initial Covered Source Air Permit

DOH issued a Covered Source permit (0802-01-C) on September 10, 2014, for the landfill that addresses air emissions and gas collection and control from the facility. The site is in compliance with the current applicable provisions of the permit. The Cell 2 expansion will not trigger a change in the Covered Source permit necessitating a permit modification.

6.1.2 Landfill Gas Collection System Design

The current GCCS design plan required by 40 CFR 60, Subpart WWW will have to be amended to include Cell 2 within 12 months of the NMOC emission rate report[2] that exceeds 34 megagrams/year based on the provisions of 40 CFR 60, Subpart XXX.

The LFG collection system design in the site's Closure Plan has been updated to incorporate the current GCCS design plan and includes components that will allow for the installation of Cell 2 expansion gas extraction wells and collection piping. The Cell 2 system design and calculations are consistent with the requirements for the NSPS. The expansion will add gas extraction wells in the Cell 2 area and change gas pipe routing to conform to the new slopes. No other significant changes to the previous design will be necessary. The existing system already includes flanges to connect future piping to Cell 2 and the condensate management system and flare have sufficient capacity to accommodate the additional landfill gas collected from Cell 2.

The proposed Cell 2 gas collection wells, gas collection piping, and control device will meet the requirements listed in 40 CFR §60.759 as follows:

- Air intrusion will be minimized by ensuring the well screen is no closer than 15 ft to the surface.
- Damage to underlying liners will be avoided by insuring the well bore does not extend closer than 10 ft to the material.
- Corrosion resistant materials will be used: polyvinyl chloride (PVC), HDPE, etc.

[1] The statutory provisions of 40 CFR 60, Subpart WWW will remain in effect until U.S. EPA enacts changes to 40 CFR 63, Subpart AAAA. Consequently, the facility must comply with both Subpart WWW and Subpart XXX.

[2] Must be submitted within 90 days after the start of construction of Cell 2.

- Extraction wells will have connector assemblies (closing valves, sampling ports, etc.) suitable for NSPS monitoring, in case it is required.
- There will be a sufficient density of extraction devices to ensure gas is collected from all areas warranting control.
- The gas system will allow for expandability and accessibility: blind flanges have been incorporated into the design in order to allow for future gas system expansions; in addition, HDPE pipe can be cut and new components fused in place to expand the system easily.
- The flexibility of the material and the modulus of elasticity of both HDPE and PVC are sufficient to prevent collapse due to settlement and to withstand planned overburden and traffic loads.
- The active gas extraction system will be designed to handle the maximum expected gas flow rate from the entire area of the landfill that warrants control, over the intended use period of the gas control system equipment.
- An enclosed flare (installed as part of the current GCCS prior to the Cell 2 expansion) is used for control and is being, and will continue to be operated in accordance with 40 CFR 60, Subpart WWW – the New Source Performance Standard for Municipal Solid Waste Landfills and will accommodate the gas collection from Cell 2.

6.2 GAS COLLECTION SYSTEM IMPLEMENTATION SCHEDULE

A LFG collection and control system designed in accordance with the NSPS was installed in 2016. The facility may also elect to install an energy recovery facility utilizing the LFG in the future. The design of the energy recovery facility, if pursued, will be provided for regulatory review and approval if required. Gas collection in the Cell 2 expansion area will be required within 5 years of initial solid waste placement in the area if it is considered active or within 2 years if closed or at final grade.

6.3 GAS COLLECTION SYSTEM DESIGN

The GCCS is designed to extract LFG at a sufficient rate to minimize the subsurface lateral migration and surface emissions of LFG. This is achieved by sizing and installing sufficient collection elements, transmission piping, blower(s), and control device(s) for the estimated maximum flow rate of LFG (see Appendix E).

The GCCS is designed to collect LFG at a sufficient rate, which per the definition in 40 CFR §60.751 means to maintain a negative [gauge] pressure at all wellheads. Application of a negative gauge pressure and minimization of air infiltration will be verified by monitoring each LFG wellhead.

Per the definition stated in 40 CFR §60.751, "sufficient density" means "any number, spacing, and combination of collection system components necessary to maintain emission and migration control as determined by measures of performance set forth in this part." Drawing No. 6 illustrates the draft gas collection system layout. Well spacing on Phase II, including Cell 2, was established based on site-specific conditions (waste age, waste density, moisture content, etc.), zone of influence (ZOI) estimates, past experience, and engineering judgment. This is consistent with spacing criteria used at other landfills and should effectively control surface emissions and subsurface migration of LFG in accordance with NSPS requirements. Many of the existing passive vent structures installed in Phase 1 will be abandoned during the 2016 GCCS construction, with a few of the vents connected to the Phase II gas collection system to expand collection to all areas of the facility that warranted control under the provisions of the NSPS.

Final gas collector placement may vary from this final design due to the filling activities of the active landfill. However, the locations will not impact system operation. Furthermore, wells/collectors may have to be replaced, redrilled, or relocated due to the conditions of the waste the wells are contained in. If this happens, the location of the well/collector may vary from the original designed location. As-

built drawings of the gas collection system will be updated and a copy of the as-built drawing will be kept on-site.

6.3.1 Landfill Gas Generation Rates and Flow Characteristics

In compliance with 40 CFR §60.752(b)(2)(ii)(A), the maximum expected LFG flow rate for the site, including Cell 2 and Phase 1 was used for sizing the GCCS.

- **§60.752(b)(2)(ii)(A)(1).** An active collection system shall be designed to handle the maximum expected gas flow rate from the entire landfill that warrants control over the intended use period of the gas control or treatment system equipment.

6.3.2 Well, Collection Device, & Pipe Network Loading

The pipe network is designed to withstand the estimated installation, static, settlement, overburden, and traffic loads. Installation loads were determined to be insignificant for GCCS components based on the installation methods used. Static loads from the vacuums applied to the GCCS components and applied loads on the GCCS were both evaluated. Vacuum loads required for the GCCS operation were compared to, and found to be less than, the allowable vacuum loads for the GCCS components.

Foundations used for GCCS components were designed to handle the applied loads. The applied loads on GCCS components within the landfill, as well as settlement forces, cannot accurately be predicted due to the non-homogeneous nature of the refuse within the landfill. The GCCS components within the landfill are consistent with those at other landfills, which have been in-place for extended periods of time and verified to be capable of withstanding applied static and settlement forces. Various sections of the header or laterals may lose grade, collect condensate, and then need to be replaced or repaired. In the event those GCCS component failures occur, they will be repaired/replaced as required to maintain NSPS compliance.

6.3.3 Landfill Gas Extraction Design

The LFG extraction is normally implemented using gas collection devices connected to a vacuum source. This section describes the design consideration of these gas collection devices. There are some specific requirements included in the NSPS that these gas collection and extraction components apply to, which are stated below:

- **§60.759(a)(1).** The collection devices within the interior and along the perimeter areas shall be certified to achieve comprehensive control of surface gas emissions by a professional engineer. The following issues shall be addressed in the design: depths of refuse, refuse gas generation rates and flow characteristics, cover properties, gas system expandability, leachate and condensate management, accessibility, compatibility with filling operations, integration with closure end use, air intrusion control, corrosion resistance, fill settlement, and resistance to the refuse decomposition heat.
- **§60.759(b)(1).** The landfill gas extraction components shall be constructed of polyvinyl chloride (PVC), high density polyethylene (HDPE) pipe, fiberglass, stainless steel, or other non-porous corrosion resistant material of suitable dimensions to: convey projected amounts of gases; withstand installation, static, and settlement forces; and withstand planned overburden or traffic loads. The collection system shall extend as necessary to comply with emission and migration standards. Collection devices such as wells and horizontal collectors shall be perforated to allow gas entry without head loss sufficient to impair performance across the intended extent of control. Perforations shall be situated with regard to the need to prevent excessive air infiltration.
- **§60.759(b)(2).** Vertical wells shall be placed so as not to endanger underlying liners and shall address the occurrence of water within the landfill. Holes and trenches constructed for

piped wells and horizontal collectors shall be of sufficient cross-section so as to allow for their proper construction and completion including, for example, centering of pipes and placement of gravel backfill. Collection devices shall be designed so as not to allow indirect short-circuiting of air into the cover or refuse into the collection system or gas into the air. Any gravel used around pipe perforations should be of a dimension so as not to penetrate or block perforations.

- **§60.759(b)(3).** Collection devices may be connected to the collection header pipes below or above the landfill surface. The connector assembly shall include a positive closing throttle valve, any necessary seals and couplings, access couplings and at least one sampling port. The collection devices shall be constructed of PVC, HDPE, fiberglass, stainless steel, or other non-porous material of suitable thickness.

In general, the collection devices are connected to the collection system via header and lateral piping. The lateral piping is connected to the header above or below the landfill surface depending on the sequencing of the refuse addition to the landfill, and the final GCCS design. These collection devices are often called, "gas wells," which include extraction wellheads (connector assemblies) that are located above grade. These wellheads include a positive closing throttle valve, necessary seals and couplings, access couplings, and a minimum of two sampling ports; all which aid in the prevention of air intrusion, allow for proper operation of the wellheads, and allow the wellheads to be sampled and monitored.

6.3.4 Depths of Extraction Wells/Collection Device

Vertical wells cannot endanger the underlying liner system and must address the occurrence of perched liquids within the landfill. All existing vertical extraction wells were installed based on this criteria. The vertical wells in Cell 2 will be installed at the appropriate depths as stated by a professional engineer in a manner to capture as much landfill gas without putting the landfill liner system at risk.

Practical site-specific factors that may change over time will impact the depths of the vertical wells. Some of these factors include the following:

- Liner construction records
- Well locations above or near liner sideslopes or other areas in which the liner elevation changes rapidly
- Obstructions or other technical difficulties that may impact the drilling operations

6.4 LFG COLLECTION DEVICES

The gas collection system is designed to prevent air infiltration through the cover, refuse contamination of the collection elements, and direct venting of LFG to the atmosphere. Vertical and horizontal gas wells are typically used in order to extract LFG from the landfill. The LFG collection system design is described in this section.

6.4.1 Vertical LFG Wells

To a large extent, a well's zone of influence is dictated by the amount of vacuum that can be applied without causing an excessive amount of air intrusion into the landfill. Typically, to reduce air intrusion and thereby increase the ZOI, the upper 1/3 of the final well depth (with a minimum of 10 ft and a maximum of 30 ft) is not slotted or backfilled with gravel. Experience has shown that this ratio provides a good balance between air intrusion control and LFG collection efficiency. Air intrusion is also minimized by using soil backfill in the upper zone of the vertical wells. In addition, a hydrated bentonite plug is used where the pipe penetrates the landfill soil cover.

LFG enters the well pipe through slots in the well pipe extending through the portion of the well backfilled with gravel. Typically, the minimum open area of the slots will be 12 square inches per ft to

provide sufficient open area for free gas flow while minimizing the impact on pipe strength. However, the design may change based on site-specific characteristics that occur at the well location.

Further, air intrusion and LFG emissions will be controlled through periodic monitoring and adjustment of the GCCS in coordination with appropriate maintenance of the landfill cover system. Final cap design will include a boot around the well casing in situations that employ a synthetic cap.

Vertical gas extraction wells will be installed in the approximate locations shown in Drawing No. 6. Gas extraction well design details are provided in Drawing No. 13.

6.4.2 Well & Collection Device Perforation/Slots & Backfill

Collectors must be perforated to allow LFG entry without excessive head loss and the surrounding gravel must be sized to prevent blocking of perforations

There are many site-specific factors that will be examined to determine the length of the slotted portion of the gas well. Due to the costs associated with new well installation, it is in the best interest of the landfill to choose the optimal slot depth to collect as much gas from the well; yet, maintain compliance with the NSPS well parameters. For example, perforated/slotted section of the gas collection device may vary based on the following conditions:

- Depths of perched liquids contained in the landfill
- Mitigation of odors (slots/perforations may be extended for this reason)
- Installation of deeper slots to extend ZOI (may be beneficial in sites with synthetic caps)

6.4.3 Well/Collection Device Backfill

Gravel, washed aggregate, or other acceptable crushed stone (with low carbonate content) of sufficient size and/or inert non-calcareous material is specified to prevent penetration or blockages of the LFG collector pipe perforations/slots. Also, an acceptable substitute may be used in lieu of the aforementioned rock as long as it prevents blockage/penetration of the collector pipe perforations/slots.

6.4.4 Accessibility

At a minimum, the system is designed to provide accessibility to the GCCS components by persons on foot.

6.4.5 Condensate Management

Condensate management has been accomplished by sloping the LFG transmission piping to low points in the GCCS piping for collection of the condensate. Condensate collection sumps/drains are located at these low points, to collect the condensate and remove it from the transmission piping. Condensate collected in drains/sumps is designed to be re-introduced into the leachate management system and ultimately managed with the leachate at the facility. In addition, a condensate storage tank is installed to allow for off-site disposal if necessary. Any connections of the leachate collection system to the GCCS are for odor control and to meet other landfill operating needs beyond regulatory compliance with the rule.

6.4.6 Control Systems

The NSPS specifically requires that LFG collected by a NSPS compliant gas collection system be sent to NSPS compliant control device(s). These regulations contained in the NSPS are listed below:

- **§60.752(b)(2)(iii).** Route all the collected gas to a control system that complies with the requirements in either paragraph (b)(2)(iii)(A), (B) or (C) of this section.

The required operational performance of these components is stipulated by §60.752(b)(2)(iii)(A), which states:

- **§60.752(b)(2)(iii)(A).** An open flare designed and operated in accordance with §60.18, except as noted in §60.754(e)
- **§60.752(b)(2)(iii)(B).** A control system designed and operated to reduce NMOC by 98 weight-percent, or, when an enclosed combustion device is used for control, to either reduce NMOC by 98 weight percent or reduce the outlet NMOC concentration to less than 20 parts per million by volume, dry basis as hexane at 3 percent oxygen. The reduction efficiency or parts per million by volume shall be established by an initial performance test, required under §60.8 using the test methods specified in §60.754(d).
- **§60.752(b)(2)(iii)(C).** Route all collected gas to a treatment system that processes the collected gas for subsequent sale or use. All emissions from any atmospheric vent from the gas treatment system shall be subject to the requirements of paragraph (b)(2)(iii)(A) or (B) of this section.

The LFG control device currently installed is an enclosed flare that conforms to the requirements of §60.752(b)(2)(iii)(B). The enclosed flare is adequately sized to handle all the gas collected by the GCCS including the gas collection from Cell 2. Future controls, such as an energy recovery project, would impose "treatment" and would conform to the applicable requirements of the standard.

The capacity of the control system may increase or decrease over time as the amount and quality of LFG produced by the landfill changes. Therefore, the control devices or systems chosen for any interim or future timeframes may vary depending on the site specific LFG characteristics, including the quantities produced. All changes to the control system will be reviewed by a professional engineer to determine if an air permit is necessary.

6.5 LFG MONITORING DEVICES

The proposed Cell 2 horizontal expansion includes the installation of a modern, lined waste cell above the northeast sideslope of the existing Phase I Kekaha Landfill. To reduce differential settlement of the Cell 2 liner system, the Phase I waste mass, which has undergone more than 20 years of consolidation, must remain intact. A Subsurface Landfill Gas Temperature Monitoring and Contingency Plan (EIL, 2017) is among the several proposed systems and management plans developed to ensure the integrity of the Cell 2 liner and Phase I cover system.

While the presence of the Cell 2 liner above the Phase I waste mass in this area will inhibit the intrusion of oxidation, and thus decrease the likelihood of subsurface oxidation events, this plan proposes the implementation of a gas and temperature monitoring system to identify, in a timely fashion, potential future subsurface landfill oxidation events under the Cell 2 liner system in order to allow for appropriate response actions. The system is designed to minimize infrastructure that can impact facility operations, while still providing data indicative of oxidation events, and also providing a mechanism to extinguish such events, if necessary.

In landfills, minimizing the potential sources of oxygen intrusion into the waste mass prevents landfill fire generation and propagation. The use of an impermeable geosynthetic liner in the construction of Cell 2 and the existing geosynthetic capping material on Phase 1 will be an effective barrier to oxygen intrusion. Furthermore, additional impermeable cap material is planned to be installed on Phase I sideslopes beyond the Cell 2 footprint, to further inhibit oxygen intrusion. To enhance this system, a network of monitoring probes will be installed along the north slope of Phase I to a depth of 30 feet. A 2-inch diameter perforated pipe will be installed in each bore to allow for the insertion of a thermocouple to measure gas temperatures under the Cell 2 liner on a real time basis, backfilled to a depth of 10 feet with granular material not smaller than 1/2 inch in diameter to allow for the detection of oxygen close to the surface. A bentonite seal will be installed on top of the gravel and the remainder of the trench will be filled with cover soils. A port installed in the side of the casing will

allow for gas quality measurement. The thermocouple will be extended down the probe in 5-foot intervals during sampling to provide data of elevated temperatures near the liner of Cell 2.

Although gas generation creates positive pressure (once active gas extraction is stopped in nearby wells) that would prevent oxygen from entering the waste mass during drilling of the bores, only one probe will be drilled at any given time. This will limit exposure and ensure that oxygen source pathways are minimized during probe installation. Each probe will be temporarily capped when completed prior to drilling another bore.

The perforated casing will allow sufficient gas to enter the piping for monitoring while providing adequate crush resistance to forces generated by the approximately 30 ft of waste above it. Supporting calculations demonstrate that the piping proposed for the monitoring probes can resist forces generated by more than 115 ft of waste at a standard safety factor of 2. The Phase I waste depths will yield a safety factor of almost 8 regarding the collapse depth of the proposed piping:

7.0 LEACHATE MANAGEMENT

Water entering KSLF and the proposed Cell 2 expansion from rain events percolates through the waste and collects in the granular drainage layer overlying the liner system. This leachate is removed by leachate collection and transmission pipes located in the granular drainage layer. The slopes, features, and details of the leachate management system are shown in Drawing Numbers 2, 3, 8, 9, and 10.

Given the narrow geometry of Cell 2, each subcell will have only one leachate collection and transmission pipe. The pipes are sloped at a minimum 2.0% slope towards the leachate collection sump. Collected leachate is then pumped to the existing leachate evaporation pond. The existing collection pipes in Phase II and Cell 1 are constructed of 8-inch diameter perforated HDPE with a standard dimension ratio (SDR) of 9.3 to protect against crushing.

The leachate collection pipes for Cell 2 were analyzed for pipe strength. The leachate collection pipes for Cell 2 are also proposed to be constructed of 8-inch diameter perforated HDPE with a minimum SDR of 9.3. Based upon the maximum final cover elevation of 120 feet, the maximum waste depth for Cell 2 overlying the leachate collection pipe on the cell floor is approximately 100 feet. This waste mass thickness is the result of the overlay onto the existing Phase II and orientation of the leachate collection pipes. The pipe strength calculations have determined that the proposed pipe sizing, perforations, and wall thickness will obtain suitable factors of safety. Supporting calculations are presented in Appendix D.

Leachate collected in the subcells of Cell 2 by the leachate collection piping will be transferred from each to a common forcemain which will discharge to the existing leachate evaporation pond.

7.1 Leachate Generation

The amount of leachate entering leachate collection and transmission pipes was estimated using the EPA HELP Model Version 3.07 computer modeling program as presented in Appendix D. The amount of leachate generated at the facility is based upon the total area of the site that is open at any given time for waste disposal, with calculations performed on a unit acre basis to conservatively estimate the daily leachate generation rate. Leachate evaporation pond sizing depends ultimately on coordination of intermediate cover placement, closure of portions of the facility at final grade, and the promotion of surface water drainage and management during interim active conditions.

Based on average monthly model output, the Cell 2 lateral expansion during open conditions is estimated to generate 653 gallons/acre/day. This result is based on historical precipitation and temperature data collected at the Kekaha landfill site and solar radiation data for Honolulu Hawaii, as well as the following assumptions:

- 50% surface water run-off for the open condition area
- Good geomembrane base liner installation (2 defects/acre) and a pinhole density of 1 pinhole per acre to account for manufacturing defects
- The waste mass is at field capacity, therefore, not absorbing water

For the Cell 2 expansion, the daily generation based upon average monthly results is 4,311 gallons per day (gpd). Existing Phase II from September 2004 through December 2010 generated approximately 610,000 gallons per year, or 1,671 gpd. For Cell 1, assumed to be near final grade when Cell 2 is operational, the estimated daily generation based on average monthly results is 328 gallons/acre/day or 2,096 gpd. Total leachate generation estimated for existing Phase II, Cell 1 and Cell 2 is 48,800 gpd. The estimated daily generation rate for Cell 2 is based upon an open condition which allows for only 50% surface water run-off thereby creating larger daily generation rates than the existing Phase II and Cell 1, which will have at least intermediate cover in place when Cell 2 is operational.

Temperature, precipitation, and average wind speed data were obtained from site records from January 2001 through December 2010. Missing data are viewed by the model as a "0". Therefore, to avoid skewing the data, missing temperature data for 2001, 2002, and 2008 were filled in using temperatures from days adjacent to the missing data. The data missing in 2004 were filled in using data from the same dates in 2003. Data are missing for a total of 5 days in April of 2001 and 2002. Data from 2004 are missing from April 9 through May 3, and May 26 through June 19. For 2008 data are missing from January 30 through February 3. There are no missing data from 2003, 2005, 2006, 2007, 2009, and 2010. Inputs into the HELP Model and the output are detailed in Appendix D.

7.2 Leachate Collection Pipe Spacing

The allowable leachate collection pipe flow was calculated using Manning's equation, conservatively assuming a full pipe as shown in Appendix D. Assuming that the pipes will have a minimum slope of 2%, an allowable flow of 801,400 gpd, was calculated. The maximum generation rate of leachate is approximately 3,705 gallons/acre/day for open conditions. One single pipe at a slope of 2% can accommodate leachate flows under open conditions from over 216 acres. Therefore, the perforated leachate collection piping capacity is adequate to convey the leachate collected from the subcells and indicates that pipe spacing for the design of the lateral expansion is not a governing dimension.

Perforation sizing of the leachate collection pipe was also verified to insure proper flow into the pipe and removal of the leachate from the base liner. Half inch perforations spaced 5 inches on center are adequate for leachate pipe flow. Calculations are provided in Appendix D.

7.3 Leachate Transfer System

Leachate accumulated in the leachate collection pipes located in each subcell gravity-drains along the 2% base grade until it reaches the edge of the recessed sump of the subcell. The leachate is then pumped from the sump through a sideslope riser pipe into a double-contained HDPE forcemain pipe which flows to the leachate evaporation pond. Leachate pump design calculations for the sumps are provided in Appendix D.

Leachate levels in the sump and sideslope riser are controlled by the use of pressure transducers set for specific elevations to trigger pump on, pump off, and high alarm, based on minimum leachate head requirements. The controls for the leachate pump are proposed to be housed in a control panel near each riser pipe. Calculations for pipe strength of the leachate collection pipes as well as the sump sideslope riser pipes are included in Appendix D, and shown in Drawing Nos. 9 and 11.

7.4 Leachate Evaporation Pond Sizing

The current and preferred means of disposing of leachate is by evaporation. With the construction of Cell 1, a new geosynthetic-lined leachate evaporation pond was also constructed. The new leachate evaporation pond was designed to ensure sufficient capacity and surface area to promote leachate evaporation (Appendix D) for all of Phase II including Cells 1 and 2.

The evaporation pond verification calculation utilized existing leachate quantities, estimated lateral expansion leachate quantities, rainfall, and evaporation rates. Existing leachate generation data from September 2004 through December 2010 were reviewed. To conservatively base the design on the months of greatest recorded leachate generation, leachate generation rates from the existing landfill for 2006 are used. Leachate generation from Cell 1 and Cell 2 lateral expansion was estimated using the HELP model. Average monthly rainfall quantities were obtained from site records from January 2001 through December 2010. Estimated pan evaporation rates per the Ground Water Atlas of the United States near Hawai'i over open-ocean is 65 inches/year (in/yr) whereas actual evaporation rates based upon the weather station in LIHUE WSO is 96.5 in/yr and HILO WSO is 64.5 in/yr. To be conservative, an evaporation rate of 64.5 in/yr was used. Actual evaporation rate at the site is likely much higher.

A maximum leachate storage capacity of approximately 895,000 gallons is required in the evaporation pond. The evaporation pond as designed will retain approximately 3,699,000 gallons. This design also provides additional volume for major storm events, possible future expansion, and placement of a drainage/protective layer. Weather data suggests the wettest months, or months generating leachate quantities greater than evaporation rates, are March and April. Months prior to and after provide negative leachate balance; therefore, the evaporation pond as sized is adequate for evaporation and retention of estimated leachate and rainfall quantities. Appendix D presents the supporting calculation for the liquid balance in the evaporation pond.

8.0 ENVIRONMENTAL PERMITS, CONSULTATIONS, AND APPROVALS

Implementation of the proposed action requires coordination and consultation with the federal, state, and county agencies for the following permits, clearances, or approvals:

Environmental Assessment (EA). The proposed project occurs on State of Hawai'i land and would use State of Hawai'i funds; therefore, the project triggers the environmental review process mandated under Hawai'i Revised Statutes Chapter 343. As part of the environmental review process, an environmental assessment was prepared for the proposed action. The administrative authority of the environmental review process was the Office of Environmental Quality Control. Two EAs were completed for Cell 2, both of which resulted in a finding of no significant impact (FONSI). The Cell 2 lateral expansion EA was completed in November 2007, and the horizontal expansion of Phase II to a maximum height of 120 ft msl was completed in October 2013.

Solid Waste Management Permit. Expansion of a MSWLF must be authorized under a Solid Waste Management Permit issued by the DOH Solid and Hazardous Waste Branch.

Historic Preservation Review. State and county projects that may affect a historic property must obtain a concurrence of "no historic properties affected" from the State Historic Preservation Division (SHPD), prior to commencement. The administrative authority is the Department of Land and Natural Resources, SHPD. This concurrence was previously obtained during the preparation of the two EAs.

Clean Water Act (CWA) § 402 National Pollutant Discharge Elimination System (NPDES) Permit(s). Section 402 of the CWA establishes the NPDES program regulating the discharge of pollutants to waters of the United States. NPDES permits are required to authorize discharges of storm water associated with construction activities that result in disturbance of one acre or more of total land area. The administrative authority is DOH Clean Water Branch (CWB). Several years ago, the County had submitted a request for Exclusion from NPDES General Permit Coverage for Storm Water Associated with Industrial Activity for KSLF to the CWB to exempt the KSLF from the NPDES requirements. We continue to work with the CWB to try to obtain the Exclusion, which also includes the Cell 2 expansion.

A NOI-C NPDES permit has been obtained for the construction of Cell 2.

Special Management Area (SMA) Permit. Like the Phase I landfill, a portion of the expansion lies within the SMA. An SMA permit for the Cell 2 expansion was obtained from the County of Kauai Planning Department in July 2012.

Conservation District Use Application (CDUA) Permit. Like the Phase I landfill, a portion of the expansion lies within a conservation district. A CDUA permit for the Cell 2 expansion was obtained from the Department of Land and Natural Resources (DLNR), Office of Conservation and Coastal Lands (OCCL), in August 2012.

Grading Permit. Per County of Kauai Ordinance 808, §22-7.6(b), a grading permit is not required for a sanitary landfill such as Kekaha Phase II, subject to applicable County, State, and Federal government regulations or laws.

9.0 CONCLUSIONS AND RECOMMENDATIONS

Based on the evaluations conducted in this report, the proposed Cell 2 lateral expansion of Phase II is feasible and will serve to alleviate the County's vital and time-sensitive needs. The existing Phase II capacity is currently being consumed and timely construction of Cell 2 is critical for the island of Kaua'i.

10.0 REFERENCES

40 Code of Federal Regulations (CFR) 60. 1996. Subpart WWW—Standards of Performance for Municipal Solid Waste Landfills.

40 Code of Federal Regulations (CFR) 70. 1992. State Operating Permit Programs.

40 Code of Federal Regulations (CFR) 71. 1996. Federal Operating Permit Programs.

40 Code of Federal Regulations (CFR) 258. 2006. *Criteria for Municipal Solid Waste Landfills*.

*42 United States Code (U.S.C.) 1963. *Clean Air Act, as amended*. Chapter 85, §§7401-7671.

AECOM Technical Services, Inc. (AECOM) 2008. Point of Compliance Assessment (POCA) Update. Kekaha Landfill Phase II, Cell 1, Lateral Expansion. September.

AECOM Technical Services, Inc., (AECOM) 2012. Point of Compliance Assessment (POCA) Update, Kekaha Landfill Phase II, Cell 2 Expansion, Appendix G in Cell 2 ER, December.

AECOM Technical Services, Inc. (AECOM). 2011. *Preliminary Engineering Report, Kekaha Landfill Phase II, Cell 2 Lateral Expansion, Kekaha, Kaua'i, Hawai'i.* Honolulu. November.

Department of Health, State of Hawaii (DOH). 1994. Hawaii Administrative Rules (HAR), Title 11, Chapter 58.1: *Solid Waste Management Control*. January.

———. 2003. Hawaii Administrative Rules (HAR), Title 11, Chapter 60.1: *Air Pollution Control*. Clean Air Branch. September.

Department of Public Works, County of Kaua'i (DPW). 2000. *New Performance Standards for New Municipal Solid Waste Landfills, Kekaha Landfill.* Correspondence with State of Hawai`i Department of Health, Clean Air Branch. May 25.

———. 2001. *Storm Water Runoff System Manual*. July.

Earth Tech, Inc. 2007 *Engineering Report, Phase II Lateral Expansion, Kekaha Sanitary Landfill, Kauai, Hawaii*. Honolulu. August.

EMCON. 1998. *Addendum Operations Manual for Kekaha Sanitary Landfill – Phase II, Kekaha, Kauai, Hawai'i.* San Jose, CA. May 27.

Harding Lawson Associates (HLA). 1993. *Geotechnical Investigation, Kekaha Landfill Development, Phase II Design*. Aiea, Hawai'i. February.

Harding Lawson Associations (HLA). 1993. Point of Compliance Assessment (POCA), Kekaha Landfill, Phase II. September.

Motan, E. Sabri; C. Benson, and B. Tuncer. *Shear Strength of Municipal Solid Waste: A Laboratory Study*.

Seed, R. B., and I. M. Idriss. 1982. *Ground Motions and Soil Liquefaction During Earthquakes*. Earthquake Engineering Research Institute.

Pacific Geotechnical Engineers (PGE), Inc. 2007. *Final Letter Report, Geotechnical Exploration, PGE Job No. 8950-020, for Horizontal Expansion of the Kekaha MSW Phase II Landfill, Kekaha, Kauai, Hawaii.* March 20.

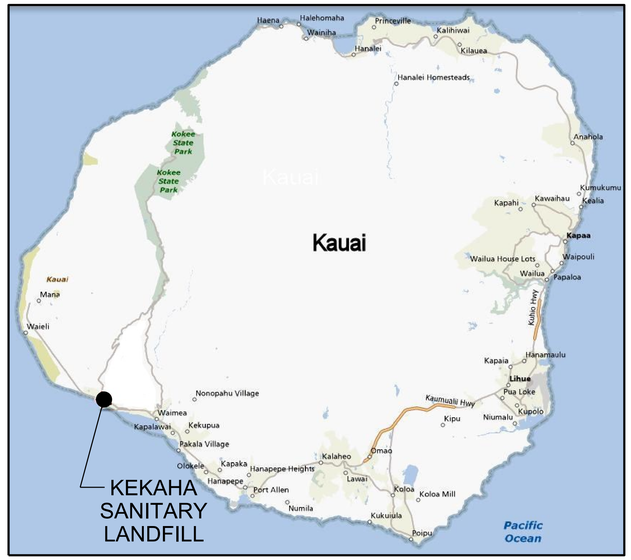
———. 2012. *Pre-Final Letter Report, Geotechnical Exploration, PGE Job No. 8950-023, for Kekaha Landfill Phase II, Cell 2 Lateral Expansion, Kekaha, Kauai, Hawaii.* August 8.

Drawings

ENGINEERING REPORT
KEKAHA LANDFILL PHASE II - CELL 2 LATERAL EXPANSION
KEKAHA SANITARY LANDFILL

KAUAI, HAWAII

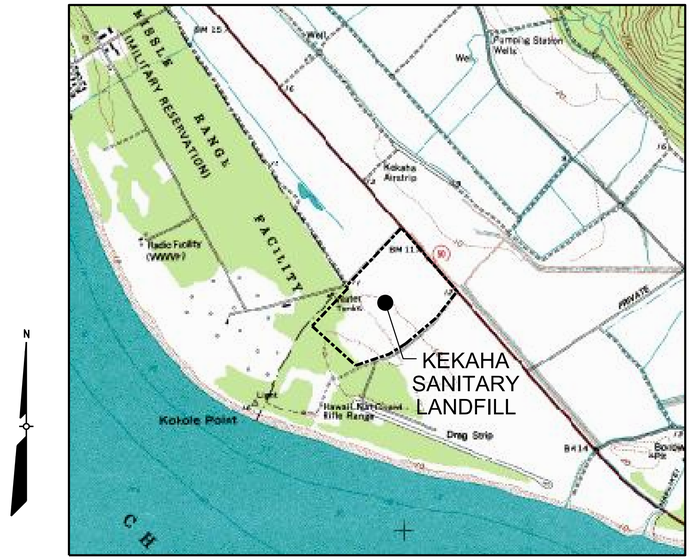
DECEMBER 2015



VICINITY MAP

INDEX OF DRAWINGS

DRAWING NO.	DRAWING TITLE
-	TITLE AND INDEX
1	EXISTING SITE CONDITIONS
2	SUBBASE GRADING PLAN
3	LEACHATE COLLECTION SYSTEM
4	FINAL COVER GRADING PLAN
5	SURFACE WATER MANAGEMENT PLAN
6	GAS MANAGEMENT SYSTEM PLAN
7	CROSS SECTIONS
8	DETAILS
9	DETAILS
10	DETAILS
11	DETAILS
12	DETAILS
13	DETAILS
14	DETAILS



SOURCE:
BASE MAP TAKEN FROM USGS 7.5 MINUTE TOPOGRAPHIC QUADRANGLE:
KAUAI QUADRANGLE, HAWAII

LOCATION MAP

PREPARED FOR

COUNTY OF KAUAI, DEPARTMENT OF PUBLIC WORKS
4444 Rice Street
Lihue, Kauai, 96766

KEKAHA LANDFILL
6900-D Kaumualii Highway
Kekaha, Hawaii
808-337-1416

PREPARED BY

AECOM

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HONOLULU, HAWAII 96813
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EXISTING CELL 1

EXISTING PHASE I

EXISTING PHASE II

INFILTRATION BASIN

LEACHATE EVAPORATION POND

SITE ENTRANCE

SCALE HOUSE

PUBLIC DROP-OFF AREA

OFFICE

MAINTENANCE BUILDING

Unpaved Parking

Paved Parking

KAUMUALII HWY

State Hwy 50

Kaumualii Hwy

MW-I-1
MW-I-2
MW-I-3
MW-II-2
MW-II-4R
MW-II-6R
MW-II-5
GP-I-A
GP-I-B
GP-I-C
GP-I-D
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LEGEND

- EXISTING TOPOGRAPHY
- PROPERTY LINE
- EXISTING LIMITS OF WASTE
- MW-I-3 — EXISTING GROUNDWATER MONITORING WELL
- EXISTING LEACHATE RISER PIPE PHASE II
- GP-II-4 — EXISTING GAS PROBE
- APPROXIMATE LOCATION OF EXISTING LANDFILL GAS VENT

NOTES:

1. EXISTING GRADE IS BASED ON TOPOGRAPHIC MAP PREPARED BY MILLER CREEK AERIAL MAPPING, LLC BURIEN, WASHINGTON. DATE OF FLIGHT: FEBRUARY 24, 2015.
2. HORIZONTAL DATUM IS BASED ON NAD83 (1986), HAWAII ZONE 4. VERTICAL DATUM IS SITE SPECIFIC AND CORRELATES TO DESIGNATED USGS BRASS MONUMENT G1000 PLUS 1.21 FEET.

SCALE 0' 50' 100' 200'

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1	UPDATED WITH FEBRUARY 24, 2015 TOPOGRAPHIC MAP	CFF	NKW	12/2015

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DES	NKW/TCR	
CHK	FLC/MRH	
APP	KJB	

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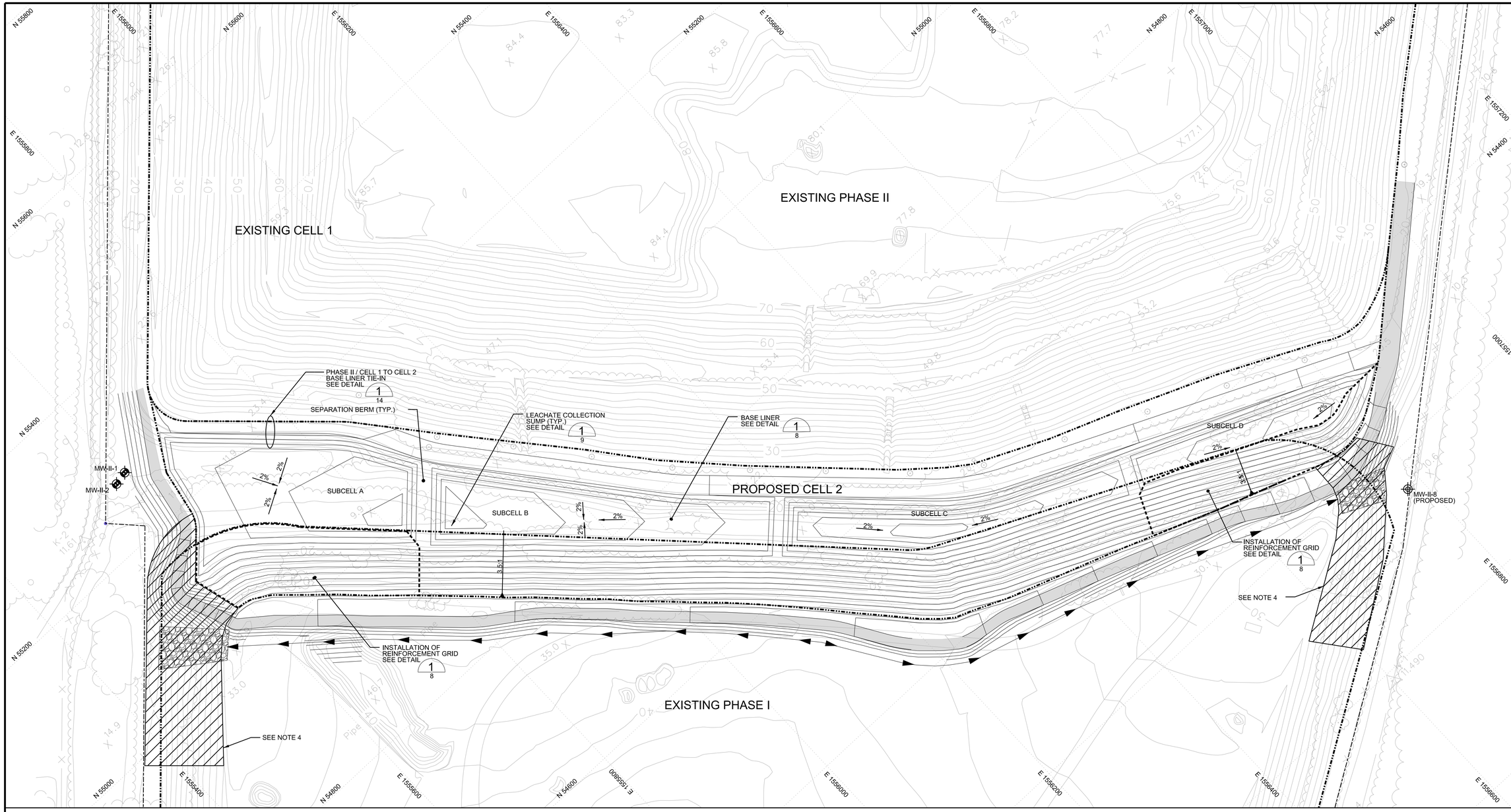
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ENGINEERING REPORT
KEKAHA LANDFILL PHASE II-CELL 2 LATERAL EXPANSION KEKAHA SANITARY LANDFILL
KAUAI, HAWAII

EXISTING SITE CONDITIONS

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LEGEND

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- PROPOSED GRADES (32)
- PROPERTY LINE
- EXISTING LIMIT OF WASTE
- PROPOSED LIMIT OF WASTE
- CELL DEVELOPMENT LIMIT
- ACCESS ROAD
- GRADE BREAK
- PHASE I FINAL COVER GEOMEMBRANE EXTENSION SEE DETAIL 8/8
- MW-II-1 EXISTING GROUNDWATER MONITORING WELL
- EXISTING LEACHATE CLEANOUT

NOTES:

1. EXISTING GRADE IS BASED ON TOPOGRAPHIC MAP PREPARED BY MILLER CREEK AERIAL MAPPING, LLC BURIEN, WASHINGTON. DATE OF FLIGHT: FEBRUARY 24, 2015.
2. HORIZONTAL DATUM IS BASED ON NAD83 (1986), HAWAII ZONE 4, VERTICAL DATUM IS SITE SPECIFIC AND CORRELATES TO DESIGNATED USGS BRASS MONUMENT G1000 PLUS 1.21 FEET.
3. GRADES SHOWN DEPICT SUBBASE AND SURFACE FOR INSTALLATION OF APPROVED BASE LINER.
4. EXTEND EXISTING PHASE I GEOMEMBRANE TO TOE OF SLOPE AND REPAIR COVER. SEE DETAIL 8/8

N

SCALE 0' 30' 60' 120'

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3	UPDATED TOPOGRAPHIC MAP	CFF	NKW	12/2015
2	REVISED ACCESS ROAD ALIGNMENT ON TOP OF PHASE 1 AND MODELED SUMPS IN CELL 2	DRB	KJB	03/2013
1	REVISED SUBBASE GRADES OVER PHASE 1, NOTED PHASE I FINAL COVER GEOMEMBRANE EXTENSION, INCREASE ROAD HEIGHT, AND SIDESLOPE REINFORCEMENT	DRB	KJB	12/2012

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ENGINEERING REPORT
KEIKAHA LANDFILL PHASE II-CELL 2 LATERAL EXPANSION KEIKAHA SANITARY LANDFILL
KAUAI, HAWAII

SUBBASE GRADING PLAN

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PUMP STATION #3 WET WELL

LEACHATE CLEANOUT RISER PIPE SEE NOTE 4

PERFORATED LEACHATE COLLECTION PIPE SEE DETAIL 3

INSTALLATION OF REINFORCEMENT GRID SEE DETAIL 1/8

LEACHATE COLLECTION SUMP (TYP.) SEE DETAIL 1/9

LEACHATE COLLECTION PIPING VERTICAL MANHOLE (TYP.) SEE DETAIL 1/10

LEACHATE EXTRACTION RISER PIPES SEE NOTE 5 AND DETAIL 1/9

EXISTING LEACHATE RISER PIPE FOR PHASE II (TYP.)

LEACHATE CLEANOUT RISER PIPE

TO EXISTING EVAPORATION POND

BELOW GRADE

ABOVE GRADE

MW-II-8 (PROPOSED)

INSTALLATION OF REINFORCEMENT GRID SEE DETAIL 1/8

LEACHATE EXTRACTION RISER PIPE

FORCEMAIN DISCHARGE TO LEACHATE EVAPORATION POND SEE DETAIL 1/11

OVERLAY BASE LINER ONTO PHASE I SEE DETAIL 6/8

OVERLAY BASE LINER TERMINATION ON PHASE II SEE DETAIL 7/8

LEGEND

- EXISTING TOPOGRAPHY (680)
- PROPERTY LINE
- EXISTING LIMIT OF WASTE
- PROPOSED LIMIT OF WASTE
- CELL DEVELOPMENT LIMIT
- GRADE BREAK
- LEACHATE COLLECTION PIPE
- CLEANOUT RISER

NOTES:

1. EXISTING GRADE IS BASED ON TOPOGRAPHIC MAP PREPARED BY MILLER CREEK AERIAL MAPPING, LLC BURIEN, WASHINGTON. DATE OF FLIGHT: FEBRUARY 24, 2015.
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3. GRADES SHOWN DEPICT SUBBASE AND SURFACE FOR INSTALLATION OF APPROVED BASE LINER.
4. THE CLEANOUT PIPES ARE CONNECTED TO THE PERFORATED LEACHATE COLLECTION PIPES.
5. THE LEACHATE EXTRACTION RISER PIPES ARE THE 18 INCH SIDE SLOPE RISER PIPES FOR THE PUMPS. SEE DETAIL DRAWING NO. 9

N

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2	REVISED PERIMETER FEATURES AND TANK LOCATION ON TOP PHASE I. MODELED SUMPS IN CELL 2. MODIFIED CLEANOUT ACCESS	DRB	KJB	03/2013
1	REVISED SUBBASE GRADES OVER PHASE 1, SUMP LOCATIONS, SUBCELL C & D NOTED PHASE I FINAL COVER GEOMEMBRANE EXTENSION. INCREASE ROAD HEIGHT, AND SIDESLOPE REINFORCEMENT	DRB	KJB	12/2012

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APP	KJB	

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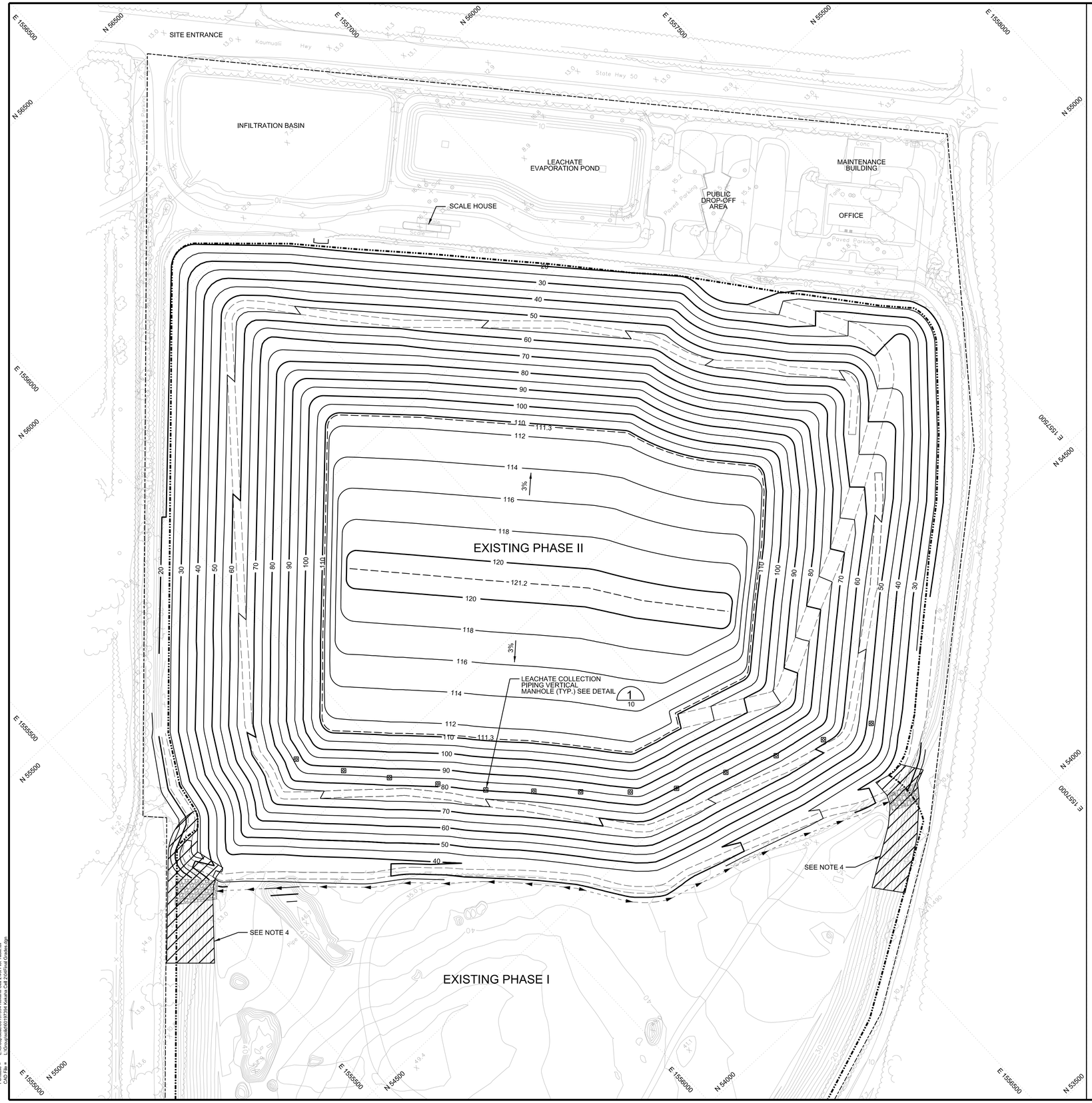
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ENGINEERING REPORT
KEKAHA LANDFILL PHASE II-CELL 2 LATERAL EXPANSION KEKAHA SANITARY LANDFILL
KAUAI, HAWAII

LEACHATE COLLECTION SYSTEM

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LEGEND

- EXISTING TOPOGRAPHY
- PROPOSED GRADES
- PROPERTY LINE
- PERMITTED LIMIT OF WASTE
- PROPOSED LIMIT OF WASTE
- ACCESS ROAD
- VERTICAL MANHOLE
- PHASE I FINAL COVER GEOMEMBRANE EXTENSION

NOTES:

1. EXISTING GRADE IS BASED ON TOPOGRAPHIC MAP PREPARED BY MILLER CREEK AERIAL MAPPING, LLC BURIEN, WASHINGTON. DATE OF FLIGHT: FEBRUARY 24, 2015.
2. HORIZONTAL DATUM IS BASED ON NAD83 (1986). HAWAII ZONE 4. VERTICAL DATUM IS SITE SPECIFIC AND CORRELATES TO DESIGNATED USGS BRASS MONUMENT G1000 PLUS 1.21 FEET.
3. GRADES SHOWN DEPICT TOP OF FINAL COVER SYSTEM. MAXIMUM FINAL COVER ELEVATION IS 120 FEET MSL; 121.2 FEET SITE DATUM.
4. THE CLEANOUT PIPES ARE CONNECTED TO THE PERFORATED LEACHATE COLLECTION PIPES.

NO	REVISIONS	DRN	CHK	DATE
3	REVISED FINAL GRADE AND ELEVATION 120' PEAK LOCATION	CFF	NKW	02/2016
2	REVISED PERIMETER FEATURES ON TOP OF PHASE 1.	DRB	KJB	03/2013
1	NOTED PHASE I FINAL COVER GEOMEMBRANE EXTENSION.	DRB	KJB	12/2012
	INCREASE ROAD HEIGHT			

NANCY K. WRIGHT
LICENSED PROFESSIONAL ENGINEER
No. 16394-C
HAWAII, U.S.A.

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SIGNATURE: Nancy K. Wright

APRIL 30, 2018
EXPIRATION DATE OF THE LICENSE

DRN	DRB	
DES	NKW/TCR	
CHK	FLC/MRH	
APP	KJB	

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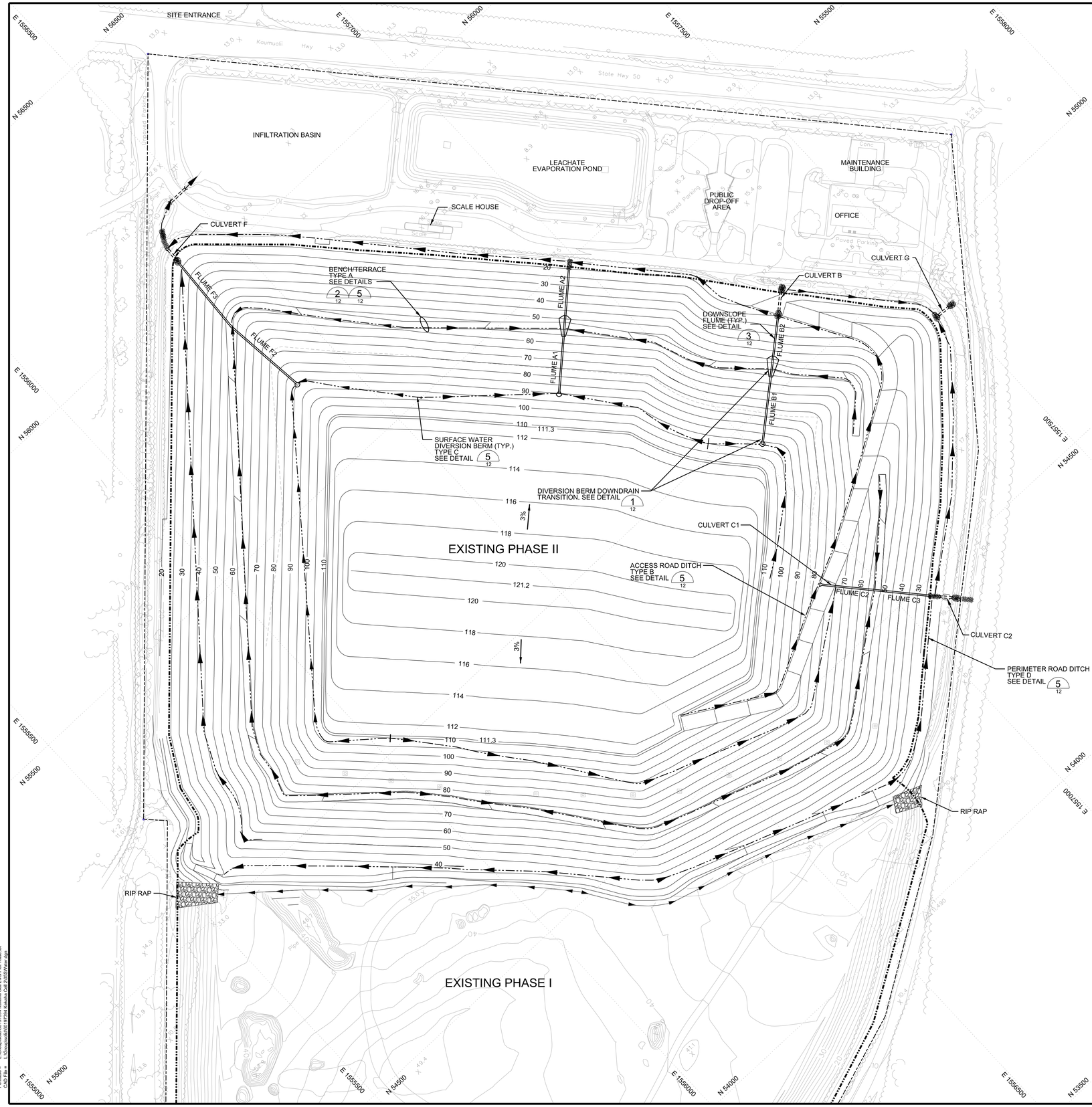
ENGINEERING REPORT
KEKAHA LANDFILL PHASE II-CELL 2 LATERAL EXPANSION KEKAHA SANITARY LANDFILL
KAUAI, HAWAII

FINAL COVER GRADING PLAN

SCALE 0' 50' 100' 200'

DATE	DECEMBER 2015
PROJECT NO	60197394
FILENAME	
SHEET NO	
DRAWING NO	4

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LEGEND

- EXISTING TOPOGRAPHY
- FINAL CONTOURS
- PROPERTY LINE
- PERMITTED LIMIT OF WASTE
- ACCESS ROAD
- VERTICAL MANHOLE
- SURFACE WATER FLOW
- CULVERT
- RIP RAP

NOTES:

1. EXISTING GRADE IS BASED ON TOPOGRAPHIC MAP PREPARED BY MILLER CREEK AERIAL MAPPING, LLC BURIEN, WASHINGTON. DATE OF FLIGHT: FEBRUARY 24, 2015.
2. HORIZONTAL DATUM IS BASED ON NAD83 (1986), HAWAII ZONE 4. VERTICAL DATUM IS SITE SPECIFIC AND CORRELATES TO DESIGNATED USGS BRASS MONUMENT G1000 PLUS 1.21 FEET.
3. GRADES SHOWN DEPICT TOP OF FINAL COVER SYSTEM. MAXIMUM FINAL COVER ELEVATION IS 120 FEET MSL; 121.2 FEET SITE DATUM. SEE FINAL COVER DETAIL 4/8
4. THE CLEANOUT PIPES ARE CONNECTED TO THE PERFORATED LEACHATE COLLECTION PIPES.

NO	REVISIONS	DRN	CHK	DATE
3	REVISED FINAL GRADES AND SURFACE WATER FEATURES	CFF	NKW	12/2015
2	REVISED PERIMETER FEATURES ON TOP OF PHASE I	DRB	KJB	03/2013
1	INCREASED ACCESS ROAD HEIGHT	DRB	KJB	12/2012

NANCY K. WRIGHT
LICENSED PROFESSIONAL ENGINEER
No. 16394-C
HAWAII U.S.A.

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APRIL 30, 2018
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CHK	FLC/MRH	
APP	KJB	

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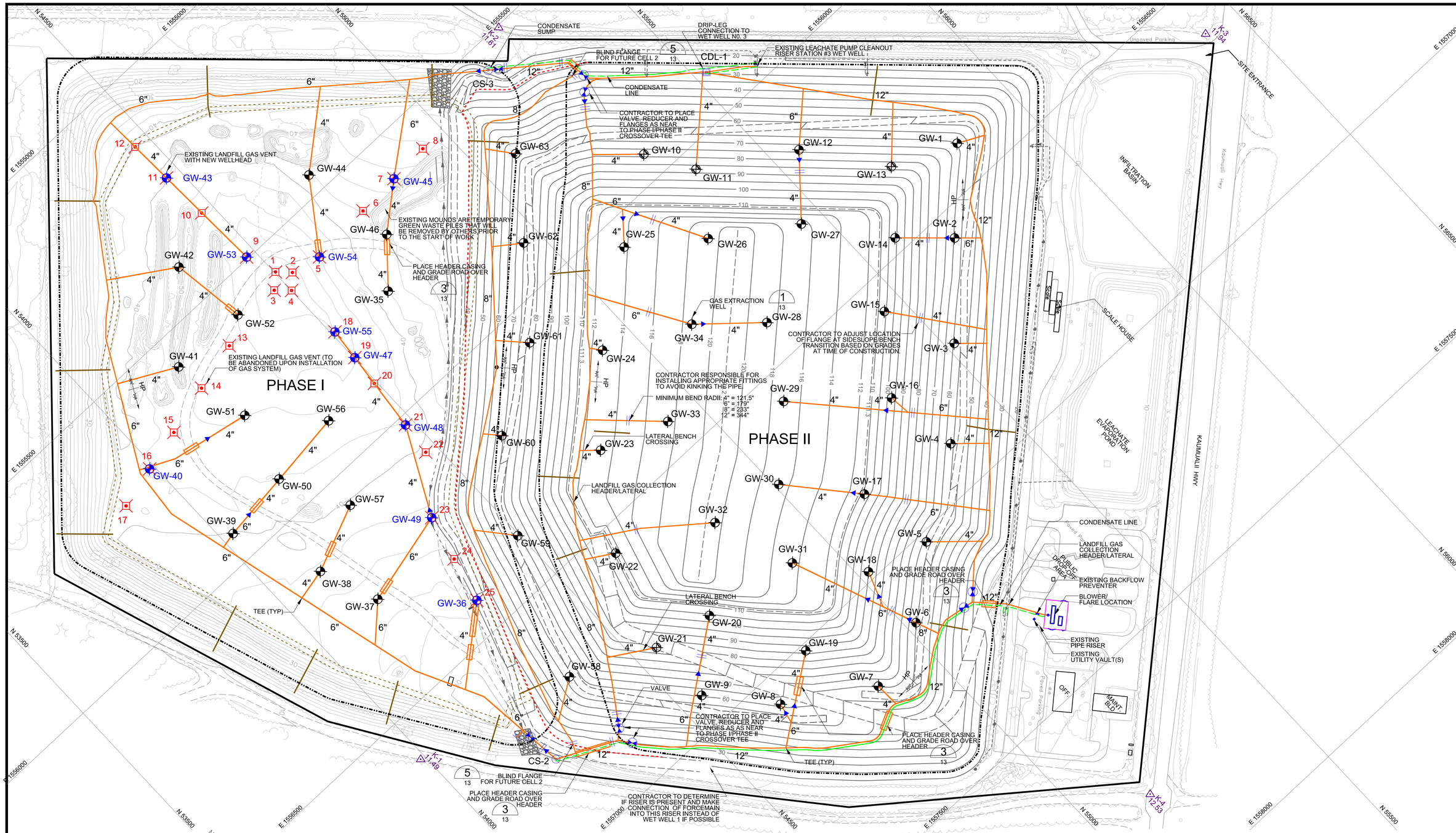
ENGINEERING REPORT
KEKAHA LANDFILL PHASE II-CELL 2 LATERAL EXPANSION KEKAHA SANITARY LANDFILL
KAUAI, HAWAII

SURFACE WATER MANAGEMENT PLAN

DATE	DECEMBER 2015
PROJECT NO	60197394
FILENAME	
SHEET NO	
DRAWING NO	5

SCALE 0' 50' 100' 200'

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KEKAHA LANDFILL SURVEY BENCHMARKS

BENCHMARK	EASTING	NORTHING	ELEVATION	LATITUDE	LONGITUDE
K-1	1,556,598.87	53,989.17	11.49	21.5855334	159.4451605
K-2	1,555,504.23	55,251.92	11.61	21.5907828	159.4503245
K-3	1,556,609.43	56,415.62	11.94	21.5919379	159.4451487
K-4	1,557,786.14	55,146.31	12.53	21.5906798	159.4438987

LEGEND:

- PROPERTY LINE
- APPROXIMATE WASTE LIMITS
- APPROXIMATE EXPANSION WASTE LIMITS
- FINAL GRADES
- ACCESS ROAD
- EXISTING TOPOGRAPHY
- COORDINATE SYSTEM
- EXISTING STRUCTURE
- APPROXIMATE EXISTING SOIL BERM
- APPROXIMATE LOCATION OF EXISTING LANDFILL GAS VENT (TO BE ABANDONED UPON INSTALLATION OF PROPOSED GAS SYSTEM)
- EXISTING BENCHMARK
- LEACHATE PUMP STATION WETWELL (APPROXIMATE)
- LEACHATE MANAGEMENT PIPING (APPROXIMATE)
- EXISTING LEACHATE COLLECTION MANHOLE (APPROXIMATE)
- APPROXIMATE EXISTING STORMWATER DRAINAGE STRUCTURE
- LANDFILL GAS COLLECTION HEADER/LATERAL
- CONDENSATE LINE
- CONDENSATE SUMP
- BLIND FLANGE
- GAS EXTRACTION WELL
- VALVE
- REDUCER
- EXISTING LANDFILL GAS VENT WITH NEW WELLHEAD
- DRIPLEG
- HEADER CASING

NOTES:

1. TOPOGRAPHIC MAP PREPARED BY MILLER CREEK AERIAL MAPPING, LLC, BURIEN, WASHINGTON. DATE OF FLIGHT: FEBRUARY 24, 2015.
2. EXISTING PHASE I LANDFILL GAS VENTS SHOWN TAKEN FROM DRAWING C-3, ENTITLED "LANDFILL GAS SYSTEM AND GEOTEXTILE 1 PLAN", DATED JANUARY 31, 1996. CREATED BY HARDING LAWSON ASSOCIATES.
3. EXISTING WATER MAIN, LEACHATE FORCEMAIN, MANHOLES, POTABLE WATER, SANITARY SEWER, AND WET WELL LOCATIONS ARE APPROXIMATE AND WERE DIGITIZED FROM BLUE PRINT DESIGN DRAWINGS, CREATED BY HARDING LAWSON ASSOCIATES, PROVIDED BY WASTE MANAGEMENT, DATED MAY 1994 AND DRAWINGS TITLED, "KEKAHA SANITARY LANDFILL, PHASE 2, KEKAHA, HAWAII", PREPARED BY HARDING LAWSON ASSOCIATES, DATED MARCH 1993.
4. FOR CLARITY, NOT ALL SITE FEATURES MAY BE SHOWN.
5. THIS PLAN PRESENTS THE MINIMUM NUMBER OF LFG WELLS PROPOSED IN THE APPROVED GCCS DESIGN PLAN. LFG WET WELL AND PIPE LOCATIONS MAY BE ADJUSTED WITH ENGINEERS APPROVAL AND ADDITIONAL LFG WELLS AND PIPES MAY BE ADDED AS NEEDED DURING AND AFTER WASTE FILLING OPERATIONS.
6. PIPING SLOPES:
   - PHASE I - MINIMUM 0.5%.
   - PHASE II - MINIMUM 3% IF ON WASTE (EXCEPT AS NOTED).
   - PHASE II - MINIMUM 0.5% IF OUT OF WASTE.

NO	REVISIONS	DRN	CHK	DATE

DRN DES CHK APP INITIAL DATE

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STAMP: ANDREW J. QUERIO, LICENSED PROFESSIONAL ENGINEER, No. 15174-C, HAWAII U.S.A., EXP. 4/30/2018

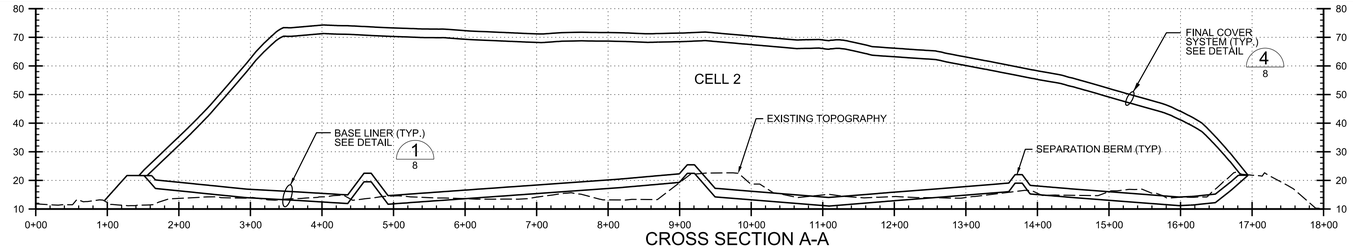
PREPARED FOR: COUNTY OF KAUAI KEKAHA SANITARY LANDFILL

PREPARED BY: EIL ENVIRONMENTAL INFORMATION LOGISTICS, LLC

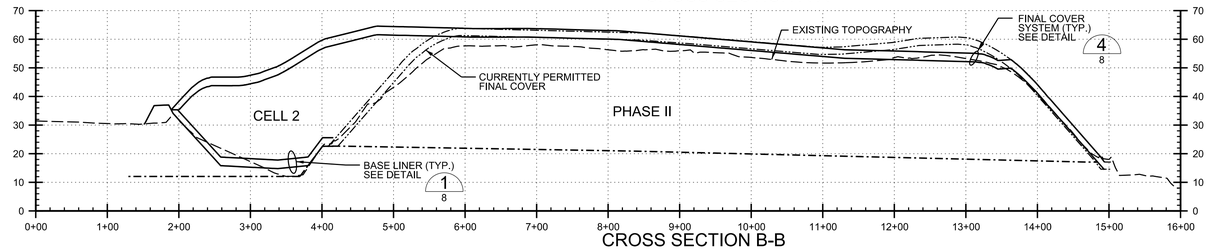
KEKAHA SANITARY LANDFILL
KEKAHA, KAUAI, HAWAII

GAS MANAGEMENT SYSTEM PLAN

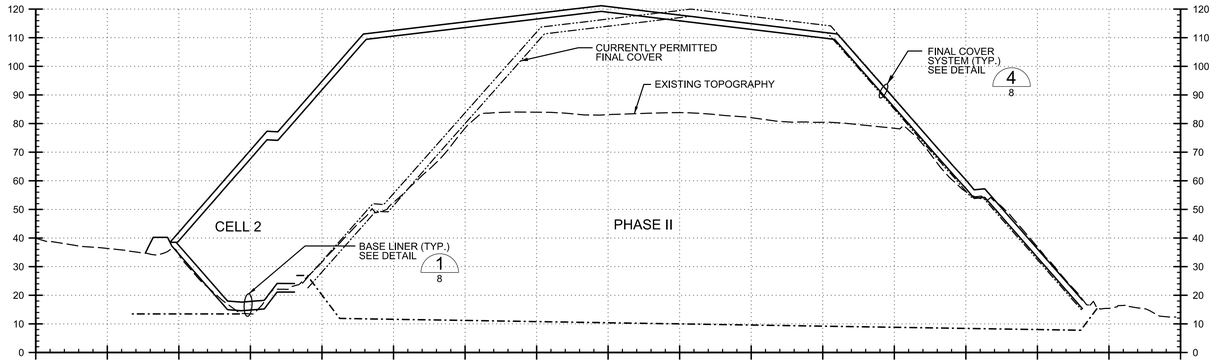
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PROJECT NO	090605
FILENAME	06 KEK GCCS PLAN 101917
SHEET NO	6
DRAWING NO	6



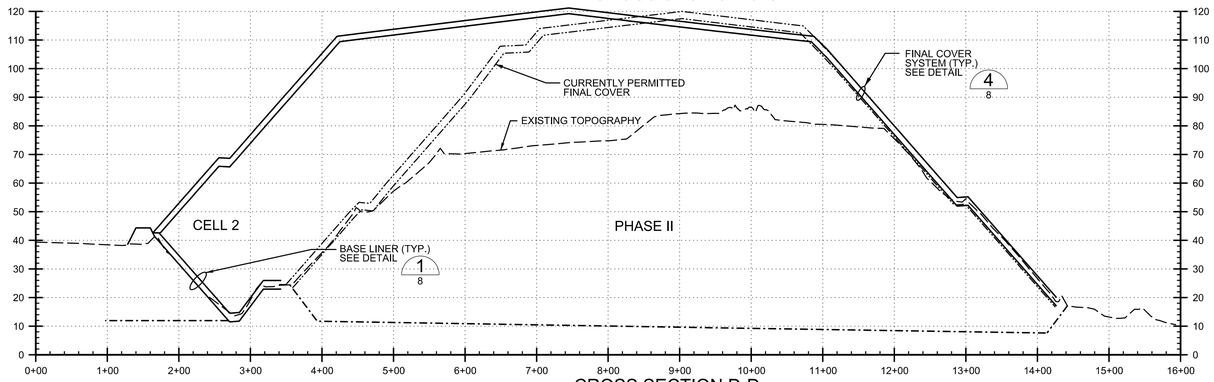
CROSS SECTION A-A



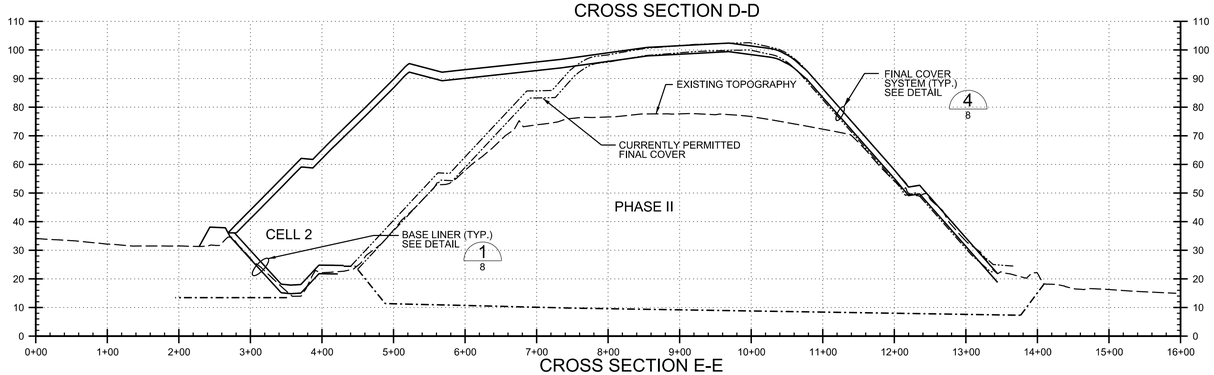
CROSS SECTION B-B



CROSS SECTION C-C



CROSS SECTION D-D

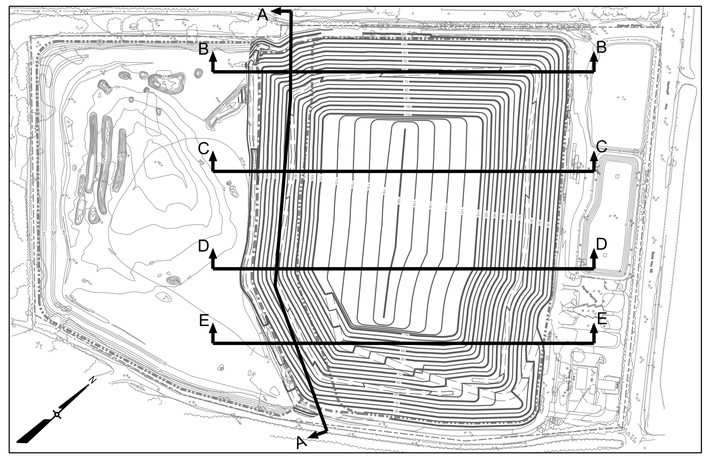


CROSS SECTION E-E

NOTES:

1. EXISTING GRADE IS BASED ON TOPOGRAPHIC MAP PREPARED BY MILLER CREEK AERIAL MAPPING, LLC BURIEN, WASHINGTON. DATE OF FLIGHT: FEBRUARY 24, 2015.
2. HORIZONTAL DATUM IS BASED ON NAD83 (1986), HAWAII ZONE 4. VERTICAL DATUM IS SITE SPECIFIC AND CORRELATES TO DESIGNATED USGS BRASS MONUMENT G1000 PLUS 1.21 FEET.

SCALE
HORIZ. 0' 50' 100' 200'
VERT. 0' 40' 80'



CROSS SECTION LOCATOR

NO	REVISIONS	DRN	CHK	DATE
3	UPDATED CROSS SECTION FOR REVISED FINAL COVER	CFF	NKW	12/2015
2	UPDATE CROSS SECTIONS FOR REVISED CELL 2 BASE GRADES	DRB	KJB	03/2013
1	UPDATE CROSS SECTIONS FOR REVISED CELL 2 BASE GRADES	DRB	KJB	12/2012
	OVER PHASE I, AND ROAD HEIGHT			

NANCY K. WRIGHT
LICENSED PROFESSIONAL ENGINEER
No. 16394-C
HAWAII, U.S.A.

THIS WORK WAS PREPARED BY ME OR UNDER MY SUPERVISION.

SIGNATURE

APRIL 30, 2018
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DRN	DRB	
DES	NKW/TCR	
CHK	FLC/MRH	
APP	KJB	

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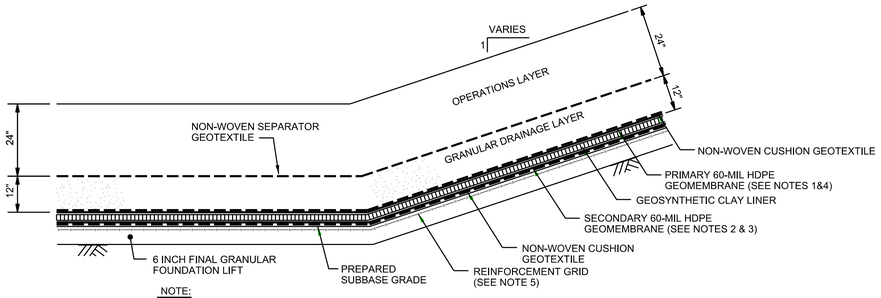
PREPARED BY AECOM

ENGINEERING REPORT
KEKAHA LANDFILL PHASE II-CELL 2 LATERAL EXPANSION KEKAHA SANITARY LANDFILL
KAUAI, HAWAII

CROSS SECTIONS

DATE DECEMBER 2015
PROJECT NO 60197394
FILENAME
SHEET NO
DRAWING NO 7

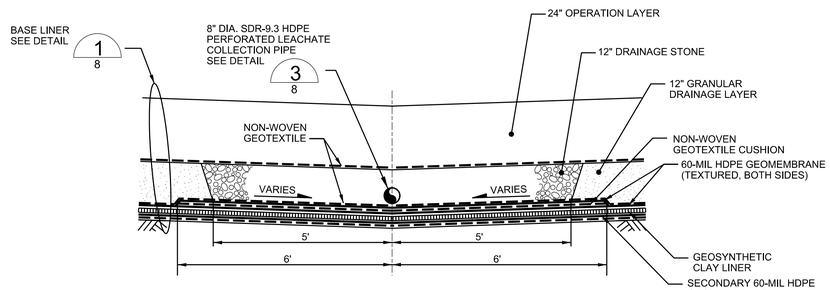
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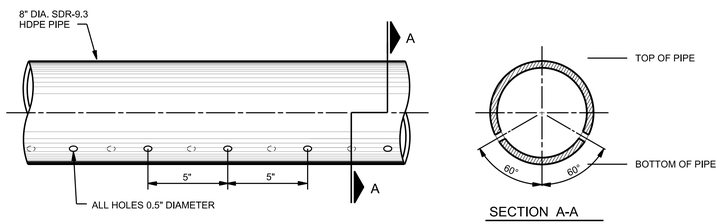
NOTE:

1. PRIMARY 60-MIL HDPE GEOMEMBRANE OVER PHASE I SIDESLOPE IS SINGLE-SIDED TEXTURED. SMOOTH SIDE OF GEOMEMBRANE IS UP AND IN DIRECT CONTACT WITH NON-WOVEN CUSHION GEOTEXTILE FOR THE GRANULAR DRAINAGE LAYER. EXTEND PRIMARY SINGLE-SIDED TEXTURED GEOMEMBRANE 5 FEET PAST TOE OF SLOPE WHERE EXTRUSION WELDED TO PRIMARY DOUBLE-SIDED TEXTURED GEOMEMBRANE.
2. SECONDARY 60-MIL HDPE GEOMEMBRANE OVER PHASE 1 SIDESLOPE IS SINGLE-SIDED TEXTURED. SMOOTH SIDE OF GEOMEMBRANE IS DOWN AND IN CONTACT WITH NON-WOVEN CUSHION GEOTEXTILE. EXTEND SECONDARY SINGLE-SIDED TEXTURED GEOMEMBRANE 5 FEET PAST TOE OF SLOPE WHERE EXTRUSION WELDED TO SECONDARY DOUBLE-SIDED TEXTURED GEOMEMBRANE.
3. SECONDARY 60-MIL HDPE GEOMEMBRANE ON PHASE II SIDESLOPE AND CELL 2 BASE IS TEXTURED ON BOTH SIDES.
4. PRIMARY 60-MIL HDPE GEOMEMBRANE ON PHASE II SIDESLOPE AND CELL 2 BASE IS TEXTURED ON BOTH SIDES.
5. REINFORCEMENT GRID TO BE INSTALLED ON PHASE I SIDESLOPES IN SUBCELLS A AND D. REFER TO APPENDIX F OF ENGINEERING REPORT FOR MATERIAL PROPERTIES. REFER TO DRAWING NO. 2 FOR LOCATIONS.

BASE LINER DETAIL 1
NTS



LEACHATE COLLECTION SYSTEM 2
NTS

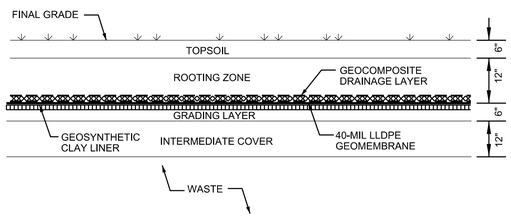


SECTION A-A

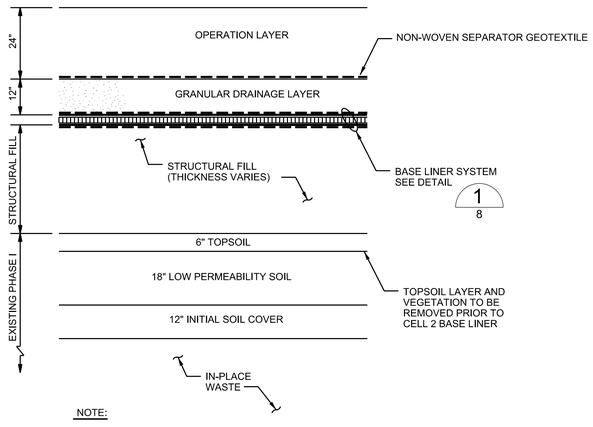
NOTE:

1. ALL HOLES TO BE ALTERNATELY STAGGERED.

PERFORATED LEACHATE COLLECTION PIPE 3
NTS



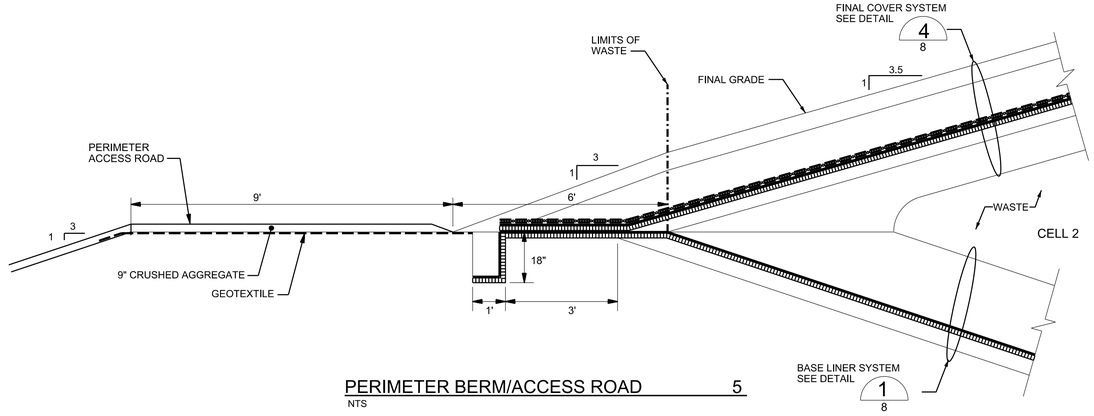
FINAL COVER SYSTEM 4
NTS



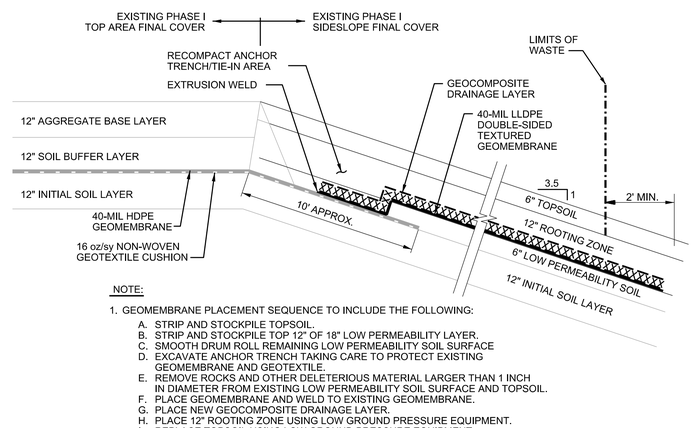
NOTE:

1. EXISTING PHASE 1 SIDESLOPE FINAL COVER CONFIGURATION TAKEN FROM DRAWING C-10 "POST CONSTRUCTION REPORT, KEKAHA SANITARY LANDFILL, PHASE I CLOSURE, KEKAHA, KAUAI, HAWAII", DATED FEBRUARY 16, 1996 BY HARDING LAWSON ASSOCIATES.

OVERLAY BASE LINER DETAIL (ONTO PHASE I) 6
NTS



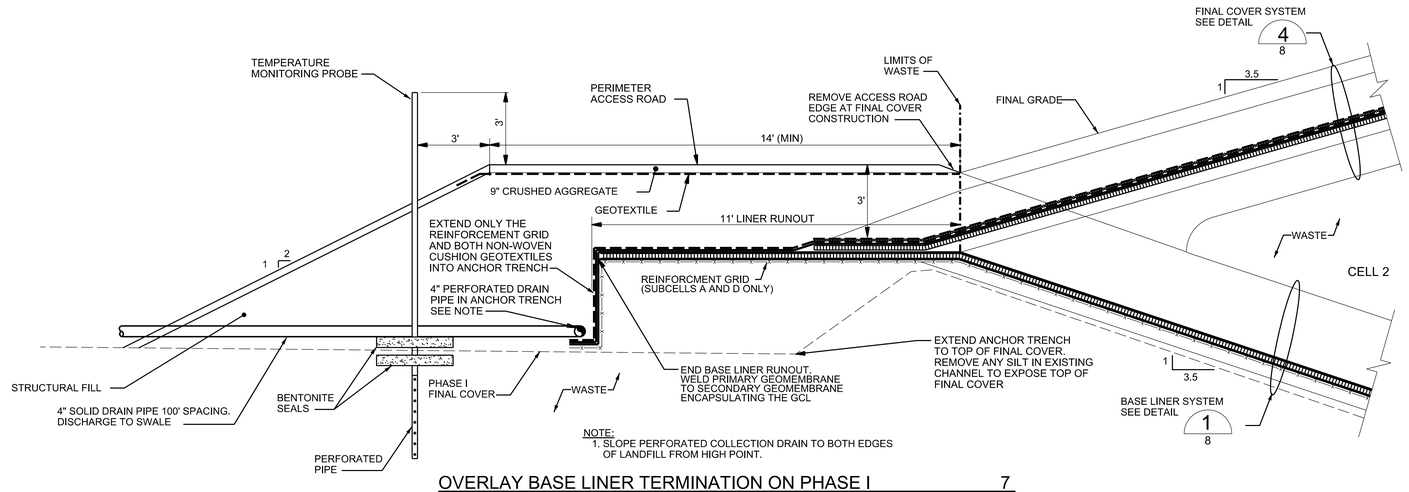
PERIMETER BERM/ACCESS ROAD 5
NTS



NOTE:

1. GEOMEMBRANE PLACEMENT SEQUENCE TO INCLUDE THE FOLLOWING:
   A. STRIP AND STOCKPILE TOPSOIL.
   B. STRIP AND STOCKPILE TOP 12" OF 18" LOW PERMEABILITY LAYER.
   C. SMOOTH DRUM ROLL REMAINING LOW PERMEABILITY SOIL SURFACE
   D. EXCAVATE ANCHOR TRENCH TAKING CARE TO PROTECT EXISTING GEOMEMBRANE AND GEOTEXTILE.
   E. REMOVE ROCKS AND OTHER DELETERIOUS MATERIAL LARGER THAN 1 INCH IN DIAMETER FROM EXISTING LOW PERMEABILITY SOIL SURFACE AND TOPSOIL.
   F. PLACE GEOMEMBRANE AND WELD TO EXISTING GEOMEMBRANE.
   G. PLACE NEW GEOCOMPOSITE DRAINAGE LAYER.
   H. PLACE 12" ROOTING ZONE USING LOW GROUND PRESSURE EQUIPMENT.
   I. REPLACE TOPSOIL USING LOW GROUND PRESSURE EQUIPMENT.
2. WHEN DEPLOYING GEOCOMPOSITE DRAINAGE LAYER OVER TEXTURED GEOMEMBRANE, A RUBSHEET SHALL BE USED BETWEEN THE TWO MATERIALS AND REMOVED AFTER FINAL MATERIAL POSITIONING.
3. LIMITS OF EXISTING GEOMEMBRANE TO BE FIELD LOCATED AND VERIFIED.
4. PLACEMENT SEQUENCE, LIMITS, AND CONFIGURATION MAY BE MODIFIED AT TIME OF CONSTRUCTION BASED ON FIELD CONDITIONS AND ENGINEER'S APPROVAL.

PHASE I GEOMEMBRANE EXTENSION 8
NTS



NOTE:

1. SLOPE PERFORATED COLLECTION DRAIN TO BOTH EDGES OF LANDFILL FROM HIGH POINT.

OVERLAY BASE LINER TERMINATION ON PHASE I 7
NTS

NO	DATE	CHK	REVISIONS
4	12/2015	NKW	MODIFIED DETAIL 1 NOTES, MODIFIED DETAILS 7 AND 8
3	03/2013	KJB	MODIFIED DETAIL 1 NOTES, ADDED DETAIL 8 AND MODIFIED DETAIL 6
2	01/2013	KJB	ADDED GCL TO COVER CROSS-SECTION ON DETAILS 4,5, AND 7
1	12/2012	KJB	UPDATED DETAILS, NOTES, MATERIAL SELECTION AND ANCHOR TRENCHING.
		DRN	CFF

NANCY K. WRIGHT LICENSED PROFESSIONAL ENGINEER No. 16394-C HAWAII U.S.A.

THIS WORK WAS PREPARED BY ME OR UNDER MY SUPERVISION.

SIGNATURE

APRIL 30, 2018
EXPIRATION DATE OF THE LICENSE

DRN	DRB	
DES	NKW/TCR	
CHK	FLC/MRH	
APP	KJB	

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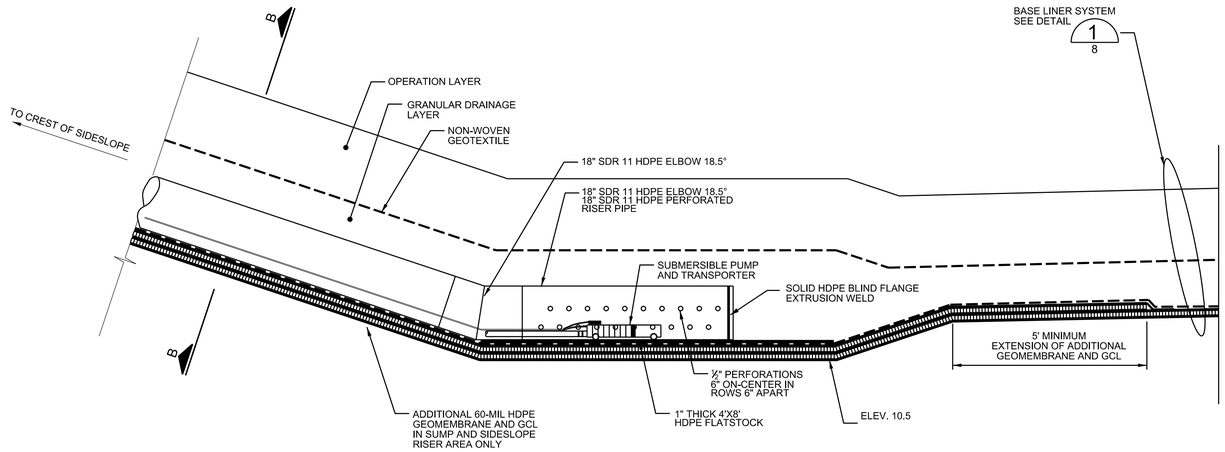
PREPARED BY AECOM

ENGINEERING REPORT
KEKAHA LANDFILL PHASE II-CELL 2 LATERAL EXPANSION KEKAHA SANITARY LANDFILL
KAUAI, HAWAII

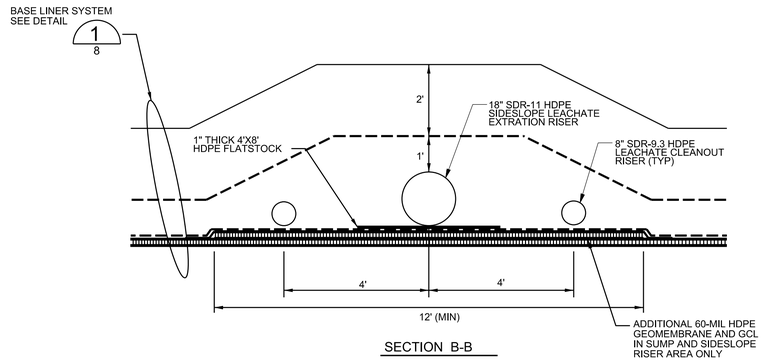
DETAILS

DATE DECEMBER 2015
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SECTION A-A



SECTION B-B

NOTES:

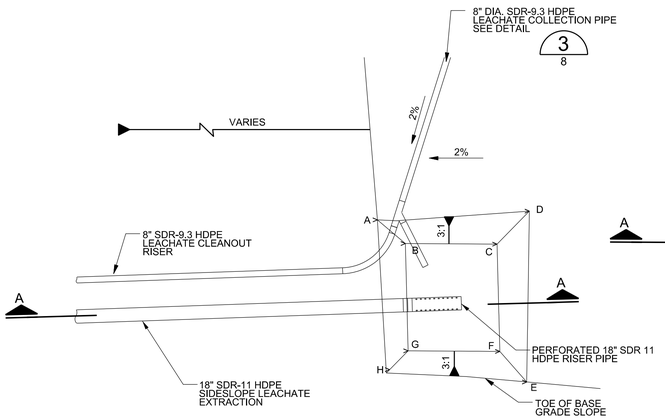
1. HDPE FLATSTOCK SHEET EDGES SHALL BE BEVELED AND CORNERS ROUNDED.

TYPICAL LEACHATE COLLECTION SUMP 1

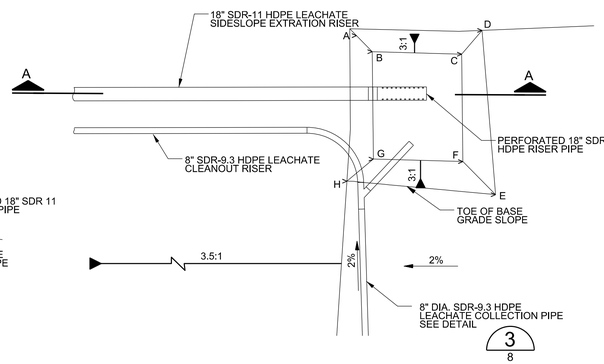
NTS

Collection Sump Subgrade Control Points

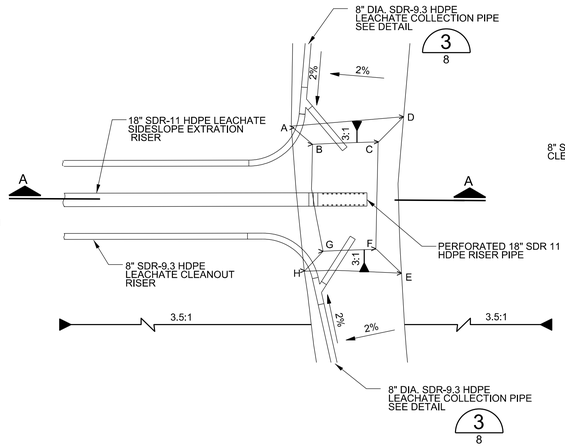
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B	1555804.61	55007.25	10.50	B	1555857.96	54952.25	10.50	B	155292.54	54530.79	10.50	B	1556728.43	54308.31	10.50
C	1555811.65	55014.70	10.50	C	1555864.99	54959.38	10.50	C	155297.38	54536.38	10.50	C	1556731.26	54318.14	10.50
D	1555811.42	55019.86	11.74	D	1555864.59	54962.86	11.27	D	155297.23	54540.35	11.41	D	1556729.32	54322.72	11.54
E	1555825.19	55006.62	11.47	E	1555879.06	54951.44	11.71	E	155309.87	54528.24	11.42	E	1556745.44	54313.78	11.45
F	1555820.64	55006.74	10.50	F	1555873.86	54951.28	10.50	F	1556305.9	54528.01	10.50	F	1556742.08	54312.36	10.50
G	1555813.60	54999.29	10.50	G	1555866.83	54944.16	10.50	G	155302.09	54523.50	10.50	G	1556739.25	54302.53	10.50
H	1555813.75	54995.82	11.21	H	1555866.62	54940.31	11.35	H	155302.29	54520.44	11.21	H	1556740.50	54298.92	11.23



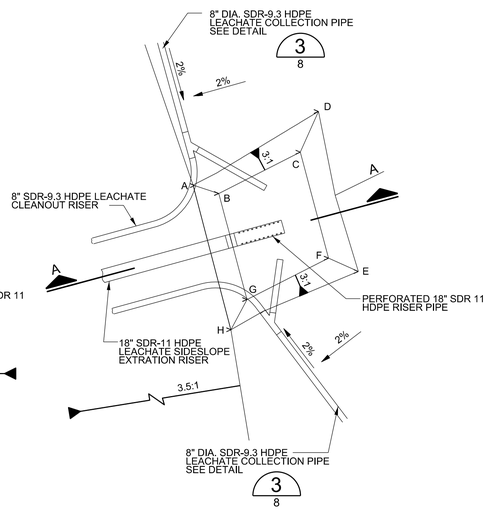
CELL A - PLAN



CELL B - PLAN



CELL C - PLAN



CELL D - PLAN

NO	REVISIONS	DRN	CHK	DATE
2	REVISED SUMP DETAILS AND PLAN VIEWS	DRB	KJB	12/2015
1	UPDATE GENERAL SUMP CONFIGURATIONS	DRB	KJB	12/2012

NANCY K. WRIGHT
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No. 16394-C
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DRN	DRB	
DES	NKW/TCR	
CHK	FLC/MRH	
APP	KJB	

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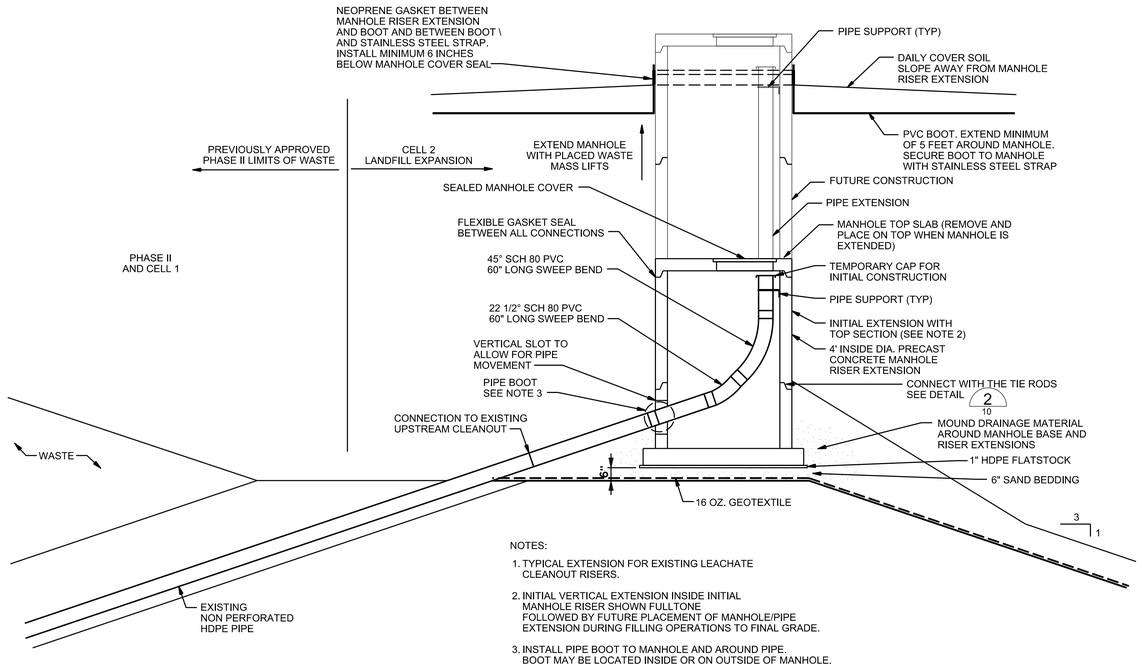
PREPARED BY AECOM

ENGINEERING REPORT
KEKAHA LANDFILL PHASE II-CELL 2 LATERAL EXPANSION KEKAHA SANITARY LANDFILL
KAUAI, HAWAII

DETAILS

DATE	DECEMBER 2015
PROJECT NO	60197394
FILENAME	
SHEET NO	
DRAWING NO	9

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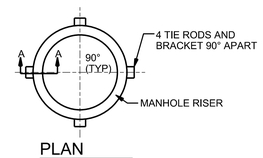


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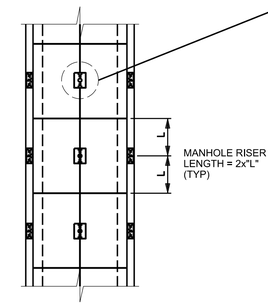
1. TYPICAL EXTENSION FOR EXISTING LEACHATE CLEANOUT RISERS.
2. INITIAL VERTICAL EXTENSION INSIDE INITIAL MANHOLE RISER SHOWN FULLTONE FOLLOWED BY FUTURE PLACEMENT OF MANHOLE/PIPE EXTENSION DURING FILLING OPERATIONS TO FINAL GRADE.
3. INSTALL PIPE BOOT TO MANHOLE AND AROUND PIPE. BOOT MAY BE LOCATED INSIDE OR ON OUTSIDE OF MANHOLE.

VERTICAL LEACHATE CLEANOUT AND MANHOLE RISER EXTENSIONS 1

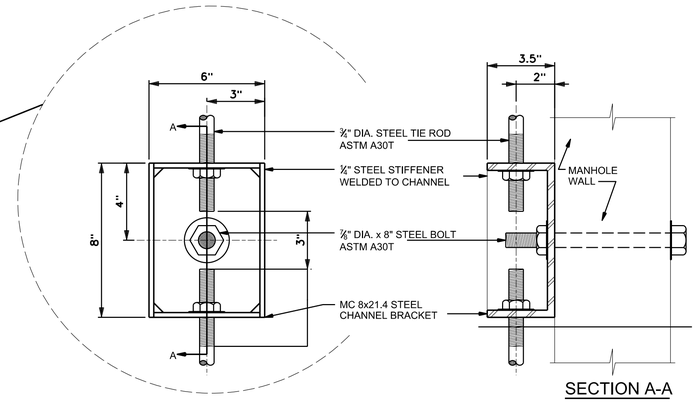
NTS



PLAN



ELEVATION



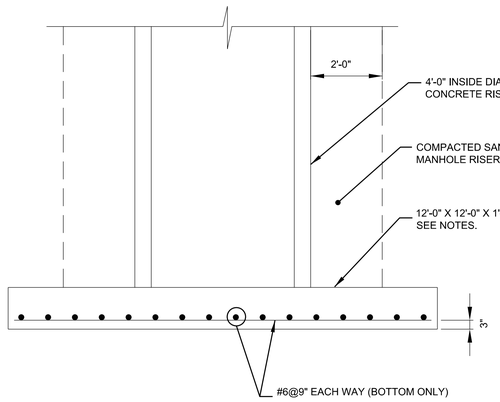
SECTION A-A

NOTES:

1. MAINTAIN ALL MANHOLE RISER SECTIONS LEVEL AND PLUMB. PROVIDE CAST IRON SPACERS BETWEEN RISER SECTIONS IF NECESSARY TO ADJUST TO LEVEL CONDITIONS.
2. EPOXY COAT ALL EXPOSED METAL SURFACES AND BOLTS IMMEDIATELY FOLLOWING INSTALLATION .
3. STAINLESS STEEL CAN BE SUBSTITUTED FOR ALL COMPONENTS.

MANHOLE TIE ROD AND BRACKET 2

NTS



MANHOLE BASE DETAIL 3

NTS

NOTES:

1. CONCRETE PAD

A. ACTUAL PAD DIMENSIONS AND REINFORCEMENT MAY VARY AT TIME OF CONSTRUCTION DUE TO FIELD CONDITIONS.

2. CONCRETE MIX

A. CLASS A:
   1. MINIMUM 6 BAGS OF CEMENT PER CUBIC YARD OF CONCRETE.
   2. MINIMUM 28 DAY COMPRESSIVE STRENGTH OF 4000 PSI.
B. FURNISH AND DELIVER CONCRETE IN ACCORDANCE WITH ASTM C94.
C. AIR CONTENT 6 +/- 1 %, ASTM C260.
D. SLUMP: 4 +/- 1 IN.
E. WATER/CEMENT RATIO: 0.45 MAXIMUM, MIXING WATER TO BE POTABLE.
F. CEMENT: ASTM C150 TYPE 1.
G. AGGREGATES: FINE TO BE NATURAL SAND, COARSE TO BE CRUSHED GRAVEL ASTM C33, SIZE (3/4 INCH MAXIMUM).
H. APPLY ASTM C309 TYPE 1 OR 1-D CURING COMPOUND IN ACCORDANCE WITH MANUFACTURER'S RECOMMENDATIONS, CURE AND SEAL J-20 BY DAYTON SUPERIOR OR EQUAL.
I. EXPOSED CONCRETE SURFACES TO HAVE SURFACE DEFECTS PATCHED AND PROJECTING FINS KNOCKED OFF.
J. PATCH HONEYCOMBING, STONE POCKETS, SPALLS, AND OTHER IRREGULARITIES WITH PATCHING MORTAR, SIKATOP BY SIKA CORPORATION OR EQUAL.
K. EXTERIOR EXPOSED SLAB TO HAVE A FLOAT AND BROOM FINISH.

3. REINFORCEMENT

A. DEFORMED BARS CONFORMING TO ASTM A615, GRADE 60.
B. DO NOT WELD OR FIELD BEND REINFORCING BARS.
C. #6 REINFORCING BAR:
   1. LAPPED SPLICE LENGTH = 30 INCHES
   2. EMBEDMENT LENGTH = 24 INCHES.

NO	REVISIONS	DRN	CHK	DATE
		CFF	NKW	12/2015
2	REMOVED DETAIL 5 LEACHATE TRANSFER PIPE TIE-IN TO LCM-1	DRB	KJB	10-23-12
1	ADD PVC PIPE BOOT TO MANHOLE. DETAIL 1	DRN	CHK	



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APP	KJB	

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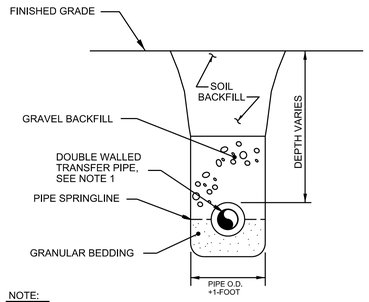
PREPARED BY AECOM

ENGINEERING REPORT
KEKAHA LANDFILL PHASE II-CELL 2 LATERAL EXPANSION KEKAHA SANITARY LANDFILL
KAUAI, HAWAII

DETAILS

DATE	DECEMBER 2015
PROJECT NO	60197394
FILENAME	
SHEET NO	
DRAWING NO	10

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NOTE:

1. TRANSFER PIPE TO BE DOUBLE-WALLED PIPE, PIPE FROM CELL 2 SUMPS TO COMBINED FORCE MAIN TO BE 2 INCH SDR-11 HDPE CARRIER PIPE INSIDE 6-INCH SDR-17 HDPE CONTAINMENT PIPE. COMBINED FORCE MAIN TO BE 4 INCH SDR-11 HDPE CARRIER PIPE INSIDE 8-INCH SDR-17 HDPE CONTAINMENT PIPE.
2. IN AREA OF PHASE I COVER, LEACHATE PIPE WILL BE ABOVE GRADE.

FORCEMAIN LEACHATE TRANSFER PIPE (TYP) 1

NTS

NO	REVISIONS	DRN	CHK	DATE
3	REMOVED STORAGE TANK DETAILS AND CONTAINMENT DETAILS 1, 2, 3 AND 6	CFF	NKW	12/2015
2	UPDATED DETAILS 1, 2, AND 3. ADD DETAIL 6	DRB	KJB	03/2012
1	UPDATED DETAILS 1 AND 2. DIMENSIONS AND NOTES.	DRB	KJB	12/2012

NANCY K. WRIGHT
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DES	NKW/TCR	
CHK	FLC/MRH	
APP	KJB	

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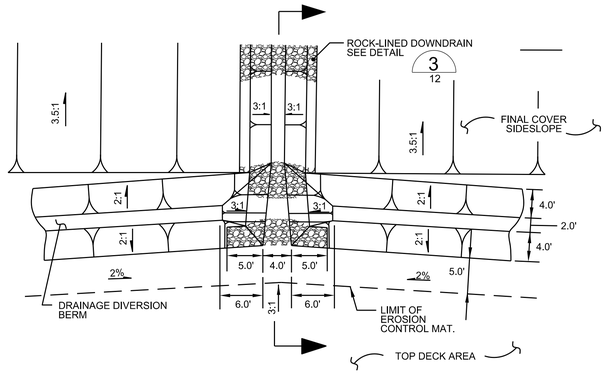
PREPARED BY AECOM

ENGINEERING REPORT
KEKAHA LANDFILL PHASE II-CELL 2 LATERAL EXPANSION KEKAHA SANITARY LANDFILL
KAUAI, HAWAII

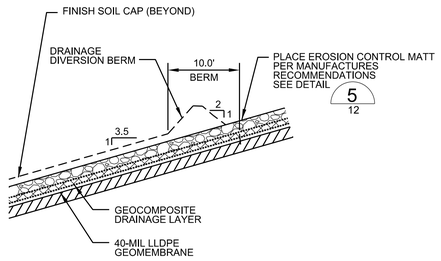
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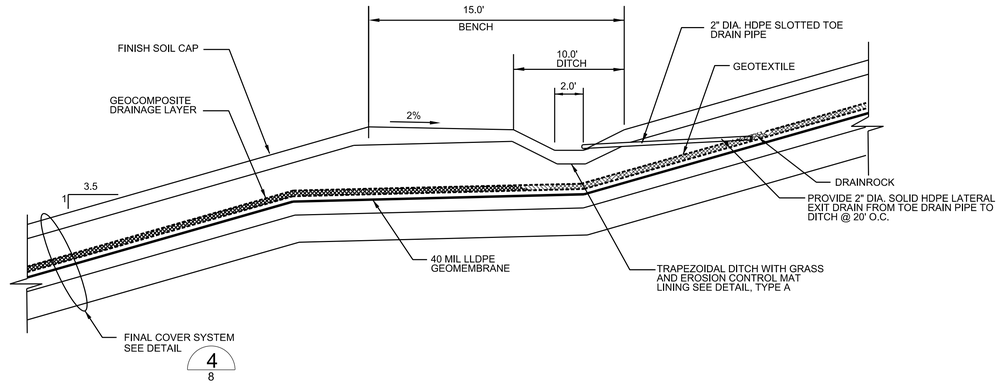
PLAN



PROFILE

DIVERSION BERM/DOWNDRAIN TRANSITION 1

NTS



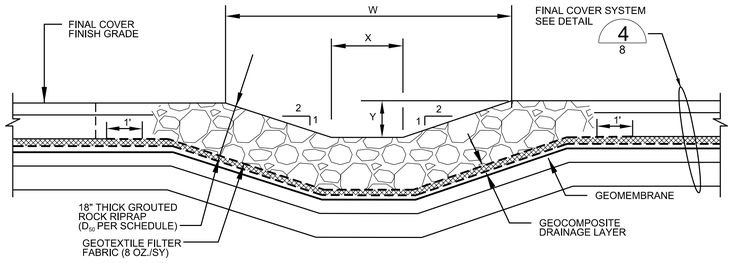
FINAL COVER BENCH PROFILE DETAIL 2

NTS

DOWNDRAIN SCHEDULE

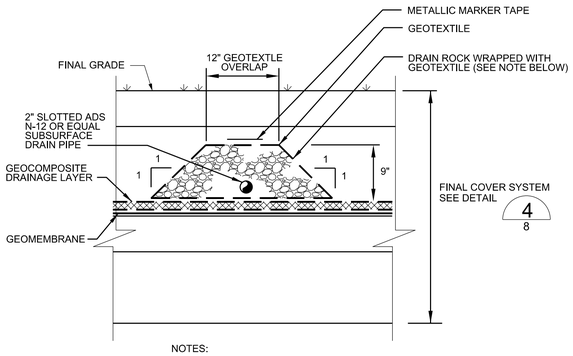
FLUME	DIMENSION W (FT)	X (FT)	Y (FT)	RIPRAP SIZE (IN) D_{MIN}	D_{50}	D_{MAX}
A1	5.0	1.0	1.0	3	6	9
A2	5.0	1.0	1.0	3	6	9
B1	6.0	2.0	1.0	3	6	9
B2	5.0	1.0	1.0	3	6	9
C2	5.0	1.0	1.0	3	6	9
C3	6.0	2.0	1.0	5	9	12
F2	5.0	1.0	1.0	3	6	9
F3	6.0	2.0	1.0	3	6	9

NOTE:
DOWNDRAIN LETTERS REFER TO FLUME DESIGNATIONS ON DRAWING NO. 5



ROCK-LINED DOWNDRAIN DETAIL 3

NTS

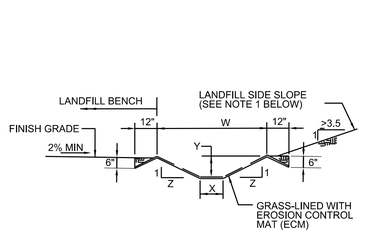


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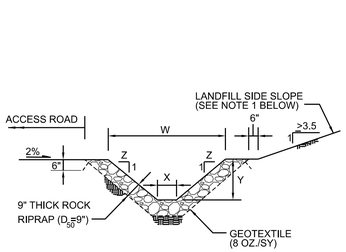
1. DRAIN ROCK PERMEABILITY MUST BE EQUAL TO OR GREATER THAN 1.0 CM/SEC.
2. DAYLIGHT 2" SUBSURFACE DRAIN PIPE INTO ROCK-LINED DOWNDRAIN

PIPE SUBDRAIN DETAIL 4

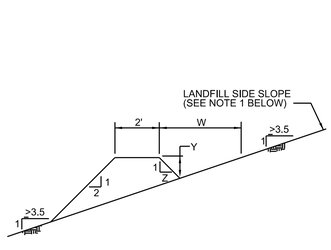
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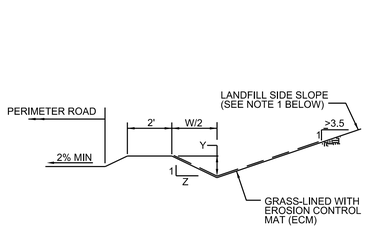
TYPE A DITCH, BENCH/TERRACE



TYPE B DITCH, ACCESS ROAD



TYPE C DITCH, SIDESLOPE DIVERSION BERM



TYPE D DITCH, PERIMETER ROAD

NOTE:

1. PLACE HAY BALES OR SILT FENCE ADJACENT TO ALL DITCHES TO PREVENT SILT AND SEDIMENT FROM WASHING INTO DITCH UNTIL VEGETATION IS ESTABLISHED.

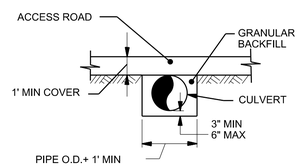
TYPE	DIMENSION W (FT)	X (FT)	Y (FT)	SIDE SLOPE Z	SHAPE	LINING
A	10.0	2.0	2.0	2.0	TRAPEZOIDAL	GRASS-LINED WITH ECM
B	5.5	1.0	1.5	1.5	TRAPEZOIDAL	ROCK RIPRAP
C	11	0	2.0	2.0	V-SHAPED	GRASS-LINED WITH ECM
D	16	0	4.0	2.0	V-SHAPED	GRASS-LINED WITH ECM

DRAINAGE DITCH DETAIL 5

NTS

CULVERT SCHEDULE

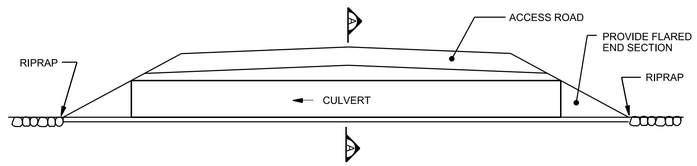
CULVERT IDENTIFICATION	NUMBER OF CILVERTS	CULVERT DIAMETER (INCHES)	MINIMUM SLOPE %
B	1	18	6
C-1	1	12	5
C-2	3	18	6
F	5	18	3
G	3	15	3



SECTION A-A

NOTES:

1. PLACE RIPRAP AT CULVERT INLET AND OUTLET LOCATIONS.
2. INSTALL FLARED END SECTIONS AT BOTH INLET AND OUTLET ENDS.
3. FIELD LOCATE EXISTING LINER ANCHOR TRENCH OF PHASE II AND DO NOT DISTURB.
4. PROVIDE MINIMUM 1.0' COVER OVER CULVERT AT ROAD EDGE.
5. A STRUCTURE HAVING EQUIVALENT HYDRAULIC PERFORMANCE MAY BE SUBSTITUTED SUBJECT TO REVIEW AND APPROVAL.



CULVERT PROFILE

CULVERT INSTALLATION DETAIL 6

NTS

NO	REVISIONS	DRN	CFF	DATE
1	REVISED DETAILS 1, 3, 5. NOTES AND SCHEDULES	NKW	CHK	12/2015

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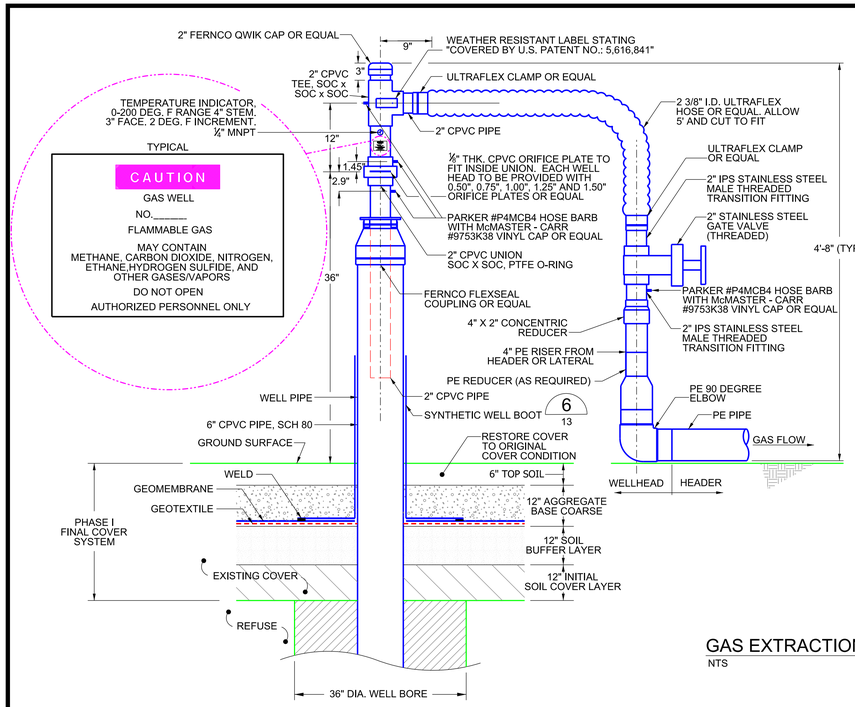
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ENGINEERING REPORT
KEKAHA LANDFILL PHASE II-CELL 2 LATERAL EXPANSION KEKAHA SANITARY LANDFILL
KAUAI, HAWAII

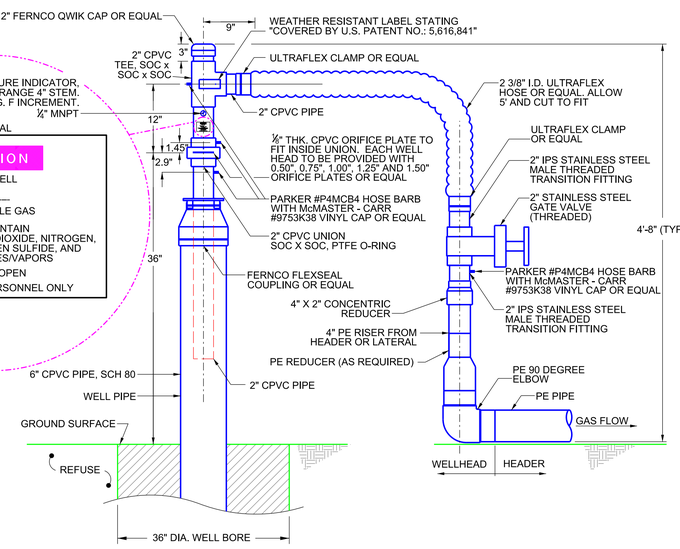
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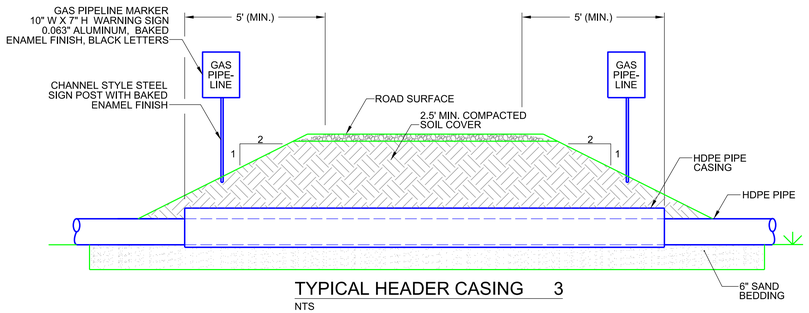
PHASE I



PHASE II

GAS EXTRACTION WELLHEAD DETAIL 1

NTS

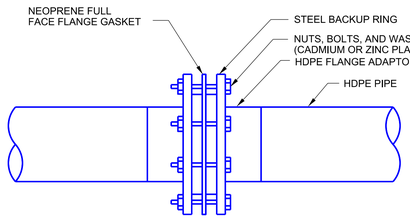


TYPICAL HEADER CASING 3

NTS

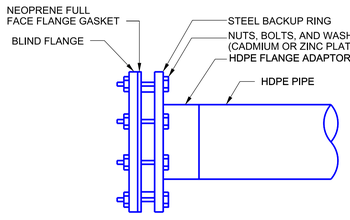
NOTES:

1. IN AREAS WHERE A HEADER CASING IS REQUIRED, USE HDPE PIPE CASING TWO PIPE SIZES LARGER THAN HEADER PIPE.
2. HEADER AND LATERAL PIPES TO BE REPAIRED OR REPLACED WHERE A HEADER CASING IS REQUIRED SHALL BE INSPECTED AND IF NECESSARY EXCAVATED. IF PIPES ARE FOUND TO BE IN GOOD CONDITION THEY CAN REMAIN OTHERWISE A NEW PIPE AND CASING IS REQUIRED.
3. LOCATION OF ALL HEADER CASINGS TO BE FIELD DETERMINED BY OWNER.
4. GRADE HEADER CASING CROSS SLOPE TO ORIGINAL ROAD OR GRADE AT A MAXIMUM 8% SLOPE.



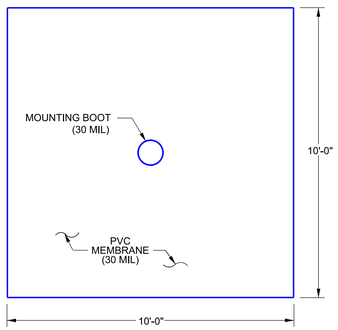
FLANGE CONNECTION (TYP) 4

NTS



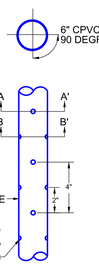
BLIND FLANGE (TYP) 5

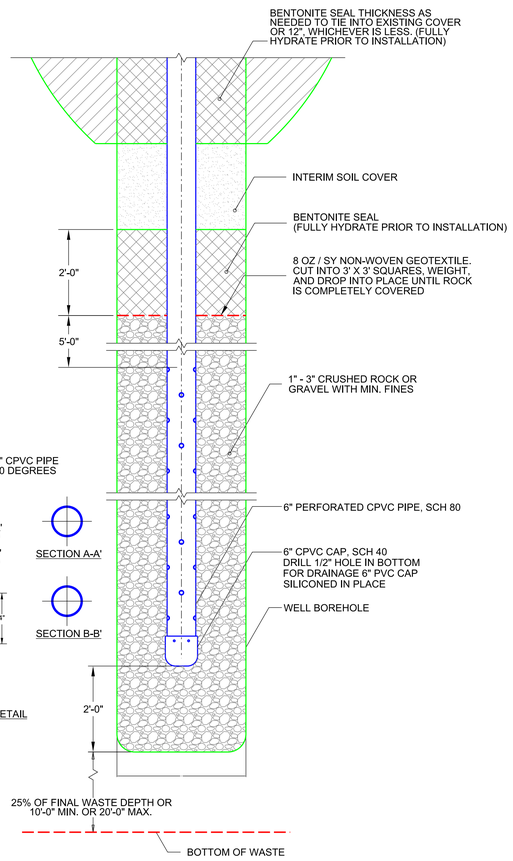
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SYNTHETIC WELL BOOT 6

NTS



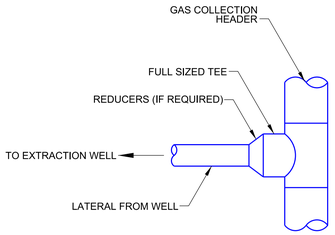


GAS EXTRACTION WELL 2

NTS

NOTES:

1. LANDFILL GAS AND CONDENSATE SYSTEM MAY CHANGE DUE TO SITE CONDITIONS AT THE TIME OF CONSTRUCTION.
2. ALL PVC PIPE AND FITTINGS ARE SCH 80 EXCEPT AS NOTED.
3. ALL HDPE PIPE AND FITTINGS GREATER THAN 12" IS SDR-26 AND ALL HDPE PIPE AND FITTINGS 12" OR LESS IS SDR-17.
4. ALL NUTS, BOLTS, AND WASHERS ARE CADMIUM PLATED UNLESS OTHERWISE NOTED.
5. COMPACTED CLEAN SAND BEDDING AND BACKFILL IN MAXIMUM LIFTS OF 9".



LATERAL CONNECTION DETAIL 7

NTS

NOTES:

1. ALL HDPE PIPE AND FITTINGS GREATER THAN 12" IS SDR-26 AND ALL HDPE PIPE AND FITTINGS 12" OR LESS IS SDR-17.
2. ALL CPVC PIPE AND FITTINGS ARE SCH 80 UNLESS OTHERWISE NOTED. HDPE PIPE MAY BE USED IN PLACE OF CPVC AS DIRECTED BY COUNTY ENGINEER AND DESIGN ENGINEER.
3. CLEAN SAND OR SELECT SITE MATERIAL BEDDING AND BACKFILL COMPACTED IN MAXIMUM LIFTS OF 9".
4. REPAIR COVER TO ORIGINAL CONDITIONS AND SPECIFICATIONS.
5. ALL NUTS, BOLTS AND WASHERS ARE CADMIUM OR ZINC PLATED UNLESS OTHERWISE STATED.
6. DIMENSIONS, LOCATION & SIZE OF SYSTEM COMPONENTS ARE SUBJECT TO CHANGE IN THE FIELD DURING CONSTRUCTION.
7. SYSTEM COMPONENTS MAY HAVE SUBSTITUTE COMPONENTS INSTALLED OF EQUAL VALUE & CAPABILITY.

NO	REVISIONS	DRN	CHK	DATE

DRN	DES	CHK	APP	INITIAL	DATE

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STAMP

ANDREW J. QUERIO
LICENSED PROFESSIONAL ENGINEER
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EXP. 4/30/2018

PREPARED FOR

COUNTY OF KAUAI KEKAHA SANITARY LANDFILL

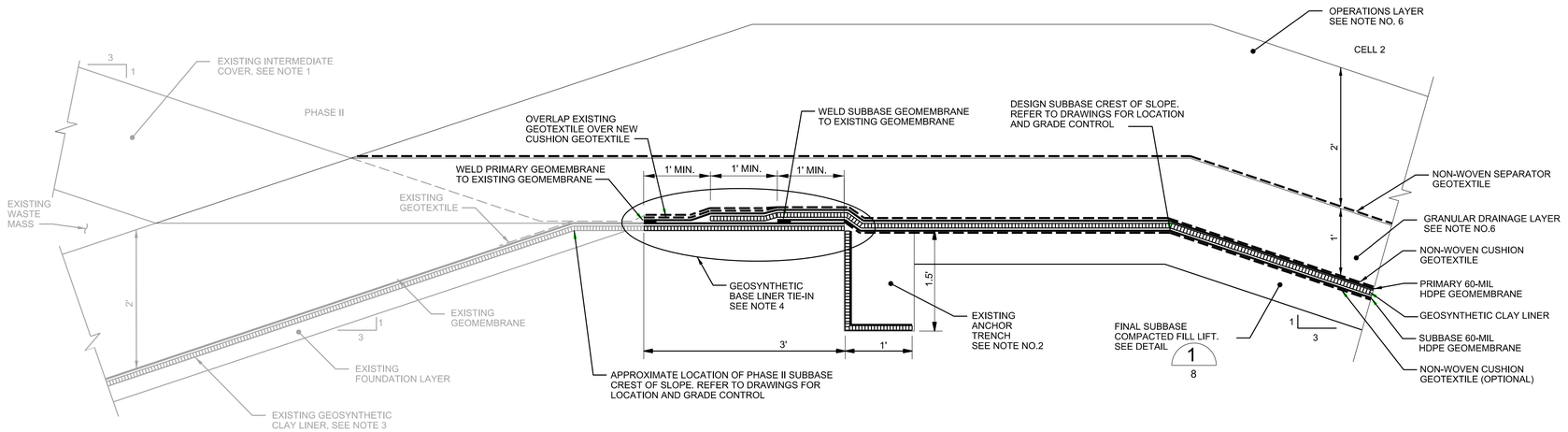
PREPARED BY

ENVIRONMENTAL INFORMATION LOGISTICS, LLC

KEKAHA SANITARY LANDFILL
KEKAHA, KAUAI, HAWAII

GAS MANAGEMENT SYSTEM DETAILS

DATE	OCTOBER 2017
PROJECT NO	090605
FILENAME	013 KEK ADD DET1 101917
SHEET NO	13
DRAWING NO	13



NOTES:

1. CONTRACTOR TO EXPOSE EXISTING ANCHOR TRENCH, GEOSYNTHETIC BASE LINER AND DRAINAGE LAYER OF PHASE II. COORDINATE WITH OWNER ACCESS TO TIE-IN AREA TO AVOID CONFLICT WITH SITE OPERATION. EXPOSURE ACTIVITIES WILL INCLUDE AT A MINIMUM REMOVAL OF EXISTING INTERMEDIATE COVER, EXISTING SOILS OVERLYING THE ANCHOR TRENCH AND BERM. CARE IS TO BE TAKEN DURING REMOVAL OF SOILS DIRECTLY IN CONTACT GEOSYNTHETICS. EXPOSURE ACTIVITIES TO BE CONSTANTLY MONITORED BY QUALITY ASSURANCE CONSULTANT AND DOCUMENTED. EXCESS DAMAGE AND RESULTING REPAIRS CAUSED FROM EXPOSING TIE-IN AREA WILL BE RESPONSIBILITY OF CONTRACTOR. CONTRACTOR TO MAINTAIN SURFACE WATER CONTROL FROM TIE-IN AREA.
2. CUT GEOSYNTHETICS ENTERING ANCHORING TRENCH AT ANCHOR TRENCH EDGE. PROTECT EXPOSED EDGE FROM DRYING AND MOISTURE. REMOVE ALL REMAINING PORTIONS OF GEOSYNTHETICS FROM ANCHOR TRENCH. BACKFILL ANCHOR TRENCH WITH STRUCTURAL FILL AND COMPACT PER PROJECT SPECIFICATIONS.
3. EXISTING GEOSYNTHETIC CLAY LINER (GCL) OF PHASE II IS COMPRISED OF A BENTONITE LAYER WITH A GEOMEMBRANE CARRIER. THE GEOMEMBRANE IS IN DIRECT CONTACT WITH THE SUBBASE. CARE TO BE TAKEN NOT TO PULL GCL FROM OVERLYING GEOMEMBRANE DURING TIE-IN CONSTRUCTION.
4. CONSTRUCT GEOSYNTHETIC BASE LINER TIE-IN IN THE FOLLOWING ORDER:
   A. INSTALL NEW SUBBASE GEOMEMBRANE AND WELD TO EXISTING GEOMEMBRANE WITH A MINIMUM 1-FOOT OVERLAP.
   B. INSTALL NEW GEOSYNTHETIC CLAY LINER (GCL) ON TOP OF SUBBASE GEOMEMBRANE WITH MINIMUM 2-FOOT OVERLAP ONTO EXISTING GEOMEMBRANE.
   C. INSTALL NEW PRIMARY GEOMEMBRANE OVER NEW GCL AND WELD TO EXISTING GEOMEMBRANE MINIMUM 1-FOOT PAST NEW GCL OVERLAP.
   D. INSTALL NEW NON-WOVEN CUSHION GEOTEXTILE OVER PRIMARY GEOMEMBRANE. OVERLAP EXISTING GEOTEXTILE ON NEW NON-WOVEN GEOTEXTILE A MINIMUM OF 2 FEET AND HEAT BURNISH FIELD OVERLAP SEAM. REPAIR THINNED OR TORN PORTIONS OF THE EXISTING GEOTEXTILE WITH NEW NON-WOVEN GEOTEXTILE AS NEEDED.
   E. INSTALL GRANULAR DRAINAGE LAYER.
   F. INSTALL NEW NON-WOVEN SEPARATOR GEOTEXTILE.
   G. INSTALL OPERATIONS LAYER.
5. WHEN DEPLOYING GCL OR GEOTEXTILES OVER TEXTURED GEOMEMBRANE OR WHEN DEPLOYING TEXTURED GEOMEMBRANE OVER GCL OR GEOTEXTILES, A RUBSHEET SHALL BE USED BETWEEN THE TWO MATERIALS AND REMOVED AFTER FINAL MATERIAL POSITIONING.
6. INSTALL GRANULAR DRAINAGE LAYER AND OPERATIONS LAYER OVER TIE-IN AREA PRIOR TO PLACEMENT OF MATERIALS ON SIDESLOPES. PLACEMENT OF MATERIALS ON SIDESLOPES TO OCCUR FROM TOE OF SLOPE UP SIDESLOPE.
7. ALL NEW GEOMEMBRANE IS TEXTURED ON BOTH SIDES.

PHASE II / CELL I TO CELL 2 BASE LINER TIE-IN DETAIL 1

NTS

NO	DATE	CHK
2	12/2015	NKW
1	03/2013	KJB
REVISIONS		DRB
NEW DRAWING, DETAIL 1 MOVED FROM DRAWING 9		CFF
REMOVED DETAIL 2		DRN

NANCY K. WRIGHT
LICENSED PROFESSIONAL ENGINEER
No. 16394-C
HAWAII, U.S.A.

THIS WORK WAS PREPARED BY ME OR UNDER MY SUPERVISION.

SIGNATURE

APRIL 30, 2018
EXPIRATION DATE OF THE LICENSE

DRN	DRB	
DES	NKW/TCR	
CHK	FLC/MRH	
APP	KJB	

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PREPARED BY AECOM

ENGINEERING REPORT
KEKAHA LANDFILL PHASE II-CELL 2 LATERAL EXPANSION KEKAHA SANITARY LANDFILL
KAUAI, HAWAII

DETAILS

DATE DECEMBER 2015
PROJECT NO 60197394
FILENAME
SHEET NO
DRAWING NO 14

10/19/2017 10:05:10 AM
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CAD File = L:\Groups\ed60\197394 Kekaha Cell 2\14Details.dgn

Appendix A
Geotechnical Exploration Letter Report

FINAL LETTER REPORT

GEOTECHNICAL EXPLORATION

PGE Job No. 8950-023

for

AECOM

KEKAHA LANDFILL PHASE II
CELL 2 LATERAL EXPANSION
KEKAHA, KAUAI, HAWAII

August 8, 2012

Submitted by:

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August 8, 2012
Job No. 8950-023

AECOM
Bishop Square, American Savings Bank Tower
1001 Bishop Street, Suite 1600
Honolulu, Hawaii 96813

Attention: Mr. Frank Cioffi, P.E.

Subject: Final Letter Report
Geotechnical Exploration
Kekaha Landfill Phase II
Cell 2 Lateral Expansion
Kekaha, Kauai, Hawaii

Gentlemen:

1.0 INTRODUCTION

This letter report presents the results of the geotechnical exploration that Pacific Geotechnical Engineers, Inc. (PGE) performed for the proposed Kekaha Landfill Phase II, Cell 2 Lateral Expansion project at Kekaha, Kauai, Hawaii. Our services were performed in general accordance with our February 1, 2011 revised proposal. A pre-final version of this letter report dated March 7, 2012 was previously submitted. This letter report incorporates our responses to AECOM's review comments regarding our pre-final letter report.

The approximate location of the project site is shown on the Map of Area, Plate 1.

2.0 PROJECT CONSIDERATIONS

This project includes the Phase II lateral expansion, designated Cell 2, of the Kekaha Landfill at Kekaha, Kauai, Hawaii. According to AECOM's Preliminary Engineering Report (PER) we received on January 11, 2012, the Cell 2 lateral expansion will include an area between the existing Phase I and Phase II landfills on the south side of the existing Phase II landfill and recent Cell 1A expansion. We understand that an initial permitted height of approximately 85 feet above Mean Sea Level (MSL) is planned for the Cell 2 lateral expansion.

According to AECOM's PER, the expansion will include a granular drainage layer with subbase liner to elevations ranging from about +11 to +21 feet MSL. A leachate collection system consisting of collection piping, sumps, holding tanks, and manholes is also included.

According to AECOM's February 1, 2011 revised draft statement of work, a geotechnical exploration is needed for the design of the lateral landfill expansion. It was specified that the geotechnical exploration include conducting a site visit, drilling five (5) soil test borings to depth

of about 40 feet, excavating eight (8) test pits, obtaining grab samples of the Phase I landfill cover material for laboratory testing, and performing field percolation testing and laboratory testing. A general layout of the site is presented on the Plot Plan, Plate 2.

3.0 SCOPE OF SERVICES

Based on the above considerations, the following scope of services was performed:

1. *Review of Readily Available Information* - Readily available subsurface information, record drawings and previous soils reports in County of Kauai, Department of Public Works (DPW) plan files, and other readily available subsurface and geologic information in our files were reviewed.

2. *Site Visit* - On November 15, 2011, our engineering personnel traveled to Kauai to observe surface conditions in readily access areas of the landfill expansion site. Surface soils, general topography, and other surface conditions in readily accessible areas of the proposed expansion area were observed from a geotechnical engineering standpoint.

3. *Coordination with AECOM and Waste Management, Staking-out Borings and Test Pits, and Utility Checking* - Prior to the start of the field work, we coordinated our work with AECOM and Waste Management. Readily available underground utility plans were reviewed to check the proposed boring and test pit locations for possible underground lines.

   The boring and test pit locations were surveyed and marked by AECOM's surveyor. Based on feedback from AECOM, we adjusted select boring and test pit locations during our site visit on November 15, 2011. The Hawaii One Call Center was contacted to review the proposed boring and test pit locations for possible underground utilities. As a final check, we toned the boring and test pit locations using a metal detector.

4. *Field Exploration* - Subsurface conditions at the proposed Cell 2 lateral expansion site were explored by performing the following soil test borings and test pits as specified:

   - Five (5) soil test borings, designated as Borings B-1 through B-5, to depths of 40 to 60.5 feet below existing grades, and

   - Eight (8) test pits, designated as Test Pits TP-1 through TP-8, to depths of 7 to 10 feet below existing grades.

The borings were drilled using a Mobile B-53 truck mounted drill rig with continuous flight augers and rotary wash drilling equipment. The test pits were excavated using a rubber-tired backhoe.

The approximate locations of the borings and test pits are shown on the Plot Plan, Plate 2. The locations and elevations of Boring B-5, Test Pits TP-1, TP-4, TP-6 and TP-8 were estimated based on information provided on the project topographic plan and our field measurements using a 100-foot tape.

The drilling of the borings and excavation of the test pits were performed under the technical observation of our engineering personnel, who maintained a log of the subsurface materials encountered in each boring and test pit, and obtained disturbed and relatively undisturbed soil samples for further examination and laboratory testing.

Relatively undisturbed and disturbed soil samples were obtained using a stainless steel split barrel sampler with an outside diameter (O.D.) of 3.3 inches and an inside diameter (I.D.) of 2.4 inches, and a Standard Penetration Test (SPT) split spoon sampler with an O.D. and I.D. of 2 and 1.4 inches, respectively. The split barrel sampler was lined with brass rings. The samplers were driven with blows from a 140-pound hammer falling about 30 inches. Each sampling attempt consisted of driving the sampler a total distance of about 18 inches, and recording the blow counts for each 6 inches of penetration. The blow counts for the last 12 inches of penetration are shown on the boring logs. The blow counts for the 3.3-inch diameter split barrel sampler on the boring logs represent actual blow counts and have not been converted to equivalent SPT-N values.

Graphical representations of the soils encountered in the borings and test pits are presented on the Log of Borings, Plates 3.1.1 through 3.5.2, and the Log of Test Pits, Plates 4.1 through 4.4. The soils encountered in the borings and test pits were initially classified according to American Society for Testing and Materials (ASTM) D2488 and Unified Soil Classification System (USCS) presented on Plates 5.1 and 5.2. Additional field data from our observation of the drilling process, soil cuttings, drilling rate, and soil sampling were used to supplement the field classifications of the soils. The field classifications were later refined according to ASTM D2487 based on the results of laboratory tests performed on select soil samples.

Ground water was encountered in all of the borings and test pits, except for Test Pits TP-2 and TP-8, at the time of our field work. The measured water levels are indicated on the boring and test pit logs.

After the completion of drilling, the submerged portions of the borings were backfilled with a bentonite, cement, and water mixture. The remainder of each hole was backfilled with soil cuttings and soil materials to existing grade. After completion of the test pit excavations, the test pits were backfilled with soil cuttings and soil materials.

5. *Percolation Testing and Sampling Phase I Cover* - As specified, a percolation test was performed near Boring B-2 at a depth of about 2 feet below existing grade. The test was performed in general accordance with the State of Hawaii Department of Health guidelines. In addition to the percolation testing, surface grab samples of the existing Phase I landfill cover soil were obtained at three (3) locations specified by AECOM for gradation and plasticity testing. The grab samples were obtained using hand tools.

6. *Laboratory Testing* - After the completion of the field work, the soil samples from the borings and test pits were shipped to our laboratory on Oahu for further examination and testing. The testing included index and classification tests, direct shear tests, consolidation tests, permeability tests, and moisture-density relations tests.

   The results of the moisture content and dry density determinations are presented on the Log of Borings and Test Pits, Plates 3.1.1 through 3.5.2 and Plates 4.1 through 4.4, respectively, at the respective sample depths. The Atterberg Limits test results are presented on Plate 6. The gradation test results are presented on Plates 7.1 through 7.5. The consolidation test data are presented on Plates 8.1 through 8.3. The direct shear test data are presented on Plates 9.1 through 9.4. The moisture-density relations test data are presented on Plates 10.1 through 10.8. The permeability test results are summarized in Table 1.

7. *Engineering Analysis and Report Preparation* - Based on the results of the data review, and field and laboratory testing, engineering was performed, and comments and recommendations were developed regarding:

   a. Site preparation,
   b. Anticipated excavation conditions,
   c. Permanent cut and fill slopes,
   d. Fill materials, placement, and compaction,
   e. Foundation subgrade treatment and support, including parameters for foundation design,
   f. Lateral earth pressure, and
   g. Permeability test results.

   The results of the geotechnical exploration complete with field and laboratory test data have been summarized in this letter report.

4.0 REVIEW OF AVAILABLE DATA

We previously performed borings and test pits along the perimeter road of the existing Phase II landfill in the vicinity of the project site for a geotechnical exploration in 2008. The general location of these borings and test pits are shown on the Plot Plan, Plate 2.

Subsurface conditions encountered in these borings and test pits generally consisted of a thin layer of fill material on the surface consisting of about 6 inches of basaltic gravel. The fill was underlain by medium dense to very dense coralline sand to the maximum depths explored in the borings at 35.5 to 40.5 feet and in the test pits at 7.5 to 7.8 feet.

Ground water was encountered in the borings at depths ranging from about 10.4 to 19 feet below existing grades. These depths correspond to elevations of about +4.5 to +5 feet. Ground water was not encountered in the test pits probably because of the relatively higher ground elevations and shallow depths of the test pits. Because of the proximity of the site to the ocean, it is anticipated that fluctuations in groundwater levels may occur due to tidal changes and rainfall landward of the site.

5.0 SITE CONDITIONS

5.1 GENERAL GEOLOGY

The Mana coastal plain is located in southwestern Kauai and lies at the foot of an ancient sea cliff composed of lava flows of the Waimea Canyon Volcanic series. The plain stretches from Waimea in the south to Polihale in the north. It is mainly composed of thick deposits of alluvium composed of clay, silt, and other detritus derived from weathered basalt. Seaward portions of the plain are generally overlain by beach and dune deposits largely composed of sand-sized calcareous sediments. Lagoonal deposits composed of a mixture of calcareous and alluvial sediments are generally present in low-lying areas of the plain, just inland of the beach and dune deposits. As a result of agricultural development of the Mana plain, a majority of the lagoonal environments in the plain have been covered by fill.

Based on geologic maps of Kauai by Macdonald and others (1960) and Sherrod and others (2007), the project site is located inland of a beach berm crest in an area composed of calcareous dune and older beach deposits. A characteristic of the calcareous deposits in this portion of the coast is the common occurrence of weakly cemented sand at or near the water table. Development of the landfill has resulted in the widespread placement of fill over the sand deposits.

According to the United States Department of Agriculture, Natural Resources Conservation Service (NRCS), the surface soil at the site is classified as Jaucas loamy fine sand

(JfB), 0 to 8 percent slopes. JfB soil is a calcareous soil that developed in wind and water-deposited calcareous sand derived from coral and marine shells.

5.2 SURFACE CONDITIONS

The Cell 2 Lateral Expansion site is bounded by the existing Phase II landfill in the east, a chain link fence and gravel road to the Kauai Raceway Park in the south, the existing Phase I landfill in the west, and a chain link fence and Lighthouse Road in the north.

The proposed landfill expansion site is generally located between the Phase I and Phase II landfills. The site appears to have been previously vegetated with trees and bushes that had been cleared. There are gravel roads in the southern portion of the site that connects the Phase I and Phase II landfills and along the west slope of the Phase II landfill. There is a dirt road along the chain link fence north of the Phase II landfill and dirt ramps connecting the gravel roads to the proposed expansion area on the southern end and near the Phase II lateral expansion in the north. The slopes of the site on the Phase I side are roughly 4 to 5 horizontal to one vertical (4H:1V to 5H:1V). The slopes of the side walls on the Phase II side are roughly 1H:1V.

The ground surface at the site generally consists of brown clayey fill material. In the northern portion of the site, surface soil on the floor generally consisted of coralline sand. Near the southern end of the site, a roughly 6- to 8-inch diameter flexible drain pipe extends from the Phase I landfill to the project site, where it is surrounded by some basaltic boulders. According to landfill staff, these boulders are held in place by concrete. The slope on the Phase II landfill side includes exposed portions of landfill gas wells and drainage swales. Vegetation at the site generally consists of about one to 3 feet tall grass, haole koa, kiawe, and various shrubs.

5.3 ANTICIPATED SUBSURFACE CONDITIONS

Subsurface conditions encountered in the borings and test pits performed at the project site are illustrated on the boring and test pit logs presented on Plates 3.1.1 through 3.5.2 and Plates 4.1 through 4.4, respectively. Because the borings and test pits are relatively widely spaced, the actual field occurrence of the geologic units, and subsurface and groundwater conditions may differ from that shown on the logs.

Subsurface conditions encountered in Borings B-1 and B-3 and Test Pit TP-8, performed on a gravel road along the west side of the Phase II landfill, generally consisted of fill material to depths of approximately 9 to 10 feet. The fill material is comprised of fat clay, basaltic gravel, and medium dense to very dense poorly graded coralline sand. Fill material was encountered to the bottom of Test Pit TP-8 at 9.5 feet. Below the fill in Borings

B-1 and B-3, subsurface conditions generally consisted of beach and/or dune deposits of medium dense to very dense poorly graded coralline sand to the bottom of these borings at a depth of approximately 40 feet.

Subsurface conditions encountered in Test Pit TP-2, performed on the dirt ramp near the Phase II lateral expansion, generally consisted of approximately 3 feet of fill material of poorly graded coralline sand with clay. Below the fill, subsurface conditions generally consisted of beach and/or dune deposits of poorly graded coralline sand to the bottom of the test pit at 10 feet.

Subsurface conditions encountered in Borings B-2, B-4, and B-5 and Test Pits TP-1, TP-3, TP-4, TP-5, TP-6, and TP-7, performed in the area between the Phase I and Phase II landfills, generally consisted of about 6 inches to 3 feet of brown, medium stiff to very stiff fat clay, and loose clayey coralline sand fill material at the surface. Fill material was not encountered in Test Pit TP-1. Subsurface conditions encountered in Test Pit TP-1 and below the fill material in the borings and test pits generally consisted of beach and/or dune deposits of medium dense to very dense poorly graded coralline sand to the bottom of the test pits at depths of 7.5 to 9 feet, to the bottom of Borings B-4 and B-5 at a depth of approximately 40.5 feet, and to a depth of about 47 feet in Boring B-2. The beach and/or dune deposits in Boring B-2 was underlain by coral reef deposits of medium dense to very dense, uncemented to moderately cemented foraminifera sand to the bottom of this boring at a depth of approximately 60.5 feet.

Ground water was encountered in all of the borings and in Test Pits TP-1, TP-3, TP-4, TP-5, TP-6, and TP-7 at depths ranging from approximately 6.5 feet to 18.6 feet below existing grades. These depths correspond to elevations of approximately +2.7 to +6.2 feet. Ground water was not encountered in Test Pits TP-2 and TP-8, which were excavated at relatively higher ground elevations. Because of the proximity of the site to the ocean, it is anticipated that fluctuations in the groundwater levels may occur due to the tidal changes and rainfall at and in the vicinity of the site.

5.4 ATTERBERG LIMITS AND GRADATION TESTS

Three (3) grab samples of near surface soil were taken from the Phase I landfill embankment for laboratory testing. The laboratory tests included Atterberg Limits and gradation to evaluate their soil classification and grain size distribution. The Atterberg Limits and gradation test results are presented on Plates 6 and 7.5, respectively. The test results indicate the tested materials generally consisted of sandy fat clay, fat clay with sand, and clayey sand.

5.5 PERCOLATION TEST

A field percolation test was performed in a test hole located near Boring B-2 at a depth of approximately 2 feet below existing grade. The test was performed in general accordance with the State of Hawaii Department of Health guidelines. The test results are summarized below.

Test Location	Subsurface Material Tested	Percolation Rate (min/inch)
Near Boring B-2	coralline sand	2

6.0 DISCUSSION

Potential main geotechnical concerns anticipated at the site for this project include:

- *Presence of variable thicknesses of fat clay at the proposed landfill expansion site.* Fat clay ranging in thickness from about 0.5 to 3 feet was encountered at the project site. Potential differential settlements may occur if the landfill is constructed on variable thicknesses of fat clay. To reduce the amount of differential settlements, it is recommended the near surface fat clay be completely removed prior to the placement of fill.

- *Presence of poorly graded sand deposits.* The on-site sand is poorly graded and appears to contain little fines. Proper compaction of poorly graded sand is anticipated to be difficult.

- *Presence of relatively shallow groundwater conditions*. The sand deposits are anticipated to be relatively permeable and will not be stable at steep slope angles. Temporary open-cut excavations may require slopes approaching the angle of repose for this material or proper sheeting and shoring may need to be installed. Excavations that extend below the groundwater table may be difficult to excavate and dewater without proper sheeting and shoring.

A more detailed description of our recommendations is presented in the following letter report sections.

7.0 RECOMMENDATIONS

7.1 SITE PREPARATION

1. Prior to grading, the areas of proposed development should be prepared by stripping the ground surface of all vegetation, trees, and the top at least 6 inches of soils containing vegetation, roots, and organic matter. Deeper stripping may be needed where vegetation and organic material extend to deeper depths. The stripped and grubbed materials should be hauled to a suitable disposal site off of the County's property.

2. Existing trees that may interfere with the proposed improvements should be completely removed or relocated. The roots and stumps of these trees should be completely grubbed and the resulting excavations backfilled with structural fill placed and compacted as described in subsection 7.5 of this report.

3. Existing underground utility lines within the proposed construction limits and any remaining underground structures that may interfere with the construction should be completely removed and/or relocated, if still in use. The remaining portions of any lines to be left in-place should be properly cut and plugged.

7.2 SUBGRADE PREPARATION

1. After clearing and grubbing, the clayey soil at the surface of the area to be developed should be excavated and completely removed to coralline sand. The clayey soil may be stockpiled for possible reuse in landscape or non-structural areas provided they conform to the project landscaping requirements.

2. After excavating and removing the surface clayey soil, the subgrade should be proof rolled using a heavy roller weighing at least 20 tons to check for yielding and soft areas. Yielding or soft areas in the subgrade should be excavated to firm material and backfilled with properly compacted structural fill.

3. Prior to placing any new fill, the subgrade beneath areas to receive fill should be scarified to a depth of at least 6 inches, thoroughly moisture conditioned to within 2 percent of the optimum moisture content for this material, and compacted to a relative compaction of at least 95 percent in structural areas and at least 90 percent in non-structural or landscape areas.

Relative compaction in this letter report is defined as the dry unit weight of the compacted material expressed as a percentage of the maximum dry unit weight of the same material based on ASTM D1557 test method.

Any soft or yielding zones detected during the subgrade compaction should be treated by removing the soft or loose materials to firm soils and replacing them with properly compacted structural fill.

7.3 ANTICIPATED EXCAVATION CONDITIONS

1. Temporary excavations, sheeting, shoring, and dewatering that may be required for constructing the proposed improvements, utility lines, and below grade structures is the responsibility of the Contractor. All construction excavations and dewatering should be performed and supported in accordance with applicable Federal and local safety regulations, including current OSHA excavation and trench safety standards.

   The Contractor's excavation support and dewatering systems should protect against instability of the excavations, boiling and/or blowout of the excavation and trench bottoms, damage to existing structures, and settlement and/or movement in existing structures, slabs, and pavements.

2. Excavations to depths required for constructing the proposed improvements are anticipated to encounter mainly clayey and sandy soils. It is anticipated that these materials can generally be excavated with conventional earthwork equipment. Excavation and removal of boulders and highly cemented layers, if encountered, may require the use of hydraulic hoe ram, rock excavator, or other suitable rock excavation equipment.

3. It is anticipated that excavations into granular deposits present at the site would tend to slough and cave-in, if not properly shored and braced. The Contractor should utilize proper sheeting and shoring to protect their workmen and surrounding properties, where necessary.

4. Ground water was encountered in the borings and most of the test pits at the time of the field exploration at elevations ranging from approximately +2.7 to +6 feet. Based on the water levels encountered at the site and the planned construction, some amount of dewatering will likely be needed in excavations that extend below the water table to allow for proper cleaning.

   To reduce the potential for settlement related damage, draw downs of the groundwater levels outside of the excavation should be minimize as much as

practical. The dewatering system should include properly designed filters and/or geotextile filter fabric to reduce the potential for removal of fines during pumping. All pumped water from the dewatering operations should be properly filtered and treated in accordance with applicable Federal and local regulations before being discharged.

7.4 PERMANENT CUT AND FILL SLOPES

1. Permanent cut slopes constructed in the natural sand deposits should not be steeper than 2H:1V.

2. We understand that permanent fill slopes for the landfill expansion is planned at 3.5H:1V. All fills placed on slopes steeper than 5H:1V should be continuously keyed and benched into the existing hillside. All fill slopes should be overbuilt and trimmed back to expose firm, compacted material at the finish grade.

3. To reduce the amount of erosion that may occur, permanent benches should be provided in cut or fill slopes that are greater than 15 feet in height. The benches should be provided every 15 feet of elevation change, except where only one bench is required. In this case, the bench should be provided at the mid-height of the slope. The benches should be at least 8 feet wide and should be sloped slightly inward to drain to a suitable drainage structure.

   All permanent cut and fill slopes should be vegetated as soon as practical to reduce overall erosion. Because of the presence of mainly sandy and clayey soils with a high potential for soil erosion, a program of surface drainage control measures, such as swales and ditches should be implemented during and after grading to reduce overall erosion during and after construction.

4. The ground surface along the top of cut and fill slopes should be graded to keep surface water from running down the slope faces or a cutoff ditch or swale should be provided along the top of the slopes. Uncontrolled surface runoff could potentially result in severe slope erosion.

7.5 FILL MATERIALS, PLACEMENT, AND COMPACTION

1. On-site soil materials classified as CH, according to the Unified Soil Classification System (USCS) and ASTM D2487 are not suitable for use as structural fill and structural backfill. These soils may be stockpiled for possible re-use in landscape

and grassed areas. Other unsuitable soils are materials with USCS classifications of MH, ML, CL, PT, OL, and OH.

2. Imported structural fill and structural backfill should consist of granular material classified as GW, GW-GM, GP-GM, SW, SW-SM, or SP-SM according to the USCS and ASTM D2487. It should be free of organic matter, debris, clayey soils, and particles larger than 3 inches in maximum dimension. It should have not more than 15 percent by weight of material passing a U.S. Standard No. 200 sieve. It should have a liquid limit of less than 25 and a plasticity index of less than 10. It should also have a California Bearing Ratio (CBR) value of at least 30 and a CBR swell of less than 2 percent after 4 days of soaking when compacted at optimum moisture content.

3. Structural fill and backfill should be placed in not more than 8-inch thick horizontal loose lifts, moisture conditioned to within 2 percent of the optimum moisture content for these materials, and compacted to a relative compaction of at least 95 percent.

4. Fill in non-structural and landscape areas should be free of organic matter, debris, trash, old pavements, deleterious material, concrete and metal debris, and particles larger than three (3) inches in maximum dimension.

   Fill in landscape and non-structural areas should be placed in not more than 10-inch thick loose, horizontal lifts, moisture conditioned to between optimum and 3 percent wet of the optimum moisture content for the material being placed, and compacted to a relative compaction of at least 90 percent for granular material and at least 85 percent for cohesive material.

5. All proposed fill materials should be checked and if appropriate, tested and approved by a qualified and experienced registered geotechnical engineer prior to their use in fills at the site. An adequate number of field density tests should be performed to check that the required degree of compaction has been achieved.

7.6 MANHOLE SUPPORT

1. According to AECOM's PER, the landfill expansion will include 4-foot diameter precast concrete manholes supported on a 12-foot square by 1'- 2" thick concrete pad. We understand that the invert of the manholes will be above the groundwater table.

2. To provide for more uniform support of the manhole, it is recommended that at least 12 inches of the subsurface materials directly below the manhole pad be over

excavated and replaced with properly compacted structural fill. The over excavation should extend at least 24 inches beyond the manhole pad limits. To reduce the potential for intermixing the poorly graded on-site sand and the new structural fill, it is recommended that a woven geotextile fabric be placed between the two materials. The geotextile fabric should be overlapped at least 12 inches along all joints.

3. The manhole walls should be backfilled with properly compacted, free draining structural fill. The fill should be placed in not more than 6-inch thick horizontal loose lifts, moisture conditioned to within 2 percent of optimum moisture content for this material, and compacted to a relative compaction of at least 90 percent.

4. The manhole walls backfilled with free draining granular fill material and level backfill conditions may be designed to resist an at-rest equivalent fluid pressure of 60 pounds per square foot per foot of depth (psf/ft) above the groundwater table.

5. Any surcharge from traffic, equipment, and embankment, if present, should be added to the above earth pressure.

8.0 PRE-FINAL PLANS/SPECS REVIEW

During the design, we plan to review the geotechnical related portions of the pre-final plans and specifications to check that the intent of the recommendations presented in this letter report are properly reflected in the contract documents.

9.0 LIMITATIONS

This letter report has been prepared specifically for the use of AECOM in accordance with generally accepted soils and foundation engineering practices for the proposed Kekaha Landfill Phase II, Cell 2 Lateral Expansion project at Kekaha, Kauai, Hawaii. No warranty or guarantee, expressed or implied, or other representation is made as to the professional advice included in this letter report and none should be inferred.

This letter report has been developed solely and exclusively for the use of AECOM for the proposed Cell 2 Lateral Expansion project as described herein. It does not contain sufficient information for the purposes of other parties or for other uses. We are not responsible for any claims, damages, or liability associated with the use of the information presented in this letter report by any other party without our expressed written consent. No third party may rely upon this letter report or any other document prepared by us unless we have agreed to such reliance in advance and in writing.

This letter report does not reflect variations which may occur in the subsurface and groundwater conditions at locations other than our boring locations. The nature and extent of variations in the subsurface and groundwater conditions may not become evident until construction.

Ground water was encountered in all of the borings and most of the test pits at the time of our field exploration. The measured water levels are indicated on the boring and test pit logs. However, fluctuations in the ground water level may occur due to variations in rainfall, seepage, perched zones, construction activities, temperature, and other factors that may be different from the conditions that existed at the time of our measurements.

The comments and recommendations presented in this letter report are based on the anticipated construction described herein. Should the actual construction differ from that described in this letter report, we should be notified and retained to check if any modifications to the recommendations presented in this letter report are needed. The comments and recommendations presented in this letter report shall not be considered valid unless the changes are reviewed by us and the recommendations of this letter report verified or modified by us in writing.

The field exploration portion of this consultation may not have disclosed the presence of underground structures, such as cesspools, drywells, storage tanks, sumps, pits, buried debris, and landfills, etc., that may be present at the site. Should these items be encountered during construction, we should be retained to provide appropriate recommendations for their disposal and/or treatment. Assessment of the presence or absence of these structures was not included in the scope of this consultation.

The scope of our services was limited to conventional geotechnical engineering services and did not include any environmental assessment or evaluation of potential subsurface and groundwater contamination. Silence in this letter report regarding any environmental aspects of the site subsurface and groundwater conditions do not include the absence of potential environmental problems.

The scope of our services specifically excluded the investigation, detection, or assessment of the presence of Biological Pollutants in or around any existing or planned structure. Accordingly, this report includes no interpretations, recommendations, findings, or conclusions for the purpose of detecting, preventing, assessing, or abating Biological Pollutants. The term “Biological Pollutants” includes, but is not limited to molds, fungi, spores, bacteria, and viruses, and/or any of their byproducts.

The following plates, table, and references are attached and complete this letter report.

Plate 1	-	Map of Area
Plate 2	-	Plot Plan
Plates 3.1.1 through 3.5.2	-	Log of Borings, B-1 through B-5
Plates 4.1 through 4.4	-	Log of Test Pits, TP-1 through TP-8
Plates 5.1 and 5.2	-	Unified Soil Classification System
Plate 6	-	Atterberg Limits
Plates 7.1 through 7.5	-	Gradation Curves
Plates 8.1 through 8.3	-	Consolidation Test Data
Plates 9.1 through 9.4	-	Direct Shear Test Data
Plates 10.1 through 10.8	-	Compaction Test Data
Table 1	-	Laboratory Permeability Test Results

References

KENNETH K. FAN
LICENSED PROFESSIONAL ENGINEER
No. 6729-C
HAWAII, U.S.A.

THIS WORK WAS PREPARED BY ME OR UNDER MY SUPERVISION
Kenneth Fan

EXP. April 30, 2014

Yours very truly,

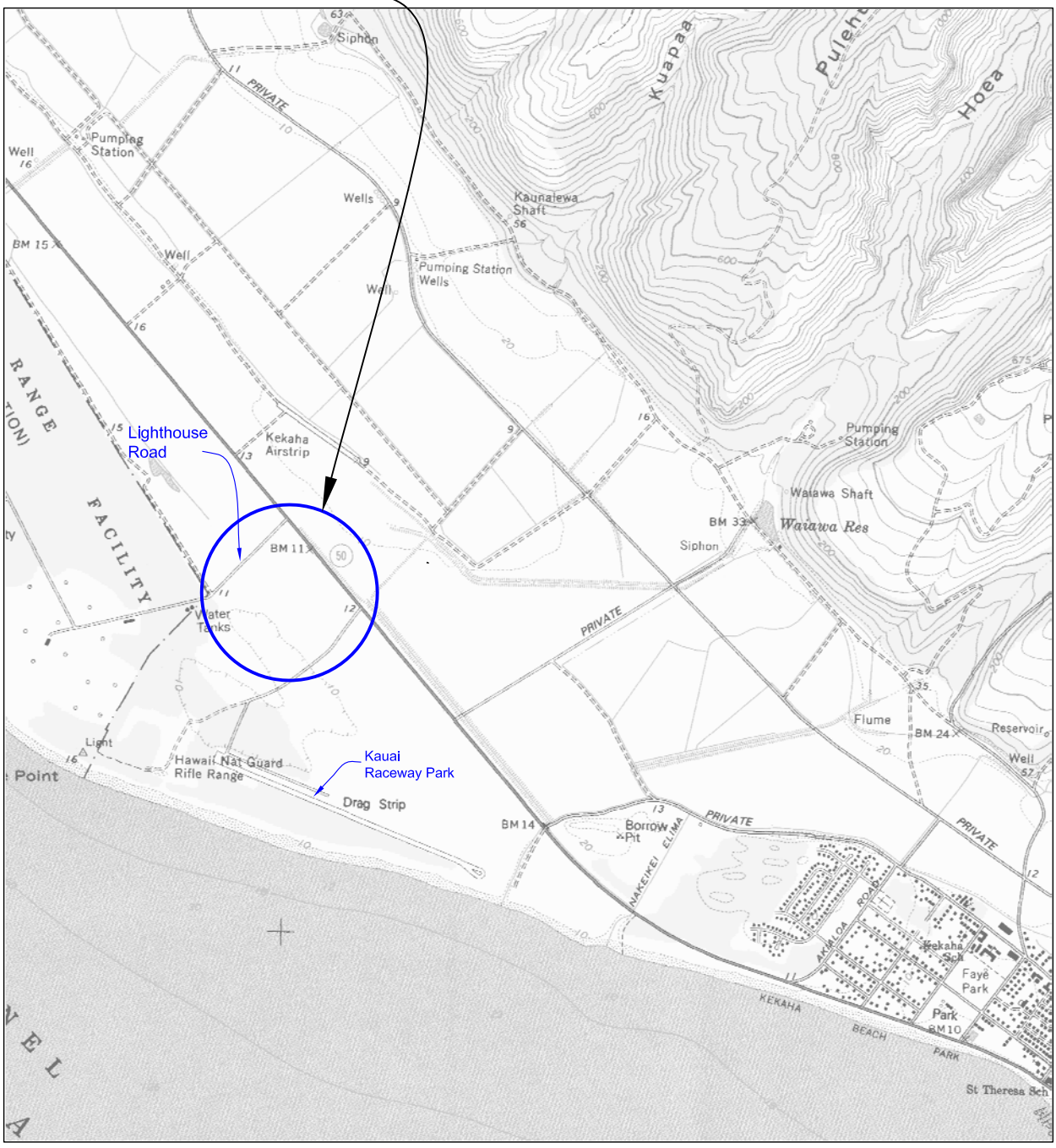
PACIFIC GEOTECHNICAL ENGINEERS, INC.

Kenneth Fan

Kenneth K. Fan, P.E.
Project Manager

Carl J. Iwasaki, P.E.
Project Engineer

KKF/CI(8950-023 final letter rpt)
(Three copies submitted)



Contour Interval 40 Feet
Datum is Mean Sea Level
Depth Curves in Feet - Datum is Mean Lower Low Water

Reference:
U.S.G.S. Topographic Map
Kekaha, Kauai, Hawaii
Dated: 1983

MAP OF AREA

Kekaha Landfill Phase II
Cell 2 Lateral Expansion
Kekaha, Kauai, Hawaii

Pacific Geotechnical Engineers, Inc.

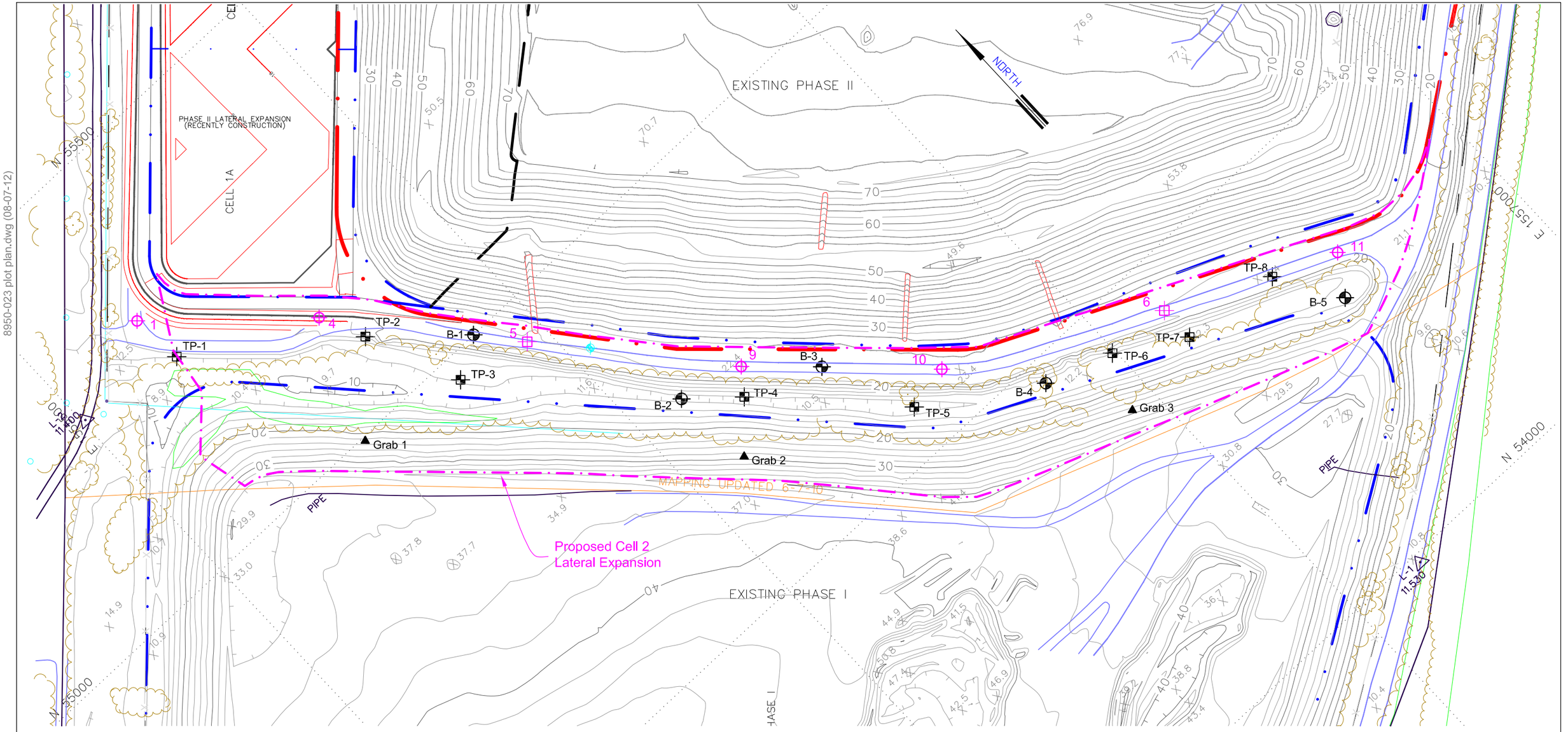
PLATE 1

8950-023 moa.dwg (08-07-12)

J:\8950-023\engineering\cad\8950-023 moa.dwg, 8/7/2012 6:46:02 PM

J:\8950-023\engineering\cad\8950-023 plot plan.dwg, 8/7/2012 6:49:50 PM

8950-023 plot plan.dwg (08-07-12)



LEGEND

B-1	APPROXIMATE LOCATION AND NUMBER OF PGE BORING
TP-1	APPROXIMATE LOCATION AND NUMBER OF PGE TEST PIT
Grab 1	APPROXIMATE LOCATION AND NUMBER OF PGE GRAB SAMPLE
680	EXISTING TOPOGRAPHY
	PROPERTY LINE
	PERMITTED LIMITS OF WASTE
GP-II-4	EXISTING GAS PROBE
1	APPROXIMATE LOCATION AND NUMBER OF PREVIOUS PGE BORING (2007)
5	APPROXIMATE LOCATION AND NUMBER OF PREVIOUS PGE TEST PIT (2007)

Reference:
Proposed Boring and Test Pit Locations
From: AECOM
Received: November 14, 2011

NOTES:

1. TOPOGRAPHIC MAP PREPARED BY AEROMETRIC, INC., SEATTLE, WASHINGTON. DATE OF FLIGHT: JUNE 7, 2010.
2. HORIZONTAL DATUM IS BASED ON NAD83 (1986), HAWAII ZONE 4. VERTICAL DATUM BASED ON LOCAL TIDAL.

0 60 120 240
Scale 1"= 120' Feet

PLOT PLAN

Kekaha Landfill Phase II
Cell 2 Lateral Expansion
Kekaha, Kauai, Hawaii

Pacific Geotechnical Engineers, Inc.

PLATE 2

Appendix B
Geotechnical Calculations

Appendix B
Geotechnical Calculations

Appendix B.1
Foundation Stability

Appendix B.1A
Cell 2 Foundation Settlement and Base Liner Strain Analysis

Appendix B.1B
Cell 2 Expansion Phase I Sideslope Settlement Analysis

Appendix B.2
Buildout Stability Analysis

Appendix B.3
Drainage Layer Stability Analysis With Equipment

Appendix B.3A
Drainage Layer Stability Analysis With Equipment
(Cell 2 Base Liner Sideslopes)

Appendix B.3B
Drainage Layer Stability Analysis With Equipment
(Cell 2 Base Liner Overlay on Phase I)

Appendix B.4
Geotextile Analysis

Appendix B.4A
Cushion Geotextile Analysis

Appendix B.4B
Separation Geotextile Analysis

Appendix B.5
Final Cover Drainage Layer Capacity and Slope Stability Analysis

Appendix B.6
Geomembrane Compatibility

Appendix B.7
Depression Settlement Calculation

Appendix B.8
Cell 2 Expansion Final Cover Settlement Analysis

Appendix B.9
Phase 1 Final Cover Geomembrane Extension Slope Stability

Appendix B.1
Foundation Stability

Appendix B.1A
Cell 2 Foundation Settlement and Base Liner Strain Analysis

Client	County of Kaua'i	**Subject**	Cell 2 Settlement	**Prepared By**	SRK	**Date** 8/14/12
Project	Kekaha Landfill		And Base Liner Strain	**Reviewed By**	JMT	**Date** 8/14/12
Phase II Lateral Expansion			Analysis	**Approved By**	KJB	**Date** 8/14/12

CELL 2 SETTLEMENT AND BASE LINER STRAIN ANALYSIS

Objective

Estimate the settlement of the foundation soils of the proposed Cell 2 expansion as the result of the increased overburden pressure from the waste. Determine if the strain on the base liner is acceptable, and determine if positive drainage will be maintained in the leachate collection line after differential settlement.

Design Criteria and Assumptions

1. The unit weight of municipal solid waste placed in Cell 2 is 62 pcf.
2. The foundation soils for Cell 2 are fine to coarse grained, relatively medium dense to very dense coralline sand (SP and SP-SM) (Reference 1).
3. Depth to bedrock is not known and is greater than 40 feet in depth from existing ground surface. The majority of settlement is expected to occur within the upper 40 feet of the foundation soil as review of borings demonstrate increased soil density base 40 feet in depth.
4. The base liner consists of two geomembranes, a geosynthetic clay liner, a 12-inch granular drainage layer, and a 24-inch operations layer and geotextiles.
5. The cover includes: a 12-inch intermediate cover, a 6-inch grading layer, a geomembrane, a geocomposite, a 12-inch rooting zone, and 6-inches of topsoil.
6. The unit weight of all soil components of the base liner and cover was estimated to be 130 pcf to be conservative.
7. Two cross sections (AA and BB) have been derived for the Phase I overlay of Cell 2. These cross sections correlate to the Final Buildout Stability analysis in Appendix B.2.
8. All references to elevations correlate to the site specific vertical datum.

Calculations

Long-term performance of the landfill base liner system and leachate collection system is related to settlement of the foundation soils and potential differential settlement can cause large strains in the liner system, and cut off drainage of the leachate collection system.

The foundation soils at the site are sandy, therefore primary consolidation and secondary consolidation settlements are negligible. Elastic settlement will make up the majority of the total settlement. Elastic settlement of sandy soil is a function of the Elastic Modulus of the soil and the change in stress.

The change in stress (added overburden) is due to the weight of the base liner drainage materials, the thickness of waste, and final cover soils. For Point A on Section AA, the change in stress is calculated as:

$$\Delta\sigma_A = (130\ pcf)(3\ ft) + (62\ pcf)(44\ ft) + (130\ pcf)(3\ ft) = 3{,}508\ psf$$

Consolidation testing performed on the foundation soils (Attachment 2) by Pacific Geotechnical Engineers, Inc. (Reference 1) indicates that the elastic strain was approximately 0.018 in/in at a stress level of 5,000 psf. Therefore, the modulus of the soil for the site's sand soils is approximately 280,000 psf for the range of stresses applicable.

$$E = \frac{5000\ psf}{0.018\ in/in} = 277{,}778\ psf \rightarrow 280{,}000\ psf$$

The total settlement is simply the product of the elastic strain and the thickness of the layer

$$S = \frac{\Delta\sigma}{E}H$$

For Point A of Section AA, total settlement is:

$$S_A = \frac{3{,}508\ psf}{280{,}000\ psf}(40\ feet) = 0.50ft$$

Similarly, for Point B of Section AA, the total settlement was calculated as 0.54 feet (see attached spreadsheet).

The final resulting slope from Point A to Point B for Section AA is equal to the settled elevations at Points A and B divided by the horizontal distance between the points. This results in:

$$Slope_{AB} = \left[\frac{(Pt.B\ Initial\ Base\ Elev - S_B) - (Pt\ A.\ Initial\ Base\ Elev - S_A)}{(Horizontal\ Distance\ Between\ Pt\ A\ and\ Pt\ B)}\right] * 100$$

$$Slope_{AB} = \left[\frac{(18.8 - 0.54\ ft) - (10.8 - 0.50\ ft)}{(400\ ft - 0\ ft)}\right] * 100 = 1.99\% \rightarrow 2.0\%$$

To determine strain or elongation of the base liner system, the pre and post settlement lengths of the base are calculated.

$$L = \sqrt{(Pt.BBase\ Elev - Pt.A\ Base\ Elev)^2 + (Horizontal\ Distance\ Between\ Pts.A\ and\ B)^2}$$

Solving base liner lengths (initial and settled) for Point A to Point B:

$$L_{initial} = \sqrt{(18.8\ ft - 10.8\ ft)^2 + (400\ ft - 0\ ft)^2} = 400.1\ ft$$

$$L_{Settled} = \sqrt{(18.3 - 10.3)^2 + (400\ ft - 0\ ft)^2} = 400.1\ ft$$

Strain (elongation) is derived by:

$$Strain = \left(\frac{L_{initial} - L_{Settled}}{L_{initial}}\right) * 100$$

Solving for Points A and B of Section AA:

$$Strain = \left(\frac{400.1\ ft - 400.1\ ft}{400.1\ ft}\right) * 100 = 0\%, no\ added\ strain.$$

By observation, settlement along the leachate collection pipes is relatively consistent resulting in minimal if any strain to the liner system.

A completed spreadsheet table is provide as Attachment 3 for the results for points A through G for the change in stress, the settlement, and the resulting base liner slope, and calculated strain.

Conclusions

The construction, waste filling, and installation of the final cover system of Cell 2 is expected to result in uniform settlement of the foundation soils of Cell 2. The amount of settlement is a function of the depth of the sandy soils, and the overburden stress of the waste. Considering differential settlement, the drainage grade of the leachate collection system is between 1.8 and 2.9% demonstrating positive flow of leachate collection upon settlement.

Strain on the base liner of Cell 2 along the leachate collection line is very minimal and near zero.

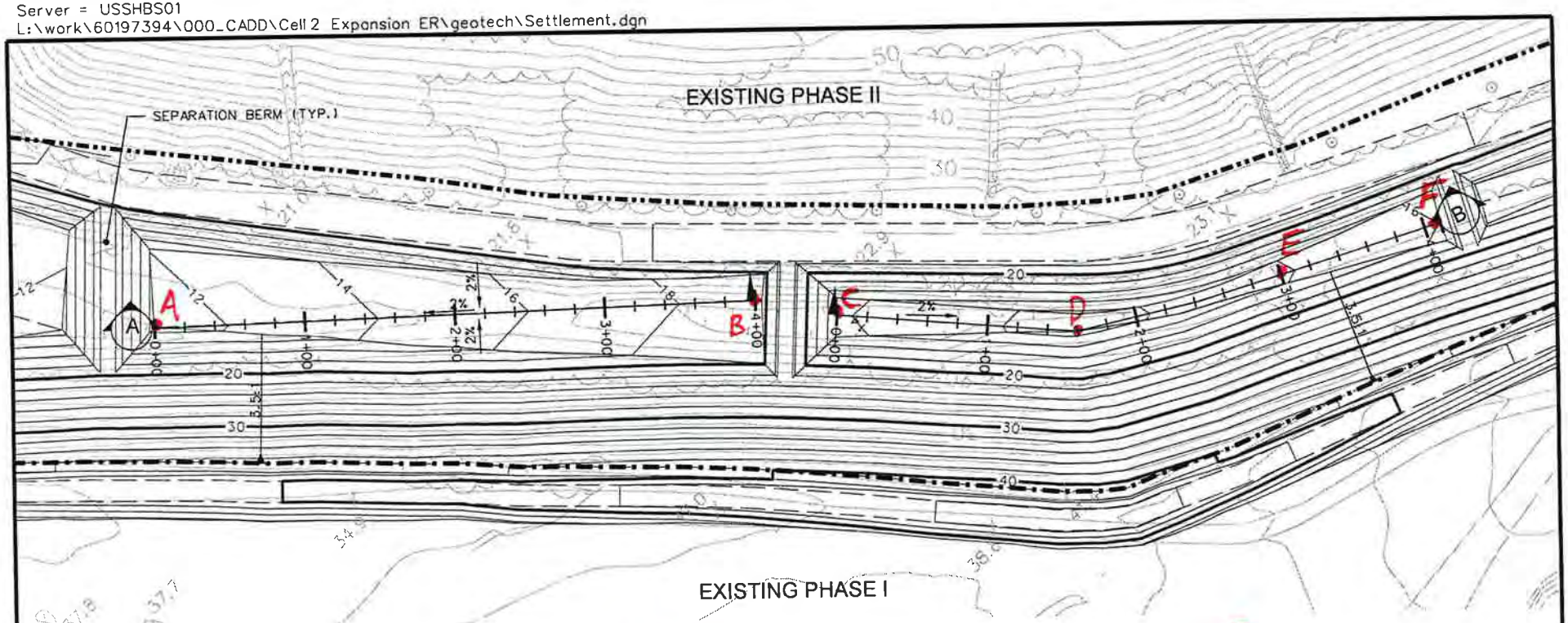
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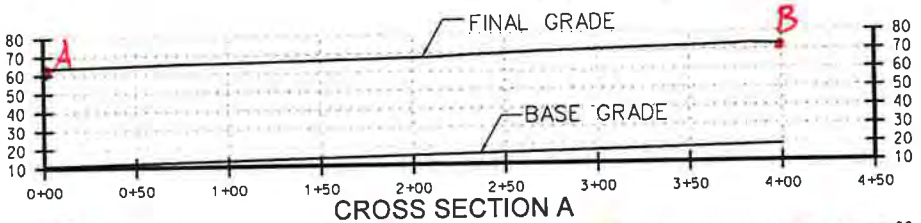
1. Pacific Geotechnical Engineers, Inc. (2012.). *Kekaha Landfill Phase II Cell 2 Lateral Expansion Kekaha Sanitary Landfill Kekaha, Kauai, Hawaii*.
2. Drawings titled "Engineering Report, Phase II - Cell 2 Lateral Expansion, Kekaha Landfill", Prepared by AECOM, dated August 2012.

ATTACHMENT 1

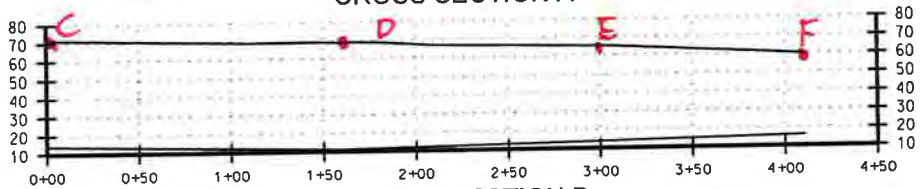
CROSS SECTION LOCATIONS FOR SETTLEMENT AND STRAIN ANALYSIS

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Server = USSHBS01
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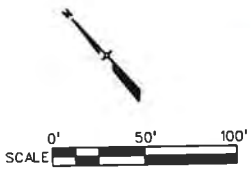




CROSS SECTION A



CROSS SECTION B



LEGEND

- EXISTING TOPOGRAPHY
- PROPERTY LINE
- EXISTING LIMIT OF WASTE
- PROPOSED LIMIT OF WASTE
- CELL DEVELOPMENT LIMIT
- ACCESS ROAD
- GRADE BREAK

NOTES:

1. TOPOGRAPHIC MAP PREPARED BY AEROMETRIC, INC., SEATTLE, WASHINGTON. DATE OF FLIGHT: APRIL 21, 2011.
2. HORIZONTAL DATUM IS BASED ON NAD83 (1986), HAWAII ZONE 4. VERTICAL DATUM BASED ON LOCAL TIDAL.
3. GRADES SHOWN DEPICT SUBBASE AND SURFACE FOR INSTALLATION OF APPROVED BASE LINER.

AECOM

SETTLEMENT ANALYSIS PLAN AND CROSS SECTIONS

ENGINEERING REPORT
KEKAHA LANDFILL PHASE II-CELL 2 LATERAL EXPANSION KEKAHA SANITARY LANDFILL
KAUAI, HAWAII

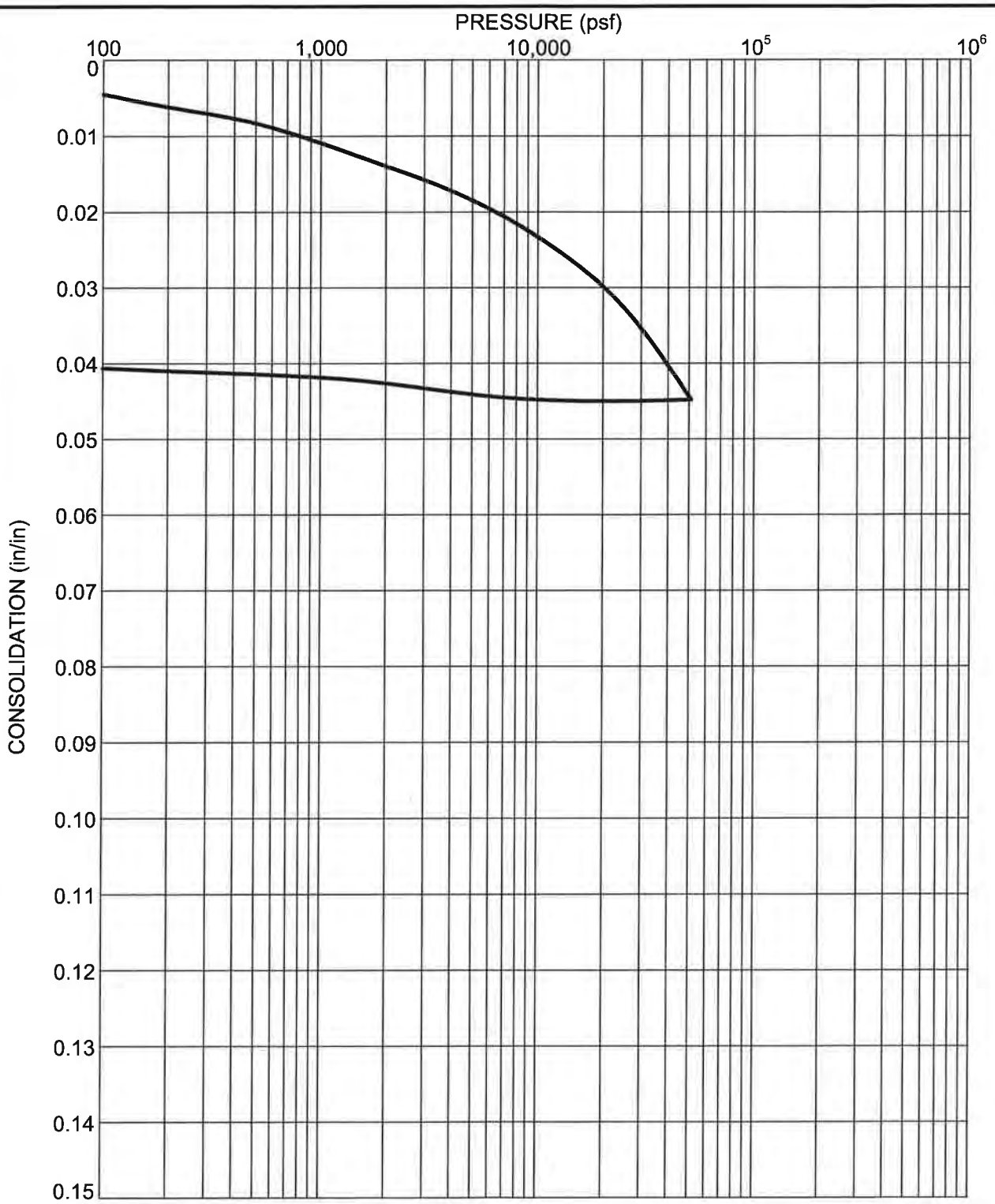
60197394 DEC. 2012

ATTACHMENT 2

CONSOLIDATION TEST RESULTS

PROJECT *Kekaha Landfill Phase II, Cell 2 Lateral Expansion* JOB NUMBER *8950-023*

LOCATION *Kekaha, Kauai, Hawaii* DATE *8/6/2012* DRAWN BY *LML*



LOCATION	SAMPLE DEPTH (feet)	SOIL DESCRIPTION
B-2	5.2	Light yellowish brown poorly graded coralline sand (SP)

	BEFORE TEST	AFTER TEST
MOISTURE CONTENT (%)	5	23
DRY DENSITY (pcf)	98	102

CONSOLIDATION TEST DATA

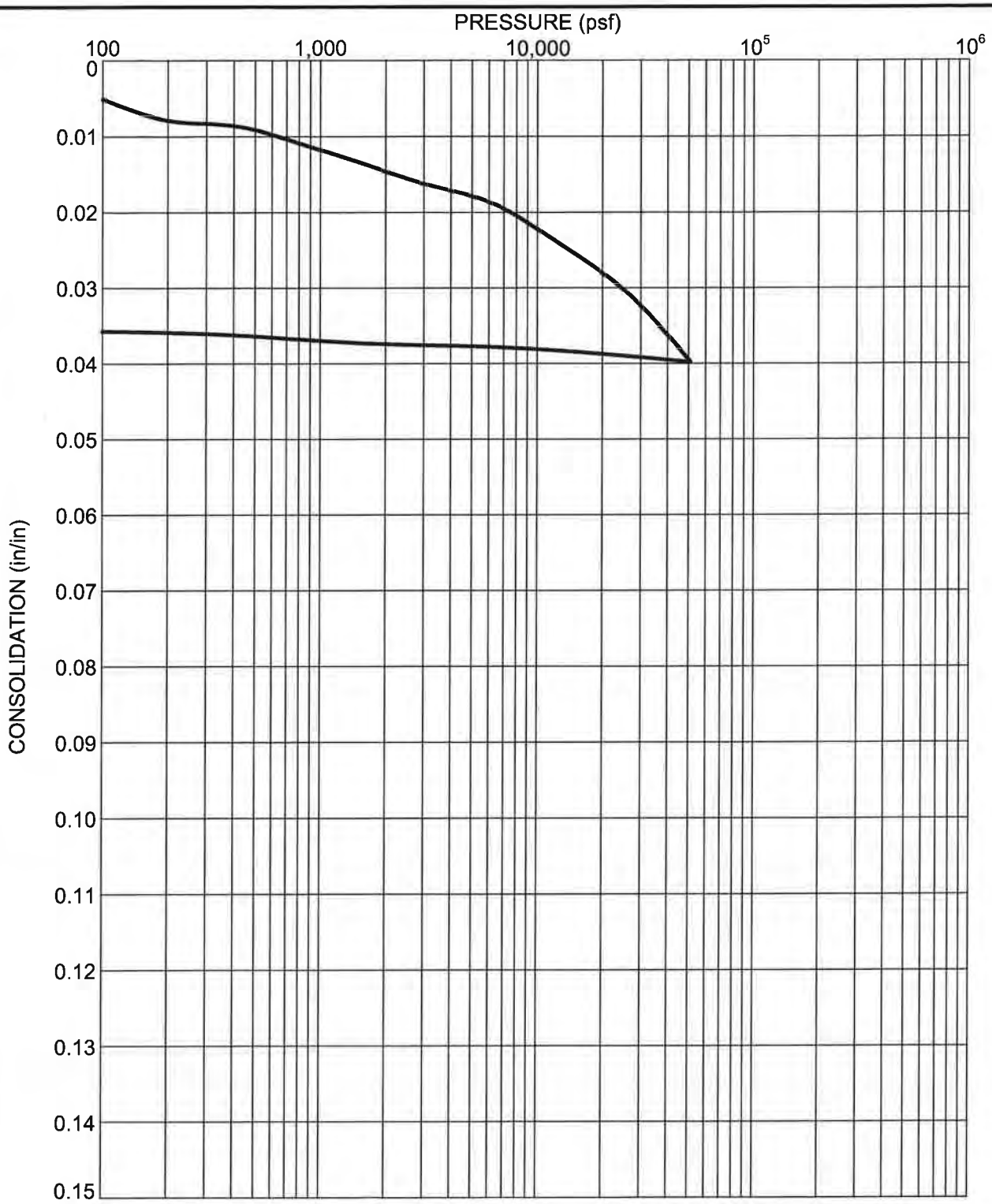
Pacific Geotechnical Engineers, Inc.

PLATE 8.1

PACIFIC GEOTECHNICAL - CONSOLIDATION - PACIFIC GEOTECHNICAL FINAL DATA TEMPLATE GDT - 8/6/12 17:04 - J:\8950-023\ENGINEERING\GINT\8950-023 B-LOGS_LAB.GPJ

PROJECT *Kekaha Landfill Phase II, Cell 2 Lateral Expansion* JOB NUMBER *8950-023*

LOCATION *Kekaha, Kauai, Hawaii* DATE *8/6/2012* DRAWN BY *LML*



LOCATION	SAMPLE DEPTH (feet)	SOIL DESCRIPTION
B-4	19.7	Light yellowish gray poorly graded coralline sand (SP)

	BEFORE TEST	AFTER TEST
MOISTURE CONTENT (%)	23	24
DRY DENSITY (pcf)	102	106

CONSOLIDATION TEST DATA

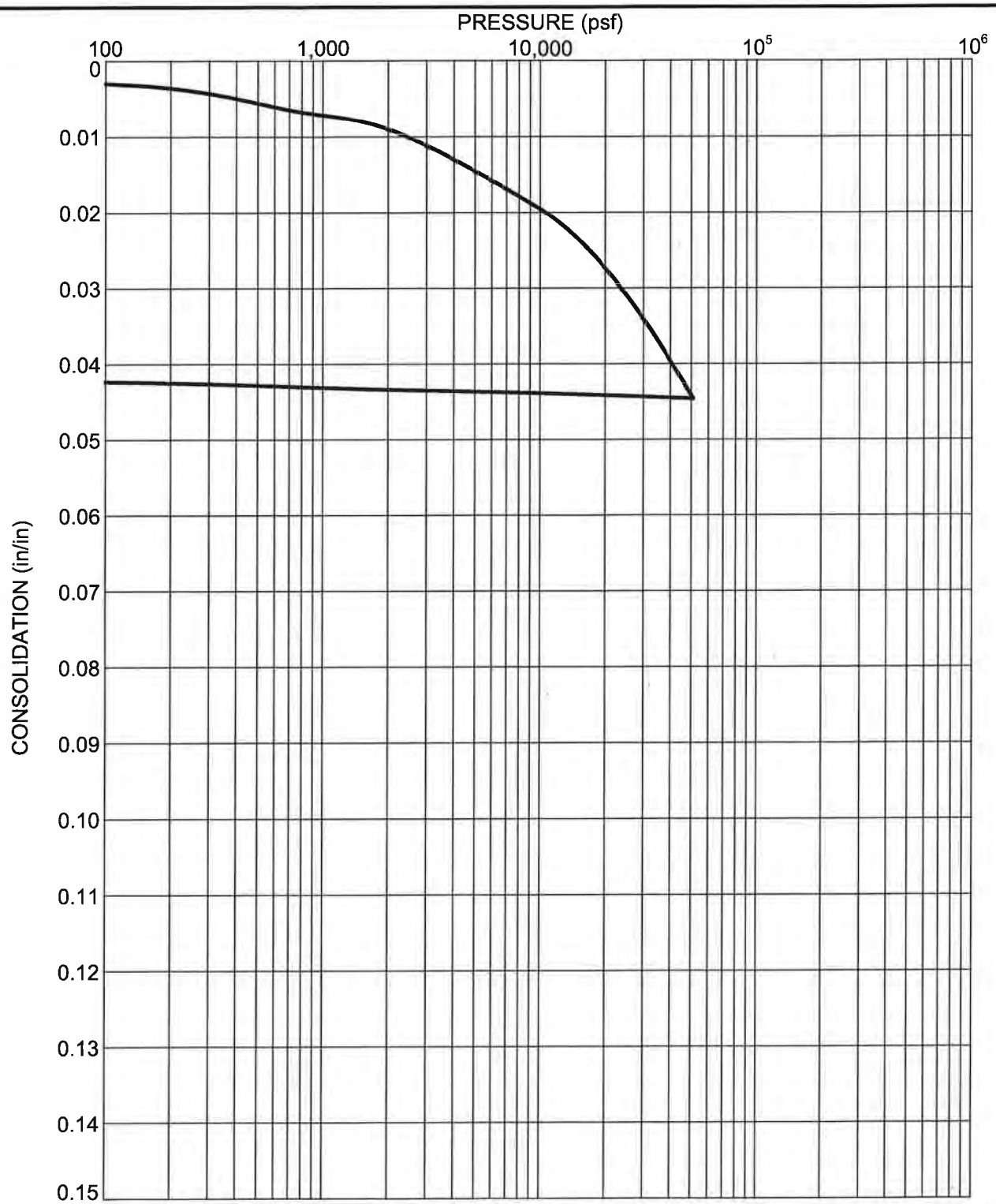
Pacific Geotechnical Engineers, Inc.

PLATE 8.2

PACIFIC GEOTECHNICAL - CONSOLIDATION - PACIFIC GEOTECHNICAL FINAL DATA TEMPLATE GDT - 8/6/12 17:04 - J:\8950-023\ENGINEERING\GINT\8950-023 B-LOGS_LAB.GPJ

PROJECT *Kekaha Landfill Phase II, Cell 2 Lateral Expansion* JOB NUMBER *8950-023*

LOCATION *Kekaha, Kauai, Hawaii* DATE *8/6/2012* DRAWN BY *LML*



LOCATION	SAMPLE DEPTH (feet)	SOIL DESCRIPTION
B-5	39.5	Light brownish gray poorly graded coralline sand (SP)

	BEFORE TEST	AFTER TEST
MOISTURE CONTENT (%)	29	28
DRY DENSITY (pcf)	93	97

CONSOLIDATION TEST DATA

Pacific Geotechnical Engineers, Inc.

PLATE 8.3

PACIFIC GEOTECHNICAL - CONSOLIDATION - PACIFIC GEOTECHNICAL FINAL DATA TEMPLATE.GDT - 8/6/12 17:04 - J:\8950-023\ENGINEERING\GINT\8950-023 B-LOGS_LAB.GPJ

ATTACHMENT 3

OUTPUT SPREADSHEET

Cell 2 Settlement Calculations

Base Geomembrane Liner Strain and Leachate Collection Line Grades

Input Parameters

MSW Unit Weight	62 pcf
Cover Soils Unit Weight	130 pcf
Drainage Layer Unit Weight	130 pcf
Drainage Layer Thickness	3 ft
Final Cover Thickness	3 ft
Foundation Soil Elastic Modulus	280000 psf
Depth of Foundation Soil	40 ft

Point	Station	Design Base Elev	Design Final Cover Elev	Design Waste Thickness (ft)	Overburden Stress Δq (psf)	Elastic Settlement (ft)	Settled Base Elev (ft)	Final Leachate Pipe Slope (%)	Design Slope Length (ft)	Settled Slope Length (ft)	Geomembrane Tensile Strain (%)
Section A-A											
A	0	11.3	63.7	52.4	4029	0.58	10.72	--	--	--	--
B	400	19.0	72.2	53.2	4078	0.58	18.42	1.92	400.07	400.07	0.00
Section B-B											
C	0	14.2	71.5	57.3	4333	0.62	13.58	--	--	--	--
D	160	11.0	70.0	59.0	4438	0.63	10.37	-2.01	160.03	160.03	0.00
E	300	13.8	67.4	53.6	4103	0.59	13.21	2.03	140.03	140.03	0.00
F	411	16.0	66.3	50.3	3899	0.56	15.44	2.01	111.02	111.02	0.00

Appendix B.1B
Cell 2 Expansion Phase I Sideslope Settlement Analysis

Client	County of Kaua'i	**Subject**	Cell 2 Expansion		
Project	Kekaha Landfill	Phase I Sideslope Settlement	**Prepared By**	SRK	**Date** 12/3/12
Cell 2 Lateral Expansion		Analysis	**Reviewed By**	JMT	**Date** 12/3/12
60197394			**Approved By**	KJB	**Date** 12/13/12

PHASE I SIDESLOPE SETTLEMENT ANALYSIS

Objective

Estimate the settlement of the Phase I waste due to the proposed Cell 2 expansion as the result of the increased overburden pressure from the Cell 2 waste. Estimate the strain in the new Cell 2 liner that will result from this settlement.

Design Criteria and Assumptions

1. The unit weight of municipal solid waste placed in Cell 2 is 62 pcf assuming the use of daily cover soils; Phase I waste was assumed to have a unit weight of approximately 72 pcf due to the Phase I age and waste type.
2. For conservative analysis, the waste mass to be placed on the overlay area is assumed to be immediate and at full depth. Actual placement of the waste mass will occur in lifts over time which will allow the existing waste mass to accommodate the incremental loading resulting in lower estimated settlement.
3. For additional conservatism, the cross section of the overlay being evaluated is assumed to be all waste mass.
4. Maximum allowable elongation at yield (strain) of HDPE geomembranes (textured and smooth) is 13%. Per Reference 1, GCLs can accommodate 6 to 20% strain.
5. All references to elevations correlate to the site specific vertical datum.

Methodology

The sideslope of Phase I will experience several forms of settlement due to the overlaying of Cell 2. This settlement includes the primary and secondary settlement of existing Phase I waste mass and the elastic settlement of the underlying inplace sandy soils. Settlement does not occur equally along the slope as the crest of slope will see minimal settlement as this is where the final cover of Cell 2 terminates, the midpoint of the slope sees settlement of existing Phase I waste mass and elastic settlement of foundation soils, and the toe of slope only see elastic settlement as Cell 2 is over foundation soils. These areas of settlement are shown in Attachment 1.

Calculations

Two cross sections were analyzed for the primary and secondary settlement and elongation of the proposed overlay liner of Cell 2 over the existing Phase I waste. Settlement was calculated in the middle of the Phase I slope. In cross section A-A', the top of Cell 2 final cover elevation, the midpoint elevation (Phase 1 sideslope, Cell 2 base liner), and the approximate base grade elevation for Phase I were 50, 26, and 15 feet, respectively. The resulting initial waste thickness, Ho, of the existing waste is 11 feet and the waste thickness of the vertical overlay, Hv, is 24 feet.

Ho = 25.9ft – 15.2ft = 10.7 ft ➔ 11.0 ft Hv = 50.2ft – 25.9ft = 24.4 ft ➔ 24.0 ft

Assuming the existing waste has a unit weight of 72 pounds per cubic foot (pcf), the resulting existing overburden pressure, Pe, at the midpoint of the existing waste is 385 pounds per square foot (psf). Assuming the overlay waste has a unit weight of 62 pcf, the resulting increment of overburden pressure, Pi, from the vertical expansion is 1512.8 psf.

$$P_e = \frac{H_o}{2} * 72\ pcf = 396\ psf$$

$$P_i = H_v * 62\ pcf = 1488\ psf$$

A logarithmic correlation between the existing overburden and increment of overburden was performed to determine the primary settlement. A modified primary compression index, C'c, of 0.22 was used and the resulting primary settlement is 1.7 feet.

$$S_{log} = \log\frac{P_e + P_i}{P_e} = 0.7$$

$$S_1 = 0.22 * H_o * S_{log} = 1.7\ ft$$

Phase I started accepting waste in 1953. The Cell 2 expansion has 5 years worth of airspace and is anticipated to start accepting waste in 2014. The assumption was made that closure of the cell would take 1 year, and post closure monitoring would take place for 30 years. The age of the existing waste mass in Phase I at the start of filling Cell 2 would be 61 years and the site overlay and monitoring period would be 96 years.

$age_{existing}$ = 2014 – 1953 = 61 years

$age_{sitelife}$ = $age_{existing}$ +5+1+30 = 97 years

Because secondary settlement is a function of time and impacted by the primary settlement, the secondary settlement was calculated using the logarithmic correlation between the site overlay life and monitoring period and the age of the existing life, the existing waste mass thickness after primary settlement, and a modified secondary compression index of 0.03. The secondary settlement was calculated to be 0.1 feet.

$$S_{log} = \log\frac{Age_{sitelife}}{Age_{existing}} = 0.2$$

$$H = H_o - S_1 = 9.3\ ft$$

$$S_2 = 0.03 * S_{log} * H = 0.1\ ft$$

Consolidation testing performed on the foundation soils by Pacific Geotechnical Engineers, Inc. indicates that the elastic strain was approximately 0.018 in/in at a stress level of 5,000 psf. Therefore, the modulus of the soil is approximately 280,000 psf for the range of stresses applicable.

The depth to bedrock is not known and is greater than 40 feet in depth from existing ground surface. The majority of elastic settlement is expected to occur within the upper 40 feet of the foundation soil due to increased inplace soil density past 40 feet in depth according to completed soil borings. The elastic settlement is calculated by taking the increment of overburden pressure from the vertical expansion multiplied by the depth settlement is likely to occur and divided by the modulus of elasticity.

$$S_e = \frac{P_i}{E} * 40\ ft$$

$$S_e = \frac{1488\ psf}{280{,}000\ psf} * 40\ ft = 0.2\ ft$$

A total settlement of 2.0 feet is calculated by adding the primary, secondary, and elastic settlements.

$$S = S_1 + S_2 + S_e = 2.0\ ft$$

The maximum allowable elongation at yield (strain) of HDPE geomembranes (textured and smooth) is 13% and 6 to 20% strain for GCLs. The geomembrane elongation was calculated by determining the existing slope length and the resulting slope length after settlement. Using a 3.5H:1V slope, the distance between the crest of the slope to the midpoint of the slope was calculated to be approximately 37 feet. The resulting existing slope is 28.6% or 15.9˚.

$$Slope_{existing} = \frac{36.6\ ft - 25.9\ ft}{37.4\ ft} * 100 = 28.6\%$$

$$Slope_{existing} = tan^{-1}(\frac{36.6\ ft - 25.9 ft}{37.4\ ft}) = 15.9^{\circ}$$

The assumption was made that the top of the slope does not move or settle, and therefore, the geomembrane does not move or settle at the top of the slope as well. The geomembrane does settle at the midpoint and the elevation is calculated using the total settlement.

$$elevation_{midpoint} = 25.9\ ft - S = 24\ ft$$

The resulting slope due to settlement is 33.7%.

Client	County of Kaua'i	**Subject**	Cell 2 Expansion		
Project	Kekaha Landfill	Phase I Sideslope Settlement	**Prepared By**	SRK	**Date** 12/3/12
Cell 2 Lateral Expansion		Analysis	**Reviewed By**	JMT	**Date** 12/3/12
60197394			**Approved By**	KJB	**Date** 12/13/12

$$Slope_{settled} = \frac{36.6\ ft - elevation_{midpoint}}{37.4\ ft} * 100 = 33.7\%$$

The geomembrane elongation is 1.29% in tension and is calculated by correlating the existing slope length to the resulting slope length.

$$length_{existing} = \sqrt{37.4ft^2 + (36.6ft - 25.9ft)^2} = 38.9\ ft$$

$$length_{settled} = \sqrt{37.4ft^2 + (36.6ft - 24.0ft)^2} = 39.4\ ft$$

$$elongation = \frac{39.4\ ft - 38.9\ ft}{38.9\ ft} * 100 = 1.29\%$$

Similar calculations can be applied to the bottom of slope. Refer to Attachment 2 for the results of the primary, secondary, elastic, and total settlement, the resulting slope from settlement, and the geomembrane elongation for the crest, midpoint, and top of slope in cross sections A-A' and B-B'.

Conclusion

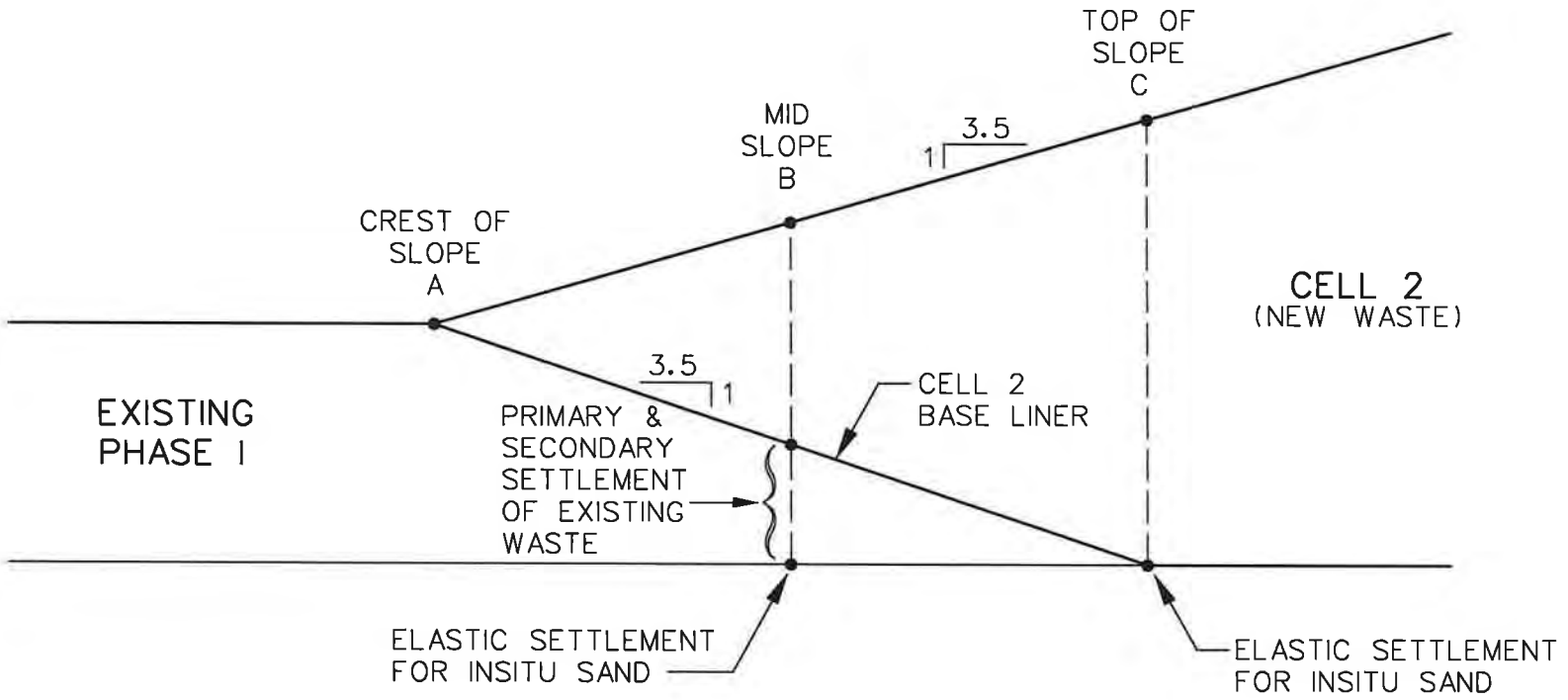
Settlement of Phase I is expected to occur due to the overburden pressure from Cell 2 horizontal overlay expansion. Total settlement ranges from 0.4 to 1.9 feet creating between 1.27 to 1.29 percent elongation (tension) in the base liner geosynthetics.

References

1. Qian, Xuede; Koerner, Robert M.; and Gray, Donald H., "Geotechnical Aspects of landfill Design and Construction", 2002, pages 449 to 451.

ATTACHMENT 1

TYPICAL PHASE I SIDESLOPE SETTLEMENT AND STRAIN CROSS SECTION



TYPICAL
PHASE I SIDESLOPE
SETTLEMENT AND STRAIN
CROSS SECTION

ATTACHMENT 2

OUTPUT SPREADSHEET

Kekaha Landfill Cell 2 Expansion
Proposed Overlay Liner Settlement and Elongation Calculation

Primary, Secondary, and Elastic Settlement of Existing Waste Mass

Unit Weight of Existing Waste, γ_{exist}: 72 pcf
Unit Weight of Overlay Waste, γ_{new}: 62 pcf

Cross Section	Modified Primary Compression Index C'c	Top of Final Cover Elevation (ft)	Top of Existing Cover Elevation (ft)	Top of Existing Cell Base Grade Elevation (ft)	Initial Waste Thickness of Existing Waste Ho (ft)	Existing Overburden Pressure at Midpoint of Existing Waste (psf)	Waste Thickness of Vertical Overlay (ft)	Increment of Overburden Pressure from Vertical Expansion (psf)	Primary Settlement (ft)	Modified Secondary Compression Index C'α	Age of Existing Waste Mass (yrs)	Site Life of Overlay and Monitoring Period (yrs)	Existing Waste Mass Thickness After Primary Settlement (ft)	Secondary Settlement (ft)	Elastic Settlement (ft)	Total Settlement (ft)
A-midslope	0.22	50.2	25.9	15.2	10.7	385.2	24.4	1512.8	**1.6**	0.03	61	97	9.1	**0.1**	**0.2**	**1.9**
A-toe of slope	0.22	60.9	15.2	15.2	0.0	0	45.7	2833.4	**0.0**	0.03	61	97	0.0	**0.0**	**0.4**	**0.4**
B-midslope	0.22	52.5	27.9	17.1	10.8	388.8	24.6	1525.2	**1.6**	0.03	61	97	9.2	**0.1**	**0.2**	**1.9**
B-toe of slope	0.22	63.3	17.1	17.1	0.0	0	46.3	2870.6	**0.0**	0.03	61	97	0.0	**0.0**	**0.4**	**0.4**

Slope Verification and Geomembrane Elongation

Cross Section and Point	Distance Between Points (ft)	Top of Existing Cover Elevation (ft)	Existing Slope (%)	Settled Existing Overlay Elevation (ft)	Resulting Slope from Settlement (%)	Existing Slope Length Between Points (ft)	Resulting Slope Length Between Points from Settlement (ft)	Geomembrane Elongation (%)
A - crest of slope	37.4	36.6	15.9453959	36.6	**-33.7**	38.9	39.4	**1.29 Tension**
A - midpoint of Phase I slope	37.4	25.9	15.9453959	24.0	**-24.5**	38.9	38.5	**-1.03 Compression**
A - toe of slope		15.2		14.8				
B - crest slope	37.9	38.7	15.9453959	38.7	**-33.6**	39.4	39.9	**1.27 Tension**
B - midpoint of Phase I slope	37.9	27.9	15.9453959	26.0	**-24.6**	39.4	39	**-1.02 Compression**
B - toe of slope		17.1		16.7				

Assumptions: Top of slope geomembrane does not move (Settled Existing Overlay Elevation = Top of Existing Cover Elevation)

Appendix B.2
Buildout Stability Analysis

AECOM

CALCULATION SHEET

Project No. 60197394

Client	County of Kaua'i	**Subject**	Final Buildout	**Prepared By** SRA	**Date** 07/24/15
Project	Kekaha Landfill		Stability	**Reviewed By** JMT	**Date** 07/24/15
	Phase II Lateral Expansion			**Approved By** NKW	**Date** 07/24/15

FINAL BUILDOUT STABILITY

Objective

Determine the stability at final buildout for the proposed Cell 2, Phase II lateral expansion at Kekaha Sanitary Landfill.

Design Criteria and Assumptions

1. The stability analysis was conducted using computer slope stability software named "Slope/W" and is part of the GeoStudio 2012 software suite. This program allows the use of several different methods for determining slope stability of circular and non-circular failure surfaces. The computer program performs an automatic search to identify potential slip surface that has the least factor of safety. The method utilized for completion of this analysis was the Morgenstern-Price Method which solves for both force and moment equilibrium.
2. Two representative cross sections were modeled for the analysis, Sections C-C and D-D. These cross section locations are presented as Figure 1 in Attachment 1.
3. The proposed final cover slope will be a maximum 3.5:1 (H:V) with a minimum 3% slope on the cover crown.
4. Leachate recirculation will not be performed.
5. Modes of Failure: The following modes of failure are considered:

    a. Circular Failure: A circular failure surface that lies entirely within the waste mass or passes through the waste mass and foundation soils.
    b. Base Liner Non-circular Failure: A non-circular failure surface originating along the base liner then up through the waste mass.

6. Factor of safety of 1.5 will be employed for determining acceptance of proposed final landfill geometry.
7. Seismic analysis will not be completed as the site specific horizontal peak ground acceleration is 0.06 g and is less than 0.10 g required by Subtitle D for seismic analysis (Reference 1).
8. All references to elevations correlate to the site specific vertical datum.

Calculations

Material Properties

Waste Mass

Waste mass properties are dependant the amount of compaction, lift thickness, composition and waste stream, moisture, and the amount of daily cover soil or alternate daily cover used. Conversations with Kekaha Landfill staff indicate a typical waste unit weight of approximately 62 pcf.

Reference 2 completed a literature review and determined resulting in internal friction angles of 25 to 51 degrees with cohesion varying from 0 to 42 kPa (0 to 877 psf) can be expected for MSW landfills. The reference concluded municipal solid waste strength properties can be considered as reasonable at friction angle of 35 degrees and cohesion 20kPa (418 psf). To provide consistency with the previous permitted documents the waste friction angle value of 33 degrees was maintained.

Waste Mass (Phase II Existing Waste and New Cell 2 Waste):

$\gamma_{buildout}$ = 62 pcf
$\phi_{buildout}$ = 33 degrees (Reference 1 and 3)
$c_{buildout}$ = varies <= 418 psf (Reference 2)

Old waste mass in Phase 1 was assumed to have a typical unit weight of 72 pcf, because it has been in place long enough to experience significant consolidation. The same strength properties were assumed as the new waste mass.

Old Waste Mass:
$\gamma_{buildout}$ = 72 pcf
$\phi_{buildout}$ = 33 degrees (References 1 and 3)
$c_{buildout}$ = varies <= 418 psf (Reference 2)

Cohesion of the waste mass was assumed to be only 200 psf for conservative analysis.

Compacted Fill

Areas of the facility that will require berming or fill soils to meet subbase grades are expected to be constructed of sandy soils. The following material properties are applied:

Compacted Fill:
γ = 115 pcf
ϕ = 30 degrees
c = 0 psf

In-situ Soils – Above Groundwater

Review of borings 1 through 5 and direct shear test results from borings 1, 3, 4 and 5 of Reference 5, which reside within the footprint of Cell 2, the typical unit weight of in-situ sandy soils was determined for soils above the groundwater elevation. The direct shear tests indicate internal friction angles between 38 to 44 degrees. The internal friction angle from boring 1 was taken for conservatism.

The following material properties are applied:

In-situ Soils:
γ = 110 pcf
ϕ = 38 degrees (conservative)
c = 0 psf (granular soils, no cohesion)

In-situ Soils – Below Groundwater

Repeating the same process for the soils located below the groundwater elevation for borings 1 through 5 of Reference 5, the typical unit weight and internal friction angle of in-situ sandy soils was determined.

The following material properties are applied:

In-situ Soils:
γ = 115 pcf
ϕ = 38 degrees (conservative)
c = 0 psf (granular soils, no cohesion)

Groundwater Elevation

Average groundwater elevation recorded during the last 15 years is slightly above 5 feet to the site specific vertical datum. For conservative design, the groundwater elevation was assumed to be 6 feet msl.

Client County of Kaua'i Subject Final Buildout Prepared By SRA Date 07/24/15
Project Kekaha Landfill Stability Reviewed By JMT Date 07/24/15
Phase II Lateral Expansion Approved By NKW Date 07/24/15

Critical Interface Friction Angle (Geosynthetics)

Interfaces exist when dissimilar materials come in contact. The frictional properties of these interfaces depend on test conditions, which include normal loading, rate of shearing, degree of submergence, size of apparatus, manufacturer, etc.

The proposed base liner system details are shown in Detail 1 (Attachment 2).

The GCL proposed for the horizontal and vertical expansion is assumed to be a composite product comprised of granular bentonite encapsulated between two nonwoven geotextiles and subsequently needle-punched.

The resulting interfaces of the proposed base liner system details and typical interface friction angles based upon Reference 3 are:

Critical Base Liner Interfaces and Friction Angles

Interface	Typical Interface Friction Test Results	
	Friction Angle (degrees)	Adhesion (kPa)
Nonwoven geotextile (separator and cushion) / granular drainage layer (gravel), operations layer (granular), or prepared subbase (granular)	33	0
Nonwoven geotextile (cushion) / textured HDPE geomembrane	25	8
Nonwoven geotextile (GCL) / textured HDPE geomembrane	23	8
Needle-punched GCL (internal friction angle)	39.7	19.9
Nonwoven geotextile / smooth HDPE Geomembrane	11	0

Upon review of the above table, the critical interface of the base liner for Cell is the interface between the nonwoven geotextile and smooth HDPE geomembrane. For conservative analysis, this interface is assumed to be 10 degrees. The critical interface was modeled as a 3-foot thick layer under the waste mass. Furthermore, a sensitivity analysis was performed for the critical section, where the friction angle of the base liner was varied from 5 degrees to 11 degrees. The purpose of the sensitivity analysis was to determine what the minimum required friction angle for the base liner is to maintain buildout stability requirements.

Conclusions

The results of the computer modeling are presented in the following table. Individual figures are presented in Attachment 3:

Section	Waste Properties (degrees, cohesion)	Failure Mode	Factor of Safety	Figure
C-C	33 degrees, 200 psf	Circular	1.83 > 1.50	1
		Block	2.77 > 1.50	2
		Foundation	3.55 > 1.50	5
D-D	33 degrees, 200 psf	Circular	2.70 > 1.50	3
		Block	2.87 > 1.50	4
		Foundation	3.42> 1.50	6

The proposed geometry of the facility and vertical expansion is expected to remain stable based upon the presented interface friction angles, soil and waste mass properties.

The slope stability results indicate that the factor of safety of the slope will remain above 1.5 for base liner friction angles greater than 6 degrees.

Interface friction angle testing will be completed prior to construction and after changes in manufacturer or significant material property changes, if possible.

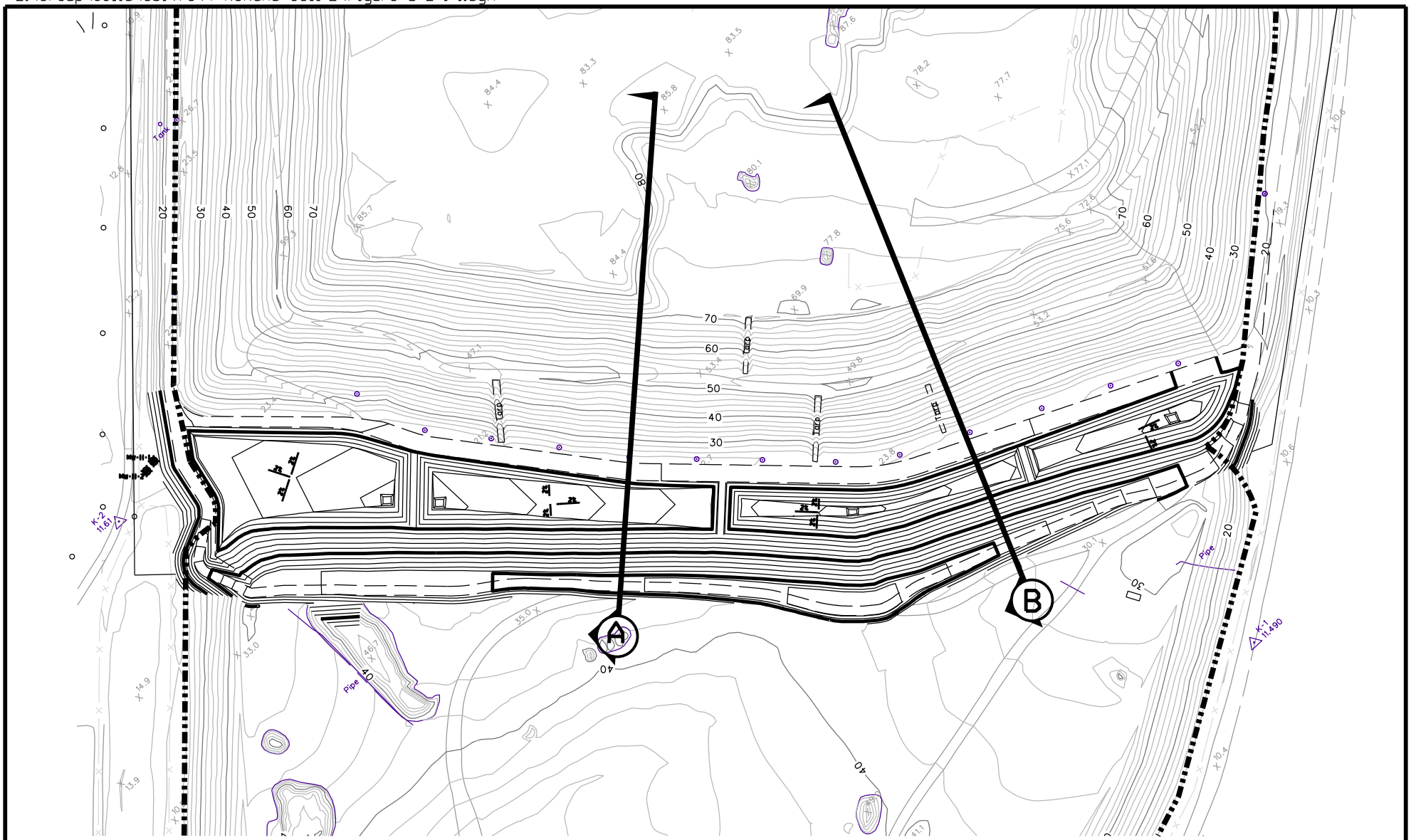
References

1. "Addendum Operations Manual For Kekaha Sanitary Landfill – Phase II," prepared by EMCON, May 1998.
2. Motan, E. Sabri; Benson, Craig H.; Tuncer, B. Edil, "Shear Strength of Municipal Solid Waste: A Laboratory Study".
3. Koerner, George R. and Najero, Dhani, "Direct Shear Database of Geosynthetic-to-Geosynthetic and Geosynthetic-to-Soil Interfaces", Geosynthetic Research Institute, GRI Report #30, June 14, 2005.
4. Drawings titled, "Engineering Report, Kekaha Landfill Phase II – Cell 2 Lateral Expansion, Kekaha Sanitary Landfill", prepared by AECOM, dated August 2012.
5. "Letter Report, Geotechnical Exploration, PGE Job No. 8950-020, for Horizontal Expansion of the Kekaha MSW Phase II Landfill, Kekaha, Kauai, Hawaii," prepared by Pacific Geotechnical Engineers, Inc., dated August 8, 2012.

ATTACHMENT 1

CROSS SECTION LOCATIONS FOR FINAL BUILDOUT STABILITY ANALYSIS

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SCALE 0' 100' 200'

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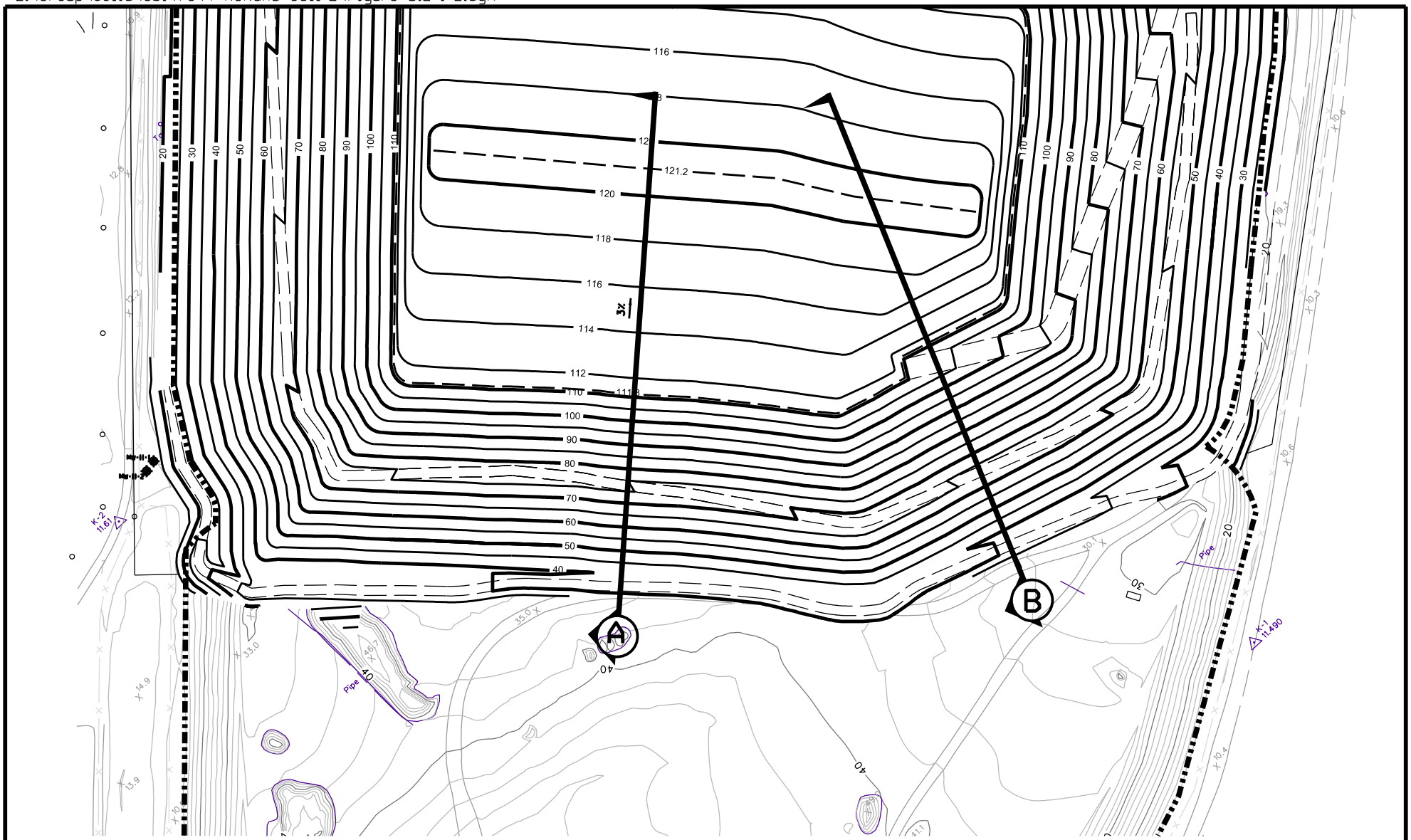
FIGURE B.2-1-1
CROSS SECTION LOCATION FOR GEOTECHNICAL ANALYSIS

CONSTRUCTION DRAWINGS PHASE II
CELL 2 LATERAL EXPANSION
KEKAHA SANITARY LANDFILL
KAUAI, HAWAII

60197394

DEC. 2015

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FIGURE B.2-1-2
CROSS SECTION LOCATION FOR GEOTECHNICAL ANALYSIS

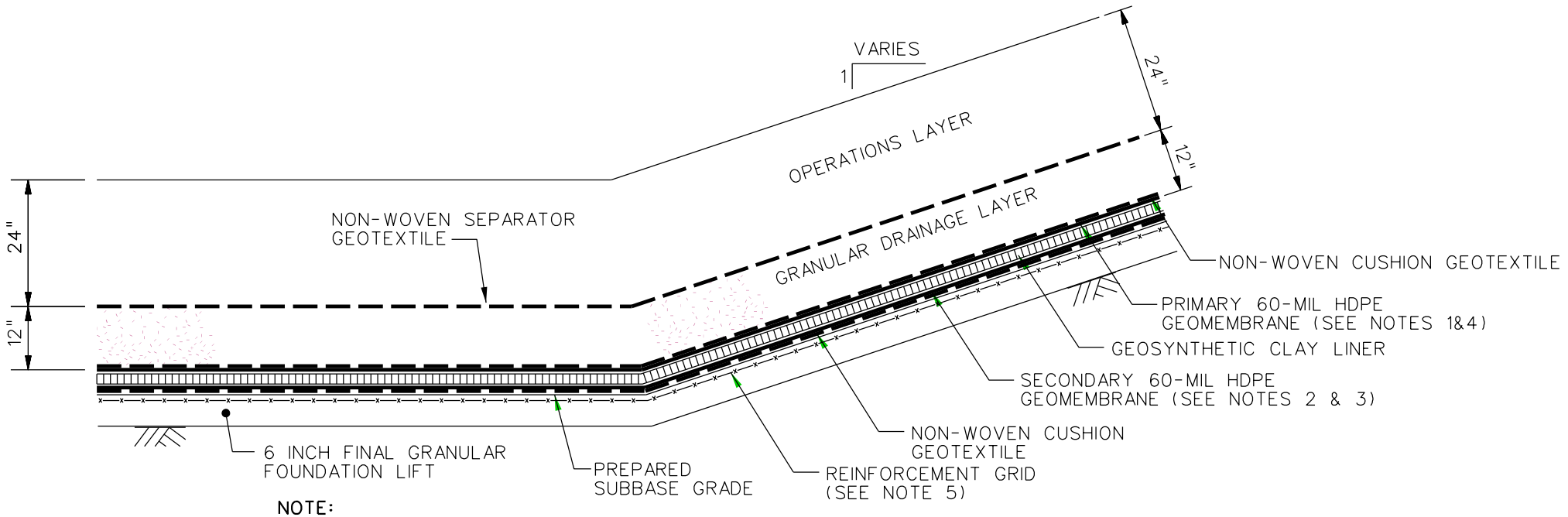
CONSTRUCTION DRAWINGS PHASE II
CELL 2 LATERAL EXPANSION
KEKAHA SANITARY LANDFILL
KAUAI, HAWAII

60197394

DEC. 2015

ATTACHMENT 2

BASE LINER SYSTEM DETAILS



NOTE:

1. PRIMARY 60-MIL HDPE GEOMEMBRANE OVER PHASE I SIDESLOPE IS SINGLE-SIDED TEXTURED. SMOOTH SIDE OF GEOMEMBRANE IS UP AND IN DIRECT CONTACT WITH NON-WOVEN CUSHION GEOTEXTILE FOR THE GRANULAR DRAINAGE LAYER. EXTEND PRIMARY GEOMEMBRANE 5 FEET PAST TOE OF SLOPE.
2. SECONDARY 60-MIL HDPE GEOMEMBRANE OVER PHASE 1 SIDESLOPE IS SINGLE-SIDED TEXTURED. SMOOTH SIDE OF GEOMEMBRANE IS DOWN AND IN CONTACT WITH NON-WOVEN CUSHION GEOTEXTILE. EXTEND GEOMEMBRANE 5 FEET PAST TOE OF SLOPE.
3. SECONDARY 60-MIL HDPE GEOMEMBRANE ON PHASE II SIDESLOPE AND CELL 2 BASE IS TEXTURED ON BOTH SIDES.
4. PRIMARY 60-MIL HDPE GEOMEMBRANE ON PHASE II SIDESLOPE AND CELL 2 BASE IS TEXTURED ON BOTH SIDES.
5. REINFORCEMENT GRID TO BE INSTALLED ON PHASE I SIDESLOPES IN SUBCELLS A AND D.

BASE LINER DETAIL 1

NTS

ATTACHMENT 3

OUTPUT FIGURES 1 THROUGH 6

Figure 1
Cross Section C-C
Circular Failure

Name: Foundation Soils - AGWT Model: Mohr-Coulomb Unit Weight: 110 pcf Cohesion': 0 psf Phi': 38 ° Phi-B: 0 ° Piezometric Line: 1
Name: Phase II Waste Model: Mohr-Coulomb Unit Weight: 62 pcf Cohesion': 200 psf Phi': 33 ° Phi-B: 0 ° Piezometric Line: 1
Name: Cover Model: Mohr-Coulomb Unit Weight: 115 pcf Cohesion': 0 psf Phi': 30 ° Phi-B: 0 ° Piezometric Line: 1
Name: Base Liner Model: Mohr-Coulomb Unit Weight: 115 pcf Cohesion': 0 psf Phi': 6 ° Phi-B: 0 ° Piezometric Line: 1
Name: Foundation Soils - BGWT Model: Mohr-Coulomb Unit Weight: 115 pcf Cohesion': 0 psf Phi': 38 ° Phi-B: 0 ° Piezometric Line: 1
Name: Phase I Waste Model: Mohr-Coulomb Unit Weight: 72 pcf Cohesion': 200 psf Phi': 33 ° Phi-B: 0 ° Piezometric Line: 1
Name: New Waste Model: Mohr-Coulomb Unit Weight: 62 pcf Cohesion': 200 psf Phi': 33 ° Phi-B: 0 ° Piezometric Line: 1

FACTOR OF SAFETY: 1.83

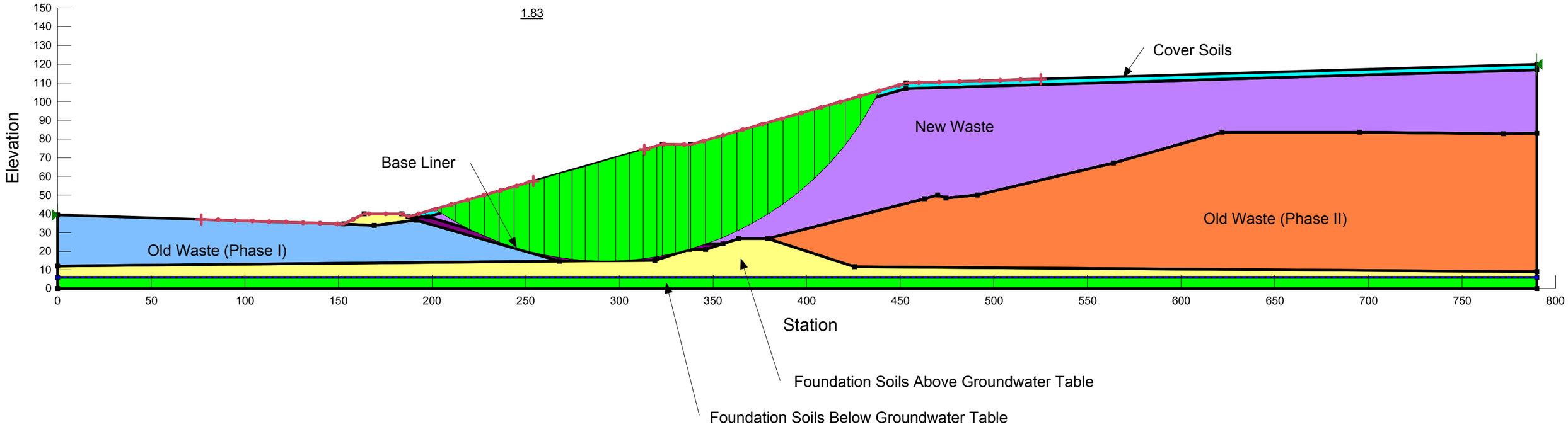


Figure 2
Cross Section C-C
Block Failure

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Name: Cover Model: Mohr-Coulomb Unit Weight: 115 pcf Cohesion': 0 psf Phi': 30 ° Phi-B: 0 ° Piezometric Line: 1
Name: Base Liner Model: Mohr-Coulomb Unit Weight: 115 pcf Cohesion': 0 psf Phi': 6 ° Phi-B: 0 ° Piezometric Line: 1
Name: Foundation Soils - BGWT Model: Mohr-Coulomb Unit Weight: 115 pcf Cohesion': 0 psf Phi': 38 ° Phi-B: 0 ° Piezometric Line: 1
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Name: New Waste Model: Mohr-Coulomb Unit Weight: 62 pcf Cohesion': 200 psf Phi': 33 ° Phi-B: 0 ° Piezometric Line: 1

FACTOR OF SAFETY: 2.77

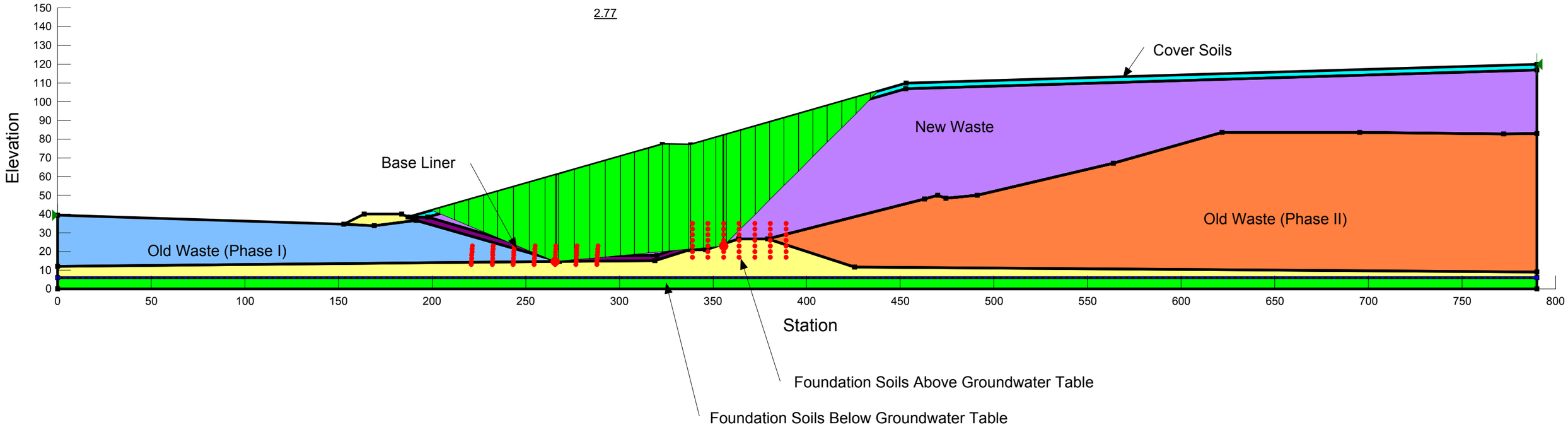


Figure 3
Cross Section D-D
Circular Failure

Name: Foundation Soils - AGWT Model: Mohr-Coulomb Unit Weight: 110 pcf Cohesion': 0 psf Phi': 38 ° Phi-B: 0 ° Piezometric Line: 1
Name: Phase II Waste Model: Mohr-Coulomb Unit Weight: 62 pcf Cohesion': 200 psf Phi': 33 ° Phi-B: 0 ° Piezometric Line: 1
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Name: Base Liner Model: Mohr-Coulomb Unit Weight: 115 pcf Cohesion': 0 psf Phi': 6 ° Phi-B: 0 ° Piezometric Line: 1
Name: Foundation Soils - BGWT Model: Mohr-Coulomb Unit Weight: 115 pcf Cohesion': 0 psf Phi': 38 ° Phi-B: 0 ° Piezometric Line: 1
Name: Phase I Waste Model: Mohr-Coulomb Unit Weight: 72 pcf Cohesion': 200 psf Phi': 33 ° Phi-B: 0 ° Piezometric Line: 1
Name: New Waste Model: Mohr-Coulomb Unit Weight: 62 pcf Cohesion': 200 psf Phi': 33 ° Phi-B: 0 ° Piezometric Line: 1

FACTOR OF SAFETY: 2.70

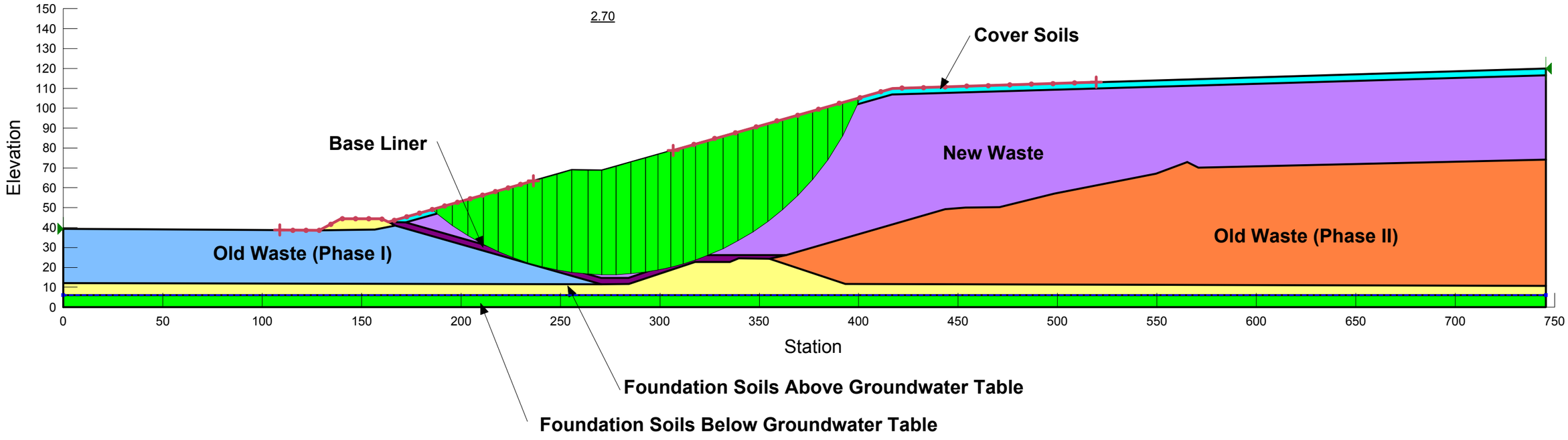


Figure 4
Cross Section D-D
Block Failure

Name: Foundation Soils - AGWT Model: Mohr-Coulomb Unit Weight: 110 pcf Cohesion': 0 psf Phi': 38 ° Phi-B: 0 ° Piezometric Line: 1
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Name: Base Liner Model: Mohr-Coulomb Unit Weight: 115 pcf Cohesion': 0 psf Phi': 6 ° Phi-B: 0 ° Piezometric Line: 1
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Name: Phase I Waste Model: Mohr-Coulomb Unit Weight: 72 pcf Cohesion': 200 psf Phi': 33 ° Phi-B: 0 ° Piezometric Line: 1
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FACTOR OF SAFETY: 2.87

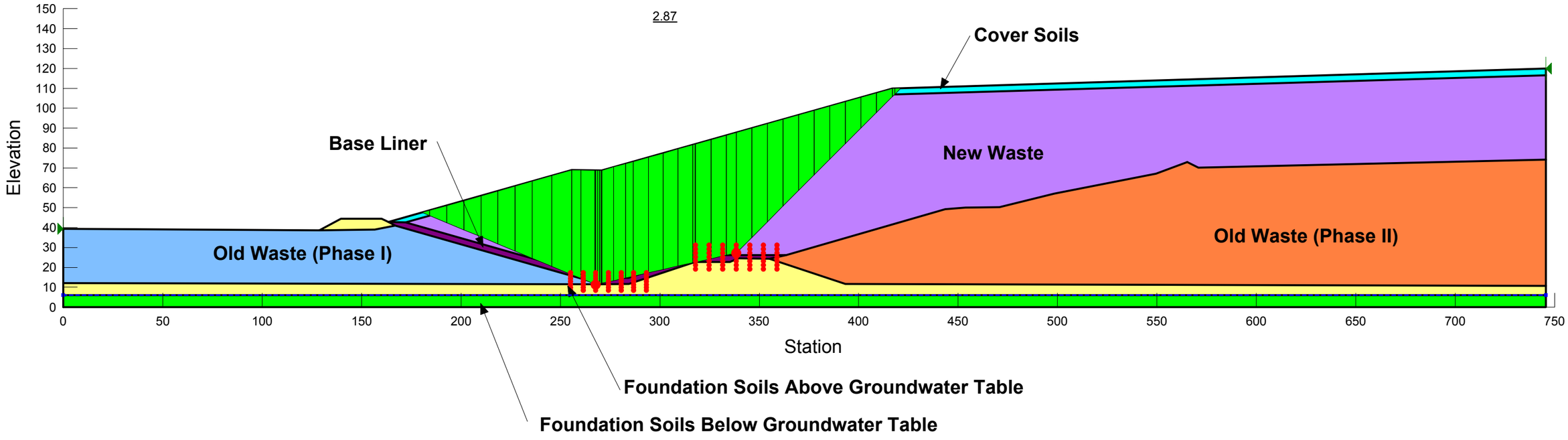


Figure 5
Cross Section C-C
Deep Circular Failure

Name: Foundation Soils - AGWT Model: Mohr-Coulomb Unit Weight: 110 pcf Cohesion': 0 psf Phi': 38 ° Phi-B: 0 ° Piezometric Line: 1
Name: Phase II Waste Model: Mohr-Coulomb Unit Weight: 62 pcf Cohesion': 200 psf Phi': 33 ° Phi-B: 0 ° Piezometric Line: 1
Name: Cover Model: Mohr-Coulomb Unit Weight: 115 pcf Cohesion': 0 psf Phi': 30 ° Phi-B: 0 ° Piezometric Line: 1
Name: Base Liner Model: Mohr-Coulomb Unit Weight: 115 pcf Cohesion': 0 psf Phi': 6 ° Phi-B: 0 ° Piezometric Line: 1
Name: Foundation Soils - BGWT Model: Mohr-Coulomb Unit Weight: 115 pcf Cohesion': 0 psf Phi': 38 ° Phi-B: 0 ° Piezometric Line: 1
Name: Phase I Waste Model: Mohr-Coulomb Unit Weight: 72 pcf Cohesion': 200 psf Phi': 33 ° Phi-B: 0 ° Piezometric Line: 1
Name: New Waste Model: Mohr-Coulomb Unit Weight: 62 pcf Cohesion': 200 psf Phi': 33 ° Phi-B: 0 ° Piezometric Line: 1

FACTOR OF SAFETY: 3.56

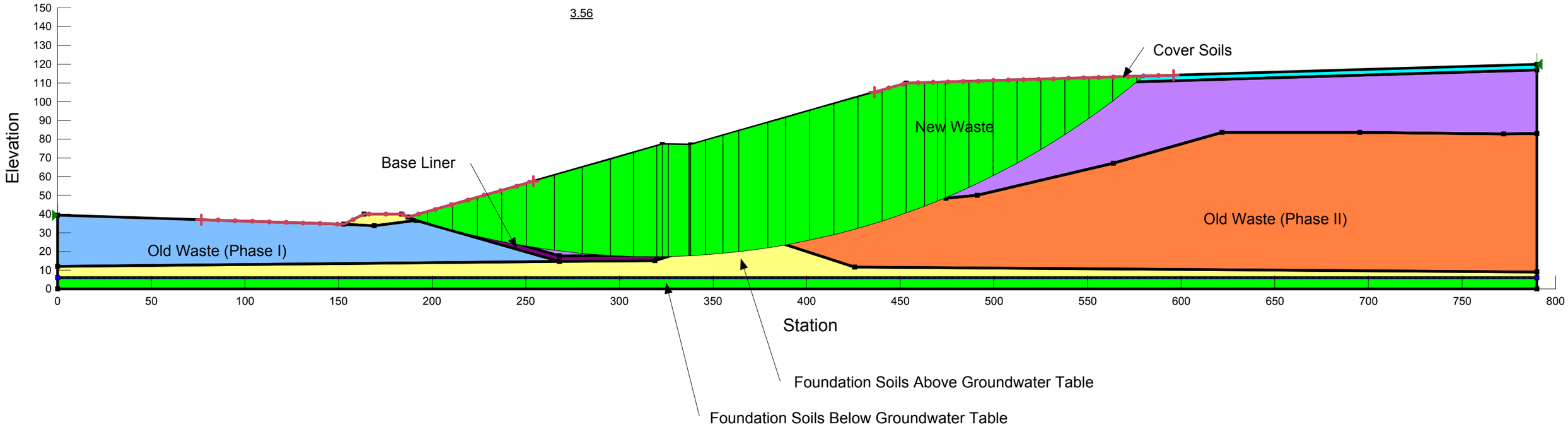
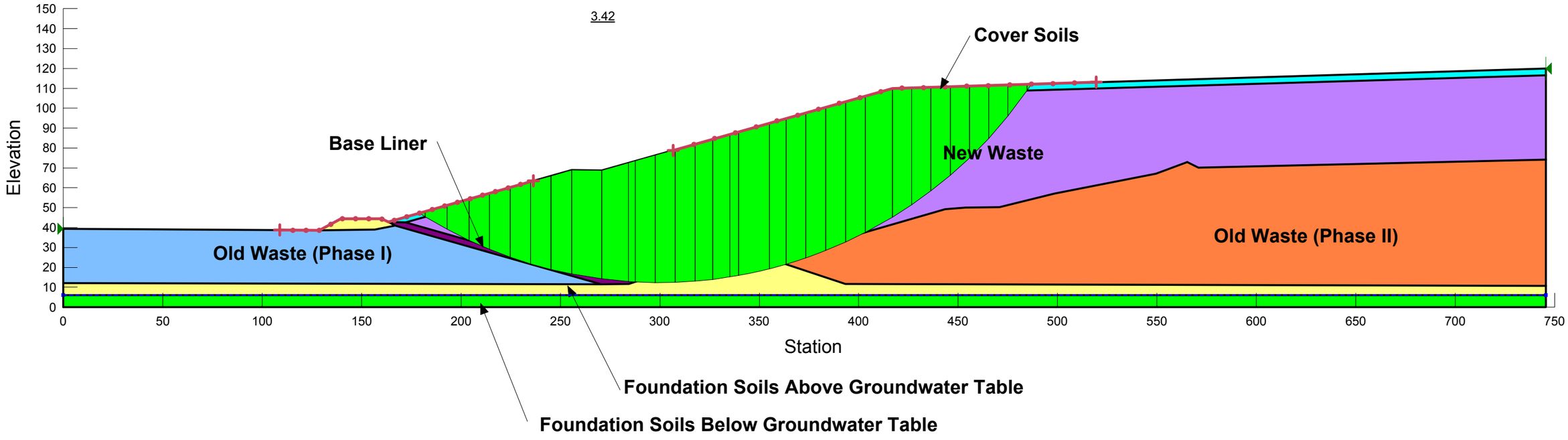


Figure 6
Cross Section D-D
Deep Circular Failure

Name: Foundation Soils - AGWT Model: Mohr-Coulomb Unit Weight: 110 pcf Cohesion': 0 psf Phi': 38 ° Phi-B: 0 ° Piezometric Line: 1
Name: Phase II Waste Model: Mohr-Coulomb Unit Weight: 62 pcf Cohesion': 200 psf Phi': 33 ° Phi-B: 0 ° Piezometric Line: 1
Name: Cover Model: Mohr-Coulomb Unit Weight: 115 pcf Cohesion': 0 psf Phi': 30 ° Phi-B: 0 ° Piezometric Line: 1
Name: Base Liner Model: Mohr-Coulomb Unit Weight: 115 pcf Cohesion': 0 psf Phi': 6 ° Phi-B: 0 ° Piezometric Line: 1
Name: Foundation Soils - BGWT Model: Mohr-Coulomb Unit Weight: 115 pcf Cohesion': 0 psf Phi': 38 ° Phi-B: 0 ° Piezometric Line: 1
Name: Phase I Waste Model: Mohr-Coulomb Unit Weight: 72 pcf Cohesion': 200 psf Phi': 33 ° Phi-B: 0 ° Piezometric Line: 1
Name: New Waste Model: Mohr-Coulomb Unit Weight: 62 pcf Cohesion': 200 psf Phi': 33 ° Phi-B: 0 ° Piezometric Line: 1

FACTOR OF SAFETY: 3.42



Appendix B.3
Drainage Layer Stability Analysis With Equipment

Appendix B.3A
Drainage Layer Stability Analysis With Equipment
(Cell 2 Base Liner Sideslopes)

Client	County of Kaua'i	**Subject**	Drainage Layer	**Prepared By**	KJB	**Date** 8/13/12
Project	Kekaha Landfill		Stability Analysis with	**Reviewed By**	TCR	**Date** 8/14/12
Phase II Lateral Expansion			Equipment Loading	**Approved By**	KJB	**Date** 8/14/12

DRAINAGE LAYER STABILITY ANALYSIS WITH EQUIPMENT LOADING

Objective

Determine the maximum slope length for granular drainage layer placement and operations layer placement over the geosynthetic base liner on the sideslopes of Cell 2.

Design Criteria and Assumptions

1. The base liner systems for Cell 2 is depicted in Figure 1 from Reference 1.
2. The drainage layer material is expected to be a sub-angular or angular gravel.
3. The placement of the granular drainage layer and the operations layer will be completed in two stages. The first stage would be placement of the 1-foot granular drainage layer and installation of the separator geotextile. After installation of the geotextile the 2-foot operations layer will be placed. This analysis will evaluate both conditions including placement of the operations layer in two 1-foot lifts as well. It is also assumed that the granular drainage layer or operations layer will be inplace across the cell floor prior to placement on the sideslope.
4. This analysis will determine the minimum critical interface friction angles required for varied slope lengths from a typical length of 10 feet to the maximum slope length for the proposed expansion. Slope length will be dependant upon equipment loadings, placement of the granular drainage layer and/or the operations layer on the slope, and assumed no tension applied to the geotextile or geomembrane.
5. Slope length for granular drainage layer and operations layer placement is defined as the actual slope length, not a vertical or horizontal length.
6. Sideslopes are a maximum 3:1 (H:V) slope.
7. The dozer to be used for deployment of the drainage layer is expected to be a Caterpillar D6H LGP. This dozer has a ground pressure of less than 5 psi, typical for placement of soil material layers over geosynthetics. Model specifications are included as Attachment 1.
8. The dozer is operated from the bottom of the slope with no abrupt braking or change in direction during granular drainage layer placement.
9. Interface friction angle testing will be completed prior to cell construction and before a resulting change of geosynthetic materials, if applicable. Material properties and interface friction angle testing requirements is presented in Appendix F.

Calculations

Caterpillar D6H LGP Dozer Properties (Attachment 1, Reference 5)

Dozer track width:	36 inches
Track length on ground:	128.5 inches (10.7 ft)
Ground pressure:	4.88 psi (702.7 psf) < 5 psi

Drainage Layer Properties

Unit Weight:	130 pcf
Internal friction angle:	*34+ degrees

*Table 3.31 of Reference 2 for SM, SP, SW, GM, GP, GW soils that could be used as drainage layer materials.

For conservatism, the internal friction angle used in the model and analysis is 33 degrees which is equal to the typical interface friction angle between sand/nonwoven geotextiles and gravel/nonwoven geotextile (Table 1, Reference 4). This will allow the model to evaluate a uniform layer thickness (granular drainage layer or with the added operations layer) with the separator geotextile included to determine the critical interface friction angle needed on the sideslope base liner system.

Maximum Slope Angle

3H:1V: $\tan^{-1}(1/3) = 18.4$ *degrees*

Maximum Slope Length

Based upon visual observation of the base liner grades shown on Drawing 4 of Reference 1, the maximum slope height for Cell 1 is approximately 26 feet (vertical) on the west sideslope of Cell 2. The resulting maximum slope length is determined to be:

$$(26)\left(\sqrt{3^2+1^2}\right) = 82.2 \text{ feet, say} \Rightarrow 85 \text{ feet}$$

Typical Interface Friction Angles

A review of Reference 4 for typical peak interface friction angles and peak adhesion in relation to the proposed base liner system include:

Table 1
Critical Base Liner Interfaces and Friction Angles

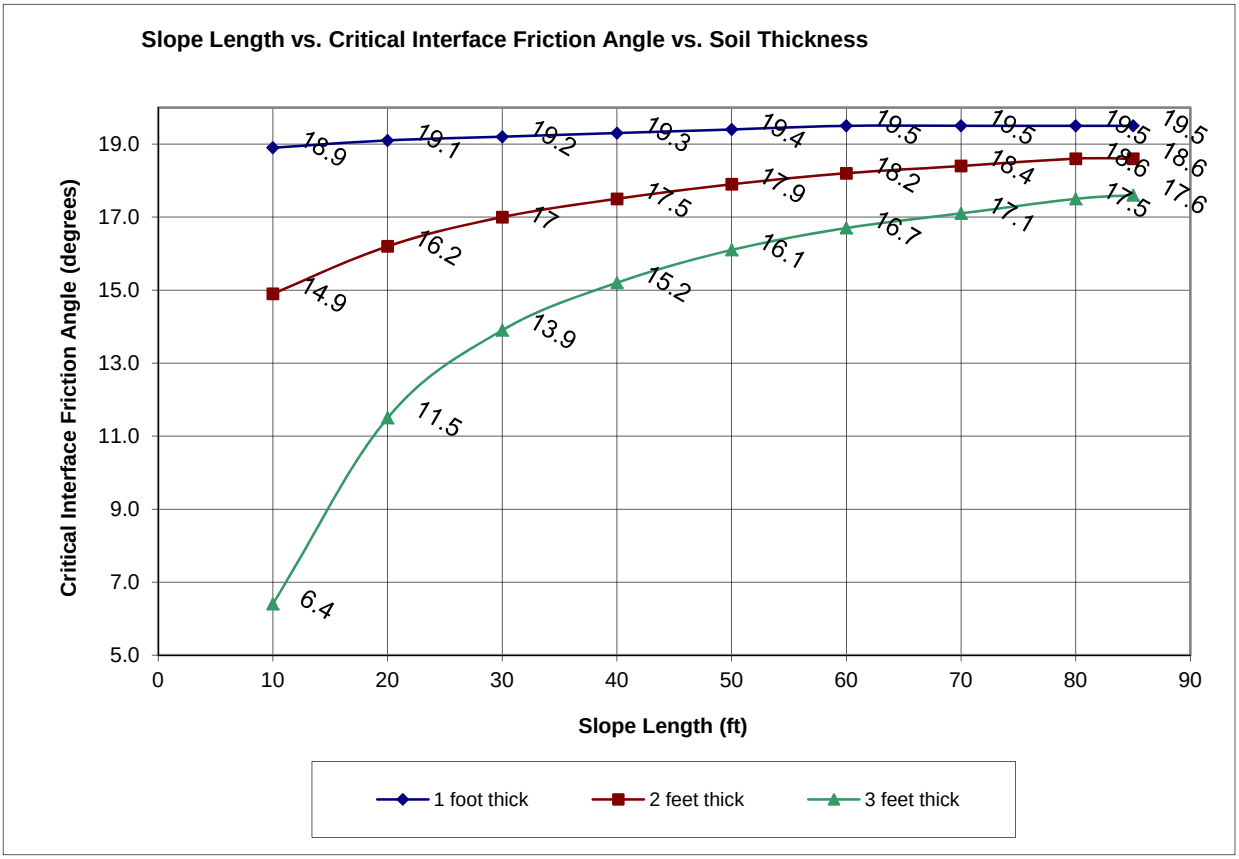
Interface	Typical Interface Friction Test Results	
	Friction Angle (degrees)	**Adhesion (kPa)**
Nonwoven geotextile (separator and cushion) / granular drainage layer (gravel), operations layer (granular), or prepared subbase (granular)	33	0
Nonwoven geotextile (cushion) / textured HDPE geomembrane	25	8
Nonwoven geotextile (GCL) / textured HDPE geomembrane	23	8
Needle-punched GCL (internal friction angle)	39.7	19.9

Client	County of Kaua'i	**Subject**	Drainage Layer	**Prepared By** KJB	**Date** 8/13/12
Project	Kekaha Landfill		Stability Analysis with	**Reviewed By** TCR	**Date** 8/14/12
Phase II Lateral Expansion			Equipment Loading	**Approved By** KJB	**Date** 8/14/12

Method of Analysis

Attached as Figures 2, 3 and 4, is a calibrated spreadsheet depicting the analysis presented by Koerner and Soong, 1998 (Reference 3). This spreadsheet depicts the driving and resisting forces experienced along the critical interface with the incorporation of equipment loading. A factor of safety of 1.1 was deemed as a qualifier for slope stability as equipment loading is a brief and temporary load condition.

Table 2
Slope Length vs. Critical Interface Friction Angles vs. Soil Thickness



By varying the slope lengths in Figures 2, 3, and 4, varying drainage layer and operations layer thickness on the sideslope, and calculating the minimum critical interface friction angle for the specific slope length, the resulting Table 2 above was derived. The table itself specifically defines the critical interface solely as friction without the application of apparent adhesion that could occur between the interfaces of geosynthetic layers to geosynthetics layers or a geosynthetic layer to a soil layer. Added conservatism is provided by not considering adhesion.

Client	County of Kaua'i	**Subject**	Drainage Layer	**Prepared By**	KJB	**Date** 8/13/12
Project	Kekaha Landfill		Stability Analysis with	**Reviewed By**	TCR	**Date** 8/14/12
Phase II Lateral Expansion			Equipment Loading	**Approved By**	KJB	**Date** 8/14/12

Conclusions

Upon review of Table 2, the critical placement period will be during the placement of the 1-foot granular drainage layer. In order to place the entire lift up the sideslope, a minimum critical interface friction angle of 19.5 degrees is required. As the lift thickness increases (the 1-foot drainage layer is then overlain by the operations layer) the critical interface requirement decreases as a result of the increased buttress effect at the toe of slope and distribution of the equipment loading through the granular soils.

Review of the typical peak values in Table 1 determines that the sideslope geomembrane should be textured to allow for placement of the granular drainage layer on the sideslopes without added tension to the underlying geotextile or geomembrane.

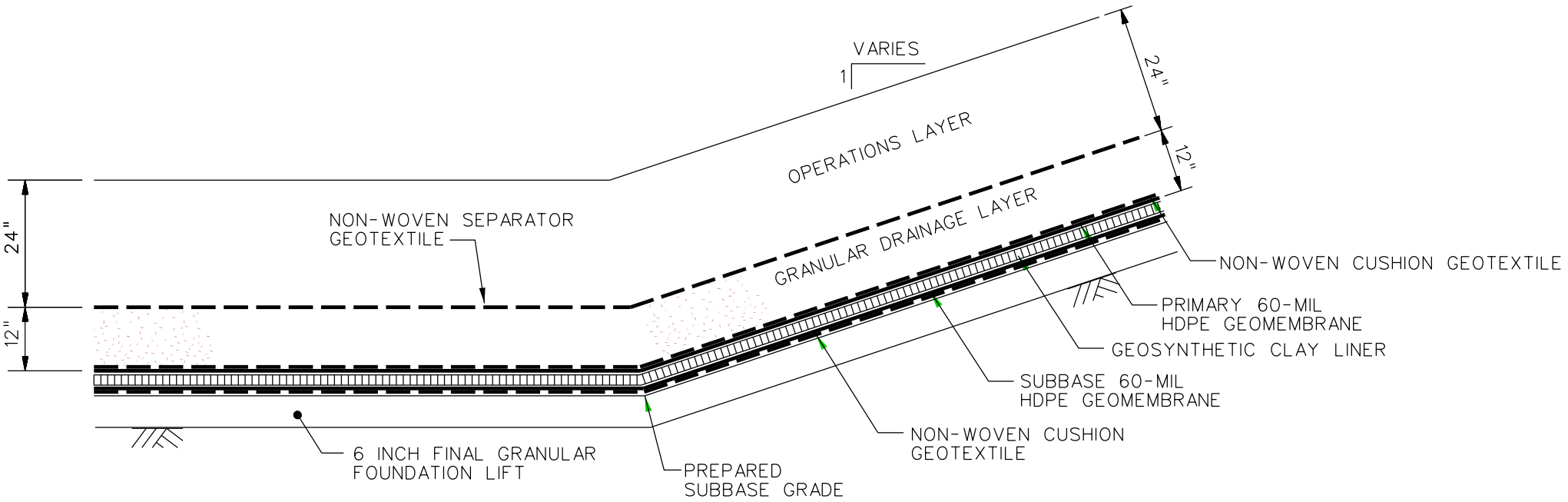
This calculation should be revisited if placement equipment is different or if there is a change in the granular drainage layer. This calculation should also be revisited for the addition of any further base liner components or removal of any of the proposed base liner components.

References

1. Drawings entitled, "Engineering Report, Phase II Cell 2 Lateral Expansion, Kekaha Landfill", prepared by AECOM, dated August 2012.
2. Table 3.31, Hunt, Roy E., "Geotechnical Engineering Investigation Manual," 1984.
3. Koerner, R. M. and Soong, T.-Y. (1998), "Analysis and Design of Veneer Cover Soils" *Proc. 6th Int. Conf. on Geosynthetics, Atlanta*, USA, IFAI, pp. 1-23.
4. Koerner, George R. and Nejaro, Dhani, "Direct Shear Database of Geosynthetic-to-Geosynthetic and Geosynthetic-to-Soil Interfaces," Geosynthetic Research Institute, GRI Report #30, June 14, 2005.
5. "Caterpillar Performance Handbook," Edition 24, pages 1-8 and 1-21.

FIGURE 1

TYPICAL CELL 2 BASE LINER SYSTEM DETAILS



NOTE:

1. ALL GEOMEMBRANE IS TEXTURED ON BOTH SIDES.

BASE LINER DETAIL 1

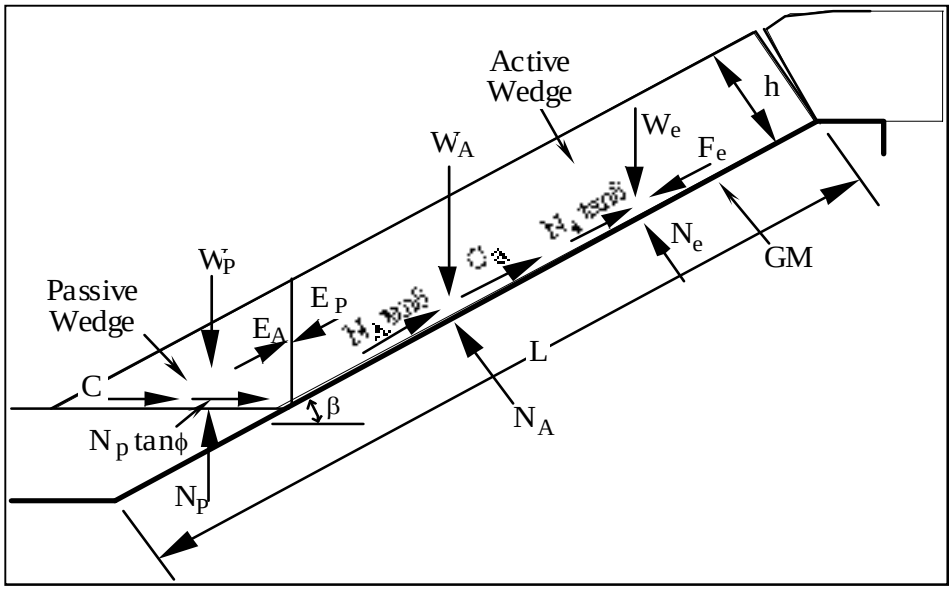
NTS

FIGURE 2

DRAINAGE LAYER STABILITY CALCULATION WITH EQUIPMENT LOADING (MAXIMUM SLOPE LENGTH ONLY, 1-FOOT GRANULAR DRAINAGE LAYER)

Figure 2

Uniform Cover Soil Thickness with the Incorporation of Equipment Loads
1-foot Thickness (1-foot Granular Drainage Layer Placed Only)



Calculation of FS

Active Wedge:

Wa=	*10616.5*	*kpounds*
Na=	*10073.8*	*kpounds*

Passive Wedge:

Wp=	*217.0*	*kpounds*

$$FS = \frac{-b + \sqrt{b^2 - 4ac}}{2a}$$

a=	*5364.2*
b=	*-7025*
c=	*1236.9*

FS= 1.10

thickness of cover soil = h =	1.00	ft	
soil slope angle beneath the geomembrane = β =	18.4	°	*= 0.32 (rad.)*
finished cover soil slope angle = ω =	18.4	°	*= 0.32 (rad.)*
length of slope measured along the geomembrane = L =	85.0	ft	
unit weight of the cover soil = γ =	130.0	pcf	
friction angle of the cover soil = ϕ =	33.0	°	*= 0.58 (rad.)*
cohesion of the cover soil = c =	0.0	psf	*C= 0 kpounds*
interface friction angle between cover soil and geomembrane = δ =	19.5	°	*= 0.34 (rad.)*
adhesion between cover soil and geomembrane = ca =	0.0	psf	*Ca= 0 kpounds*

thickness of cover soil = h =	1.00	ft	*b/h= 3.0*
equipment ground pressure (= wt. of equipment/(2wb)) = q =	702.7	psf	*We=q w l= 7293.3*
length of each equipment track = w =	10.7	ft	*Ne=Wecos β = 6920.5*
width of each equipment track = b =	3.0	ft	*Fe=We (a/g) = 0.0*
influence factor* at geomembrane interface = I =	0.97		
acceleration/deceleration of the bulldozer = a =	0.00	g	

*Influence Factor Default Values

Cover Soil Thickness	Equipment Track Width		
	Very Wide	Wide	Standard
[2] 300 mm	1.00	0.97	0.94
300-1000 mm	0.97	0.92	0.70
[3] 1000 mm	0.95	0.75	0.30

Note: numbers in boxes are input values

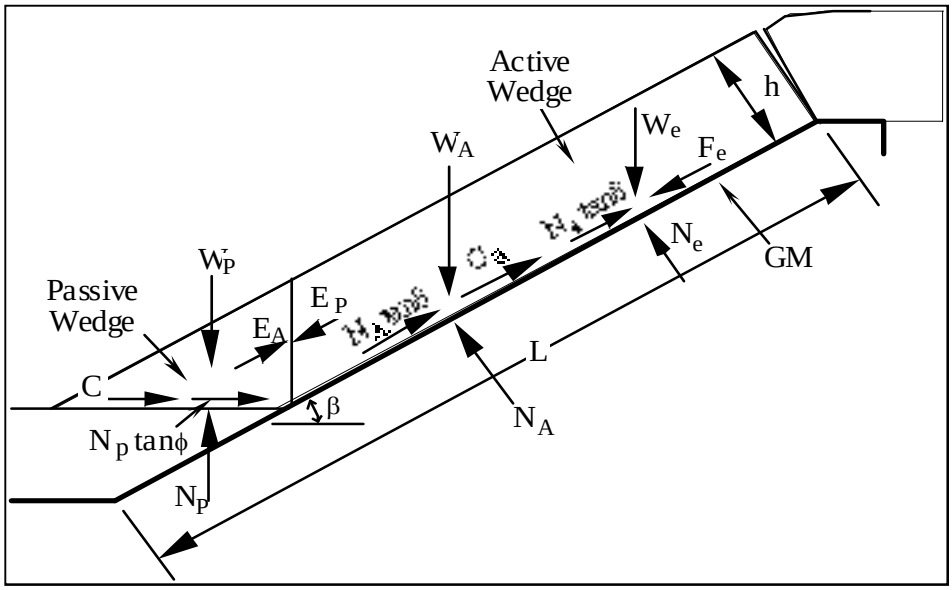
numbers in Italics are calculated values

FIGURE 3

DRAINAGE LAYER STABILITY CALCULATION WITH EQUIPMENT LOADING (MAXIMUM SLOPE LENGTH ONLY, 1-FOOT GRANULAR DRAINAGE LAYER WITH 1 FOOT OF OPERATIONS LAYER)

Figure 3

Uniform Cover Soil Thickness with the Incorporation of Equipment Loads
2-foot Thickness (1-foot Granular Drainage Layer and 1-foot Operations Layer)



Calculation of FS

Active Wedge:

Wa=	*20366.1*	*kpounds*
Na=	*19324.9*	*kpounds*

Passive Wedge:

Wp=	*868.1*	*kpounds*

$$FS = \frac{-b + \sqrt{b^2 - 4ac}}{2a}$$

a=	*8171.7*
b=	*-10616*
c=	*1790.3*

FS= 1.10

thickness of cover soil = h =	2.00	ft	
soil slope angle beneath the geomembrane = β =	18.4	°	*= 0.32 (rad.)*
finished cover soil slope angle = ω =	18.4	°	*= 0.32 (rad.)*
length of slope measured along the geomembrane = L =	85.0	ft	
unit weight of the cover soil = γ =	130.0	pcf	
friction angle of the cover soil = ϕ =	33.0	°	*= 0.58 (rad.)*
cohesion of the cover soil = c =	0.0	psf	*C= 0 kpounds*
interface friction angle between cover soil and geomembrane = δ =	18.6	°	*= 0.33 (rad.)*
adhesion between cover soil and geomembrane = ca =	0.0	psf	*Ca= 0 kpounds*

thickness of cover soil = h =	2.00	ft	*b/h= 1.5*
equipment ground pressure (= wt. of equipment/(2wb)) = q =	702.7	psf	*We=q w l= 6917.4*
length of each equipment track = w =	10.7	ft	*Ne=Wecos β = 6563.7*
width of each equipment track = b =	3.0	ft	*Fe=We (a/g) = 0.0*
influence factor* at geomembrane interface = I =	0.92		
acceleration/deceleration of the bulldozer = a =	0.00	g	

*Influence Factor Default Values

Cover Soil Thickness	Equipment Track Width		
	Very Wide	Wide	Standard
[2] 300 mm	1.00	0.97	0.94
300-1000 mm	0.97	0.92	0.70
[3] 1000 mm	0.95	0.75	0.30

Note: numbers in boxes are input values

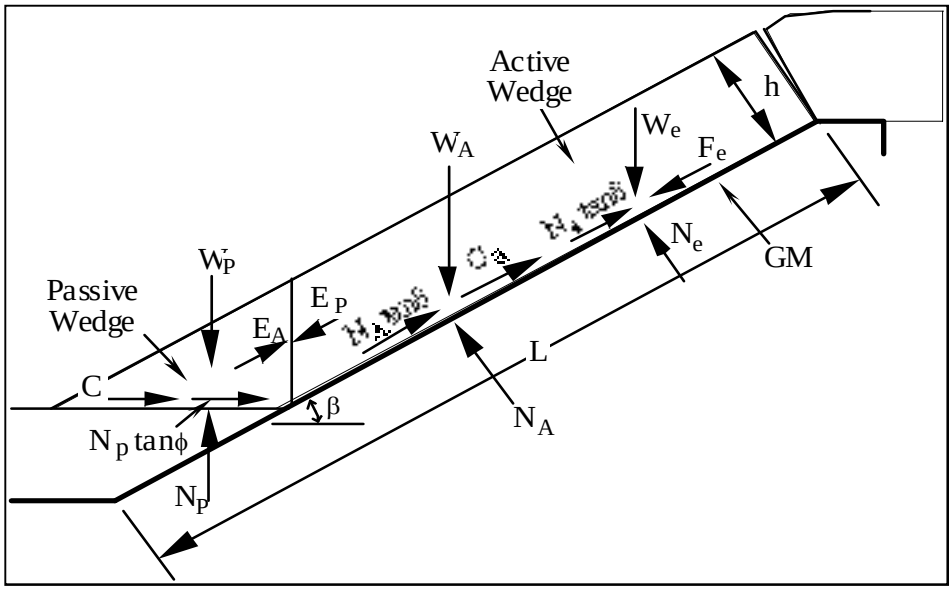
numbers in Italics are calculated values

FIGURE 4

DRAINAGE LAYER STABILITY CALCULATION WITH EQUIPMENT LOADING (MAXIMUM SLOPE LENGTH ONLY, 1-FOOT GRANULAR DRAINAGE LAYER WITH 2 FEET OF OPERATIONS LAYER)

Figure 4

Uniform Cover Soil Thickness with the Incorporation of Equipment Loads
3-foot Thickness (1-foot Granular Drainage Layer and 2-foot Operations Layer)



Calculation of FS

Active Wedge:

Wa=	*29248.7*	*kpounds*
Na=	*27753.4*	*kpounds*

Passive Wedge:

Wp=	*1953.2*	*kpounds*

$$FS = \frac{-b + \sqrt{b^2 - 4ac}}{2a}$$

a=	*10832.2*
b=	*-13953*
c=	*2234.8*

FS= 1.10

thickness of cover soil = h =	3.00	ft	
soil slope angle beneath the geomembrane = β =	18.4	°	*= 0.32 (rad.)*
finished cover soil slope angle = ω =	18.4	°	*= 0.32 (rad.)*
length of slope measured along the geomembrane = L =	85.0	ft	
unit weight of the cover soil = γ =	130.0	pcf	
friction angle of the cover soil = ϕ =	33.0	°	*= 0.58 (rad.)*
cohesion of the cover soil = c =	0.0	psf	*C= 0 kpounds*
interface friction angle between cover soil and geomembrane = δ =	17.6	°	*= 0.31 (rad.)*
adhesion between cover soil and geomembrane = ca =	0.0	psf	*Ca= 0 kpounds*

thickness of cover soil = h =	3.00	ft	*b/h= 1.0*
equipment ground pressure (= wt. of equipment/(2wb)) = q =	702.7	psf	*We=q w l= 6917.4*
length of each equipment track = w =	10.7	ft	*Ne=Wecos β = 6563.7*
width of each equipment track = b =	3.0	ft	*Fe=We (a/g) = 0.0*
influence factor* at geomembrane interface = I =	0.92		
acceleration/deceleration of the bulldozer = a =	0.00	g	

*Influence Factor Default Values

Cover Soil Thickness	Equipment Track Width		
	Very Wide	Wide	Standard
[2] 300 mm	1.00	0.97	0.94
300-1000 mm	0.97	0.92	0.70
[3] 1000 mm	0.95	0.75	0.30

Note: numbers in boxes are input values

numbers in Italics are calculated values

ATTACHMENT 1

EQUIPMENT PROPERTIES FOR CATERPILLAR D6H LGP DOZER






MODEL	D6H XR Series II		D6H LGP Series II		D7G		D7H Series II	
Flywheel Power	130 kW	175 hp	127 kW	170 hp	149 kW	200 hp	160 kW	215 hp
Operating Weight*								
(Power Shift)	18 684 kg	41,192 lb	19 814 kg	43,590 lb	20 666 kg	45,560 lb	24 694 kg	54,401 lb
(Direct Drive)	—		19 989 kg	43,976 lb	20 510 kg	45,218 lb	—	
(Power Shift Differential Steer)	18 799 kg	41,444 lb	—		—		24 993 kg	55,100 lb
Engine Model	3306		3306		3306		3306	
Rated Engine RPM	1900		1900		2000		2100	
No. of Cylinders	6		6		6		6	
Bore	121 mm	4.75"	121 mm	4.75"	121 mm	4.75"	121 mm	4.75"
Stroke	152 mm	6"	152 mm	6"	152 mm	6"	152 mm	6"
Displacement	10.5 L	638 in³	10.5 L	638 in³	10.5 L	638 in³	10.5 L	638 in³
Track Rollers (Each Side)	7		8		6		7	
Width of Standard Track Shoe	560 mm	22"	915 mm	36"	508 mm	20"	560 mm	22"
Length of Track on Ground	2.77 m	9'1"	3.27 m	10'8.5"	2.70 m	8'11"	2.90 m	9'6"
Ground Contact Area (W/Std. Shoe)	3.11 m²	4815 in²	5.97 m²	9254 in²	2.76 m²	4280 in²	3.24 m²	5016 in²
Track Gauge	1.88 m	6'2"	2.23 m	7'3"	1.98 m	6'5"	1.98 m	6'6"
GENERAL DIMENSIONS:								
Height (Stripped Top)**	2.26 m	7'5"	2.32 m	7'7"	2.27 m	7'5"	2.44 m	8'0"
Height (To Top of ROPS)	3.12 m	10'3"	3.16 m	10'5"	3.20 m	10'6"	3.33 m	10'11"
Height (To Top of Cab ROPS)	3.12 m	10'3"	3.16 m	10'5"	—		3.42	11'3"
Height (To Top of ROPS Canopy)	—		3.16 m	10'5"	—		—	
Overall Length (With P Blade)	—		5.18 m	17'0"	—		—	
(Without Blade)	—		4.49 m	14'9"	—		—	
Overall Length (With S Blade)	5.26 m	17'3"	—		5.28 m	17'4"	6.03 m	19'9"
(Without Blade)	4.21 m	13'10"	—		4.19 m	13'9"	4.73 m	15'6"
Width (Over Trunnion)	2.64 m	8'8"	3.43 m	11'3"	—		2.86 m	9'5"
Width (W/O Trunnion — Std. Shoe)	2.44 m	8'0"	3.14 m	10'3.6"	2.55 m	8'5"	2.54 m	8'4"
Ground Clearance	377 mm	14.8"	382 mm	15"	347 mm	13.7"	406 mm	16"
Blade Types and Widths:								
Straight	3.36 m	11'0"	3.99 m	13'1"	3.66 m	12'0"	3.91 m	12'10"
Angle	—		—		4.27 m	14'0"	—	
Angle Straight	4.16 m	13'8"	—		—		4.49 m	14'9"
Full Angle	3.78 m	12'5"	—		—		4.08 m	13'5"
Universal	—		—		3.81 m	12'6"	3.98 m	13'1"
Semi-U	3.26 m	10'8"	—		—		3.68 m	12'1"
Fuel Tank Refill Capacity	397 L	105 U.S. gal	337 L	89 U.S. gal	435 L	115 U.S. gal	479 L	127 U.S. gal

*Operating Weight includes ROPS canopy, operator, lubricants, coolant, full fuel tank, hydraulic controls and fluid, straight dozer with tilt, horn, back-up alarm, retrieval hitch and front pull hook.
— D7G includes end track guiding guards.
**Height (stripped top) — without ROPS canopy, exhaust, seat back or other easily removed encumbrances.



GROUND PRESSURES

Pressures computed from operating weights given earlier in this section in the specifications tables.

MODEL	SHOE WIDTH		CONTACT AREA		GROUND PRESSURE	
	mm	in	m^2	in^2	kPa	psi
D3C Series II ◄	356	14	1.35	2097	52	7.2
	406	16	1.54	2394	44	6.45
D3C LGP Series II	635	25	2.61	4045	29	4.2
D3C LGP-S Series II	990	39	4.85	7524	17	2.52
D4C Series II	406	16	1.67	2589	44	6.44
D4C LGP Series II	635	25	2.61	4045	30	4.3
D5C	457	18	1.92	3038	42.3	6.14
D5C LGP	660	26	2.83	4389	31.0	4.51
D4H Series II	360	14	1.59	2464	64	9.1
	410	16	1.82	2826	56	8.0
◄	460	18	2.05	3168	50	7.1
D4H XL Series III ◄	510	20	2.45	3799	47	6.73
	560	22	2.69	4172	44	6.17
D4H LGP Series III ◄	610	24	3.20	4953	38	5.34
	760	30	4.03	6252	31	4.39
	770	30	3.98	6170	31	4.41
D5E	406	16	1.77	2745	62	9.0
◄	457	18	1.99	3085	55	7.98
	508	20	2.21	3426	49.8	7.22
	560	22	2.44	3784	45	6.53
D5H Series II	460	18	2.11	3276	50.9	8.69
◄	510	20	2.35	3646	54.4	7.9
D5H XL Series II	560	22	2.78	4309	50	7.10
◄	600	24	3.63	4689	47	6.67
D5H LGP Series II	710	28	4.43	6866	36	5.15
◄	860	34	5.37	8321	30	4.24
	865	34	5.40	8369	29	4.17
D6D	457	18	2.17	3364	65	9.42
◄	508	20	2.41	3736	59	8.56
	560	22	2.65	4108	53	7.68
	610	24	2.89	4480	49	7.10
D6E	457	18	2.43	3766	60	8.70
◄	508	20	2.71	4200	54	7.83
	560	22	2.98	4619	49	7.10
	610	24	3.25	5040	45	6.54
D6H Series II	508	20	2.67	4140	65.0	9.44
◄	560	22	2.94	4564	59.0	8.56
	610	24	3.21	4971	54.2	7.86

◄Standard Shoe

MODEL	SHOE WIDTH		CONTACT AREA		GROUND PRESSURE	
	mm	in	m^2	in^2	kPa	psi
D6H XL Series II	510	20	2.89	4484	66	9.42
◄	560	22	3.18	4923	60	8.49
	610	24	3.46	5363	55	7.88
D6H XR Series II	508	20	2.83	4386	67	9.48
◄	560	22	3.11	4815	60	8.55
	610	24	3.38	5246	56	7.93
D6H LGP Series II ◄	760	30	4.96	7296	40	5.74
	915	36	5.97	9261	34	4.88
	1000	39	6.53	10,122	32	4.48
D7G ◄	508	20	2.76	4280	73	10.6
◄	559	22	3.04	4708	66	9.6
	610	24	3.31	5136	60	8.8
D7H Series II	510	20	2.94	4560	82	11.7
◄	560	22	3.24	5016	75	10.6
	610	24	3.53	5472	69	9.8
	660	26	3.82	5928	64	9.1
D7H XR Series II	560	22	3.43	5315	71.5	10.16
◄	610	24	3.75	5808	65.9	9.37
	660	26	4.06	6282	61.2	8.70
D7H LGP Series II	760	30	4.8	7504	54	7.74
	915	36	5.82	9029	46	6.55
D8N ◄	560	22	3.59	5565	100.6	14.6
	610	24	3.91	6062	92.3	13.4
	660	26	4.23	6559	85.4	12.4
	710	28	4.55	7056	79.2	11.5
D8N LGP*	965	38	6.2	9576	53.7	7.8
D9N	560	22	3.86	6009	107.0	15.51
◄	610	24	4.21	6555	99.1	14.37
	685	27	4.74	7374	89.2	12.94
	760	30	5.26	8194	79.4	11.51
D10N ◄	610	24	4.73	7326	124.1	18.0
	710	28	5.50	8527	107.8	15.6
	860	31.5	6.66	10,328	89.8	13.0
D11N ◄	710	28	6.31	9781	150.2	21.8
	810	32	7.20	11,159	132.7	19.3
	915	36	8.13	12,605	118.6	17.2

◄Standard shoe. *Offered as a Custom Product.

NOTE: Ground contact area = width of track shoe x length of track on ground x 2.

$$\text{Ground pressure} = \frac{\text{operating weight}}{\text{ground contact area}}$$

ATTACHMENT 2

TABLE 3.31 TYPICAL PROPERTIES OF COMPACTED SOILS

TABLE 3.31
TYPICAL PROPERTIES OF COMPACTED SOILS*

Group symbol	Soil type	Range of maximum dry unit weight, pcf	Range of optimum moisture, %	Typical value of compression: Percent of original height, At 1.4 tsf (20 psi)	Typical value of compression: Percent of original height, At 3.6 tsf (50 psi)	Typical strength characteristics: Cohesion (as compacted), psf	Cohesion (saturated), psf	Effective stress envelope ϕ, degrees	tan ϕ	Typical coefficient of permeability, ft/min	Range of CBR values	Range of subgrade modulus k_s, lb/in^3
GW	Well-graded clean gravels, gravel-sand mixtures	125–135	11–8	0.3	0.6	0	0	>38	>0.79	5×10^{-2}	40–80	300–500
GP	Poorly graded clean gravels, gravel-sand mix	115–125	14–11	0.4	0.9	0	0	>37	>0.74	10^{-1}	30–60	250–400
GM	Silty gravels, poorly graded gravel-sand silt	120–135	12–8	0.5	1.1	...	...	>34	>0.67	$>10^{-6}$	20–60	100–400
GC	Clayey gravels, poorly graded gravel-sand-clay	115–130	14–9	0.7	1.6	...	...	>31	>0.60	$>10^{-7}$	20–40	100–300
SW	Well-graded clean sands, gravelly sands	110–130	16–9	0.6	1.2	0	0	38	0.79	$>10^{-3}$	20–40	200–300
SP	Poorly-graded clean sands, sand-gravel mix	100–120	21–12	0.8	1.4	0	0	37	0.74	$>10^{-3}$	10–40	200–300
SM	Silty sands, poorly graded sand-silt mix	110–125	16–11	0.8	1.6	1050	420	34	0.67	5×10^{-5}	10–40	100–300
SM-SC	Sand-silt clay mix with slightly plastic fines	110–130	15–11	0.8	1.4	1050	300	33	0.66	2×10^{-6}	...	
SC	Clayey sands, poorly graded sand-clay mix	105–125	19–11	1.1	2.2	1550	230	31	0.60	5×10^{-7}	5–20	100–300
ML	Inorganic silts and clayey silts	95–120	24–12	0.9	1.7	1400	190	32	0.62	10^{-5}	15 or less	100–200
ML-CL	Mixture of inorganic silt and clay	100–120	22–12	1.0	2.2	1350	460	32	0.62	5×10^{-7}	...	
CL	Inorganic clays of low to medium plasticity	95–120	24–12	1.3	2.5	1800	270	28	0.54	10^{-7}	15 or less	50–200
OL	Organic silts and silt-clays, low plasticity	80–100	33–21	...	...	...	...	...	...	...	5 or less	50–100
MH	Inorganic clayey silts, elastic silts	70–95	40–24	2.0	3.8	1500	420	25	0.47	5×10^{-7}	10 or less	50–100
CH	Inorganic clays of high plasticity	75–105	36–19	2.6	3.9	2150	230	19	0.35	10^{-7}	15 or less	50–150
OH	Organic clays and silty clays	65–100	45–21	...	...	...	...	...	...	...	5 or less	25–100

*From NAVFAC Manual DM 7 (1971).[5] All properties are for condition of "standard Proctor" maximum density, except values of k and CBR which are for "modified Proctor" maximum density. Typical strength characteristics are for effective strength envelopes and are obtained from USBR data. Compression values are for vertical loading with complete lateral confinement. [...] Indicates insufficient data available for an estimate.

Appendix B.3B
Drainage Layer Stability Analysis With Equipment
(Cell 2 Base Liner Overlay on Phase I)

Client	County of Kaua'i	**Subject**	Drainage Layer	**Prepared By** SRA	**Date** 3/27/13
Project	Kekaha Landfill		Stability Analysis with	**Reviewed By** JMT	**Date** 3/27/13
Phase II Lateral Expansion			Equipment Loading	**Approved By** KJB	**Date** 3/27/13

DRAINAGE LAYER STABILITY ANALYSIS WITH EQUIPMENT LOADING

Objective

Determine the maximum slope length for granular drainage layer placement and operations layer placement over the geosynthetic base liner on the 3.5:1 sideslopes of Cell 2 which overlays Phase I.

Design Criteria and Assumptions

1. The base liner systems for Cell 2 is depicted in Figure 1 from Reference 1.
2. The drainage layer material is expected to be a sub-angular or angular gravel.
3. The placement of the granular drainage layer and the operations layer will be completed in two stages. The first stage would be placement of the 1-foot granular drainage layer and installation of the separator geotextile. After installation of the geotextile the 2-foot operations layer will be placed. This analysis will evaluate both conditions including placement of the operations layer in two 1-foot lifts as well. It is also assumed that the granular drainage layer or operations layer will be inplace across the cell floor prior to placement on the sideslope.
4. This analysis will determine the minimum critical interface friction angles required for varied slope lengths from a typical length of 10 feet to the maximum slope length for the proposed expansion. Slope length will be dependant upon equipment loadings, placement of the granular drainage layer and/or the operations layer on the slope, and assumed no tension applied to the geotextile or geomembrane.
5. The geomembrane is assumed to be smooth between the interface with the geotextile cushion and textured between the interface with the waste so that the geomembrane is designed to slide. Derived friction angles are assumed to represent the interface of the textured surface and the geotextile cushion.
6. Slope length for granular drainage layer and operations layer placement is defined as the actual slope length, not a vertical or horizontal length.
7. Sideslopes are a maximum 3.5:1 (H:V) slope.
8. The dozer to be used for deployment of the drainage layer is expected to be a Caterpillar D6H LGP. This dozer has a ground pressure of less than 5 psi, typical for placement of soil material layers over geosynthetics. Model specifications are included as Attachment 1.
9. The dozer is operated from the bottom of the slope with no abrupt braking or change in direction during granular drainage layer placement.
10. Interface friction angle testing will be completed prior to cell construction and before a resulting change of geosynthetic materials, if applicable. Material properties and interface friction angle testing requirements is presented in Appendix F.

Calculations

Caterpillar D6H LGP Dozer Properties (Attachment 1, Reference 5)

Dozer track width:	36 inches
Track length on ground:	128.5 inches (10.7 ft)
Ground pressure:	4.88 psi (702.7 psf) < 5 psi

Drainage Layer Properties

Unit Weight: 130 pcf
Internal friction angle: *34+ degrees

*Table 3.31 of Reference 2 for SM, SP, SW, GM, GP, GW soils that could be used as drainage layer materials.

For conservatism, the internal friction angle used in the model and analysis is 33 degrees which is equal to the typical interface friction angle between sand/nonwoven geotextiles and gravel/nonwoven geotextile (Table 1, Reference 4). This will allow the model to evaluate a uniform layer thickness (granular drainage layer or with the added operations layer) with the separator geotextile included to determine the critical interface friction angle needed on the sideslope base liner system.

Maximum Slope Angle

3.5H:1V: $\tan^{-1}(1/3.5) = 15.9$ degrees

Typical Interface Friction Angles

A review of Reference 4 for typical peak interface friction angles and peak adhesion in relation to the proposed base liner system include:

Table 1
Critical Base Liner Interfaces and Friction Angles

Interface	Typical Interface Friction Test Results	
	Friction Angle (degrees)	Adhesion (kPa)
Nonwoven geotextile (separator and cushion) / granular drainage layer (gravel), operations layer (granular), or prepared subbase (granular)	33	0
Nonwoven geotextile (cushion) / textured HDPE geomembrane	25	8
Nonwoven geotextile (GCL) / textured HDPE geomembrane	23	8
Needle-punched GCL (internal friction angle)	39.7	19.9
Nonwoven geotextile / smooth HDPE geomebrane	11	0

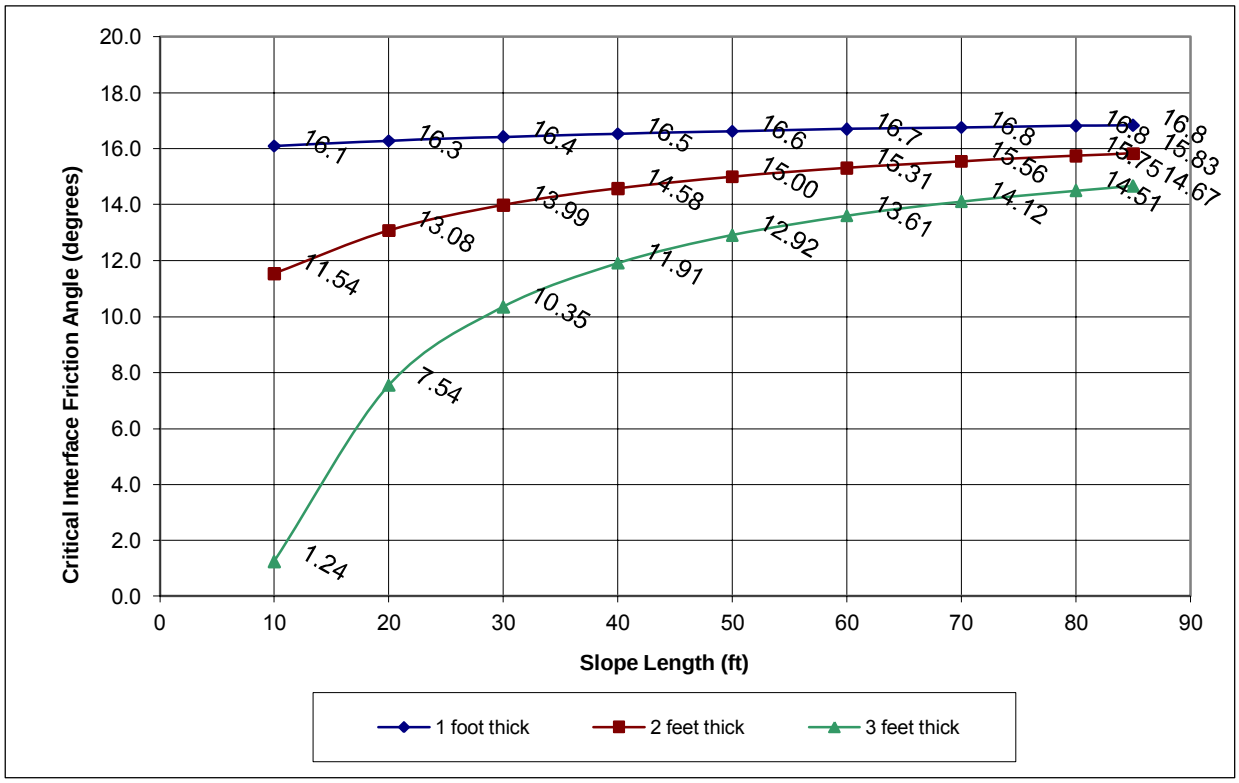
Method of Analysis

Attached as Figures 2, 3 and 4, is a calibrated spreadsheet depicting the analysis presented by Koerner and Soong, 1998 (Reference 3). This spreadsheet depicts the driving and resisting forces experienced along the critical interface with the incorporation of equipment loading. A factor of safety of 1.1 was deemed as a qualifier for slope stability as equipment loading is a

brief and temporary load condition.

The minimum interface friction angle for the interface between the smooth geomembrane and the geotextile is 11 degrees but for conservatism 10 degrees will be used. The analysis determined that a factor of safety equal to 1.1 with a minimum friction angle of 10 degrees is not achievable for a drainage thickness of 1 foot with equipment loading. In addition, the maximum length material that can be placed with the 2-foot thickness including equipment loading is 4.1 feet. Further analysis determined that the maximum length material can be placed with the 1-foot thickness without equipment loading is 15 feet, see Figure 2 included in the attachments. Therefore, we recommend the 1-foot granular drainage layer be placed up the slope 15 feet by an excavator, then place the separation geotextile, then push the remaining 2 feet of granular drainage material over the geotextile by a dozer. Although the resulting slope lengths at which granular material can be placed 3-feet thick with no equipment loading is 28.4 feet, the maximum slope length is controlled by the placed 1-foot thick drainage layer.

Table 2
Slope Length vs. Critical Interface Friction Angles vs. Soil Thickness With Equipment Loading Applies



By varying the slope lengths in Figures 2, 3, and 4, applying equipment loads, varying drainage layer and operations layer thickness on the sideslope, and calculating the minimum critical interface friction angle for the specific slope length, the resulting Table 2 above was derived. The table itself specifically defines the critical interface solely as friction without the application

of apparent adhesion that could occur between the interfaces of geosynthetic layers to geosynthetics layers or a geosynthetic layer to a soil layer. Added conservatism is provided by not considering adhesion.

Conclusions

Upon review of Table 2, the critical placement period will be during the placement of the 1-foot granular drainage layer on the 3.5:1 slope. As the lift thickness increases (the 1-foot drainage layer is then overlain by the operations layer) the critical interface requirement decreases as a result of the increased buttress effect at the toe of slope and distribution of the equipment loading through the granular soils.

However, given the typical peak values in Table 1 it is determined that the smooth sideslope geomembrane is only feasible for 1 foot of placement of the granular drainage layer on the sideslopes (without added tension to the underlying geotextile or geomembrane) if the granular drainage layer is placed. Without additional equipment loading, as shown in Figure 2, the 1-foot layer of drainage layer will need to be placed by an excavator reaching up the sideslope from the toe slope. The maximum slope length for placement is 15 feet. Additional 15-foot sections can be placed as waste mass lifts are completed.

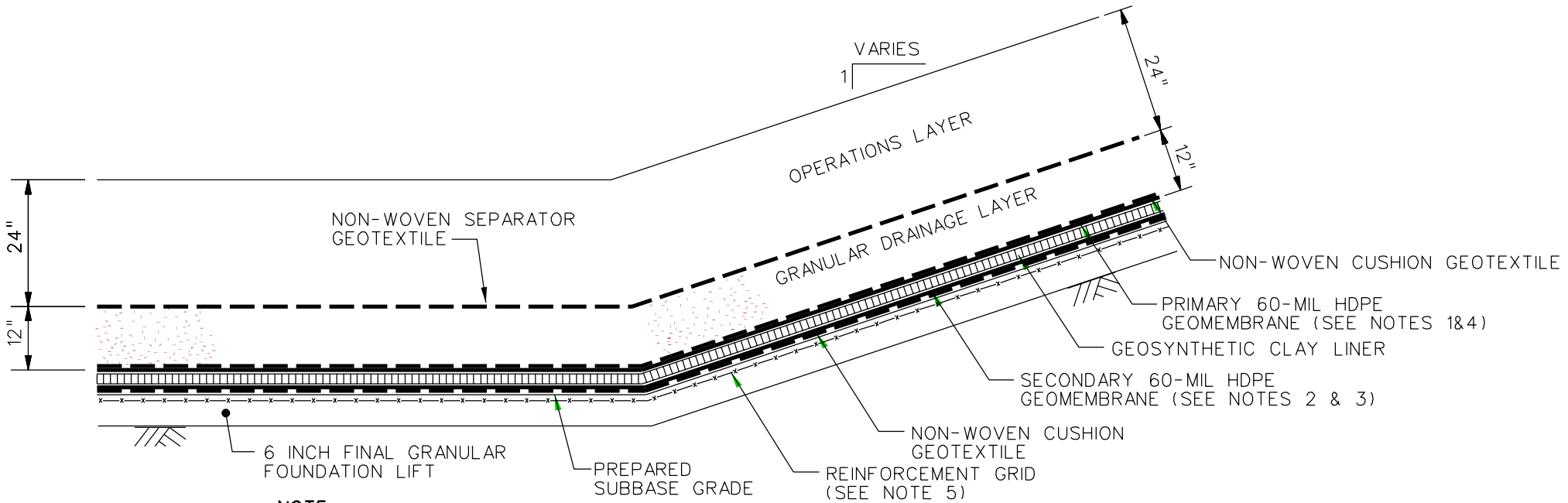
This calculation should be revisited if placement equipment is different or if there is a change in the granular drainage layer. This calculation should also be revisited for the addition of any further base liner components or removal of any of the proposed base liner components.

References

1. Drawings entitled, "Engineering Report, Phase II Cell 2 Lateral Expansion, Kekaha Landfill", prepared by AECOM, dated August 2012.
2. Table 3.31, Hunt, Roy E., "Geotechnical Engineering Investigation Manual," 1984.
3. Koerner, R. M. and Soong, T.-Y. (1998), "Analysis and Design of Veneer Cover Soils" *Proc. 6th Int. Conf. on Geosynthetics, Atlanta*, USA, IFAI, pp. 1-23.
4. Koerner, George R. and Nejaro, Dhani, "Direct Shear Database of Geosynthetic-to-Geosynthetic and Geosynthetic-to-Soil Interfaces," Geosynthetic Research Institute, GRI Report #30, June 14, 2005.
5. "Caterpillar Performance Handbook," Edition 24, pages 1-8 and 1-21.

FIGURE 1

TYPICAL CELL 2 BASE LINER SYSTEM DETAILS



NOTE:

1. PRIMARY 60-MIL HDPE GEOMEMBRANE OVER PHASE I SIDESLOPE IS SINGLE-SIDED TEXTURED. SMOOTH SIDE OF GEOMEMBRANE IS UP AND IN DIRECT CONTACT WITH NON-WOVEN CUSHION GEOTEXTILE FOR THE GRANULAR DRAINAGE LAYER. EXTEND PRIMARY GEOMEMBRANE 5 FEET PAST TOE OF SLOPE.
2. SECONDARY 60-MIL HDPE GEOMEMBRANE OVER PHASE 1 SIDESLOPE IS SINGLE-SIDED TEXTURED. SMOOTH SIDE OF GEOMEMBRANE IS DOWN AND IN CONTACT WITH NON-WOVEN CUSHION GEOTEXTILE. EXTEND GEOMEMBRANE 5 FEET PAST TOE OF SLOPE.
3. SECONDARY 60-MIL HDPE GEOMEMBRANE ON PHASE II SIDESLOPE AND CELL 2 BASE IS TEXTURED ON BOTH SIDES.
4. PRIMARY 60-MIL HDPE GEOMEMBRANE ON PHASE II SIDESLOPE AND CELL 2 BASE IS TEXTURED ON BOTH SIDES.
5. REINFORCEMENT GRID TO BE INSTALLED ON PHASE I SIDESLOPES IN SUBCELLS A AND D.

BASE LINER DETAIL 1

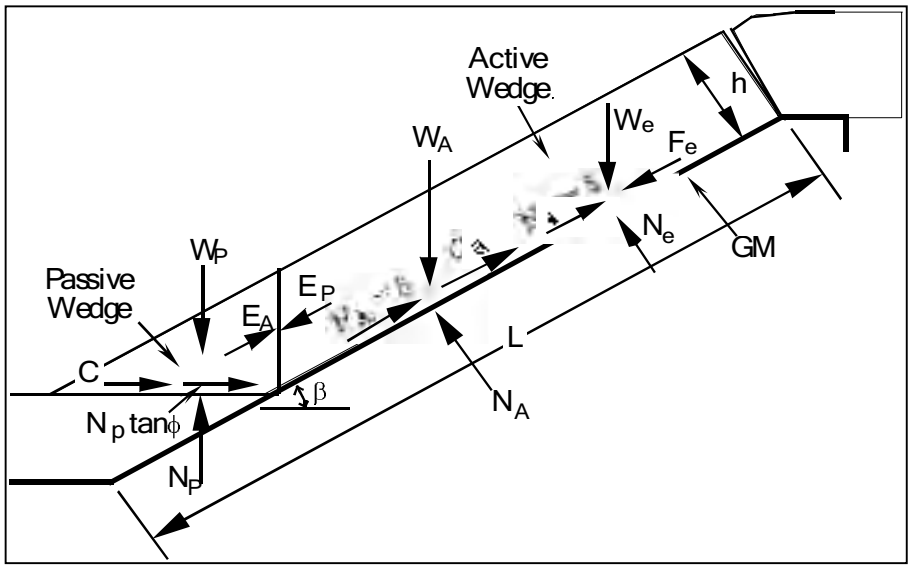
NTS

FIGURE 2

DRAINAGE LAYER STABILITY CALCULATION WITHOUT EQUIPMENT LOADING (MAXIMUM SLOPE LENGTH ONLY, 1-FOOT GRANULAR DRAINAGE LAYER)

Figure 2

Uniform Cover Soil Thickness with the Incorporation of Equipment Loads
1-foot Thickness (1-foot Granular Drainage Layer Placed Only)



Calculation of FS

Active Wedge:

Wa=	*1507.0*	*kpounds*
Na=	*1449.0*	*kpounds*

Passive Wedge:

Wp=	*246.1*	*kpounds*

$$FS = \frac{-b + \sqrt{b^2 - 4ac}}{2a}$$

a=	*398.1*
b=	*-479*
c=	*45.6*

FS= 1.10

thickness of cover soil = h =	1.00	ft		
soil slope angle beneath the geomembrane = ⌴ =	15.9	°	*= 0.28 (rad.)*	
finished cover soil slope angle = ⌴ =	15.9	°	*= 0.28 (rad.)*	
length of slope measured along the geomembrane = L =	15.4	ft		
unit weight of the cover soil = ⌴=	130.0	pcf		
friction angle of the cover soil = ⌴=	33.0	°	*= 0.58 (rad.)*	
cohesion of the cover soil = c =	0.0	psf	*C= 0*	*kpounds*
interface friction angle between cover soil and geomembrane = ⌴=	10.0	°	*= 0.17 (rad.)*	
adhesion between cover soil and geomembrane = ca =	0.0	psf	*Ca= 0*	*kpounds*

thickness of cover soil = h =	1.00	ft	*b/h=*	*0.0*
equipment ground pressure (= wt. of equipment/(2wb)) = q =	0.0	psf	*We=q w l=*	*0.0*
length of each equipment track = w =	0.0	ft	*Ne=Wecos b =*	*0.0*
width of each equipment track = b =	0.0	ft	*Fe=We (a/g) =*	*0.0*
influence factor* at geomembrane interface = I =	0.00			
acceleration/deceleration of the bulldozer = a =	0.00	g		

*Influence Factor Default Values

Cover Soil Thickness	Equipment Track Width		
	Very Wide	Wide	Standard
[2] 300 mm	1.00	0.97	0.94
300-1000 mm	0.97	0.92	0.70
[3] 1000 mm	0.95	0.75	0.30

Note: numbers in boxes are input values

numbers in Italics are calculated values

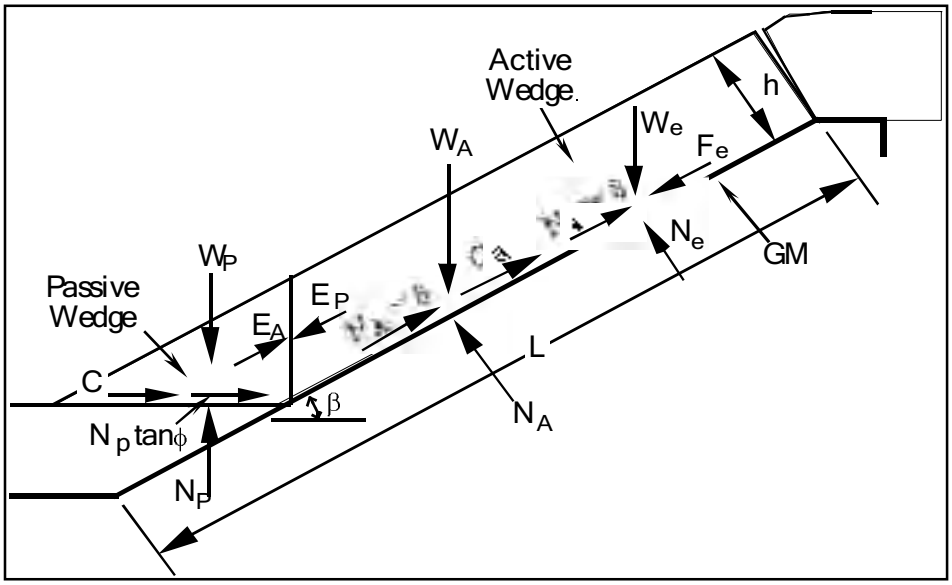
*Note: No equipment load applied to achieve FS=1.1

FIGURE 3

DRAINAGE LAYER STABILITY CALCULATION WITHOUT EQUIPMENT LOADING (MAXIMUM SLOPE LENGTH ONLY, 1-FOOT GRANULAR DRAINAGE LAYER WITH 1 FOOT OF OPERATIONS LAYER)

Figure 3

Uniform Cover Soil Thickness with the Incorporation of Equipment Loads
2-foot Thickness (1-foot Granular Drainage Layer and 1-foot Operations Layer)



Calculation of FS

Active Wedge:

Wa= 6026.9 kpounds

Na= 5795.0 kpounds

Passive Wedge:

Wp= 984.3 kpounds

$$FS = \frac{-b + \sqrt{b^2 - 4ac}}{2a}$$

a= 1592.0

b= -1917

c= 182.3

FS= 1.10

thickness of cover soil = h =	2.00 ft		
soil slope angle beneath the geomembrane = □ =	15.9 °	*= 0.28 (rad.)*	
finished cover soil slope angle = □ =	15.9 °	*= 0.28 (rad.)*	
length of slope measured along the geomembrane = L =	30.7 ft		
unit weight of the cover soil = □=	130.0 pcf		
friction angle of the cover soil = □ =	33.0 °	*= 0.58 (rad.)*	
cohesion of the cover soil = c =	0.0 psf	*C= 0*	*kpounds*
interface friction angle between cover soil and geomembrane = □ =	10.0 °	*= 0.17 (rad.)*	
adhesion between cover soil and geomembrane = ca =	0.0 psf	*Ca= 0*	*kpounds*

thickness of cover soil = h =	2.00 ft	*b/h=*	*0.0*
equipment ground pressure (= wt. of equipment/(2wb)) = q =	0.0 psf	*We=q w l=*	*0.0*
length of each equipment track = w =	0.0 ft	*Ne=Wecos b =*	*0.0*
width of each equipment track = b =	0.0 ft	*Fe=We (a/g) =*	*0.0*
influence factor* at geomembrane interface = I =	0.00		
acceleration/deceleration of the bulldozer = a =	0.00 g		

*Influence Factor Default Values

Cover Soil Thickness	Equipment Track Width		
	Very Wide	Wide	Standard
[2] 300 mm	1.00	0.97	0.94
300-1000 mm	0.97	0.92	0.70
[3] 1000 mm	0.95	0.75	0.30

Note: numbers in boxes are input values

numbers in Italics are calculated values

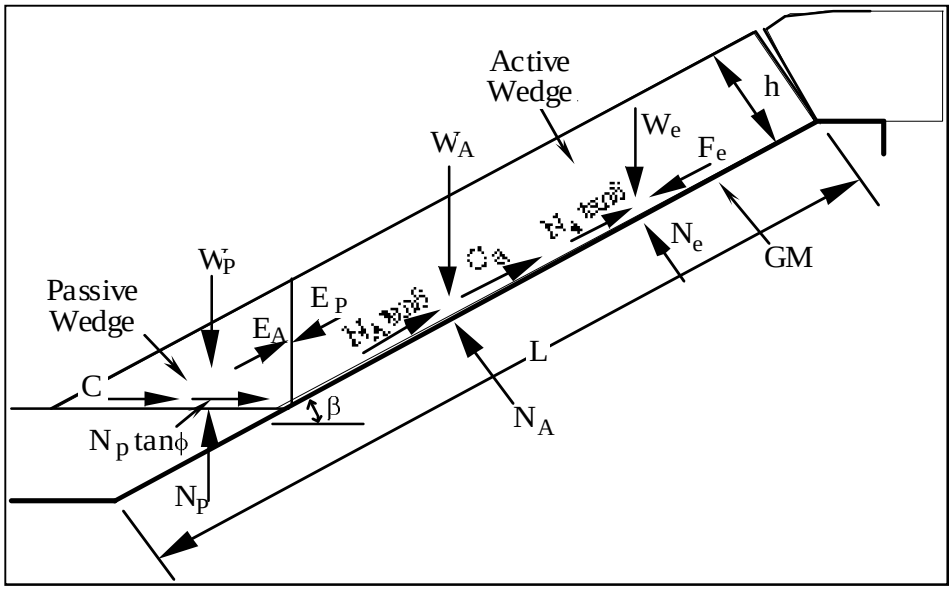
*Note: No equipment load applied to achieve FS=1.1

FIGURE 4

DRAINAGE LAYER STABILITY CALCULATION WITH EQUIPMENT LOADING (MAXIMUM SLOPE LENGTH ONLY, 1-FOOT GRANULAR DRAINAGE LAYER WITH 2 FEET OF OPERATIONS LAYER)

Figure 4

Uniform Cover Soil Thickness with the Incorporation of Equipment Loads
3-foot Thickness (1-foot Granular Drainage Layer and 2-foot Operations Layer)



Calculation of FS

Active Wedge:

Wa=	*6635.9*	*kpounds*
Na=	*6380.6*	*kpounds*

Passive Wedge:

Wp=	*2214.6*	*kpounds*

$$FS = \frac{-b + \sqrt{b^2 - 4ac}}{2a}$$

a=	*3580.1*
b=	*-4312*
c=	*410.0*

FS= 1.10

thickness of cover soil = h =	3.00	ft		
soil slope angle beneath the geomembrane = β =	15.9	°	*= 0.28 (rad.)*	
finished cover soil slope angle = ω =	15.9	°	*= 0.28 (rad.)*	
length of slope measured along the geomembrane = L =	28.4	ft		
unit weight of the cover soil = γ =	130.0	pcf		
friction angle of the cover soil = ϕ =	33.0	°	*= 0.58 (rad.)*	
cohesion of the cover soil = c =	0.0	psf	*C= 0*	*kpounds*
interface friction angle between cover soil and geomembrane = δ =	10.0	°	*= 0.17 (rad.)*	
adhesion between cover soil and geomembrane = ca =	0.0	psf	*Ca= 0*	*kpounds*

thickness of cover soil = h =	3.00	ft	*b/h=*	*1.0*
equipment ground pressure (= wt. of equipment/(2wb)) = q =	702.7	psf	*We=q w l=*	*6917.4*
length of each equipment track = w =	10.7	ft	*Ne=Wecos β =*	*6651.2*
width of each equipment track = b =	3.0	ft	*Fe=We (a/g) =*	*0.0*
influence factor* at geomembrane interface = I =	0.92			
acceleration/deceleration of the bulldozer = a =	0.00	g		

*Influence Factor Default Values

Cover Soil Thickness	Equipment Track Width		
	Very Wide	Wide	Standard
[2] 300 mm	1.00	0.97	0.94
300-1000 mm	0.97	0.92	0.70
[3] 1000 mm	0.95	0.75	0.30

Note: numbers in boxes are input values

numbers in Italics are calculated values

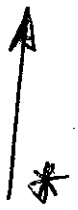
ATTACHMENT 1

EQUIPMENT PROPERTIES FOR CATERPILLAR D6H LGP DOZER



MODEL	D6H XR Series II		D6H LGP Series II		D7G		D7H Series II	
Flywheel Power	130 kW	175 hp	127 kW	170 hp	149 kW	200 hp	160 kW	215 hp
Operating Weight*								
(Power Shift)	18 684 kg	41,192 lb	19 814 kg	43,590 lb	20 666 kg	45,560 lb	24 694 kg	54,401 lb
(Direct Drive)	—		19 989 kg	43,976 lb	20 510 kg	45,218 lb	—	
(Power Shift Differential Steer)	18 799 kg	41,444 lb	—		—		24 993 kg	55,100 lb
Engine Model	3306		3306		3306		3306	
Rated Engine RPM	1900		1900		2000		2100	
No. of Cylinders	6		6		6		6	
Bore	121 mm	4.75"	121 mm	4.75"	121 mm	4.75"	121 mm	4.75"
Stroke	152 mm	6"	152 mm	6"	152 mm	6"	152 mm	6"
Displacement	10.5 L	638 in³	10.5 L	638 in³	10.5 L	638 in³	10.5 L	638 in³
Track Rollers (Each Side)	7		8		6		7	
Width of Standard Track Shoe	560 mm	22"	915 mm	36"	508 mm	20"	560 mm	22"
Length of Track on Ground	2.77 m	9'1"	3.27 m	10'8.5"	2.70 m	8'11"	2.90 m	9'6"
Ground Contact Area (W/Std. Shoe)	3.11 m²	4815 in²	5.97 m²	9254 in²	2.76 m²	4280 in²	3.24 m²	5016 in²
Track Gauge	1.88 m	6'2"	2.23 m	7'3"	1.98 m	6'5"	1.98 m	6'6"
GENERAL DIMENSIONS:								
Height (Stripped Top)**	2.26 m	7'5"	2.32 m	7'7"	2.27 m	7'5"	2.44 m	8'0"
Height (To Top of ROPS)	3.12 m	10'3"	3.16 m	10'5"	3.20 m	10'6"	3.33 m	10'11"
Height (To Top of Cab ROPS)	3.12 m	10'3"	3.16 m	10'5"	—		3.42	11'3"
Height (To Top of ROPS Canopy)	—		3.16 m	10'5"	—		—	
Overall Length (With P Blade)	—		5.18 m	17'0"	—		—	
(Without Blade)	—		4.49 m	14'9"	—		—	
Overall Length (With S Blade)	5.26 m	17'3"	—		5.28 m	17'4"	6.03 m	19'9"
(Without Blade)	4.21 m	13'10"	—		4.19 m	13'9"	4.73 m	15'6"
Width (Over Trunnion)	2.64 m	8'8"	3.43 m	11'3"	—		2.86 m	9'5"
Width (W/O Trunnion — Std. Shoe)	2.44 m	8'0"	3.14 m	10'3.6"	2.55 m	8'5"	2.54 m	8'4"
Ground Clearance	377 mm	14.8"	382 mm	15"	347 mm	13.7"	406 mm	16"
Blade Types and Widths:								
Straight	3.36 m	11'0"	3.99 m	13'1"	3.66 m	12'0"	3.91 m	12'10"
Angle	—		—		4.27 m	14'0"	—	
Angle Straight	4.16 m	13'8"	—		—		4.49 m	14'9"
Full Angle	3.78 m	12'5"	—		—		4.08 m	13'5"
Universal	—		—		3.81 m	12'6"	3.98 m	13'1"
Semi-U	3.26 m	10'8"	—		—		3.68 m	12'1"
Fuel Tank Refill Capacity	397 L	105 U.S. gal	337 L	89 U.S. gal	435 L	115 U.S. gal	479 L	127 U.S. gal

*Operating Weight includes ROPS canopy, operator, lubricants, coolant, full fuel tank, hydraulic controls and fluid, straight dozer with tilt, horn, back-up alarm, retrieval hitch and front pull hook.
— D7G includes end track guiding guards.
**Height (stripped top) — without ROPS canopy, exhaust, seat back or other easily removed encumbrances.



GROUND PRESSURES

Pressures computed from operating weights given earlier in this section in the specifications tables.

MODEL	SHOE WIDTH		CONTACT AREA		GROUND PRESSURE	
	mm	in	m^2	in^2	kPa	psi
D3C Series II ◄	356	14	1.35	2097	52	7.2
	406	16	1.54	2394	44	6.45
D3C LGP Series II	635	25	2.61	4045	29	4.2
D3C LGP-S Series II	990	39	4.85	7524	17	2.52
D4C Series II	406	16	1.67	2589	44	6.44
D4C LGP Series II	635	25	2.61	4045	30	4.3
D5C	457	18	1.92	3038	42.3	6.14
D5C LGP	660	26	2.834	4389	31.0	4.51
D4H Series II	360	14	1.59	2464	64	9.1
	410	16	1.82	2826	56	8.0
◄	460	18	2.05	3168	50	7.1
D4H XL Series III ◄	510	20	2.45	3799	47	6.73
	560	22	2.69	4172	44	6.17
D4H LGP Series III ◄	610	24	3.20	4953	38	5.34
	760	30	4.03	6252	31	4.39
	770	30	3.98	6170	31	4.41
D5E	406	16	1.77	2745	62	9.0
◄	457	18	1.99	3085	55	7.98
	508	20	2.21	3426	49.8	7.22
	560	22	2.44	3784	45	6.53
D5H Series II	460	18	2.11	3276	50.9	8.69
◄	510	20	2.35	3646	54.4	7.9
D5H XL Series II	560	22	2.78	4309	50	7.10
◄	600	24	3.63	4689	47	6.67
D5H LGP Series II	710	28	4.43	6866	36	5.15
◄	860	34	5.37	8321	30	4.24
	865	34	5.40	8369	29	4.17
D6D	457	18	2.17	3364	65	9.42
◄	508	20	2.41	3736	59	8.56
	560	22	2.65	4108	53	7.68
	610	24	2.89	4480	49	7.10
D6E	457	18	2.43	3766	60	8.70
◄	508	20	2.71	4200	54	7.83
	560	22	2.98	4619	49	7.10
	610	24	3.25	5040	45	6.54
D6H Series II	508	20	2.67	4140	65.0	9.44
◄	560	22	2.94	4564	59.0	8.56
	610	24	3.21	4971	54.2	7.86
D6H XL Series II	510	20	2.89	4484	66	9.42
◄	560	22	3.18	4923	60	8.49
	610	24	3.46	5363	55	7.88
D6H XR Series II	508	20	2.83	4386	67	9.48
◄	560	22	3.11	4815	60	8.55
	610	24	3.38	5246	56	7.93
D6H LGP Series II ◄	760	30	4.96	7296	40	5.74
	915	36	5.97	9261	34	4.88
	1000	39	6.53	10,122	32	4.48
D7G ◄	508	20	2.76	4280	73	10.6
◄	559	22	3.04	4708	66	9.6
	610	24	3.31	5136	60	8.8
D7H Series II	510	20	2.94	4560	82	11.7
◄	560	22	3.24	5016	75	10.6
	610	24	3.53	5472	69	9.8
	660	26	3.82	5928	64	9.1
D7H XR Series II	560	22	3.43	5315	71.5	10.16
◄	610	24	3.75	5808	65.9	9.37
	660	26	4.06	6282	61.2	8.70
D7H LGP Series II	760	30	4.8	7504	54	7.74
	915	36	5.82	9029	46	6.55
D8N ◄	560	22	3.59	5565	100.6	14.6
	610	24	3.91	6062	92.3	13.4
	660	26	4.23	6559	85.4	12.4
	710	28	4.55	7056	79.2	11.5
D8N LGP*	965	38	6.2	9576	53.7	7.8
D9N	560	22	3.86	6009	107.0	15.51
◄	610	24	4.21	6555	99.1	14.37
	685	27	4.74	7374	89.2	12.94
	760	30	5.26	8194	79.4	11.51
D10N ◄	610	24	4.73	7326	124.1	18.0
	710	28	5.50	8527	107.8	15.6
	860	31.5	6.66	10,328	89.8	13.0
D11N ◄	710	28	6.31	9781	150.2	21.8
	810	32	7.20	11,159	132.7	19.3
	915	36	8.13	12,605	118.6	17.2

◄Standard shoe.

*Offered as a Custom Product.

NOTE: Ground contact area = width of track shoe x length of track on ground x 2.

$$\text{Ground pressure} = \frac{\text{operating weight}}{\text{ground contact area}}$$

ATTACHMENT 2

TABLE 3.31 TYPICAL PROPERTIES OF COMPACTED SOILS

TABLE 3.31
TYPICAL PROPERTIES OF COMPACTED SOILS*

Group symbol	Soil type	Range of maximum dry unit weight, pcf	Range of optimum moisture, %	Typical value of compression: Percent of original height, At 1.4 tsf (20 psi)	Typical value of compression: Percent of original height, At 3.6 tsf (50 psi)	Typical strength characteristics: Cohesion (as compacted), psf	Typical strength characteristics: Cohesion (saturated), psf	Typical strength characteristics: Effective stress envelope ϕ, degrees	Typical strength characteristics: tan ϕ	Typical coefficient of permeability, ft/min	Range of CBR values	Range of subgrade modulus k_s, lb/in^3
GW	Well-graded clean gravels, gravel-sand mixtures	125–135	11–8	0.3	0.6	0	0	>38	>0.79	5×10^{-2}	40–80	300–500
GP	Poorly graded clean gravels, gravel-sand mix	115–125	14–11	0.4	0.9	0	0	>37	>0.74	10^{-1}	30–60	250–400
GM	Silty gravels, poorly graded gravel-sand silt	120–135	12–8	0.5	1.1	...	...	>34	>0.67	$>10^{-6}$	20–60	100–400
GC	Clayey gravels, poorly graded gravel-sand-clay	115–130	14–9	0.7	1.6	...	...	>31	>0.60	$>10^{-7}$	20–40	100–300
SW	Well-graded clean sands, gravelly sands	110–130	16–9	0.6	1.2	0	0	38	0.79	$>10^{-3}$	20–40	200–300
SP	Poorly-graded clean sands, sand-gravel mix	100–120	21–12	0.8	1.4	0	0	37	0.74	$>10^{-3}$	10–40	200–300
SM	Silty sands, poorly graded sand-silt mix	110–125	16–11	0.8	1.6	1050	420	34	0.67	5×10^{-5}	10–40	100–300
SM-SC	Sand-silt clay mix with slightly plastic fines	110–130	15–11	0.8	1.4	1050	300	33	0.66	2×10^{-6}	...	
SC	Clayey sands, poorly graded sand-clay mix	105–125	19–11	1.1	2.2	1550	230	31	0.60	5×10^{-7}	5–20	100–300
ML	Inorganic silts and clayey silts	95–120	24–12	0.9	1.7	1400	190	32	0.62	10^{-5}	15 or less	100–200
ML-CL	Mixture of inorganic silt and clay	100–120	22–12	1.0	2.2	1350	460	32	0.62	5×10^{-7}	...	
CL	Inorganic clays of low to medium plasticity	95–120	24–12	1.3	2.5	1800	270	28	0.54	10^{-7}	15 or less	50–200
OL	Organic silts and silt-clays, low plasticity	80–100	33–21	...	...	...	...	...	...	...	5 or less	50–100
MH	Inorganic clayey silts, elastic silts	70–95	40–24	2.0	3.8	1500	420	25	0.47	5×10^{-7}	10 or less	50–100
CH	Inorganic clays of high plasticity	75–105	36–19	2.6	3.9	2150	230	19	0.35	10^{-7}	15 or less	50–150
OH	Organic clays and silty clays	65–100	45–21	...	...	...	...	...	...	...	5 or less	25–100

*From NAVFAC Manual DM 7 (1971).[6] All properties are for condition of "standard Proctor" maximum density, except values of k and CBR which are for "modified Proctor" maximum density. Typical strength characteristics are for effective strength envelopes and are obtained from USBR data. Compression values are for vertical loading with complete lateral confinement. (...) Indicates insufficient data available for an estimate.

Appendix B.4
Geotextile Analysis

Appendix B.4A
Cushion Geotextile Analysis

AECOM

CALCULATION SHEET

Project No. 60197394

Client	County of Kaua'i	**Subject**	Geotextile Cushion	**Prepared By**	KJB	**Date**	8/13/12
Project	Kekaha Landfill	Analysis		**Reviewed By**	TCR	**Date**	8/14/12
Phase II Lateral Expansion				**Approved By**	KJB	**Date**	8/14/12

GEOTEXTILE CUSHION ANALYSIS

Objective

Determine the geotextile required to resist damage due to impact and puncture during the placement of the granular drainage layer over the geotextile/geomembrane as well as from overburden pressures from in place waste mass and final cover system of Cell 2 of the proposed lateral expansion of Phase II. This analysis also applies to the cushion geotextile immediately underlying the subbase geomembrane.

Design Criteria and Assumptions

1. The base liner system details for Cell 2 are depicted in Figure 1 from Reference 1.
2. The proposed final cover system for Cell 2 is depicted in Figure 2 from Reference 1.
3. The drainage layer material is assumed to be gravel with particles that are angular in shape with an average particle size (d_{50} of the sieve analysis) ranging from 0.6 inch or 15 mm to 1 inch or 25 mm. The geotextile cushion analysis was conducted using the boundary values.
4. The base liner geomembrane is a 60-mil HDPE geomembrane.
5. The geotextile proposed as a cushion layers will be a nonwoven needle punched product and a minimum mass per unit area of 16 oz/yd^2.
6. The maximum surface pressure allowed on top of the 1-foot granular drainage layer by a dozer is 5 psi. This surface pressure represents the placement of the granular drainage layer with low ground pressure equipment such as a Caterpillar D6H LGP dozer. If larger or different equipment is utilized, this analysis should be reviewed.
7. All vehicles that are not low ground pressure, such as dump trucks, will be required to traverse over the base liner on a minimum a 3-foot thick gravel access road.
8. The maximum proposed final cover elevation for Cell 1 is elevation 85. For conservative analysis and potential vertical expansions, a final cover elevation of 120 is assumed.
9. The lowest elevation of the base liner is the sump area. This elevation is 10.50.
10. All references to elevations correlate to the site specific vertical datum.

Calculations

Granular Drainage Layer (Base Liner)

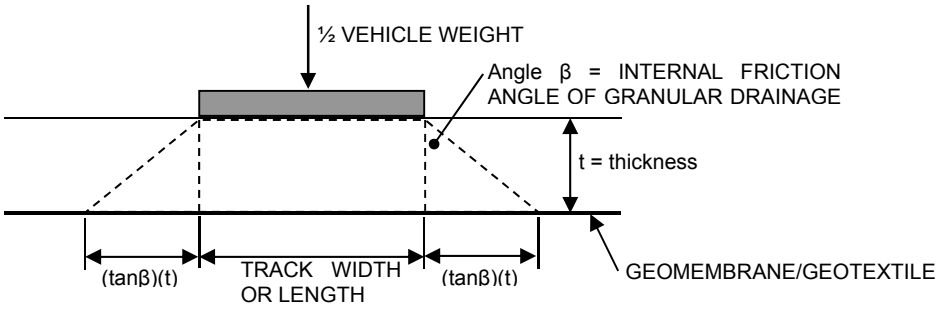
Unit Weight:	γ_{moist} = 130 pcf (average for all drainage/ops soils)
Thickness:	1 foot
Internal Friction Angle:	34 degrees
Overburden Pressure:	(1 foot)(130 pcf * 1 foot) = 130 psf = 6.2 KPa
Average Particle Size (d_{50}):	0.6 inches (15 mm) to 1 inch (25 mm)

Vehicle Pressure on Geomembrane (Dozer) (Attachment 4 from Reference 3)

Vehicle weight	43,976 lbs., say 44,000 lbs.
Dozer track width:	36 inches (3 ft)
Track length on ground:	128.5 inches (10.7 ft)
Ground pressure:	4.88 psi (702.7 psf)

Client	County of Kaua'i	**Subject**	Geotextile Cushion	**Prepared By**	KJB	**Date**	8/13/12
Project	Kekaha Landfill	Analysis		**Reviewed By**	TCR	**Date**	8/14/12
Phase II Lateral Expansion				**Approved By**	KJB	**Date**	8/14/12

Pressure on the geomembrane from a Caterpillar D6H LPG dozer can be determined using the following pressure prism:



Solving:

Surface area of the prism on the geomembrane/geotextile:

= [3 feet + (2((tan 34)(1 foot)))] * [10.7 feet + (2((tan 34)(1 foot)))]
= 52.4 sf

Resulting pressure on the geomembrane/geotextile is:

= ((1/2)(44,000 lbs)) / 52.4 sf = 420 psf (2.9 psi) = 20.1 kPa (kN/m^2)

Overburden Pressure on Geomembrane (Soils and Waste Mass)

Total overburden pressure on geomembrane/geotextile is determined by accounting for maximum waste thickness, final cover thickness, and granular drainage layer. The resulting overburden as calculated based upon Figures 1 and 2 is provided as Attachment 1 and estimated as 7,168 psf or 343 KPa.

Methodology

The following method is based upon landfill design for drainage layers in which a nonwoven geotextile can provide puncture protection. The method based upon 60-mil HDPE geomembrane (1.5 mm thick) for a firm base liner system subbase. Based upon Reference 2, the method is:

$$FS = \frac{p_{allow}}{p_{actual}}$$

where: FS = Factor of safety against geomembrane puncture
p_{allow} = allowable pressure for different types of geotextiles and site-specific conditions
p_{actual} = actual pressure due to surface impoundment

modifying for vehicle loading:

$$FS = \frac{p_{allow}}{p_{actual} + p_{vehicle}}$$

where: $p_{vehicle}$ = distributed vehicle surface pressure on the geomembrane surface

Based on Reference 2 numerous ASTM 5514 experiments an empirical formula for p_{allow} has been determined:

$$p_{allow} = \left(50 + 0.00045\frac{M}{H^2}\right)\left[\frac{1}{(MF_S)(MF_{PD})(MF_A)}\right]\left[\frac{1}{(RF_{CR})(RF_{CBD})}\right]$$

where: p_{allow} = allowable pressure (kPa)
M = geotextile mass per unit area (g/m^2)
H = protrusion height (m)
MF_S = modification factor for protrusion shape
MF_{PD} = modification factor for packing density
MF_A = modification factor for arching solids
RF_{CR} = reduction for long-term creep
RF_{CBD} = reduction factor for long-term chemical/biological degradation

All modification and reduction factors are based upon Table 5.18 of Reference 2 (Attachment 2). Note that all MF values are $\leq$ 1.0 and RF values are $\geq$ 1.0.

Client	County of Kaua'i	**Subject**	Geotextile Cushion	**Prepared By**	KJB	**Date**	8/13/12
Project	Kekaha Landfill	Analysis		**Reviewed By**	TCR	**Date**	8/14/12
Phase II Lateral Expansion				**Approved By**	KJB	**Date**	8/14/12

Modification and Reduction Factor Selection						
Factors	**Vehicle Loading**		**Buildout Loading (Protrusion height (H) limited to half the average particle size)**		**Buildout Loading (Protrusion height (H) equal to the average particle size)**	
	d_{50}=0.6 inches	d_{50}=1 inch	d_{50}=0.6 inches	d_{50}=1 inch	d_{50}=0.6 inches	d_{50}=1 inch
MF_S	1.0	1.0	1.0	1.0	1.0	1.0
	angular particles		angular particles		angular particles	
MF_{PD}	0.54	0.67	0.54	0.67	0.54	0.67
	granular drainage layer installed as a continuous layer		granular drainage layer installed as a continuous layer		granular drainage layer installed as a continuous layer	
MF_A	0.75	0.75	0.25	0.25	0.25	0.25
	granular drainage layer being installed, no waste mass in place)		waste mass and final cover installed		waste mass and final cover installed	
RF_{CBD}	1.0	1.0	1.3	1.3	1.3	1.3
	no leachate present at time of granular drainage layer placement		moderate leachate, typical of site leachate		moderate leachate, typical of site leachate	
RF_{CR}	2.0	2.0	1.5	1.5	2.0	2.0
	assume average particle as protrusion height into geomembrane, very conservative		protrusion height assumed to be half of the average particle size, conservative		protrusion height assumed to be average particle size, very conservative	

For ease and repetitive calculations, the above calculation was repeated in the following table/spreadsheet:

Particle Shape	Protrusion Height, H (m)	Geomembrane Protection Modification/Reduction Factors					Geotextile Mass, M (g/m^2)	Pallow (kPa)	Pactual (kPa)	Vehicle Type	Vehicle (kPa)	FS
		MFs	MFpd	Mfa	RFcbd	RFcr						
Gravel (d50 = 15 mm or 0.015 m)												
Angular	0.015	1.00	0.54	0.75	1.00	2.00	542	1400.0	6.2	Dozer	20.1	53.2
Angular	0.0075	1.00	0.54	0.25	1.30	1.50	542	16661.0	343	None	0.0	48.6
Angular	0.015	1.00	0.54	0.25	1.30	2.00	542	3230.8	343	None	0.0	9.4
Gravel (d50 = 25 mm or 0.025 m)												
Angular	0.025	1.00	0.67	0.75	1.00	2.00	542	438.0	6.2	Dozer	20.1	16.7
Angular	0.013	1.00	0.67	0.25	1.30	1.50	542	4571.6	343	None	0.0	13.3
Angular	0.025	1.00	0.67	0.25	1.30	2.00	542	1010.9	343	None	0.0	2.9

Conclusions

Based upon the above calculations, material properties, and design and equipment assumptions, the proposed 16 oz/yd^2 nonwoven geotextile will provide adequate puncture resistance of the geomembrane during granular drainage layer placement and after achieving final buildout geometry of the facility.

The geotextile shall have minimum material properties for nonwoven geotextile in accordance with Appendix F.

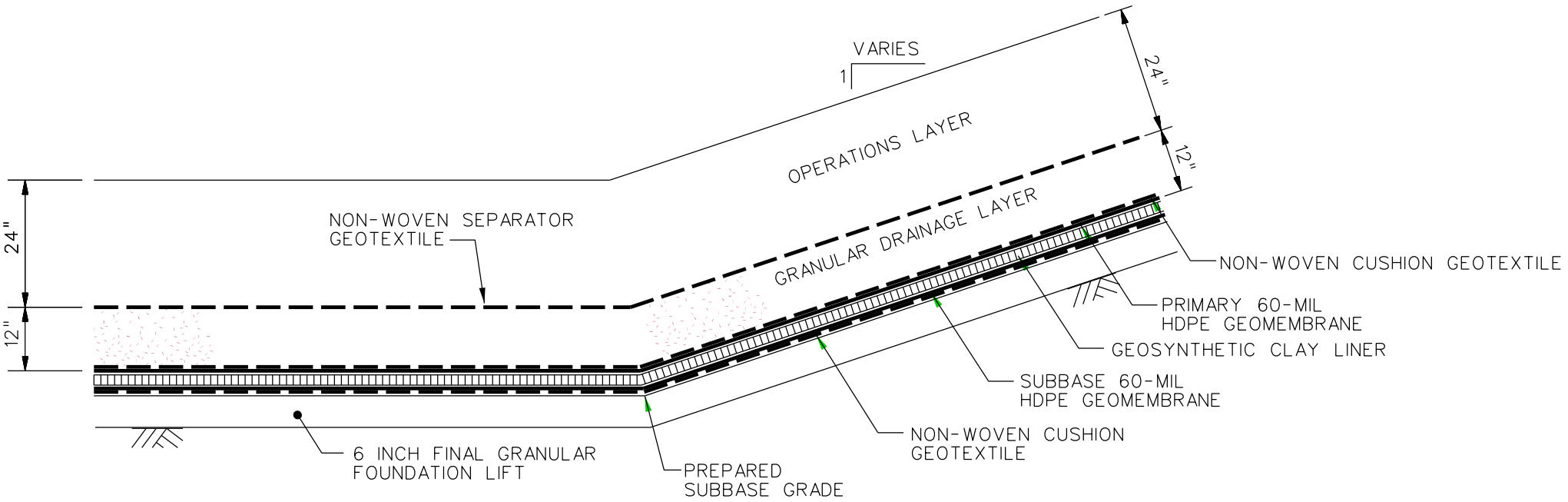
If different equipment, different drainage layer material proposed, or alternate geotextile materials are proposed at the time of construction, this calculation should be revisited.

References

1. Drawings entitled, "Engineering Report, Phase II Cell 2 Lateral Expansion, Kekaha Landfill," prepared by AECOM, dated August 2012.
2. Koerner, Robert M., "Designing with Geosynthetics," Fourth Edition, pps. 535-537.
3. "Caterpillar Performance Handbook," Edition 24, pages 1-8 and 1-21.
4. Koerner, Robert M., "Modification to the GRI-Method for the RF_{CR}-Factor Used in the Design of Geotextiles for Puncture Protection of Geomembranes," GRI White Paper #14, Geosynthetic Institute, November 2008.

FIGURE 1

TYPICAL CELL 2 BASE LINER SYSTEM DETAILS



NOTE:

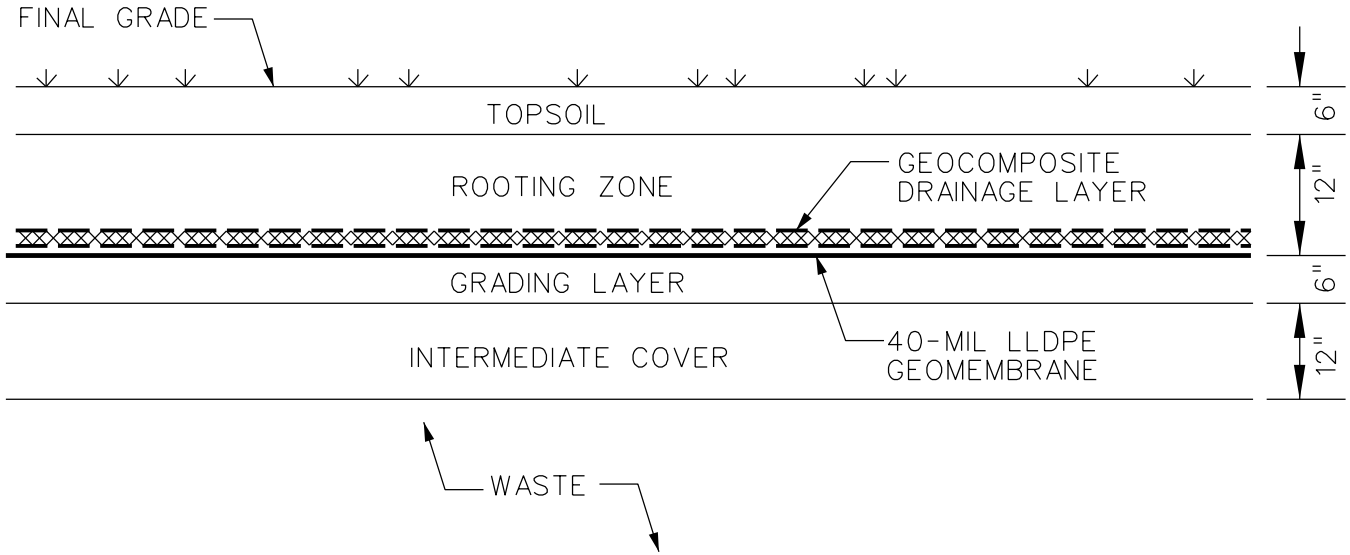
1. ALL GEOMEMBRANE IS TEXTURED ON BOTH SIDES.

BASE LINER DETAIL 1

NTS

FIGURE 2

FINAL COVER SYSTEM



FINAL COVER SYSTEM 4

NTS

ATTACHMENT 1

OVERBURDEN CALCULATION

Attachment 1
Cell 2 Overburden Pressure - Cushion Geotextile Analysis

Calculate Δσ			Overburden Pressure		
Proposed Base Liner System	(Figure 1)				
Granular drainage layer thickness		1 feet			=Design thickness
Granular drainage layer unit weight		130 pcf	130	psf	=Phase I aggregate layer
Operations layer thickness		2 feet			=Design thickness
Operations layer unit weight		130 pcf	260	psf	=Phase I soil buffer layer
Cell waste mass thickness*		104 feet			=Top of final cover minus subbase minus 3' for base liner section and 3' for final cover section)
Cell waste mass unit weight		62 pcf	6448	psf	
Proposed Overlay Final Cover	(Figure 2)				
Int. cover and grading layer thickness		1.5 feet			=Design thickness
Int. cover and grading layer unit weight		100 pcf	150	psf	(Phase I Closure/Postclosure Plan)
Rooting zone/topsoil layer thickness		1.5 feet			=Design thickness
Rooting zone/topsoil layer unit weight		120 pcf	180	psf	(Phase I Closure/Postclosure Plan)
Total overburden increase from vertical expansion Δσ=			**7168**	**psf**	
			343	**KPa**	

*Maximum overall thickness (subbase to top of cover) = 120' (Top of Final cover) - 10.5' (Sump elevation) - 3' (Granular/Ops layers) - 3' (Final Cover) = 103.5', 104'.

ATTACHMENT 2

TABLE 5.18 MODIFICATION FACTORS AND REDUCTION FACTORS FOR GEOMEMBRANE PROTECTION DESIGN USING NONWOVEN NEEDLE-PUNCHED GEOTEXTILES

TABLE 5.18 MODIFICATION FACTORS AND REDUCTION FACTORS FOR GEOMEMBRANE PROTECTION DESIGN USING NONWOVEN NEEDLE-PUNCHED GEOTEXTILES

Modification Factors

MF_S		MF_{PD}		MF_A	
Angular	1.0	Isolated	1.0	Hydrostatic	1.0
Subrounded	0.5	Dense, 38 mm	0.83	Geostatic, shallow	0.75
Rounded	0.25	Dense, 25 mm	0.67	Geostatic, mod.	0.50
		Dense, 12 mm	0.50	Geostatic, deep	0.25

Reduction Factors

		RF_{CR}			
			Protrusion (mm)		
RF_{CBD}		Mass per unit area (g/m^2)	38	25	12
Mild leachate	1.1	Geomembrane alone	N/R	N/R	N/R
Moderate leachate	1.3	270	N/R	N/R	> 1.5
Harsh leachate	1.5	550	N/R	1.5	1.3
		1100	1.3	1.2	1.1
		> 1100	≅ 1.2	≅ 1.1	≅ 1.0

N/R = Not recommended

ATTACHMENT 3

TABLE 3 REVISED VALUES FOR "RF_{CR}" TO BE USED INGEOTEXTILE PROTECTION MATERIALS DESIGN (FROM GRI WHITE PAPER #14)

Table 3. Revised values for "RF_{CR}" to be used in Equation 2 for geotextile protection materials design.

Mass per unit area (g/m^2)	"RF_{CR}"-Values Protrusion Height (mm)		
	38	25	12
Geomembrane alone	N/R	N/R	N/R
270	N/R	N/R	N/R
550	N/R	N/R	>1.5
1100	N/R	1.5	1.3
>1100	1.3	1.2	1.1

Abbreviation: N/R = Not recommended

Lastly, the entry of ">1.5" for a 12 mm cone height associated with a 550 g/m^2 geotextile is felt to be appropriate considering the following items.

- The geotextiles used at present are made from polypropylene fibers versus the tested geotextiles which were made from polyester fibers. Since the specific gravity of PP is 0.91 and that of PET is between 1.22 and 1.38, one has from 25% to 34% more filaments in an equivalent mass per unit area geotextile using polypropylene fibers. This provides for considerably greater protection capability.
- The area of yield for the six $\simeq$ 12 mm cone heights was extremely small and the thicknesses of the remaining geomembrane was such that considerable deformation could still be sustained before break is even close to occurring.
- The ">1.5" recommendation is precisely for additional conservatism and safety and if a designer wishes to be more conservative than the new recommended table suggests he/she is free to do so.

ATTACHMENT 4

EQUIPMENT PROPERTIES FOR CATERPILLAR D6H LGP DOZER






MODEL	D6H XR Series II		D6H LGP Series II		D7G		D7H Series II	
Flywheel Power	130 kW	175 hp	127 kW	170 hp	149 kW	200 hp	160 kW	215 hp
Operating Weight*								
(Power Shift)	18 684 kg	41,192 lb	19 814 kg	43,590 lb	20 666 kg	45,560 lb	24 694 kg	54,401 lb
(Direct Drive)	—		19 989 kg	43,976 lb	20 510 kg	45,218 lb	—	
(Power Shift Differential Steer)	18 799 kg	41,444 lb	—		—		24 993 kg	55,100 lb
Engine Model	3306		3306		3306		3306	
Rated Engine RPM	1900		1900		2000		2100	
No. of Cylinders	6		6		6		6	
Bore	121 mm	4.75"	121 mm	4.75"	121 mm	4.75"	121 mm	4.75"
Stroke	152 mm	6"	152 mm	6"	152 mm	6"	152 mm	6"
Displacement	10.5 L	638 in³	10.5 L	638 in³	10.5 L	638 in³	10.5 L	638 in³
Track Rollers (Each Side)	7		8		6		7	
Width of Standard Track Shoe	560 mm	22"	915 mm	36"	508 mm	20"	560 mm	22"
Length of Track on Ground	2.77 m	9'1"	3.27 m	10'8.5"	2.70 m	8'11"	2.90 m	9'6"
Ground Contact Area (W/Std. Shoe)	3.11 m²	4815 in²	5.97 m²	9254 in²	2.76 m²	4280 in²	3.24 m²	5016 in²
Track Gauge	1.88 m	6'2"	2.23 m	7'3"	1.98 m	6'5"	1.98 m	6'6"
GENERAL DIMENSIONS:								
Height (Stripped Top)**	2.26 m	7'5"	2.32 m	7'7"	2.27 m	7'5"	2.44 m	8'0"
Height (To Top of ROPS)	3.12 m	10'3"	3.16 m	10'5"	3.20 m	10'6"	3.33 m	10'11"
Height (To Top of Cab ROPS)	3.12 m	10'3"	3.16 m	10'5"	—		3.42	11'3"
Height (To Top of ROPS Canopy)	—		3.16 m	10'5"	—		—	
Overall Length (With P Blade)	—		5.18 m	17'0"	—		—	
(Without Blade)	—		4.49 m	14'9"	—		—	
Overall Length (With S Blade)	5.26 m	17'3"	—		5.28 m	17'4"	6.03 m	19'9"
(Without Blade)	4.21 m	13'10"	—		4.19 m	13'9"	4.73 m	15'6"
Width (Over Trunnion)	2.64 m	8'8"	3.43 m	11'3"	—		2.86 m	9'5"
Width (W/O Trunnion — Std. Shoe)	2.44 m	8'0"	3.14 m	10'3.6"	2.55 m	8'5"	2.54 m	8'4"
Ground Clearance	377 mm	14.8"	382 mm	15"	347 mm	13.7"	406 mm	16"
Blade Types and Widths:								
Straight	3.36 m	11'0"	3.99 m	13'1"	3.66 m	12'0"	3.91 m	12'10"
Angle	—		—		4.27 m	14'0"	—	
Angle Straight	4.16 m	13'8"	—		—		4.49 m	14'9"
Full Angle	3.78 m	12'5"	—		—		4.08 m	13'5"
Universal	—		—		3.81 m	12'6"	3.98 m	13'1"
Semi-U	3.26 m	10'8"	—		—		3.68 m	12'1"
Fuel Tank Refill Capacity	397 L	105 U.S. gal	337 L	89 U.S. gal	435 L	115 U.S. gal	479 L	127 U.S. gal

*Operating Weight includes ROPS canopy, operator, lubricants, coolant, full fuel tank, hydraulic controls and fluid, straight dozer with tilt, horn, back-up alarm, retrieval hitch and front pull hook.
— D7G includes end track guiding guards.
**Height (stripped top) — without ROPS canopy, exhaust, seat back or other easily removed encumbrances.



GROUND PRESSURES

Pressures computed from operating weights given earlier in this section in the specifications tables.

MODEL	SHOE WIDTH		CONTACT AREA		GROUND PRESSURE	
	mm	in	m^2	in^2	kPa	psi
D3C Series II ◄	356	14	1.35	2097	52	7.2
	406	16	1.54	2394	44	6.45
D3C LGP Series II	635	25	2.61	4045	29	4.2
D3C LGP-S Series II	990	39	4.85	7524	17	2.52
D4C Series II	406	16	1.67	2589	44	6.44
D4C LGP Series II	635	25	2.61	4045	30	4.3
D5C	457	18	1.92	3038	42.3	6.14
D5C LGP	660	26	2.83	4389	31.0	4.51
D4H Series II	360	14	1.59	2464	64	9.1
	410	16	1.82	2826	56	8.0
◄	460	18	2.05	3168	50	7.1
D4H XL Series III ◄	510	20	2.45	3799	47	6.73
	560	22	2.69	4172	44	6.17
D4H LGP Series III ◄	610	24	3.20	4953	38	5.34
	760	30	4.03	6252	31	4.39
	770	30	3.98	6170	31	4.41
D5E	406	16	1.77	2745	62	9.0
◄	457	18	1.99	3085	55	7.98
	508	20	2.21	3426	49.8	7.22
	560	22	2.44	3784	45	6.53
D5H Series II	460	18	2.11	3276	50.9	8.69
◄	510	20	2.35	3646	54.4	7.9
D5H XL Series II	560	22	2.78	4309	50	7.10
◄	600	24	3.63	4689	47	6.67
D5H LGP Series II	710	28	4.43	6866	36	5.15
◄	860	34	5.37	8321	30	4.24
	865	34	5.40	8369	29	4.17
D6D	457	18	2.17	3364	65	9.42
◄	508	20	2.41	3736	59	8.56
	560	22	2.65	4108	53	7.68
	610	24	2.89	4480	49	7.10
D6E	457	18	2.43	3766	60	8.70
◄	508	20	2.71	4200	54	7.83
	560	22	2.98	4619	49	7.10
	610	24	3.25	5040	45	6.54
D6H Series II	508	20	2.67	4140	65.0	9.44
◄	560	22	2.94	4564	59.0	8.56
	610	24	3.21	4971	54.2	7.86

◄Standard Shoe

MODEL	SHOE WIDTH		CONTACT AREA		GROUND PRESSURE	
	mm	in	m^2	in^2	kPa	psi
D6H XL Series II	510	20	2.89	4484	66	9.42
◄	560	22	3.18	4923	60	8.49
	610	24	3.46	5363	55	7.88
D6H XR Series II	508	20	2.83	4386	67	9.48
◄	560	22	3.11	4815	60	8.55
	610	24	3.38	5246	56	7.93
D6H LGP Series II ◄	760	30	4.96	7296	40	5.74
	915	36	5.97	9261	34	4.88
	1000	39	6.53	10,122	32	4.48
D7G ◄	508	20	2.76	4280	73	10.6
◄	559	22	3.04	4708	66	9.6
	610	24	3.31	5136	60	8.8
D7H Series II	510	20	2.94	4560	82	11.7
◄	560	22	3.24	5016	75	10.6
	610	24	3.53	5472	69	9.8
	660	26	3.82	5928	64	9.1
D7H XR Series II	560	22	3.43	5315	71.5	10.16
◄	610	24	3.75	5808	65.9	9.37
	660	26	4.06	6282	61.2	8.70
D7H LGP Series II	760	30	4.8	7504	54	7.74
	915	36	5.82	9029	46	6.55
D8N ◄	560	22	3.59	5565	100.6	14.6
	610	24	3.91	6062	92.3	13.4
	660	26	4.23	6559	85.4	12.4
	710	28	4.55	7056	79.2	11.5
D8N LGP*	965	38	6.2	9576	53.7	7.8
D9N	560	22	3.86	6009	107.0	15.51
◄	610	24	4.21	6555	99.1	14.37
	685	27	4.74	7374	89.2	12.94
	760	30	5.26	8194	79.4	11.51
D10N ◄	610	24	4.73	7326	124.1	18.0
	710	28	5.50	8527	107.8	15.6
	860	31.5	6.66	10,328	89.8	13.0
D11N ◄	710	28	6.31	9781	150.2	21.8
	810	32	7.20	11,159	132.7	19.3
	915	36	8.13	12,605	118.6	17.2

◄Standard shoe. *Offered as a Custom Product.

NOTE: Ground contact area = width of track shoe x length of track on ground x 2.

$$\text{Ground pressure} = \frac{\text{operating weight}}{\text{ground contact area}}$$

Appendix B.4B
Separation Geotextile Analysis

Client	County of Kaua'i	**Subject**	Geotextile	**Prepared By** KJB	**Date** 8/13/12
Project	Kekaha Landfill		Separation Analysis	**Reviewed By** TCR	**Date** 8/14/12
	Phase II Lateral Expansion			**Approved By** KJB	**Date** 8/14/12

GEOTEXTILE SEPARATION ANALYSIS

OBJECTIVE

A nonwoven geotextile is proposed as a filter and as a separation layer between the operations layer and the granular drainage layer on the base of Cell 2. The use of the nonwoven geotextile will be used to prevent fines from entering the drainage layer from the operations layer and overlaying waste mass. Therefore, the geotextile filter was evaluated for 1) soil retention, 2) permeability, and 3) long-term effectiveness.

DESIGN CRITERIA AND ASSUMPTIONS

1. The application of a geotextile as a separation layer requires the review of specific material properties including Apparent Opening Size (AOS), permittivity, and permeability.
2. The separation geotextile will be a nonwoven geotextile with a minimum mass per unit area of 6 oz/yd^2.

CALCULATIONS

Material Properties

The material design properties for the separation geotextile are:

Table 1 – Separation Geotextile Properties

Property	ASTM Test Method	Value
Mass per unit area (oz/yd^2)	D5261	6
Apparent Opening Size (US Sieve)	D4751	70
Permittivity (sec^{-1})	D4491	1.5
Permeability (cm/sec)	D4491	0.30
Geotextile Structure	Nonwoven, needle-punched	
Notes: (1) All values are minimum average roll value (MARV) except for Apparent Opening Size which is a maximum average roll value.		

Soil Retention Analysis

Geotextile functioning as a filtration media should have an AOS small enough to retain fines on the upstream of the fabric. For an adequate AOS, the Taskforce #25 (AASHTO - ABC - ARBTA) specification M-288-96, shown below as Table 2, was used to evaluate the capability of geotextile filtration:

Table 2 - M-288-96 Specification
Maximum Apparent Opening Size Requirements

Application	1996 - Percent In-Situ Soil Passing 0.075 mm (US Sieve No. 200)		
	<15	15 to 50	>50
Subsurface Drainage	0.43 mm	0.25 mm	0.22 mm
Temporary Erosion Control	0.43 mm	0.25 mm	0.22 mm
Separation		Class 2 geotextile with 0.60 mm	
Stabilization		Class 1 geotextile with 0.43 mm	

Based upon Table 2, the geotextile should have an AOS = 0.22 mm or smaller openings. For the proposed 6 oz/yd^2 geotextile an AOS equal to the US No. 70 sieve which is equivalent square opening of 0.212 mm is specified. Therefore, the geotextile will have an AOS small enough to retain fines for all conditions and soil types listed in Table 2, especially separation and for soils with more than 50% passing a US No. 200 sieve.

Geotextile Permeability

Geotextile functioning as a filtration media should have an adequate permeability to percolate liquid into the leachate granular drainage layer and to provide effective clogging protection. The M-288-96 specification for geotextile permittivity is shown in Table 3.

Table 3 - M-288-96 Specification
Permittivity Requirements

Application	1996 - Percent In-Situ Soil Passing 0.075 mm (US Sieve No. 200)		
	<15	15 to 50	>50
Subsurface Drainage	0.5 sec^{-1}	0.2 sec^{-1}	0.1 sec^{-1}
Temporary Erosion Control	0.7 sec^{-1}	0.2 sec^{-1}	0.1 sec^{-1}
Separation	---	Class 2 geotextile with 0.2 sec^{-1}	---
Stabilization	---	Class 1 geotextile with 0.5 sec^{-1}	---

The design permittivity of the selected geotextile shall be 1.5 sec^{-1} (Table 1) based on the geotextiles MARV values.

The design permittivity is greater than the minimum M-288-96 specification requirement of 0.1 sec^{-1}, for geotextiles used in subsurface drainage or 0.2 sec^{-1} for separation indicating an adequate permittivity of the geotextile. As a comparison, the factor of safety would be equal to (1.5 sec^{-1} / 0.1 sec^{-1}) = 15.0.

Long-Term Flow Equilibrium

The following equation from Reference 2 is used to evaluate the geotextile long-term flow equilibrium:

$$F.S. = \frac{K_{allowable}}{K_{required} \times DCF}$$

where: $K_{allowable}$ = the allowable geotextile permeability
$K_{required}$ = the required flow rate per unit area
DCF = Drainage Correction Factor

The required flow capacity was determined through use of the average monthly rate for open conditions as calculated by HELP. An average monthly rate of 651.9 gallons/acre/day (refer to Appendix D-1) will be used.

Converting gallons/acre/day to cm/sec for $K_{required}$:

$K_{required}$ = (651.9 gallons/acre/day)(0.13368 ft^3/gallon)(1 acre/43,560 ft^2) = 0.002001 ft/day
(0.002001 ft/day) (12 inches / ft) (1day / 86,400 sec) = 2.8 $x10^{-7}$ inches /day
(2.8 $x10^{-7}$ inches /day) (2.54 cm /in) = 7.1 $x10^{-7}$ cm/sec

$K_{required}$ = 7.1 $x10^{-7}$ cm/sec

The design permeability of the proposed geotextile is 0.30 cm/sec, or $K_{allowable}$ = 3$x10^{-1}$ cm/sec.

According to Reference 2, Figure 6 (a) presented as Attachment 1, for the case of geotextile to be used as the entire cell filter, DCF = 1. Therefore,

$$F.S. = \frac{Kallowable}{Krequired \times DCF} = \frac{3x10^{-1}\ cm/sec}{(7.1x10^{-7}\ cm/sec)(1)} = 4.2x10^{5}$$

The drainage correction factor DCF is defined as the total cell area divided by the actual area for flow into the downstream drains of the leachate collection system. Therefore, the value of DCF could vary in a wide range based upon the various designs of the leachate collection system of landfills as shown in Figure 6 (Attachment 1). The above calculated factor of safety for permeability is extremely large and takes into consideration various conditions for the leachate collection system of the landfill.

Conclusion

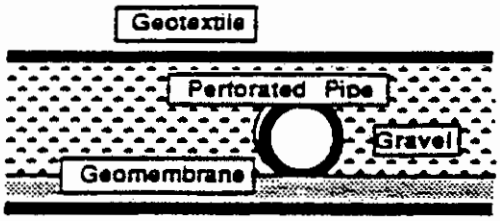
The proposed geotextile minimum material properties for a readily available nonwoven geotextile demonstrated the geotextile will perform well as separation geotextile for soil retention, permeability, and long-term flow.

References

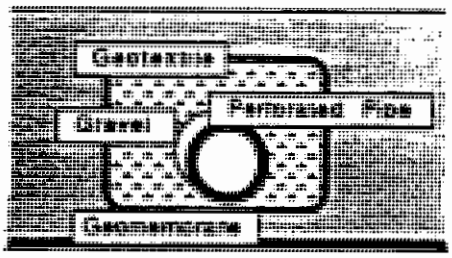
1. L. David Suits, "Review of the Updated American Association of State Highway and Transportation Officials Geotextile Specification," Volume 2, Geosynthetics 1997 Conference Proceedings, Long Beach, CA, March 1997.
2. G. R. Koerner, "Geotextile Filters Used for Leachate Collection Systems: Testing, Design and Field Behavior," GRI-151, 1993.

ATTACHMENT 1

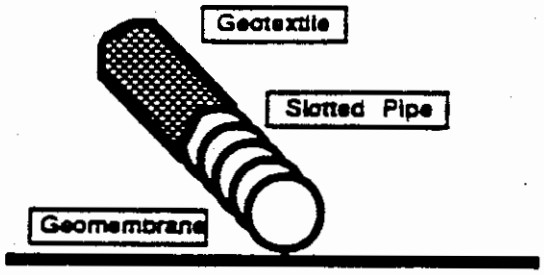
G. R. Koerner, "Geotextile Filters Used for Leachate Collection Systems: Testing, Design and Field Behavior," GRI-151, 1993 - FIGURE 6, "Range of Drain Correction Factors (DCF) for Several Different Leachate Collection Drainage Configurations"



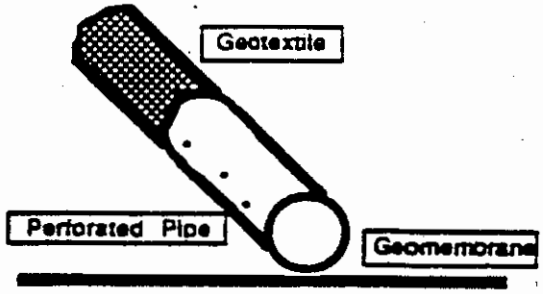
a) Entire cell filter (DCF = 1)



b) Geotextile wrapped drain (DCF = 10 - 40)



c) Socked corrugated pipe (DCF = 60 -260)



d) Socked smooth walled perforated pipe (DCF = 7,500 -24,000)

Fig. 6. Range of Drain Correction Factors (DCF) for Several Different Leachate Collection Drainage Configurations.

Appendix B.5
Final Cover Drainage Layer Capacity and Slope Stability Analysis

Client	County of Kaua'i	**Subject**	Final Cover	**Prepared By** KJB	**Date** 8/26/13
Project	Kekaha Landfill		Drainage Layer Capacity	**Reviewed By** BPS	**Date** 8/26/13
Phase II Vertical Expansion			and Slope Stability	**Approved By** KJB	**Date** 8/26/13

FINAL COVER DRAINAGE LAYER CAPACITY AND SLOPE STABILITY

Objective

Evaluate the final cover slope stability under static and seepage conditions for the proposed Phase II and Cell 1 Vertical Expansion at Kekaha Landfill that incorporates the use of geosynthetics and soil materials.

Design Criteria & Assumptions

1. The attached final cover cross section is applicable (Figure 1).
2. A 100-year 24-hour rain event of 12 inches is the assumed rain event for the seepage condition.
3. Interface friction angle testing as well as transmissivity testing should be completed prior to construction and use of the proposed materials. This calculation is completed to evaluate the acceptability of the proposed drainage materials under expected normal loadings and minimum interface friction angles. Material and interface friction angle testing requirements are presented in accompanying Engineering Report.
4. The geocomposite drainage layer will consist of a HDPE geonet with a nonwoven geotextile heat-bonded to both sides.
5. A factor of safety of 1.5 will be used to determine slope stability under static conditions (consistent with past permitted approvals). A factor of safety of 1.2 and a drainage layer capacity greater than one (DLC >=1) will be used to assess stability for seepage conditions (short-term condition) in defining minimum material requirements. By defining a DLC >=1 all infiltration through the protective cover soil will be managed in the thickness of the geocomposite drainage layer. Under short-term conditions, the final cover protective cover soil layer above the geocomposite drainage layer is assumed to be at field capacity, a temporary condition.
6. This analysis is completed only on the final cover sideslope (3.5:1) as this represents the steepest slopes, required overall stability of the cover system, and drainage performance of the geocomposite.
7. The calculation also assumes that the drainage from the proposed crown of the facility is day lighted or drained prior to the 3.5:1 slopes.
8. Seismic analysis will not be completed as the site specific horizontal peak ground acceleration is 0.06 g and is less than 0.10 g required by Subtitle D for analysis (Reference 1).

Calculations

The stability of the final cover is primarily dependent upon the shear resistance of the weakest interface, slope length, slope inclination, and the seepage condition. The following information was collected/selected before conducting the actual stability analysis.

Material/Design Properties

Final cover material properties are based upon Appendix B of Reference 1 for unit weight. Final cover soil is expected to be imported to the facility for construction.

Protective Cover Soil (well graded sand, Reference 1):

Unit weight, moist	γ_{moist} = 110 pcf = 17.3 kN/m^3
Unit weight, saturated (assumed)	$\gamma_{saturated}$ = 120 pcf = 18.9 kN/m^3
Permeability	$k_{c.s.}$ = varies
Thickness (permitted)	$h_{c.s.}$ = 18 inches = 457.2 mm
Internal friction angle	$\phi_{c.s.}$ = 30 degrees (conservative)

Runoff Coefficient:

Runoff coefficient (RC) is an estimated percentage of precipitation allowed to runoff the final cover. The runoff coefficient is dependent upon the type of soils and slope steepness. The proposed protective cover soil will consist of on-site well graded sands. The runoff coefficient based upon Reference 5 (Attachment 1) is estimated at:

Clayey soils, slopes greater than 7% RC = 0.25 to 0.35
Sandy soils, slopes greater than 7% RC = 0.15 to 0.20

For conservative design, a runoff coefficient of RC = 0.15 has been used for the 3.5:1 slope (28.6% slope). If a lower permeable soil is used (i.e. clayey soils), then the amount of expected runoff will increase and reduce the overall infiltration to the geocomposite, further increasing this calculation conservatism.

Slope Inclination:

Slope inclination: β = 3.5H:1V = $\tan^{-1}$ (1/3.5) = 15.9 degrees

Design rainfall intensity (Reference 7):

100-year, 24-hour storm event	I = 12 inches/24 hours
	I = (12/24) inches/hour = 0.50 inches/hr
	I = 12.7 mm/hr

Geocomposite Drainage Layer Properties:

Geocomposite thickness (geonet only): $t_{composite}$ = 200 mil or 5 mm
(Per GSE, Attachment 2)

Transmissivity is determined by applying the following equation (References 8 and 9)

$$\theta_{allow} = \left(\frac{\theta_{100}}{\mathrm{RF}_{\mathrm{CR}} * RF_{CC} * RF_{BC}} \right)$$

where:

θ_{allow} = allowable transmissivity, m^2/sec
θ_{100} = 100 hour transmissivity data, m^2/sec
RF_{CR}= Reduction factor for creep
RF_{CC}= Reduction factor for chemical clogging
RF_{BC}= Reduction factor for biological clogging

Gradient for transmissivity testing is equal to sin β (Reference 9) or sin (15.9 degrees) = 0.274.

Since this is a final cover system, the overburden or loading applied to the geocomposite is equal to the thickness of the final cover soils multiplied by the unit weight. For the proposed final cover cross section the overburden is (1.5 feet * 120 pcf) = 180 psf. Accounting for construction equipment placing the soils, which would include a dozer with a maximum 5 psi surface pressure, the resulting overburden is (180 psf +(5 psi * 144 in^2/sf)) = 900 psf. Equipment loading is considered temporary. Therefore, assuming an overburden loading of 1,000 psf for 100 hours is conservative.

The 100 hour transmissivity data (θ_{100}) for 200-mil GSE Fabrinet geocomposite with a 6 or 8-oz geotextile with a boundary condition of soil to geocomposite to geomembrane is determined using Figure A-3 of Reference 9 (Attachment 3) for 1,000-psf loading, 100-hour seat time, at a gradient of 0.27:

$\theta_{100} = 1.7 \times 10^{-4}$ m^2/sec, say → 1.5×10^{-4} m^2/sec

Reduction factors of RF_{CC} and RF_{BC} are selected from Table 4.4 of Reference 9:

RF_{CC}= 1.1 (assumes low alkaline soils)
RF_{BC}= 1.5 (assumes very little to no root growth into geocomposite)

Reduction factor of RF_{CR} (creep) is determined based upon increasing overburden stress. Creep reduction factors for cover applications are much smaller than those for liner systems. Based upon Section 4.3.4 and Appendix B - Table B-1 of Reference 9, RF_{CR} = 1.1 for GSE 200 mil HyperNet Geonet (geonet core for the geocomposite).

Solving allowable transmissivity:

$$\theta_{allow} = \left(\frac{1.5x10^{-4} m^2/\sec}{1.1*1.1*1.5}\right) = \left(\frac{1.5x10^{-4} m^2/\sec}{1.82}\right) = 8.24x10^{-5} m^2/\sec$$

Permeability:

$$k_d = \left(\frac{8.24\text{x}10^{-5}\ \text{m}^2/\text{sec}}{5\,\text{mm}}\right)\left(\frac{100\,\text{cm}}{1\,\text{m}}\right)^2\left(\frac{10\,\text{mm}}{1\,\text{cm}}\right) = 1.65\,\text{cm/sec}$$

Maximum slope length (Reference 2):

Maximum slope length for the 3.5:1 cover slope (east sideslope across from the leachate evaporation pond) based upon longest change in elevation

$$(\text{El.}114 - \text{El.}52)\left(\sqrt{3.5^2 + 1^2}\right) = 225.7\ \text{feet} \Rightarrow 230\ \text{feet or}\ 70.1\,\text{m}$$

Infinite Slope Stability

The infinite slope stability is determined by force summation along the sideslope angle resulting in a factor of safety (Reference 6) without a buttress effect at the toe of slope or:

$$FS = \frac{\text{resisting forces}}{\text{driving forces}} = \frac{F}{W\sin\beta} = \frac{N\tan\delta}{W\sin\beta} = \frac{W\cos\beta\tan\delta}{W\sin\beta}$$

$$FS = \frac{\tan\delta}{\tan\beta}$$

where:

FS = Factor of Safety
δ = critical interface friction angle
β = sideslope angle

Solving for 3.5:1 slope (15.9 degrees) and Factor of Safety of 1.5, the minimum critical interface friction angle is:

$$FS = \frac{\tan\delta}{\tan\beta} \Rightarrow 1.5 = \frac{\tan\delta_{3.5:1}}{\tan 15.9} \Rightarrow \delta_{3.5:1} = \tan^{-1}(1.5 * \tan 15.9)$$

$$\delta_{3.5:1} = 23.1\,\text{degrees}$$

The infinite slope stability is not dependent upon cover soil thickness, slope length, or cover soil permeability.

Client	County of Kaua'i	**Subject**	Final Cover	**Prepared By** KJB	**Date** 8/26/13
Project	Kekaha Landfill		Drainage Layer Capacity	**Reviewed By** BPS	**Date** 8/26/13
Phase II Vertical Expansion			and Slope Stability	**Approved By** KJB	**Date** 8/26/13

Methodology for Drainage Layer Capacity Analysis

The degree of submergence of the final cover under the design precipitation should be determined prior to conducting the final cover stability analysis. In order to quantify the degree of submergence, the drainage layer capacity (DLC) of the design option will be determined first. The procedure is based upon those presented by Soong and Koerner, 1997 (Reference 3).

The procedure is:

$$DLC = \frac{FLUX_{allow}}{FLUX_{req'd}}$$

For geocomposite drainage layer:

$FLUX_{allow}$ = Allowable flux (flow rate per unit width of final cover)
= $k_d iA$
= $k_d \sin\beta(t*1)$
= $(k_d*t)\sin\beta$
= $\theta\sin\beta$

where:

k_d = permeability of drainage material
i = hydraulic gradient = $\sin\beta$
β= slope angle
A = cross-sectional area of the drainage layer per unit width of final cover
t = thickness of drainage layer
θ = transmissivity of geocomposite

$FLUX_{req'd}$ = Required flux per unit width of sideslope
= (PERC)L*cosβ(1.0)

where:

PERC = percolation = $\begin{cases} I & (k_{\text{cover soil}} \geq I) \\ k_{\text{cover soils.}} & (k_{\text{cover soils}} < I) \end{cases}$

I = design rainfall intensity
$k_{cover\ soils}$ = permeability of vegetative/protective cover
L = slope length
β = slope angle

Methodology for Seepage Slope Stability Analysis

The final cover stability analysis is conducted using procedures developed specifically for veneer cover situations; see Koerner and Soong, 1998 (Reference 4). A spreadsheet program, based on the procedures recommended in the above referenced literature for seepage conditions was constructed, calibrated, and used in this analysis. Note that the input values were previously converted as needed into SI units for ease of spreadsheet input and analysis.

Results

The following conditions have been considered in the final cover stability analyses:

1. Static condition: Determine the minimum interface friction angle based upon infinite slope stability analysis (see "infinite slope stability" section above) to achieve a factor of safety of 1.5; and,

2. Seepage condition: Determine the correlations between the maximum length of continuous geocomposite drainage layer, permeability of protective soils and the critical interface friction angle. A minimum factor of safety of 1.2 is targeted for the seepage (short term) condition.

The typical permeability of the imported final cover soil was varied between $1x10^{-4}$ cm/sec and $1.0x10^{-5}$ cm/sec for the seepage analysis to assess the maximum allowable uninterrupted slope installation length for the proposed geocomposite while soil thickness, design slope, precipitation, soil parameters, and liner shear strength were held constant.

The results of the static final cover stability have been completed (see "infinite slope stability" section above)and on spreadsheets with the results attached and summarized below in Table 1.

Table 1
Summary of Slope Stability Results For Static and Seepage Conditions

Condition	Slope (H:V)	Rain Event	Maximum Protective Cover Permeability (cm/sec)	Slope Length Feet, (meters)	Cover Soil Thickness Feet, (mm)	Critical Friction Angle	Factor of Safety	DLC	Figure
Static	3.5:1	Infinite slope stability analysis, these parameters are not applied.				23.1	1.50 = 1.50	NA	NA
Seepage	3.5:1	100-year	*$1x10^{-4}$	77.1 (23.5)	1.5 (457.2)	23.1	1.55 > 1.20	1.0 = 1.0	2
			$3.35x10^{-5}$	230 (70.1)			1.51 > 1.20	1.0 = 1.0	3
			$1x10^{-5}$	770.8 (234.9)			1.49 > 1.20	1.0 = 1.0	4

* Equals maximum allowable permeability per the final cover/base liner equivalency calculation.

Conclusions and Discussion

Based upon the above static infinite slope analysis, the weakest peak shear strength as represented by the friction angle phi (Φ) for any interface within the final cover system should be a minimum of 23.1 degrees. Based on the seepage analysis, the maximum 3.5:1 slope length (230 feet or 70.1 meters) can be covered with a continuous layer of geocomposite if the protective cover soil layer immediately overlying the geocomposite has a maximum permeability of 3.35×10^{-5} cm/sec. If the permeability of the cover soils is greater than 3.35×10^{-5} cm/sec, then the specific geocomposite proposed for this project will need to be day lighted along the slope length at the maximum slope length intervals listed.

If the transmissivity and correlating permeability of a site specific geocomposite is greater than the design requirement, this calculation should be revisited and slope lengths may potentially increase for a continuous layer of geocomposite with higher permeability protective cover soils. Please note continuous geocomposite slope lengths cannot be increased by increasing the critical friction angle since the DLC factor governs the suitability of the geocomposite. The overall factor of safety would increase, but the performance of the geocomposite still governs maximum continuous geocomposite slope length.

Interface friction angle testing should be completed prior to final acceptance of final cover liner system materials. Transmissivity testing should also be performed at 100 hour seat time at 1,000 psf load with actual site soil and geosynthetic materials to further substantiate the design assumptions.

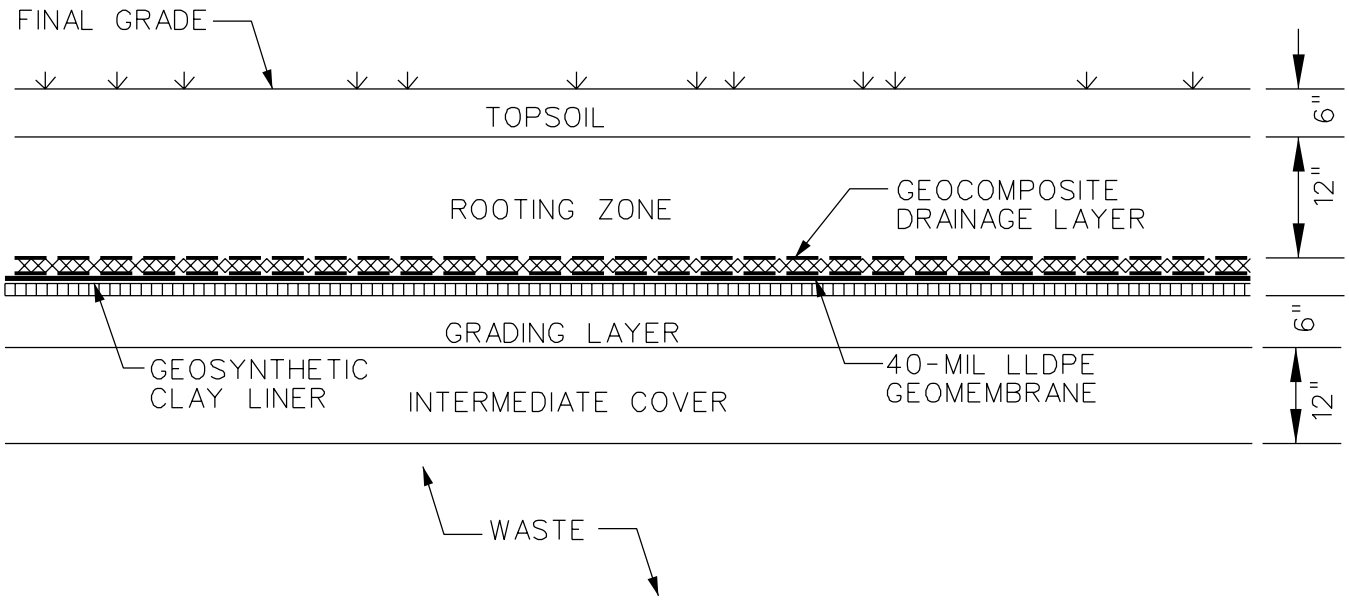
This calculation should be revisited if a geocomposite other than GSE FabriNet geocomposite is selected for final cover installation.

References

1. "Addendum Operations Manual For Kekaha Sanitary Landfill – Phase II," prepared by EMCON, May 1998.
2. Drawings titled, "Engineering Report, Kekaha Landfill Phase II Vertical Expansion, Kekaha Sanitary Landfill," prepared by AECOM, dated August 2013.
3. Soong, T.-Y. and Koerner, R.M. (1997) "The Design of Drainage Systems Over Geosynthetically Lined Slopes," Report #19, Geosynthetic Research Institute, Philadelphia, PA 19104, 88 pgs.
4. Koerner, R. M. and Soong, T.-Y. (1998), "Analysis and Design of Veneer Cover Soils" *Proc. 6th Int. Conf. on Geosynthetics, Atlanta*, USA, IFAI, pp. 1-23.
5. Daniel, David E., "Geotechnical Practice for Waste Disposal," page 216.
6. Koerner, R. M., "Designing with Geosynthetics," Fourth Edition, pps. 478 and 480.
7. Department of Engineering, County of Kauai. 2001. Stormwater Runoff System Manual. July. Lihue, Kauai, Hawaii
8. Standard Guide for "Determination of the Allowable Flow Rate of a Drainage Geocomposite," Geosynthetic Research Institute, GRI Standard GC8, Revision 1: January 9, 2013.
9. "The GSE Drainage Design Manual," Second Edition, GSE Environmental.

FIGURE 1

TYPICAL FINAL COVER LINER CROSS SECTION



FINAL COVER SYSTEM 1

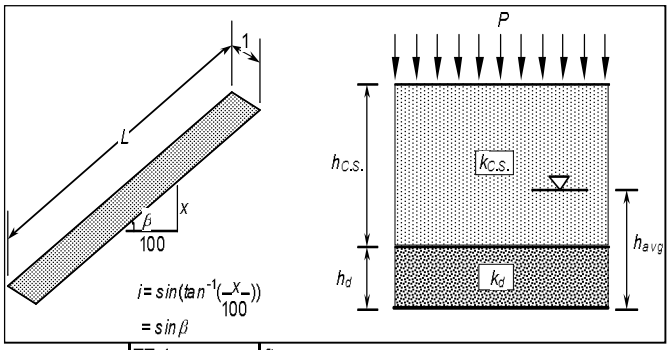
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FIGURES 2 THROUGH 4

SLOPE STABILITY UNDER SEEPAGE CONDITIONS FIGURES
3.5:1 SLOPE, VARIOUS SLOPE LENGTHS, 18-INCH PROTECTIVE COVER THICKNESS

Calculation of stability under seepage condition

Calculation of DLC and PSR



	77.1	ft			
L =	23.5	m	i =	0.2740	
β =	15.9	°	$L(\cos\beta)$ =	22.60	m
			x =	6.44	m
$h_{C.S.}$ =	457.2	mm	$h_{C.S.}$ =	0.5	m
h_d or t_{GS} =	5.0	mm	h_d or t_{GS} =	0.01	m
			$h_{C.S.} + h_d$ =	0.46	m
$k_{C.S.}$ =	1.00E-04	cm/s	$k_{C.S.}$ =	1.0E-06	m/s
k_d or k_{GS} =	1.65E+00	cm/s	k_d or k_{GS} =	1.7E-02	m/s
P =	12.70	mm/hr	P (RC) =	1.9	mm/hr
RC =	0.15		Actual runoff =	9.10	mm/hr
			PERC =	3.60	mm/h
			$FLUX_{actual}$ =	0.081	m^3/hr
			$FLUX_{allow}$ =	0.081	m^3/hr

Results:

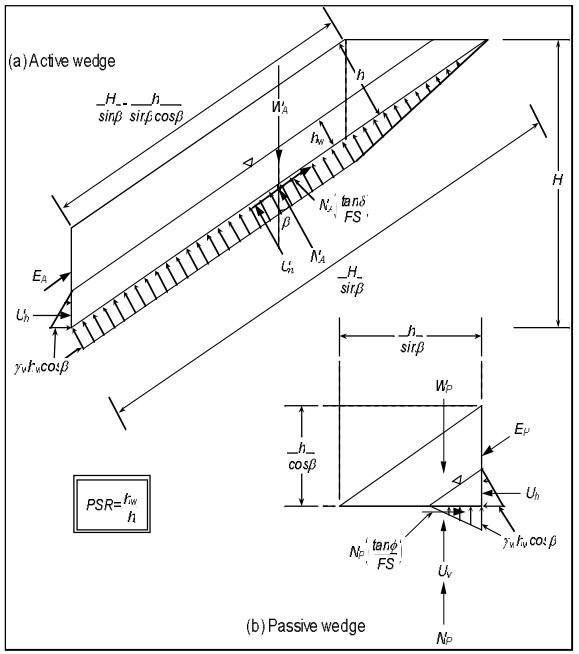
DLC	1.000
PSR	0.020
FS	1.550

DLC = 0.9999

q = 2.3E-05 m^3/sec

h_{avg} = 0.01 m

PSR = 0.020



thickness of cover soil = h = 0.46 m

length of slope measured along the top of clay layer = L = 24 m

soil slope angle along top of clay = β = 15.9 ° = 0.28 (rad.)

vertical height of the slope measured from the toe = H = 6.4 m

parallel submergence ratio = PSR = 0.02

depth of the water surface measured from the top of clay layer = h_w = 0.01 m

dry unit weight of the cover soil = γ_{moist} =	17.3	kN/m^3	
saturated unit weight of the cover soil = $\gamma_{sat'd}$ =	18.9	kN/m^3	
unit weight of water = γ_w =	9.81	kN/m^3	
friction angle of the cover soil = ϕ =	30.0	°	= 0.52 (rad.)
critical interface = δ =	23.1	°	= 0.40 (rad.)

Calculation of FS

Active Wedge:

W_A = 181.262 kN

U_n = 2.07626 kN

U_h = 0.00043 kN

N_A = 172.251 kN

Passive Wedge:

W_P = 7.01372 kN

U_V = 0.00151 kN

$$FS = \frac{-b + \sqrt{b^2 - 4ac}}{2a}$$

where a = 47.8

b = -81.5

c = 11.6

FS= 1.550

Note: numbers in boxes are input values

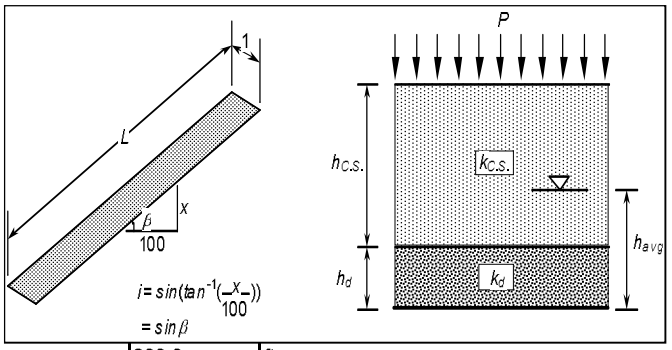
numbers in Italics are calculated values

Constructed by Te-Yang Soong

Figure 2

Calculation of stability under seepage condition

Calculation of DLC and PSR



	230.0	ft			
L =	70.1	m	i =	0.2740	
β =	15.9	°	$L(\cos\beta)$ =	67.42	m
			x =	19.21	m
$h_{C.S.}$ =	457.2	mm	$h_{C.S.}$ =	0.5	m
h_d or t_{GS} =	5.0	mm	h_d or t_{GS} =	0.01	m
			$h_{C.S.} + h_d$ =	0.46	m
$k_{C.S.}$ =	3.35E-05	cm/s	$k_{C.S.}$ =	3.4E-07	m/s
k_d or k_{GS} =	1.65E+00	cm/s	k_d or k_{GS} =	1.7E-02	m/s
P =	12.70	mm/hr	P (RC) =	1.9	mm/hr
RC =	0.15		Actual runoff =	11.49	mm/hr
			PERC =	1.21	mm/h
			$FLUX_{actual}$ =	0.081	m^3/hr
			$FLUX_{allow}$ =	0.081	m^3/hr

Results:

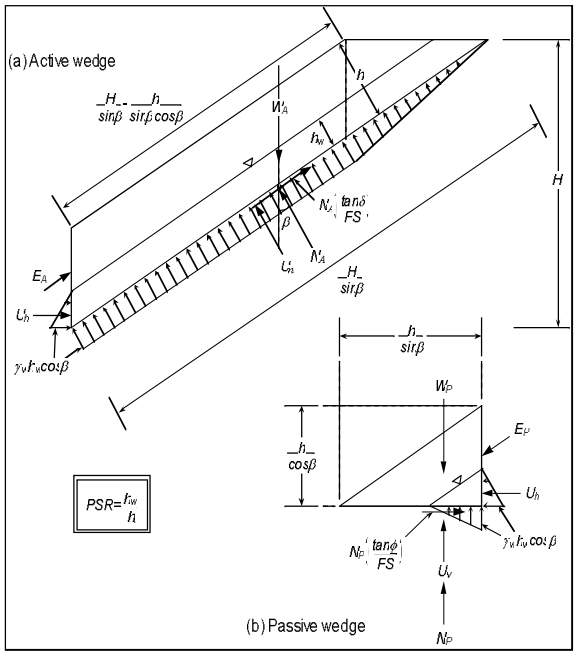
DLC	1.000
PSR	0.011
FS	1.511

DLC = 1.0003

q = 2.3E-05 m^3/sec

h_{avg} = 0.00 m

PSR = 0.011



Calculation of FS

Active Wedge:

W_A = 554.103 kN
U_n = 3.30568 kN
U_h = 0.00012 kN
N_A = 529.598 kN

Passive Wedge:

W_P = 7.01353 kN
U_V = 0.00043 kN

$$FS = \frac{-b + \sqrt{b^2 - 4ac}}{2a}$$

where a = 146.0
b = -244.3
c = 35.7

FS= 1.511

thickness of cover soil = h = 0.46 m
length of slope measured along the top of clay layer = L = 70 m
soil slope angle along top of clay = β = 15.9 ° = 0.28 (rad.)
vertical height of the slope measured from the toe = H = 19.2 m
parallel submergence ratio = PSR = 0.01
depth of the water surface measured from the top of clay layer = h_w = 0.00 m

dry unit weight of the cover soil = γ_{moist} =	17.3	kN/m^3
saturated unit weight of the cover soil = $\gamma_{sat'd}$ =	18.9	kN/m^3
unit weight of water = γ_w =	9.81	kN/m^3
friction angle of the cover soil = ϕ =	30.0	° = 0.52 (rad.)
critical interface = δ =	23.1	° = 0.40 (rad.)

Note: numbers in boxes are input values

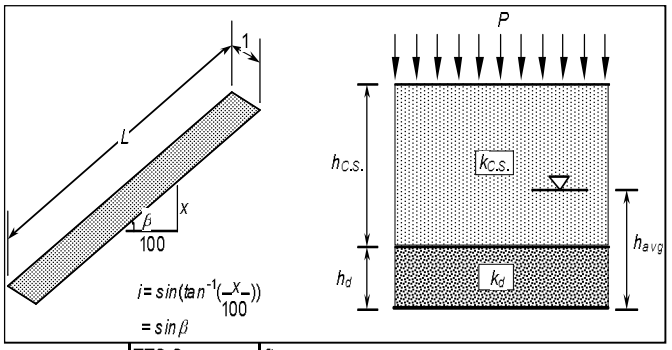
numbers in Italics are calculated values

Constructed by Te-Yang Soong

Figure 3

Calculation of stability under seepage condition

Calculation of DLC and PSR



	770.8	ft			
L =	234.9	m	i =	0.2740	
β =	15.9	°	$L(\cos\beta)$ =	225.95	m
			x =	64.36	m
$h_{C.S.}$ =	457.2	mm	$h_{C.S.}$ =	0.5	m
h_d or t_{GS} =	5.0	mm	h_d or t_{GS} =	0.01	m
			$h_{C.S.} + h_d$ =	0.46	m
$k_{C.S.}$ =	1.00E-05	cm/s	$k_{C.S.}$ =	1.0E-07	m/s
k_d or k_{GS} =	1.65E+00	cm/s	k_d or k_{GS} =	1.7E-02	m/s
P =	12.70	mm/hr	P (RC) =	1.9	mm/hr
RC =	0.15		*Actual runoff* =	12.34	mm/hr
			PERC =	0.36	mm/h
			$FLUX_{actual}$ =	0.081	m^3/hr
			$FLUX_{allow}$ =	0.081	m^3/hr

Results:

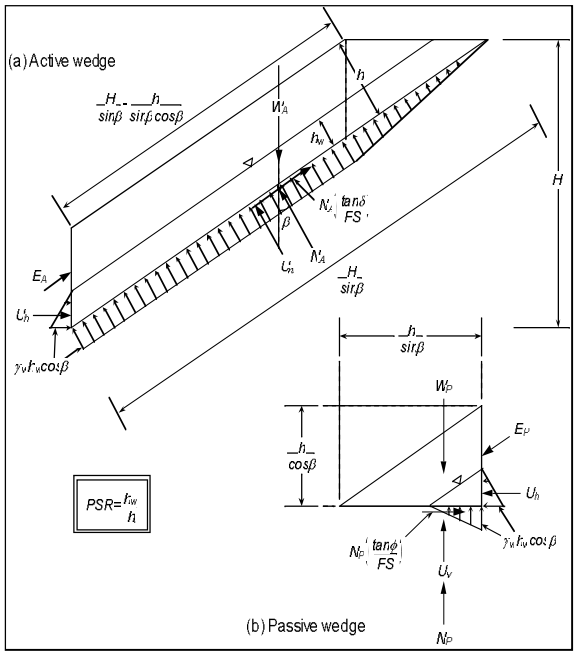
DLC	***1.000***
PSR	***0.011***
FS	***1.495***

DLC = 1.0003

q = 2.3E-05 m^3/sec

h_{avg} = 0.00 m

PSR = 0.011



thickness of cover soil = h = 0.46 m

length of slope measured along the top of clay layer = L = 235 m

soil slope angle along top of clay = β = 15.9 ° = 0.28 (rad.)

vertical height of the slope measured from the toe = H = 64.4 m

parallel submergence ratio = PSR = 0.01

depth of the water surface measured from the top of clay layer = h_w = 0.00 m

dry unit weight of the cover soil = γ_{moist} =	17.3	kN/m^3	
saturated unit weight of the cover soil = $\gamma_{sat'd}$ =	18.9	kN/m^3	
unit weight of water = γ_w =	9.81	kN/m^3	
friction angle of the cover soil = ϕ =	30.0	°	= 0.52 (rad.)
critical interface = δ =	23.1	°	= 0.40 (rad.)

Calculation of FS

Active Wedge:

W_A = 1873.47 kN

U_n = 11.0795 kN

U_h = 0.00012 kN

N_A = 1790.72 kN

Passive Wedge:

W_P = 7.01353 kN

U_V = 0.00043 kN

$$FS = \frac{-b + \sqrt{b^2 - 4ac}}{2a}$$

where a = 493.6

b = -818.8

c = 120.8

FS= 1.495

Note: numbers in boxes are input values

numbers in Italics are calculated values

Constructed by Te-Yang Soong

Figure 4

ATTACHMENT 1

RUNOFF COEFFICIENT REFERENCE

10.2.1 Thornthwaite method

The earliest, comprehensive method for water balance analysis was developed by Thornthwaite and Mather (1957). Fenn *et al.* (1975) developed for the US Environmental Protection Agency (EPA) a water balance method for predicting leachate generation for solid waste disposal sites based on the Thornthwaite method, proposing runoff coefficients for landfills and moisture storage values for municipal waste. The water balance method employs average monthly values of precipitation and other climatic parameters. To determine monthly infiltration (*IN*) into the cover, one subtracts monthly runoff (*R*) from monthly precipitation (*P*).

$$IN = P - R \tag{10.1}$$

Runoff can be calculated from precipitation, as follows

$$R = CP \tag{10.2}$$

where C is a runoff coefficient that can be estimated from the guidance provided by Fenn *et al.* (1975):

description of the grass-covered soil	slope of ground surface	runoff coefficient (C)
sandy soil	flat (<2%)	0.05–0.10
	mild (2 – 7%)	0.10–0.15
	steep (>7%)	0.15–0.20
clayey soil	flat (<2%)	0.13–0.17
	mild (2 – 7%)	0.18–0.22
	steep (>7%)	0.25–0.35

Potential evapotranspiration (*PET*), which depends on mean temperature, heat index, and hours of sunlight, can be calculated from tables provided by Thornthwaite and Mather (1957). The cumulative monthly infiltration minus potential evapotranspiration (*IN* – *PET*) is calculated. A negative number for *IN* – *PET* indicates that the cover has a tendency to dry out; a positive value indicates that there is a tendency for the cover soil to become wetter. If *IN* – *PET* is negative, water may evapotranspire from the cover soil, but if the soil is already dry, no further drying will occur. The amount of drying depends not only on *IN* – *PET* but also on the water content of the soil.

If *IN* – *PET* is positive, water may be stored in the cover soil (which would produce an increase in water content). However, if the water content is already very high, the soil can store no additional water and water will percolate downward through the cover soil. The field capacity of the soil is the maximum water content that a soil can attain without draining water by gravity. Field capacity can be determined by

REFERENCE: "GEOTECHNICAL PRACTICE FOR WASTE DISPOSAL", DANIEL, DAVID E., PG 216.

ATTACHMENT 2

TYPICAL GEOCOMPOSITE PROPERTIES

PRODUCT DATA SHEET

GSE FabriNet Geocomposite

GSE FabriNet geocomposite consists of a 200 mil thick GSE HyperNet geonet heat-laminated on one or both sides with a GSE nonwoven needle-punched geotextile. The geotextile is available in mass per unit area range of 6 oz/yd^2 to 16 oz/yd^2. The geocomposite is designed and formulated to perform drainage function under a range of anticipated site loads, gradients and boundary conditions.

AT THE CORE:

A 200 mil thick HyperNet geonet heat-laminated on one or both sides with a nonwoven needlepunched geotextile.

Product Specifications

Tested Property	Test Method	Frequency	Minimum Average Roll Value		
Geocomposite			6 oz/yd^2	8 oz/yd^2	10 oz/yd^2
Transmissivity[2], gal/min/ft, (m^2/sec) Double-Sided Composite Single-Sided Composite	ASTM D 4716	1/540,000 ft^2	 0.5 (1x10^{-4}) 4.8 (1x10^{-3})	 0.5 (1x10^{-4}) 4.8 (1x10^{-3})	 0.4 (9x10^{-5}) 4.3 (9x10^{-4})
Ply Adhesion, lb/in	ASTM D 7005	1/50,000 ft^2	1.0	1.0	1.0
Geonet Core[3] – GSE HyperNet					
Transmissivity[2], gal/min/ft (m^2/sec)	ASTM D 4716		9.6 (2 x 10^{-3})	9.6 (2 x 10^{-3})	9.6 (2 x 10^{-3})
Density, g/cm^3	ASTM D 1505	1/50,000 ft^2	0.94	0.94	0.94
Tensile Strength (MD), lb/in	ASTM D 5035/7179	1/50,000 ft^2	45	45	45
Carbon Black Content, %	ASTM D 1603[6]/4218	1/50,000 ft^2	2.0	2.0	2.0
Geotextile[3,4]					
Mass per Unit Area, oz/yd^2	ASTM D 5261	1/90,000 ft^2	6	8	10
Grab Tensile, lb	ASTM D 4632	1/90,000 ft^2	160	220	260
Puncture Strength, lb	ASTM D 4833	1/90,000 ft^2	90	120	165
AOS, US sieve[1], (mm)	ASTM D 4751	1/540,000 ft^2	70 (0.212)	80 (0.180)	100 (0.150)
Permittivity, sec^{-1}	ASTM D 4491	1/540,000 ft^2	1.5	1.3	1.0
Flow Rate, gpm/ft^2	ASTM D 4491	1/540,000 ft^2	110	95	75
UV Resistance, % retained	ASTM D 4355 (after 500 hours)	per formulation	70	70	70
NOMINAL ROLL DIMENSIONS					
Geonet Core Thickness, mil	ASTM D 5199	1/50,000 ft^2	200	200	200
Roll Width[5], ft			14.5	14.5	14.5
Roll Length[5], ft	Double-Sided Composite Single-Sided Composite		270 300	260 300	230 290
Roll Area, ft^2	Double-Sided Composite Single-Sided Composite		3,915 4,350	3,770 4,350	3,335 4,205

[Product specifications continued on back]

AT THE CORE:

A 200 mil thick GSE HyperNet geonet heat-laminated on one or both sides with a nonwoven needlepunched geotextile.

Product Specifications [continued]

NOTES:

- (1)AOS in mm is a maximum average roll value.
- (2)Gradient of 0.1, normal load of 10,000 psf, water at 70˚F between steel plates for 15 minutes. Contact GSE for performance transmissivity value for use in design.
- (3)Component properties prior to lamination.
- (4)Refer to geotextile product data sheet for additional specifications.
- (5)Roll widths and lengths have a tolerance of ±1%.
- (6)Modified.

GSE is a leading manufacturer and marketer of geosynthetic lining products and services. We've built a reputation of reliability through our dedication to providing consistency of product, price and protection to our global customers.

Our commitment to innovation, our focus on quality and our industry expertise allow us the flexibility to collaborate with our clients to develop a custom, purpose-fit solution.

GSE ENVIRONMENTAL™

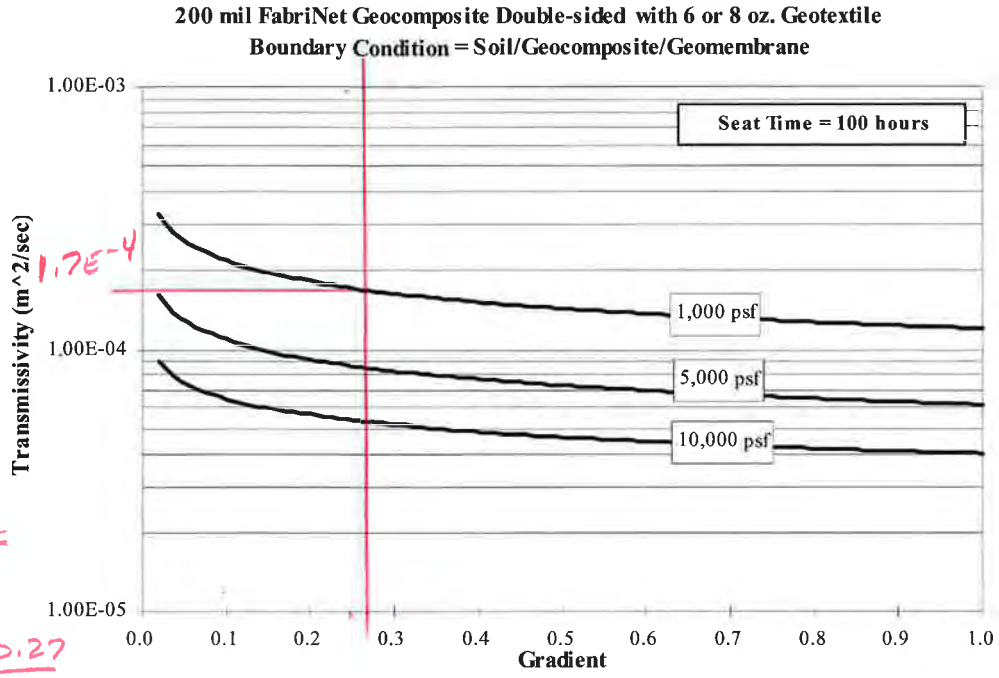
DURABILITY RUNS DEEP

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ATTACHMENT 3

Figure A-3, Performance Transmissivity of a 200 mil FabriNet Geocomposite under Soil



GRADIENT =
SIN B =
SIN (15.9) = 0.27

Figure A-3. Performance Transmissivity of a 200 mil FabriNet Geocomposite under Soil.

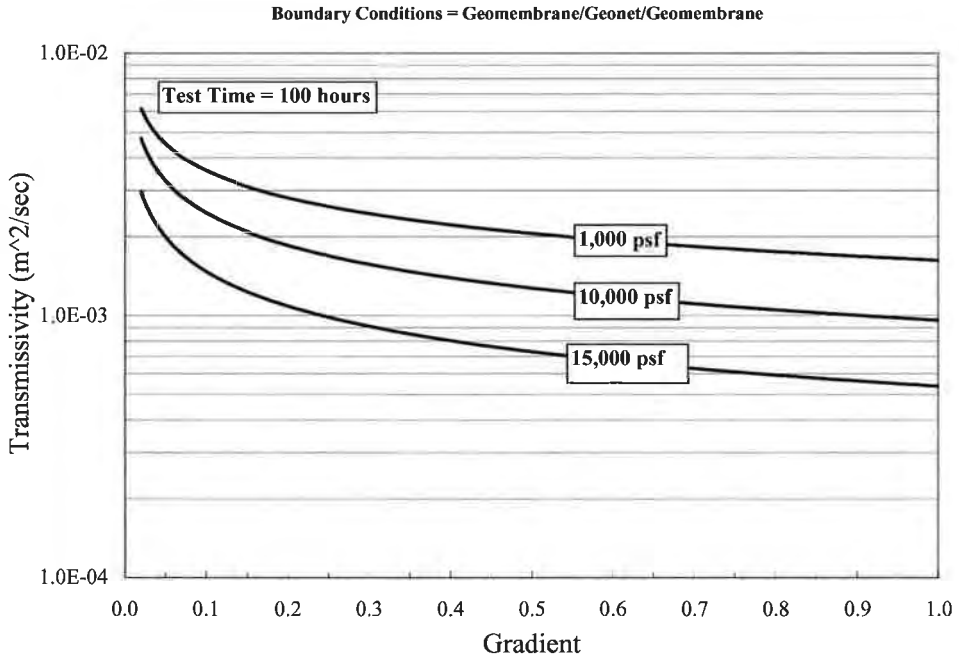


Figure A-4. Performance Transmissivity of a 250 mil GSE HyperNet HF geonet.

Appendix B.6
Geomembrane Compatibility

Technical Note

Chemical Resistance Chart

GSE is the world's leading supplier of high quality, polyethylene geomembranes. GSE polyethylene geomembranes are resistant to a great number and combinations of chemicals. Note that the effect of chemicals on any material is influenced by a number of variable factors such as temperature, concentration, exposed area and duration. Many tests have been performed that use geomembranes and certain specific chemical mixtures. Naturally, however, every mixture of chemicals cannot be tested for, and various criteria may be used to judge performance. Reported performance ratings may not apply to all applications of a given material in the same chemical. Therefore, these ratings are offered as a guide only. This information is provided for reference purposes only and is not intended as a warranty or guarantee. GSE assumes no liability in connection with the use of this information.

Medium	Concentration	Resistance at: 20 °C (68 °F)	60 °C (140 °F)
A			
Acetic acid	100%	S	L
Acetic acid	10%	S	S
Acetic acid anhydride	100%	S	L
Acetone	100%	L	L
Adipic acid	sat. sol.	S	S
Allyl alcohol	96%	S	S
Aluminum chloride	sat. sol.	S	S
Aluminum fluoride	sat. sol.	S	S
Aluminum sulfate	sat. sol.	S	S
Alum	sol.	S	S
Ammonia, aqueous	dil. sol.	S	S
Ammonia, gaseous dry	100%	S	S
Ammonia, liquid	100%	S	S
Ammonium chloride	sat. sol.	S	S
Ammonium fluoride	sol.	S	S
Ammonium nitrate	sat. sol.	S	S
Ammonium sulfate	sat. sol.	S	S
Ammonium sulfide	sol.	S	S
Amyl acetate	100%	S	L
Amyl alcohol	100%	S	L
Aniline	100%	S	L
Antimony trichloride	90%	S	S
Arsenic acid	sat. sol.	S	S
Aqua regia	HCI-HNO3	U	U
B			
Barium carbonate	sat. sol.	S	S
Barium chloride	sat. sol.	S	S
Barium hydroxide	sat. sol.	S	S
Barium sulfate	sat. sol.	S	S
Barium sulfide	sol.	S	S
Benzaldehyde	100%	S	L
Benzene	—	L	L
Benzoic acid	sat. sol.	S	S
Beer	—	S	S
Borax (sodium tetraborate)	sat. sol.	S	S
Boric acid	sat. sol.	S	S
Bromine, gaseous dry	100%	U	U
Bromine, liquid	100%	U	U
Butane, gaseous	100%	S	S
1-Butanol	100%	S	S
Butyric acid	100%	S	L
C			
Calcium carbonate	sat. sol.	S	S
Calcium chlorate	sat. sol.	S	S
Calcium chloride	sat. sol.	S	S
Calcium nitrate	sat. sol.	S	S
Calcium sulfate	sat. sol.	S	S
Calcium sulfide	dil. sol.	L	L
Carbon dioxide, gaseous dry	100%	S	S
Carbon disulfide	100%	L	U
Carbon monoxide	100%	S	S
Chloracetic acid	sol.	S	S
Carbon tetrachloride	100%	L	U
Chlorine, aqueous solution	sat. sol.	L	U
Chlorine, gaseous dry	100%	L	U
Chloroform	100%	U	U
Chromic acid	20%	S	L
Chromic acid	50%	S	L
Citric acid	sat. sol.	S	S

Medium	Concentration	Resistance at: 20 °C (68 °F)	60 °C (140 °F)
Copper chloride	sat. sol.	S	S
Copper nitrate	sat. sol.	S	S
Copper sulfate	sat. sol.	S	S
Cresylic acid	sat. sol.	L	—
Cyclohexanol	100%	S	S
Cyclohexanone	100%	S	L
D			
Decahydronaphthalene	100%	S	L
Dextrine	sol.	S	S
Diethyl ether	100%	L	—
Dioctylphthalate	100%	S	L
Dioxane	100%	S	S
E			
Ethanediol	100%	S	S
Ethanol	40%	S	L
Ethyl acetate	100%	S	U
Ethylene trichloride	100%	U	U
F			
Ferric chloride	sat. sol.	S	S
Ferric nitrate	sol.	S	S
Ferric sulfate	sat. sol.	S	S
Ferrous chloride	sat. sol.	S	S
Ferrous sulfate	sat. sol.	S	S
Fluorine, gaseous	100%	U	U
Fluorosilicic acid	40%	S	S
Formaldehyde	40%	S	S
Formic acid	50%	S	S
Formic acid	98-100%	S	S
Furfuryl alcohol	100%	S	L
G			
Gasoline	—	S	L
Glacial acetic acid	96%	S	L
Glucose	sat. sol.	S	S
Glycerine	100%	S	S
Glycol	sol.	S	S
H			
Heptane	100%	S	U
Hydrobromic acid	50%	S	S
Hydrobromic acid	100%	S	S
Hydrochloric acid	10%	S	S
Hydrochloric acid	35%	S	S
Hydrocyanic acid	10%	S	S
Hydrofluoric acid	4%	S	S
Hydrofluoric acid	60%	S	L
Hydrogen	100%	S	S
Hydrogen peroxide	30%	S	L
Hydrogen peroxide	90%	S	U
Hydrogen sulfide, gaseous	100%	S	S
L			
Lactic acid	100%	S	S
Lead acetate	sat. sol.	S	—
M			
Magnesium carbonate	sat. sol.	S	S
Magnesium chloride	sat. sol.	S	S
Magnesium hydroxide	sat. sol.	S	S
Magnesium nitrate	sat. sol.	S	S
Maleic acid	sat. sol.	S	S
Mercuric chloride	sat. sol.	S	S

- Continued -

Medium	Concentration	Resistance at: 20 °C (68 °F)	60 °C (140 °F)
Mercuric cyanide	sat. sol.	S	S
Mercuric nitrate	sol.	S	S
Mercury	100%	S	S
Methanol	100%	S	S
Methylene chloride	100%	L	—
Milk	—	S	S
Molasses	—	S	S
N			
Nickel chloride	sat. sol.	S	S
Nickel nitrate	sat. sol.	S	S
Nickel sulfate	sat. sol.	S	S
Nicotinic acid	dil. sol.	S	—
Nitric acid	25%	S	S
Nitric acid	50%	S	U
Nitric acid	75%	U	U
Nitric acid	100%	U	U
O			
Oils and Grease	—	S	L
Oleic acid	100%	S	L
Orthophosphoric acid	50%	S	S
Orthophosphoric acid	95%	S	L
Oxalic acid	sat. sol.	S	S
Oxygen	100%	S	L
Ozone	100%	L	U
P			
Petroleum (kerosene)	—	S	L
Phenol	sol.	S	S
Phosphorus trichloride	100%	S	L
Photographic developer	cust. conc.	S	S
Picric acid	sat. sol.	S	—
Potassium bicarbonate	sat. sol.	S	S
Potassium bisulfide	sol.	S	S
Potassium bromate	sat. sol.	S	S
Potassium bromide	sat. sol.	S	S
Potassium carbonate	sat. sol.	S	S
Potassium chlorate	sat. sol.	S	S
Potassium chloride	sat. sol.	S	S
Potassium chromate	sat. sol.	S	S
Potassium cyanide	sol.	S	S
Potassium dichromate	sat. sol.	S	S
Potassium ferricyanide	sat. sol.	S	S
Potassium ferrocyanide	sat. sol.	S	S
Potassium fluoride	sat. sol.	S	S
Potassium hydroxide	10%	S	S
Potassium hydroxide	sol.	S	S
Potassium hypochlorite	sol.	S	L
Potassium nitrate	sat. sol.	S	S
Potassium orthophosphate	sat. sol.	S	S
Potassium perchlorate	sat. sol.	S	S
Potassium permanganate	20%	S	S
Potassium persulfate	sat. sol.	S	S
Potassium sulfate	sat. sol.	S	S
Potassium sulfite	sol.	S	S
Propionic acid	50%	S	S
Propionic acid	100%	S	L
Pyridine	100%	S	L
Q			
Quinol (Hydroquinone)	sat. sol.	S	S
S			
Salicylic acid	sat. sol.	S	S
Silver acetate	sat. sol.	S	S
Silver cyanide	sat. sol.	S	S
Silver nitrate	sat. sol.	S	S
Sodium benzoate	sat. sol.	S	S
Sodium bicarbonate	sat. sol.	S	S
Sodium biphosphate	sat. sol.	S	S
Sodium bisulfite	sol.	S	S
Sodium bromide	sat. sol.	S	S
Sodium carbonate	sat. sol.	S	S
Sodium chlorate	sat. sol.	S	S
Sodium chloride	sat. sol.	S	S
Sodium cyanide	sat. sol.	S	S
Sodium ferricyanide	sat. sol.	S	S
Sodium ferrocyanide	sat. sol.	S	S
Sodium fluoride	sat. sol.	S	S
Sodium hydroxide	40%	S	S
Sodium hydroxide	sat. sol.	S	S
Sodium hypochlorite	15% active chlorine	S	S
Sodium nitrate	sat. sol.	S	S
Sodium nitrite	sat. sol.	S	S
Sodium orthophosphate	sat. sol.	S	S
Sodium sulfate	sat. sol.	S	S
Sodium sulfide	sat. sol.	S	S
Sulfur dioxide, dry	100%	S	S
Sulfur trioxide	100%	U	U
Sulfuric acid	10%	S	S
Sulfuric acid	50%	S	S
Sulfuric acid	98%	S	U
Sulfuric acid	fuming	U	U
Sulfurous acid	30%	S	S
T			
Tannic acid	sol.	S	S
Tartaric acid	sol.	S	S
Thionyl chloride	100%	L	U
Toluene	100%	L	U
Triethylamine	sol.	S	L
U			
Urea	sol.	S	S
Urine	—	S	S
W			
Water	—	S	S
Wine vinegar	—	S	S
Wines and liquors	—	S	S
X			
Xylenes	100%	L	U
Y			
Yeast	sol.	S	S
Z			
Zinc carbonate	sat. sol.	S	S
Zinc chloride	sat. sol.	S	S
Zinc (II) chloride	sat. sol.	S	S
Zinc (IV) chloride	sat. sol.	S	S
Zinc oxide	sat. sol.	S	S
Zinc sulfate	sat. sol.	S	S

Specific immersion testing should be undertaken to ascertain the suitability of chemicals not listed above with reference to special requirements.

NOTES:

(S) Satisfactory: *Liner material is resistant to the given reagent at the given concentration and temperature. No mechanical or chemical degradation is observed.*

(L) Limited Application Possible: *Liner material may reflect some attack. Factors such as concentration, pressure and temperature directly affect liner performance against the given media. Application, however, is possible under less severe conditions, e.g. lower concentration, secondary containment, additional liner protections, etc.*

(U) Unsatisfactory: ***Liner material is not resistant to the given reagent at the given concentration and temperature. Mechanical and/or chemical degradation is observed.***

(–) Not tested

sat. sol. = *Saturated aqueous solution, prepared at 20°C (68°F)*

sol. = *aqueous solution with concentration above 10% but below saturation level*

dil. sol. = *diluted aqueous solution with concentration below 10%*

cust. conc. = *customary service concentration*

TN032 ResistChart R03/17/06

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Marlex® POLYETHYLENE

Premium Extrusion and Rigid Packaging Resins

CPChem

ACOUSTICAL

Sound Velocity in Polyethylene - The sound velocity in Marlex HDPE is 2.21 x 10^3 m/sec.*

*Data from Naval Weapons Laboratory in Dahlgren, Virginia

CHEMICAL

Fungus Resistance

The fungus resistance of Marlex HDPE was determined using a test method described in Military Specification, MIL-E-5262A. In this procedure, tensile bars were sprayed with a suspension containing a specified mixture of five groups of fungi drawn from 16-day old cultures. The exposed tensile specimens were suspended over water in a vessel that was then sealed. To produce 95 ± 5% relative humidity in the chamber, the entire assembly was heated up to 86 ± 2°F (30° ± 1°C) and so maintained for a period of 28 days. After conditioning, the exposed samples were washed in alcohol, wiped dry and tested for tensile strength according to standard ASTM procedures. As a control, unexposed samples were conditioned and tested in the same manner.

Marlex HDPE showed no evidence of deterioration or corrosion due to fungus exposure.

TABLE 1

Fungus Resistance of Marlex High Density Polyethylene

	Control	Tensile Bars Sprayed with Fungus* Suspension
Tensile, psi (MPa)	4530 (31.2)	4530 (31.2)
Elongation, %	21	21

*Group I	Chaetonium globosum
Group II	Rhizopus higricans
Group III	Aspergillus flavus
Group IV	Penicillium funiculosum
Group V	Fusarium moniliforme

Chemical Resistance

Marlex polyethylenes are very resistant to chemical attack. However, they are sometimes affected by aggressive chemical environments. A few strong oxidizing agents and mineral acids can react with the resins. Other low molecular weight chemicals and oils absorb into and permeate through the resins. The absorption of liquid can affect appearance and dimensional accuracy of formed parts. When dispersed throughout the HDPE, a few liquids can act as an internal lubricant and reduce its toughness and mechanical strength. To determine the effects of most typical commercial and industrial chemicals, Marlex HDPE has been tested before and after immersion in a number of liquid chemicals selected to represent most of the types in common use. Both chemical reactivity and absorption depend heavily on temperature so the entire test procedure was conducted at room and elevated temperatures to quantify that dependency.

Effect of Chemicals on Tensile and Elongation - Tensile specimens as specified by ASTM D638 were immersed in various test liquids contained in glass jars. The jars were then sealed and brought to test temperatures in forced draft ovens held at 80°, 120° and 150°F (27, 49 and 66°C). After exposure for three months, the specimens were removed and tested on an Instron tensile testing machine. Unexposed samples were tested for control. A rapid strain rate of 20 inches (508 mm) per minute was employed to emphasize any decay of strength.

Most significant changes in mechanical strength were accompanied by some visible change in appearance. Oxidizing acids may not only cause a slight increase in weight but also cause color changes to occur. For example, concentrated sulfuric acid at 150°F (66°C) oxidized the samples so much that they turned black. Concentrated nitric acid caused yellowing.

To determine the extent of absorption, the change in weight was determined. A coupon measuring 1 inch x 2 inch (25.4 x 50.8 mm) was weighed and then immersed for three months at a temperature of 80°, 120° or 150°F (27, 49 or 66°C) in the test material. After exposure, the coupon was removed, wiped dry and weighed. Most of the organic chemicals that were used caused an increase in weight of the specimens due to a permeability of the chemical into the HDPE sample.

An examination of this data reflects the excellent chemical resistance of Marlex HDPE. Exposure periods longer than 3 months result in very little additional change in properties.

Marlex® POLYETHYLENE

Premium Extrusion and Rigid Packaging Resins

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TABLE 2

Ultimate Tensile Strength, Elongation and Absorption of Marlex® HDPE after Three Months

Chemical	Temp. °F	Temp. °C	Tensile Strength psi.	Tensile Strength MPa	Tensile Strength % Change	Elongation % Elong.	Elongation % Change	% Weight Change
Battery Acid (36%)	80	27	4480	30.9	-2.6	22	-4.3	-0.01
	120	49	4360	30.0	-5.2	27	+17.4	-0.01
	150	66	4670	32.2	+1.5	31	+34.8	-0.03
Nitric Acid (70%)	80	27	4670	32.2	+1.5	20	-13.0	+1.09
Hydrofluoric Acid	80	27	4460	30.7	-3.1	26	+13.0	+0.01
(52 – 55%)	120	49	4500	31.0	-2.1	28	+21.7	–
Phosphoric Acid (85%)	80	27	4580	31.6	-0.4	23	+0.0	+0.00
	120	49	4490	31.0	-2.3	30	+30.4	+0.00
	150	66	4680	32.2	-1.7	29	+26.1	-0.06
Chromic Acid (25%)	80	27	4610	31.8	+0.2	23	+0.0	-0.55
	120	49	4500	31.0	-2.2	28	+21.7	-0.07
	150	66	4300	29.6	-5.9	31	+34.8	-0.51
Hydrochloric Acid (37%)	80	27	4620	31.8	-0.4	22	-4.3	+0.04
	120	49	4370	30.1	-5.1	20	-13.0	+0.07
	150	66	4590	31.6	-0.2	30	+30.4	-0.03
Glacial Acetic Acid	80	27	4580	31.6	-0.4	26	+13.0	+0.81
	120	49	4440	30.6	-3.4	32	+39.1	+0.73
	150	66	4550	31.4	-1.0	33	+43.5	+0.39
NaOH (50%)	80	27	4590	31.6	-0.3	24	+4.3	+0.07
	120	49	4320	29.8	-0.6	24	+4.3	+0.09
	150	66	4650	32.0	-1.0	33	+43.5	-0.02
Nitrobenzene	80	27	4420	30.5	-3.9	30	+30.4	+2.25
	120	49	4070	28.0	-11.6	41	+78.3	+2.94
	150	66	4410	30.4	-4.1	40	+73.9	+3.47
Phenol	80	27	4590	31.6	-0.2	24	+4.3	+2.70
	120	49	4540	31.3	-1.3	28	+21.7	+0.41
	150	66	4730	32.6	+2.8	29	+26.1	+0.54
10 Wt. Motor Oil	80	27	4440	30.6	-3.5	28	+21.7	+1.87
	120	49	3770	26.0	-8.0	49	+113.0	+6.31
	150	66	4130	28.5	-10.1	53	+130.4	+6.86
Hydrogen Peroxide (30%)	80	27	4440	30.6	-3.4	25	+8.7	+0.01
	120	49	4410	30.4	-4.1	26	+13.0	-0.05
	150	66	4710	32.5	+0.5	32	+39.1	-0.03
Clorox	80	27	4550	31.4	-1.0	25	+1.1	+0.01
	120	49	4520	31.1	-1.8	25	+8.7	-0.06
	150	66	4580	31.5	-0.5	37	+60.9	-0.24
Methyl Salicylate	80	27	4260	29.4	-7.3	32	+39.1	+3.76
	120	49	4050	27.9	-12.0	42	+82.6	+4.92
	150	66	4110	28.3	-10.7	47	+104.3	+6.63
Ethylene Glycol	80	27	4520	31.1	-1.8	21	-4.3	+0.01
	120	49	4450	30.6	-3.3	27	+17.4	-0.06
	150	66	4710	32.5	-2.4	30	+30.4	-0.01
Tetralin	80	27	4020	27.7	-12.7	42	+82.6	+7.08
	120	49	3590	24.7	-22.0	53	+130.4	+7.08
	150	66	3620	24.9	-6.9	48	+108.7	+16.06
Benzaldehyde	80	27	4280	29.5	-7.0	27	+17.6	+2.08
	120	49	4130	28.5	-10.2	37	+61.0	+2.76
	150	66	4280	29.5	-6.9	37	+65.2	+16.06

TABLE 2 - continued

Ultimate Tensile Strength, Elongation and Absorption of Marlex® HDPE after Three Months

Chemical	Temp. °F	Temp. °C	Tensile Strength psi	Tensile Strength MPa	Tensile Strength % Change	Elongation % Elong.	Elongation % Change	% Weight Change
Acetophenone	80	27	4200	28.9	-8.7	33	+43.5	+2.13
	120	49	4110	28.3	-10.7	37	+61.0	+2.79
	150	66	4250	29.3	-7.5	37	+60.9	+3.90
Chlorobenzene	80	27	4280	29.5	-6.9	31	+34.8	+5.33
	120	49	3980	27.4	-13.5	43	+87.0	+4.91
	150	66	4150	28.6	-9.7	40	+73.9	+9.25
Butyl Carbitol	80	27	4390	30.2	-4.6	27	+17.6	+0.53
	120	49	4420	30.5	-3.8	29	+26.1	+1.38
	150	66	4150	28.6	-9.7	40	+73.9	+1.66
Toluene	80	27	4060	28.0	-11.8	36	+56.5	+4.40
	120	49	3950	27.2	-11.9	43	+87.0	+3.86
	150	66	4110	28.3	-2.0	43	+87.0	+5.58
Mineral Oil	80	27	4510	31.1	-2.0	29	+26.0	+0.82
	120	49	4200	29.0	-8.8	45	+95.7	+5.70
	150	66	4040	27.9	12.1	55	+139.1	+5.86
Aniline	80	27	4520	31.1	-1.7	24	+4.3	+0.76
	120	49	4300	29.6	-6.5	30	+30.4	+1.15
	150	66	4530	32.0	-1.6	37	+60.9	+1.45
Xylene	80	27	3980	27.4	-13.3	49	+113.0	+5.29
	120	49	3900	26.9	-15.2	44	+91.3	+5.02
	150	66	4040	27.9	-12.1	44	+91.3	+7.07
Diethylene Triamine	80	27	4440	30.6	-3.5	22	-4.3	+0.17
	120	49	4410	30.4	-4.1	32	+39.1	+0.39
	150	66	4610	31.8	-2.1	31	+34.6	+0.23
5% Soap Solution	80	27	4460	30.8	-3.0	30	+30.4	+0.06
	120	49	4620	31.9	+0.5	23	+0.0	+0.10
	150	66	4740	32.7	+3.0	30	+30.4	+0.04
Water	80	27	4530	31.2	-1.5	25	+8.7	+0.02
	120	49	4570	31.5	-0.7	28	+21.7	+0.02
	150	66	4620	31.8	+0.4	32	+39.1	+0.02
Air	80	27	4590	31.6	-0.3	23	+0.0	-0.03
	120	49	4430	30.5	-3.8	25	+8.7	+0.02
	150	66	4710	32.4	-2.3	27	+17.4	-0.02
Ethyl Alcohol	80	27	4580	31.6	-0.4	27	+17.4	+0.26
Amyl Acetate	80	27	4110	28.3	-10.6	47	+104.3	+3.31
Diethyl Ether	80	27	4200	29.0	-8.8	49	+113.0	+3.07
Turpentine	80	27	3970	27.4	-13.6	58	+152.2	+6.65
Tetrachloroethane	80	27	4110	28.3	-10.7	40	+74.0	+8.59
N-Heptane	80	27	4100	28.3	-10.8	45	+95.7	+4.99
Cyclohexane	80	27	3940	27.1	-14.3	45	+95.7	+6.52
Ethyl Acetate	80	27	4260	29.4	-7.4	45	+95.7	+2.23
Carbon Tetrachloride	80	27	4070	28.1	-11.5	34	+47.8	+14.00
Benzene	80	27	4300	29.6	-6.5	24	+4.3	+5.50
Bromine Water	80	27	4530	31.2	-1.6	24	+4.3	+3.75
NH_4OH (28%)	80	27	4640	32.0	+0.9	23	+0.0	+0.02

Additional information regarding the chemical resistance of Marlex polyethylene is presented in other PTC documents. This data is provided for use only as a guideline in the preliminary determination of packageability, because chemical compatibility is highly dependent on storage and use conditions. Furthermore, many products are combinations of chemicals, so the ultimate compatibility with the packaging material involves testing the combination of the product material and its proposed container.

Technical Note

GSE Polyethylene Geomembranes

Polyethylene has proved to be the most popular geomembrane lining material. This popularity is due to polyethylene's high UV and chemical resistance in addition to its flexibility. Through developments in resin technology, today's polyethylene geomembranes exhibit outstanding resistance to stress cracking and thermal aging.

Crude oil is the primary source for a wide range of intermediate organic products including gases and liquid mixtures. One of the products obtained from the refinement of crude oil is ethylene. The ethylene molecule is two carbon atoms bonded with a double bond and two hydrogen atoms attached to each carbon.

Ethylene molecules are able to participate in a chemical reaction called "polymerization". Polymerization is the process by which small molecules are combined to form large molecules called polymers. The polymerization of ethylene molecules occurs in a stepwise fashion. First, two ethylene molecules are bonded together, then another is added and so on until the reaction terminates. In this way, long molecules that fold, bend and intermingle are formed.

The reaction occurs in large reactors that can be pressurized and heated. A catalyst is typically required and a comonomer is often employed. Selection of pressure, temperature, catalyst and comonomer determine the particular grade of polyethylene that is produced. Pressure and elevated temperatures force the gaseous ethylene molecules together in close proximity. A comonomer can be used to further control the molecular structure of the finished product. A comonomer is commonly added in small amounts during polymerization to control or alter the molecular structure, in particular the branching, and performance of the polymer. With the great number of variations and combinations, there can be a great number of unique types of polyethylene materials.

The finished product is characterized primarily by: molecular weight, molecular weight distribution and degree of branching. Molecular weight is a quantitative measure of a single molecule's mass. Polymerization reactions do not result in molecules that all demonstrate the same molecular weight. As a result, the finished product contains a range of molecular weights; that is quantified by the molecular weight distribution. Since the molecules exist as discrete chains, the way those chains are ordered is important. The degree of crystallinity is closely related to density and has an effect on the material's stress crack resistance. Variations of these polymer characteristics may have a significant effect on both the processing characteristics and the life expectancy of the finished product.

There are two primary types of extrusion processes used to manufacture polyethylene into sheet goods, those with a round die and those with a flat die. Each method requires the polyethylene resin used to exhibit some range of properties. The world's polyethylene geomembrane manufacturers are charged with the responsibility of bridging the gap between resin processing characteristics and long term survivability as a containment liner.

Considerations for environmental projects include resistance to chemical, UV and thermal degradation. Polyethylene's stress crack resistance is not always a concern to resin suppliers since they supply so many different markets. This is in strong contrast to the environmental market where long term performance is of the utmost importance. As a result, polyethylene geomembrane manufacturers must work closely with their resin supplier(s) to achieve the longest lifespan of the material. Since the finished product is only as good as the raw material, special relationships between the resin supplier and the geomembrane manufacturer must be established and maintained.

TN010 PolyethyleneGeomem R03/17/06

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Appendix B.7
Depression Settlement Calculation

Client County of Kaua'i	**Subject** Cell 2 Expansion		
Project Kekaha Landfill	Depression Settlement	**Prepared By** JMT	**Date** 12/7/12
Cell 2 Lateral Expansion	Analysis	**Reviewed By** KJB	**Date** 12/7/12
60197394		**Approved By** KJB	**Date** 12/13/12

DEPRESSION SETTLEMENT ANALYSIS

Objective

On the Phase I sideslope and overlay, determine the minimum required runout length of the geomembrane base liner and liner system tensile strength to resist the effects of a depression forming within the Phase 1 waste mass which will support the liner system for Cell 2.

Design Criteria and Assumptions

1. Maximum allowable elongation at yield (strain) of HDPE geomembranes (textured and smooth) is 13%. Per Reference 1, GCLs can accommodate 6 to 20% strain.
2. All references to elevations correlate to the site-specific vertical datum.

Basis of Calculation

A landfill fire has been experienced at the site. As a result of this fire, a depression of approximately 100 feet by 100 feet and up to 5 feet deep, has been observed in the waste slopes. In the event that a fire occurs after the construction of the Cell 2 liner system, the liner will need to have the strength and flexibility to either span one of these depressions or have sufficient length to conform to the shape of the depression if slippage occurs.

The basis for our assumptions for the reinforcement and run out length of the liner is that any depressions caused by a "fire" would be at or near a source of oxygen. Given this we do know that the lower portion of the Phase 1 sideslopes are soil lined and do not have a geomembrane. The area in which Kekaha resides is semi-arid and the final cover soils are prone to cracking and can be the entry point for air into the waste mass creating an aerobic environment. The inherent design of the Cell 2 overlay is the installation of a full Subtitle D landfill liner for the entire slope of Phase 1 being overlain, all of the infiltration ditch, and along Phase 2. This thereby cuts off the source of air on this sideslope.

With the installation of the liner system the air source on Phase 1 along the Phase 2 side will be eliminated. The potential for air to enter from the sides (east and west ends), however, could potentially remain. Following the similar mindset as above, it is being proposed to extend the existing geomembrane of the Phase 1 final cover down the remainder of the Phase I sideslopes. The length of slope which would receive extended geomembrane would be for a minimum of 200 feet on each the west (Cell 2, Subcell A) and east (Cell 2, Subcell d) sideslopes of Phase 1.

The liner system design will also account for other potential oxygen sources. By looking at the geometry of the Phase 1 and the overlay system, the biggest potential for oxygen to enter the landfill would be the geomembrane of the Phase 1 cover (non-slope area). This would occur from liner penetrations (gas wells, etc.) which rely on geomembrane boots and an operating LFG system.

Based upon discussion above and the anticipated settlement from the Cell 2 waste mass, the following reinforcement and geotechnical characteristic for the overlay liner onto Phase 1 are proposed:

1. A designed slip surface of nonwoven geotextile to smooth geomembrane will be provided on the Phase 1 sideslope. This surface will reside in two places: 1) The bottom of the secondary geomembrane to the bottom cushion geotextile and, 2) the top of the primary geomembrane to the cushion geotextile of the drainage layer. The remaining interfaces of both the primary and

secondary geomembranes will be textured and are in direct contact with the GCL. The slip surface will be only on the Phase 1 overlay and 5 feet onto the floor of Cell 2. The Phase 2 side of the cell liner will have all texturing on all sides of the geomembrane to allow placement of the operations layers.

2. The run out length and liner tensile strength will be designed for a depression that starts at the crest of slope and propagates down the sideslope. The entire Phase 1 sideslope overlay will be reinforced for this scenario.

3. Reinforcement on the east and west ends will be designed to resist the effects of a mid-slope depression. This level of reinforcement will only be placed over the Subcells A and D overlay slopes.

Calculations

The maximum allowable elongation at yield (strain) of HDPE geomembranes (textured and smooth) is 13% and 6 to 20% strain for GCLs. The required geomembrane elongation was calculated by determining the existing slope length and the resulting slope length after depression settlement. A sensitivity analysis was performed to determine the amount of strain for varying diameters of depression with a depth of 5 feet. The initial length of the liner has been taken as the length of the depression; the deformed length of the geomembrane has been determined by modeling the depression as half of an ellipse. To determine the arch length of the depression a calculation for the approximate length of the ellipse was used. The geometry of the ellipse can be defined with the following parameters.

$$half\ length\ of\ major\ axis = a = varies$$

$$half\ length\ of\ minor\ axis = b = 5\ feet$$

The exact length of the perimeter of an ellipse cannot be calculated with a closed form solution. To simplify the calculation, the following approximation was used.

$$L_stretched = 1/2 * \pi * [3 * (a + b) - \sqrt{(3a + b)(a + 3b)}]$$

The elongation in the membrane was calculated as follows:

$$elongation = \frac{L_{stretched} - 2 * a}{2 * a} * 100$$

The maximum acceptable elongation in the liner system is 6% (controlled by the GCL). An iterative calculation was performed to determine the minimum depression size required to limit the strain. To limit the strain, the depression would need to be larger than 68 feet in diameter. For all depressions smaller than this, the strain required for the liner to conform to the depression will be larger than acceptable levels.

As the diameter of the depression increases, the tensile force applied to the liner also increases. At a certain point, the tensile force in the liner will exceed the pullout resistance along the liner and the liner will slide into the depression that forms. This sliding mechanism has intentionally been introduced into the liner system to limit the magnitude of the tensile force in the liner. As such, there will not be an

anchor trench for some components of the liner to allow them to slide into the depression. The pull out resistance of the liner has been calculated based on the friction between the bottom of the Cell 2 liner and Phase 1 plus the friction on the top and bottom of the liner in the run out length at the crest of the slope.

We will provide a minimum of 3 feet of cover over the top of the run out to protect the geomembrane from being damaged by traffic. The run out will occur below the access road. The required runout length is actually controlled by the smallest depression that we need to design for, which will occur at the crest of the slope. The force required to cause the membrane to slide will be controlled by the pullout resistance of the liner run out. The controlling interface friction angle will be 11 degrees between the smooth HDPE membrane and the nonwoven geotextile layers. The pullout resistance is simply the product of the overburden stress and the tangent of the interface friction angle. Assuming the fill which will provide cover for the membrane has an average unit weight of 125 pounds per cubic foot (pcf), the pullout resistance can be calculated as follows:

$$\sigma_v = 3\ ft * 125\ pcf$$

$$F = 2 * \sigma_v * L * \tan(11)$$

In the above equation, L is the run out length for the liner system. The runout length is a minimum of 3 feet to provide the overlap into the cover. An additional 5 feet of material is provided for the maximum amount of liner that will be needed to conform to a 5-foot deep depression. An additional 3 feet has been added for additional factor of safety to ensure the liner system will stay intact. Based on this the runout length for the calculations has been set to 11 feet. Using the above equations and a run out length of 11 feet, the pull out resistance of the liner is 1,600 pounds/foot. The factor of 2 in the above calculations accounts for friction on the top and bottom of the liner run out.

$$F = 2 * 375\ psf * 11ft * tan(11) = 1{,}600\ pounds/ft$$

This calculation is applicable to depressions which form at the top of the slope. For mid-slope depression, the overburden stress and friction which develops on the sideslope is also included in the pull out resistance of the liner.

The tension that is induced in the liner is a function of the overburden stress applied to the liner and the radius of the depression in plan dimensions. The tension can be calculated as follows:

$$T = p * r * \Omega$$

$$p = applied\ vertical\ pressure = 2 * \gamma * r * (1 - e^{-0.5 * \frac{H}{r}})$$

$$\gamma = unit\ weight\ of\ waste = 62\ pcf$$

$$r = radius\ of\ depression\ (ft)$$

$$H = thickness\ of\ waste\ above\ centerline\ of\ depression\ (ft)$$

$$\Omega = dimensionless\ strain\ factor = 0.90\ for\ 6\%\ strain$$

The applied vertical pressure and resulting tension were calculated for varying sizes of the depression. The change in vertical height of the waste was varied as the centerline of the depression progressed

down the slope. Radius increments of 1-foot were used for the calculation which is presented in the attachments. It was determined that at a radius of 8 feet, the tensile force induced in the membrane will exceed the pullout resistance in the run out and at that point, the liner will slip into the depression. In the attached table you can see that:

$$T = 1{,}840\ pounds\ per\ foot$$

We suggest a factor of safety of 1.3 is provided on this force resulting in a design tensile resistance of 2,400 pounds per foot over the full area of the liner.

Similar procedures can be followed for the design of the reinforcement for the mid-slope depression. Our calculations indicate the induced tension in the liner exceeds the pullout resistance when a mid-slope depression reaches a radius of 9 feet. The tension induced in the liner for a 9-foot radius mid-slope depression is 6,555 pounds per foot. Using a factor of safety of 1.3, the liner will be designed to resist a tensile force of 8,520 pounds per foot.

The proposed liner system, from the bottom up, will consist of the following:

- 16 oz/sq yd nonwoven cushion geotextile
- 60-mil HDPE secondary geomembrane
- Geosynthetic clay layer with needle punched nonwoven geotextile top and bottom
- 60-mil HDPE primary geomembrane
- 16 oz/sq yd cushion geotextile

From the information provided to us by Agru, the cushion geotextiles have a tensile strength of 2,100 pounds per lineal foot. Using information from GRI, 60-mil HDPE has a yield strength of 1,510 pounds per lineal foot. Tensile capacity in the geosynthetics clay liner was not accounted for which is a conservative assumption. From a strength standpoint, no additional reinforcement will be necessary across Subcells B and C. In Subcells A and D, an additional reinforcement with a minimum weak direction tensile strength of 1,300 pounds per foot will be required.

Conclusion

We recommend a horizontal liner runout of 11 feet is provided along the full length of Cell 2. No additional reinforcement is required for Subcells B and C. Additional reinforcement with a minimum tensile strength of 1,300 pounds per foot in the weak direction is required for Subcells A and D.

References

1. Qian, Xuede; Koerner, Robert M.; and Gray, Donald H., "Geotechnical Aspects of landfill Design and Construction," 2002, pages 560 to 573.

ATTACHMENT 1

TOP OF SLOPE AND MID-SLOPE DEPRESSION CALCULATION TABLES

AECOM

Analysis of Depression Settlement
Liner Tensile Requirements
Top of Slope Depression

Calculated by JMT
Checked by KJB
Approved by KJB

L	**78**	**ft**
H	**0**	**ft**
γ	**62**	**pcf**
pmid	**0**	**psf**
Ω	**0.9**	
Φ	**11**	**deg**

r	pave	Hcl	parch	Treq	N	F	
ft	**psf**	**ft**	**psf**	**lb/ft**	**lb**	**lb**	**Pull Out**
0.5	18.5	0.3	16.0	7.2	4125.0	1603.6	
1.0	37.0	0.6	32.0	28.8	4125.0	1603.6	
2.0	74.0	1.2	64.0	115.2	4125.0	1603.6	
3.0	111.0	1.8	96.0	259.1	4125.0	1603.6	
4.0	148.0	2.4	128.0	460.7	4125.0	1603.6	
5.0	185.0	3.0	160.0	719.8	4125.0	1603.6	
6.0	222.0	3.6	191.9	1036.5	4125.0	1603.6	
7.0	259.0	4.2	223.9	1410.8	4125.0	1603.6	
8.0	296.0	4.8	255.9	1842.6	4125.0	1603.6	Slip

AECOM

Analysis of Depression Settlement
Liner Tensile Requirements
Mid-Slope Depression

Calculated by JMT
Checked by KJB
Approved by KJB

L	**78**	**ft**
H	**46.5**	**ft**
γ	**62**	**pcf**
pmid	**1441.5**	**psf**
Omega	**0.9**	
Φ	**11**	**deg**

r	pave	Hcl	parch	Treq	N	F	
ft	**psf**	**ft**	**psf**	**lb/ft**	**lb**	**lb**	**Pull Out**
0.5	1441.5	23.3	62.0	27.9	31517.8	6928.2	
1.0	1441.5	23.3	124.0	111.6	34935.5	7592.6	
2.0	1441.5	23.3	247.3	445.1	33548.8	7323.0	
3.0	1441.5	23.3	364.3	983.6	32199.0	7060.7	
4.0	1441.5	23.3	468.9	1688.0	30886.3	6805.5	
5.0	1441.5	23.3	559.4	2517.2	29610.5	6557.5	
6.0	1441.5	23.3	636.8	3438.8	28371.8	6316.7	
7.0	1441.5	23.3	703.1	4429.4	27170.0	6083.1	
8.0	1441.5	23.3	760.0	5472.2	26005.3	5856.7	
9.0	1441.5	23.3	809.3	6555.4	24877.5	5637.5	Slip

Appendix B.8
Cell 2 Expansion Final Cover Settlement Analysis

Client County of Kaua'i	**Subject** Cell 2 Expansion		
Project Kekaha Landfill	Final Cover Settlement	**Prepared By** SRK	**Date** 12/5/12
Cell 2 Lateral Expansion	Analysis	**Reviewed By** JMT	**Date** 12/5/12
60197394		**Approved By** KJB	**Date** 12/13/12

FINAL COVER SETTLEMENT ANALYSIS

Objective

Estimate the settlement of the new waste in the proposed Cell 2 expansion. Estimate the strain in the new Cell 2 final cover liner that will result from this settlement.

Design Criteria and Assumptions

1. The unit weight of municipal solid waste placed in Cell 2 is 62 pcf assuming the use of daily cover soils.
2. For conservative analysis, the waste mass to be placed on the overlay area is assumed to be immediate and at full depth. Actual placement of the waste mass will occur in lifts over time which will allow the existing waste mass to accommodate the incremental loading resulting in lower estimated settlement.
3. For additional conservatism, the cross section of the overlay being evaluated is assumed to be all waste mass. Foundation soils are assumed to be under the portion of the expansion that is not over the overlay.
4. The Final Cover System consists of a 40-mil LLDPE geomembrane, a geocomposite drainage layer, 12 inches of rooting zone and 6 inches of topsoil.
5. All references to elevations correlate to the site specific vertical datum.

Methodology

Municipal solid waste is not a uniform material or product. As a result, differential settlement due to isolated focused settlement can occur. To account for this type of settlement a reinforcement calculation is performed to determine survivability of the geosynthetic components of the final cover liner system. Supporting calculations follow similar techniques presented in Reference 1.

Calculations

Two cross sections were analyzed for the primary and secondary settlement and elongation of the proposed Cell 2 expansion over the existing Phase I waste. Cross sections are shown in the attachments. Settlement was calculated for the final cover in the middle of the Phase I slope, points F1, F2, F3, and F4, and the midpoint between points F3 and F4 shown on the cross sections. In cross section A-A', the final cover elevations for the midpoint of point of the overlay and the final cover elevation of the sideslope is 50.4 and 26 feet, respectively. The resulting initial waste thickness, Ho, of the new waste at the midpoint of the overlay is 24.4 feet.

$$Ho = 50.4ft - 26.0ft = 24.4\ ft$$

The previously applied pressure in the waste layer, P_e, is assumed to equal the conservative compaction pressure, 1,000 pounds per cubic foot (pcf). Assuming the new waste has a unit

weight of 62 pounds per cubic foot (pcf), the resulting total overburden pressure, P_i, applied at the mid-level of the waste layer is 756 psf.

$$P_e = 1{,}000\ psf$$

$$P_i = \frac{1}{2} * H_0 * 62\ pcf = 756.4 psf$$

A logarithmic correlation between the existing overburden and increment of overburden was performed to determine the primary settlement. A modified primary compression index, C'c, of 0.22 was used and the resulting primary settlement is 0.7 feet in heave. The assumption was made that if the soil is going to swell, the maximum swell will occur immediately after compaction. Since we assume immediate loading at full depth, the assumption was made that if the previously applied pressure, P_e, is greater than the total overburden pressure, P_i no additional swelling will occur resulting in primary settlement equal to 0 feet. This results in a more conservative settlement.

$$S_{log} = \log\frac{P_i}{P_e} = -0.1$$

$$S_1 = 0.22 * H_o * S_{log} = -0.7\ ft \rightarrow say\ 0.0\ ft$$

Cell 2 expansion has 5 years' worth of airspace. The assumption was made that closure of the cell would take 1 year, and post closure monitoring would take place for 30 years. The average age of the waste mass in Cell 2 at the start of capping of Cell 2 is assumed to be 2 years.

$$\text{age}_{\text{existing}} = \frac{5\ year}{2} = 2.5\ years$$

$$\text{age}_{\text{sitelife}} = 5\ years + 1\ year + 30\ years = 36$$

Because secondary settlement is a function of time and impacted by the primary settlement, the secondary settlement was calculated using the logarithmic correlation between the site overlay life and monitoring period and the age of the existing life, the existing waste mass thickness after primary settlement, and a modified secondary compression index of 0.03. The secondary settlement was calculated to be 0.9 feet.

$$S_{log} = \log\frac{Age_{sitelife}}{Age_{existing}} = 1.16$$

$$H = H_o - S_1 = 24.2\ ft$$

$$S_2 = 0.03 * S_{log} * H = 0.9 ft$$

Consolidation testing performed on the foundation soils by Pacific Geotechnical Engineers, Inc. indicates that the elastic strain was approximately 0.018 in/in at a stress level of 5,000 psf.

Therefore, the modulus of the soil is approximately 280,000 psf for the range of stresses applicable.

The depth to bedrock is not known and is greater than 40 feet in depth from existing ground surface. The majority of elastic settlement is expected to occur within the upper 40 feet of the foundation soil. The elastic settlement is calculated by taking the total overburden pressure from the vertical expansion multiplied by the depth settlement is likely to occur and divided by the modulus of elasticity.

$$S_e = \frac{P_i}{E} * 40\ ft$$

$$S_e = \frac{756.4\ psf}{280{,}000\ psf} * 40\ ft = 0.1\ ft$$

A total settlement of 2.9 feet is calculated by adding the primary, secondary, and elastic settlements and the settlement of the Phase I sideslope, $S_{Phase\ I}$, due to the overlay of Cell 2. Phase I sideslope settlement calculations are included in the Cell 2 Expansion Phase I Sideslope Settlement Analysis, and ranged from 0.4 to 1.9 feet. It should be noted that the total settlement calculations for points F2, F3, F4, and the midpoint between points F3 and F4 do not include the settlement of the Phase I sideslope. It is assumed that the waste within Phase I ends at the toe of the sideslope. Foundation soils are assumed to be below the remaining Cell 2 expansion.

$$S = S_1 + S_2 + S_e + S_{Phase\ I} = 2.9\ ft$$

The geomembrane strain was calculated by determining the existing slope length and the resulting slope length after settlement. Using a 3.5H:1V slope, the distance between the midpoint of the slope to the toe of the slope was calculated to be approximately 48.5 feet. The resulting existing slope is 28.6%.

$$Slope_{existing} = \frac{50.43\ ft - 36.56\ ft}{48.5\ ft} * 100 = 28.6\%$$

The assumption was made that the toe of the Cell 2 slope (point F1) does not move or settle, and therefore the geomembrane does not move or settle at that point. The geomembrane does settle at the midpoint and the elevation is calculated using the total settlement.

$$elevation_{midpoint} = 50.4\ ft - S = 47.5\ ft$$

The resulting slope due to settlement is 22.6%.

$$Slope_{settled} = \frac{36.6\ ft - elevation_{midpoint}}{48.5\ ft} * 100 = -22.6\%$$

Client County of Kaua'i	**Subject** Cell 2 Expansion		
Project Kekaha Landfill	Final Cover Settlement	**Prepared By** SRK	**Date** 12/5/12
Cell 2 Lateral Expansion	Analysis	**Reviewed By** JMT	**Date** 12/5/12
60197394		**Approved By** KJB	**Date** 12/13/12

The geomembrane strain is 1.39% in compression and is calculated by correlating the existing slope length to the resulting slope length.

$$length_{existing} = \sqrt{48.5ft^2 + (50.43ft - 36.56ft)^2} = 50.5\ ft$$

$$length_{settled} = \sqrt{48.5ft^2 + (47.5ft - 36.6ft)^2} = 49.8\ ft$$

$$strain = \frac{49.8ft - 50.5\ ft}{50.5\ ft} * 100 = -1.39\%$$

Similar calculations can be applied to the remaining analysis points. Refer to Attachment 1 for the results of the primary, secondary, elastic, Phase I sideslope and total settlement, the resulting slope from settlement, and the geomembrane strain for the analysis points previously stated in cross sections A-A' and B-B'.

Conclusion

Settlement of the new waste is expected to occur. Total settlement ranges from 0 to 14 feet creating between 0.0 to 3.12 percent strain in the geo membrane between analysis points. All strain in the membrane is determined to be compression.

References

1. Qian, Xuede; Koerner, Robert M.; and Gray, Donald H., "Geotechnical Aspects of Landfill Design and Construction," 2002, pages 560 to 573.
2. Pacific Geotechnical Engineers, Inc. (2012.). *Kekaha Landfill Phase II Cell 2 Lateral Expansion Kekaha Sanitary Landfill Kekaha, Kauai, Hawaii*.
3. Drawings titled "Engineering Report, Phase II - Cell 2 Lateral Expansion, Kekaha Landfill," prepared by AECOM, dated August 2012.

ATTACHMENT 1

SETTLEMENT AND STRAIN ANAYLSIS FOR CROSS SECTIONS A-A' AND B-B'

Kekaha Landfill Cell 2 Expansion

Primary, Secondary, and Elastic Settlement of Existing Waste Mass

Unit Weight of Existing Waste, γ_{exist}: 72 pcf
Unit Weight of Overlay Waste, γ_{new}: 62 pcf

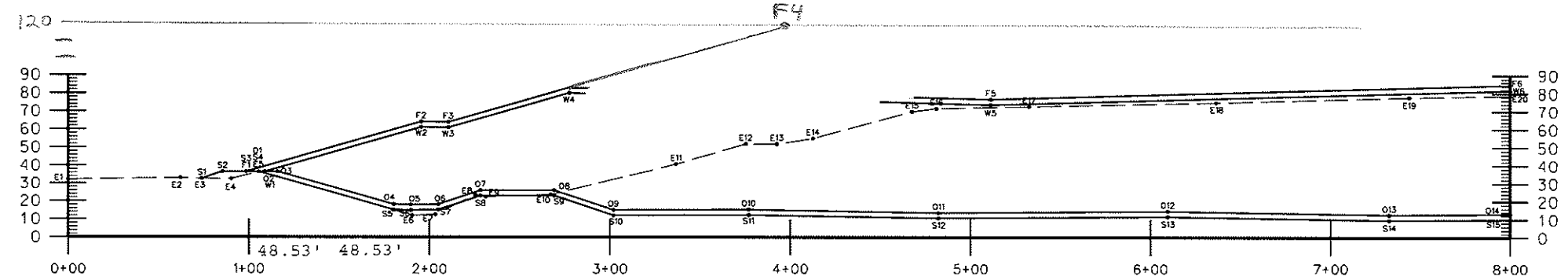
Cross Section	Corresponding Point on XS	Modified Primary Compression Index C'c	Top of Final Cover Elevation (ft)	Approximate Top of Base Cover or Phase II Waste Elevation (ft)	Initial Waste Thickness of New Waste H_o (ft)	Previously Applied Pressure in the Waste Layer (Assumed Equal to Compaction Pressure), P_e (psf)	Total Overburden Pressureat mid level of the Waste Layer, P_i (psf)	Primary Settlement (ft)	Modified Secondary Compression Index C'α	Starting Time of the Time Period for which Long-Term Settlement of the Layer is desired (yrs)	Ending Time of the Time Period for which Long-Term Settlement of the Layer is desired (yrs)	Existing Waste Mass Thickness After Primary Settlement (ft)	Secondary Settlement (ft)	Elastic Settlement (ft)	Phase I Sideslope, Phase II Toe Berm Sideslope, and/or Phase II Base Liner Settlement	Total Settlement (ft)
A	F4	0.22	120	23	97.0	1000	3007.0	**10.2**	0.03	2.5	36	86.8	**3.0**	**0.4**	**0.3**	**14.0**
A	Midpt F4 & F3	0.22	92	25	67.0	1000	2077.0	**4.7**	0.03	2.5	36	62.3	**2.2**	**0.3**	**3.7**	**10.9**
A	F3	0.22	64.0	21.0	43.0	1000	1333.0	**1.2**	0.03	2.5	36	41.8	**1.5**	**0.2**	**0.0**	**2.9**
A	F2	0.22	64.2	18.0	46.2	1000	1432.2	**1.6**	0.03	2.5	36	44.6	**1.6**	**0.2**	**0.0**	**3.4**
A	Midpt E5 & S5	0.22	50.4	26.0	24.4	1000	756.4	**0.0**	0.03	2.5	36	24.4	**0.8**	**0.1**	**1.9**	**2.8**
A	F1	0.22	36.6	36.6	0.0	1000	0.0	**0.0**	0.03	2.5	36	0.0	**0.0**	**0.0**	**0.0**	**0.0**
B	F4	0.22	84.9	22.0	62.9	1000	1949.9	**4.0**	0.03	2.5	36	58.9	**2.0**	**0.3**	**0.3**	**6.6**
B	Midpt F4 & F3	0.22	80.0	24.0	56.0	1000	1736.0	**3.0**	0.03	2.5	36	53.0	**1.8**	**0.2**	**0.0**	**5.0**
B	F3	0.22	75.1	24.0	51.1	1000	1584.1	**2.2**	0.03	2.5	36	48.9	**1.7**	**0.2**	**0.4**	**4.6**
B	F2	0.22	75.2	20.0	55.2	1000	1711.2	**2.8**	0.03	2.5	36	52.4	**1.8**	**0.2**	**0.4**	**5.3**
B	Midpt E5 & S5	0.22	52.5	27.9	24.6	1000	762.6	**0.0**	0.03	2.5	36	24.6	**0.9**	**0.1**	**1.9**	**2.9**
B	F1	0.22	38.5	38.5	0.0	1000	0.0	**0.0**	0.03	2.5	36	0.0	**0.0**	**0.0**	**0.0**	**0.0**

Slope Verification and Geomembrane Elongation

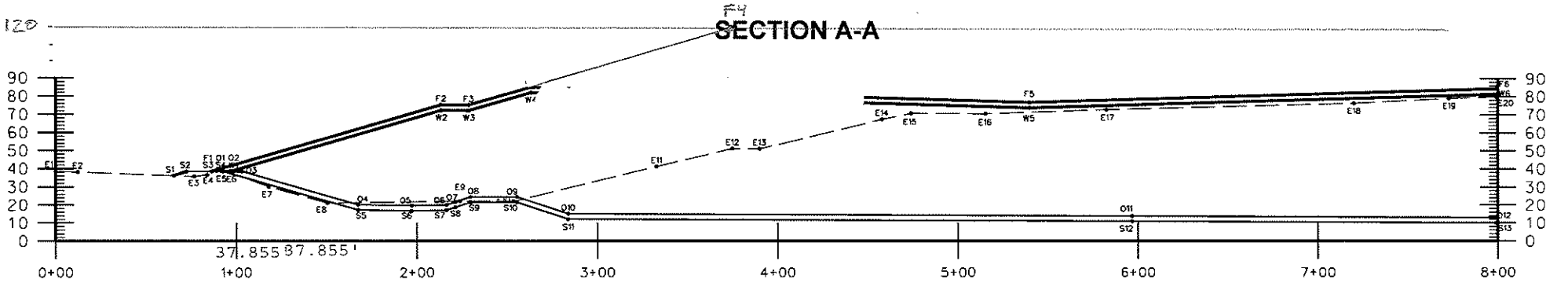
Cross Section	Corresponding Point on XS	Distance Between Points (Point and Next Point in Spreadsheet) (ft)	Top of Existing Cover Elevation (ft)	Existing Slope (%)	Settled New Waste Elevation (ft)	Resulting Slope from Settlement (%)	Existing Slope Length Between Points (ft)	Resulting Slope Length Between Points from Settlement (ft)	Geomembrane Strain (%)
A	F4	75.0	120.0	-37.3	106.0	**-33.2**	80.1	79	**-1.37 Compression**
A	Midpt F4 & F3	75.0	92.0	-37.3	81.1	**-26.7**	80.1	77.6	**-3.12 Compression**
A	F3	15.1	64.0	1.3	61.1	**-2.1**	15.1	15.1	**0.00 Compression**
A	F2	52.7	64.2	-26.2	60.8	**-25.1**	54.5	54.3	**-0.37 Compression**
A	Midpt E5 & S5	48.5	50.4	-24.5	47.6	**-18.7**	49.9	49.3	**-1.20 Compression**
A	F1	--	36.6		36.6				
B	F4	50.0	84.9	-9.8	78.3	**-6.6**	50.2	50.1	**-0.20 Compression**
B	Midpt F4 & F3	50.0	80.0	-9.8	75.0	**-8.9**	50.2	50.2	**0.00 Compression**
B	F3	15.3	75.1	0.7	70.5	**-4**	15.3	15.3	**0.00 Compression**
B	F2	83.8	75.2	-27.1	69.9	**-24.3**	86.8	86.2	**-0.69 Compression**
B	Midpt E5 & S5	63.6	52.5	-22	49.6	**-17.4**	65.1	64.6	**-0.77 Compression**
B	F1	--	38.5		38.5				

ATTACHMENT 2

CROSS SECTIONS A-A' AND B-B'



SECTION A-A



SECTION B-B

Appendix B.9
Phase 1 Final Cover Geomembrane Extension Slope Stability

Client	County of Kaua'i	**Subject**	Phase 1 Final	**Prepared By** KJB	**Date** 3/27/13
Project	Kekaha Landfill		Cover Geomembrane	**Reviewed By** JMT	**Date** 3/27/13
	Phase II Lateral Expansion		Extension Slope Stability	**Approved By** KJB	**Date** 3/27/13

PHASE 1 FINAL COVER GEOMEMBRANE EXTENSION SLOPE STABILITY

Objective

Evaluate the final cover slope stability under static and equipment loading conditions for the proposed Phase I final cover geomembrane extension at Kekaha Landfill that incorporates the use of geosynthetics and soil materials.

Design Criteria & Assumptions

1. An approximate 200-foot section of the Phase 1 sideslope adjacent to Subcells A and D of the proposed Cell 2 expansion are proposed to receive a geomembrane extension. The extension will be connected to the inplace geomembrane installed on the top portion of Phase 1 final cover which is estimated to extend 10 feet down the sideslope. The attached final cover cross section is applicable (Figure 1).
2. Interface friction angle testing should be completed prior to construction and use of the proposed materials. Material and interface friction angle testing requirements is presented in Appendix F.
3. A factor of safety of 1.5 will be used to determine slope stability under static conditions (consistent with past permitted approvals). A factor of safety of 1.1 is used for equipment loadings.
4. Slope length for cover soil placement is defined as the actual slope length, not a vertical or horizontal length.
5. Sideslopes are a maximum 3.5:1 (H:V) slope.
6. The dozer to be used for deployment of the drainage layer is expected to be a Caterpillar D6H LGP. This dozer has a ground pressure of less than 5 psi, typical for placement of soil material layers over geosynthetics. Model specifications are included as Attachment 1.
7. The dozer is operated from the bottom of the slope with no abrupt braking or change in direction during granular drainage layer placement.
8. Seismic analysis will not be completed as the site specific horizontal peak ground acceleration is 0.06 g and is less than 0.10 g required by Subtitle D for analysis (Reference 1).

Calculations

The stability of the final cover is primarily dependent upon the shear resistance of the weakest interface, slope length, and slope inclination. The following information was collected/selected before conducting the actual stability analysis.

Material/Design Properties

Topsoil:

Unit weight, moist	γ_{moist} = 110 pcf
Thickness (permitted)	$h_{c.s.}$ = 6 inches
Internal friction angle	$\phi_{c.s.}$ = 25 degrees (conservative)

Slope Inclination:

Slope inclination: β = 3.5H:1V = $\tan^{-1}$ (1/3.5) = 15.9 degrees

Maximum slope length (Reference 2):

Maximum slope length for the 3.5:1 Phase 1 cover slope receiving geomembrane extension is based upon longest change in elevation

$$(22\ ft)\left(\sqrt{3.5^2+1^2}\right) = 80.0 \text{ feet} \Rightarrow 100 \text{ feet (conservatism)}$$

Caterpillar D6H LGP Dozer Properties (Attachment 1, Reference 5)

Dozer track width:	36 inches
Track length on ground:	128.5 inches (10.7 ft)
Ground pressure:	4.88 psi (702.7 psf) < 5 psi

Infinite Slope Stability

The infinite slope stability is determined by force summation along the sideslope angle resulting in a factor of safety (Reference 4) without a buttress effect at the toe of slope or:

$$FS = \frac{\text{resisting forces}}{\text{driving forces}} = \frac{F}{W\sin\beta} = \frac{N\tan\delta}{W\sin\beta} = \frac{W\cos\beta\tan\delta}{W\sin\beta}$$

$$FS = \frac{\tan\delta}{\tan\beta}$$

where:

FS = Factor of Safety
δ = critical interface friction angle
β = sideslope angle

Solving for 3.5:1 slope (15.9 degrees) and Factor of Safety of 1.5, the minimum critical interface friction angle is:

$$FS = \frac{\tan\delta}{\tan\beta} \Rightarrow 1.5 = \frac{\tan\delta_{3.5:1}}{\tan 15.9} \Rightarrow \delta_{3.5:1} = \tan^{-1}(1.5 * \tan 15.9)$$

$$\delta_{3.5:1} = 23.1 \text{ degrees}$$

The infinite slope stability is not dependent upon cover soil thickness, slope length, or cover soil permeability.

Cover Soil Method of Analysis

Attached as Figure 2 is a calibrated spreadsheet depicting the analysis presented by Koerner and Soong, 1998 (Reference 3). This spreadsheet depicts the driving and resisting forces experienced along the critical interface with the incorporation of equipment loading. A factor of safety of 1.1 was deemed as a qualifier for slope stability as equipment loading is a brief and temporary load condition. Added conservatism is provided by not considering adhesion.

Conclusions

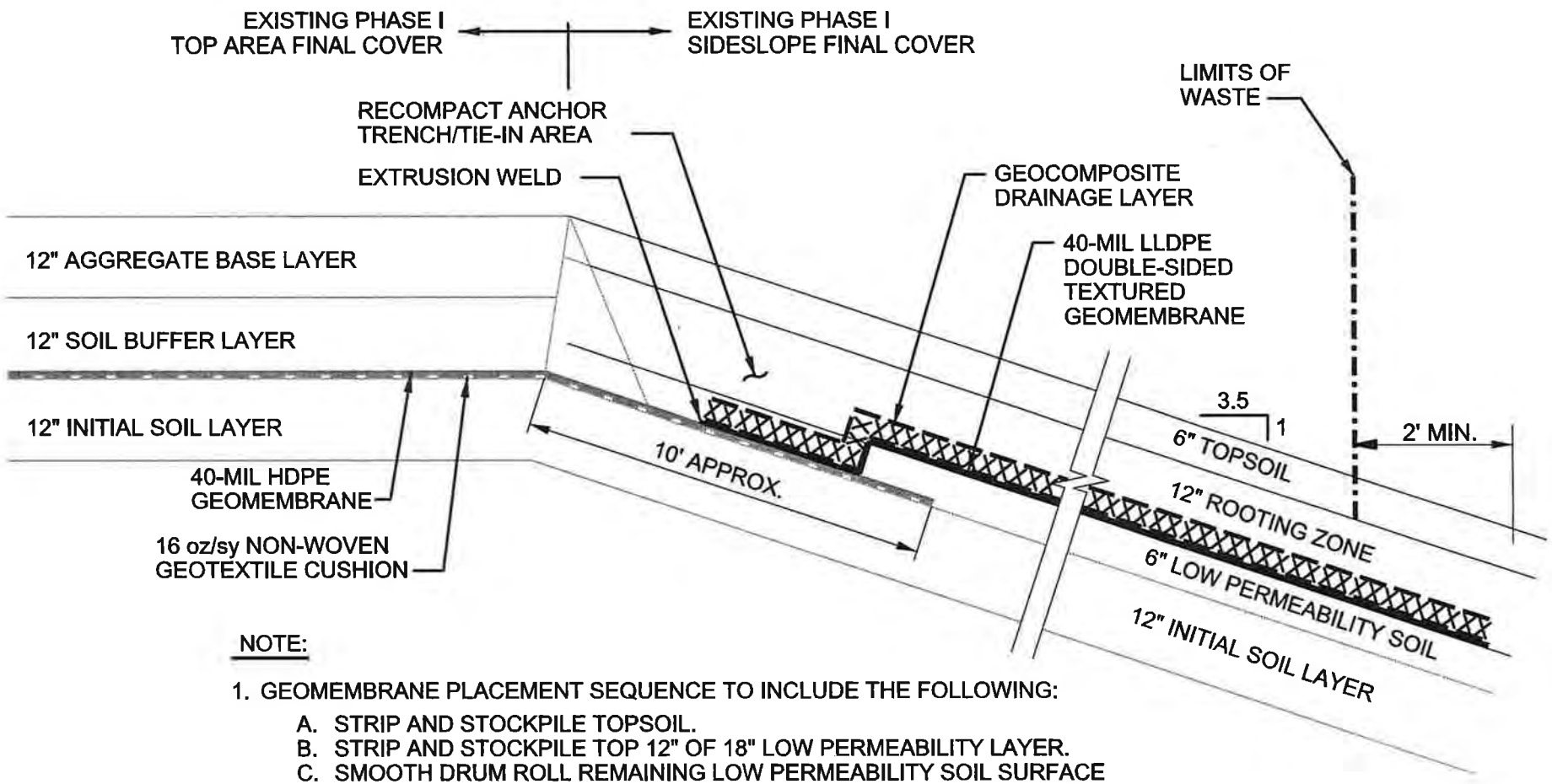
Based upon the infinite slope stability analysis, a minimum critical interface of 23.1 degrees is required for a slope with no buttress effect at the toe of slope and a static factor of safety of 1.5. Using the critical interface friction angle of 23.1 degrees and a Caterpillar D6H low ground pressure dozer for equipment loadings, placement of the 6-inch topsoil layer has resulting factor of safety of 1.54 > 1.1 for temporary loading conditions. Placement of the topsoil over the geomembrane extension and geotextile cushion layer is acceptable.

References

1. "Addendum Operations Manual For Kekaha Sanitary Landfill – Phase II," prepared by EMCON, May 1998.
2. Drawings titled, "Engineering Report, Phase II Cell 2 Lateral Expansion, Kekaha Landfill," prepared by AECOM, dated August 2012.
3. Koerner, R. M. and Soong, T.-Y. (1998), "Analysis and Design of Veneer Cover Soils" *Proc. 6th Int. Conf. on Geosynthetics, Atlanta*, USA, IFAI, pp. 1-23.
4. Koerner, R. M., "Designing with Geosynthetics," Fourth Edition, pps. 478 and 480.
5. "Caterpillar Performance Handbook," Edition 24, pages 1-8 and 1-21.

FIGURE 1

TYPICAL FINAL COVER LINER CROSS SECTION



NOTE:

1. GEOMEMBRANE PLACEMENT SEQUENCE TO INCLUDE THE FOLLOWING:
   - A. STRIP AND STOCKPILE TOPSOIL.
   - B. STRIP AND STOCKPILE TOP 12" OF 18" LOW PERMEABILITY LAYER.
   - C. SMOOTH DRUM ROLL REMAINING LOW PERMEABILITY SOIL SURFACE
   - D. EXCAVATE ANCHOR TRENCH TAKING CARE TO PROTECT EXISTING GEOMEMBRANE AND GEOTEXTILE.
   - E. REMOVE ROCKS AND OTHER DELETERIOUS MATERIAL LARGER THAN 1 INCH IN DIAMETER FROM EXISTING LOW PERMEABILITY SOIL SURFACE AND TOPSOIL.
   - F. PLACE GEOMEMBRANE AND WELD TO EXISTING GEOMEMBRANE.
   - G. PLACE NEW GEOCOMPOSITE DRAINAGE LAYER.
   - H. PLACE 12" ROOTING ZONE USING LOW GROUND PRESSURE EQUIPMENT.
   - I. REPLACE TOPSOIL USING LOW GROUND PRESSURE EQUIPMENT.
2. WHEN DEPLOYING GEOCOMPOSITE DRAINAGE LAYER OVER TEXTURED GEOMEMBRANE, A RUBSHEET SHALL BE USED BETWEEN THE TWO MATERIALS AND REMOVED AFTER FINAL MATERIAL POSITIONING.
3. LIMITS OF EXISTING GEOMEMBRANE TO BE FIELD LOCATED AND VERIFIED.
4. PLACEMENT SEQUENCE, LIMITS, AND CONFIGURATION MAY BE MODIFIED AT TIME OF CONSTRUCTION BASED ON FIELD CONDITIONS AND ENGINEER'S APPROVAL.

PHASE I GEOMEMBRANE EXTENSION 8

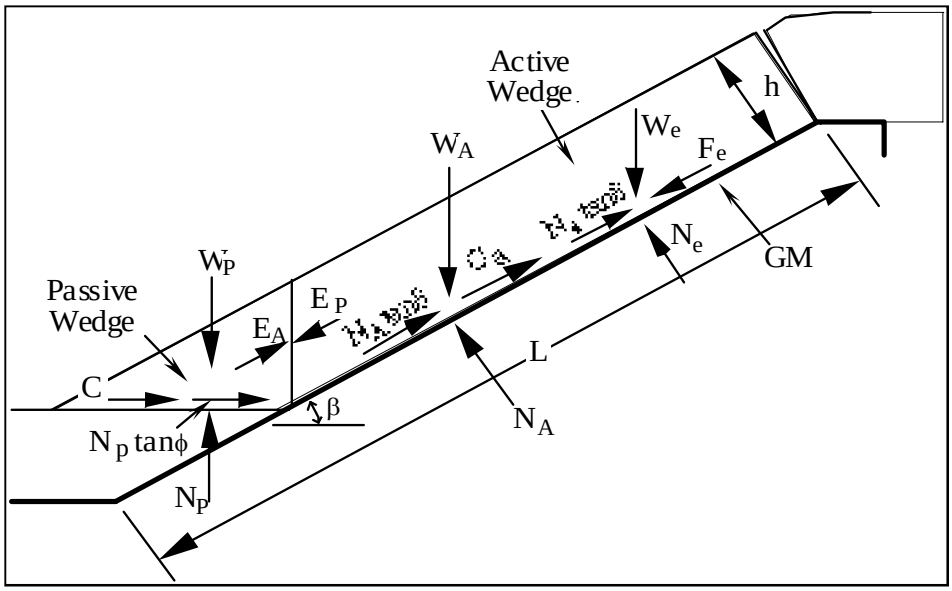
NTS

FIGURE 2

SLOPE STABILITY UNDER EQUIPMENT LOADINGS CONDITIONS FIGURES
3.5:1 SLOPE, 100’ SLOPE LENGTH, 6-INCH TOPSOIL COVER THICKNESS

Figure 2

Uniform Cover Soil Thickness with the Incorporation of Equipment Loads
0.5-foot Thickness (6" Topsoil Layer on Phase 1 Geomembrane Extension)



Calculation of FS

Active Wedge:

Wa= 15561.3 kpounds

Na= 14966.0 kpounds

Passive Wedge:

Wp= 469.7 kpounds

$$FS = \frac{-b + \sqrt{b^2 - 4ac}}{2a}$$

a= 6021.7

b= -10036

c= 1197.7

FS= 1.54

thickness of cover soil = h =	1.50	ft	
soil slope angle beneath the geomembrane = β =	15.9	°	*= 0.28 (rad.)*
finished cover soil slope angle = ω =	15.9	°	*= 0.28 (rad.)*
length of slope measured along the geomembrane = L =	100.0	ft	
unit weight of the cover soil = γ =	110.0	pcf	
friction angle of the cover soil = ϕ =	25.0	°	*= 0.44 (rad.)*
cohesion of the cover soil = c =	0.0	psf	*C= 0 kpounds*
interface friction angle between cover soil and geomembrane = δ =	23.1	°	*= 0.40 (rad.)*
adhesion between cover soil and geomembrane = ca =	0.0	psf	*Ca= 0 kpounds*

thickness of cover soil = h =	1.50	ft	*b/h= 2.0*
equipment ground pressure (= wt. of equipment/(2wb)) = q =	702.7	psf	*We=q w l= 7293.3*
length of each equipment track = w =	10.7	ft	*Ne=Wecos β = 7014.3*
width of each equipment track = b =	3.0	ft	*Fe=We (a/g) = 0.0*
influence factor* at geomembrane interface = I =	0.97		
acceleration/deceleration of the bulldozer = a =	0.00	g	

*Influence Factor Default Values

Cover Soil Thickness	Equipment Track Width		
	Very Wide	Wide	Standard
[2] 300 mm	1.00	0.97	0.94
300-1000 mm	0.97	0.92	0.70
[3] 1000 mm	0.95	0.75	0.30

Note: numbers in boxes are input values

numbers in Italics are calculated values

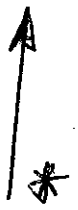
ATTACHMENT 1

EQUIPMENT PROPERTIES FOR CATERPILLAR D6H LGP DOZER



MODEL	D6H XR Series II		D6H LGP Series II		D7G		D7H Series II	
Flywheel Power	130 kW	175 hp	127 kW	170 hp	149 kW	200 hp	160 kW	215 hp
Operating Weight*								
(Power Shift)	18 684 kg	41,192 lb	19 814 kg	43,590 lb	20 666 kg	45,560 lb	24 694 kg	54,401 lb
(Direct Drive)	—		19 989 kg	43,976 lb	20 510 kg	45,218 lb	—	
(Power Shift Differential Steer)	18 799 kg	41,444 lb	—		—		24 993 kg	55,100 lb
Engine Model	3306		3306		3306		3306	
Rated Engine RPM	1900		1900		2000		2100	
No. of Cylinders	6		6		6		6	
Bore	121 mm	4.75"	121 mm	4.75"	121 mm	4.75"	121 mm	4.75"
Stroke	152 mm	6"	152 mm	6"	152 mm	6"	152 mm	6"
Displacement	10.5 L	638 in³	10.5 L	638 in³	10.5 L	638 in³	10.5 L	638 in³
Track Rollers (Each Side)	7		8		6		7	
Width of Standard Track Shoe	560 mm	22"	915 mm	36"	508 mm	20"	560 mm	22"
Length of Track on Ground	2.77 m	9'1"	3.27 m	10'8.5"	2.70 m	8'11"	2.90 m	9'6"
Ground Contact Area (W/Std. Shoe)	3.11 m²	4815 in²	5.97 m²	9254 in²	2.76 m²	4280 in²	3.24 m²	5016 in²
Track Gauge	1.88 m	6'2"	2.23 m	7'3"	1.98 m	6'5"	1.98 m	6'6"
GENERAL DIMENSIONS:								
Height (Stripped Top)**	2.26 m	7'5"	2.32 m	7'7"	2.27 m	7'5"	2.44 m	8'0"
Height (To Top of ROPS)	3.12 m	10'3"	3.16 m	10'5"	3.20 m	10'6"	3.33 m	10'11"
Height (To Top of Cab ROPS)	3.12 m	10'3"	3.16 m	10'5"	—		3.42	11'3"
Height (To Top of ROPS Canopy)	—		3.16 m	10'5"	—		—	
Overall Length (With P Blade)	—		5.18 m	17'0"	—		—	
(Without Blade)	—		4.49 m	14'9"	—		—	
Overall Length (With S Blade)	5.26 m	17'3"	—		5.28 m	17'4"	6.03 m	19'9"
(Without Blade)	4.21 m	13'10"	—		4.19 m	13'9"	4.73 m	15'6"
Width (Over Trunnion)	2.64 m	8'8"	3.43 m	11'3"	—		2.86 m	9'5"
Width (W/O Trunnion — Std. Shoe)	2.44 m	8'0"	3.14 m	10'3.6"	2.55 m	8'5"	2.54 m	8'4"
Ground Clearance	377 mm	14.8"	382 mm	15"	347 mm	13.7"	406 mm	16"
Blade Types and Widths:								
Straight	3.36 m	11'0"	3.99 m	13'1"	3.66 m	12'0"	3.91 m	12'10"
Angle	—		—		4.27 m	14'0"	—	
Angle Straight	4.16 m	13'8"	—		—		4.49 m	14'9"
Full Angle	3.78 m	12'5"	—		—		4.08 m	13'5"
Universal	—		—		3.81 m	12'6"	3.98 m	13'1"
Semi-U	3.26 m	10'8"	—		—		3.68 m	12'1"
Fuel Tank Refill Capacity	397 L	105 U.S. gal	337 L	89 U.S. gal	435 L	115 U.S. gal	479 L	127 U.S. gal

*Operating Weight includes ROPS canopy, operator, lubricants, coolant, full fuel tank, hydraulic controls and fluid, straight dozer with tilt, horn, back-up alarm, retrieval hitch and front pull hook.
— D7G includes end track guiding guards.
**Height (stripped top) — without ROPS canopy, exhaust, seat back or other easily removed encumbrances.



GROUND PRESSURES

Pressures computed from operating weights given earlier in this section in the specifications tables.

MODEL	SHOE WIDTH		CONTACT AREA		GROUND PRESSURE	
	mm	in	m^2	in^2	kPa	psi
D3C Series II ◄	356	14	1.35	2097	52	7.2
	406	16	1.54	2394	44	6.45
D3C LGP Series II	635	25	2.61	4045	29	4.2
D3C LGP-S Series II	990	39	4.85	7524	17	2.52
D4C Series II	406	16	1.67	2589	44	6.44
D4C LGP Series II	635	25	2.61	4045	30	4.3
D5C	457	18	1.92	3038	42.3	6.14
D5C LGP	660	26	2.834	4389	31.0	4.51
D4H Series II	360	14	1.59	2464	64	9.1
	410	16	1.82	2826	56	8.0
◄	460	18	2.05	3168	50	7.1
D4H XL Series III ◄	510	20	2.45	3799	47	6.73
	560	22	2.69	4172	44	6.17
D4H LGP Series III ◄	610	24	3.20	4953	38	5.34
	760	30	4.03	6252	31	4.39
	770	30	3.98	6170	31	4.41
D5E	406	16	1.77	2745	62	9.0
◄	457	18	1.99	3085	55	7.98
	508	20	2.21	3426	49.8	7.22
	560	22	2.44	3784	45	6.53
D5H Series II	460	18	2.11	3276	50.9	8.69
◄	510	20	2.35	3646	54.4	7.9
D5H XL Series II	560	22	2.78	4309	50	7.10
◄	600	24	3.63	4689	47	6.67
D5H LGP Series II	710	28	4.43	6866	36	5.15
◄	860	34	5.37	8321	30	4.24
	865	34	5.40	8369	29	4.17
D6D	457	18	2.17	3364	65	9.42
◄	508	20	2.41	3736	59	8.56
	560	22	2.65	4108	53	7.68
	610	24	2.89	4480	49	7.10
D6E	457	18	2.43	3766	60	8.70
◄	508	20	2.71	4200	54	7.83
	560	22	2.98	4619	49	7.10
	610	24	3.25	5040	45	6.54
D6H Series II	508	20	2.67	4140	65.0	9.44
◄	560	22	2.94	4564	59.0	8.56
	610	24	3.21	4971	54.2	7.86
D6H XL Series II	510	20	2.89	4484	66	9.42
◄	560	22	3.18	4923	60	8.49
	610	24	3.46	5363	55	7.88
D6H XR Series II	508	20	2.83	4386	67	9.48
◄	560	22	3.11	4815	60	8.55
	610	24	3.38	5246	56	7.93
D6H LGP Series II ◄	760	30	4.96	7296	40	5.74
	915	36	5.97	9261	34	4.88
	1000	39	6.53	10,122	32	4.48
D7G ◄	508	20	2.76	4280	73	10.6
◄	559	22	3.04	4708	66	9.6
	610	24	3.31	5136	60	8.8
D7H Series II	510	20	2.94	4560	82	11.7
◄	560	22	3.24	5016	75	10.6
	610	24	3.53	5472	69	9.8
	660	26	3.82	5928	64	9.1
D7H XR Series II	560	22	3.43	5315	71.5	10.16
◄	610	24	3.75	5808	65.9	9.37
	660	26	4.06	6282	61.2	8.70
D7H LGP Series II	760	30	4.8	7504	54	7.74
	915	36	5.82	9029	46	6.55
D8N ◄	560	22	3.59	5565	100.6	14.6
	610	24	3.91	6062	92.3	13.4
	660	26	4.23	6559	85.4	12.4
	710	28	4.55	7056	79.2	11.5
D8N LGP*	965	38	6.2	9576	53.7	7.8
D9N	560	22	3.86	6009	107.0	15.51
◄	610	24	4.21	6555	99.1	14.37
	685	27	4.74	7374	89.2	12.94
	760	30	5.26	8194	79.4	11.51
D10N ◄	610	24	4.73	7326	124.1	18.0
	710	28	5.50	8527	107.8	15.6
	860	31.5	6.66	10,328	89.8	13.0
D11N ◄	710	28	6.31	9781	150.2	21.8
	810	32	7.20	11,159	132.7	19.3
	915	36	8.13	12,605	118.6	17.2

◄Standard shoe.

*Offered as a Custom Product.

NOTE: Ground contact area = width of track shoe x length of track on ground x 2.

$$\text{Ground pressure} = \frac{\text{operating weight}}{\text{ground contact area}}$$

Appendix C
Surface Water Management Calculations

Appendix C
Surface Water Management Calculations

Appendix C.1
Surface Water Runoff Parameters and Design Storm Requirements

Appendix C.2
Surface Water Infiltration Basin Analysis

Appendix C.3
Surface Water Diversion Berm Analysis

Appendix C.4
Surface Water Downdrain Analysis

Appendix C.5
Surface Water Culvert Analysis

Appendix C.1
Surface Water Runoff Parameters and Design Storm Requirements

Client	County of Kaua'i	**Subject**	Surface Water	**Prepared By**	NKW	**Date**	8/9/12
Project	Kekaha Landfill - ER		Runoff Parameters and	**Reviewed By**	TCR	**Date**	8/9/12
Phase II, Cell 2 Lateral Exp.			Design Storm Requirements	**Approved By**	KJB	**Date**	8/10/12

SURFACE WATER RUNOFF PARAMETERS AND DESIGN STORM REQUIREMENTS

Objective

Determine rainfall value and curve number to be used as input in SEDCAD+. The curve number is used to determine how much precipitation becomes runoff. The curve number is primarily a function of soil type, cover type and treatment, and hydrologic condition. SEDCAD+ uses SCS methods to compute runoff volume.

Design Criteria and Assumptions

1. SEDCAD+, a computer program, was used to design and evaluate the surface water management system for the Kekaha Landfill Expansion. SEDCAD+ is a program developed at the University of Kentucky to assist in the design and evaluation of surface water management system components.

   The program is capable of computing peak flow, calculating runoff volume, sizing channels and culverts, and computing the hydraulic performance of sedimentation basins. SEDCAD+ utilizes the following methods within the program:

   - SCS Upland Curve Method for calculating time of concentration.
   - Muskingum's Method for routing between structures.
   - SCS Technical Release - 55 (TR-55) parameter for runoff volume.

   Within this appendix are calculations for each element of the surface water management system along with SEDCAD+ program input/output.

2. There are no surface water discharges from the Kekaha Landfill site. The Kekaha Landfill property is located on soils with high infiltration rates. Testing at the site revealed a minimum percolation rate of 1 inch per 6 minutes. The current surface water management system design is for surface water runoff to be routed to infiltration ditches located around the perimeter of the permitted landfill and to the infiltration basin. Any collected surface water percolates into the ground within the ditches and basin.

   This submittal proposes to route surface water to the existing infiltration ditches and to the northern corner of the site to the infiltration basin. The water is detained until it percolates into the ground.

3. Evaluate the following storm events for surface water management structures and infiltration basin:

   a. 100-year, 1-hour storm event.
   b. 25-year, 24-hour storm event.

4. Determine peak rainfall values based on the attached Design Plates from the County of Kauai, Department of Public Works Stormwater Runoff System Manual.

5. Use SCS storm distribution Type I, per attached Figure 5 of 8.

6. Determine curve number, assuming final cover conditions are pasture, grassland, or range, fair condition, and hydrologic soil group C. Determine curve number from tables provided in SCS TR-55. See attached table for curve number determination.

7. All references to elevations correlate to the site-specific vertical datum.

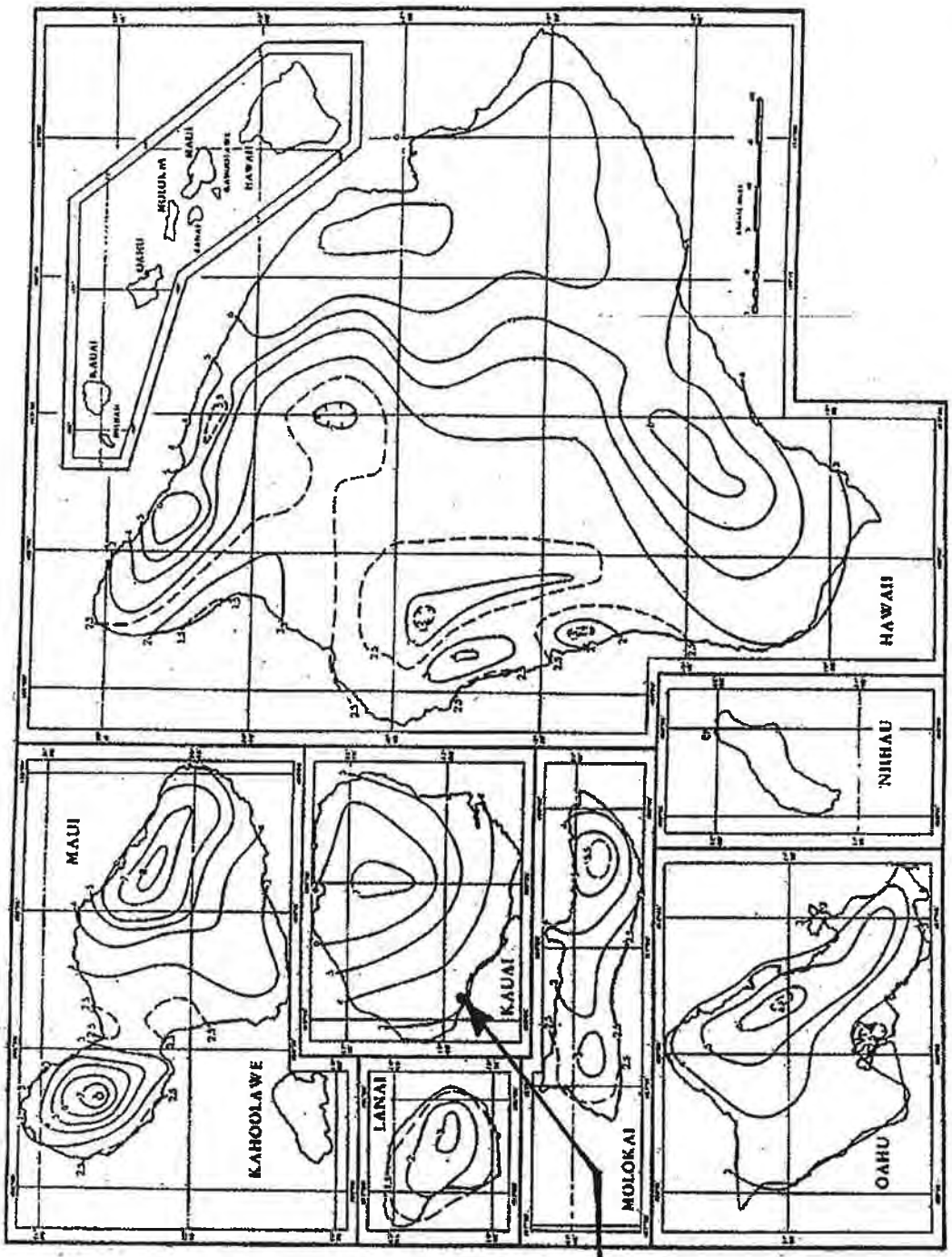
Client	County of Kaua'i	**Subject**	Surface Water	**Prepared By**	NKW	**Date**	8/9/12
Project	Kekaha Landfill - ER		Runoff Parameters and	**Reviewed By**	TCR	**Date**	8/9/12
Phase II, Cell 2 Lateral Exp.			Design Storm Requirements	**Approved By**	KJB	**Date**	8/10/12

Conclusions

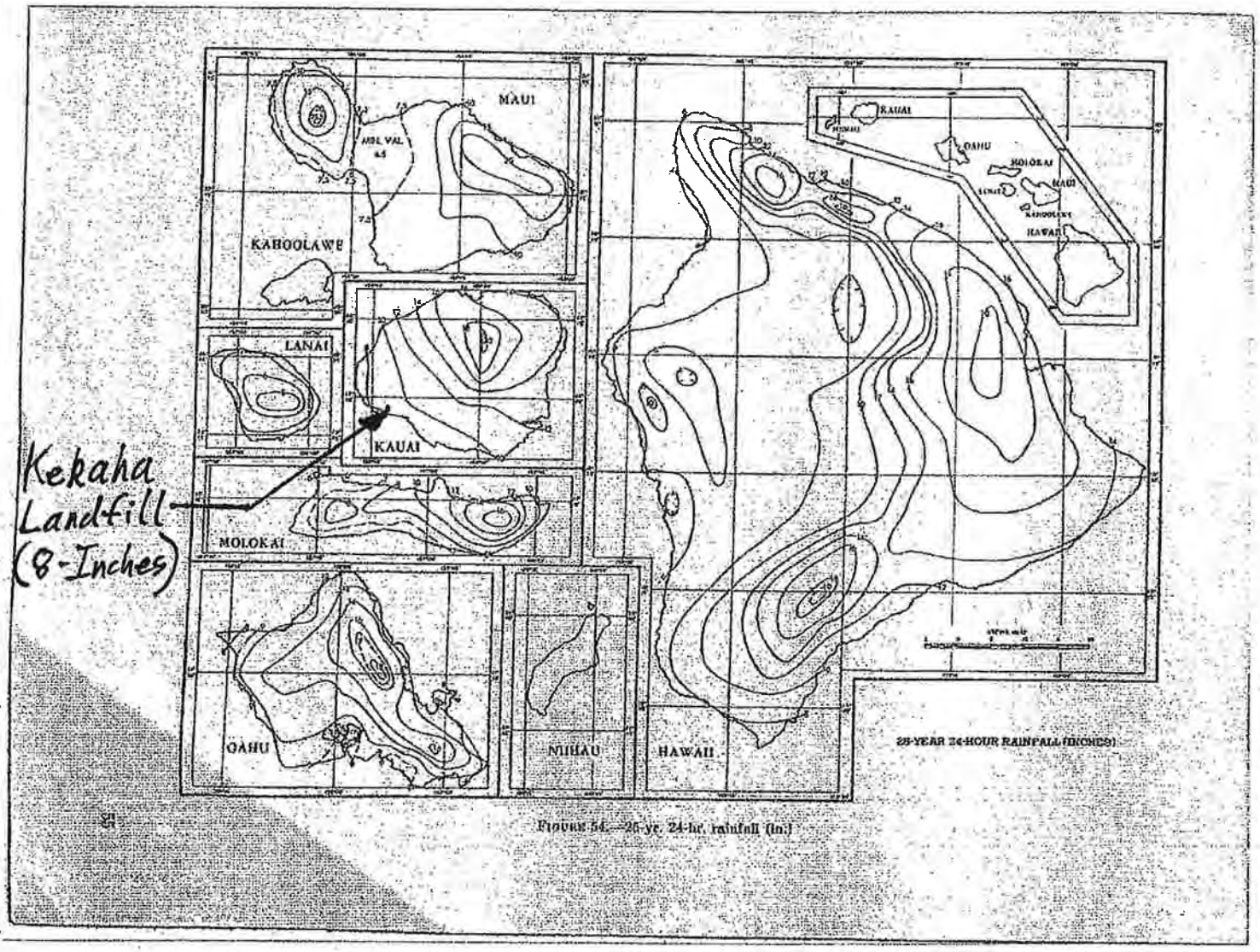
1. A 100-year, 1-hour storm event yields 3.0 inches of rainfall.
2. A 25-year, 24-hour storm event yields 8 inches of rainfall.
3. The curve number is 79 for the landfill final cover condition.

3/8

PLATE 4



100-Year 1-Hour Rainfall (in.)



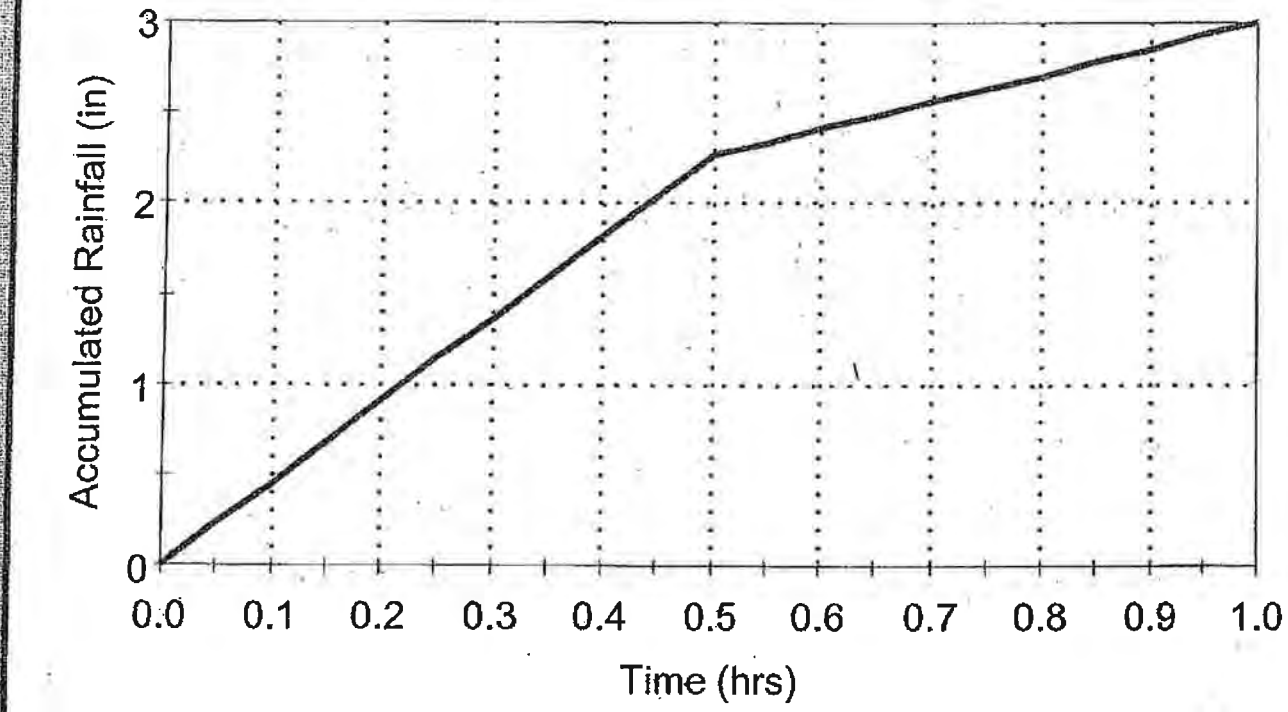
Kekaha Landfill (8-Inches)

Figure 54.—25-yr. 24-hr. rainfall (in.)

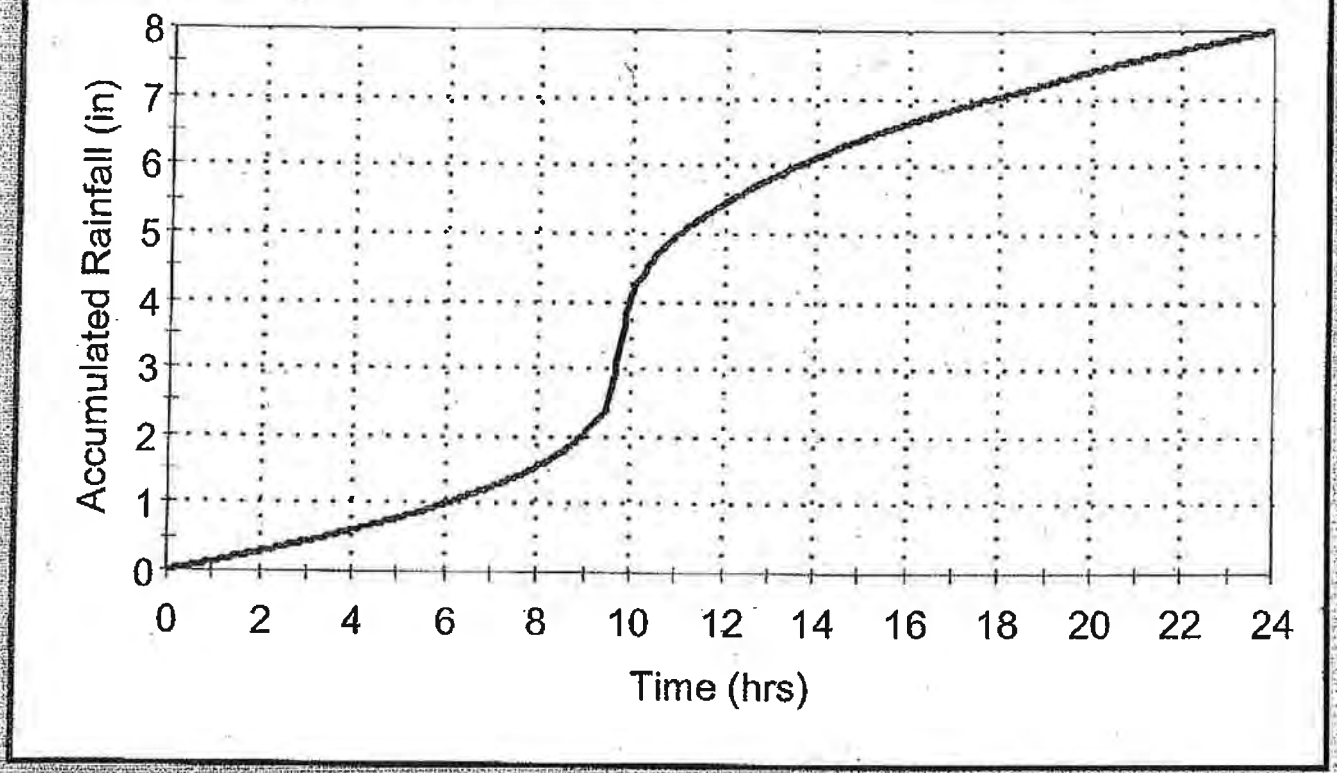
5/8

Figure B-2.—Approximate geographic boundaries for SCS rainfall distributions.

NRCS Type I, 100 yr - 1 hr Storm



NRCS Type I, 25 yr - 24 hr Storm



8/8

Table 2-2c.—Runoff curve numbers for other agricultural lands[1]

Cover description		Curve numbers for hydrologic soil group—			
Cover type	Hydrologic condition	A	B	C	D
Pasture, grassland, or range—continuous forage for grazing.[2]	Poor	68	79	86	89
	Fair	49	69	79	84
	Good	39	61	74	80
Meadow—continuous grass, protected from grazing and generally mowed for hay.	—	30	58	71	78
Brush—brush-weed-grass mixture with brush the major element.[3]	Poor	48	67	77	83
	Fair	35	56	70	77
	Good	430	48	65	73
Woods—grass combination (orchard or tree farm).[5]	Poor	57	73	82	86
	Fair	43	65	76	82
	Good	32	58	72	79
Woods.[6]	Poor	45	66	77	83
	Fair	36	60	73	79
	Good	430	55	70	77
Farmsteads—buildings, lanes, driveways, and surrounding lots.	—	59	74	82	86

[1]Average runoff condition, and I_a = 0.2S.

[2]*Poor:* <50% ground cover or heavily grazed with no mulch.
Fair: 50 to 75% ground cover and not heavily grazed.
Good: >75% ground cover and lightly or only occasionally grazed.

[3]*Poor:* <50% ground cover.
Fair: 50 to 75% ground cover.
Good: >75% ground cover.

[4]Actual curve number is less than 30; use CN = 30 for runoff computations.

[5]CN's shown were computed for areas with 50% woods and 50% grass (pasture) cover. Other combinations of conditions may be computed from the CN's for woods and pasture.

[6]*Poor:* Forest litter, small trees, and brush are destroyed by heavy grazing or regular burning.
Fair: Woods are grazed but not burned, and some forest litter covers the soil.
Good: Woods are protected from grazing, and litter and brush adequately cover the soil.

Appendix C.2
Surface Water Infiltration Basin Analysis

CALCULATION SHEET

Project No. 60197394

Client	County of Kaua'i	**Subject**	Surface Water	**Prepared By**	NKW	**Date**	8/26/15
Project	Kekaha Landfill - ER		Infiltration Basin Analysis	**Reviewed By**	DFP	**Date**	12/15/15
Phase II, Cell 2 Lateral Exp.				**Approved By**	NKW	**Date**	12/15/15

SURFACE WATER INFILTRATION BASIN ANALYSIS

Objective

Verify the design of the infiltration basin and infiltration ditches for the expansion at Kekaha Landfill.

Design Criteria and Assumptions

1. Existing perimeter infiltration ditches are used in combination with the existing infiltration basin located near the site entrance.

2. Analyze the infiltration basin and ditch for final cover conditions for the elevation 120' proposed permit condition. Assume entire landfill has final cover. The basin and ditch system is designed to comply with the requirements of the County of Kauai Department of Public Works Stormwater Runoff System Manual and the Hawaii Administrative Rules and Federal Regulations.

3. The basin and ditch system is designed to detain the design storm events. The basin and ditch will empty by infiltration within one day of the storm event.

4. There are no discharge structures in the basin. The SEDCAD+ surface water model shows user-defined discharge which increases as the depth of water in the basin and channel increases. This discharge is based upon measured percolation rates for the soils on-site.

5. Figure 1 shows the surface water management system network with subbasin drainage areas and structure labels.

6. All references to elevations correlate to the site-specific vertical datum.

Calculations

1. SEDCAD+ was used to determine the stage-storage-discharge relationships for the basin. SEDCAD+ output for the basin is attached. The basin was analyzed for the 100-year, 1-hour and the 25-year, 24-hour storm events. Table 1 summarizes the results from the SEDCAD+ analyses. SEDCAD+ output files are presented herein in Attachments 2A and 2B.

2. The slowest measured percolation rate at Kekaha Landfill is:

   1 inch/6 minutes = 10 inches/hour

   Pond dewatering rate =

$$\frac{1\ inch}{6\ \text{minute}} \ x \ \frac{1\ \text{minute}}{60\ \text{seconds}} \ x \ \frac{1\ ft}{12\ \text{inches}} \ \frac{43{,}560\ ft^2}{acre} = \frac{10.08\ cfs}{acre}$$

   For the infiltration basin, the output in cfs for the different elevations depends on the areas (in acres) of the pond contours for each different elevation multiplied by 10.083.

				Project No.	60197394		
Client	County of Kaua'i	Subject	Surface Water	Prepared By	NKW	Date	8/26/15
Project	Kekaha Landfill - ER	Infiltration Basin Analysis		Reviewed By	DFP	Date	12/15/15
Phase II, Cell 2 Lateral Exp.				Approved By	NKW	Date	12/15/15

Conclusions

1. The existing infiltration basin and infiltration channels will detain the design storm events.

2. The following Table 1 summarizes the results from the SEDCAD+ analysis for final elevation 120'.

TABLE 1 – SUMMARY OF INFILTRATION BASIN ANALYSES FOR PHASE II FINAL CONDITION

Parameter	Infiltration Basin	
	100-Year, 1-Hour Storm	25-Year, 24-Hour Storm
Total Drainage Area (ac)	50.46	
Peak Flow In (cfs)	98.5	108.8
Peak Flow Out (cfs)	26.6	30.46
Peak Water Elevation (ft.)	8.61	9.19
Dewatering Time (days)	0.23	0.72
Volume of Permanent Pool (ac-ft)	0.02	
Top of Berm Elevation	10.0	
Bottom of Basin Elevation	7.5	

TABLE 2 – PEAK FLOWS FOR DESIGN OF STRUCTURES

Structure No.	Type/ Name of Structure	Peak Flows	
		25-Year, 24-Hour Storm (cfs)	100-Year, 1-Hour Storm (cfs)
1	Top Berm A-1	6.93	7.01
2	Top Berm A-2	7.37	7.46
3	Flume A-1	14.3	14.47
4	Terrace Diversion Berm A-1	3.23	3.27
5	Terrace Diversion Berm A-2	3.31	3.35
6	Flume A-2	20.84	21.09
7	Channel #1	31.77	28.67
8	Channel #2	4.20	4.25
9	Top Berm B-1	2.70	2.73
10	Top Berm B-2	0.95	0.96
11	Flume B-1	3.65	3.69
12	Terrace Diversion Berm B-1	1.22	1.24
13	Terrace Diversion Berm B-2	1.84	1.86
14	Flume B-2	6.71	6.79
15	Channel #3	~2.0	~2.0
16	Roadside Ditch #2	5.40	5.46
18	Top Berm C-1	14.54	12.53
20	Roadside Ditch #1 to Culvert C-1	4.48	4.53
21	Culvert C-1	4.48	4.53
22	Flume C-2	18.77	14.73
23	Terrace Berm DB-C-1	0.97	0.99
24	Terrace Berm DB-C-2	8.03	7.03
25	Flume C-3	27.77	22.75
26	Channel #6	6.80	4.87
27	Lower Culvert C-2	34.57	27.62
28	Culvert B	12.13	12.28
29	Channel #4	13.34	12.47
30	Channel #5	5.04	5.10
31	Culvert G to Infiltration Ditch and Pond	18.38	17.56
33	Top Berm F-1	5.73	5.03
34	Top Berm F-2	4.37	4.42
35	Flume F-2	10.0	8.47
36	Terrace Berm DBF-1	9.41	8.20
37	Terrace Berm DBF-2	3.14	3.18
38	Lower Flume F-3	22.40	18.28
39	Lower Berm DBF-4	8.24	6.83
40	Channel & Culvert F	62.25	49.36
41	Infiltration Basin and Ditches	98.54	108.77

Note:
1. The purpose of this table is to identify the worst-case flow condition for each structure for design of each structure.
2. Shaded structures and corresponding shaded flows indicate the designed structure and its corresponding worst-case flow condition.

ATTACHMENT 2A

INFILTRATION BASIN DESIGN FINAL ELEVATION 120'
SEDCAD+ OUTPUT
100-YEAR, 1-HOUR STORM EVENT



LEGEND

- EXISTING TOPOGRAPHY
- FINAL CONTOURS
- PROPERTY LINE
- EXISTING LIMITS OF WASTE
- PPROPOSED LIMIT OF WASTE
- CELL DEVELOPMENT LIMIT

NOTES:

1. TOPOGRAPHIC MAP PREPARED BY WALKER AND ASSOCIATES, SEATTLE, WASHINGTON. DATE OF FLIGHT: 2/24/2015 AND PORTIONS ARE ANTICIPATED TO BE UPDATED BASED ON GROUND SURVEY.
2. HORIZONTAL DATUM IS BASED ON NAD83 (1986), HAWAII ZONE 4. VERTICAL DATUM BASED ON LOCAL TIDAL.

SCALE 0' 50' 100' 200'

NO	REVISIONS	DRN	CHK	DATE
1	REVISED FIGURE NUMBER, FINAL GRADES AND SURFACE WATER FEATURES	CFF	NKW	02/2016

NANCY K. WRIGHT
LICENSED PROFESSIONAL ENGINEER
No. 16394-C
HAWAII U.S.A.

THIS WORK WAS PREPARED BY ME OR UNDER MY SUPERVISION.

SIGNATURE

APRIL 30, 2016
EXPIRATION DATE OF THE LICENSE

Sheboygan, Wisconsin

DRN	DRB	
DES	NKW/TCR	
CHK	FLC/MRH	
APP	KJB	

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PREPARED BY AECOM

ENGINEERING REPORT
KEKAHA LANDFILL PHASE II-CELL 2 LATERAL EXPANSION KEKAHA SANITARY LANDFILL
KAUAI, HAWAII

SURFACE WATER MANAGEMENT NETWORK

DATE AUGUST 2012

PROJECT NO 60197394

FILENAME

SHEET NO

DRAWING NO FIGURE 1

8/30/2015 1:02:27 PM
Pentable = L:\Groups\ds60\197394 Kekaha Cell 2\v8\ Plot Table.tbl
CAD File = L:\Groups\ds60\197394 Kekaha Cell 2\Figure 1 SW Area.dgn

ATTACHMENT 2A

INFILTRATION BASIN DESIGN FINAL ELEVATION 120'
SEDCAD+ OUTPUT
100-YEAR, 1-HOUR STORM EVENT

Kekaha Landfill
Phase II, Cell 2, Horizontal Expansion
Engineering Report
Final Elevation 120'
February, 2016

100-Year, 1-Hour Storm Event

Nancy K. Wright

General Information

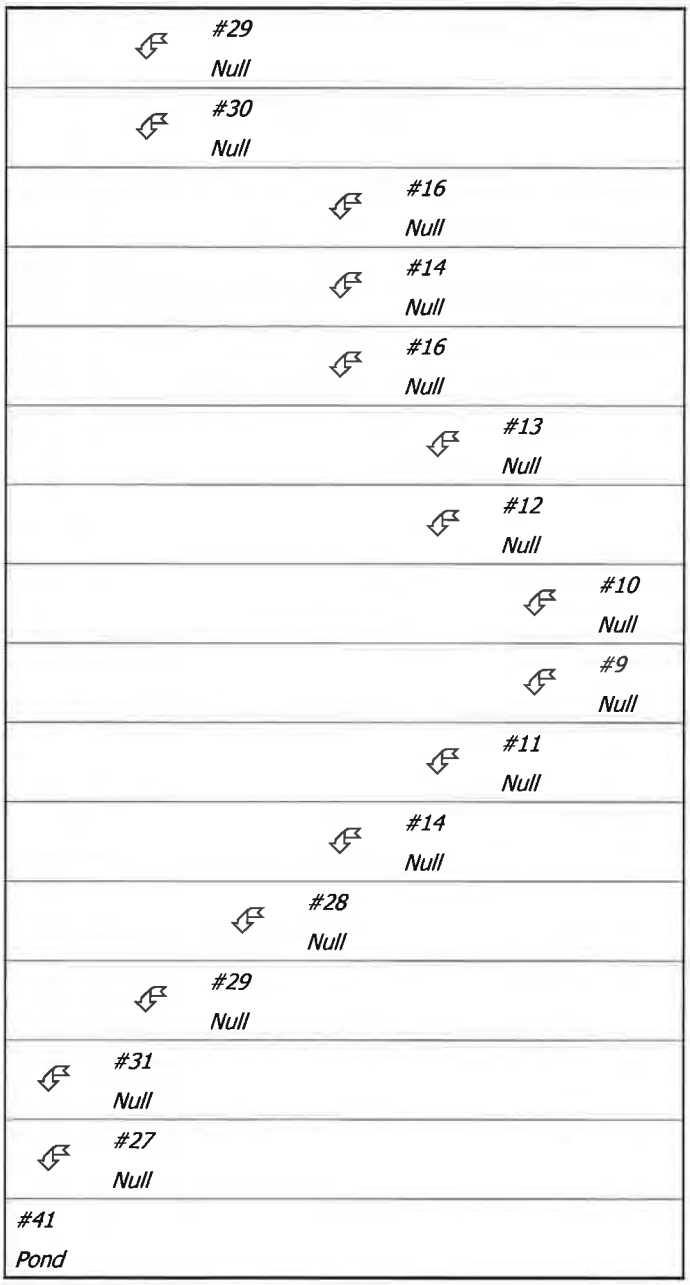
Storm Information:

Storm Type:	NRCS Type I
Design Storm:	100 yr - 1 hr
Rainfall Depth:	3.000 inches

Structure Networking:

Type	Stru #	(flows into)	Stru #	Musk. K (hrs)	Musk. X	Description
Null	#1	==>	#3	0.002	0.452	Top Berm A-1
Null	#2	==>	#3	0.002	0.452	Top Berm A-2
Null	#3	==>	#6	0.002	0.452	Top Flume A-1
Null	#4	==>	#6	0.002	0.452	Terrace Berm DB-A-1
Null	#5	==>	#6	0.002	0.452	Terrace Berm DB-A-2
Null	#6	==>	#7	0.105	0.234	Lower Flume A
Null	#7	==>	#40	0.000	0.000	Channel #1
Null	#8	==>	#7	0.105	0.234	Channel #2
Null	#9	==>	#11	0.002	0.452	Top Berm B-1
Null	#10	==>	#11	0.002	0.452	Top Berm B-2
Null	#11	==>	#14	0.001	0.452	Top Flume B-1
Null	#12	==>	#14	0.001	0.452	Terrace Berm DB-B-1
Null	#13	==>	#14	0.001	0.452	Terrace Berm DB-B-2
Null	#14	==>	#28	0.000	0.000	Lower Flume B
Null	#15	==>	#8	0.048	0.234	Channel #3
Null	#16	==>	#28	0.000	0.000	Roadside Ditch #2 to Culvert B.
Null	#18	==>	#22	0.001	0.452	Top Berm C-1
Null	#20	==>	#21	0.002	0.368	Roadside Ditch #1 to Culvert C-1
Null	#21	==>	#22	0.001	0.452	Culvert C-1
Null	#22	==>	#25	0.001	0.452	Flume C-2
Null	#23	==>	#25	0.001	0.452	Terrace Berm DB-C-1
Null	#24	==>	#25	0.001	0.452	Terrace Berm DB-C-2
Null	#25	==>	#27	0.000	0.000	Flume C-3
Null	#26	==>	#27	0.000	0.000	Channel #6
Null	#27	==>	#41	0.000	0.000	Lower Culvert C-2
Null	#28	==>	#29	0.064	0.234	Culvert B
Null	#29	==>	#31	0.000	0.000	Channel #4
Null	#30	==>	#31	0.000	0.000	Channel #5
Null	#31	==>	#41	0.000	0.000	Culvert G to Infiltration Ditch and Pond
Null	#33	==>	#35	0.003	0.441	Top Berm F-1
Null	#34	==>	#35	0.004	0.441	Top Berm F-2
Null	#35	==>	#38	0.004	0.444	Flume F-2
Null	#36	==>	#38	0.004	0.444	Terrace Berm DB-F-1
Null	#37	==>	#38	0.004	0.444	Terrace Berm DB-F-2
Null	#38	==>	#40	0.000	0.000	Lower Flume F-3
Null	#39	==>	#40	0.000	0.000	Lower Berm DB-F-4
Null	#40	==>	#41	0.000	0.000	Channel & Culvert F
Pond	#41	==>	End	0.000	0.000	Infiltration Pond and Ditches

#39 Null				
	#37 Null			
	#36 Null			
		#34 Null		
		#33 Null		
	#35 Null			
#38 Null				
	#8 Null			
		#5 Null		
		#4 Null		
			#2 Null	
			#1 Null	
		#3 Null		
	#6 Null			
	#24 Null			
	#23 Null			
		#21 Null		
		#18 Null		
	#22 Null			
#7 Null				
#40 Null				
	#30 Null			



Structure Routing Details:

Stru #	Land Flow Condition	Slope (%)	Vert. Dist. (ft)	Horiz. Dist. (ft)	Velocity (fps)	Time (hrs)
#1	8. Large gullies, diversions, and low flowing streams	28.57	48.56	169.96	16.03	0.002
#1	**Muskingum K:**					**0.002**
#2	8. Large gullies, diversions, and low flowing streams	28.57	48.56	170.00	16.03	0.002
#2	**Muskingum K:**					**0.002**

Stru #	Land Flow Condition	Slope (%)	Vert. Dist. (ft)	Horiz. Dist. (ft)	Velocity (fps)	Time (hrs)
#3	8. Large gullies, diversions, and low flowing streams	28.57	37.14	130.00	16.03	0.002
#3	**Muskingum K:**					**0.002**
#4	8. Large gullies, diversions, and low flowing streams	28.57	37.14	130.00	16.03	0.002
#4	**Muskingum K:**					**0.002**
#5	8. Large gullies, diversions, and low flowing streams	28.57	37.14	129.99	16.03	0.002
#5	**Muskingum K:**					**0.002**
#6	6. Grassed waterway	1.00	5.70	570.00	1.50	0.105
#6	**Muskingum K:**					**0.105**
#8	6. Grassed waterway	1.00	5.70	570.00	1.50	0.105
#8	**Muskingum K:**					**0.105**
#9	8. Large gullies, diversions, and low flowing streams	28.57	47.14	165.00	16.03	0.002
#9	**Muskingum K:**					**0.002**
#10	8. Large gullies, diversions, and low flowing streams	28.57	47.14	165.00	16.03	0.002
#10	**Muskingum K:**					**0.002**
#11	8. Large gullies, diversions, and low flowing streams	28.57	28.57	100.00	16.03	0.001
#11	**Muskingum K:**					**0.001**
#15	6. Grassed waterway	1.00	2.60	260.00	1.50	0.048
#15	**Muskingum K:**					**0.048**
#18	8. Large gullies, diversions, and low flowing streams	28.57	22.85	80.00	16.03	0.001
#18	**Muskingum K:**					**0.001**
#20	8. Large gullies, diversions, and low flowing streams	2.50	1.00	40.00	4.74	0.002
#20	**Muskingum K:**					**0.002**
#21	8. Large gullies, diversions, and low flowing streams	28.57	22.85	80.00	16.03	0.001
#21	**Muskingum K:**					**0.001**
#22	8. Large gullies, diversions, and low flowing streams	28.57	31.42	109.97	16.03	0.001
#22	**Muskingum K:**					**0.001**
#23	8. Large gullies, diversions, and low flowing streams	28.57	31.42	109.97	16.03	0.001
#23	**Muskingum K:**					**0.001**
#24	8. Large gullies, diversions, and low flowing streams	28.57	31.42	109.97	16.03	0.001
#24	**Muskingum K:**					**0.001**
#28	6. Grassed waterway	1.00	3.50	350.00	1.50	0.064
#28	**Muskingum K:**					**0.064**
#33	8. Large gullies, diversions, and low flowing streams	18.24	31.00	170.00	12.81	0.003
#33	**Muskingum K:**					**0.003**

Stru #	Land Flow Condition	Slope (%)	Vert. Dist. (ft)	Horiz. Dist. (ft)	Velocity (fps)	Time (hrs)
#34	8. Large gullies, diversions, and low flowing streams	18.10	38.00	210.00	12.76	0.004
#34	**Muskingum K:**					**0.004**
#35	8. Large gullies, diversions, and low flowing streams	20.49	42.00	205.00	13.57	0.004
#35	**Muskingum K:**					**0.004**

Structure Summary:

	Immediate Contributing Area (ac)	Total Contributing Area (ac)	Peak Discharge (cfs)	Total Runoff Volume (ac-ft)
#39	3.130	3.130	6.83	0.31
#37	1.130	1.130	3.18	0.11
#36	3.470	3.470	8.20	0.34
#34	1.570	1.570	4.42	0.16
#33	2.100	2.100	5.03	0.21
#35	0.000	3.670	8.47	0.36
#38	0.000	8.270	18.28	0.82
#8	0.730	1.510	4.25	0.15
#5	1.190	1.190	3.35	0.12
#4	1.160	1.160	3.27	0.11
#2	2.650	2.650	7.46	0.26
#1	2.490	2.490	7.01	0.25
#3	0.000	5.140	14.47	0.51
#6	0.000	7.490	21.09	0.74
#24	2.950	2.950	7.03	0.29
#23	0.350	0.350	0.99	0.03
#21	0.000	1.610	4.53	0.16
#18	5.440	5.440	12.53	0.54
#22	0.000	7.050	14.73	0.70
#7	2.780	11.780	28.67	1.17
#40	0.000	23.180	49.36	2.30
#30	1.810	1.810	5.10	0.18
#29	0.510	4.870	12.47	0.59
#30	1.810	1.810	5.10	0.18
#16	1.940	1.940	5.46	0.19
#14	0.000	2.410	6.79	0.35
#16	1.940	1.940	5.46	0.19
#13	0.660	0.660	1.86	0.07
#12	0.440	0.440	1.24	0.04
#10	0.340	0.340	0.96	0.03
#9	0.970	0.970	2.73	0.10
#11	0.000	1.310	3.69	0.13
#14	0.000	2.410	6.79	0.35
#28	0.010	4.360	12.28	0.54
#29	0.510	4.870	12.47	0.59
#31	0.000	6.680	17.56	0.77
#27	0.000	12.930	27.62	0.26

		Immediate Contributing Area (ac)	Total Contributing Area (ac)	Peak Discharge (cfs)	Total Runoff Volume (ac-ft)
#41	In	7.672	50.462	98.54	4.70
	Out			26.61	4.70

Structure Detail:

Structure #39 (Null)

Lower Berm DB-F-4

Structure #37 (Null)

Terrace Berm DB-F-2

Structure #36 (Null)

Terrace Berm DB-F-1

Structure #34 (Null)

Top Berm F-2

Structure #33 (Null)

Top Berm F-1

Structure #35 (Null)

Flume F-2

Structure #38 (Null)

Lower Flume F-3

Structure #8 (Null)

Channel #2

Structure #5 (Null)

Terrace Berm DB-A-2

Structure #4 (Null)

Terrace Berm DB-A-1

Structure #2 (Null)

Top Berm A-2

Structure #1 (Null)

Top Berm A-1

Structure #3 (Null)

Top Flume A-1

Structure #6 (Null)

Lower Flume A

Structure #24 (Null)

Terrace Berm DB-C-2

Structure #23 (Null)

Terrace Berm DB-C-1

Structure #21 (Null)

Culvert C-1

Structure #18 (Null)

Top Berm C-1

Structure #22 (Null)

Flume C-2

Structure #7 (Null)

Channel #1

Structure #40 (Null)

Channel & Culvert F

Structure #30 (Null)

Channel #5

Structure #29 (Null)

Channel #4

Structure #30 (Null)

Channel #5

Structure #16 (Null)

Roadside Ditch #2 to Culvert B.

Structure #14 (Null)

Lower Flume B

Structure #16 (Null)

Roadside Ditch #2 to Culvert B.

Structure #13 (Null)

Terrace Berm DB-B-2

Structure #12 (Null)

Terrace Berm DB-B-1

Structure #10 (Null)

Top Berm B-2

Structure #9 (Null)

Top Berm B-1

Structure #11 (Null)

Top Flume B-1

Structure #14 (Null)

Lower Flume B

Structure #28 (Null)

Culvert B

Structure #29 (Null)

Channel #4

Structure #31 (Null)

Culvert G to Infiltration Ditch and Pond

Structure #27 (Null)

Lower Culvert C-2

Structure #41 (Pond)

Infiltration Pond and Ditches

Pond Inputs:

Initial Pool Elev:	7.51
Initial Pool:	0.02 ac-ft

Pond Results:

Peak Elevation:	8.61
Dewater Time:	0.23 days

Dewatering time is calculated from peak stage to lowest spillway

Elevation-Capacity-Discharge Table

Elevation	Area (ac)	Capacity (ac-ft)	Discharge (cfs)	Dewater Time (hrs)	
7.50	1.897	0.000	0.000		
7.51	1.898	0.019	0.000		
8.00	2.332	1.054	23.514	4.50	
8.50	2.565	2.277	25.862	0.60	
8.61	2.678	2.581	26.609	0.30	Peak Stage
9.00	2.898	3.642	29.221		
9.50	3.229	5.173	32.558		
10.00	3.562	6.870	35.916		

Detailed Discharge Table

Elevation	User-input discharge (cfs)	Combined Total Discharge (cfs)
7.50	0.000	0.000
7.51	0.000	0.000
8.00	23.514	23.514
8.50	25.862	25.862
9.00	29.221	29.221
9.50	32.558	32.558
10.00	35.916	35.916

Subwatershed Hydrology Detail:

Stru #	SWS #	SWS Area (ac)	Time of Conc (hrs)	Musk K (hrs)	Musk X	Curve Number	UHS	Peak Discharge (cfs)	Runoff Volume (ac-ft)
#39	1	3.130	0.236	0.000	0.000	79.000	TR55	6.83	0.310
	Σ	**3.130**						**6.83**	**0.310**
#37	1	1.130	0.058	0.000	0.000	79.000	TR55	3.18	0.112
	Σ	**1.130**						**3.18**	**0.112**
#36	1	3.470	0.175	0.000	0.000	79.000	TR55	8.20	0.344
	Σ	**3.470**						**8.20**	**0.344**
#34	1	1.570	0.095	0.000	0.000	79.000	TR55	4.42	0.155
	Σ	**1.570**						**4.42**	**0.155**
#33	1	2.100	0.161	0.000	0.000	79.000	TR55	5.03	0.208
	Σ	**2.100**						**5.03**	**0.208**
#35	Σ	**3.670**						**8.47**	**0.363**
#38	Σ	**8.270**						**18.28**	**0.819**
#8	1	0.730	0.056	0.000	0.000	79.000	TR55	2.06	0.072
	Σ	**1.510**						**4.25**	**0.150**
#5	1	1.190	0.053	0.000	0.000	79.000	TR55	3.35	0.118
	Σ	**1.190**						**3.35**	**0.118**
#4	1	1.160	0.052	0.000	0.000	79.000	TR55	3.27	0.115
	Σ	**1.160**						**3.27**	**0.115**
#2	1	2.650	0.099	0.000	0.000	79.000	TR55	7.46	0.262
	Σ	**2.650**						**7.46**	**0.262**
#1	1	2.490	0.097	0.000	0.000	79.000	TR55	7.01	0.247
	Σ	**2.490**						**7.01**	**0.247**
#3	Σ	**5.140**						**14.47**	**0.509**
#6	Σ	**7.490**						**21.09**	**0.742**
#24	1	2.950	0.170	0.000	0.000	79.000	TR55	7.03	0.292
	Σ	**2.950**						**7.03**	**0.292**
#23	1	0.350	0.029	0.000	0.000	79.000	TR55	0.99	0.035
	Σ	**0.350**						**0.99**	**0.035**
#21	Σ	**1.610**						**4.53**	**0.159**

Stru #	SWS #	SWS Area (ac)	Time of Conc (hrs)	Musk K (hrs)	Musk X	Curve Number	UHS	Peak Discharge (cfs)	Runoff Volume (ac-ft)
#18	1	5.440	0.189	0.000	0.000	79.000	TR55	12.53	0.539
	Σ	**5.440**						**12.53**	**0.539**
#22	Σ	**7.050**						**14.73**	**0.698**
#7	1	2.780	0.165	0.000	0.000	79.000	TR55	6.66	0.275
	Σ	**11.780**						**28.67**	**1.167**
#40	Σ	**23.180**						**49.36**	**2.296**
#30	1	1.810	0.117	0.000	0.000	79.000	TR55	5.10	0.179
	Σ	**1.810**						**5.10**	**0.179**
#29	1	0.510	0.085	0.000	0.000	79.000	TR55	1.44	0.051
	Σ	**4.870**						**12.47**	**0.592**
#30	1	1.810	0.117	0.000	0.000	79.000	TR55	5.10	0.179
	Σ	**1.810**						**5.10**	**0.179**
#16	1	1.940	0.056	0.000	0.000	79.000	TR55	5.46	0.192
	Σ	**1.940**						**5.46**	**0.192**
#14	Σ	**2.410**						**6.79**	**0.348**
#16	1	1.940	0.056	0.000	0.000	79.000	TR55	5.46	0.192
	Σ	**1.940**						**5.46**	**0.192**
#13	1	0.660	0.046	0.000	0.000	79.000	TR55	1.86	0.065
	Σ	**0.660**						**1.86**	**0.065**
#12	1	0.440	0.023	0.000	0.000	79.000	TR55	1.24	0.044
	Σ	**0.440**						**1.24**	**0.044**
#10	1	0.340	0.034	0.000	0.000	79.000	TR55	0.96	0.034
	Σ	**0.340**						**0.96**	**0.034**
#9	1	0.970	0.075	0.000	0.000	79.000	TR55	2.73	0.096
	Σ	**0.970**						**2.73**	**0.096**
#11	Σ	**1.310**						**3.69**	**0.130**
#14	Σ	**2.410**						**6.79**	**0.348**
#28	1	0.010	0.001	0.000	0.000	79.000	TR55	0.03	0.001
	Σ	**4.360**						**12.28**	**0.541**
#29	1	0.510	0.085	0.000	0.000	79.000	TR55	1.44	0.051
	Σ	**4.870**						**12.47**	**0.592**
#31	Σ	**6.680**						**17.56**	**0.772**

Stru #	SWS #	SWS Area (ac)	Time of Conc (hrs)	Musk K (hrs)	Musk X	Curve Number	UHS	Peak Discharge (cfs)	Runoff Volume (ac-ft)
#27	**∑**	**12.930**						**27.62**	**0.256**
#41	1	7.672	0.000	0.000	0.000	92.000	TR55	31.87	1.382
	∑	**50.462**						**98.54**	**4.704**

Subwatershed Time of Concentration Details:

Stru #	SWS #	Land Flow Condition	Slope (%)	Vert. Dist. (ft)	Horiz. Dist. (ft)	Velocity (fps)	Time (hrs)
#1	1	3. Short grass pasture	3.00	9.89	330.00	1.380	0.066
		3. Short grass pasture	28.57	11.42	40.00	4.270	0.002
		6. Grassed waterway	3.00	8.39	279.66	2.590	0.029
#1	**1**	**Time of Concentration:**					**0.097**
#2	1	3. Short grass pasture	3.00	9.30	310.00	1.380	0.062
		3. Short grass pasture	28.57	9.99	35.00	4.270	0.002
		6. Grassed waterway	3.00	9.89	329.66	2.590	0.035
#2	**1**	**Time of Concentration:**					**0.099**
#4	1	3. Short grass pasture	28.57	39.99	140.00	4.270	0.009
		6. Grassed waterway	2.00	6.60	330.00	2.120	0.043
#4	**1**	**Time of Concentration:**					**0.052**
#5	1	3. Short grass pasture	28.57	45.71	160.00	4.270	0.010
		6. Grassed waterway	2.00	6.70	335.00	2.120	0.043
#5	**1**	**Time of Concentration:**					**0.053**
#7	1	3. Short grass pasture	28.57	37.14	129.99	4.270	0.008
		6. Grassed waterway	1.00	8.50	850.00	1.500	0.157
#7	**1**	**Time of Concentration:**					**0.165**
#8	1	3. Short grass pasture	28.57	37.14	130.00	4.270	0.008
		6. Grassed waterway	1.00	2.60	260.00	1.500	0.048
#8	**1**	**Time of Concentration:**					**0.056**
#9	1	3. Short grass pasture	3.00	9.30	310.00	1.380	0.062
		3. Short grass pasture	28.57	9.99	34.96	4.270	0.002
		6. Grassed waterway	3.00	3.30	110.00	2.590	0.011
#9	**1**	**Time of Concentration:**					**0.075**
#10	1	3. Short grass pasture	28.57	12.85	45.00	4.270	0.002
		6. Grassed waterway	3.00	9.00	300.00	2.590	0.032
#10	**1**	**Time of Concentration:**					**0.034**
#12	1	3. Short grass pasture	28.57	45.71	160.00	4.270	0.010
		6. Grassed waterway	2.00	2.09	105.00	2.120	0.013
#12	**1**	**Time of Concentration:**					**0.023**
#13	1	3. Short grass pasture	26.67	40.00	150.00	4.130	0.010
		6. Grassed waterway	2.00	5.60	280.00	2.120	0.036

Stru #	SWS #	Land Flow Condition	Slope (%)	Vert. Dist. (ft)	Horiz. Dist. (ft)	Velocity (fps)	Time (hrs)
#13	**1**	**Time of Concentration:**					**0.046**
#15	1	3. Short grass pasture	28.57	28.57	100.00	4.270	0.006
		6. Grassed waterway	3.00	6.00	200.00	2.590	0.021
#15	**1**	**Time of Concentration:**					**0.027**
#16	1	3. Short grass pasture	28.57	28.57	100.00	4.270	0.006
		6. Grassed waterway	7.17	52.00	725.00	4.010	0.050
#16	**1**	**Time of Concentration:**					**0.056**
#18	1	3. Short grass pasture	3.00	10.20	340.00	1.380	0.068
		6. Grassed waterway	3.00	33.89	1,130.00	2.590	0.121
#18	**1**	**Time of Concentration:**					**0.189**
#20	1	3. Short grass pasture	3.00	6.89	230.00	1.380	0.046
		6. Grassed waterway	7.73	34.00	440.00	4.160	0.029
#20	**1**	**Time of Concentration:**					**0.075**
#23	1	3. Short grass pasture	13.64	3.00	22.00	2.950	0.002
		6. Grassed waterway	2.00	4.19	210.00	2.120	0.027
#23	**1**	**Time of Concentration:**					**0.029**
#24	1	3. Short grass pasture	28.57	31.42	110.00	4.270	0.007
		6. Grassed waterway	2.00	25.00	1,250.00	2.120	0.163
#24	**1**	**Time of Concentration:**					**0.170**
#26	1	3. Short grass pasture	28.57	27.14	95.00	4.270	0.006
		6. Grassed waterway	2.00	19.60	980.00	2.120	0.128
#26	**1**	**Time of Concentration:**					**0.134**
#28	1	6. Grassed waterway	2.00	0.19	10.00	2.120	0.001
#28	**1**	**Time of Concentration:**					**0.001**
#29	1	3. Short grass pasture	2.00	0.20	10.00	1.130	0.002
		6. Grassed waterway	1.00	4.50	450.00	1.500	0.083
#29	**1**	**Time of Concentration:**					**0.085**
#30	1	3. Short grass pasture	28.57	34.28	120.00	4.270	0.007
		6. Grassed waterway	1.00	5.95	595.00	1.500	0.110
#30	**1**	**Time of Concentration:**					**0.117**
#33	1	3. Short grass pasture	3.00	10.20	340.00	1.380	0.068
		6. Grassed waterway	3.00	26.25	875.00	2.590	0.093
#33	**1**	**Time of Concentration:**					**0.161**
#34	1	3. Short grass pasture	3.00	9.89	329.66	1.380	0.066
		3. Short grass pasture	28.57	11.42	39.97	4.270	0.002
		6. Grassed waterway	3.00	7.80	260.00	2.590	0.027
#34	**1**	**Time of Concentration:**					**0.095**
#36	1	3. Short grass pasture	28.57	31.42	110.00	4.270	0.007
		6. Grassed waterway	2.00	25.70	1,285.00	2.120	0.168
#36	**1**	**Time of Concentration:**					**0.175**
#37	1	3. Short grass pasture	28.57	39.99	140.00	4.270	0.009
		6. Grassed waterway	2.00	7.60	380.00	2.120	0.049

Stru #	SWS #	Land Flow Condition	Slope (%)	Vert. Dist. (ft)	Horiz. Dist. (ft)	Velocity (fps)	Time (hrs)
#37	**1**	**Time of Concentration:**					**0.058**
#39	1	3. Short grass pasture	28.57	14.28	50.00	4.270	0.003
		6. Grassed waterway	2.00	35.70	1,785.00	2.120	0.233
#39	**1**	**Time of Concentration:**					**0.236**

ATTACHMENT 2B

INFILTRATION BASIN DESIGN FINAL ELEVATION 120'
SEDCAD+ OUTPUT
25-YEAR, 24-HOUR STORM EVENT

Kekaha Landfill
Phase II, Cell 2, Horizontal Expansion
Engineering Report
Final Elevation 120'
February, 2016

25-Year, 24-Hour Storm Event

Nancy K. Wright

General Information

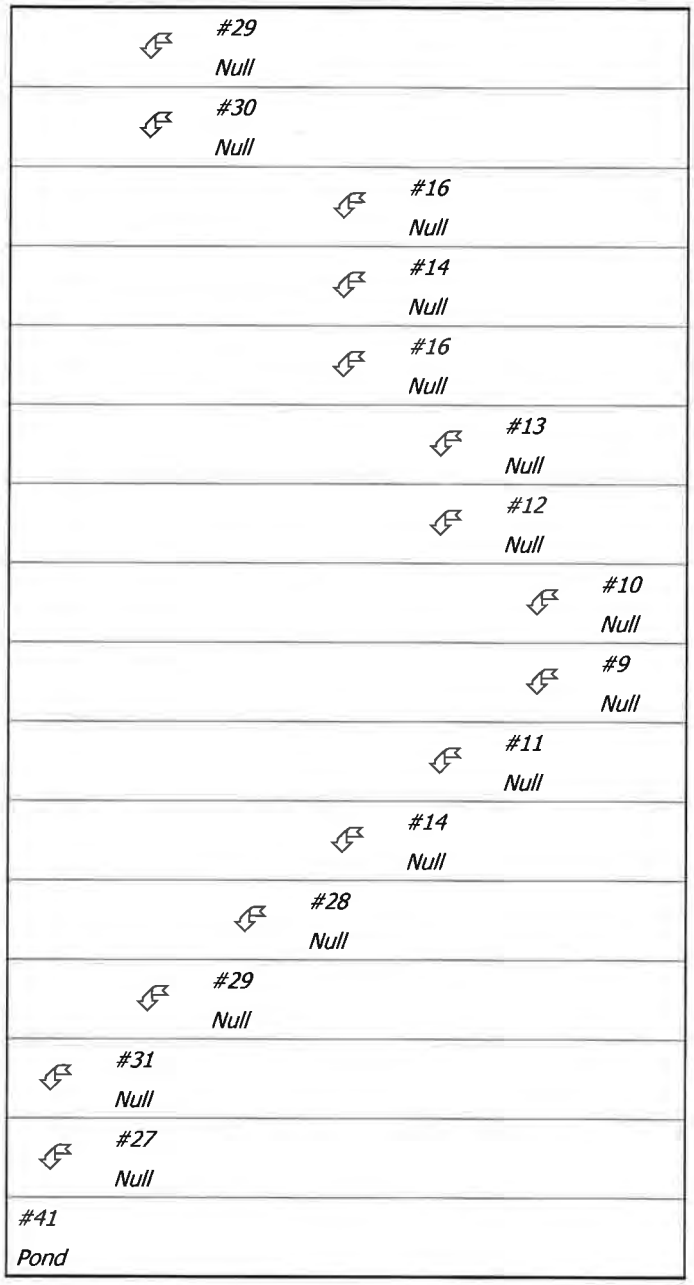
Storm Information:

Storm Type:	NRCS Type I
Design Storm:	25 yr - 24 hr
Rainfall Depth:	8.000 inches

Structure Networking:

Type	Stru #	(flows into)	Stru #	Musk. K (hrs)	Musk. X	Description
Null	#1	==>	#3	0.002	0.452	Top Berm A-1
Null	#2	==>	#3	0.002	0.452	Top Berm A-2
Null	#3	==>	#6	0.002	0.452	Top Flume A-1
Null	#4	==>	#6	0.002	0.452	Terrace Berm DB-A-1
Null	#5	==>	#6	0.002	0.452	Terrace Berm DB-A-2
Null	#6	==>	#7	0.105	0.234	Lower Flume A
Null	#7	==>	#40	0.000	0.000	Channel #1
Null	#8	==>	#7	0.105	0.234	Channel #2
Null	#9	==>	#11	0.002	0.452	Top Berm B-1
Null	#10	==>	#11	0.002	0.452	Top Berm B-2
Null	#11	==>	#14	0.001	0.452	Top Flume B-1
Null	#12	==>	#14	0.001	0.452	Terrace Berm DB-B-1
Null	#13	==>	#14	0.001	0.452	Terrace Berm DB-B-2
Null	#14	==>	#28	0.000	0.000	Lower Flume B
Null	#15	==>	#8	0.000	0.000	Channel #3
Null	#16	==>	#28	0.000	0.000	Roadside Ditch #2 to Culvert B.
Null	#18	==>	#22	0.001	0.452	Top Berm C-1
Null	#20	==>	#21	0.000	0.000	Roadside Ditch #1 to Culvert C-1
Null	#21	==>	#22	0.001	0.452	Culvert C-1
Null	#22	==>	#25	0.001	0.452	Flume C-2
Null	#23	==>	#25	0.001	0.452	Terrace Berm DB-C-1
Null	#24	==>	#25	0.001	0.452	Terrace Berm DB-C-2
Null	#25	==>	#27	0.001	0.424	Flume C-3
Null	#26	==>	#27	0.001	0.424	Channel #6
Null	#27	==>	#41	0.000	0.000	Lower Culvert C-2
Null	#28	==>	#29	0.064	0.234	Culvert B
Null	#29	==>	#31	0.000	0.000	Channel #4
Null	#30	==>	#31	0.000	0.000	Channel #5
Null	#31	==>	#41	0.000	0.000	Culvert G to Infiltration Ditch and Pond
Null	#33	==>	#35	0.003	0.441	Top Berm F-1
Null	#34	==>	#35	0.004	0.441	Top Berm F-2
Null	#35	==>	#38	0.004	0.444	Flume F-2
Null	#36	==>	#38	0.004	0.444	Terrace Berm DB-F-1
Null	#37	==>	#38	0.004	0.444	Terrace Berm DB-F-2
Null	#38	==>	#40	0.000	0.000	Lower Flume F-3
Null	#39	==>	#40	0.000	0.000	Lower Berm DB-F-4
Null	#40	==>	#41	0.000	0.000	Channel & Culvert F
Pond	#41	==>	End	0.000	0.000	Infiltration Pond and Ditches

#39 Null									
	#37 Null								
	#36 Null								
		#34 Null							
		#33 Null							
	#35 Null								
#38 Null									
	#8 Null								
		#5 Null							
		#4 Null							
			#2 Null						
			#1 Null						
		#3 Null							
	#6 Null								
	#24 Null								
	#23 Null								
		#21 Null							
		#18 Null							
	#22 Null								
#7 Null									
#40 Null									
	#30 Null								



Structure Routing Details:

Stru #	Land Flow Condition	Slope (%)	Vert. Dist. (ft)	Horiz. Dist. (ft)	Velocity (fps)	Time (hrs)
#1	8. Large gullies, diversions, and low flowing streams	28.57	48.56	169.96	16.03	0.002
#1	**Muskingum K:**					**0.002**
#2	8. Large gullies, diversions, and low flowing streams	28.57	48.56	170.00	16.03	0.002
#2	**Muskingum K:**					**0.002**

Stru #	Land Flow Condition	Slope (%)	Vert. Dist. (ft)	Horiz. Dist. (ft)	Velocity (fps)	Time (hrs)
#3	8. Large gullies, diversions, and low flowing streams	28.57	37.14	130.00	16.03	0.002
#3	**Muskingum K:**					**0.002**
#4	8. Large gullies, diversions, and low flowing streams	28.57	37.14	130.00	16.03	0.002
#4	**Muskingum K:**					**0.002**
#5	8. Large gullies, diversions, and low flowing streams	28.57	37.14	129.99	16.03	0.002
#5	**Muskingum K:**					**0.002**
#6	6. Grassed waterway	1.00	5.70	570.00	1.50	0.105
#6	**Muskingum K:**					**0.105**
#8	6. Grassed waterway	1.00	5.70	570.00	1.50	0.105
#8	**Muskingum K:**					**0.105**
#9	8. Large gullies, diversions, and low flowing streams	28.57	47.14	165.00	16.03	0.002
#9	**Muskingum K:**					**0.002**
#10	8. Large gullies, diversions, and low flowing streams	28.57	47.14	165.00	16.03	0.002
#10	**Muskingum K:**					**0.002**
#11	8. Large gullies, diversions, and low flowing streams	28.57	28.57	100.00	16.03	0.001
#11	**Muskingum K:**					**0.001**
#18	8. Large gullies, diversions, and low flowing streams	28.57	22.85	80.00	16.03	0.001
#18	**Muskingum K:**					**0.001**
#21	8. Large gullies, diversions, and low flowing streams	28.57	28.57	100.00	16.03	0.001
#21	**Muskingum K:**					**0.001**
#22	8. Large gullies, diversions, and low flowing streams	28.57	31.42	109.97	16.03	0.001
#22	**Muskingum K:**					**0.001**
#23	8. Large gullies, diversions, and low flowing streams	28.57	31.42	109.97	16.03	0.001
#23	**Muskingum K:**					**0.001**
#24	8. Large gullies, diversions, and low flowing streams	28.57	31.42	109.97	16.03	0.001
#24	**Muskingum K:**					**0.001**
#25	8. Large gullies, diversions, and low flowing streams	10.00	4.00	40.00	9.48	0.001
#25	**Muskingum K:**					**0.001**
#26	8. Large gullies, diversions, and low flowing streams	10.00	4.00	40.00	9.48	0.001
#26	**Muskingum K:**					**0.001**
#28	6. Grassed waterway	1.00	3.50	350.00	1.50	0.064
#28	**Muskingum K:**					**0.064**
#33	8. Large gullies, diversions, and low flowing streams	18.24	31.00	170.00	12.81	0.003

Stru #	Land Flow Condition	Slope (%)	Vert. Dist. (ft)	Horiz. Dist. (ft)	Velocity (fps)	Time (hrs)
#33	**Muskingum K:**					**0.003**
#34	8. Large gullies, diversions, and low flowing streams	18.10	38.00	210.00	12.76	0.004
#34	**Muskingum K:**					**0.004**
#35	8. Large gullies, diversions, and low flowing streams	20.49	42.00	205.00	13.57	0.004
#35	**Muskingum K:**					**0.004**

Structure Summary:

	Immediate Contributing Area (ac)	Total Contributing Area (ac)	Peak Discharge (cfs)	Total Runoff Volume (ac-ft)
#39	3.130	3.130	8.24	1.44
#37	1.130	1.130	3.14	0.52
#36	3.470	3.470	9.41	1.59
#34	1.570	1.570	4.37	0.72
#33	2.100	2.100	5.73	0.96
#35	0.000	3.670	10.00	1.68
#38	0.000	8.270	22.40	3.80
#8	1.510	1.510	4.20	0.69
#5	1.190	1.190	3.31	0.55
#4	1.160	1.160	3.23	0.53
#2	2.650	2.650	7.37	1.22
#1	2.490	2.490	6.93	1.14
#3	0.000	5.140	14.30	2.36
#6	0.000	7.490	20.84	3.44
#24	2.950	2.950	8.03	1.35
#23	0.350	0.350	0.97	0.16
#21	1.610	1.610	4.48	0.74
#18	5.440	5.440	14.54	2.50
#22	0.000	7.050	18.77	3.24
#7	2.780	11.780	31.77	5.41
#40	0.000	23.180	62.25	10.64
#30	1.810	1.810	5.04	0.83
#29	0.510	4.870	13.34	2.74
#30	1.810	1.810	5.04	0.83
#16	1.940	1.940	5.40	0.89
#14	0.000	2.410	6.71	1.61
#16	1.940	1.940	5.40	0.89
#13	0.660	0.660	1.84	0.30
#12	0.440	0.440	1.22	0.20
#10	0.340	0.340	0.95	0.16
#9	0.970	0.970	2.70	0.45
#11	0.000	1.310	3.65	0.60
#14	0.000	2.410	6.71	1.61
#28	0.010	4.360	12.13	2.51
#29	0.510	4.870	13.34	2.74
#31	0.000	6.680	18.38	3.58
#27	0.000	12.930	34.57	1.18

		Immediate Contributing Area (ac)	Total Contributing Area (ac)	Peak Discharge (cfs)	Total Runoff Volume (ac-ft)
#41	In	7.672	50.462	108.77	19.90
	Out			30.46	19.90

Structure Detail:

Structure #39 (Null)

Lower Berm DB-F-4

Structure #37 (Null)

Terrace Berm DB-F-2

Structure #36 (Null)

Terrace Berm DB-F-1

Structure #34 (Null)

Top Berm F-2

Structure #33 (Null)

Top Berm F-1

Structure #35 (Null)

Flume F-2

Structure #38 (Null)

Lower Flume F-3

Structure #8 (Null)

Channel #2

Structure #5 (Null)

Terrace Berm DB-A-2

Structure #4 (Null)

Terrace Berm DB-A-1

Structure #2 (Null)

Top Berm A-2

Structure #1 (Null)

Top Berm A-1

Structure #3 (Null)

Top Flume A-1

Structure #6 (Null)

Lower Flume A

Structure #24 (Null)

Terrace Berm DB-C-2

Structure #23 (Null)

Terrace Berm DB-C-1

Structure #21 (Null)

Culvert C-1

Structure #18 (Null)

Top Berm C-1

Structure #22 (Null)

Flume C-2

Structure #7 (Null)

Channel #1

Structure #40 (Null)

Channel & Culvert F

Structure #30 (Null)

Channel #5

Structure #29 (Null)

Channel #4

Structure #30 (Null)

Channel #5

Structure #16 (Null)

Roadside Ditch #2 to Culvert B.

Structure #14 (Null)

Lower Flume B

Structure #16 (Null)

Roadside Ditch #2 to Culvert B.

Structure #13 (Null)

Terrace Berm DB-B-2

Structure #12 (Null)

Terrace Berm DB-B-1

Structure #10 (Null)

Top Berm B-2

Structure #9 (Null)

Top Berm B-1

Structure #11 (Null)

Top Flume B-1

Structure #14 (Null)

Lower Flume B

Structure #28 (Null)

Culvert B

Structure #29 (Null)

Channel #4

Structure #31 (Null)

Culvert G to Infiltration Ditch and Pond

Structure #27 (Null)

Lower Culvert C-2

Structure #41 (Pond)

Infiltration Pond and Ditches

Pond Inputs:

Initial Pool Elev:	7.51
Initial Pool:	0.02 ac-ft

Pond Results:

Peak Elevation:	9.19
Dewater Time:	0.72 days

Dewatering time is calculated from peak stage to lowest spillway

Elevation-Capacity-Discharge Table

Elevation	Area (ac)	Capacity (ac-ft)	Discharge (cfs)	Dewater Time (hrs)	
7.50	1.897	0.000	0.000		
7.51	1.898	0.019	0.000		
8.00	2.332	1.054	23.514	14.00	
8.50	2.565	2.277	25.862	1.15	
9.00	2.898	3.642	29.221	1.25	
9.19	3.020	4.209	30.457	0.85	Peak Stage
9.50	3.229	5.173	32.558		
10.00	3.562	6.870	35.916		

Detailed Discharge Table

Elevation	User-input discharge (cfs)	Combined Total Discharge (cfs)
7.50	0.000	0.000
7.51	0.000	0.000
8.00	23.514	23.514
8.50	25.862	25.862
9.00	29.221	29.221
9.50	32.558	32.558
10.00	35.916	35.916

Subwatershed Hydrology Detail:

Stru #	SWS #	SWS Area (ac)	Time of Conc (hrs)	Musk K (hrs)	Musk X	Curve Number	UHS	Peak Discharge (cfs)	Runoff Volume (ac-ft)
#39	1	3.130	0.236	0.000	0.000	79.000	TR55	8.24	1.437
	Σ	**3.130**						**8.24**	**1.437**
#37	1	1.130	0.058	0.000	0.000	79.000	TR55	3.14	0.519
	Σ	**1.130**						**3.14**	**0.519**
#36	1	3.470	0.175	0.000	0.000	79.000	TR55	9.41	1.593
	Σ	**3.470**						**9.41**	**1.593**
#34	1	1.570	0.095	0.000	0.000	79.000	TR55	4.37	0.721
	Σ	**1.570**						**4.37**	**0.721**
#33	1	2.100	0.161	0.000	0.000	79.000	TR55	5.73	0.964
	Σ	**2.100**						**5.73**	**0.964**
#35	Σ	**3.670**						**10.00**	**1.685**
#38	Σ	**8.270**						**22.40**	**3.796**
#8	1	1.510	0.091	0.000	0.000	79.000	TR55	4.20	0.693
	Σ	**1.510**						**4.20**	**0.693**
#5	1	1.190	0.053	0.000	0.000	79.000	TR55	3.31	0.546
	Σ	**1.190**						**3.31**	**0.546**
#4	1	1.160	0.052	0.000	0.000	79.000	TR55	3.23	0.532
	Σ	**1.160**						**3.23**	**0.532**
#2	1	2.650	0.099	0.000	0.000	79.000	TR55	7.37	1.216
	Σ	**2.650**						**7.37**	**1.216**
#1	1	2.490	0.097	0.000	0.000	79.000	TR55	6.93	1.143
	Σ	**2.490**						**6.93**	**1.143**
#3	Σ	**5.140**						**14.30**	**2.359**
#6	Σ	**7.490**						**20.84**	**3.438**
#24	1	2.950	0.170	0.000	0.000	79.000	TR55	8.03	1.354
	Σ	**2.950**						**8.03**	**1.354**
#23	1	0.350	0.029	0.000	0.000	79.000	TR55	0.97	0.161
	Σ	**0.350**						**0.97**	**0.161**
#21	1	1.610	0.076	0.000	0.000	79.000	TR55	4.48	0.739

Stru #	SWS #	SWS Area (ac)	Time of Conc (hrs)	Musk K (hrs)	Musk X	Curve Number	UHS	Peak Discharge (cfs)	Runoff Volume (ac-ft)
	Σ	**1.610**						**4.48**	**0.739**
#18	1	5.440	0.189	0.000	0.000	79.000	TR55	14.54	2.497
	Σ	**5.440**						**14.54**	**2.497**
#22	**Σ**	**7.050**						**18.77**	**3.236**
#7	1	2.780	0.165	0.000	0.000	79.000	TR55	7.59	1.276
	Σ	**11.780**						**31.77**	**5.407**
#40	**Σ**	**23.180**						**62.25**	**10.639**
#30	1	1.810	0.117	0.000	0.000	79.000	TR55	5.04	0.831
	Σ	**1.810**						**5.04**	**0.831**
#29	1	0.510	0.085	0.000	0.000	79.000	TR55	1.42	0.234
	Σ	**4.870**						**13.34**	**2.745**
#30	1	1.810	0.117	0.000	0.000	79.000	TR55	5.04	0.831
	Σ	**1.810**						**5.04**	**0.831**
#16	1	1.940	0.056	0.000	0.000	79.000	TR55	5.40	0.890
	Σ	**1.940**						**5.40**	**0.890**
#14	**Σ**	**2.410**						**6.71**	**1.611**
#16	1	1.940	0.056	0.000	0.000	79.000	TR55	5.40	0.890
	Σ	**1.940**						**5.40**	**0.890**
#13	1	0.660	0.046	0.000	0.000	79.000	TR55	1.84	0.303
	Σ	**0.660**						**1.84**	**0.303**
#12	1	0.440	0.023	0.000	0.000	79.000	TR55	1.22	0.202
	Σ	**0.440**						**1.22**	**0.202**
#10	1	0.340	0.034	0.000	0.000	79.000	TR55	0.95	0.156
	Σ	**0.340**						**0.95**	**0.156**
#9	1	0.970	0.075	0.000	0.000	79.000	TR55	2.70	0.445
	Σ	**0.970**						**2.70**	**0.445**
#11	**Σ**	**1.310**						**3.65**	**0.601**
#14	**Σ**	**2.410**						**6.71**	**1.611**
#28	1	0.010	0.001	0.000	0.000	79.000	TR55	0.03	0.005
	Σ	**4.360**						**12.13**	**2.506**
#29	1	0.510	0.085	0.000	0.000	79.000	TR55	1.42	0.234
	Σ	**4.870**						**13.34**	**2.745**

Stru #	SWS #	SWS Area (ac)	Time of Conc (hrs)	Musk K (hrs)	Musk X	Curve Number	UHS	Peak Discharge (cfs)	Runoff Volume (ac-ft)
#31	**∑**	**6.680**						**18.38**	**3.576**
#27	**∑**	**12.930**						**34.57**	**1.184**
#41	1	7.672	0.000	0.000	0.000	92.000	TR55	25.35	4.503
	∑	**50.462**						**108.77**	**19.902**

Subwatershed Time of Concentration Details:

Stru #	SWS #	Land Flow Condition	Slope (%)	Vert. Dist. (ft)	Horiz. Dist. (ft)	Velocity (fps)	Time (hrs)
#1	1	3. Short grass pasture	3.00	9.89	330.00	1.380	0.066
		3. Short grass pasture	28.57	11.42	40.00	4.270	0.002
		6. Grassed waterway	3.00	8.39	279.66	2.590	0.029
#1	**1**	**Time of Concentration:**					**0.097**
#2	1	3. Short grass pasture	3.00	9.30	310.00	1.380	0.062
		3. Short grass pasture	28.57	9.99	35.00	4.270	0.002
		6. Grassed waterway	3.00	9.89	329.66	2.590	0.035
#2	**1**	**Time of Concentration:**					**0.099**
#4	1	3. Short grass pasture	28.57	39.99	140.00	4.270	0.009
		6. Grassed waterway	2.00	6.60	330.00	2.120	0.043
#4	**1**	**Time of Concentration:**					**0.052**
#5	1	3. Short grass pasture	28.57	45.71	160.00	4.270	0.010
		6. Grassed waterway	2.00	6.70	335.00	2.120	0.043
#5	**1**	**Time of Concentration:**					**0.053**
#7	1	3. Short grass pasture	28.57	37.14	129.99	4.270	0.008
		6. Grassed waterway	1.00	8.50	850.00	1.500	0.157
#7	**1**	**Time of Concentration:**					**0.165**
#8	1	3. Short grass pasture	28.57	28.57	100.00	4.270	0.006
		6. Grassed waterway	1.00	4.60	460.00	1.500	0.085
#8	**1**	**Time of Concentration:**					**0.091**
#9	1	3. Short grass pasture	3.00	9.30	310.00	1.380	0.062
		3. Short grass pasture	28.57	9.99	34.96	4.270	0.002
		6. Grassed waterway	3.00	3.30	110.00	2.590	0.011
#9	**1**	**Time of Concentration:**					**0.075**
#10	1	3. Short grass pasture	28.57	12.85	45.00	4.270	0.002
		6. Grassed waterway	3.00	9.00	300.00	2.590	0.032
#10	**1**	**Time of Concentration:**					**0.034**
#12	1	3. Short grass pasture	28.57	45.71	160.00	4.270	0.010
		6. Grassed waterway	2.00	2.09	105.00	2.120	0.013
#12	**1**	**Time of Concentration:**					**0.023**
#13	1	3. Short grass pasture	26.67	40.00	150.00	4.130	0.010

Stru #	SWS #	Land Flow Condition	Slope (%)	Vert. Dist. (ft)	Horiz. Dist. (ft)	Velocity (fps)	Time (hrs)
		6. Grassed waterway	2.00	5.60	280.00	2.120	0.036
#13	**1**	**Time of Concentration:**					**0.046**
#16	1	3. Short grass pasture	28.57	28.57	100.00	4.270	0.006
		6. Grassed waterway	7.17	52.00	725.00	4.010	0.050
#16	**1**	**Time of Concentration:**					**0.056**
#18	1	3. Short grass pasture	3.00	10.20	340.00	1.380	0.068
		6. Grassed waterway	3.00	33.89	1,130.00	2.590	0.121
#18	**1**	**Time of Concentration:**					**0.189**
#20	1	3. Short grass pasture	3.00	6.89	230.00	1.380	0.046
		6. Grassed waterway	7.73	34.00	440.00	4.160	0.029
#20	**1**	**Time of Concentration:**					**0.000**
#21	1	3. Short grass pasture	3.00	6.89	230.00	1.380	0.046
		6. Grassed waterway	7.73	34.00	440.00	4.160	0.029
		8. Large gullies, diversions, and low flowing streams	5.00	2.00	40.00	6.700	0.001
#21	**1**	**Time of Concentration:**					**0.076**
#23	1	3. Short grass pasture	13.64	3.00	22.00	2.950	0.002
		6. Grassed waterway	2.00	4.19	210.00	2.120	0.027
#23	**1**	**Time of Concentration:**					**0.029**
#24	1	3. Short grass pasture	28.57	31.42	110.00	4.270	0.007
		6. Grassed waterway	2.00	25.00	1,250.00	2.120	0.163
#24	**1**	**Time of Concentration:**					**0.170**
#26	1	3. Short grass pasture	28.57	27.14	95.00	4.270	0.006
		6. Grassed waterway	2.00	19.60	980.00	2.120	0.128
#26	**1**	**Time of Concentration:**					**0.134**
#28	1	6. Grassed waterway	2.00	0.19	10.00	2.120	0.001
#28	**1**	**Time of Concentration:**					**0.001**
#29	1	3. Short grass pasture	2.00	0.20	10.00	1.130	0.002
		6. Grassed waterway	1.00	4.50	450.00	1.500	0.083
#29	**1**	**Time of Concentration:**					**0.085**
#30	1	3. Short grass pasture	28.57	34.28	120.00	4.270	0.007
		6. Grassed waterway	1.00	5.95	595.00	1.500	0.110
#30	**1**	**Time of Concentration:**					**0.117**
#33	1	3. Short grass pasture	3.00	10.20	340.00	1.380	0.068
		6. Grassed waterway	3.00	26.25	875.00	2.590	0.093
#33	**1**	**Time of Concentration:**					**0.161**
#34	1	3. Short grass pasture	3.00	9.89	329.66	1.380	0.066
		3. Short grass pasture	28.57	11.42	39.97	4.270	0.002
		6. Grassed waterway	3.00	7.80	260.00	2.590	0.027
#34	**1**	**Time of Concentration:**					**0.095**
#36	1	3. Short grass pasture	28.57	31.42	110.00	4.270	0.007
		6. Grassed waterway	2.00	25.70	1,285.00	2.120	0.168
#36	**1**	**Time of Concentration:**					**0.175**

Stru #	SWS #	Land Flow Condition	Slope (%)	Vert. Dist. (ft)	Horiz. Dist. (ft)	Velocity (fps)	Time (hrs)
#37	1	3. Short grass pasture	28.57	39.99	140.00	4.270	0.009
		6. Grassed waterway	2.00	7.60	380.00	2.120	0.049
#37	**1**	**Time of Concentration:**					**0.058**
#39	1	3. Short grass pasture	28.57	14.28	50.00	4.270	0.003
		6. Grassed waterway	2.00	35.70	1,785.00	2.120	0.233
#39	**1**	**Time of Concentration:**					**0.236**

Subwatershed Muskingum Routing Details:

Stru #	SWS #	Land Flow Condition	Slope (%)	Vert. Dist. (ft)	Horiz. Dist. (ft)	Velocity (fps)	Time (hrs)
#21	1	8. Large gullies, diversions, and low flowing streams	28.57	0.00	0.00	0.000	0.000
#21	**1**	**Muskingum K:**					**0.000**

Appendix C.3
Surface Water Diversion Berm Analysis

CALCULATION SHEET

				Project No.	60197394		
Client	County of Kaua'i	**Subject**	Surface Water	**Prepared By**	NKW	**Date**	8/2615
Project	Kekaha Landfill - ER		Diversion Berm Analysis	**Reviewed By**	DFP	**Date**	12/15/15
Phase II, Cell 2 Lateral Exp.				**Approved By**	NKW	**Date**	12/15/15

SURFACE WATER DIVERSION BERM ANALYSIS

Objective

Design a diversion berm to intercept surface water runoff from the final cover and divert surface water runoff to the downdrain channels.

Design Criteria and Assumptions

1. Two types of diversion berms are designed for Kekaha Landfill as follows:

   a. Bench (terrace) drainage channel (Type A ditch).
   b. Sideslope drainage diversion berm (Type C ditch).

2. Diversion berms will be conservatively designed to convey the maximum runoff rate from either a 100-year, 1-hour storm event or from a 25-year, 24-hour storm event with 0.1 foot of freeboard.

3. Diversion berm designs will be based on worst case flow conditions. Peak flows are derived from SEDCAD+ output included in attachments of Appendix C.2.

4. All references to elevations correlate to the site-specific vertical datum.

Calculations

One analysis was performed to design each type of diversion berm for the Expansion. The design was performed using the peak flow rates obtained from both the 100-year, 1-hour storm event and the 25-year, 24-hour storm event. SEDCAD+ results are contained in Appendix C.2. Diversion berms are labeled as shown on Figure 1 in Appendix C.2.

1. Bench/Terrace Diversion Berm

The largest flow is estimated to occur in the bench/terrace diversion berm designated as DB-F-1, contributing to Downdrain F. The flow in this diversion berm is estimated to be 9.41 cfs for the 25-year, 24-hour storm, as shown on the SEDCAD+ output files presented in Appendix C.2. The SEDCAD+ Channel Design Utility was used to evaluate the diversion berm for the highest estimated flow. The results of this analysis are presented herein on page 3.

2. Sideslope Drainage Diversion Berms

The largest flow is estimated to occur in the sideslope drainage diversion berm designated as TB-C-1, contributing to Downdrain Flume C-2. The flow in this drainage diversion berm is estimated to be 14.54 cfs for the 25-year, 24-hour storm. Flows are shown in the SEDCAD+ output included in Appendix C.2. The SEDCAD+ channel design utility was used to evaluate the sideslope diversion berms for the highest estimated flow. The results of this analysis are presented herein on Page 4.

Conclusions

Berm details resulting from this calculation are included on the design detail drawing in the plan set. Following are descriptions of the channel designs:

1. The bench/terrace diversion berm (Type A ditch) forms a 2-foot deep, trapezoidal, grass channel with a 2-foot flat bottom and 2:1 sideslopes on both sides. The channel slopes at 2 percent toward the downdrain.

2. The sideslope drainage diversion berm (Type C ditch) forms a 2-foot deep, v-notch, grass channel with a 3.5:1 (H:V) sideslope on one side and a 2:1 (H:V) sideslope on the other side. The channel slopes at 3 percent toward the downdrain.

Bench/Terrace Diversion Berm

TYPE A DITCH

Material: Grass mixture

Trapezoidal Channel

Bottom Width (ft)	Left Sideslope Ratio	Right Sideslope Ratio	Slope (%)	Retardance Classes	Freeboard Depth (ft)	Freeboard % of Depth	Freeboard Mult. x (VxD)	Limiting Velocity (fps)
2.00	2.0:1	2.0:1	2.0	D, B	0.10			5.0

	Stability Class D w/o Freeboard	Stability Class D w/ Freeboard	Capacity Class B w/o Freeboard	Capacity Class B w/ Freeboard
Design Discharge:	9.41 cfs		9.41 cfs	
Depth:	0.88 ft	0.98 ft	1.46 ft	1.56 ft
Top Width:	5.54 ft	5.94 ft	7.84 ft	8.24 ft
Velocity:	2.82 fps		1.31 fps	
X-Section Area:	3.34 sq ft		7.19 sq ft	
Hydraulic Radius:	0.560		0.843	
Froude Number:	0.64		0.24	
Roughness Coefficient:	0.0507		0.1436	

Sideslope Drainage Diversion Berm

TYPE C DITCH
Material: Grass mixture

Triangular Channel

Left Sideslope Ratio	Right Sideslope Ratio	Slope (%)	Retardance Classes	Freeboard Depth (ft)	Freeboard % of Depth	Freeboard Mult. x (VxD)	Limiting Velocity (fps)
3.5:1	2.0:1	3.0	D, B	0.10			5.0

	Stability Class D w/o Freeboard	Stability Class D w/ Freeboard	Capacity Class B w/o Freeboard	Capacity Class B w/ Freeboard
Design Discharge:	14.54 cfs		14.54 cfs	
Depth:	1.19 ft	1.29 ft	1.72 ft	1.82 ft
Top Width:	6.53 ft	7.08 ft	9.44 ft	9.99 ft
Velocity:	3.75 fps		1.80 fps	
X-Section Area:	3.87 sq ft		8.10 sq ft	
Hydraulic Radius:	0.558		0.806	
Froude Number:	0.86		0.34	
Roughness Coefficient:	0.0465		0.1244	

Appendix C.4
Surface Water Downdrain Analysis

CALCULATION SHEET

Project No. 60197394

Client	County of Kaua'i	**Subject**	Surface Water	**Prepared By** NKW	**Date** 8/26/15
Project	Kekaha Landfill - ER		Downdrain Analysis	**Reviewed By** DFP	**Date** 12/15/15
Phase II, Cell 2 Lateral Exp.				**Approved By** NKW	**Date** 12/15/15

SURFACE WATER DOWNDRAIN ANALYSIS

Objective

Design a downslope downdrain (flume) to convey surface water runoff from the diversion berms and diversion terraces to the perimeter channels, culverts, and infiltration ditches.

Design Criteria and Assumptions

1. Downslope downdrains (flumes) will be able to convey runoff from the highest peak flow storm; either a 100-year, 1-hour storm event or a 25-year, 24-hour storm event, without overflow.

2. Each downslope channel will be designed based on the actual worst-case estimated flow condition.

3. Downslope channels will be sloped from approximately 19 percent to approximately 29 percent toward the perimeter channels. Downslope channels will be designed for worst case slope and flow. Channel depth will depend on the shallowest slope condition. Riprap design will depend on the steepest slope condition.

4. Downslope channels will be trapezoidal in shape. The channels will cut into the landfill, final cover installed, then riprap installed.

5. Downslope channel sideslopes will be 2:1 (horizontal:vertical).

6. Downslope channel bottom width will be either 1 foot or 2 feet.

7. Downslope channels will be lined with riprap for erosion protection.

8. All references to elevations correlate to the site-specific vertical datum.

Calculations

Each of the eight downslope channels is analyzed. The peak flow rate associated with each downslope channel is shown on the SEDCAD+ output files presented in Appendix C.2. Downslope channel locations are shown on Figure 1, provided in Appendix C.2.

The SEDCAD+ Channel Design Utility was used to evaluate the downslope channels. The results of these analyses are presented on pages 3 through 18.

Conclusions

The typical cross-section of the downslope channels was developed as shown on the Design Detail Drawing in the drawing set to meet the requirements of the SEDCAD+ analyses. The depths and widths of the channels vary according to the location and peak flow. Dimensions identified by the SEDCAD+ analyses were rounded up to the nearest 1.0 foot to obtain the typical cross-section. Each peak flow rate and specific slope was input into the SEDCAD+ Channel Design Utility. The results are presented on pages 3 through 18. The following schedule was developed to summarize the downslope channel designs.

CALCULATION SHEET

Project No. 60197394

Client	County of Kaua'i	**Subject**	Surface Water	**Prepared By**	NKW **Date** 8/26/15
Project	Kekaha Landfill - ER		Downdrain Analysis	**Reviewed By**	DFP **Date** 12/15/15
	Phase II, Cell 2 Lateral Exp.			**Approved By**	NKW **Date** 12/15/15

Downslope Channel Schedule

SEDCAD Structure Number	Downdrain Flume	25-Year Peak Flow (cfs)	100-Year, Max. 1-Hour Flow (cfs)	Slope (Percent)		Width (ft)	Depth (ft)		Rip-Rap Size (in)			Att. Pgs.
				Min.	Max.		SEDCAD[+1]	Design	D_{min}	D_{50}	D_{max}	
3	A-1	14.3	14.47	25	29	1	0.85	1.0	3	6	9	3&4
6	A-2	20.8	21.09	25	29	2	0.83	1.0	3	6	9	5&6
11	B-1	3.65	3.69	25	29	1	0.46	1.0	3	6	9	7&8
14	B-2	6.71	6.79	25	29	1	0.61	1.0	3	6	9	9&10
22	C-2	18.77	14.73	25	29	2	0.80	1.0	3	6	9	11&12
25	C-3	27.8	22.8	25	29	2	0.93	1.0	5	9	12	13&14
35	F-2	10.00	8.47	20	25	1	0.76	1.0	3	6	9	15 & 16
38	F-3	22.40	18.28	20	25	2	0.89	1.0	3	6	9	17&18

Note:
1. SEDCAD[+] depth includes 0.1-foot freeboard.

Downdrain/Flume A-1 -- 100-Yr, 1-Hr Peak Flow -- 25% Slope

Material: Riprap

Trapezoidal Channel

Bottom Width (ft)	Left Sideslope Ratio	Right Sideslope Ratio	Slope (%)	Freeboard Depth (ft)	Freeboard % of Depth	Freeboard Mult. x (VxD)
1.00	2.0:1	2.0:1	25.0	0.10		

PADER Method - Steep Slope Design

	w/o Freeboard	w/ Freeboard
Design Discharge:	14.50 cfs	
Depth:	0.75 ft	0.85 ft
Top Width:	4.00 ft	4.40 ft
Velocity:	7.73 fps	
X-Section Area:	1.88 sq ft	
Hydraulic Radius:	0.431	
Froude Number:	1.99	
Manning's n:	0.0550	
Dmin:	3.00 in	
D50:	6.00 in	
Dmax:	9.00 in	

Downdrain/Flume A-1 -- 100-Yr, 1-Hr Peak Flow -- 29% Slope

Material: Riprap

Trapezoidal Channel

Bottom Width (ft)	Left Sideslope Ratio	Right Sideslope Ratio	Slope (%)	Freeboard Depth (ft)	Freeboard % of Depth	Freeboard Mult. x (VxD)
1.00	2.0:1	2.0:1	29.0	0.10		

PADER Method - Steep Slope Design

	w/o Freeboard	w/ Freeboard
Design Discharge:	14.50 cfs	
Depth:	0.72 ft	0.82 ft
Top Width:	3.90 ft	4.30 ft
Velocity:	8.16 fps	
X-Section Area:	1.78 sq ft	
Hydraulic Radius:	0.419	
Froude Number:	2.13	
Manning's n:	0.0550	
Dmin:	3.00 in	
D50:	6.00 in	
Dmax:	9.00 in	

Downdrain/Flume A-2 -- 100-Yr, 1-Hr Peak Flow -- 25% Slope

Material: Riprap

Trapezoidal Channel

Bottom Width (ft)	Left Sideslope Ratio	Right Sideslope Ratio	Slope (%)	Freeboard Depth (ft)	Freeboard % of Depth	Freeboard Mult. x (VxD)
2.00	2.0:1	2.0:1	25.0	0.10		

PADER Method - Steep Slope Design

	w/o Freeboard	w/ Freeboard
Design Discharge:	21.09 cfs	
Depth:	0.73 ft	0.83 ft
Top Width:	4.93 ft	5.33 ft
Velocity:	8.31 fps	
X-Section Area:	2.54 sq ft	
Hydraulic Radius:	0.481	
Froude Number:	2.04	
Manning's n:	0.0550	
Dmin:	3.00 in	
D50:	6.00 in	
Dmax:	9.00 in	

Downdrain/Flume A-2 -- 100-Yr, 1-Hr Peak Flow -- 29% Slope

Material: Riprap

Trapezoidal Channel

Bottom Width (ft)	Left Sideslope Ratio	Right Sideslope Ratio	Slope (%)	Freeboard Depth (ft)	Freeboard % of Depth	Freeboard Mult. x (VxD)
2.00	2.0:1	2.0:1	29.0	0.10		

PADER Method - Steep Slope Design

	w/o Freeboard	w/ Freeboard
Design Discharge:	21.09 cfs	
Depth:	0.71 ft	0.81 ft
Top Width:	4.85 ft	5.25 ft
Velocity:	8.66 fps	
X-Section Area:	2.43 sq ft	
Hydraulic Radius:	0.470	
Froude Number:	2.15	
Manning's n:	0.0560	
Dmin:	3.00 in	
D50:	6.00 in	
Dmax:	9.00 in	

Downdrain/Flume B-1 -- 100-Yr, 1-Hr Peak Flow -- 25% Slope

Material: Riprap

Trapezoidal Channel

Bottom Width (ft)	Left Sideslope Ratio	Right Sideslope Ratio	Slope (%)	Freeboard Depth (ft)	Freeboard % of Depth	Freeboard Mult. x (VxD)
1.00	2.0:1	2.0:1	25.0	0.10		

PADER Method - Steep Slope Design

	w/o Freeboard	w/ Freeboard
Design Discharge:	3.69 cfs	
Depth:	0.36 ft	0.46 ft
Top Width:	2.45 ft	2.85 ft
Velocity:	5.86 fps	
X-Section Area:	0.63 sq ft	
Hydraulic Radius:	0.239	
Froude Number:	2.04	
Manning's n:	0.0490	
Dmin:	2.00 in	
D50:	3.00 in	
Dmax:	4.50 in	

Downdrain/Flume B-1 -- 100-Yr, 1-Hr Peak Flow -- 29% Slope

Material: Riprap

Trapezoidal Channel

Bottom Width (ft)	Left Sideslope Ratio	Right Sideslope Ratio	Slope (%)	Freeboard Depth (ft)	Freeboard % of Depth	Freeboard Mult. x (VxD)
1.00	2.0:1	2.0:1	29.0	0.10		

PADER Method - Steep Slope Design

	w/o Freeboard	w/ Freeboard
Design Discharge:	3.69 cfs	
Depth:	0.35 ft	0.45 ft
Top Width:	2.42 ft	2.82 ft
Velocity:	6.09 fps	
X-Section Area:	0.60 sq ft	
Hydraulic Radius:	0.234	
Froude Number:	2.15	
Manning's n:	0.0500	
Dmin:	2.00 in	
D50:	3.00 in	
Dmax:	4.50 in	

Downdrain/Flume B-2 -- 100-Yr, 1-Hr Peak Flow -- 25% Slope

Material: Riprap

Trapezoidal Channel

Bottom Width (ft)	Left Sideslope Ratio	Right Sideslope Ratio	Slope (%)	Freeboard Depth (ft)	Freeboard % of Depth	Freeboard Mult. x (VxD)
1.00	2.0:1	2.0:1	25.0	0.10		

PADER Method - Steep Slope Design

	w/o Freeboard	w/ Freeboard
Design Discharge:	6.79 cfs	
Depth:	0.51 ft	0.61 ft
Top Width:	3.05 ft	3.45 ft
Velocity:	6.52 fps	
X-Section Area:	1.04 sq ft	
Hydraulic Radius:	0.316	
Froude Number:	1.97	
Manning's n:	0.0530	
Dmin:	3.00 in	
D50:	6.00 in	
Dmax:	9.00 in	

Downdrain/Flume B-2 -- 100-Yr, 1-Hr Peak Flow -- 29% Slope

Material: Riprap

Trapezoidal Channel

Bottom Width (ft)	Left Sideslope Ratio	Right Sideslope Ratio	Slope (%)	Freeboard Depth (ft)	Freeboard % of Depth	Freeboard Mult. x (VxD)
1.00	2.0:1	2.0:1	29.0	0.10		

PADER Method - Steep Slope Design

	w/o Freeboard	w/ Freeboard
Design Discharge:	6.79 cfs	
Depth:	0.51 ft	0.61 ft
Top Width:	3.05 ft	3.45 ft
Velocity:	6.53 fps	
X-Section Area:	1.04 sq ft	
Hydraulic Radius:	0.316	
Froude Number:	1.97	
Manning's n:	0.0570	
Dmin:	3.00 in	
D50:	6.00 in	
Dmax:	9.00 in	

Downdrain/Flume C-2 -- 25-Yr, 24-Hr Peak Flow -- 25% Slope

Material: Riprap

Trapezoidal Channel

Bottom Width (ft)	Left Sideslope Ratio	Right Sideslope Ratio	Slope (%)	Freeboard Depth (ft)	Freeboard % of Depth	Freeboard Mult. x (VxD)
2.00	2.0:1	2.0:1	25.0	0.10		

PADER Method - Steep Slope Design

	w/o Freeboard	w/ Freeboard
Design Discharge:	18.77 cfs	
Depth:	0.70 ft	0.80 ft
Top Width:	4.78 ft	5.18 ft
Velocity:	7.95 fps	
X-Section Area:	2.36 sq ft	
Hydraulic Radius:	0.462	
Froude Number:	1.99	
Manning's n:	0.0560	
Dmin:	3.00 in	
D50:	6.00 in	
Dmax:	9.00 in	

Downdrain/Flume C-2 -- 25-Yr, 24-Hr Peak Flow -- 29% Slope

Material: Riprap

Trapezoidal Channel

Bottom Width (ft)	Left Sideslope Ratio	Right Sideslope Ratio	Slope (%)	Freeboard Depth (ft)	Freeboard % of Depth	Freeboard Mult. x (VxD)
2.00	2.0:1	2.0:1	29.0	0.10		

PADER Method - Steep Slope Design

	w/o Freeboard	w/ Freeboard
Design Discharge:	18.77 cfs	
Depth:	0.68 ft	0.78 ft
Top Width:	4.70 ft	5.10 ft
Velocity:	8.28 fps	
X-Section Area:	2.27 sq ft	
Hydraulic Radius:	0.451	
Froude Number:	2.10	
Manning's n:	0.0570	
Dmin:	3.00 in	
D50:	6.00 in	
Dmax:	9.00 in	

Downdrain/Flume C-3 -- 25-Yr, 24-Hr Peak Flow -- 25% Slope

Material: Riprap

Trapezoidal Channel

Bottom Width (ft)	Left Sideslope Ratio	Right Sideslope Ratio	Slope (%)	Freeboard Depth (ft)	Freeboard % of Depth	Freeboard Mult. x (VxD)
2.00	2.0:1	2.0:1	25.0	0.10		

PADER Method - Steep Slope Design

	w/o Freeboard	w/ Freeboard
Design Discharge:	27.80 cfs	
Depth:	0.83 ft	0.93 ft
Top Width:	5.34 ft	5.74 ft
Velocity:	9.08 fps	
X-Section Area:	3.06 sq ft	
Hydraulic Radius:	0.534	
Froude Number:	2.11	
Manning's n:	0.0540	
Dmin:	5.00 in	
D50:	9.00 in	
Dmax:	12.00 in	

Downdrain/Flume C-3 -- 25-Yr, 24-Hr Peak Flow -- 29% Slope

Material: Riprap

Trapezoidal Channel

Bottom Width (ft)	Left Sideslope Ratio	Right Sideslope Ratio	Slope (%)	Freeboard Depth (ft)	Freeboard % of Depth	Freeboard Mult. x (VxD)
2.00	2.0:1	2.0:1	29.0	0.10		

PADER Method - Steep Slope Design

	w/o Freeboard	w/ Freeboard
Design Discharge:	27.80 cfs	
Depth:	0.83 ft	0.93 ft
Top Width:	5.33 ft	5.73 ft
Velocity:	9.10 fps	
X-Section Area:	3.05 sq ft	
Hydraulic Radius:	0.533	
Froude Number:	2.12	
Manning's n:	0.0580	
Dmin:	5.00 in	
D50:	9.00 in	
Dmax:	12.00 in	

Downdrain/Flume F-2 -- 25-Yr, 24-Hr Peak Flow -- 20% Slope

Material: Riprap

Trapezoidal Channel

Bottom Width (ft)	Left Sideslope Ratio	Right Sideslope Ratio	Slope (%)	Freeboard Depth (ft)	Freeboard % of Depth	Freeboard Mult. x (VxD)
1.00	2.0:1	2.0:1	20.0	0.10		

PADER Method - Steep Slope Design

	w/o Freeboard	w/ Freeboard
Design Discharge:	10.00 cfs	
Depth:	0.66 ft	0.76 ft
Top Width:	3.63 ft	4.03 ft
Velocity:	6.55 fps	
X-Section Area:	1.53 sq ft	
Hydraulic Radius:	0.387	
Froude Number:	1.78	
Manning's n:	0.0540	
Dmin:	3.00 in	
D50:	6.00 in	
Dmax:	9.00 in	

Downdrain/Flume F-2 -- 25-Yr, 24-Hr Peak Flow -- 25% Slope

Material: Riprap

Trapezoidal Channel

Bottom Width (ft)	Left Sideslope Ratio	Right Sideslope Ratio	Slope (%)	Freeboard Depth (ft)	Freeboard % of Depth	Freeboard Mult. x (VxD)
1.00	2.0:1	2.0:1	25.0	0.10		

PADER Method - Steep Slope Design

	w/o Freeboard	w/ Freeboard
Design Discharge:	10.00 cfs	
Depth:	0.65 ft	0.75 ft
Top Width:	3.59 ft	3.99 ft
Velocity:	6.75 fps	
X-Section Area:	1.48 sq ft	
Hydraulic Radius:	0.381	
Froude Number:	1.85	
Manning's n:	0.0580	
Dmin:	3.00 in	
D50:	6.00 in	
Dmax:	9.00 in	

Downdrain/Flume F-3 -- 25-Yr, 24-Hr Peak Flow -- 20% Slope

Material: Riprap

Trapezoidal Channel

Bottom Width (ft)	Left Sideslope Ratio	Right Sideslope Ratio	Slope (%)	Freeboard Depth (ft)	Freeboard % of Depth	Freeboard Mult. x (VxD)
2.00	2.0:1	2.0:1	20.0	0.10		

PADER Method - Steep Slope Design

	w/o Freeboard	w/ Freeboard
Design Discharge:	22.40 cfs	
Depth:	0.79 ft	0.89 ft
Top Width:	5.17 ft	5.57 ft
Velocity:	7.90 fps	
X-Section Area:	2.84 sq ft	
Hydraulic Radius:	0.512	
Froude Number:	1.88	
Manning's n:	0.0540	
Dmin:	3.00 in	
D50:	6.00 in	
Dmax:	9.00 in	

Downdrain/Flume F-3 -- 25-Yr, 24-Hr Peak Flow -- 25% Slope

Material: Riprap

Trapezoidal Channel

Bottom Width (ft)	Left Sideslope Ratio	Right Sideslope Ratio	Slope (%)	Freeboard Depth (ft)	Freeboard % of Depth	Freeboard Mult. x (VxD)
2.00	2.0:1	2.0:1	25.0	0.10		

PADER Method - Steep Slope Design

	w/o Freeboard	w/ Freeboard
Design Discharge:	22.40 cfs	
Depth:	0.75 ft	0.85 ft
Top Width:	4.99 ft	5.39 ft
Velocity:	8.57 fps	
X-Section Area:	2.61 sq ft	
Hydraulic Radius:	0.489	
Froude Number:	2.09	
Manning's n:	0.0540	
Dmin:	3.00 in	
D50:	6.00 in	
Dmax:	9.00 in	

Appendix C.5
Surface Water Culvert Analysis

CALCULATION SHEET

Project No. 60197394

Client	County of Kaua'I	**Subject**	Culverts	**Prepared By** NKW	**Date** 8/26/15
Project	Kekaha Landfill - ER			**Reviewed By** DFP	**Date** 12/15/15
Phase II, Cell 2 Lateral Exp.				**Approved By** NKW	**Date** 12/15/15

OBJECTIVE

Design culverts under the final cover access road and under the perimeter access road. These pipes will convey surface water runoff from the downdrain channels to the infiltration channels and basin system.

DESIGN CRITERIA AND ASSUMPTIONS

1. Culverts will be sized to convey the greater of the peak flow from the 100-year, 1-hour storm event or the 25-year, 24-hour storm event.
2. Assume corrugated metal pipe (CMP) will be used for all culverts.
3. Use a Manning's Roughness Coefficient "n" of 0.024 for corrugated metal pipe.
4. Assume flared end sections will be attached at both inlet and outlet ends of the culverts to provide a smooth transition of flow.
5. Assume rip rap will be used for erosion protection at the culvert inlet and outlet. Alternative materials to riprap, such as a geotextile erosion control mat, may be used to provide erosion protection.
6. Culverts recommended in these calculations may be substituted with a structure that provides equal hydraulic performance (i.e., box culvert, pipe arches, etc.)
7. Culvert slope is assumed, actual slopes and resulting capacity to be field verified.
8. Culverts will be sized using the SEDCAD+ utilities culvert design program. Output is attached.
9. Assume a minimum of 1 foot and 10 inches of cover material over the top of culverts (10 inches gravel road plus 1 foot of cover). All culverts were designed to convey flow through the culvert without overtopping the road.
10. Tailwater depths for the culverts were calculated based on free outlet conditions since the high water level in the infiltration channel and basin system is lower than the invert at the culvert outlets.
11. All references to elevations correlate to the site-specific vertical datum.

CALCULATIONS

The expected flow rate to each culvert is equal to the discharge from each downdrain channel. These flow rates are presented within the SEDCAD+ output for the infiltration basin system analyses included in Appendix C.2. Culverts will be field fit according to location. Length will vary from approximately 20 to 30 feet.

Culvert B

One 18-inch diameter culvert conveys 13.2 cfs without overtopping the road. Culvert B must pass 12.30 cfs. 13.2 > 12.30. Therefore, Culvert B will pass the design flow rate.

Culvert C-1

One 12-inch diameter culvert conveys 5.42 cfs without overtopping the road. Culvert C-1 must pass 4.53 cfs. 5.42 > 4.53. Therefore, Culvert C-1 will pass the design flow rate.

Culvert C-2

Three 15-inch diameter culverts convey 39.6 cfs without overtopping the road. Culvert C-2 must pass 34.6 cfs. 39.6 > 34.6. Therefore, Culvert C-2 will pass the design flow rate.

Culvert F

Five 18-inch diameter culverts convey 65.95 cfs without overtopping the road. Culvert F must pass 62.3 cfs. 65.95 > 62.3. Therefore, Culvert F will pass the design flow rate.

Culvert G

Three 15-inch diameter culverts convey 25.6 cfs without overtopping the road. Culvert G must pass 18.6 cfs. 25.6 > 18.6. Therefore, Culvert G will pass the design flow rate.

CONCLUSIONS

Structure Number	Culvert ID	Number of Culverts	Culvert Diameter (inches)	Culvert Slope (%)	Headwater Elevation (ft)	Design Flow (cfs)	Peak Flow During 100-Year, 1-Hour Storm (cfs)	Peak Flow During 25-Year, 24-Hour Storm (cfs)
28	B	1	18	6	3.33	**12.3**	12.28	12.13
21	C-1	1	12	5	2.83	**4.53**	4.53	4.48
27	C-2	3	18	6	3.33	**34.6**	27.62	34.57
40	F	5	18	3	3.33	**62.3**	49.36	62.25
31	G	3	15	3	3.08	**18.4**	17.56	18.38

Notes:

1. Place riprap at culvert inlet and outlet locations.
2. Install flared end sections at both inlet and outlet ends.
3. Culverts recommended in these calculations may be substituted with an equivalent structure that provides equal hydraulic performance (i.e., box culvert, pipe arch, etc.).
4. Culvert slope is approximate. Actual slopes at final design will need to verify adequacy of pipe sizes.
5. Headwater is assumed to be pipe size plus 1'-10". Actual headwater will need to verified at final design to verify adequacy of pipe sizes.

Culvert B -- One 18-inch Culvert

Culvert Inputs:

Length (ft)	Slope (%)	Manning's n	Max. Headwater (ft)	Tailwater (ft)	Entrance Loss Coef. (Ke)
30.00	6.00	0.0240	3.33	0.00	0.50

Culvert Results:

Minimum pipe diameter: 1 - 18 inch pipe(s) required

Detailed Performance Curves

Design Discharge = 12.30 cfs

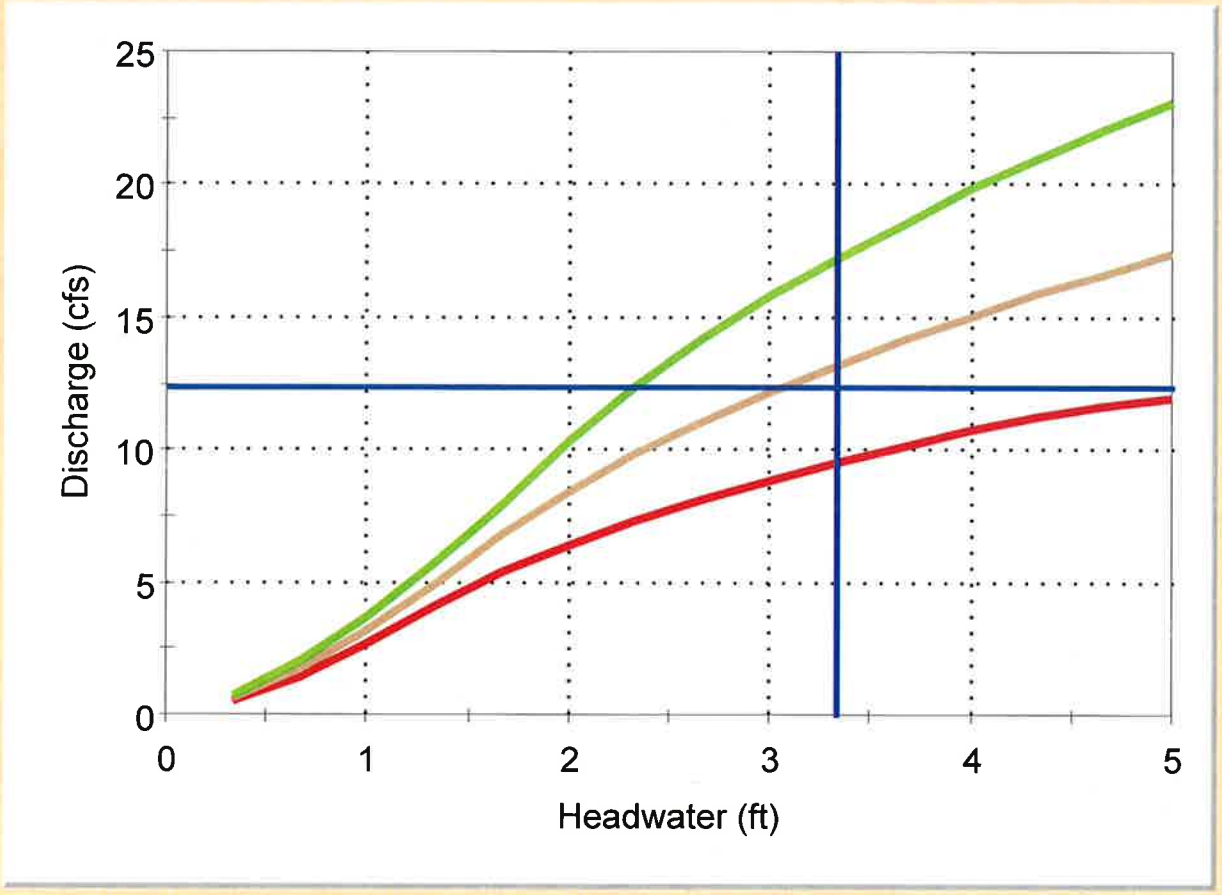
Maximum Headwater = 3.33 ft

(BOLD indicates design pipe size)

Headwater (ft)	Discharge (cfs) (15 in)	**Discharge (cfs) (18 in)**	Discharge (cfs) (21 in)
0.33	0.52	**0.62**	0.72
0.67	1.43	**1.71**	2.00
1.00	2.62	**3.14**	3.66
1.33	4.03	**4.83**	5.63
1.66	5.34	**6.75**	7.87
2.00	6.40	**8.42**	10.35
2.33	7.30	**9.83**	12.38
2.66	8.11	**11.06**	14.17
3.00	8.84	**12.17**	15.77
3.33	9.51	**13.19**	17.21
3.66	10.15	**14.12**	18.55
4.00	10.74	**15.01**	19.80
4.33	11.24	**15.85**	20.96
4.66	11.62	**16.64**	22.07
4.99	11.97	**17.39**	23.12

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Culvert Performance Curves - Structure # 0



Culvert C-1 -- One 12-inch Culvert

Culvert Inputs:

Length (ft)	Slope (%)	Manning's n	Max. Headwater (ft)	Tailwater (ft)	Entrance Loss Coef. (Ke)
30.00	5.00	0.0240	2.83	0.00	0.50

Culvert Results:

Minimum pipe diameter: 1 - 12 inch pipe(s) required

Detailed Performance Curves

Design Discharge = 4.53 cfs

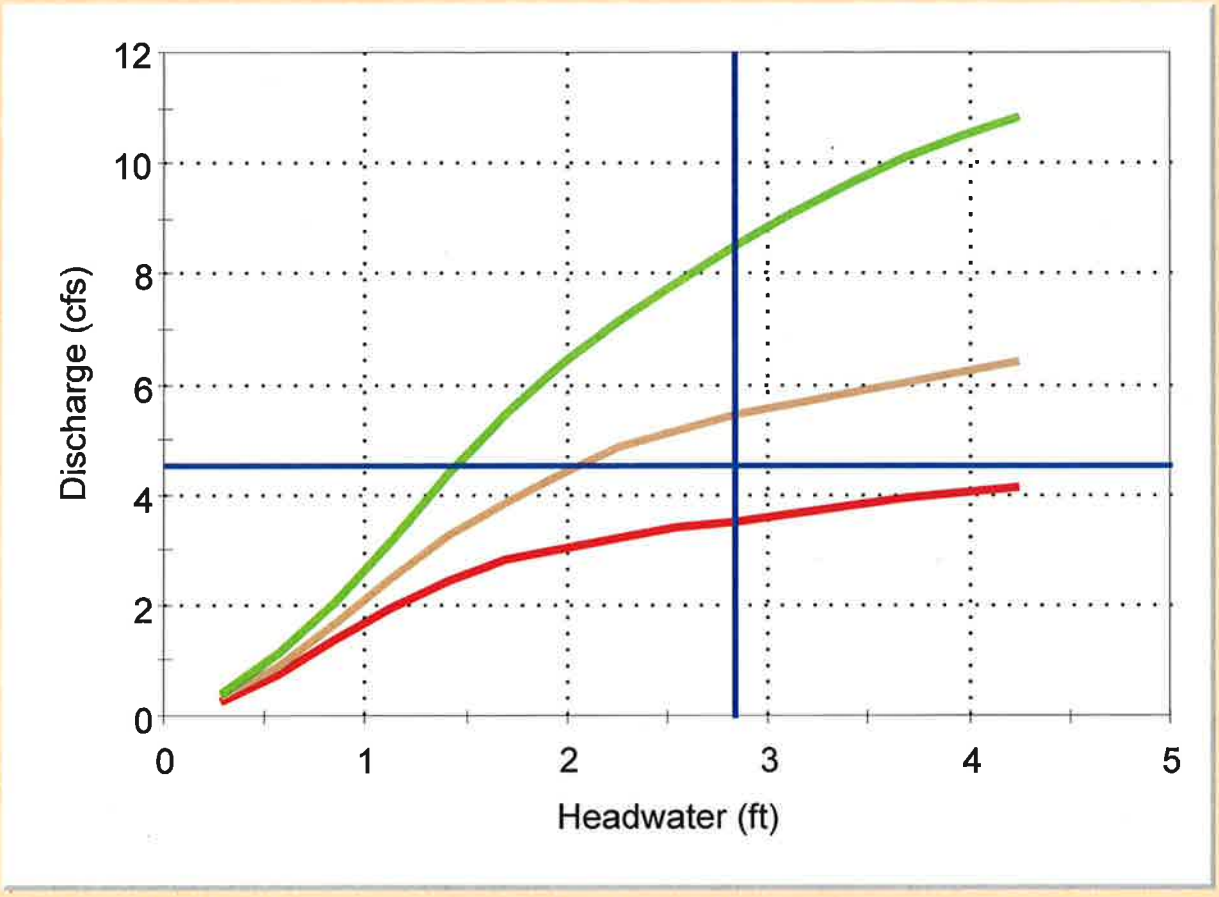
Maximum Headwater = 2.83 ft

(BOLD indicates design pipe size)

Headwater (ft)	Discharge (cfs) (10 in)	**Discharge (cfs) (12 in)**	Discharge (cfs) (15 in)
0.28	0.26	**0.32**	0.41
0.57	0.75	**0.90**	1.12
0.85	1.36	**1.64**	2.05
1.13	1.97	**2.50**	3.16
1.41	2.44	**3.25**	4.39
1.70	2.82	**3.86**	5.45
1.98	3.02	**4.38**	6.35
2.26	3.22	**4.84**	7.13
2.55	3.38	**5.16**	7.83
2.83	3.52	**5.42**	8.48
3.11	3.65	**5.63**	9.08
3.40	3.79	**5.84**	9.64
3.68	3.91	**6.04**	10.11
3.96	4.03	**6.24**	10.49
4.24	4.15	**6.43**	10.82

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Culvert Performance Curves - Structure # 0



Culvert C-2 -- Three 18-inch Culverts

Culvert Inputs:

Length (ft)	Slope (%)	Manning's n	Max. Headwater (ft)	Tailwater (ft)	Entrance Loss Coef. (Ke)
30.00	6.00	0.0240	3.33	0.00	0.50

Culvert Results:

Minimum pipe diameter: 1 - 18 inch pipe(s) required

Detailed Performance Curves

Design Discharge = 11.53 cfs

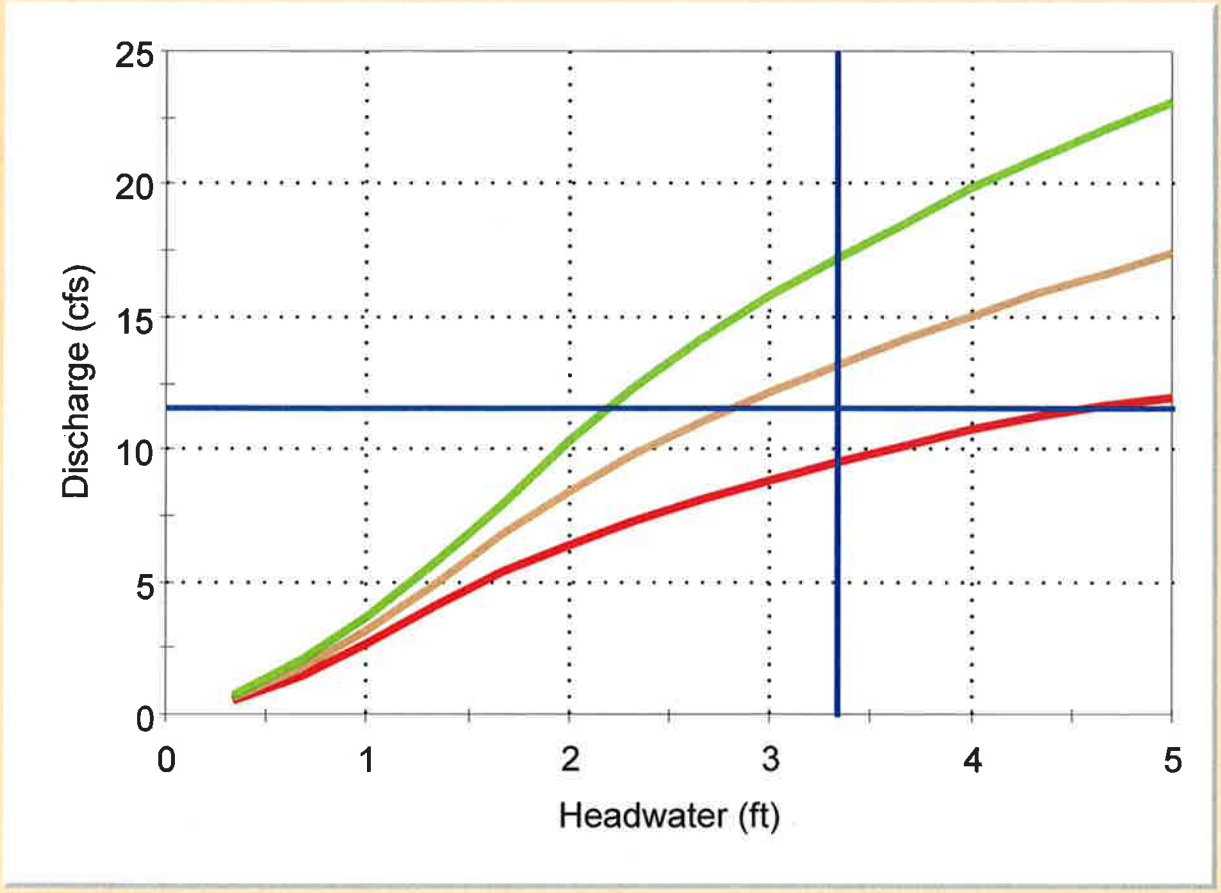
Maximum Headwater = 3.33 ft

(BOLD indicates design pipe size)

Headwater (ft)	Discharge (cfs) (15 in)	**Discharge (cfs) (18 in)**	Discharge (cfs) (21 in)
0.33	0.52	**0.62**	0.72
0.67	1.43	**1.71**	2.00
1.00	2.62	**3.14**	3.66
1.33	4.03	**4.83**	5.63
1.66	5.34	**6.75**	7.87
2.00	6.40	**8.42**	10.35
2.33	7.30	**9.83**	12.38
2.66	8.11	**11.06**	14.17
3.00	8.84	**12.17**	15.77
3.33	9.51	**13.19**	17.21
3.66	10.15	**14.12**	18.55
4.00	10.74	**15.01**	19.80
4.33	11.24	**15.85**	20.96
4.66	11.62	**16.64**	22.07
4.99	11.97	**17.39**	23.12

8/12

Culvert Performance Curves - Structure # 0



Culvert F -- Five 18-inch Culverts

Culvert Inputs:

Length (ft)	Slope (%)	Manning's n	Max. Headwater (ft)	Tailwater (ft)	Entrance Loss Coef. (Ke)
30.00	3.00	0.0240	3.33	0.00	0.50

Culvert Results:

Minimum pipe diameter: 1 - 18 inch pipe(s) required

Detailed Performance Curves

Design Discharge = 12.46 cfs

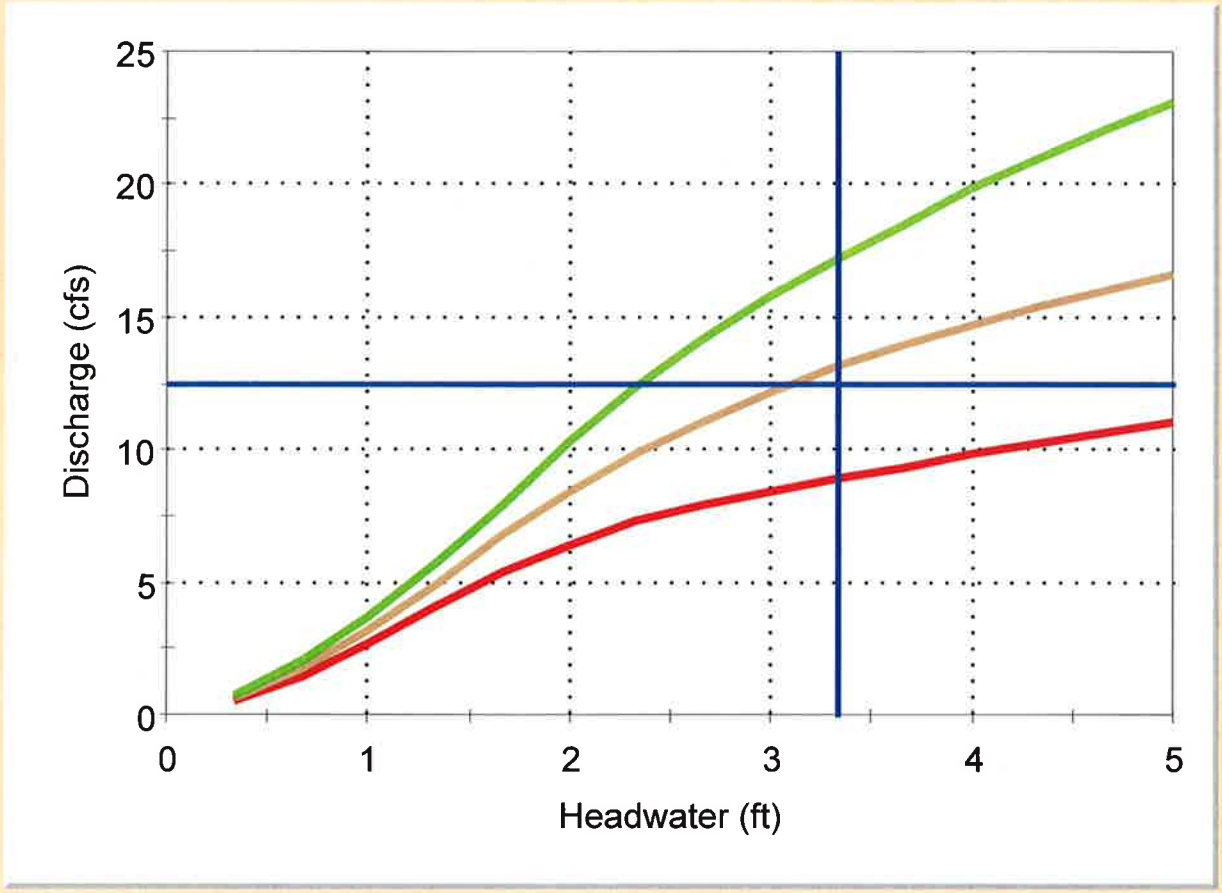
Maximum Headwater = 3.33 ft

(BOLD indicates design pipe size)

Headwater (ft)	Discharge (cfs) (15 in)	**Discharge (cfs) (18 in)**	Discharge (cfs) (21 in)
0.33	0.52	**0.62**	0.72
0.67	1.43	**1.71**	2.00
1.00	2.62	**3.14**	3.66
1.33	4.03	**4.83**	5.63
1.66	5.34	**6.75**	7.87
2.00	6.40	**8.42**	10.35
2.33	7.30	**9.83**	12.38
2.66	7.91	**11.06**	14.17
3.00	8.42	**12.17**	15.77
3.33	8.90	**13.19**	17.21
3.66	9.35	**14.01**	18.55
4.00	9.79	**14.71**	19.80
4.33	10.21	**15.36**	20.96
4.66	10.60	**16.00**	22.07
4.99	10.99	**16.61**	23.12

10/12

Culvert Performance Curves - Structure # 0



Culvert G -- Three 15-inch Culverts

Culvert Inputs:

Length (ft)	Slope (%)	Manning's n	Max. Headwater (ft)	Tailwater (ft)	Entrance Loss Coef. (Ke)
30.00	3.00	0.0240	3.08	0.00	0.50

Culvert Results:

Minimum pipe diameter: 1 - 15 inch pipe(s) required

Detailed Performance Curves

Design Discharge = 6.13 cfs

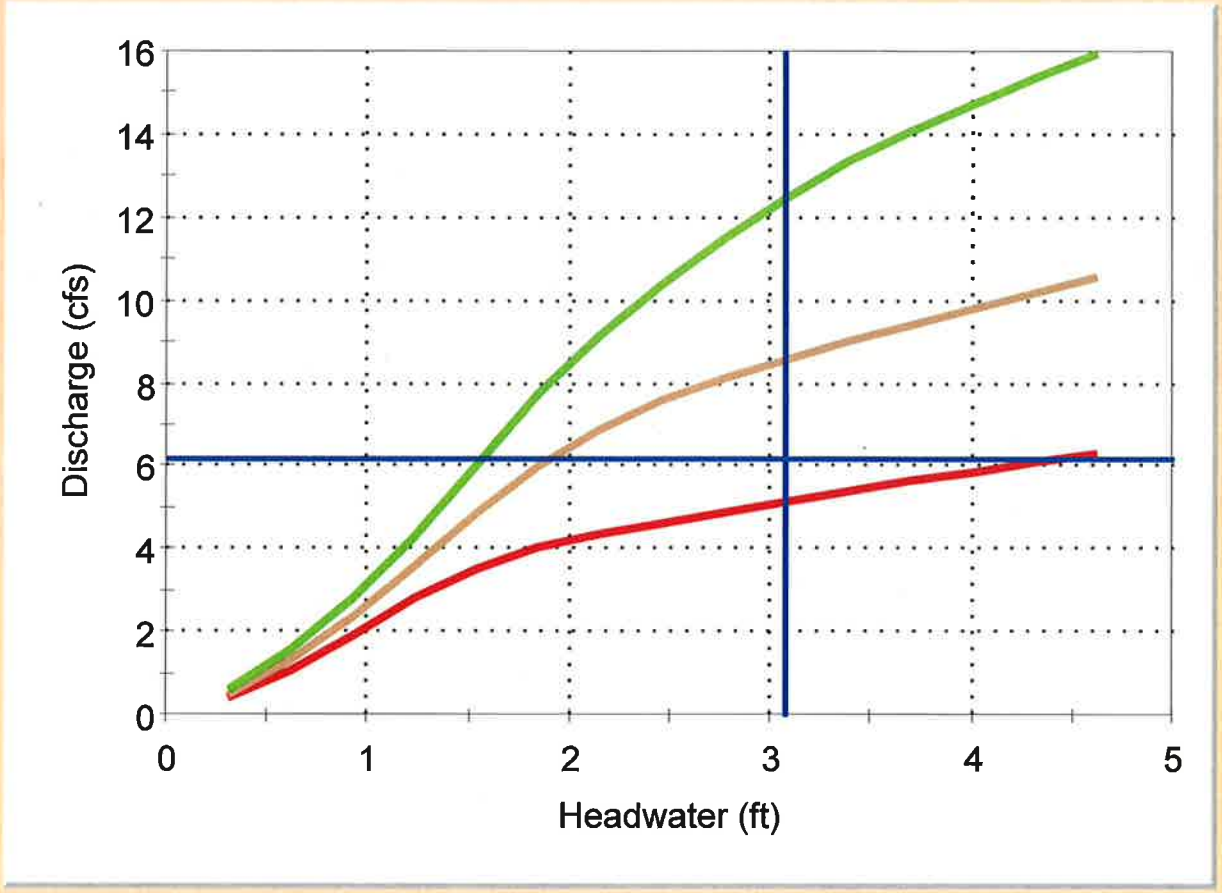
Maximum Headwater = 3.08 ft

(BOLD indicates design pipe size)

Headwater (ft)	Discharge (cfs) (12 in)	**Discharge (cfs) (15 in)**	Discharge (cfs) (18 in)
0.31	0.37	**0.47**	0.56
0.62	1.02	**1.27**	1.53
0.92	1.86	**2.33**	2.79
1.23	2.80	**3.58**	4.29
1.54	3.53	**4.88**	6.00
1.85	3.99	**5.94**	7.72
2.16	4.32	**6.84**	9.12
2.46	4.60	**7.55**	10.34
2.77	4.88	**8.07**	11.43
3.08	5.14	**8.54**	12.43
3.39	5.39	**8.98**	13.35
3.70	5.62	**9.40**	14.08
4.00	5.85	**9.80**	14.72
4.31	6.07	**10.18**	15.33
4.62	6.28	**10.55**	15.92

12/12

Culvert Performance Curves - Structure # 0



Appendix D
Leachate Collection System Calculations

Appendix D
Leachate Collection System Calculations

Appendix D.1
Leachate Generation Analysis

Appendix D.2
Leachate Collection Pipe

Appendix D.2A
Leachate Collection Pipe Flow Capacity Analysis

Appendix D.2B
Leachate Collection Pipe Flow Perforation Sizing

Appendix D.3
Leachate Collection and Extraction Riser Pipe Strength Analysis

Appendix D.4
Leachate Evaporation Pond Analysis

Appendix D.5
Leachate Flows – Cell 2

Appendix D.6
Pump Sizing – Cell 2 Pumps

Appendix D.7
Final Cover Equivalency

Appendix D.1
Leachate Generation Analysis

Client County of Kaua'i	**Subject** Leachate	**Prepared By**	NKW	**Date**	12/10/15
Project Kekaha Landfill	Generation Analysis	**Reviewed By**	DFP	**Date**	12/11/15
Cell 2, Phase II		**Approved By**	NKW	**Date**	12/14/15

LEACHATE GENERATION ANALYSIS

Objective

Estimate the quantity of leachate generated for the entire Kekaha Landfill during filling of Phase II, Cell 2, using the USEPA Hydraulic Evaluation of Landfill Performance Version 3.07 (HELP) computer modeling program and site data.

Design Criteria and Assumptions

1. This analysis was performed to estimate total site leachate generation rates including Cell 1, Cell 2, and Phase II based on total site development conditions that yield the greatest leachate quantities.

2. Active conditions are to be modeled as a conservative analysis. Active conditions refer to the situation of all of the base areas developed and active waste placement and with 12 inches of soil cover in place (no final cover constructed).

3. All references to elevations correlate to mean sea level (msl) datum.

4. The actual final cover elevation is 120 feet.

5. The average leachate generation rate from the existing landfill from September 2004 through December 2010 (approximately 610,000 gallons per year) is used to estimate leachate generation from the existing landfill. This quantity includes leachate generated from the active area of the existing landfill Phase II but does not include Cell 1.

HELP Model Version 3.07 Input:

6. Solar radiation data can be default data, user input or synthetically generated by the HELP model. Synthetically generated solar radiation data for Honolulu, Hawaii was used to simulate site weather conditions.

7. Temperature, precipitation, and average wind speed data was obtained from site records from January 2001 through December 2010. Missing data is viewed by the model as a "0". Therefore, to avoid skewing the data, missing temperature data for 2001, 2002, and 2008 was filled in using temperatures from days adjacent to the missing data. The data missing in 2004 was filled in using data from the same dates in 2003. Data is missing for a total of 5 days in April of 2001 and 2002. Data from 2004 is missing from April 9 through May 3, and May 26 through June 19. For 2008, data is missing from January 30 through February 3. There is no missing data from 2003, 2005, 2006, 2007, 2009, and 2010.

8. Geomembrane liner pinhole density and size were assumed to account for possible manufacturing defects during geomembrane production. The HELP Model User's Guide for Version 3.07 states that the pinhole density for a typical geomembrane is 0.5 to 1 pinhole per acre. One pinhole per acre was assumed to be present.

9. The placement quality for the geomembrane liner was assumed to be "good". A placement quality of "good" according to the HELP Model User's Guide "assumes good field installation with well prepared, smooth soil surface and geomembrane wrinkle control". The HELP Model User's Guide suggests 1 to 4 installation defects for an installation quality of "good". Two installation defects per acre were assumed for the geomembrane liner. This also reflects full time CQA during geomembrane installation.

10. For the Cell 2 analysis, it is assumed that 50 percent of the area will allow rainwater runoff. This assumption is based on anticipated areas of intermediate cover that will route surface water off the landfill.

11. For the Cell 1 analysis, it is assumed that 100 percent of the area will allow rainwater runoff. This scenario represents conditions after filling begins in Cell 2 when final grades are reached in Cell 1 but prior to final cover placement.

12. The HELP model was used to calculate the curve number for the cell based on the intermediate cover soil type, a slope of 28.5 percent (3.5:1) and a typical slope length of 150 feet.

13. The initial moisture content of the waste layer was conservatively assumed to be at field capacity.

14. Soil layers were generally modeled using HELP model default soil characteristics (porosity, field capacity, wilting point, saturated hydraulic conductivity). The exceptions are the geosynthetic clay liner (bentonite mat) and the operations layer. The default saturated hydraulic conductivity of the bentonite mat is adjusted to reflect current manufacturer data. The operations layer is assumed to have a hydraulic conductivity of 6.0×10^{-3} cm/sec.

15. The longest flow path and shallowest slope along the base liner was input, resulting in the highest potential leachate head estimate. For the Cell 1 liner, the longest flow is 260 feet and the minimum slope is approximately 2.8 percent. The longest flow path occurs in Cell 1a. For the Cell 2 liner, the longest flow path is 190 feet and the minimum slope is approximately 2.8 percent. The longest slope occurs in the west subcell.

16. A 1-acre design area was used for modeling purposes to compute unit quantities.

17. Active Condition Inputs (see also Table 1):

    - For the elevation 120 final cover: 104 feet of waste (assumed maximum waste depth in Cells 1 and 2)
    - Bare ground conditions (no vegetation)
    - Evaporative zone depth = 10 in. (recommended by HELP Model for bare ground for Honolulu, Hawaii)
    - Maximum leaf area index = 0 (recommended by HELP Model for bare ground)
    - Fraction of area allowing runoff = 100 percent (for Cell 1), 50 percent (for Cell 2)
    - Runoff curve number = 95.5 (calculated by HELP model)
    - Length of model run = 10 years

18. The area of the Cell 1 and Cell 2 lateral expansions to Phase II are 6.4 and 6.6 acres respectively.

Help Model General Layout

Table 1: Active Condition – Cells 1 and 2 HELP Model Layout

(Layer Number) Layer Description	Flow	Thickness	Saturated Hydraulic Conductivity	Soil Type/ Texture Number
(1)vertical percolation	↓	12 inches	$4.2x10^{-5}$ cm/sec	Intermediate cover/#12
(2)vertical percolation	↓	1,248 inches	$1.0x10^{-3}$ cm/sec	waste/#18
(3)vertical percolation	↓	24 inches	$6.0x10^{-3}$ cm/sec	operations layer/ #61
(4)lateral drainage	→	12 inches	$3.0x10^{-1}$ cm/sec	gravel/ #21
(5)60-mil geomembrane Liner	↓	0.06 inch	$2.0x10^{-13}$ cm/sec	geomembrane/#35
(6)Geosynthetic Clay Layer	↓	0.24 inches	$5.0x10^{-9}$ cm/sec	Bentonite mat/#62

Table 2: Soil Texture Properties

Soil Texture	Soil Classification		Comments, Properties, and Uses
	USDA	USCS	
62	---	---	Based on HELP model default parameter for a bentonite mat (0.6 cm or 0.24 inches thick) with the hydraulic conductivity changed from $3x10^{-9}$ cm/sec to $5x10^{-9}$ cm/sec.
61	S	SW	Operations layer. Based on HELP model default soil parameter No. 2 with the hydraulic conductivity changed from $5.8x10^{-3}$ cm/sec to $6x10^{-3}$ cm/sec.
35	---	---	HELP model default parameters for High Density Polyethylene (HDPE) geomembrane.
21	---	---	Gravel drainage layer. HELP model default soil parameter for gravel, hydraulic conductivity of $3.0x10^{-1}$ cm/sec
18	---	---	HELP model default parameter for municipal solid waste.
12	SiCL	CL	HELP model default soil used to model intermediate cover.

Note: All soil properties are defaults of the HELP Model, Version 3.07 unless otherwise designated.

Calculations

Calculations performed by the HELP Model are included in the output located in the following pages. See Table 3 for the modeling results based on peak daily, average monthly, and peak average annual values. The HELP Model presents peak daily results in inches per day and cubic feet per day. Results for average monthly values are based on the average values for the month with the highest leachate quantity (January for Cell 1 and February for Cell 2). The HELP Model presents the average annual results in inches per year and cubic feet per year. See the sample conversions below for Cell .

Peak daily lateral drainage collected from layer 4 (leachate quantity) for the 1-acre design area:

$$\left(\frac{472.69\,\text{cf}}{\text{day-acre}}\right)\left(\frac{7.48\,\text{gallons}}{1\,\text{cf}}\right) = 3{,}535.8\,\text{gal/acre/day}$$

Average monthly lateral drainage collected from layer 4 (leachate quantity) for the 1-acre design area:

$$\left(\frac{0.3377\,\text{inches}}{\text{month}}\right)\left(\frac{1\,\text{foot}}{12\,\text{inches}}\right)\left(\frac{43{,}560\,\text{sf}}{\text{acre}}\right)\left(\frac{7.48\,\text{gallons}}{1\,\text{cf}}\right)\left(\frac{\text{month}}{28\,\text{days (for February)}}\right) = 327.5\,\text{gal/acre/day}$$

Average annual lateral drainage collected from layer 4 (leachate quantity) for the 1-acre design area:

$$\left(\frac{6{,}130\,\text{cf}}{\text{year–acre}}\right)\left(\frac{7.48\,\text{gallons}}{1\,\text{cf}}\right)\left(\frac{\text{year}}{365\,\text{days}}\right) = 125.6\,gal/acre/day$$

Table 3: Elevation 120 Final Cover
Active Condition Calculated Results From HELP Analysis Output

Parameter	Peak Daily Value	Average Monthly Value	Peak Average Annual Value
Cell 1: Leachate Head on Base (filling at final grade)	1.349 inches	0.0595 inches	0.025 inches
Cell 1: Leachate Quantity (filling at final grade)	3,535.8 gallons/acre/day	327.5 gallons/acre/day	125.6 gallons/acre/day
Cell 2: Leachate Head on Base	1.031 inches	0.0950 inches	0.043 inches
Cell 2: Leachate Quantity	3,704.8 gallons/acre/day	653.2 gallons/acre/day	293.4 gallons/acre/day

NOTE: Refer to pages 4 through 6 of each of the attached HELP Model output (filenames: CELL1A.OUT, and CELL2A.OUT).

Daily Lateral Expansion Leachate Estimation:

Based on peak daily leachate generation:
Cell 1: 6.4 acres x 3,535.8 gal/acre/day = ***22,629 gal/day***
Cell 2: 6.6 acres x 3,704.8 gal/acre/day = ***24,452 gal/day***

Based on average monthly leachate generation:
Cell 1: 6.4 acres x 327.5 gal/acre/day = ***2,096 gal/day***
Cell 2: 6.6 acres x 653.2 gal/acre/day = ***4,311 gal/day***

Based on peak average annual leachate generation:
Cell 1: 6.4 acres x 125.6 gal/acre/day = ***803.8 gal/day***
Cell 2: 6.6 acres x 293.4 gal/acre/day = **1,936.4 gal/day**

Existing Landfill (Except Ph. II, Cell 1) Leachate Generation = 610,000 gal/yr or 1,671 gal/day

Estimated Total Site Leachate Generation:
22,629 gal/day + 24,452 gal/day + 1,671 gal/day = 48,752 gal/day

Conclusions

Active conditions when Cell 1 is at final grade with intermediate cover in place and Cell 2 of Phase II is actively receiving waste, final cover elevation 120 feet, results in the largest leachate generation. The estimated maximum daily total leachate generation for Phase II, including Cells 1 and 2, is approximately 48,800 gallons

These values are estimated using a waste with an initial water content at field capacity. The HELP Model was used to calculate leachate volumes and should be considered only an estimate of potential field conditions.

ATTACHMENT 1

EL. 120 FINAL COVER HELP MODEL OUTPUT
(FILENAME: CELL1A.OUT)

```
**********************************************************************
**********************************************************************
**                                                                  **
**                                                                  **
**              HYDROLOGIC EVALUATION OF LANDFILL PERFORMANCE        **
**                HELP MODEL VERSION 3.07  (1 NOVEMBER 1997)        **
**                  DEVELOPED BY ENVIRONMENTAL LABORATORY           **
**                    USAE WATERWAYS EXPERIMENT STATION             **
**             FOR USEPA RISK REDUCTION ENGINEERING LABORATORY       **
**                                                                  **
**                                                                  **
**********************************************************************
**********************************************************************

PRECIPITATION DATA FILE:    c:\temp\KEK10YR.D4
TEMPERATURE DATA FILE:      C:\temp\KEK10YR.D7
SOLAR RADIATION DATA FILE:  c:\temp\KEK10YR.D13
EVAPOTRANSPIRATION DATA:    c:\temp\KEK10YR.D11
SOIL AND DESIGN DATA FILE:  c:\temp\CELL1A.D10
OUTPUT DATA FILE:           c:\temp\CELL1A.OUT

TIME:  15: 2     DATE:   8/ 2/2012

**********************************************************************

     TITLE:  Kekaha Landfill Phase II- Cell 1, Final Grade, Active Cond.

**********************************************************************

     NOTE:  INITIAL MOISTURE CONTENT OF THE LAYERS AND SNOW WATER WERE
              COMPUTED AS NEARLY STEADY-STATE VALUES BY THE PROGRAM.

                                LAYER  1
                                --------

                TYPE 1 - VERTICAL PERCOLATION LAYER
                    MATERIAL TEXTURE NUMBER  12
          THICKNESS                   =     12.00   INCHES
          POROSITY                    =      0.4710 VOL/VOL
          FIELD CAPACITY              =      0.3420 VOL/VOL
          WILTING POINT               =      0.2100 VOL/VOL
          INITIAL SOIL WATER CONTENT  =      0.2623 VOL/VOL
          EFFECTIVE SAT. HYD. COND.   =  0.419999997000E-04 CM/SEC
```

LAYER 2

TYPE 1 - VERTICAL PERCOLATION LAYER
MATERIAL TEXTURE NUMBER 18

THICKNESS	=	1248.00	INCHES
POROSITY	=	0.6710	VOL/VOL
FIELD CAPACITY	=	0.2920	VOL/VOL
WILTING POINT	=	0.0770	VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.2920	VOL/VOL
EFFECTIVE SAT. HYD. COND.	=	0.100000005000E-02	CM/SEC

LAYER 3

TYPE 1 - VERTICAL PERCOLATION LAYER
MATERIAL TEXTURE NUMBER 61

THICKNESS	=	24.00	INCHES
POROSITY	=	0.4370	VOL/VOL
FIELD CAPACITY	=	0.0620	VOL/VOL
WILTING POINT	=	0.0240	VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.0705	VOL/VOL
EFFECTIVE SAT. HYD. COND.	=	0.600000005000E-02	CM/SEC

LAYER 4

TYPE 2 - LATERAL DRAINAGE LAYER
MATERIAL TEXTURE NUMBER 21

THICKNESS	=	12.00	INCHES
POROSITY	=	0.3970	VOL/VOL
FIELD CAPACITY	=	0.0320	VOL/VOL
WILTING POINT	=	0.0130	VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.0320	VOL/VOL
EFFECTIVE SAT. HYD. COND.	=	0.300000012000	CM/SEC
SLOPE	=	2.80	PERCENT
DRAINAGE LENGTH	=	260.0	FEET

LAYER 5

TYPE 4 - FLEXIBLE MEMBRANE LINER
MATERIAL TEXTURE NUMBER 35

THICKNESS	=	0.06	INCHES
POROSITY	=	0.0000	VOL/VOL
FIELD CAPACITY	=	0.0000	VOL/VOL
WILTING POINT	=	0.0000	VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.0000	VOL/VOL
EFFECTIVE SAT. HYD. COND.	=	0.199999996000E-12	CM/SEC

FML PINHOLE DENSITY	=	1.00	HOLES/ACRE
FML INSTALLATION DEFECTS	=	2.00	HOLES/ACRE
FML PLACEMENT QUALITY	=	3 - GOOD	

LAYER 6

TYPE 3 - BARRIER SOIL LINER
MATERIAL TEXTURE NUMBER 62

THICKNESS	=	0.24	INCHES
POROSITY	=	0.7500	VOL/VOL
FIELD CAPACITY	=	0.7470	VOL/VOL
WILTING POINT	=	0.4000	VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.7500	VOL/VOL
EFFECTIVE SAT. HYD. COND.	=	0.499999997000E-08	CM/SEC

GENERAL DESIGN AND EVAPORATIVE ZONE DATA

NOTE: SCS RUNOFF CURVE NUMBER WAS COMPUTED FROM DEFAULT SOIL DATA BASE USING SOIL TEXTURE #12 WITH BARE GROUND CONDITIONS, A SURFACE SLOPE OF 28.% AND A SLOPE LENGTH OF 150. FEET.

SCS RUNOFF CURVE NUMBER	=	95.50	
FRACTION OF AREA ALLOWING RUNOFF	=	100.0	PERCENT
AREA PROJECTED ON HORIZONTAL PLANE	=	1.000	ACRES
EVAPORATIVE ZONE DEPTH	=	10.0	INCHES
INITIAL WATER IN EVAPORATIVE ZONE	=	2.463	INCHES
UPPER LIMIT OF EVAPORATIVE STORAGE	=	4.710	INCHES
LOWER LIMIT OF EVAPORATIVE STORAGE	=	2.100	INCHES
INITIAL SNOW WATER	=	0.000	INCHES
INITIAL WATER IN LAYER MATERIALS	=	369.820	INCHES
TOTAL INITIAL WATER	=	369.820	INCHES
TOTAL SUBSURFACE INFLOW	=	0.00	INCHES/YEAR

EVAPOTRANSPIRATION AND WEATHER DATA

NOTE: EVAPOTRANSPIRATION DATA WAS OBTAINED FROM HONOLULU HAWAII

STATION LATITUDE	=	21.33	DEGREES
MAXIMUM LEAF AREA INDEX	=	0.00	
START OF GROWING SEASON (JULIAN DATE)	=	0	
END OF GROWING SEASON (JULIAN DATE)	=	367	
EVAPORATIVE ZONE DEPTH	=	10.0	INCHES
AVERAGE ANNUAL WIND SPEED	=	3.47	MPH

AVERAGE 1ST QUARTER RELATIVE HUMIDITY = 72.00 %
AVERAGE 2ND QUARTER RELATIVE HUMIDITY = 66.00 %
AVERAGE 3RD QUARTER RELATIVE HUMIDITY = 66.00 %
AVERAGE 4TH QUARTER RELATIVE HUMIDITY = 70.00 %

NOTE: PRECIPITATION DATA FOR KEKAHA/SITE HAWAII
WAS ENTERED FROM AN ASCII DATA FILE.

NOTE: TEMPERATURE DATA FOR KEKAHA/SITE HAWAII
WAS ENTERED FROM AN ASCII DATA FILE.

NOTE: SOLAR RADIATION DATA WAS SYNTHETICALLY GENERATED USING
COEFFICIENTS FOR HONOLULU HAWAII
AND STATION LATITUDE = 21.33 DEGREES

****

AVERAGE MONTHLY VALUES IN INCHES FOR YEARS 2001 THROUGH 2010

	JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
PRECIPITATION						
TOTALS	2.17	1.55	3.20	0.84	0.45	0.73
	0.28	1.03	0.97	1.08	2.15	4.48
STD. DEVIATIONS	2.80	1.80	6.51	0.68	0.39	0.46
	0.25	1.33	0.63	1.60	2.10	5.94
RUNOFF						
TOTALS	0.648	0.622	1.532	0.155	0.031	0.108
	0.005	0.347	0.141	0.281	0.829	2.720
STD. DEVIATIONS	1.629	1.194	4.092	0.221	0.071	0.161
	0.012	0.761	0.187	0.756	1.384	4.305
EVAPOTRANSPIRATION						
TOTALS	1.257	0.964	1.207	0.787	0.414	0.476
	0.293	0.467	0.718	0.647	1.111	1.183
STD. DEVIATIONS	1.168	0.536	1.182	0.802	0.431	0.452
	0.079	0.630	0.664	0.698	0.749	0.955
LATERAL DRAINAGE COLLECTED FROM LAYER 4						
TOTALS	**0.3377**	0.2763	0.1038	0.2083	0.3185	0.0938
	0.0606	0.0473	0.0331	0.0622	0.0675	0.0796

STD. DEVIATIONS	0.6448	0.5880	0.1233	0.3660	0.6662	0.1007
	0.0508	0.0325	0.0220	0.0746	0.0854	0.1066

PERCOLATION/LEAKAGE THROUGH LAYER 6

TOTALS	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
STD. DEVIATIONS	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000

AVERAGES OF MONTHLY AVERAGED DAILY HEADS (INCHES)

DAILY AVERAGE HEAD ON TOP OF LAYER 5

AVERAGES	**0.0595**	0.0536	0.0183	0.0379	0.0561	0.0171
	0.0107	0.0083	0.0060	0.0110	0.0123	0.0140
STD. DEVIATIONS	0.1137	0.1148	0.0217	0.0667	0.1174	0.0183
	0.0090	0.0057	0.0040	0.0131	0.0156	0.0188

AVERAGE ANNUAL TOTALS & (STD. DEVIATIONS) FOR YEARS 2001 THROUGH 2010

	INCHES		CU. FEET	PERCENT
PRECIPITATION	18.94	(8.940)	68748.6	100.00
RUNOFF	7.420	(5.6631)	26936.16	39.181
EVAPOTRANSPIRATION	9.524	(2.4970)	34572.55	50.288
LATERAL DRAINAGE COLLECTED FROM LAYER 4	1.68860	(1.54981)	**6129.629**	8.91601
PERCOLATION/LEAKAGE THROUGH LAYER 6	0.00000	(0.00000)	0.017	0.00003
AVERAGE HEAD ON TOP OF LAYER 5	**0.025**	(0.023)		
CHANGE IN WATER STORAGE	0.306	(1.4981)	1110.21	1.615

PEAK DAILY VALUES FOR YEARS 2001 THROUGH 2010

	(INCHES)	(CU. FT.)
PRECIPITATION	10.69	38804.699
RUNOFF	9.982	36234.1172
DRAINAGE COLLECTED FROM LAYER 4	0.13022	**472.69492**
PERCOLATION/LEAKAGE THROUGH LAYER 6	0.000000	0.00086
AVERAGE HEAD ON TOP OF LAYER 5	0.712	
MAXIMUM HEAD ON TOP OF LAYER 5	**1.349**	
LOCATION OF MAXIMUM HEAD IN LAYER 4 (DISTANCE FROM DRAIN)	13.3 FEET	
SNOW WATER	0.00	0.0000

MAXIMUM VEG. SOIL WATER (VOL/VOL) 0.3976

MINIMUM VEG. SOIL WATER (VOL/VOL) 0.2100

*** Maximum heads are computed using McEnroe's equations. ***

Reference: Maximum Saturated Depth over Landfill Liner
by Bruce M. McEnroe, University of Kansas
ASCE Journal of Environmental Engineering
Vol. 119, No. 2, March 1993, pp. 262-270.

****
****

FINAL WATER STORAGE AT END OF YEAR 2010

LAYER	(INCHES)	(VOL/VOL)
1	3.9058	0.3255
2	365.2119	0.2926
3	3.1963	0.1332
4	0.3844	0.0320
5	0.0000	0.0000
6	0.1800	0.7500
SNOW WATER	0.000	

****
****

ATTACHMENT 2

EL. 120 FINAL COVER HELP MODEL OUTPUT
(FILENAME: CELL2A.OUT)

```
**********************************************************************
**********************************************************************
**                                                                  **
**                                                                  **
**              HYDROLOGIC EVALUATION OF LANDFILL PERFORMANCE        **
**                HELP MODEL VERSION 3.07  (1 NOVEMBER 1997)        **
**                  DEVELOPED BY ENVIRONMENTAL LABORATORY           **
**                    USAE WATERWAYS EXPERIMENT STATION             **
**             FOR USEPA RISK REDUCTION ENGINEERING LABORATORY       **
**                                                                  **
**                                                                  **
**********************************************************************
**********************************************************************


PRECIPITATION DATA FILE:    c:\temp\KEK10YR.D4
TEMPERATURE DATA FILE:      C:\temp\KEK10YR.D7
SOLAR RADIATION DATA FILE:  c:\temp\KEK10YR.D13
EVAPOTRANSPIRATION DATA:    c:\temp\KEK10YR.D11
SOIL AND DESIGN DATA FILE:  c:\temp\CELL2A.D10
OUTPUT DATA FILE:           c:\temp\CELL2A.OUT


TIME:  15: 3     DATE:   8/ 2/2012


**********************************************************************

     TITLE:  Kekaha Landfill Phase II- Cell 2, Active Condition

**********************************************************************


     NOTE:  INITIAL MOISTURE CONTENT OF THE LAYERS AND SNOW WATER WERE
              COMPUTED AS NEARLY STEADY-STATE VALUES BY THE PROGRAM.


                                LAYER  1
                                --------

                  TYPE 1 - VERTICAL PERCOLATION LAYER
                      MATERIAL TEXTURE NUMBER  12
          THICKNESS                    =     12.00   INCHES
          POROSITY                     =      0.4710 VOL/VOL
          FIELD CAPACITY               =      0.3420 VOL/VOL
          WILTING POINT                =      0.2100 VOL/VOL
          INITIAL SOIL WATER CONTENT   =      0.2534 VOL/VOL
          EFFECTIVE SAT. HYD. COND.    =  0.419999997000E-04 CM/SEC
```

LAYER 2

TYPE 1 - VERTICAL PERCOLATION LAYER
MATERIAL TEXTURE NUMBER 18

THICKNESS	=	1248.00	INCHES
POROSITY	=	0.6710	VOL/VOL
FIELD CAPACITY	=	0.2920	VOL/VOL
WILTING POINT	=	0.0770	VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.2920	VOL/VOL
EFFECTIVE SAT. HYD. COND.	=	0.100000005000E-02	CM/SEC

LAYER 3

TYPE 1 - VERTICAL PERCOLATION LAYER
MATERIAL TEXTURE NUMBER 61

THICKNESS	=	24.00	INCHES
POROSITY	=	0.4370	VOL/VOL
FIELD CAPACITY	=	0.0620	VOL/VOL
WILTING POINT	=	0.0240	VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.0903	VOL/VOL
EFFECTIVE SAT. HYD. COND.	=	0.600000005000E-02	CM/SEC

LAYER 4

TYPE 2 - LATERAL DRAINAGE LAYER
MATERIAL TEXTURE NUMBER 21

THICKNESS	=	12.00	INCHES
POROSITY	=	0.3970	VOL/VOL
FIELD CAPACITY	=	0.0320	VOL/VOL
WILTING POINT	=	0.0130	VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.0320	VOL/VOL
EFFECTIVE SAT. HYD. COND.	=	0.300000012000	CM/SEC
SLOPE	=	2.80	PERCENT
DRAINAGE LENGTH	=	190.0	FEET

LAYER 5

TYPE 4 - FLEXIBLE MEMBRANE LINER
MATERIAL TEXTURE NUMBER 35

THICKNESS	=	0.06	INCHES
POROSITY	=	0.0000	VOL/VOL
FIELD CAPACITY	=	0.0000	VOL/VOL
WILTING POINT	=	0.0000	VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.0000	VOL/VOL
EFFECTIVE SAT. HYD. COND.	=	0.199999996000E-12	CM/SEC

FML PINHOLE DENSITY	=	1.00	HOLES/ACRE
FML INSTALLATION DEFECTS	=	2.00	HOLES/ACRE
FML PLACEMENT QUALITY	=	3 - GOOD	

LAYER 6

TYPE 3 - BARRIER SOIL LINER
MATERIAL TEXTURE NUMBER 62

THICKNESS	=	0.24	INCHES
POROSITY	=	0.7500	VOL/VOL
FIELD CAPACITY	=	0.7470	VOL/VOL
WILTING POINT	=	0.4000	VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.7500	VOL/VOL
EFFECTIVE SAT. HYD. COND.	=	0.499999997000E-08	CM/SEC

GENERAL DESIGN AND EVAPORATIVE ZONE DATA

NOTE: SCS RUNOFF CURVE NUMBER WAS COMPUTED FROM DEFAULT SOIL DATA BASE USING SOIL TEXTURE #12 WITH BARE GROUND CONDITIONS, A SURFACE SLOPE OF 28.% AND A SLOPE LENGTH OF 150. FEET.

SCS RUNOFF CURVE NUMBER	=	95.50	
FRACTION OF AREA ALLOWING RUNOFF	=	50.0	PERCENT
AREA PROJECTED ON HORIZONTAL PLANE	=	1.000	ACRES
EVAPORATIVE ZONE DEPTH	=	10.0	INCHES
INITIAL WATER IN EVAPORATIVE ZONE	=	2.357	INCHES
UPPER LIMIT OF EVAPORATIVE STORAGE	=	4.710	INCHES
LOWER LIMIT OF EVAPORATIVE STORAGE	=	2.100	INCHES
INITIAL SNOW WATER	=	0.000	INCHES
INITIAL WATER IN LAYER MATERIALS	=	370.189	INCHES
TOTAL INITIAL WATER	=	370.189	INCHES
TOTAL SUBSURFACE INFLOW	=	0.00	INCHES/YEAR

EVAPOTRANSPIRATION AND WEATHER DATA

NOTE: EVAPOTRANSPIRATION DATA WAS OBTAINED FROM HONOLULU HAWAII

STATION LATITUDE	=	21.33	DEGREES
MAXIMUM LEAF AREA INDEX	=	0.00	
START OF GROWING SEASON (JULIAN DATE)	=	0	
END OF GROWING SEASON (JULIAN DATE)	=	367	
EVAPORATIVE ZONE DEPTH	=	10.0	INCHES
AVERAGE ANNUAL WIND SPEED	=	3.47	MPH

AVERAGE 1ST QUARTER RELATIVE HUMIDITY = 72.00 %
AVERAGE 2ND QUARTER RELATIVE HUMIDITY = 66.00 %
AVERAGE 3RD QUARTER RELATIVE HUMIDITY = 66.00 %
AVERAGE 4TH QUARTER RELATIVE HUMIDITY = 70.00 %

NOTE: PRECIPITATION DATA FOR KEKAHA/SITE HAWAII
WAS ENTERED FROM AN ASCII DATA FILE.

NOTE: TEMPERATURE DATA FOR KEKAHA/SITE HAWAII
WAS ENTERED FROM AN ASCII DATA FILE.

NOTE: SOLAR RADIATION DATA WAS SYNTHETICALLY GENERATED USING
COEFFICIENTS FOR HONOLULU HAWAII
AND STATION LATITUDE = 21.33 DEGREES

****

AVERAGE MONTHLY VALUES IN INCHES FOR YEARS 2001 THROUGH 2010

	JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
PRECIPITATION						
TOTALS	2.17	1.55	3.20	0.84	0.45	0.73
	0.28	1.03	0.97	1.08	2.15	4.48
STD. DEVIATIONS	2.80	1.80	6.51	0.68	0.39	0.46
	0.25	1.33	0.63	1.60	2.10	5.94
RUNOFF						
TOTALS	0.371	0.337	0.826	0.078	0.016	0.055
	0.003	0.204	0.062	0.155	0.481	1.732
STD. DEVIATIONS	0.952	0.657	2.228	0.111	0.037	0.083
	0.006	0.475	0.077	0.422	0.841	2.956
EVAPOTRANSPIRATION						
TOTALS	1.177	0.978	1.318	0.851	0.549	0.419
	0.275	0.616	0.718	0.734	1.177	1.324
STD. DEVIATIONS	1.167	0.537	1.198	0.805	0.566	0.491
	0.124	0.698	0.650	0.846	0.593	1.011
LATERAL DRAINAGE COLLECTED FROM LAYER 4						
TOTALS	0.5898	**0.6736**	0.5197	0.4497	0.5213	0.3993
	0.2613	0.0942	0.1020	0.1030	0.1164	0.1137

STD. DEVIATIONS	1.1174	1.0305	0.9690	0.3862	1.0300	0.9433
	0.5966	0.1096	0.1197	0.1337	0.2142	0.1057

PERCOLATION/LEAKAGE THROUGH LAYER 6

TOTALS	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
STD. DEVIATIONS	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000

AVERAGES OF MONTHLY AVERAGED DAILY HEADS (INCHES)

DAILY AVERAGE HEAD ON TOP OF LAYER 5

AVERAGES	0.0760	**0.0950**	0.0669	0.0599	0.0671	0.0531
	0.0337	0.0121	0.0136	0.0133	0.0155	0.0147
STD. DEVIATIONS	0.1439	0.1456	0.1248	0.0514	0.1327	0.1256
	0.0768	0.0141	0.0159	0.0172	0.0285	0.0136

AVERAGE ANNUAL TOTALS & (STD. DEVIATIONS) FOR YEARS 2001 THROUGH 2010

	INCHES		CU. FEET	PERCENT
PRECIPITATION	18.94	(8.940)	68748.6	100.00
RUNOFF	4.320	(3.4689)	15682.85	22.812
EVAPOTRANSPIRATION	10.136	(2.7212)	36792.87	53.518
LATERAL DRAINAGE COLLECTED FROM LAYER 4	3.94408	(3.79058)	**14317.002**	20.82516
PERCOLATION/LEAKAGE THROUGH LAYER 6	0.00001	(0.00000)	0.025	0.00004
AVERAGE HEAD ON TOP OF LAYER 5	**0.043**	(0.042)		
CHANGE IN WATER STORAGE	0.539	(3.5681)	1955.80	2.845

PEAK DAILY VALUES FOR YEARS 2001 THROUGH 2010

	(INCHES)	(CU. FT.)
PRECIPITATION	10.69	38804.699
RUNOFF	7.054	25606.6113
DRAINAGE COLLECTED FROM LAYER 4	0.13644	**495.28577**
PERCOLATION/LEAKAGE THROUGH LAYER 6	0.000000	0.00065
AVERAGE HEAD ON TOP OF LAYER 5	0.545	
MAXIMUM HEAD ON TOP OF LAYER 5	**1.031**	
LOCATION OF MAXIMUM HEAD IN LAYER 4 (DISTANCE FROM DRAIN)	10.0 FEET	
SNOW WATER	0.00	0.0000

MAXIMUM VEG. SOIL WATER (VOL/VOL)	0.4253
MINIMUM VEG. SOIL WATER (VOL/VOL)	0.2100

*** Maximum heads are computed using McEnroe's equations. ***

Reference: Maximum Saturated Depth over Landfill Liner
by Bruce M. McEnroe, University of Kansas
ASCE Journal of Environmental Engineering
Vol. 119, No. 2, March 1993, pp. 262-270.

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FINAL WATER STORAGE AT END OF YEAR 2010

LAYER	(INCHES)	(VOL/VOL)
1	3.7858	0.3155
2	367.7162	0.2946
3	3.5107	0.1463
4	0.3841	0.0320
5	0.0000	0.0000
6	0.1800	0.7500
SNOW WATER	0.000	

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Appendix D.2
Leachate Collection Pipe

Appendix D.2A
Leachate Collection Pipe Flow Capacity Analysis

CALCULATION SHEET

AECOM

Project No. 60197394

Client County of Kaua'i	**Subject** Leachate Pipe	**Prepared By**	NKW	**Date**	12/10/15
Project Kekaha Landfill	Flow Capacity Analysis	**Reviewed By**	DFP	**Date**	12/11/15
Cell 2, Phase II		**Approved By**	NKW	**Date**	12/14/15

LEACHATE PIPE FLOW CAPACITY ANALYSIS

Objective

To evaluate the leachate collection and transfer pipe flow capacity of the proposed Cell 2 leachate management system and determine the maximum allowable area draining to the leachate collection and transfer pipes based on pipe capacity.

Design Criteria and Assumptions

1. Use Manning's equation to calculate the allowable leachate collection pipe flow.

2. The existing leachate collection and transfer pipe is an 8-inch HDPE SDR 9.3 pipe. The outside diameter (OD) of the pipe is about 8.625 inches, and the inside diameter (ID) is about 6.66 inches, see Attachment 1.

3. The Manning's "n" for gravity open channel flow in HDPE pipe is approximately 0.011.

4. The 8-inch diameter pipes will be evaluated for a 2 percent slope.

5. The maximum leachate generation rate in Cell 2 is 3,705 gal/acre/day under active conditions based upon the peak daily leachate generation results provided in Appendix D.1.

6. Cell 2, Phase II is 6.6 acres.

Calculations

Allowable Flow:

Determine the maximum allowable flow, assuming the pipe is flowing full. (Note: Gravity pipes carry more liquid when running 85 percent to 95 percent full than when 100 percent full):

Manning's Equation:

$$Q_{allow} = \frac{1.49}{n} A R_h^{2/3} S^{1/2}$$

Where
Q_{allow} = allowable flow rate, ft^3/sec
n = Manning's roughness coefficient $\approx$ 0.011 for HDPE pipe
A = cross-sectional area of the pipe, ft^2
R_h = hydraulic radius, ft
= ID/4 for full flow
S = slope of pipe

For a 2 Percent Slope:

Determine the values for each factor:

$A = \frac{(6.66)^2 \pi}{4} = 34.8 \text{ in}^2 = 0.24 \text{ ft}^2$

R_h = ID/4 = (6.66)/4 inches= 1.67 inches = 0.14 ft

S = 0.020

$$Q_{allow} = \frac{1.49}{n} A R_h^{2/3} S^{0.5}$$

$$Q_{allow} = \frac{1.49}{0.011}(0.24)(0.14)^{2/3}(0.020)^{0.5} = 1.24 \text{ cfs} = 557 \text{ gpm} = 801{,}400 \text{ gpd}$$

Allowable Drainage Area for One Leachate Collection Pipe at 2 Percent Slope:

$$= \frac{\text{Allowable Flow}}{\text{Leachate Generated per acre}} = \frac{801{,}400 \text{ gpd}}{3{,}705 \text{ gpad}} = 216 \text{ acres}$$

Conclusion

Based on the above calculations, the 8-inch diameter HDPE SDR 9.3 leachate collection or transfer pipe will provide sufficient flow capacity to convey the anticipated amount of leachate generated under active conditions in approximately 216 acres for a pipe at a 2 percent slope. Therefore, the leachate pipe has sufficient capacity to collect and transmit leachate flows in the 6.6-acre Cell 2 of Phase II at Kekaha Landfill.

Attachment 1

OD			Pipe inside diameter (d)	Minimum Wall Thickness (t)	Weight (w)
Nominal in.	Actual in.	SDR	in.	in.	lb. per foot
		7	3.88	0.795	5.172
		7.3	3.95	0.762	4.996
		9	4.25	0.618	4.182
		9.3	4.29	0.598	4.065
		11	4.49	0.506	3.505
5	5.563	11.5	4.54	0.484	3.368
		13.5	4.69	0.412	2.912
		15.5	4.80	0.359	2.564
		17	4.87	0.327	2.353
		21	5.00	0.265	1.929
		26	5.11	0.214	1.574
		32.5	5.20	0.171	1.270
		7	4.62	0.946	7.336
		7.3	4.70	0.908	7.086
		9	5.06	0.736	5.932
		9.3	5.11	0.712	5.765
		11	5.35	0.602	4.971
6	6.625	11.5	5.40	0.576	4.777
		13.5	5.58	0.491	4.130
		15.5	5.72	0.427	3.637
		17	5.80	0.390	3.338
		21	5.96	0.315	2.736
		26	6.08	0.255	2.233
		32.5	6.19	0.204	1.801
		7	6.01	1.232	12.433
		7.3	6.12	1.182	12.010
		9	6.59	0.958	10.054
		9.3	6.66	0.927	9.771
		11	6.96	0.784	8.425
8	8.625	11.5	7.04	0.750	8.096
		13.5	7.27	0.639	7.001
		15.5	7.45	0.556	6.164
		17	7.55	0.507	5.657
		21	7.75	0.411	4.637
		26	7.92	0.332	3.784

Appendix D.2B
Leachate Collection Pipe Flow Perforation Sizing

CALCULATION SHEET

Project No. 60197394

Client	County of Kaua'i	**Subject**	Leachate Collection Pipe Perforation Sizing	**Prepared By** NKW	**Date** 12/10/15
Project	Kekaha Landfill Cell 2, Phase II			**Reviewed By** DFP	**Date** 12/11/15
				Approved By NKW	**Date** 12/14/15

LEACHATE COLLECTION PIPE PERFORATION SIZING

Objective

Determine if the proposed pipe perforations in the leachate collection pipe of Cell 2 will be able to maintain leachate flow into the pipe for the estimated leachate quantity and verify proposed granular drainage layer particle sizing is adequate for the proposed perforation sizing.

Design Criteria and Assumptions

1. Cell 2 is divided into four sump areas as shown on Drawing 2. The shortest length of perforated leachate collection pipe draining to one sump is approximately 300 feet. The largest area draining to one sump is approximately 2.0 acres.

2. Proposed pipe perforations are located in two rows along the bottom of the pipe and alternately spaced. Spacing of perforations, center-to-center, in each row is 5 inches. The diameter of each pipe perforation is 0.5 inches.

3. For conservatism, the peak daily leachate generation rate is used in verifying the flow through the perforations. From the leachate generation analysis calculation (Appendix D.1), the leachate generation rate is 3,705 gallons/acre/day.

4. The average particle size (D_{50}) of the granular drainage layer is 1.0 inch.

Calculations

Largest Sump Area Leachate Flow

2.0 ac x 3,705 gal/ac/day = 7,410 gal/day

Total Pipe Perforations

Determine the total number of perforation available on the proposed leachate collection pipe.

Where: P = Total number of perforations available
L = Length of perforated pipe (ft).
S = Spacing center-to-center of perforations (in).
H = Number of perforation rows

Solving:

$$P_{total} = L * \left(\frac{1}{S}\right) * \left(\frac{12''}{1ft}\right) * (H)$$

For the Shortest perforated pipe draining to one sump:

$$P_{2W} = 300\,ft * \left(\frac{1}{5''}\right) * \left(\frac{12''}{1\,ft}\right) * (2) = 1{,}440$$

The shortest proposed pipe in Cell 2 will have 1,440 perforations.

Flow Per Perforation

The flow through each perforation can be determined by applying Q=VAC, where:

Q = flow rate
A = area of perforation, or $(D^2/4)*\pi = 0.00136$ ft^2.
V = in-flow velocity through each perforation, 0.1 feet per second, assumed (Qian, 2002)
C = discharge coefficient, 0.62 (Qian, 2002)

Solving:

$$Q = (A) * (V) * (C)$$

$$Q = (0.00136sf) * (0.1\,fps) * (0.62\,fps)$$

$$Q = 8.43x10^{-5}\,cfs * \left(\frac{7.48\,\text{gal}}{1\,\text{cf}}\right) * \left(\frac{60\,\text{sec}}{1\,\text{min}}\right) * \left(\frac{60\,\text{min}}{1\,\text{hr}}\right)\left(\frac{24\,\text{hr}}{1\,\text{day}}\right)$$

$$Q = 54.5\,\text{gal/day}$$

The maximum daily flow of leachate through each perforation is 54.5 gallons (54.5 gal/day/perforation).

Required Total Number of Perforations

The required total number of perforations needed to accommodate the Peak Daily Flow of leachate in the largest area draining to one sump is:

$$P_{required} = (7{,}410\,gal/day) * \left(\frac{day - perforation}{54.5\,gal}\right) = 136$$

The total number of perforations required is 136. Since 136 is less than the 1,440 perforations available, the leachate collection pipe is sufficient. 9.5 percent of the available perforations will be required.

Grain Size Criteria of Filter Materials in Relation to Pipe Perforations

To verify specified granular drainage layer particle sizing verse perforations in the leachate collection piping and sideslope riser pipes, the U.S. Bureau of Reclamation formula for grain size criteria of filter materials next to perforations in pipes (Reference 1) is applied. The formula is:

(D_{85} of the filter nearest the pipe) / (perforation diameter) = 2 or greater

Solving the equation based upon a pipe perforation diameter of 0.5 inches we obtain:

$$\frac{D_{85}}{0.5"} = 2, solving: D_{85} \geq 1.0"$$

The required granular drainage layer against the pipe perforations at D_{85} (or the particle size at which 85 percent of the sample is smaller) should be 1 inch. The specified average particle size for the granular drainage layer, or D_{50}, is 1 inch. Therefore, the D_{85} of the granular drainage layer will be larger than 1 inch and meets the requirements for particle size versus pipe perforations.

Conclusions

The leachate collection pipe as proposed contains more than enough perforations to handle the estimated Peak Daily Flow of leachate. The extra perforations provide a factor of safety for perforations that become partially plugged.

The specified granular drainage layer with an average particle size of 1 inch will meet the requirements for placement along piping perforated with 0.5-inch diameter holes.

References

1. Qian, Xuede; Koerner, Robert M.; Gray, Donald H., "Geotechnical Aspects of Landfill Design and Construction", 2002, pages 300-302.

Appendix D.3
Leachate Collection and Extraction Riser Pipe Strength Analysis

PIPE STRENGTH ANALYSIS

Objective

Leachate collection pipes will be located at the low points on the base liner of Cell 2, Phase II. The pipes will be subjected to increased overburden pressures as waste filling progresses. The pipes will be evaluated for compressive ring thrust (wall crushing), ring deflection, and wall buckling. Leachate extraction riser pipes placed within the leachate collection sumps will be evaluated for the same criteria.

Design Criteria and Assumptions

1. Leachate collection pipe will be 8-inch diameter, SDR 9.3, HDPE pipe O.D. = 8.625 inches, minimum wall thickness = t = 0.927, from reference 1 (refer to page 9 in Attachment 1).

2. Largest waste height at closure over the leachate collection pipe will be approximately 104 feet (difference between final grade and base grade less 3 feet of final cover soils and 3 feet of granular and operations layers above base grade). Note that the largest waste height over the leachate collection pipe is conservatively set equal to the maximum waste depth.

3. Leachate extraction riser pipe will be 18-inch diameter, SDR 11, HDPE pipe O.D. = 18.0-inch, minimum wall thickness = t = 1.636 inches, from reference 1 (refer to page 11 of Attachment 1).

4. Largest waste height at closure over the leachate extraction riser pipes will be approximately 104 feet (difference between final grade and base grade less 3 feet of final cover soils and 3 feet of granular and operations layers above base grade). Note that the largest waste height over the riser pipes is conservatively set equal to the maximum waste depth.

5. The unit weight of cover soil, granular layer, and operations layer material was assumed to be 130 lbs/ft^3. The unit weight of waste was assumed to be 62 lbs/ft^3.

6. The leachate collection pipes will be perforated in the lower quarters. Research (Reference 2) indicates that inclusion of perforations in the lower quarters of the pipe has no significant changes on pipe deflection versus load performance. Thus, the perforated pipe stiffness is assumed the same as that of the solid pipe.

7. The portion of the leachate extraction and detection riser pipes in the sump will be perforated around the entire diameter of the pipe. An effective pipe wall thickness will be calculated based on the pipe material removed due to perforations and will be used in the pipe strength calculations for the leachate extraction riser pipe.

8. Assume the portion of the leachate extraction riser pipe in the sump will have rows of ½-inch perforations 3-inches apart (four perforations per foot).

9. An approximately 120,000-pound compactor as the live load (Caterpillar 836).

10. To be conservative, calculate the live load as a concentrated load.

11. Conservatively assume the temperature of the leachate drainage layer is equal to the temperature of the pipes and is 100°F (37.8°C) (see Attachment 2). The apparent modulus of elasticity for HDPE pipe is 23,000 psi (see Table 3-1 in Attachment 1).

Methodology Reference:

1. Evaluation of buried pipes will follow procedure outlined in the "PPI Handbook of Polyethylene Pipe," downloaded May 2006 www.plasticpipe.org/general/ppihandbook.php.

2. Daniel P. Dietzler, "Structural Design of Landfill Leachate Collection System." Sixth Annual Madison Conference of Applied Research & Practice on Municipal & Industrial Waste, September 14-15, 1983.

Methodology

(1) Calculate the total external pressure, P_T, at the top of the pipe. P_T is composed of three components: pressure due to static loads, pressure due to live loads, and apparent pressure due to internal vacuum. The total external pressure is determined for two cases, one with shallow soil height over the pipe, one with maximum soil height over pipe:

$$P_T = P_s + P_L + P_i \qquad < 1a >$$

Where: P_s = total "static load" pressure, soil dead load
P_L = total "live load" pressure
P_i = total effective external pressure due to internal vacuum

The total "static load" pressure, soil dead load, P_s, at the top of the pipe for the two cases is conservatively calculated using a prism load.

$$P_s = \Sigma(w * H) \qquad < 1b >$$

Where: w = unit weight of waste or soil above the pipe
H = height of waste or soil above the pipe

The total "live load" pressure, vehicle traffic load, P_L, at the top of the pipe for the two cases is calculated using Boussinesq's Equation.

$$P_L = \frac{3 \times I_F \times W_L \times H^3}{2 \times \pi \times r^5} \qquad < 2 >$$

Where: P_L = live load, psf
I_F = impact factor
W_L = wheel load, lb
H = depth of cover, ft.
r = distance from the point of load application to pipe crown, ft.

(2) Examine compressive ring thrust (wall crushing) by calculating the compressive stress, S_A:

$$S_A = \frac{(P_{RD}) \times DR}{288} \qquad < 3 >$$

Where: S_A = compressive stress, psi
P_{RD} = radial directed earth pressure, psf
DR = dimension Ration, D_O/t

D_O = pipe outside diameter, in.
t = pipe wall thickness, in.

$$P_{RD} = (VAF) \times w \times H = (VAF) \times P_T \quad < 4 >$$

Where: P_{RD} = radial directed earth pressure, psf
VAF = Vertical Arching Factor
w = unit weight of soil, pcf
H = depth of cover, ft.
P_T = total external pressure, psf

$$VAF = 0.88 - 0.71 \times \frac{S_R - 1}{S_R + 2.5} \quad < 5 >$$

Where: VAF = Vertical Arching Factor
S_R = hoop thrust stiffness ratio

$$S_R = \frac{1.43 \times M_S \times r_{CENT}}{E \times t} \quad < 6 >$$

Where: S_R = hoop thrust stiffness ratio
M_S = one-dimensional modulus of soil, psi
r_{CENT} = radius to centroidal axis of pipe, in.
E = apparent modulus of elasticity of pipe material, psi
t = pipe wall thickness

The recommended long-term compressive strength design value is 780 psi for PE 3408 pipe at 100 F (1,000 psi x 0.78, see Tables 1-3 and 2-12 in Attachment 1)

$S_A \leq 780$ psi

(3) Examine ring deflection of the pipe by calculating the deflection, $\Delta X/D_M$:

$$\Delta X/D_M = D_F \times \varepsilon \quad < 7 >$$

Where: $\Delta X/D_M$ = deflection
D_F = deformation factor
ε = soil strain

$$\varepsilon = \frac{w \times H}{144 \times E_S} = \frac{P_T}{144 \times E_S} \quad < 8 >$$

Where: P_T = total external pressure, psf
w = unit weight of soil, pcf
H = depth of soil cover, ft.
E_S = secant modulus of the soil, psi

D_F is determined from Figure 2-6 of Attachment 1 using the rigidity factor, R_F

$$R_F = \frac{12 \times E_S \times (DR - 1)^3}{E} \quad < 9 >$$

Where: E_S = secant modulus of the soil (soil strain), psi
DR = dimension ratio
E = apparent modulus of elasticity of pipe material, psi

$$E_S = M_S \times \frac{(1 + \mu) \times (1 - 2 \times \mu)}{(1 - \mu)} \qquad < 10 >$$

Where: M_S = one-dimensional modulus
μ = poisson's ratio of soil

The recommended allowable ring deflection for non-pressure HDPE pipe and accounting for a large safety factor is 7.5% (see page 2 in Attachment 1).

(4) Calculate the allowable pipe wall buckling pressure, $P_{CR.}$

$$P_{CR} = \frac{2.4 \times \varphi \times R_H}{D_M} \times (E \times I)^{1/3} \times (E_S{}^*)^{2/3} \qquad < 11 >$$

Where:
- P_{CR} = allowable buckling pressure, psi
- φ = calibration factor, 0.55 for granular soils (see page 7 in Attachment 1)
- R_H = geometry factor, 1 (for deep burial, see page 7 in Attachment 1)
- D_M = mean diameter (D_O – t), in.
  D_O = pipe outside diameter, in.
  t = pipe wall thickness, in.
- E = apparent modulus of elasticity of pipe material, psi
- I = pipe wall moment of inertia, in^4/in ($t^3/12$, if solid wall construction)
- $E_S{}^*$ = $E_S/(1-\mu)$
  E_S = secant modulus of the soil, psi
  μ = poisson's ratio of soil

The allowable buckling pressure should be compared to the total "static load" pressure, P_T, on the pipe.

$$P_{CR} \geq P_T$$

Calculations

1. Calculate total external pressure on the leachate collection pipe.

$$P_T = P_s + P_L + P_i \qquad < 1a >$$

Where:
- P_T = total external pressure, psf
- P_s = total "static load" pressure, soil dead load
- P_L = total "live load" pressure
- P_i = total effective external pressure due to internal vacuum

The leachate collection pipe will have no internal pressure. Therefore, P_i = 0.0 psi for both shallow and largest soil height cases.

Total "static load" pressure on pipe, equation < 1b >

$$P_s = \Sigma(w * H) \qquad < 1b >$$

Where: w_{soil} = 130 pcf, unit weight of soil above pipe
w_{waste} = 62 pcf, unit weight of waste

Shallow waste height (during waste filling):

H_{soil} = 3 ft (granular drainage and operation layers)
H_{waste} = 5 ft
P_s = [(130 pcf x 3 ft) + (62 pcf x 5 ft)] x 1 $ft^2/144\ in^2$
P_s = 4.86 psi

Largest waste height:

H_{soil} = 3 ft cover + 3 ft granular drainage layer
H_{waste} = 104 ft, largest height of waste over pipe
P_s = [(130 pcf x 6 ft) + (62 pcf x 104 ft)] x 1 $ft^2/144\ in^2$
P_s = 50.2 psi

The total "live load" pressure, vehicle traffic load, P_L, at the top of the pipe for the two cases is calculated using Boussinesq's Equation.

$$P_L = \frac{3 \times I_F \times W_L \times H^3}{2 \times \pi \times r^5} \qquad < 2 >$$

Where
P_L = total "live load" pressure, psf
I_F = impact factor = 1.5, assumed for slow vehicle velocity see Attachment 3
W_L = wheel load = 60,000 lbs (approximately 120,000 lbs for Caterpillar 836 compactor divided by 2 drums)
H = depth of cover, ft.
r = distance from the point of load application to pipe crown (same as H), ft.

For the Shallow Waste Height Case:

Where H = r = depth of cover; 8 ft (5 ft of waste + 3 ft drainage and operational layers)

$$P_L = \frac{3 \times 1.5 \times 60{,}000 \times (8)^3}{2 \times \pi \times (8)^5} = 671.4\ psf \times 1\ ft^2/144\ in^2 = 4.66\ psi$$

For the Largest Waste Height Case:

The "live load" pressure, P_L, applied at the top of the final cover will be negligible due to the distance (over 60 feet) from the top of the pipe. Therefore, $P_L = 0$. At the burial depths greater than 5 feet, loads from an AASHTO Standard H-20 truck are "not usually significant because the load is attenuated significantly compared (with) loads under 1 or 2 feet of cover." The H-20 loading represents a 40,000-lb. dual axle truck (see Attachment 3).

Shallow waste height:

Total external pressure, $P_T = P_s + P_L + P_i = 4.86 \text{ psi} + 4.66 \text{ psi} + 0 = 9.52 \text{ psi}$

Largest waste height:

Total external pressure, $P_T = P_s + P_L + P_i = 50.2 \text{ psi} + 0 + 0 \text{ psi} = 50.2 \text{ psi}$

$50.2 \text{ psi} \times 144 \text{ in}^2/\text{ft}^2 = 7{,}229 \text{ psf}$

The worst case situation is the largest waste height case and will therefore be used in the leacahate collection pipe analysis.

2. Calculate total external pressure on the leachate extraction riser pipe.

$$P_T = P_s + P_L + P_i \qquad < 2 >$$

Where: P_s = total "static load" pressure soil dead load
P_L = total "live load" pressure
P_i = total effective external pressure due to internal vacuum

The extraction riser pipe will have no internal pressure. Therefore, $P_i = 0.0$ psi for both shallow and largest waste height cases.

Total static load on pipe, equation < 1 >

$$P_s = \Sigma(r * H) \qquad < 1 >$$

Where: r_{soil} = 130 pcf, unit weight of soil above pipe
r_{waste} = 62 pcf, unit weight of waste

Shallow waste height (during waste filling):

H_{soil} = 4 ft (operations layer, granular drainage layer, and sump depth)
H_{waste} = 5 ft
$P_s = [(130 \text{ pcf} \times 4 \text{ ft}) + (62 \text{ pcf} \times 5 \text{ ft})] \times 1 \text{ ft}^2/144 \text{ in}^2$
$P_s = 5.76$ psi

Largest waste height:

H_{soil} = 3 ft cover + 4 ft operations layer, granular drainage layer, and sump depth = 7 ft
H_{waste} = 104 ft, largest height of waste
$P_s = [(130 \text{ pcf} \times 7 \text{ ft}) + (62 \text{ pcf} \times 104 \text{ ft})] \times 1 \text{ ft}^2/144 \text{ in}^2$
$P_s = 51.1$ psi

The total "live load" pressure, vehicle traffic load, P_L, at the top of the pipe for the two cases is calculated using Boussinesq's Equation.

$$P_L = \frac{3 \times I_F \times W_L \times H^3}{2 \times \pi \times r^5} \qquad < 2 >$$

Where
- P_L = live load, psf
- I_F = impact factor = 1.5, assumed for slow vehicle velocity (see Attachment 3)
- W_L = wheel load = 60,000 lbs (approximately 120,000 lbs for Caterpillar 836 compactor divided by 2 drums)
- H = depth of cover, ft.
- r = distance from the point of load application to pipe crown (same as H), ft.

For the Shallow Waste Height Case:

Where: H = r = depth of cover; 10 ft (5 ft of waste + 4 ft operations layer, drainage blanket, and sump depth)

$$P_L = \frac{3 \times 1.5 \times 60{,}000 \times (9)^3}{2 \times \pi \times (9)^5} = 530.52 \text{ psf} \times 1 \text{ ft}^2/144 \text{ in}^2 = 3.68 \text{ psi}$$

For the Largest Waste Height Case:

The live load, P_L, applied at the top of the final cover will be negligible due to the distance (over 40 feet) from the top of the pipe. Therefore, $P_L = 0$. At the burial depths greater than 5 feet, loads from an AASHTO Standard H-20 truck are "not usually significant because the load is attenuated significantly compared (with) loads under 1 or 2 feet of cover." The H-20 loading represents a 40,000-lb. dual axle truck.

Shallow waste height:

Total external pressure, $P_T = P_s + P_L + P_i = 5.76 \text{ psi} + 3.68 \text{ psi} + 0 = 9.44 \text{ psi}$

Largest waste height:

Total external pressure, $P_T = P_s + P_L + P_i = 51.1 \text{ psi} + 0 + 0 \text{ psi} = 51.1 \text{ psi}$

$51.1 \text{ psi} \times 144 \text{ in}^2/\text{ft}^2 = 7{,}358 \text{ psf}$

The worst case situation is the largest waste height case and will therefore be used in the analysis of the leachate extraction riser pipe.

3. Calculate the effective wall thickness of the leachate extraction riser pipe.

To account for pipe perforations, calculate the effective wall thickness for use in extraction riser pipe strength calculations.

$$t_{eff} = t \times \frac{12 - \left(D_{perf} \times n\right)}{12}$$

Where: t_{eff} = effective pipe wall thickness
t = actual pipe wall thickness, 1.636 in.
D_{perf} = diameter of perforation, 0.5 in.
n = number of rows of perforations in 1 ft. of pipe, 4

$$t_{eff} = 1.636\,\text{in.} \times \frac{12 - (0.5\,\text{in.} \times 4)}{12} = 1.363\,\text{in.}$$

4. Evaluate the compressive ring thrust (wall crushing), ring deflection, and wall buckling for the leachate collection pipe and extraction riser pipe.

   An excel spreadsheet is used to evaluate the leachate collection pipe and extraction riser pipe. The spreadsheet calculation inputs are summarized in Tables 1 and 2. The spreadsheet results are summarized in Table 3 and provided in detail in Attachment 4. Sample calculations for the leachate extraction pipe are provided below.

Table 1: General Calculation Inputs

			8" Collection Pipe	18" Extraction Pipe	
D_O =	pipe outside diameter, in.	=	8.625	18.0	in.
t =	pipe wall thickness, in.	=	0.927	1.363	in.
P_T =	total external pressure, psf	=	7,229	7,358	psf
		=	50.2	50.1	psi
E =	apparent modulus of elasticity of pipe material, psi	=	23,000		psi
M_S =	one-dimensional modulus, (for 90% standard proctor and vertical waste and soil stress (Table 2-14, Plastic Pipe Institute, 2006)	=	2,304	2,322	(for total pressure stated above)
μ =	Poisson's ratio of soil, to model granular drainage layer, (Table 2-15, Plastic Pipe Institute, 2006)	=	0.15		(value for coarse sand)
The recommended long-term compressive strength design value for HDPE pipe (psi) (Tables 1-3 & 2-12, Plastic Pipe Institute, 2006)		=	780		
D_F =	Deformation Factor, (Figure 2-6, Plastic Pipe Institute, 2006) for R_F as in ring deflection.	=	1.15	1.4	
φ =	Calibration Factor	=	0.55		(for granular soils)
R_H =	Geometry Factor, (page 7 of Attachment 1, Plastic Pipe Institute, 2006)	=	1		(for deep burial)

I. Sample Calculations 8-inch HDPE Leachate Collection Pipe

A. Compressive Ring Thrust (Wall Crushing) Analysis

The compressive stress, S_A, by equation 3:

$$S_A = \frac{(P_{RD}) \times DR}{288} \qquad < 3 >$$

Where: S_A = compressive stress, psi
P_{RD} = radial directed earth pressure, psf
DR = dimension Ratio, D_O/t
D_O = pipe outside diameter, in.

Calculate the hoop stiffness ratio, S_R, using equation < 6 > to determine the vertical arching factor, VAF, using equation < 5 >. The radial directed earth pressure, P_{RD}, can then be calculated using equation < 4 >:

$$S_R = \frac{1.43 \times M_S \times r_{CENT}}{E \times t} \qquad < 6 >$$

Where: S_R = hoop thrust stiffness ratio
M_S = one-dimensional modulus of soil, 2,304 psi (for 90% standard proctor and 50.2 psi vertical soil stress, see Table 2-14 included in Attachment 1)
r_{CENT} = radius to centroidal axis of pipe, 3.85 in. ($r_{cent} = r_{pipe} – t/2 = 8.625/2 - 0.927$)
E = apparent modulus of elasticity of pipe material, 23,000 psi
t = pipe wall thickness, 0.927 in.

$$S_R = \frac{1.43 \times M_S \times r_{CENT}}{E \times t} = \frac{1.43 \times 2{,}304 \text{ psi} \times 3.85 \text{ in}}{23{,}000 \text{ psi} \times 0.927 \text{ in}} = 0.595$$

$$VAF = 0.88 – 0.71 \times \frac{S_R - 1}{S_R + 2.5} = 0.88 – 0.71 \times \frac{0.595 – 1}{0.595 + 2.5} = 0.973$$

Where: VAF = Vertical Arching Factor
S_R = hoop thrust stiffness ratio

$P_{RD} = (VAF) \times w \times H = (VAF) \times P_T = 0.973 \times 7{,}229 \text{ psf} = 7{,}034 \text{ psf}$

Where: P_{RD} = radial directed earth pressure, psf
VAF = Vertical Arching Factor
w = unit weight of soil, pcf
H = depth of cover, ft.
P_T = total external pressure, psf

$$S_A = \frac{(P_{RD}) \times DR}{288} = \frac{7{,}034 \text{ psf} \times (8.625 \text{ in} / 0.927 \text{ in})}{288} = 227 \text{ psi}$$

The recommended long-term compressive strength design value is 780 psi at 100ºF for PE 3408 pipe, (see Tables 1-3 and 2-12 in Attachment 1).

P_{RD} = 227 psi ≤ 780 psi

The factor of safety for wall crushing is defined by:

F.S. = Allowable / Actual = 780 / 227 = 3.4

A factor of safety of 3.4 demonstrates that the pipe will perform without wall crushing.

B. Calculate the secant modulus of the soil, E_S, using equation 10, determine the rigidity factor, R_F, using equation 9 and the soil strain using equation 8. The rigidity factor is used to determine the deformation factor which is used to calculate the percent deflection, $\Delta X/D_M$ x 100, of the pipe using equation 7.

$$E_S = M_S \times \frac{(1 + \mu) \times (1 - 2 \times \mu)}{(1 - \mu)} \qquad < 10 >$$

Where: E_S = secant modulus of the soil, psi
M_S = one-dimensional modulus, 2,304 psi (for 90% standard proctor and 50.2 psi vertical soil stress, see Table 2-14 included in Attachment 1)
μ = poisson's ratio of soil, 0.15 (used to be conservative, value for coarse sand from Table 2-15 included in Attachment 1).

$$E_S = 2{,}304 \text{ psi} \times \frac{(1 + 0.15) \times (1 - 2 \times 0.15)}{(1 - 0.15)} = 2{,}182 \text{ psi}$$

$$R_F = \frac{12 \times E_S \times (DR - 1)^3}{E} \qquad < 9 >$$

Where: R_F =rigidity factor
E_S = secant modulus of the soil, psi (see calculated value above)
DR = dimension ratio, D_O/t, 9.3, (8.625/0.927)
E = apparent modulus of elasticity of pipe material, 23,000 psi

$$R_F = \frac{12 \times E_S \times (DR - 1)^3}{E} = \frac{12 \times 2{,}182 \text{ psi} \times (9.3 - 1)^3}{23{,}000 \text{ psi}} = 651$$

$$\varepsilon = \frac{P_T}{144 \times E_S} \qquad < 8 >$$

Where: P_T = total external pressure, 7,229 psf
E_S = secant modulus of the soil, psi (see calculated value above)

$$\varepsilon = \frac{P_T}{144 \times E_S} = \frac{7{,}229 \text{ psf}}{144 \times 2{,}182 \text{ psi}} = 0.0230$$

$$\Delta X/D_M = D_F \times \varepsilon \qquad < 7 >$$

Where: $\Delta X/D_M$ = deflection
D_F = deformation factor, 1.15 (from Figure 2-6 included in Attachment 1 for R_F as calculated above).
ε = soil strain

$\Delta X/D_M = D_F \times \varepsilon = 1.15 \times 0.0230 = 0.0265$

Percent Deflection = 0.0265 x 100 = 2.65%

The recommended allowable ring deflection for non-pressure pipe and accounting for a large safety factor is 7.5% (see page 2 in Attachment 1). Therefore, the pipe will not deflect beyond the allowable limit.

C. Calculate the allowable pipe wall buckling pressure, P_{CR}.

$$P_{CR} = \frac{2.4 \times \varphi \times R_H}{D_M} \times (E \times I)^{1/3} \times (E_S^*)^{2/3} \qquad < 11 >$$

Where: P_{CR} = allowable buckling pressure, psi
φ = calibration factor, 0.55 for granular soils (see page 7 in Attachment 1)
R_H = geometry factor, 1 (for deep burial, see page 7 in Attachment 1)
D_M = mean diameter (D_O – t), 7.698 in. (8.625 – 0.927)
D_O = pipe outside diameter, 8.625 in.
t = pipe wall thickness, 0.927 in.
E = Apparent modulus of elasticity of pipe material, 23,000 psi
I = Pipe wall moment of inertia, 0.0664 in^4/in ($t^3/12 = 0.927^3/12$)
$E_S^* = E_S/(1-\mu) = 2{,}182 \text{ psi}/(1 - 0.15) = 2{,}567$ psi
E_S = secant modulus of the soil, 2,182 psi (see calculation above)
μ = poisson's ratio of soil, 0.15 (used to be conservative, value for coarse sand from Table 2-15 included in Attachment 1).

$$P_{CR} = \frac{2.4 \times 0.55 \times 1}{7.698} \times (23{,}000 \times 0.0664)^{1/3} \times (2{,}567)^{2/3} = 370 \text{ psi}$$

The allowable buckling pressure should be compared to the total external pressure, P_T, on the pipe.

370 psi = $P_{CR} \geq$ 50.1 psi = P_T

Factor of safety for wall buckling is:

F.S. = P_{CR} / P_T = 370 psi / 50.1 psi = 7.4

A factor of safety of 7.4 demonstrates that the pipe will perform without wall buckling.

Conclusions

The results of the above calculations are summarized below. The proposed leachate collection pipes and leachate extraction pipes will perform as designed without strength failure.

Table 2: Results Summary

	8" SDR 9.3 HDPE Leachate Collection Pipe	18" SDR 11 HDPE Leachate Extraction Riser Pipe
Compressive Ring Thrust (Wall Crushing)	F.S. = 3.4	F.S. = 2.5
Ring Deflection (7.5% allowable)	2.65%	3.25%
Wall Buckling	F.S. = 7.4	F.S. = 4.9

Attachment 1

Source: "PPI Handbook of Polyethylene Pipe", downloaded May, 2006
www.plasticpipe.org/general/ppi_handbook.php

As indicated in Table 1-2, polyethylene pipe which meets the requirements of ASTM D2513 may be used for the transport of liquefied petroleum gas (LPG). NFPA 58 recommends a maximum operating pressure of 30 psig for LPG gas applications involving polyethylene pipe. This design limit is established in recognition of the higher condensation temperature for LPG as compared to that of natural gas and, thus, the maximum operating pressure is recommended to ensure that plastic pipe is not subjected to excessive exposure to LPG condensates. For further information the reader is referred to PPI's TR-22, Polyethylene Piping Distribution Systems for Components of Liquid Petroleum Gases.[14]

TABLE 1-3
Service Temperature Design Factors, F_T

Maximum Continuously Applied Service Temp., °F(°C)	Temperature Compensation Factor, FT, for PE3408
≤ 80 (26)	1.00
≤ 90 (32)	0.90
≤ 100 (38)	0.78
≤ 110 (43)	0.75
≤ 120 (49)	0.63
≤ 130 (54)	0.60
≤ 140 (60)	0.50



Fluid Flow in Polyethylene Piping

Head Loss in Pipes – Darcy-Weisbach/Fanning/Colebrook/Moody

Viscous shear stresses within the liquid and friction along the pipe walls create resistance to flow within a pipe. This resistance within a pipe results in a pressure drop, or loss of head in the piping system.

The Darcy-Weisbach or Fanning formula, Equation 1-7, and the Colebrook formula, Equation 1-10, are generally accepted methods for calculating friction losses due to liquids flowing in full pipes.[15,16] These formulas recognize dependence on pipe bore and pipe surface characteristics, liquid viscosity and flow velocity.

The Darcy-Weisbach formula is:

(1-7)

$$h_f = f \frac{L V^2}{d' 2g}$$

Attachment 1

Bending strain occurs in the pipe wall as a result of ring deflection—outer-fiber tensile strain at the pipe springline and outer-fiber compressive strain at the crown and invert. While strain limits of 5% have been proposed, Jansen [12] reported that, on tests of PE pipe manufactured from pressure-rated resins and subjected to soil pressure only, "no upper limit from a practical design point of view seems to exist for the bending strain." In other words, as deflection increases, the pipe's performance limit will not be overstraining but reverse curvature collapse.

Thus, for non-pressure applications, a 7.5 percent deflection limit provides a large safety factor against instability and strain and is considered a safe design deflection. Some engineers will design profile wall pipe and other non-pressure pipe applications to a 5% deflection limit, but allow spot deflections up to 7.5% during field inspection.

The deflection limits for pressurized pipe are generally lower than for non-pressurized pipe. This is primarily due to strain considerations. Hoop strain from pressurization adds to the outer-fiber tensile strain. But the internal pressure acts to reround the pipe and, therefore, Eq. 2-10 overpredicts the actual long-term deflection for pressurized pipe. Safe allowable deflections for pressurized pipe are given in Table 2-11. Spangler and Handy [13] give equations for correcting deflection to account for rerounding.

TABLE 2-11
Safe Deflection Limits for Pressurized Pipe

DR or SDR	Safe Deflection as % of Diameter
32.5	7.5
26	7.5
21	7.5
17	6.0
13.5	6.0
11	5.0
9	4.0
7.3	3.0

*Based on Long-Term Design Deflection of Buried Pressurized Pipe given in ASTM F1962.

Compressive Ring Thrust

Earth pressure exerts a radial-directed force around the circumference of a pipe that results in a compressive ring thrust in the pipe wall. (This thrust is exactly opposite to the tensile hoop thrust induced when a pipe is pressurized.) See Figure 2-1b. Excessive ring compressive thrust may lead to two different performance limits:

Attachment 1

The compressive stress in the pipe wall can be compared to the pipe material allowable compressive stress. If the calculated compressive stress exceeds the allowable stress, then a lower DR (heavier wall thickness) or heavier profile wall is required.

Allowable Compressive Stress

Table 2-12 gives allowable long-term compressive stress values for PE 3408 and PE 2406 material.

TABLE 2-12
Long-Term Compressive Stress at 73°F (23°C)

Material	Long-Term Compressive Stress, lb/in^2
PE 3408	1000
PE 2406	800



The long-term compressive stress value should be reduced for elevated temperature pipeline operation. Temperature design factors used for hydrostatic pressure may be used, i.e. 0.5 @ 140°F. Additional temperature design factors may be obtained by reference to Table 1-11 in Section 1 of this chapter.

Ring Compression Example

Find the pipe wall compressive ring stress in a DR 32.5 HDPE pipe buried under 46 ft of cover. The ground water level is at the surface, the saturated weight of the insitu silty-clay soil is 120 lbs/ft^3.

SOLUTION: Find the vertical earth pressure acting on the pipe. Use Equation 2-1.

Although the net soil pressure is equal to the buoyant weight of the soil, the water pressure is also acting on the pipe. Therefore the total pressure (water and earth load) can be found using the saturated unit weight of the soil.

Next, solve for the compressive stress.

$$P_E = (120\ pcf)(46\ ft) = 5520\ psf$$

$$S = \frac{(5520\ lb/\ ft^2)(32.5)}{288} = 623\ lb/inch^2$$

The compressive stress is within the 1000 lb/in^2 allowable stress for HDPE given in Table 2-12.

Attachment 1

(2-21) $$VAF = 0.88 - 0.71\frac{S_A - 1}{S_A + 2.5}$$

WHERE

VAF = Vertical Arching Factor

S_A = Hoop Thrust Stiffness Ratio

(2-22) $$S_A = \frac{1.43\, M_S\, r_{CENT}}{EA}$$

WHERE

r_{CENT} = radius to centroidal axis of pipe, in

M_S= one-dimensional modulus of soil, psi

E = apparent modulus of elasticity of pipe material, psi

A= profile wall average cross-sectional area, in^2/in, or wall thickness (in) for DR pipe

One-dimensional modulus values for soil can be obtained from soil testing, geotechnical texts, or Table 2-14 which gives typical values. The typical values in Table 2-14 were obtained by converting values from McGrath[20].

TABLE 2-14
Typical Values of M_S, One-Dimensional Modulus of Soil

Vertical Soil Stress[1] (psi)	Gravelly Sand/Gravels 95% Std. Proctor (psi)	Gravelly Sand/Gravels 90% Std. Proctor (psi)	Gravelly Sand/Gravels 85% Std. Proctor (psi)
10	3000	1600	550
20	3500	1800	650
40	4200	2100	800
60	5000	2500	1000
80	6000	2900	1300
100	6500	3200	1450

*Adapted and extended from values given by McGrath[20]. For depths not shown in McGrath[20], the MS values were approximated using the hyperbolic soil model with appropriate values for K and n where n=0.4 and K=200, K=100, and K=45 for 95% Proctor, 90% Proctor, and 85% Proctor, respectively.

[1] Vertical Soil Stress (psi) = [soil depth (ft) x soil density (pcf)]/144

The radial directed earth pressure can be found by multiplying the prism load (pressure) by the vertical arching factor as shown in Eq. 2-23.

(2-23) $$P_{RD} = (VAF)wH$$

Attachment 1

TABLE 2-15
Typical range of Poisson's Ratio for Soil (Bowles [21])

Soil Type	Poisson Ratio, μ
Saturated Clay	0.4-0.5
Unsaturated Clay	0.1-0.3
Sandy Clay	0.2-0.3
Silt	0.3-0.35
Sand (Dense)	0.2-0.4
Coarse Sand (Void Ratio 0.4-0.7)	0.15
Fine-grained Sand (Void Ratio 0.4-0.7)	0.25



Next, the designer determines the Deformation Factor, D_F , by entering the Watkins-Gaube Graph with the Rigidity Factor. See Fig. 2-6. The Deformation Factor is the proportionality constant between vertical deflection (compression) of the soil layer containing the pipe and the deflection of the pipe. Thus, pipe deflection can be obtained by multiplying the proportionality constant D_F times the soil settlement. If D_F is less than 1.0 in Fig. 2-6, use 1.0.

The soil layer surrounding the pipe bears the entire load of the overburden above it without arching. Therefore, settlement (compression) of the soil layer is proportional to the prism load and not the radial directed earth pressure. Soil strain, ε_S, may be determined from geotechnical analysis or from the following equation:

(2-27)

$$\varepsilon_S = \frac{wH}{144E_S}$$

WHERE

w = unit weight of soil, pcf

H = depth of cover (height of fill above pipe crown), ft

E_S = secant modulus of the soil, psi

The designer can find the pipe deflection as a percent of the diameter by multiplying the soil strain, in percent, by the deformation factor:

Attachment 1

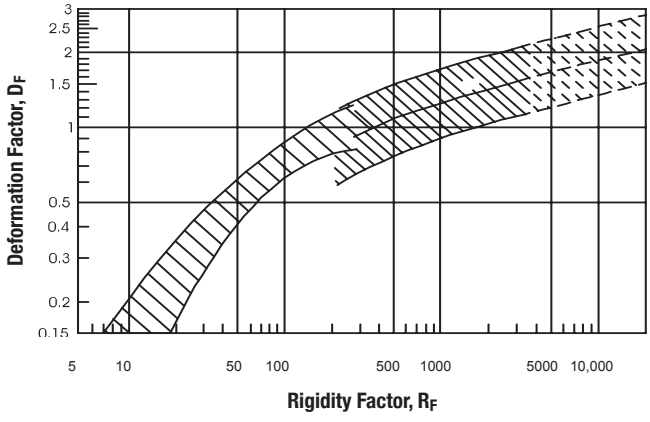


Figure 2-6 Watkins-Gaube Graph

(2-28) $$\frac{\Delta X}{D_M}(100) = D_F \varepsilon_S$$

WHERE

$\Delta X/D_M$ multiplied by 100 gives percent deflection.

Watkins – Gaube Calculation Technique

Find the deflection of a 6" SDR 11 pipe under 140 ft of fill with granular embedment containing 12% or less fines, compacted at 90% of standard proctor. The fill weighs 75 pcf.

SOLUTION: First, calculate the vertical soil pressure equation, Eq. 2-1.

Eq. 2-1: $P_E = wH$

$P_E = (75 lb/ft^3)(140 ft)$

$P_E = 10{,}500 lb/ft^2$ or 72.9 psi

The M_S is obtained by interpolation from Table 2-14 and equals 2700. The secant modulus can be found assuming a Poisson Ratio of 0.30

$$E_S = \frac{2700\ psi\,(1+0.30)(1-2(0.30))}{(1-0.30)} = 2005\ psi$$

The rigidity factor is obtained from Equation 2-24.

$$R_F = \frac{12\,(2005)\,(11-1)^3}{28250} = 852$$

Attachment 1

Using Figure 2-6, the deformation factor is found to be 1.2. The soil strain is calculated by Equation 2-27.

$$\varepsilon_S = \frac{75pcf * 140ft}{144 * 2005\frac{lbs}{inch^2}} \bullet 100 = 3.6\%$$

The deflection is found by multiplying the soil strain by the deformation factor:

$$\frac{\Delta X}{D_M}(100) = 1.2 * 3.6 = 4.4\%$$

Moore-Selig Equation for Constrained Buckling in Dry Ground

As discussed previously, a compressive thrust stress exists in buried pipe. When this thrust stress approaches a critical value, the pipe can experience a local instability or large deformation and collapse. In an earlier section of this chapter, Luscher's equation was given for constrained buckling under ground water. Moore and Selig[17] have used an alternate approach called the continuum theory to develop design equations for contrained buckling due to soil pressure (buckling of embedded pipes). The particular version of their equations given below is more appropriate for dry applications than Luscher's equation. Where ground water is present, Luscher's equation should be used.

The Moore-Selig Equation for critical buckling pressure follows: (Critical buckling pressure is the pressure at which buckling will occur. A safety factor should be provided.)

(2-29)

$$P_{CR} = \frac{2.4\,\varphi\,R_H}{D_M}(EI)^{\frac{1}{3}}(E_S^*)^{\frac{2}{3}}$$

WHERE

P_{CR} = Critical constrained buckling pressure, psi

φ = Calibration Factor, 0.55 for granular soils

R_H = Geometry Factor

E = Apparent modulus of elasticity of pipe material, psi

I = Pipe wall moment of Inertia, in^4/in ($t^3/12$, if solid wall construction)

$E_S^* = ES/(1-\mu)$

E_S = Secant modulus of the soil, psi

μ_s = Poisson's Ratio of Soil

The geometry factor is dependent on the depth of burial and the relative stiffness between the embedment soil and the insitu soil. Moore has shown that for deep burials in uniform fills, R_H equals 1.0.

Attachment 1

change cannot occur, so a longitudinal tensile stress is created along the pipe. The magnitude of this stress can be determined using Equation 3-2.

(3-2) $\sigma = E \alpha \Delta T$

Where terms are as defined above, and

σ = longitudinal stress in pipe, psi

E = apparent modulus elasticity of pipe material, psi

The value of the apparent modulus of elasticity of the pipe material has a large impact on the calculated stress. As with all thermoplastic materials, polyethylene's modulus, and therefore its stiffness, is dependent on temperature and the duration of the applied load. Therefore, the appropriate elastic modulus should be selected based on these two variables. When determining the appropriate time interval, it is important to consider that heat transfer occurs at relatively slow rates through the wall of polyethylene pipe; therefore temperature changes do not occur rapidly. Because the temperature change does not happen rapidly, the average temperature is often chosen for the modulus selection.

TABLE 3-1
Apparent Modulus Elasticity for HDPE Pipe Material at Various Temperatures

Load Duration	PE 3408 Apparent Elastic Modulus†, 1000 psi (MPa), at Temperature, °F (°C)							
	-20 (-29)	0 (-18)	40 (4)	60 (16)	73 (23)	100 (38)	120 (49)	140 (60)
Short-Term	300.0 (2069)	260.0 (1793)	170.0 (1172)	130.0 (896)	110.0 (758)	100.0 (690)	65.0 (448)	50.0 (345)
10 h	140.8 (971)	122.0 (841)	79.8 (550)	61.0 (421)	57.5 (396)	46.9 (323)	30.5 (210)	23.5 (162)
100 h	125.4 (865)	108.7 (749)	71.0 (490)	54.3 (374)	51.2 (353)	41.8 (288)	27.2 (188)	20.9 (144)
1000 h	107.0 (738)	92.8 (640)	60.7 (419)	46.4 (320)	43.7 (301)	35.7 (246)	23.2 (160)	17.8 (123)
1 y	93.0 (641)	80.6 (556)	52.7 (363)	40.3 (278)	38.0 (262)	31.0 (214)	20.2 (139)	15.5 (107)
10 y	77.4 (534)	67.1 (463)	43.9 (303)	33.5 (231)	31.6 (218)	25.8 (178)	16.8 (116)	12.9 (89)
50 y	69.1 (476)	59.9 (413)	39.1 (270)	29.9 (206)	28.2 (194)	23.0 (159)	15.0 (103)	11.5 (79)

† Typical values based on ASTM D 638 testing of molded plaque material specimens. An elastic modulus for PE 2406 may be estimated by multiplying the PE 3408 modulus value by 0.875.

Attachment 1

OD			Pipe inside diameter (d)	Minimum Wall Thickness (t)	Weight (w)
Nominal in.	**Actual in.**	**SDR**	**in.**	**in.**	**lb. per foot**
		7	3.88	0.795	5.172
		7.3	3.95	0.762	4.996
		9	4.25	0.618	4.182
		9.3	4.29	0.598	4.065
		11	4.49	0.506	3.505
5	5.563	11.5	4.54	0.484	3.368
		13.5	4.69	0.412	2.912
		15.5	4.80	0.359	2.564
		17	4.87	0.327	2.353
		21	5.00	0.265	1.929
		26	5.11	0.214	1.574
		32.5	5.20	0.171	1.270
		7	4.62	0.946	7.336
		7.3	4.70	0.908	7.086
		9	5.06	0.736	5.932
		9.3	5.11	0.712	5.765
		11	5.35	0.602	4.971
6	6.625	11.5	5.40	0.576	4.777
		13.5	5.58	0.491	4.130
		15.5	5.72	0.427	3.637
		17	5.80	0.390	3.338
		21	5.96	0.315	2.736
		26	6.08	0.255	2.233
		32.5	6.19	0.204	1.801
		7	6.01	1.232	12.433
		7.3	6.12	1.182	12.010
		9	6.59	0.958	10.054
		9.3	6.66	0.927	9.771
		11	6.96	0.784	8.425
8	8.625	11.5	7.04	0.750	8.096
		13.5	7.27	0.639	7.001
		15.5	7.45	0.556	6.164
		17	7.55	0.507	5.657
		21	7.75	0.411	4.637
		26	7.92	0.332	3.784

Attachment 1

OD			Pipe inside diameter (d)	Minimum Wall Thickness (t)	Weight (w)
Nominal in.	**Actual in.**	**SDR**	**in.**	**in.**	**lb. per foot**
		7	7.49	1.536	19.314
		7.3	7.63	1.473	18.656
		9	8.22	1.194	15.618
		9.3	8.30	1.156	15.179
		11	8.68	0.977	13.089
10	10.750	11.5	8.77	0.935	12.578
		13.5	9.06	0.796	10.875
		15.5	9.28	0.694	9.576
		17	9.41	0.632	8.788
		21	9.66	0.512	7.204
		26	9.87	0.413	5.878
		32.5	10.05	0.331	4.742
		7	8.89	1.821	27.170
		7.3	9.05	1.747	26.244
		9	9.75	1.417	21.970
		9.3	9.84	1.371	21.353
		11	10.29	1.159	18.412
12	12.750	11.5	10.40	1.109	17.693
		13.5	10.75	0.944	15.298
		15.5	11.01	0.823	13.471
		17	11.16	0.750	12.362
		21	11.46	0.607	10.134
		26	11.71	0.490	8.269
		32.5	11.92	0.392	6.671
		7	9.76	2.000	32.758
		7.3	9.93	1.918	31.642
		9	10.70	1.556	26.489
		9.3	10.81	1.505	25.745
		11	11.30	1.273	22.199
14	14.000	11.5	11.42	1.217	21.332
		13.5	11.80	1.037	18.445
		15.5	12.09	0.903	16.242
		17	12.25	0.824	14.905
		21	12.59	0.667	12.218
		26	12.86	0.538	9.970
		32.5	13.09	0.431	8.044

Attachment 1

OD			Pipe inside diameter (d)	Minimum Wall Thickness (t)	Weight (w)
Nominal in.	**Actual in.**	**SDR**	**in.**	**in.**	**lb. per foot**
		7	11.15	2.286	42.786
		7.3	11.35	2.192	41.329
		9	12.23	1.778	34.598
		9.3	12.35	1.720	33.626
		11	12.92	1.455	28.994
16	16.000	11.5	13.05	1.391	27.862
		13.5	13.49	1.185	24.092
		15.5	13.81	1.032	21.214
		17	14.00	0.941	19.467
		21	14.38	0.762	15.959
		26	14.70	0.615	13.022
		7	12.55	2.571	54.151
		7.3	12.77	2.466	52.307
		9	13.76	2.000	43.788
		9.3	13.90	1.935	42.558
		11	14.53	1.636	36.696
18	18.000	11.5	14.68	1.565	35.263
		13.5	15.17	1.333	30.491
		15.5	15.54	1.161	26.849
		17	15.76	1.059	24.638
		21	16.18	0.857	20.198
		26	16.53	0.692	16.480
		32.5	16.83	0.554	13.296
		7	13.94	2.857	66.853
		7.3	14.19	2.740	64.576
		9	15.29	2.222	54.059
		9.3	15.44	2.151	52.541
		11	16.15	1.818	45.304
20	20.000	11.5	16.31	1.739	43.535
		13.5	16.86	1.481	37.643
		15.5	17.26	1.290	33.146
		17	17.51	1.176	30.418
		21	17.98	0.952	24.936
		26	18.37	0.769	20.346
		32.5	18.70	0.615	16.415

Attachment 1

OD			Pipe inside diameter (d)	Minimum Wall Thickness (t)	Weight (w)
Nominal in.	Actual in.	SDR	in.	in.	lb. per foot
		9	16.82	2.444	65.412
		9.3	16.98	2.366	63.574
		11	17.76	2.000	54.818
		11.5	17.94	1.913	52.677
22	22.000	13.5	18.55	1.630	45.548
		15.5	18.99	1.419	40.107
		17	19.26	1.294	36.805
		21	19.78	1.048	30.172
		26	20.21	0.846	24.619
		32.5	20.56	0.677	19.863
		9	18.35	2.667	77.845
		9.3	18.53	2.581	75.658
		11	19.37	2.182	65.237
		11.5	19.58	2.087	62.690
24	24.000	13.5	20.23	1.778	54.206
		15.5	20.72	1.548	47.731
		17	21.01	1.412	43.801
		21	21.58	1.143	35.907
		26	22.04	0.923	29.299
		32.5	22.43	0.738	23.638
		11	22.60	2.545	88.795
		11.5	22.84	2.435	85.329
		13.5	23.60	2.074	73.781
		15.5	24.17	1.806	64.967
28	28.000	17	24.51	1.647	59.618
		21	25.17	1.333	48.874
		26	25.72	1.077	39.879
		32.5	26.17	0.862	32.174
		11	24.22	2.727	101.934
		11.5	24.47	2.609	97.954
		13.5	25.29	2.222	84.697
		15.5	25.90	1.935	74.580
30	30.000	17	26.26	1.765	68.439
		21	26.97	1.429	56.105
		26	27.55	1.154	45.779
		32.5	28.04	0.923	36.934

Attachment 2

Source: "Monitoring Approaches for Landfill Bioreactors", Carson, et. al, December 2004, EPA/600/R-04/301.

4.2 Analytical Monitoring Parameters

4.2.1 Leachate Monitoring

Suggested leachate monitoring parameters for MSW bioreactor landfills to enhance operational control under RCRA and the RD&D (FR 2004) rule are divided into primary (Table 3) and secondary parameters (Table 4). The primary parameters are relatively inexpensive and easy to examine. Parameters presented in the secondary list are more research-oriented, and are more time intensive and as a result are relatively costly. Suggested analysis methods and monitoring frequencies are presented in the tables. It is the responsibility of the owner/operator of a landfill to comply with all existing local, state and federal regulations with regard to existing monitoring parameters in addition to parameters agreed upon under an RD&D project. A few monitoring parameters are evaluated further in the following sections.

Sample duplication is necessary to account for the statistical relevance of monitoring data. Caution must also be taken to hydraulically isolate bioreactor from adjacent conventional landfill cells. The separation would allow for an accurate evaluation of conditions only within the bioreactor. Hydraulic separation may be achieved in an as-built cell; however, such a separation is more difficult in retrofit bioreactor landfill cells. In the case of a retrofit bioreactor landfill, it is suggested that the zone of influence of the liquid application area be examined (see Reinhart and Townsend, 1998 for more details). Leachate samples representing the bioreactor section of the landfill should be collected only from areas directly under the zone of influence.

4.2.1.1 Leachate Temperature

Research suggests that anaerobic processes occur best within either mesophilic (30-38°C) or thermophilic (50 to 60°C) temperature ranges (McCarty 1964; Parkin and Owen 1986). Optimum methane generation from solid wastes, however, occurred

at 41°C (Harts et al. 1982). Regardless of the operational temperatures, the maintenance of a uniform temperature is considered to be fundamental to anaerobic stabilization process efficiencies. Historically, conventional landfills leachate temperature ranged from 7 to 25°C while bioreactor landfills leachate ranged from 6 to 37°C (EPA 2003).

Because landfill temperature is not externally controlled, it reflects a combination of ambient temperature conditions, microbial activities and the extent and effectiveness of insulation provided by the landfill configuration. In cold climates, for example, leachate temperatures could initially be as low as 6°C. Soon after recirculation the leachate temperatures should steadily increase. Leachate temperatures at the Outer Loop landfill in Kentucky were initially around 7°C. The leachate temperatures at the landfill steadily increased to above 30°C within a few months of the bioreactor operation (EPA 2003). While an increase in leachate temperature is reflective of waste degradation in a landfill, it is not solely indicative of biological activity.

4.2.1.2 Leachate pH

The optimum pH range for anaerobic systems ranges between 6.5 and 7.6 (Parkin and Owen 1986). Gas generation and stabilization rates have been reported to be the highest at near neutral pH levels (Pohland et al. 1993). Initially the leachate pH may be neutral, however after the onset of anaerobic conditions there may be a pH drop especially during the acid forming phase (see section 3.2). The pH drop is most likely caused by VFAs production and accumulation in the leachate. The pH, however, will tend to move to neutrality as methanogens consume these acids. Historically there may not be a measured difference between the leachate pH measured in conventional and in bioreactor landfills. The leachate pH ranged between 4.7-8.8 for conventional landfills (EPA 2003; Kjeldsen et al. 2002; Chu et al. 1994; Krung and Ham 1991) and from 5.4-8.6 for bioreactor landfills (EPA 2003; Pohland and Harper 1986).

Off-Highway and Unpaved Road Loads

Off-highway vehicles may be considerably heavier than H20 or HS20 trucks, and these vehicles frequently operate on unpaved roads that may have uneven surfaces. Thus impact factors higher than 1.5 may be reached depending on the vehicle speed. Except for slow traffic, an impact factor of 2.0 to 3.0 should be considered.

During construction, both permanent and temporary underground pipelines may be subjected to heavy vehicle loading from construction equipment. A designated vehicle crossing with special design measures such as temporary pavement or structural sheeting may be prudent, as well as vehicle speed controls to limit impact loading.

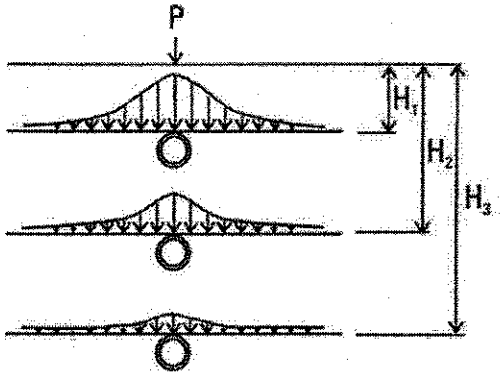
Vehicular Loads As Point Loads

There are generally two approaches for calculating vehicle live load surcharge pressure. The more conservative approach is to treat the wheel load as a concentrated (point) load. The other is to treat it as a distributed load spread over the contact area of the tire with the ground (imprint area). The pressure due to a distributed load and the pressure due to a concentrated load begin to approach the same value at a depth of about twice the square root of the loaded area.

The distributed load method gives more realistic values where the depth equals less than twice the square root of the loaded area, whereas for deeper depths concentrated loads are preferred because the calculations are simpler and typically more conservative.

The pressure distribution under a concentrated load varies with depth as illustrated in Figure 7-9. When the live load is calculated using the point load methods in the following sections, a conservative approach is to assume that the maximum pressure at the pipe crown is distributed across the entire pipe.

Figure 7-9 Concentrated Vehicular Load Pressure Distribution at Various



A key consideration in determining live load pressure on the pipe is the location of vehicle wheels relative to the pipe. A higher pressure may occur below a point between two vehicles passing in adjacent lanes than directly under a single vehicle wheel. This depends on the depth of cover.

When depths are not greater than four or five feet, the combined H20 load for two separate wheels straddling the pipe is greater than that for a single wheel directly over the pipe. Deeper than five feet, H20 loads are not usually significant because the load is attenuated significantly compared loads under one or two feet of cover. However, greater live loads may produce design significant effects at depths greater than five feet. Therefore, the designer should check load conditions for a single wheel directly over the pipe, and for two wheels spaced six feet apart and centered over the pipe.

Single Wheel Load Centered On Pipe

To check a single wheel load centered directly over the pipe, a method based on Holl's integration of Boussinesq's equation assumes that the wheel load is a concentrated (point) load. Holl's integration finds the pressure at the depth of the pipe crown that is distributed over a surface three feet long and the width of the pipe outside diameter.

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Compressive Ring Thrust (Wall Crushing)

8" Leachate Collection Pipes

$$S_A = \frac{(P_{RD}) \times DR}{288}$$

Where:	S_A	= compressive stress, psi	=	220
	P_{RD}	= radial directed waste or soil pressure, psf	=	6,809
	DR	= Dimension Ratio, D_O/t	=	9.3
	D_O	= pipe outside diameter, in.	=	8.625
	t	= pipe wall thickness, in.	=	0.927

$$P_{RD} = (VAF) \times \gamma \times H = (VAF) \times PT \qquad < 4 >$$

Where:	P_{RD}	= radial directed earth waste or soil pressure, psf	=	6,809
	VAF	= Vertical Arching Factor	=	0.975
	P_T	= total external pressure, psf	=	6,984

$$VAF = 0.88 - 0.71 \times \frac{S_R - 1}{S_R + 2.5}, \qquad < 5 >$$

Where:	VAF	= Vertical Arching Factor	=	0.975
	S_R	= hoop thrust stiffness Ratio	=	0.586

$$S_R = \frac{1.43 \times M_S \times r_{cent}}{E \times t} \qquad < 6 >$$

Where:	S_R	= hoop thrust stiffness Ratio	=	0.586
	M_S	= one-dimensional modulus of waste and soil, psi	=	2,270
	r_{cent}	= radius to centroidal axis of pipe, in.		
		$=r_{pipe}$ - t/2	=	3.85
	E	= apparent modulus of elasticity of pipe material, psi	=	23,000
	t	= pipe wall thickness	=	0.927

Allowable = Recommended long-term compressive strength design value for HDPE pipe from Plastic Pipe Institute, 2006

Allowable (psi), $S_A \leq$ 780

Actual (psi), $S_A =$ 220

FS = Allowable/actual

FS = 3.5

Ring Deflection Analysis

8" Leachate Collection Pipes

Calculate the secant modulus of the soil, E_S, using equation 10; determine the rigidity factor, R_F, using equation 9 and the soil strain using equation 8. The rigidity factor is used to determine the deformation factor which is used to calculate the percent deflection, $\Delta X/D_M$ x 100, of the pipe using equation 7.

$$E_S = M_S \times \frac{(1 + \mu) \times (1 - 2 \times \mu)}{(1 - \mu)} \qquad < 10 >$$

Where:	E_S =	secant modulus of the granular drainage layer, psi	=	2,150
	M_S =	one-dimensional modulus, (for 90% standard proctor and vertical waste and soil stress (Table 2-14, Plastic Pipe Institute, 2006)	=	2,270
	μ =	Poisson's ratio of soil, used to represent the granular drainage layer material (Table 2-15, Plastic Pipe Institute, 2006)	=	0.15

$$R_F = \frac{12 \times E_S \times (DR - 1)^3}{E} \qquad < 9 >$$

Where:	R_F =	rigidity factor	=	641
	E_S =	Secant modulus of the soil, psi (see calculated value above)	=	2,150
	DR =	dimension ratio, (DR = D_o/t)	=	9.3
			D_o =	8.625
			t =	0.927
	E =	apparent modulus of elasticity of pipe material, psi	=	23,000

$$\varepsilon = \frac{P_T}{144 \times E_S} \qquad < 8 >$$

Where:	P_T =	total external pressure, psf	=	6,984.00
	E_S =	Secant modulus of the soil, psi (see calculated value above)	=	2,150
	ε	= soil strain	=	0.0226

$$\Delta X/D_M = D_F \times \varepsilon \qquad < 7 >$$

Where:	$\Delta X/D_M$	= deflection	=	0.0260
	D_F	= Deformation Factor, (Figure 2-6, Plastic Pipe Institute, 2006) for R_F as calculated above.	=	1.15
	ε	= soil strain	=	0.0226

Percent Deflection = deflection x 100 = 2.599

The recommended allowable ring deflection for non-pressure pipe is 7.5% which provides a large factor of safety against instability and strain (Plastic Pipe Institute, Chapter 6, 2006).

Is the ring deflection allowable? (actual < allowable)	**Yes**

Pipe Wall Buckling Analysis
8" Leachate Collection Pipes

Calculate the allowable pipe wall buckling pressure, $P_{CR.}$

$$P_{CR} = \frac{2.4 \times \varphi \times R_H}{D_M} \times (E \times I)^{1/3} \times (E_S^*)^{2/3} \qquad < 11 >$$

Where:	P_{CR}	= allowable buckling pressure, psi	=	367
	φ	= Calibration Factor	=	0.55
	R_H	= Geometry Factor, (equation 2-29, Plastic Pipe Institute, 2006)	=	1
	D_M	= mean diameter (D_O – t), in.	=	7.698
	D_O	= pipe outside diameter, in.	=	8.625
	t	= pipe wall thickness, in.	=	0.927
	E	= apparent modulus of elasticity of pipe material, psi	=	23,000
	I	= pipe wall moment of inertia, in^4/in ($t^3/12$)	=	0.0664
	E_S^*	= $E_S/(1-\mu)$, psi	=	2,529
	E_S	= Secant modulus of the soil, (see calculation in Ring Deflection)	=	2,150
	μ	= Poisson's ratio of soil, (from Table 2-15, used to represent the granular drainage layer material, Plastic Pipe Institute, 2006)	=	0.15

The allowable buckling pressure should be compared to the total "static load" pressure, P_T, on the pipe.

Actual, psi: P_T = 48.5

Allowable, psi: P_{CR} = 367

Factor of safety for wall buckling is:

F.S. = P_{CR} / P_T = 7.57

Compressive Ring Thrust (Wall Crushing)

18" Leachate Extraction Riser Pipe

$$S_A = \frac{(P_{RD}) \times DR}{288}$$

Where:	S_A	= compressive stress, psi	=	291
	P_{RD}	= radial directed waste or soil pressure, psf	=	6,348
	DR	= Dimension Ratio, D_O/t	=	13.2
	D_O	= pipe outside diameter, in.	=	18
	t	= pipe wall thickness, in.	=	1.363

P_{RD} = (VAF) x γ x H = (VAF) x PT < 4 >

Where:	P_{RD}	= radial directed earth waste or soil pressure, psf	=	6,348
	VAF	= Vertical Arching Factor	=	0.909
	P_T	= total external pressure, psf	=	6,984

$$VAF = 0.88 - 0.71 \times \frac{S_R - 1}{S_R + 2.5}, \quad < 5 >$$

Where:	VAF	= Vertical Arching Factor	=	0.909
	S_R	= hoop thrust stiffness Ratio	=	0.862

$$S_R = \frac{1.43 \times M_S \times r_{cent}}{E \times t} \quad < 6 >$$

Where:	S_R	= hoop thrust stiffness Ratio	=	0.862
	M_S	= one-dimensional modulus of waste and soil, psi	=	2,270
	r_{cent}	= radius to centroidal axis of pipe, in.		
		$=r_{pipe}$ - t/2	=	8.32
	E	= apparent modulus of elasticity of pipe material, psi	=	23,000
	t	= pipe wall thickness	=	1.363

Allowable = Recommended long-term compressive strength design value for HDPE pipe from Plastic Pipe Institute, 2006

Allowable (psi), $S_A \leq$ 780

Actual (psi), $S_A =$ 291

FS = Allowable/actual

FS = 2.6

Ring Deflection Analysis

18" Leachate Extraction Riser Pipe

Calculate the secant modulus of the soil, E_S, using equation 10; determine the rigidity factor, R_F, using equation 9 and the soil strain using equation 8. The rigidity factor is used to determine the deformation factor which is used to calculate the percent deflection, $\Delta X/D_M$ x 100, of the pipe using equation 7.

$$E_S = M_S \times \frac{(1 + \mu) \times (1 - 2 \times \mu)}{(1 - \mu)} \qquad < 10 >$$

Where:	E_S =	secant modulus of the granular drainage layer, psi	=	2,150
	M_S =	one-dimensional modulus, (for 90% standard proctor and vertical waste and soil stress (Table 2-14, Plastic Pipe Institute, 2006)	=	2,270
	μ =	Poisson's ratio of soil, used to represent the granular drainage layer material (Table 2-15, Plastic Pipe Institute, 2006)	=	0.15

$$R_F = \frac{12 \times E_S \times (DR - 1)^3}{E} \qquad < 9 >$$

Where:	R_F =	rigidity factor	=	2,037
	E_S =	Secant modulus of the soil, psi (see calculated value above)	=	2,150
	DR =	dimension ratio, (DR = D_o/t)	=	13.2
			D_o =	18
			t =	1.363
	E =	apparent modulus of elasticity of pipe material, psi	=	23,000

$$\varepsilon = \frac{P_T}{144 \times E_S} \qquad < 8 >$$

Where:	P_T =	total external pressure, psf	=	6,984.00
	E_S =	Secant modulus of the soil, psi (see calculated value above)	=	2,150
	ε	= soil strain	=	0.0226

$$\Delta X/D_M = D_F \times \varepsilon \qquad < 7 >$$

Where:	$\Delta X/D_M$	= deflection	=	0.0316
	D_F	= Deformation Factor, (Figure 2-6, Plastic Pipe Institute, 2006) for R_F as calculated above.	=	1.4
	ε	= soil strain	=	0.0226

Percent Deflection = deflection x 100 = 3.164

The recommended allowable ring deflection for non-pressure pipe is 7.5% which provides a large factor of safety against instability and strain (Plastic Pipe Institute, Chapter 6, 2006).

Is the ring deflection allowable? (actual < allowable)	**Yes**

Pipe Wall Buckling Analysis
18" Leachate Extraction Riser Pipe

Calculate the allowable pipe wall buckling pressure, $P_{CR.}$

$$P_{CR} = \frac{2.4 \times \varphi \times R_H}{D_M} \times (E \times I)^{1/3} \times (E_S^*)^{2/3} \qquad < 11 >$$

Where:	P_{CR}	= allowable buckling pressure, psi	=	249
	φ	= Calibration Factor	=	0.55
	R_H	= Geometry Factor, (equation 2-29, Plastic Pipe Institute, 2006)	=	1
	D_M	= mean diameter (D_O – t), in.	=	16.637
	D_O	= pipe outside diameter, in.	=	18
	t	= pipe wall thickness, in.	=	1.363
	E	= apparent modulus of elasticity of pipe material, psi	=	23,000
	I	= pipe wall moment of inertia, in^4/in ($t^3/12$)	=	0.211
	E_S^*	= $E_S/(1-\mu)$, psi	=	2,529
	E_S	= Secant modulus of the soil, (see calculation in Ring Deflection)	=	2,150
	μ	= Poisson's ratio of soil, (from Table 2-15, used to represent the granular drainage layer material, Plastic Pipe Institute, 2006)	=	0.15

The allowable buckling pressure should be compared to the total "static load" pressure, P_T, on the pipe.

Actual, psi: P_T = 48.5

Allowable, psi: P_{CR} = 249

Factor of safety for wall buckling is:

F.S. = P_{CR} / P_T = 5.13

Appendix D.4
Leachate Evaporation Pond Analysis

Client	County of Kaua'i	**Subject**	Leachate	**Prepared By** NKW	**Date** 8/19/15
Project	Kekaha Landfill		Evaporation Lagoon Analysis	**Reviewed By** DFP	**Date** 12/11/15
	Cell 2, Phase II			**Approved By** NKW	**Date** 12/14/15

LEACHATE EVAPORATION LAGOON ANALYSIS

Objective

Evaluate the ability of the proposed leachate evaporation lagoon to properly process the Cell 2 lateral expansion estimated leachate quantities, Cell 1 leachate quantities, Phase II leachate quantities, and rainfall.

Design Criteria and Assumptions

1. The leachate evaporation lagoon will function by storing leachate during times of excess leachate generation until the excess leachate can be evaporated. To facilitate the analysis the lagoon is evaluated on a monthly basis.

2. The proposed evaporation lagoon is evaluated based on a water balance, comparing the liquid entering the evaporation lagoon to anticipated evaporation quantities.

3. It is assumed that the evaporation rates from the evaporation lagoon will be similar to pan evaporation rates due to the confined nature, relatively shallow depth, and black geomembrane lining in the lagoon.

4. Estimated pan evaporation rates from two sources were reviewed (see Attachment 1). To be very conservative, the pan evaporation rate of 64.5 inches annually for the weather station at HILO WSO, Hawaii was used in this calculation. For comparison, the pan evaporation rate at the weather station in LIHUE WSO, is 96.5 inches/year.

5. Estimated monthly leachate generation rates based on active conditions for Cells 1 and 2 of the Phase II lateral expansion were used in this analysis (see the Leachate Generation Analysis calculation).

6. Average monthly rainfall data based on the ten years (2001 through 2010) of available site data was used in this analysis.

7. Monthly leachate generation data between September 2004 and December 2010 for Kekaha Landfill was reviewed. To be conservative the year with the greatest leachate generation, 2006, was used in this analysis. The average leachate generation was also considered.

8. All references to elevations correlate to the site-specific vertical datum.

9. The planar area of the evaporation lagoon available for evaporation, 1.63 acres, was determined at the average depth, elevation 12 feet.

10. The planar area of Cell 1 of the Phase II Lateral Expansion is 6.4 acres. The planar area of Cell 2 is 6.6 acres.

11. The gross capacity of the leachate evaporation lagoon is approximately 21,000 cy or 4,241,000 gallons. The bottom of the lagoon has 1-foot of gravel and the sideslopes have 8 inches of drainage material supported in place by a geoweb system. The total volume of base and sideslope material is approximately 2,682 cy or 542,000 gallons. Assuming no porosity of the base and sideslope material (conservative), the net volume of the evaporation lagoon is 3,699,000 gallons.

Calculations

The calculation of the annual water balance is based on month-to-month additions to the leachate evaporation lagoon due to rainfall and leachate using the year with greatest leachate generation and subtractions due to average pan evaporation for each month. The calculations for this worst-case scenario are summarized in Table 1, Evaporation Lagoon Month-to-Month Water Balance – Maximum Annual Leachate Generation, included as Attachment 2.

Calculations of the annual water balance based on the average leachate generation are summarized in Table 2, Evaporation Lagoon Month-to-Month Water Balance – Average Annual Leachate Generation, included as Attachment 3.

Sample calculations from Attachment 2 for the month with the greatest leachate generation, March, are provided below.

Cell 2 leachate (convert HELP model output from in/ac to gallons) for the month of March:

$$\text{Cell 2 Leachate} = \left(\frac{0.5197 \text{ inches}}{\text{month}}\right)\left(\frac{1 \text{ foot}}{12 \text{ in.}}\right)\left(\frac{43{,}560 \text{ sf}}{\text{acre}}\right)\left(\frac{7.48 \text{ gal.}}{1 \text{ cf}}\right) \text{x } 6.6 \text{ ac.} = 93{,}133 \text{ gal/ month}$$

Total leachate generated (in inches of evaporation lagoon) for the month of March:

Total leachate = (existing leachate + Cell 1 leachate + Cell 2 Leachate)
average lagoon surface area

$$= \left(\frac{\left(670{,}304 \text{ gal}/\text{month} + 18{,}037.9 \text{ gal}/\text{month} + 92{,}506 \text{ gal}/\text{month}\right) \text{x } 1 \text{ cf}/7.48 \text{ gal}}{1.63 \text{ acres x } 43{,}560 \text{ sf}/\text{acre}}\right) \text{x } \frac{12 \text{ in}}{1 \text{ ft}} = 17.64 \text{ in/month}$$

Pan Evaporation for March: 5.15 inches

Monthly water balance (inches) for March:

Monthly water balance = Total leachate + Rainfall – evaporation
= 17.64 inches + 3.20 inches – 5.15 inches = 15.69 inches

Cumulative Required Storage from inches to cubic yards for the month of March:

Cumulative required storage = Cumulative water storage * average lagoon area

$$= 15.71 \text{ inches} * \left(\frac{1 \text{ ft}}{12 \text{ inches}}\right) * \left(\frac{43{,}560 \text{ sf}}{1 \text{ acre}}\right) * (1.63 \text{ acres}) * \left(\frac{1 \text{ cy}}{27 \text{ cf}}\right) = 3{,}443 \text{ cy} = 695{,}300 \text{ gallons}$$

When considering maximum annual leachate generation, Attachment 2, the estimated required maximum leachate and rainfall storage volume of 894,500 gallons occurs during the month of April as shown in Attachment 2.

Factor of Safety:

The factor of safety, F.S., for the leachate evaporation lagoon, when maximum monthly cumulative leachate volume is considered, is calculated as follows:

F.S. = Storage Volume Available / Volume required
= 3,699,000 gallons/ 894,500 gallons
= 4.14

Note that when average leachate quantities are considered, as in Attachment 3, the factor of safety is larger.

Conclusion

Based on estimated average monthly leachate flows (Table 2 in Attachment 3), evaporation and rainfall, the proposed evaporation lagoon is able to process anticipated leachate and rainfall quantities. The proposed evaporation lagoon has a net volume of 3,699,000 gallons. When maximum annual leachate generation is considered (Table 1 in Attachment 2), the estimated maximum required leachate and rainfall storage volume occurs during the month of April, and is approximately 895,000 gallons providing a factor of safety of over 4.1. Leachate that may be remaining in the lagoon after years when leachate generation exceeds evaporation will be evaporated during the subsequent years when leachate generation is less than evaporation.

Attachment 1
Hawaii Pan Evaporation Rates

Source 1: GROUND WATER ATLAS of the UNITED STATES
Alaska, Hawaii, Puerto Rico and the U. S. Virgin Islands
HA 730-N
http://capp.water.usgs.gov/gwa/ch_n/N-HItext1.html

Evapotranspiration, which is the loss of water to the atmosphere by the combination of transpiration of plants and direct evaporation from land and water surfaces, is a major component of the hydrologic budget of the islands. In the Honolulu area of Oahu, for example, actual evapotranspiration was estimated to be about 40 percent of the total water (rainfall plus irrigation) falling on or applied to the ground surface during 1946-75. Pan evaporation is the main measurement used in Hawaii to assess the amount of water loss by evapotranspiration. Over the open ocean, the estimated annual pan-evaporation rate is 65 inches. As with precipitation, pan-evaporation rates in Hawaii are related to topography. At altitudes between 2,000 and 4,000 feet, where humidity is high and sunlight intensity is reduced because of clouds, pan-evaporation rates are reduced to as low as 25 percent of the open-ocean rate. In the leeward coastal areas, wind carrying dry, warm air increases annual pan-evaporation rates to as much as 100 inches. At the summits of Mauna Kea and Mauna Loa on the island of Hawaii, annual pan-evaporation rates exceed 70 inches because of clear skies and dry air.

Source 2: Western Regional Climate Center http://www.wrcc.dri.edu/htmlfiles/westevap.final.html#HAWAII/PACIFIC%20ISLANDS

Evaporation Stations
Standard daily pan evaporation is measured using the 4-foot diameter Class A evaporation pan. The pan water level reading is adjusted when precipitation is measure to obtain the actual evaporation. Most Class A pans are installed above ground, allowing effects such as radiation on the side walls and heat exchanges with the pan material. These effects tend to increase the evaporation totals. The amounts can then be adjusted by multiplying the totals by 0.70 or 0.80 to more closely estimate the evaporation from naturally existing surfaces such as a shallow lake, wet soil or other moist natural surfaces.

HAWAII
MONTHLY AVERAGE PAN EVAPORATION (INCHES)

	Period Of Record	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Year
HILO WSO AP 87	*1949-2005*	*5.06*	*4.87*	*5.15*	*5.40*	*5.66*	*6.49*	*6.44*	*6.13*	*5.40*	*5.37*	*4.06*	*4.46*	*64.49*
HONOLULU OBSRVY 702.2	*1962-2005*	*4.72*	*5.23*	*7.01*	*7.84*	*8.93*	*9.41*	*10.10*	*10.09*	*8.82*	*7.68*	*5.94*	*5.08*	*90.85*
LIHUE WSO AP 1020.1	*1950-2005*	*5.62*	*6.22*	*7.62*	*8.22*	*9.21*	*9.85*	*10.40*	*10.21*	*9.18*	*8.04*	*6.27*	*5.67*	*96.51*

For Lihue, Kaua'i: 96.51 inches (annual pan evaporation) x 0.70 (correction factor from above) = 67.56 inches
96.51 inches (annual pan evaporation) x 0.80 (correction factor from above) = 77.21 inches

Attachment 2

Table 1: Evaporation Lagoon Month-to-Month Water Balance - Maximum Annual Leachate Generation
Kekaha Landfill
Phase II Lateral Expansion, Cell 2

Month	Rainfall[1] (in)	Existing Leachate[2] (gal)	Cell 1[1] (in/ac)	Cell 1 Leachate (gal)	Cell 2[1] (in/ac)	Cell 2 Leachate (gal)	Total Leachate[3] (in)	Pan Evap.[4] (in)	Monthly Water Balance[5] (in)	Cumulative Water Balance[6] (in)	Cumulative Required Storage Volume (cy)	Cumulative Required Storage Volume (gal)
Jan	2.17	12,578	0.3377	58,683.9	0.5898	105,695.6	4.00	5.06	1.11	1.11	243	49,100
Feb	1.55	18,496	0.2763	48,014.1	0.6736	120,713.1	4.23	4.87	0.91	0.91	199	40,300
Mar	3.2	670,304	0.1038	18,037.9	0.5197	93,133.3	17.66	5.15	15.71	15.71	3,443	695,300
Apr	0.84	284,102	0.2083	36,197.4	0.4497	80,588.9	9.06	5.40	4.50	20.21	4,429	894,500
May	0.45	48,090	0.3185	55,347.5	0.5213	93,420.0	4.45	5.66	-0.76	19.45	4,262	860,800
Jun	0.73	36,993	0.0938	16,300.1	0.3993	71,556.9	2.82	6.49	-2.94	16.51	3,618	730,700
Jul	0.28	19,237	0.0606	10,530.8	0.2613	46,826.5	1.73	6.44	-4.43	12.08	2,647	534,600
Aug	1.03	36,993	0.0473	8,219.6	0.0942	16,881.2	1.40	6.13	-3.70	8.38	1,836	370,900
Sept	0.97	27,374	0.0331	5,752.0	0.1020	18,279.0	1.16	5.40	-3.27	5.11	1,120	226,200
Oct	1.08	68,806	0.0622	10,808.8	0.1030	18,458.2	2.22	5.37	-2.07	3.04	666	134,500
Nov	2.15	30,334	0.0675	11,729.8	0.1164	20,859.6	1.42	4.06	-0.49	2.55	559	112,900
Dec	4.48	19,236	0.0796	13,832.5	0.1137	20,375.7	1.21	4.46	1.23	3.78	828	167,300
TOTAL:	18.93	1,272,543	1.6887	293,454.5	3.944	706,787.8	51.36	64.49				

Notes:
1. Rainfall in inches and expansion leachate generation in inches/acre obtained from HELP Model output included in the "Leachate Generation Analysis" calculation.
2. Existing leachate generation quantities determined from the maximum annual existing leachate generation based on site data from September 2004 through December 2010.
3. Total leachate generation is displayed in inches of evaporation pond depth at average pond area.
4. Pan evaporation data from the weather station at HILO WSO as indicated in Attachment 1.
5. Negative monthly water balance indicates evaporation is greater than the liquid entering the basin.
6. Negative water balance quantities can not be accumulated, therefore, the minimum cumulative water balance is 0.

Attachment 3

Table 2: Evaporation Lagoon Month-to-Month Water Balance - Average Leachate Quantities
Kekaha Landfill
Phase II Lateral Expansion, Cell 2

Month	Rainfall[1] (in)	Existing Leachate[2] (gal)	Cell 1[3] (in/ac)	Cell 1 Leachate (gal)	Cell 2[1] (in/ac)	Cell 2 Leachate (gal)	Total Leachate[3] (in)	Pan Evap.[4] (in)	Monthly Water Balance[5] (in)	Cumulative Water Balance[6] (in)	Cumulative Required Storage Volume (cy)	Cumulative Required Storage Volume (gal)
Jan	2.17	55,000	0.3377	58,683.9	0.5898	105,695.6	4.96	5.06	2.07	2.07	454	91,600
Feb	1.55	35,373	0.2763	48,014.1	0.6736	120,713.1	4.61	4.87	1.29	1.29	283	57,100
Mar	3.2	130,983	0.1038	18,037.9	0.5197	93,133.3	5.47	5.15	3.52	3.52	771	155,800
Apr	0.84	59,193	0.2083	36,197.4	0.4497	80,588.9	3.98	5.40	-0.58	2.94	644	130,100
May	0.45	18,552	0.3185	55,347.5	0.5213	93,420.0	3.78	5.66	-1.43	1.51	331	66,800
Jun	0.73	17,349	0.0938	16,300.1	0.3993	71,556.9	2.38	6.49	-3.38	0.00	0	0
Jul	0.28	12,907	0.0606	10,530.8	0.2613	46,826.5	1.59	6.44	-4.57	0.00	0	0
Aug	1.03	17,750	0.0473	8,219.6	0.0942	16,881.2	0.97	6.13	-4.13	0.00	0	0
Sept	0.97	19,062	0.0331	5,752.0	0.1020	18,279.0	0.97	5.40	-3.46	0.00	0	0
Oct	1.08	19,849	0.0622	10,808.8	0.1030	18,458.2	1.11	5.37	-3.18	0.00	0	0
Nov	2.15	21,899	0.0675	11,729.8	0.1164	20,859.6	1.23	4.06	-0.68	0.00	0	0
Dec	4.48	193,593	0.0796	13,832.5	0.1137	20,375.7	5.15	4.46	5.17	5.17	1,133	228,800
TOTAL:	18.93	601,508	1.6887	293,454.5	3.944	706,787.8	36.20	64.49				

Notes:
1. Rainfall in inches and expansion leachate generation in inches/acre obtained from HELP Model output included in the "Leachate Generation Analysis" calculation.
2. Existing leachate generation quantities determined from the average of site data from September 2004 through December 2010.
3. Total leachate generation is displayed in inches of evaporation pond depth at average pond area.
4. Pan evaporation data from the weather station at HILO WSO as indicated in Attachment 1.
5. Negative monthly water balance indicates evaporation is greater than the liquid entering the basin.
6. Negative water balance quantities can not be accumulated, therefore, the minimum cumulative water balance is 0.

Appendix D.5
Leachate Flows – Cell 2

CALCULATION SHEET

AECOM

Project No. 60197394

Client	County of Kaua'i	**Subject**	Leachate Flows	**Prepared By** NKW	**Date** 8/19/15
Project	Kekaha Landfill		For Cell 2 Leachate	**Reviewed By** DFP	**Date** 12/11/15
	Cell 2, Phase II		System Sizing	**Approved By** NKW	**Date** 12/14/15

LEACHATE FLOWS FOR CELL 2 LEACHATE SYSTEM SIZING

Objective

Determine peak leachate flows for pipe sizing, pump station sizing, and pump selection. The alternate piping system from Cell 2 to Wetwell #3 may or may not be installed in the future.

Design Criteria and Assumptions

1. Use the peak daily leachate generation rates described in Appendix D.1, of 3,705 gal/acre/day for Cell 2 when actively accepting waste and 3,554.8 gal/acre/day for Cell 1 with intermediate cover.

2. Estimate leachate quantities flowing to each of the two Cell 2 sub-cell areas.

3. As stated in the #3 Wetwell and Forcemain Design calculation dated December 2008 in the Kekaha Landfill Phase II Lateral Expansion, Wetwell #3 was designed to include a flow of 8.6 gpm from the Cell 2 area and a flow of 12.6 gpm from Cell 1.

4. As stated in the Leachate Flows for Cell 1 Leachate System Sizing calculation dated December 2008 in the Kekaha Landfill Phase II Lateral Expansion, acreage of areas of Cell 1 flowing to Wetwell #3 are 4.82 acres.

Calculations

1. Convert peak daily leachate generation from Leachate Generation Analysis in Appendix D.1 from gal/acre/day to gal/acre/min.

Cell 1: $$3{,}554.8\frac{gal}{acre-day}\ x\ \frac{1\,day}{24\,hr}\ x\ \frac{1\,hr}{60\,\text{min}}\ = 2.47\frac{gpm}{acre}$$

Cell 2: $$3{,}705\frac{gal}{acre-day}\ x\ \frac{1\,day}{24\,hr}\ x\ \frac{1\,hr}{60\,\text{min}}\ = 2.57\frac{gpm}{acre}$$

2. Calculate leachate flow per pair of Cell 2 sub-cell area.

   a. Sub-cell Area West:

      3.5 ac x 2.57 gpm/ac = 9.0 gpm

   b. Sub-cell Area East:

      3.1 ac x 2.57 gpm/ac = 8.0 gpm

3. Calculate leachate flow for Wetwell #3 from Cell 1 with intermediate cover:

   4.82 ac x 2.47 gpm/ac = 11.9 gpm

4. Calculate the changes in design flow for Wetwell #3:

   Flow into Wetwell #3 = portion of Cell 1 flow + portion of Cell 2 flow
   Note: Original design called for flow from Cell 2 East to go to Wetwell #3. Although this is not current design, we review it as a potential outcome.

   Original Design Flow = 12.6 gpm + 8.6 gpm = 21.2 gpm

   Current Design Flow = 11.9 gpm + 0 gpm = 11.9 gpm

   Possible Design Flow = 11.9 gpm + 9.1 gpm = 21.0 gpm
   Potential Change in Wetwell #3 design flow = Original design – potential design
   = 21.2 gpm – 21.0 gpm = -0.2 gpm

   Since the change in design flow to Wetwell #3 is negative, the original design pumps can handle the current design flow from Cell 2, if the alternate piping is installed and used in the future.

Conclusion

Following are the peak leachate generation rates contributing to the Cell 2 leachate transfer system:

Cell 2 West Sub-Cell Area	9.1 gpm
Cell 2 East Sub-Cell Area	8.0 gpm
Potential Net Change for Wet Well #3	-0.2 gpm <=No design changes needed for Wetwell #3.

Appendix D.6
Pump Sizing – Cell 2 Pumps

Client	County of Kaua'i	**Subject**	Leachate Pump	**Prepared By** NKW	**Date** 8/19/15
Project	Kekaha Landfill		Sizing Cell 2 Sumps	**Reviewed By** DFP	**Date** 12/11/15
	Cell 2, Phase II			**Approved By** NKW	**Date** 12/14/15

LEACHATE PUMP SIZING CELL 2 SUMPS

Objective

Evaluate the required pump size, and forcemain size from the Cell 2 sumps to the leachate evaporation pond. Determine pump on/off levels based on pump requirements and head on the liner requirements.

Design Criteria and Assumptions

1. Assume the same pump will be used in each Cell 2 sump to pump leachate to the leachate evaporation pond.

2. Use the peak daily leachate generation rates described in Appendix D.1, 3,705 gal/acre/day.

3. Determine the required pump size based on the largest area draining to one sump, approximately 2.0 acres.

4. The pumps will be installed in each sump through sideslope riser pipes. Pump selection is based on the peak daily leachate collection rate and the system head curve, which consists of the static head, friction loss, and minor losses.

5. Assume the pump discharge piping will be 1.5-inch diameter within the sideslope risers and 4.0-inch SDR 11 HDPE pipe between the sideslope risers and leachate evaporation pond.

6. All references to elevations correlate to the site-specific vertical datum.

7. Highest "pump-on" elevation is 16 inches off the bottom of the sump (10 inches above the landfill floor) to maintain leachate head levels below 1 foot on the landfill floor.

8. Lowest "pump-off" elevation is at least 10 inches above the bottom of the sump (considers flatstock thickness, riser pipe wall diameter and allows for liquid to cover pump).

9. Assume that the highest point in the system to pump out of the sumps and into above grade combined force main pipe is 3 feet above the topography in the Cell 2 riser locations on Cell 1.

10. Based on the default soil parameters used in the leachate generation analysis presented in Appendix D.1, assume the porosity of the granular drainage layer to be 0.397.

Calculations

1. Peak flow to a sump;
   Flow in = 2.0 acres x 3,705 gal/acre/day x 1 day/24 hrs x 1 hr/60 min = 5.18 gpm

2. Static Head:

High point of system	=	43 (see design criteria 9, ground elevation 40)
Low point of system	=	10.5 (conservatively the sump floor elevation)
Static head	=	32.5 feet

3. Minor loss:

The minor loss can be calculated as follows:

$$h_L = K_L \frac{V^2}{2g}$$

Where

h_L = head loss (ft)
K_L = minor loss coefficient
V = velocity (fps)
g = gravitational constant (32.2 ft/s^2)

4. Friction Loss

The Hazen-Williams equation is used to calculate the friction loss in the system:

$$V = 1.318 C_{hw} R^{0.63} S^{0.54}$$

Where,

V = flow velocity (fps)
C_{hw} = Hazen-Williams coefficient
R = Hydraulic radius
S = Slope of the energy grade line (head loss per unit length of pipe) (ft/ft)
= h_f / L
h_f = friction loss (ft)
L = pipe length (ft)

Rearranging the equation to solve for the slope of the energy grade line yields:

$$S = \left(\frac{V}{1.318 C_{hw} R^{0.63}} \right)^{1.85}$$

Solving for the friction loss yields:

$$h_f = \left(\frac{V}{1.318 C_{hw} R^{0.63}} \right)^{1.85} \times L$$

5. System Head

The system head is the summation of the static head, minor losses and friction losses at each flow rate. The calculations are included as Attachment 1.

6. Pump Selection

An EPG SurePump™ Wheeled Sump Drainer with level sensors (WSDPT) Model 5-3 with 0.75 HP motor was selected as the pump for the Cell 2 leachate sumps. The pump characteristic curve is included as Attachment 2. The intersection of the pump characteristic curve with the system head curve yields the operating point of the system. The plot of the system head curve versus the pump characteristic curves is included in Attachment 1. The pump operates near the end of its volume (gpm) range. If vibration occurs, the ball valves should be closed enough to end vibration. Closing the ball valves a bit will increase system head. The operating points are summarized in Table 1.

Table 1 - Pump Operating Points

Pump	Number of Pumps in Parallel	Operating Point	
		Flow (Each Pump) (gpm)	System Head (ft)
EPG WSDPT 5-3	4	22	55

7. Pump Control Levels

a. Pump Off level is set at 10 inches above the bottom of the sump.
b. Pump #1 On level is set at 16 inches above the bottom of the sump.
c. High Level Alarm level is set at 18 inches above the bottom of the sump.

8. Storage Volume within Pump Control Levels:
The liquid storage volume within the sump control levels is the pore space of the granular drainage layer between the pump on level and the base grades. Due to the slope of the base grades the storage volume is a triangular shape.

Therefore:

Storage Volume = ½ x area x depth x granular material porosity

Area = 1,300 sf (area at elevation 11.83, 16 inches above the bottom of the sump; the "pump on" elevation)
Depth = 0.5 ft (pump on elevation – pump off elevation)
Granular material porosity = 0.397

Storage Volume = ½ x 1,300 sf x 0.5 ft. x 0.397 = 129.0 cf = 965 gal

9. Pump Cycle Time, t_{pump}:

$t_{pump} = t_{run} + t_{off}$

Where: t_{run} = pump running time
t_{off} = pump off time (time to refill volume between pump off and pump on levels)

t_{run} = time to empty the storage volume,
t_{run} can be calculated when: flow out x time – flow in x time = storage volume
965 gal = 22 gpm x t_{run} – 5.81 gpm x t_{run}, therefore t_{run} = 60 minutes

t_{off} = storage volume / flow in = 965 gal / 5.81 gpm = 166 minutes

t_{pump} = 60 min + 166 min = 226 min

Pump cycle time will not cause excessive wear on pump.

Client	County of Kaua'i	**Subject**	Leachate Pump	**Prepared By** NKW	**Date** 8/19/15
Project	Kekaha Landfill		Sizing Cell 2 Sumps	**Reviewed By** DFP	**Date** 12/11/15
Cell 2, Phase II				**Approved By** NKW	**Date** 12/14/15

Conclusion

The design of the pumps located in the Cell 2 sumps is based on the sump requiring the largest pump, the ball valve may need to be partially closed for some sumps to increase system head and optimize pump operation:

- 1.5-inch diameter piping from the sump to the intersection with the common forcemain and 4-inch diameter common forcemain piping to the leachate evaporation pond;
- EPG SurePump WSDPT Model 5-3 with a 0.75 HP motor as the pump for the Cell 2 sumps.
- Pump control levels for the pumping system for each sump are:

High Level Alarm	18 inches above the sump floor (12 inches above the landfill floor)
Pump On	16 inches above the sump floor (10 inches above the landfill floor)
Pump Off	10 inches above the sump floor (4 inches above the landfill floor)

ATTACHMENT 1

PROPOSED CELL 2 LEACHATE SUMP PUMP DESIGN

PROJECT NO.: 60197394

CLIENT: County of Kaua'i
PROJECT: Kekaha Landfill
Phase II, Cell 2 Expansion

BY: NKW
DATE: 8/18/2015
CHK: DFP
DATE:

SUBJECT: Proposed Cell 2 Sump and Forcemain to Leachate Evaporation Pond

PIPE LENGTH

Pipe Size	Average Inner Diameter (in)	Average Inner Diameter (ft)	Length (ft)	Description
1.5	1.500	0.1250	108	1.5" dia. pipe inside sideslope riser
4.0	3.633	0.3028	2900	4.0" dia SDR 11 HDPE pipe between sideslope risers and evap. pond

STATIC HEAD

High Point of System = EL	43	ft	(3 ft above topography in force main area)
Low Point of System = EL	10.5	ft	(Conservatively the sump floor elevation)
Static Head =	32.5	ft	

MINOR LOSSES

Minor Losses for 1.5" diameter pipe inside sideslope riser

Fitting Type	Loss Coefficient K_L	No. of Fittings	Total Loss Coefficient K_L	Reference
90^o Elbow	0.30	1	0.30	4
45^o Elbow	0.20	1	0.20	4
Ball Valve	2.00	1	2.00	4
Check Valve	1.00	1	1.00	4
Branch Tee	1.00	0	0.00	4
Total			3.50	

Minor Losses for 4" diameter SDR 11 force main

Fitting Type	Loss Coefficient K_L	No. of Fittings	Total Loss Coefficient K_L	Reference
45^o Elbow	0.20	2	0.40	4
90^o Elbow	0.30	4	1.20	4
Ball Valve	2.00	2	4.00	4
Branch Tee	1.00	2	2.00	4
Running Tee	0.20	2	0.40	4
Total			8.00	

OPERATING POINTS

Pump	No. of Pumps in Parallel	Operating Point Flow (gpm)	Operating Point Head (ft)
EPG SurePump, WSDPT Series 5-3, 0.75 hp	4	27	45

CLIENT: County of Kaua'i
PROJECT: Kekaha Landfill
Phase II, Cell 2 Expansion

BY: NKW
DATE: 8/18/2015
CHK: DFP
DATE:

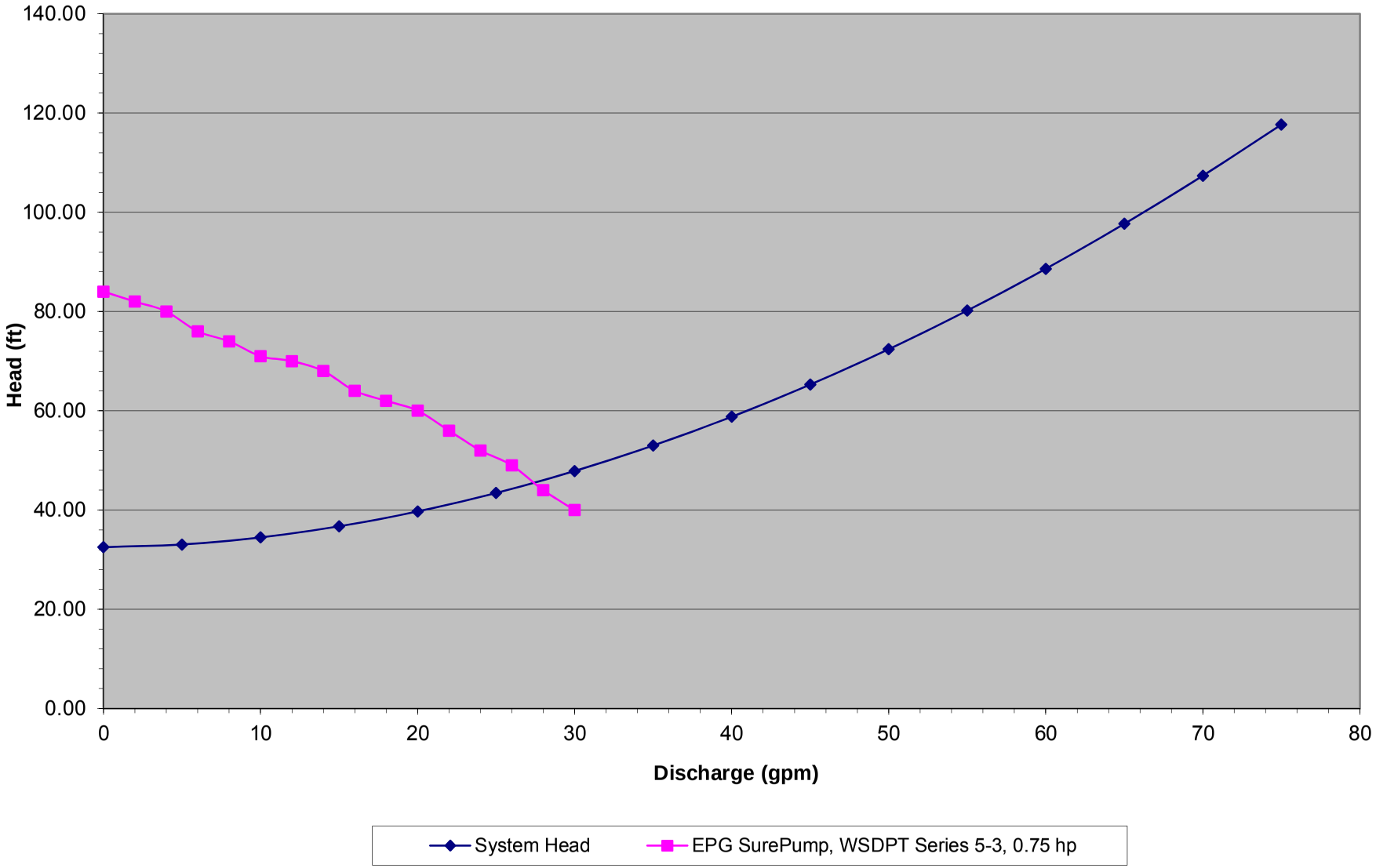
SUBJECT: Proposed Cell 2 Sump and Forcemain to Leachate Evaporation Pond
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	1.5-IN	4-IN SDR 11 HDPE
LENGTH (FT) =	108	2900
INSIDE DIAMETER (FT) =	0.1250	0.3028
FLOW AREA (SF) =	0.01227	0.07199
HYDRAULIC RADIUS (FT) =	0.0313	0.0757
C_{HW} =	130	140
TOTAL LOSS COEFFICIENT (K_L) =	3.50	8.00

STATIC HEAD = 32.5 ft

Q	1.5-IN				4-IN SDR 11 HDPE				$h_L + h_f$	TDH
(gpm)	V (ft/sec)	h_L (ft)	S (ft/ft)	h_f (ft)	V (ft/sec)	h_L (ft)	S (ft/ft)	h_f (ft)	(ft)	(ft)
0	0.00	0.00	0.0000	0.00	0.00	0.00	0.0000	0.00	0.00	32.50
5	0.91	0.04	0.0035	0.38	0.15	0.00	0.0000	0.12	0.54	33.04
10	1.82	0.18	0.0126	1.36	0.31	0.01	0.0001	0.43	1.97	34.47
15	2.72	0.40	0.0266	2.87	0.46	0.03	0.0003	0.91	4.21	36.71
20	3.63	0.72	0.0453	4.90	0.62	0.05	0.0005	1.54	7.20	39.70
25	4.54	1.12	0.0685	7.40	0.77	0.07	0.0008	2.33	10.92	43.42
30	5.45	1.61	0.0960	10.37	0.93	0.11	0.0011	3.27	15.36	47.86
35	6.35	2.19	0.1278	13.80	1.08	0.15	0.0015	4.35	20.49	52.99
40	7.26	2.87	0.1636	17.67	1.24	0.19	0.0019	5.57	26.29	58.79
45	8.17	3.63	0.2035	21.98	1.39	0.24	0.0024	6.92	32.77	65.27
50	9.08	4.48	0.2473	26.71	1.55	0.30	0.0029	8.41	39.90	72.40
55	9.99	5.42	0.2951	31.87	1.70	0.36	0.0035	10.04	47.68	80.18
60	10.89	6.45	0.3467	37.44	1.86	0.43	0.0041	11.79	56.11	88.61
65	11.80	7.57	0.4020	43.42	2.01	0.50	0.0047	13.68	65.17	97.67
70	12.71	8.78	0.4612	49.81	2.17	0.58	0.0054	15.69	74.86	107.36
75	13.62	10.08	0.5240	56.60	2.32	0.67	0.0061	17.83	85.17	117.67

Pump Station Design



Pump 1:
EPG SurePump, WSDPT Series 5-3, 0.75 hp

	Pump 1
Discharge (gpm)	Head (ft)
0	84
2	82
4	80
6	76
8	74
10	71
12	70
14	68
16	64
18	62
20	60
22	56
24	52
26	49
28	44
30	40

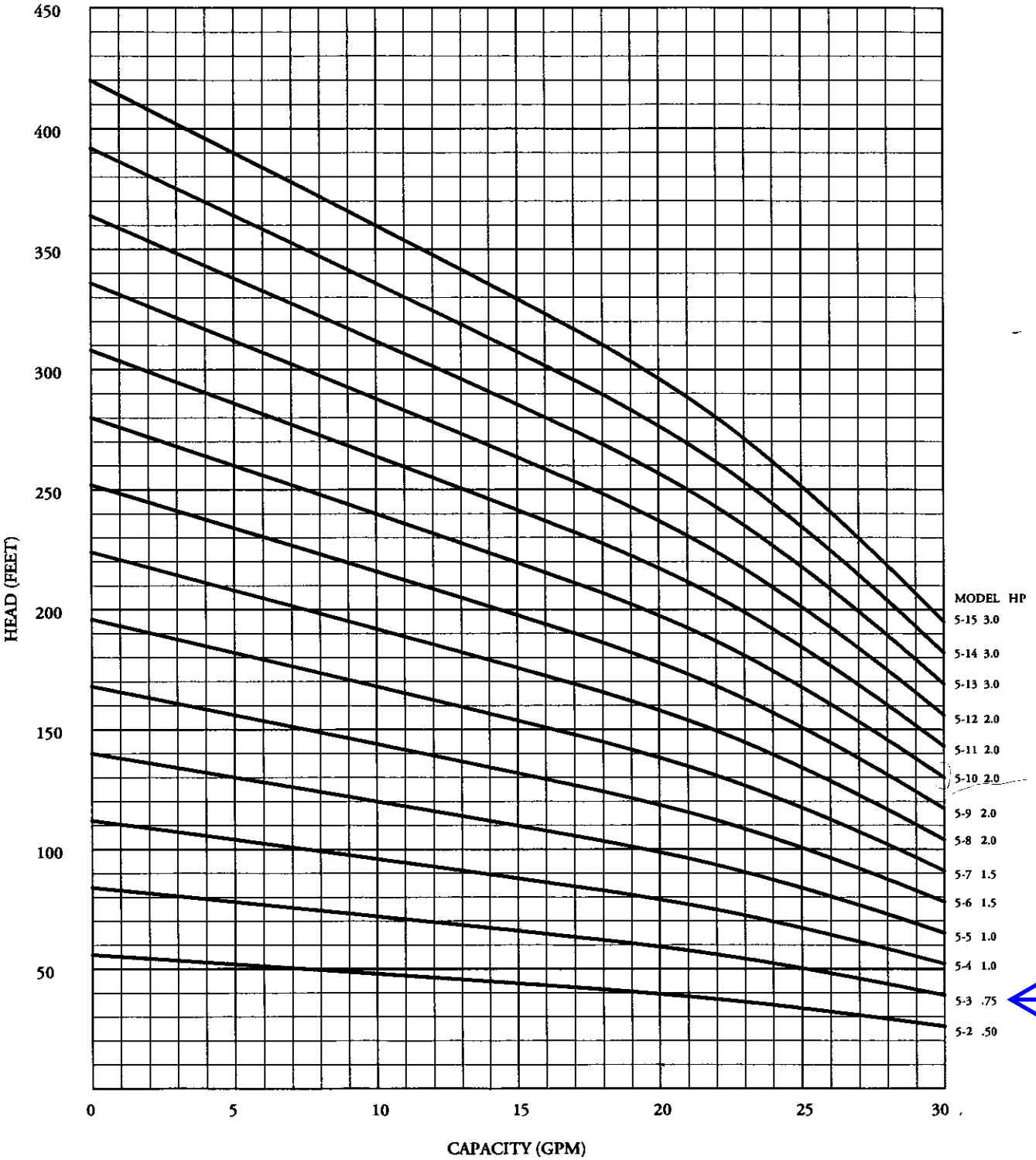
ATTACHMENT 2

PUMP CHACTERISTIC CURVE

SERIES 5 SurePump™

Flow Range 15-30 GPM

60 Hz



DATA SUBJECT TO CHANGE WITHOUT NOTICE

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SERIES 5 SUMP DRAINER SELECTION GUIDE

To select the appropriate sump drainer:

1. Select SINGLE or THREE PHASE, motor HORSEPOWER, and system supply VOLTAGE.
2. Determine the DISTANCE from the power supply to the pump.
3. Use the sump drainer size from the SUMP DRAINER column to find its respective dimensions on the following pages. MOTOR LEAD column shows minimum motor power cable size.

SINGLE PHASE MOTORS				
HORSEPOWER	VOLTAGE	DISTANCE FROM SERVICE ENTRANCE TO PUMP IN FEET	SUMP DRAINER	MOTOR LEAD
0.5	230	0 TO 360'	SIZE 4	14 GA
0.5	230	360' TO 585'	SIZE 4	12 GA
0.75	230	0 TO 270'	SIZE 4	14 GA
0.75	230	270' TO 430'	SIZE 4	12 GA
1	230	0 TO 225'	SIZE 4	14 GA
1	230	225' TO 360'	SIZE 4	12 GA
1.5	230	0 TO 170'	SIZE 4	14 GA
1.5	230	170' TO 280'	SIZE 4	12 GA
1.5	230	280' TO 430'	SIZE 5	10 GA
1.5	230	430' TO 690'	SIZE 6	8 GA
2	230	0 TO 135'	SIZE 4	14 GA
2	230	135' TO 225'	SIZE 4	12 GA
2	230	225' TO 350'	SIZE 5	10 GA
2	230	350' TO 555'	SIZE 6	8 GA
3	230	0 TO 95'	SIZE 4	14 GA
3	230	95' TO 170'	SIZE 4	12 GA
3	230	170' TO 270'	SIZE 5	10 GA
3	230	270' TO 420'	SIZE 6	8 GA
3	230	420' TO 675'	SIZE 6	6 GA

continued on back

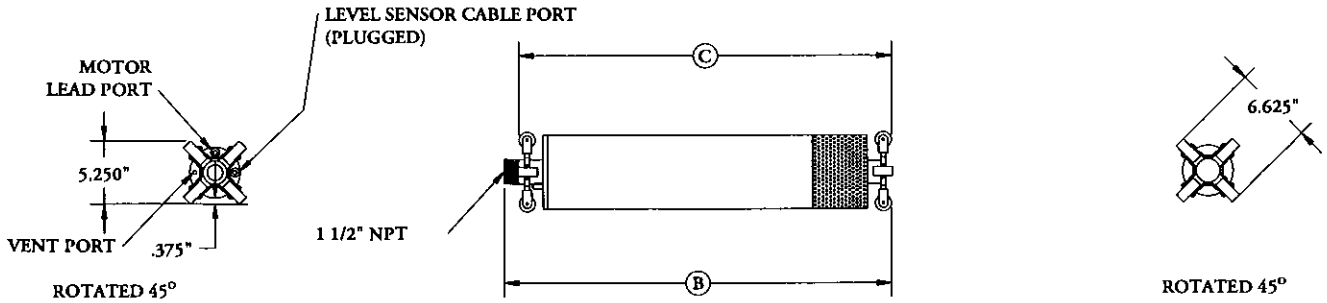
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SERIES 5 SUMP DRAINER SELECTION GUIDE

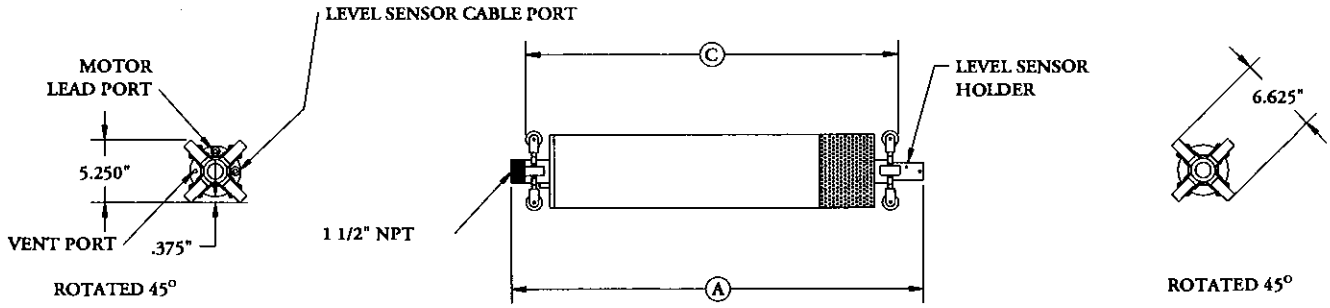
THREE PHASE MOTORS				
HORSEPOWER	VOLTAGE	DISTANCE FROM SERVICE ENTRANCE TO PUMP IN FEET	SUMP DRAINER	MOTOR LEAD
0.5	200	0 TO 640'	SIZE 4	14 GA
0.5	200	640' TO 1025'	SIZE 4	12 GA
0.5	230	0 TO 835'	SIZE 4	14 GA
0.5	230	835' TO 1340'	SIZE 4	12 GA
0.5	460	0 TO 3390'	SIZE 4	14 GA
0.5	460	3390' TO 5420'	SIZE 4	12 GA
0.75	200	0 TO 460'	SIZE 4	14 GA
0.75	200	460' TO 730'	SIZE 4	12 GA
0.75	230	0 TO 600'	SIZE 4	14 GA
0.75	230	600' TO 970'	SIZE 4	12 GA
0.75	460	0 TO 2455'	SIZE 4	14 GA
0.75	460	2455' TO 3915'	SIZE 4	12 GA
1	200	0 TO 385'	SIZE 4	14 GA
1	200	385' TO 620'	SIZE 4	12 GA
1	230	0 TO 505'	SIZE 4	14 GA
1	230	505' TO 820'	SIZE 4	12 GA
1	460	0 TO 2070'	SIZE 4	14 GA
1	460	2070' TO 3300'	SIZE 4	12 GA
1.5	200	0 TO 280'	SIZE 4	14 GA
1.5	200	280' TO 450'	SIZE 4	12 GA
1.5	230	0 TO 380'	SIZE 4	14 GA
1.5	230	380' TO 600'	SIZE 4	12 GA
1.5	460	0 TO 1530'	SIZE 4	14 GA
1.5	460	1530' TO 2440'	SIZE 4	12 GA
2	200	0 TO 215'	SIZE 4	14 GA
2	200	215' TO 350'	SIZE 4	12 GA
2	200	350' TO 550'	SIZE 5	10 GA
2	230	0 TO 290'	SIZE 4	14 GA
2	230	290' TO 460'	SIZE 4	12 GA
2	460	0 TO 1170'	SIZE 4	14 GA
2	460	1170' TO 1860'	SIZE 4	12 GA
3	200	0 TO 160'	SIZE 4	14 GA
3	200	160' TO 260'	SIZE 4	12 GA
3	200	260' TO 420'	SIZE 5	10 GA
3	200	420' TO 665'	SIZE 6	8 GA
3	230	0 TO 215'	SIZE 4	14 GA
3	230	215' TO 350'	SIZE 4	12 GA
3	230	350' TO 560'	SIZE 5	10 GA
3	460	0 TO 900'	SIZE 4	14 GA
3	460	900' TO 1440'	SIZE 4	12 GA

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SERIES 5 SIZE 4 WHEELED SUMP DRAINER



WSD



WSDPT

MODEL	HP	PHASE	A	B	C	*APPROX. SHIPPING WEIGHT	
						WSD	WSDPT
5-2	0.50	1	31.02	29.90	29.15	61.47	66.47
5-2	0.50	3	31.02	29.90	29.15	61.47	66.47
5-3	0.75	1	32.98	31.86	31.11	66.24	71.24
5-3	0.75	3	32.98	31.86	31.11	66.24	71.24
5-4	1.00	1	34.90	33.78	33.03	70.98	75.98
5-4	1.00	3	34.90	33.78	33.03	70.98	75.98
5-5	1.00	1	35.73	34.61	33.86	71.88	76.88
5-5	1.00	3	35.73	34.61	33.86	71.88	76.88
5-6	1.50	1	38.43	37.31	36.56	79.22	84.22
5-6	1.50	3	36.56	35.44	34.69	72.77	77.77
5-7	1.50	1	39.26	38.14	37.39	80.12	85.12
5-7	1.50	3	37.39	36.27	35.52	73.67	78.67
5-8	2.00	1	41.59	40.47	39.72	85.17	90.17
5-8	2.00	3	40.09	38.97	38.22	81.01	86.01

MODEL	HP	PHASE	A	B	C	*APPROX. SHIPPING WEIGHT	
						WSD	WSDPT
5-9	2.00	1	42.42	41.30	40.55	86.07	91.07
5-9	2.00	3	40.92	39.80	39.05	81.90	86.90
5-10	2.00	1	43.25	42.13	41.38	86.96	91.96
5-10	2.00	3	41.75	40.63	39.88	82.80	87.80
5-11	2.00	1	44.08	42.96	42.21	87.86	92.86
5-11	2.00	3	42.58	41.46	40.71	83.69	88.69
5-12	2.00	1	44.91	43.79	43.04	88.75	93.75
5-12	2.00	3	53.41	52.29	51.54	92.35	97.35
5-13	3.00	1	54.24	53.12	52.37	119.24	124.24
5-13	3.00	3	51.24	50.12	49.37	105.91	110.91
5-14	3.00	1	55.07	53.95	53.20	120.14	125.14
5-14	3.00	3	52.07	50.95	50.20	106.81	111.81
5-15	3.00	1	55.90	54.78	54.03	121.03	126.03
5-15	3.00	3	52.90	51.78	51.03	107.70	112.70

NOTE: ALL DIMENSIONS ARE IN INCHES.

*SHIPPING WEIGHT INCLUDES
WSD: CRATE, 50' OF 14-4 MOTOR LEAD, 50' OF 1/8" SS CABLE.
WSDPT: CRATE, 50' OF 14-4 MOTOR LEAD, 50' OF 1/8" SS CABLE,
LEVEL SENSOR AND CABLE.

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Appendix D.7
Final Cover Equivalency

FINAL COVER EQUIVALENCY ANALYSIS

Objective

Evaluate and compare the infiltration through the proposed Kekaha Landfill Phase II cover and the permitted bottom liner to determine the effectiveness of the proposed cover. Infiltration through the cover and bottom liner is estimated using the USEPA Hydraulic Evaluation of Landfill Performance Version 3.07 (HELP) computer modeling program.

Design Criteria and Assumptions

1. This analysis was performed to evaluate the effectiveness of the proposed final cover for the Phase II landfill.

2. The required cover specified in HAR Section 11-58.1-17 requires the same or less infiltration as the bottom liner.

3. The bottom liner will be evaluated with 12 inches of head, which is the maximum head allowed by HAR during the post closure care period.

HELP Model Version 3.07 Input:

1. Solar radiation data can be default data, user input or synthetically generated by the HELP model. Synthetically generated solar radiation data for Honolulu, Hawaii was used to simulate site weather conditions.

2. Temperature, precipitation, and average wind speed data was obtained from site records from January 2001 through December 2010. Missing data is viewed by the model as a "0". Therefore, to avoid skewing the data, missing temperature data for 2001, 2002, and 2008 was filled in using temperatures from days adjacent to the missing data. The data missing in 2004 was filled in using data from the same dates in 2003. Data is missing for a total of 5 days in April of 2001 and 2002. Data from 2004 is missing from April 9 through May 3, and May 26 through June 19. For 2008 data is missing from January 30 through February 3rd. There is no missing data from 2003, 2005, 2006, 2007, 2009, and 2010.

3. Geomembrane liner pinhole density and size were assumed to account for possible manufacturing defects during geomembrane production. The HELP Model User's Guide for Version 3.07 states that the pinhole density for a typical geomembrane is 0.5 to 1 pinhole per acre. One pinhole per acre was assumed to be present.

4. The placement quality for the geomembrane liner was assumed to be "good". A placement quality of "good" according to the HELP Model User's Guide "assumes good field installation with well prepared, smooth soil surface and geomembrane wrinkle control". The HELP Model User's Guide suggests 1 to 4 installation defects for an installation quality of "good". Two installation defects per acre were assumed for the geomembrane liner. This also reflects full time CQA during geomembrane installation.

5. Only the relatively gentle top slopes of the landfill are used in the HELP analysis. The relatively gentle slopes allow the maximum infiltration through the cover.

6. The initial moisture content of the final cover layers was computed by the HELP model software to be steady state conditions.

7. For the cover analysis the curve number was calculated by the HELP model based on the soil type and final cover slopes. Through the use of top deck diversion berms or other engineered features, the shallowest, longest slope for the cover will be 3 percent and 200 feet. The slope and length are based on the flatter top areas of the landfill.

8. The HAR allows a maximum of 1 foot of head on the bottom liner during the post closure period. The head on the bottom liner directly affects the amount of infiltration through the bottom liner. A larger head produces greater infiltration. To determine the performance of the permitted bottom liner with a maximum of 1 foot of head the HELP model analysis of the bottom liner includes the GCL, geomembrane and only 1 foot of the drainage layer. The bottom liner HELP analysis was performed only to determine infiltration through the liner with a maximum of 1 foot of head. Therefore, rainfall and weather data does not impact the results but were input to allow the HELP model software to perform the desired calculations.

9. To maintain a head at a maximum of 1 foot during the bottom liner analysis, only the bottom liner was modeled, no run-off was allowed, and the top sand layer was modeled as a vertical percolation layer. This analysis setup maintained the head on the liner near the HAR allowed depth.

10. The permitted bottom liner system consists of a GCL and 60-mil HDPE geomembrane.

11. A one-acre design area was used for modeling purposes to compute unit quantities.

12. Permitted Bottom Liner Inputs for existing Phase II (see also Table 2):

    - 12 inches of drainage layer
    - Geomembrane
    - GCL
    - No vegetation on cover (does not affect this analysis)
    - Evaporative zone depth = 1 inch (minimum required for HELP Model software requirements) (does not affect this analysis)
    - Maximum leaf area index = 0 (recommended by HELP Model for Bare ground conditions) (does not affect this analysis)
    - Fraction of area allowing runoff = 0 percent
    - Runoff curve number = 84.7 (HELP model calculated, Soil No. 2, 0.51 percent slope-minimum allowed by HELP software, 1-foot slope length) (does not affect this analysis)
    - Length of model run = 10 years (length of run based on length of available data, solar radiation data was synthetically generated for 10 years)

13. Proposed Final Cover Inputs (see also Table 1):

    - 6 inch topsoil/vegetative layer
    - 12 inch vegetative/protective soil layer
    - Geocomposite drainage layer
    - Geomembrane
    - 6 inches soil grading layer (the top 6 inches of intermediate cover)
    - Good vegetation on cover
    - Evaporative zone depth = 22 inches (recommended by HELP Model for fair vegetation conditions for Honolulu, Hawaii) Note that the HELP model uses an actual evaporative zone depth based on the cover soil depths. The maximum evaporative zone depth is

limited to 22 inches.

- Maximum leaf area index = 2 (recommended by HELP Model for fair vegetation conditions)
- Fraction of area allowing runoff = 100 percent
- Runoff curve number = 82.2 (HELP model calculated, Soil No. 9, 3 percent slope, 200-foot slope length)
- Length of model run = 10 years (length of run based on length of available data, solar radiation data was synthetically generated for 10 years)

Help Model General Layout

Table 1: Proposed Final Cover – HELP Model Layout

(Layer Number) Layer Description	Flow	Thickness	Saturated Hydraulic Conductivity	Soil Type/ Texture Number
(1)vertical percolation	↓	6 inches	1.9×10^{-4} cm/sec	Topsoil/Vegetative Layer/#9
(1)vertical percolation	↓	12 inches	4.2×10^{-5} cm/sec	Vegetative/Protective Soil Layer/#12
(12)lateral drainage	→	0.197 inches	$1.0 \times 10^{+1}$ cm/sec	Geocomposite/ #20
(13)40-mil LDPE geomembrane Liner	↓	0.04 inch	4.0×10^{-13} cm/sec	LDPE geomembrane/#36
(14)barrier soil layer	↓	6 inches	1.2×10^{-4} cm/sec	Soil grading layer /#10

Table 2: Bottom Liner – HELP Model Layout

(Layer Number) Layer Description	Flow	Thickness	Saturated Hydraulic Conductivity	Soil Type/ Texture Number
(1)vertical percolation	↓	12 inches	6.0×10^{-3} cm/sec	Drainage Layer/#61
(13)60-mil HDPE geomembrane Liner	↓	0.06 inch	2.0×10^{-13} cm/sec	HDPE geomembrane/#35
(14)Geosynthetic Clay Layer	↓	0.24 inches	3.0×10^{-9} cm/sec	Bentonite mat/#17

Table 3: Soil Texture Properties

Soil Texture	Soil Classification		Comments, Properties, and Uses
	USDA	USCS	
61	FS	SW	Sand drainage layer. Based on HELP model default soil no. 2 with the hydraulic conductivity changed from $5.8x10^{-3}$ cm/sec to $6x10^{-3}$ cm/sec.
36	---	---	HELP model default parameters for Low Density Polyethylene (LDPE) geomembrane.
20	---	---	HELP model default parameters for geocomposite (drainage net, 0.5 cm/ 0.197 inches thick)
18	---	---	HELP model default parameter for municipal solid waste.
17	---	---	HELP model default parameter for a bentonite mat (0.6 cm/0.24 in. thick).
12	SiCL	CL	Used for vegetative/protective cover. HELP model default parameter.
10	SCL	SC	Used for soil grading layer. HELP model default parameter.
9	SiL	ML	Used for topsoil/vegetative cover. HELP model default parameter.

Note: All soil properties are defaults of the HELP Model, Version 3.07 unless otherwise designated.

Calculations

Calculations performed by the HELP Model are included in the output located in Attachments 1 and 2. See Table 4 for the modeling results based on peak average annual values.

Table 4: Calculated Results From HELP Analysis Output

Parameter	Average Annual Value	
	Proposed Final Cover	Permitted Bottom Liner
Total Rainfall	18.94 inches	18.94 inches
Average head: Cover Geomembrane	0.017 inches	---
Infiltration through final cover	6.755 cf/acre	---
Average head: Bottom Liner Geomembrane	---	11.450 inches
Infiltration through bottom liner system	---	8.019 cf/acre

NOTE: Refer to the attached HELP model output files Attachment 1 for the bottom liner, file name "BotLine1.out", and Attachment 2 for the proposed final cover, file name "PropCov2.out".

Conclusion

The evaluation of the proposed cover shows that the proposed cover allows less infiltration than the permitted bottom liner (see Table 4). The proposed final cover which includes a geocomposite, 40-mil geomembrane, and grading layer allows approximately 6.755 cf/acre of surface water infiltration. The permitted bottom liner which includes a 60-mil geomembrane and GCL allows approximately 8.019 cf/acre of infiltration.

References

1. "Engineering Report, Kekaha Landfill Phase II, Kekaha, Kauai, Hawaii," prepared by Harding Lawson Associates, dated August 5, 1993.
2. "Engineering Report, Kekaha Landfill Phase II, Second Vertical Expansion to 85' MSL, Kekaha, Kauai, Hawaii," prepared by Earth Tech, dated November 2004.

Attachment 1

Permitted Bottom Liner HELP Model Output

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**********************************************************************************
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**                                                                              **
**                                                                              **
**               HYDROLOGIC EVALUATION OF LANDFILL PERFORMANCE                  **
**                 HELP MODEL VERSION 3.07  (1 NOVEMBER 1997)                   **
**                   DEVELOPED BY ENVIRONMENTAL LABORATORY                      **
**                     USAE WATERWAYS EXPERIMENT STATION                        **
**              FOR USEPA RISK REDUCTION ENGINEERING LABORATORY                 **
**                                                                              **
**                                                                              **
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PRECIPITATION DATA FILE:    c:\temp\KEK10BOT.D4
TEMPERATURE DATA FILE:      C:\temp\KEK10BOT.D7
SOLAR RADIATION DATA FILE:  c:\temp\KEK10BOT.D13
EVAPOTRANSPIRATION DATA:    c:\temp\KEK10BOT.D11
SOIL AND DESIGN DATA FILE:  c:\temp\BOTLINER.D10
OUTPUT DATA FILE:           c:\temp\BOTLINE1.OUT


TIME:  23:10     DATE:  11/30/2011


**********************************************************************************

     TITLE:  Kekaha Landfill Phase II- Bottom Liner Analysis- 12" Head

**********************************************************************************


     NOTE:  INITIAL MOISTURE CONTENT OF THE LAYERS AND SNOW WATER WERE
              COMPUTED AS NEARLY STEADY-STATE VALUES BY THE PROGRAM.


                                LAYER  1
                                --------

                  TYPE 1 - VERTICAL PERCOLATION LAYER
                      MATERIAL TEXTURE NUMBER  61
          THICKNESS                    =     12.00   INCHES
          POROSITY                     =      0.4370 VOL/VOL
          FIELD CAPACITY               =      0.0620 VOL/VOL
          WILTING POINT                =      0.0240 VOL/VOL
          INITIAL SOIL WATER CONTENT   =      0.4153 VOL/VOL
          EFFECTIVE SAT. HYD. COND.    =  0.600000005000E-02 CM/SEC
```

LAYER 2

TYPE 4 - FLEXIBLE MEMBRANE LINER
MATERIAL TEXTURE NUMBER 35

THICKNESS	=	0.06 INCHES
POROSITY	=	0.0000 VOL/VOL
FIELD CAPACITY	=	0.0000 VOL/VOL
WILTING POINT	=	0.0000 VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.0000 VOL/VOL
EFFECTIVE SAT. HYD. COND.	=	0.199999996000E-12 CM/SEC
FML PINHOLE DENSITY	=	1.00 HOLES/ACRE
FML INSTALLATION DEFECTS	=	2.00 HOLES/ACRE
FML PLACEMENT QUALITY	=	3 - GOOD

LAYER 3

TYPE 3 - BARRIER SOIL LINER
MATERIAL TEXTURE NUMBER 17

THICKNESS	=	0.24 INCHES
POROSITY	=	0.7500 VOL/VOL
FIELD CAPACITY	=	0.7470 VOL/VOL
WILTING POINT	=	0.4000 VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.7500 VOL/VOL
EFFECTIVE SAT. HYD. COND.	=	0.300000003000E-08 CM/SEC

GENERAL DESIGN AND EVAPORATIVE ZONE DATA

NOTE: SCS RUNOFF CURVE NUMBER WAS COMPUTED FROM DEFAULT SOIL DATA BASE USING SOIL TEXTURE # 2 WITH BARE GROUND CONDITIONS, A SURFACE SLOPE OF 1.% AND A SLOPE LENGTH OF 1. FEET.

SCS RUNOFF CURVE NUMBER	=	84.70	
FRACTION OF AREA ALLOWING RUNOFF	=	0.0	PERCENT
AREA PROJECTED ON HORIZONTAL PLANE	=	1.000	ACRES
EVAPORATIVE ZONE DEPTH	=	1.0	INCHES
INITIAL WATER IN EVAPORATIVE ZONE	=	0.176	INCHES
UPPER LIMIT OF EVAPORATIVE STORAGE	=	0.437	INCHES
LOWER LIMIT OF EVAPORATIVE STORAGE	=	0.024	INCHES
INITIAL SNOW WATER	=	0.000	INCHES
INITIAL WATER IN LAYER MATERIALS	=	5.163	INCHES
TOTAL INITIAL WATER	=	5.163	INCHES
TOTAL SUBSURFACE INFLOW	=	0.00	INCHES/YEAR

EVAPOTRANSPIRATION AND WEATHER DATA

NOTE: EVAPOTRANSPIRATION DATA WAS OBTAINED FROM
HONOLULU HAWAII

STATION LATITUDE	=	21.33 DEGREES
MAXIMUM LEAF AREA INDEX	=	0.00
START OF GROWING SEASON (JULIAN DATE)	=	0
END OF GROWING SEASON (JULIAN DATE)	=	367
EVAPORATIVE ZONE DEPTH	=	1.0 INCHES
AVERAGE ANNUAL WIND SPEED	=	3.47 MPH
AVERAGE 1ST QUARTER RELATIVE HUMIDITY	=	72.00 %
AVERAGE 2ND QUARTER RELATIVE HUMIDITY	=	66.00 %
AVERAGE 3RD QUARTER RELATIVE HUMIDITY	=	66.00 %
AVERAGE 4TH QUARTER RELATIVE HUMIDITY	=	70.00 %

NOTE: PRECIPITATION DATA FOR KEKAHA/SITE HAWAII
WAS ENTERED FROM AN ASCII DATA FILE.

NOTE: TEMPERATURE DATA FOR KEKAHA/SITE HAWAII
WAS ENTERED FROM AN ASCII DATA FILE.

NOTE: SOLAR RADIATION DATA WAS SYNTHETICALLY GENERATED USING
COEFFICIENTS FOR HONOLULU HAWAII
AND STATION LATITUDE = 21.33 DEGREES

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ANNUAL TOTALS FOR YEAR 2001

	INCHES	CU. FEET	PERCENT
PRECIPITATION	9.41	34158.316	100.00
RUNOFF	0.000	0.000	0.00
EVAPOTRANSPIRATION	9.397	34110.551	99.86
PERC./LEAKAGE THROUGH LAYER 3	0.002223	8.069	0.02
AVG. HEAD ON TOP OF LAYER 2	11.5016		
CHANGE IN WATER STORAGE	0.011	39.692	0.12
SOIL WATER AT START OF YEAR	5.163	18742.600	
SOIL WATER AT END OF YEAR	5.174	18782.291	
SNOW WATER AT START OF YEAR	0.000	0.000	0.00

SNOW WATER AT END OF YEAR	0.000	0.000	0.00
ANNUAL WATER BUDGET BALANCE	0.0000	0.006	0.00

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ANNUAL TOTALS FOR YEAR 2002

	INCHES	CU. FEET	PERCENT
PRECIPITATION	6.03	21888.908	100.00
RUNOFF	0.000	0.000	0.00
EVAPOTRANSPIRATION	6.191	22474.006	102.67
PERC./LEAKAGE THROUGH LAYER 3	0.002199	7.983	0.04
AVG. HEAD ON TOP OF LAYER 2	11.4234		
CHANGE IN WATER STORAGE	-0.163	-593.073	-2.71
SOIL WATER AT START OF YEAR	5.174	18782.291	
SOIL WATER AT END OF YEAR	5.011	18189.217	
SNOW WATER AT START OF YEAR	0.000	0.000	0.00
SNOW WATER AT END OF YEAR	0.000	0.000	0.00
ANNUAL WATER BUDGET BALANCE	0.0000	-0.009	0.00

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ANNUAL TOTALS FOR YEAR 2003

	INCHES	CU. FEET	PERCENT
PRECIPITATION	13.37	48533.125	100.00
RUNOFF	0.000	0.000	0.00
EVAPOTRANSPIRATION	12.366	44889.953	92.49
PERC./LEAKAGE THROUGH LAYER 3	0.002184	7.929	0.02
AVG. HEAD ON TOP OF LAYER 2	11.3682		

CHANGE IN WATER STORAGE	1.001	3635.228	7.49
SOIL WATER AT START OF YEAR	5.011	18189.217	
SOIL WATER AT END OF YEAR	5.424	19688.801	
SNOW WATER AT START OF YEAR	0.000	0.000	0.00
SNOW WATER AT END OF YEAR	0.588	2135.645	4.40
ANNUAL WATER BUDGET BALANCE	0.0000	0.012	0.00

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ANNUAL TOTALS FOR YEAR 2004

	INCHES	CU. FEET	PERCENT
PRECIPITATION	23.64	85813.227	100.00
RUNOFF	0.000	0.000	0.00
EVAPOTRANSPIRATION	18.271	66324.883	77.29
PERC./LEAKAGE THROUGH LAYER 3	0.002202	7.995	0.01
AVG. HEAD ON TOP OF LAYER 2	11.4074		
CHANGE IN WATER STORAGE	5.366	19480.322	22.70
SOIL WATER AT START OF YEAR	5.424	19688.801	
SOIL WATER AT END OF YEAR	5.424	19688.801	
SNOW WATER AT START OF YEAR	0.588	2135.645	2.49
SNOW WATER AT END OF YEAR	5.955	21615.967	25.19
ANNUAL WATER BUDGET BALANCE	0.0000	0.029	0.00

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ANNUAL TOTALS FOR YEAR 2005

	INCHES	CU. FEET	PERCENT
PRECIPITATION	15.52	56337.633	100.00

RUNOFF	0.000	0.000	0.00
EVAPOTRANSPIRATION	21.886	79444.984	141.02
PERC./LEAKAGE THROUGH LAYER 3	0.002249	8.164	0.01
AVG. HEAD ON TOP OF LAYER 2	11.5948		
CHANGE IN WATER STORAGE	-6.368	-23115.621	-41.03
SOIL WATER AT START OF YEAR	5.424	19688.801	
SOIL WATER AT END OF YEAR	5.011	18189.146	
SNOW WATER AT START OF YEAR	5.955	21615.967	38.37
SNOW WATER AT END OF YEAR	0.000	0.000	0.00
ANNUAL WATER BUDGET BALANCE	0.0000	0.103	0.00

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ANNUAL TOTALS FOR YEAR 2006

	INCHES	CU. FEET	PERCENT
PRECIPITATION	34.76	126178.836	100.00
RUNOFF	0.000	0.000	0.00
EVAPOTRANSPIRATION	34.758	126171.070	99.99
PERC./LEAKAGE THROUGH LAYER 3	0.002230	8.096	0.01
AVG. HEAD ON TOP OF LAYER 2	11.5252		
CHANGE IN WATER STORAGE	0.000	-0.325	0.00
SOIL WATER AT START OF YEAR	5.011	18189.146	
SOIL WATER AT END OF YEAR	5.011	18188.820	
SNOW WATER AT START OF YEAR	0.000	0.000	0.00
SNOW WATER AT END OF YEAR	0.000	0.000	0.00
ANNUAL WATER BUDGET BALANCE	0.0000	-0.003	0.00

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ANNUAL TOTALS FOR YEAR 2007

	INCHES	CU. FEET	PERCENT
PRECIPITATION	23.58	85595.414	100.00
RUNOFF	0.000	0.000	0.00
EVAPOTRANSPIRATION	15.167	55056.668	64.32
PERC./LEAKAGE THROUGH LAYER 3	0.002170	7.879	0.01
AVG. HEAD ON TOP OF LAYER 2	11.3164		
CHANGE IN WATER STORAGE	8.411	30530.820	35.67
SOIL WATER AT START OF YEAR	5.011	18188.820	
SOIL WATER AT END OF YEAR	5.424	19688.801	
SNOW WATER AT START OF YEAR	0.000	0.000	0.00
SNOW WATER AT END OF YEAR	7.997	29030.840	33.92
ANNUAL WATER BUDGET BALANCE	0.0000	0.049	0.00

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ANNUAL TOTALS FOR YEAR 2008

	INCHES	CU. FEET	PERCENT
PRECIPITATION	28.54	103600.219	100.00
RUNOFF	0.000	0.000	0.00
EVAPOTRANSPIRATION	21.823	79217.461	76.46
PERC./LEAKAGE THROUGH LAYER 3	0.002246	8.153	0.01
AVG. HEAD ON TOP OF LAYER 2	11.5604		
CHANGE IN WATER STORAGE	6.715	24374.525	23.53
SOIL WATER AT START OF YEAR	5.424	19688.801	
SOIL WATER AT END OF YEAR	5.424	19688.801	
SNOW WATER AT START OF YEAR	7.997	29030.840	28.02

SNOW WATER AT END OF YEAR	14.712	53405.367	51.55
ANNUAL WATER BUDGET BALANCE	0.0000	0.075	0.00

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ANNUAL TOTALS FOR YEAR 2009

	INCHES	CU. FEET	PERCENT
PRECIPITATION	13.56	49222.816	100.00
RUNOFF	0.000	0.000	0.00
EVAPOTRANSPIRATION	28.275	102639.484	208.52
PERC./LEAKAGE THROUGH LAYER 3	0.002233	8.106	0.02
AVG. HEAD ON TOP OF LAYER 2	11.5383		
CHANGE IN WATER STORAGE	-14.718	-53424.859	-108.54
SOIL WATER AT START OF YEAR	5.424	19688.801	
SOIL WATER AT END OF YEAR	5.419	19669.305	
SNOW WATER AT START OF YEAR	14.712	53405.367	108.50
SNOW WATER AT END OF YEAR	0.000	0.000	0.00
ANNUAL WATER BUDGET BALANCE	0.0000	0.090	0.00

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ANNUAL TOTALS FOR YEAR 2010

	INCHES	CU. FEET	PERCENT
PRECIPITATION	20.98	76157.414	100.00
RUNOFF	0.000	0.000	0.00
EVAPOTRANSPIRATION	12.484	45317.312	59.50
PERC./LEAKAGE THROUGH LAYER 3	0.002153	7.816	0.01
AVG. HEAD ON TOP OF LAYER 2	11.2593		

CHANGE IN WATER STORAGE	8.494	30832.242	40.48
SOIL WATER AT START OF YEAR	5.419	19669.305	
SOIL WATER AT END OF YEAR	5.424	19688.801	
SNOW WATER AT START OF YEAR	0.000	0.000	0.00
SNOW WATER AT END OF YEAR	8.488	30812.748	40.46
ANNUAL WATER BUDGET BALANCE	0.0000	0.041	0.00

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AVERAGE MONTHLY VALUES IN INCHES FOR YEARS 2001 THROUGH 2010

	JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
PRECIPITATION						
TOTALS	2.17	1.55	3.20	0.84	0.45	0.73
	0.28	1.03	0.97	1.08	2.15	4.48
STD. DEVIATIONS	2.80	1.80	6.51	0.68	0.39	0.46
	0.25	1.33	0.63	1.60	2.10	5.94
RUNOFF						
TOTALS	0.000	0.000	0.000	0.000	0.000	0.000
	0.000	0.000	0.000	0.000	0.000	0.000
STD. DEVIATIONS	0.000	0.000	0.000	0.000	0.000	0.000
	0.000	0.000	0.000	0.000	0.000	0.000
EVAPOTRANSPIRATION						
TOTALS	1.988	2.003	2.934	2.451	1.360	1.208
	0.264	1.008	0.996	0.791	1.544	1.516
STD. DEVIATIONS	1.356	1.537	1.702	2.363	2.030	1.869
	0.196	1.206	0.695	0.950	0.781	1.401
PERCOLATION/LEAKAGE THROUGH LAYER 3						
TOTALS	0.0002	0.0002	0.0002	0.0002	0.0002	0.0002
	0.0002	0.0002	0.0002	0.0002	0.0002	0.0002
STD. DEVIATIONS	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000

AVERAGES OF MONTHLY AVERAGED DAILY HEADS (INCHES)

DAILY AVERAGE HEAD ON TOP OF LAYER 2

AVERAGES	11.5862	11.5424	11.6712	11.4411	11.2597	11.3906
	11.2798	11.3418	11.4298	11.4853	11.5576	11.4088
STD. DEVIATIONS	0.3431	0.3721	0.3121	0.4196	0.3200	0.3595
	0.2892	0.3351	0.2780	0.3853	0.2597	0.3964

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AVERAGE ANNUAL TOTALS & (STD. DEVIATIONS) FOR YEARS 2001 THROUGH 2010

	INCHES		CU. FEET	PERCENT
PRECIPITATION	18.94	(8.940)	68748.6	100.00
RUNOFF	0.000	(0.0000)	0.00	0.000
EVAPOTRANSPIRATION	18.062	(8.8180)	65564.64	95.369
PERCOLATION/LEAKAGE THROUGH LAYER 3	0.00221	(0.00003)	8.019	0.01166
AVERAGE HEAD ON TOP OF LAYER 2	11.450	(0.112)		
CHANGE IN WATER STORAGE	0.875	(7.2057)	3175.89	4.620

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PEAK DAILY VALUES FOR YEARS 2001 THROUGH 2010

	(INCHES)	(CU. FT.)
PRECIPITATION	10.69	38804.699
RUNOFF	0.000	0.0000
PERCOLATION/LEAKAGE THROUGH LAYER 3	0.000006	0.02355
AVERAGE HEAD ON TOP OF LAYER 2	12.000	
SNOW WATER	0.00	0.0000

MAXIMUM VEG. SOIL WATER (VOL/VOL)	0.4370
MINIMUM VEG. SOIL WATER (VOL/VOL)	0.0240

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FINAL WATER STORAGE AT END OF YEAR 2010

LAYER	(INCHES)	(VOL/VOL)
1	5.2439	0.4370
2	0.0000	0.0000
3	0.1800	0.7500
SNOW WATER	8.488	

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**              HYDROLOGIC EVALUATION OF LANDFILL PERFORMANCE        **
**                HELP MODEL VERSION 3.07  (1 NOVEMBER 1997)        **
**                  DEVELOPED BY ENVIRONMENTAL LABORATORY           **
**                    USAE WATERWAYS EXPERIMENT STATION             **
**              FOR USEPA RISK REDUCTION ENGINEERING LABORATORY      **
**                                                                  **
**                                                                  **
**********************************************************************
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PRECIPITATION DATA FILE:     c:\temp\KEK10TOP.D4
TEMPERATURE DATA FILE:       C:\temp\KEK10TOP.D7
SOLAR RADIATION DATA FILE:   c:\temp\KEK10TOP.D13
EVAPOTRANSPIRATION DATA:     c:\temp\KEK10TOP.D11
SOIL AND DESIGN DATA FILE:   c:\temp\PROPCOV3.D10
OUTPUT DATA FILE:            c:\temp\PROPCOV3.OUT

TIME:  13:44     DATE:   1/10/2013

**********************************************************************

     TITLE:  Kekaha Landfill Phase II- Lateral Exp. Final Cover Analysis

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     NOTE:  INITIAL MOISTURE CONTENT OF THE LAYERS AND SNOW WATER WERE
              COMPUTED AS NEARLY STEADY-STATE VALUES BY THE PROGRAM.


                                LAYER  1
                                --------

                  TYPE 1 - VERTICAL PERCOLATION LAYER
                      MATERIAL TEXTURE NUMBER   9
        THICKNESS                     =      6.00   INCHES
        POROSITY                      =      0.5010 VOL/VOL
        FIELD CAPACITY                =      0.2840 VOL/VOL
        WILTING POINT                 =      0.1350 VOL/VOL
        INITIAL SOIL WATER CONTENT    =      0.1493 VOL/VOL
        EFFECTIVE SAT. HYD. COND.     =  0.190000006000E-03 CM/SEC
     NOTE:  SATURATED HYDRAULIC CONDUCTIVITY IS MULTIPLIED BY  3.00
              FOR ROOT CHANNELS IN TOP HALF OF EVAPORATIVE ZONE.
```

LAYER 2

TYPE 1 - VERTICAL PERCOLATION LAYER
MATERIAL TEXTURE NUMBER 12

THICKNESS	=	12.00	INCHES
POROSITY	=	0.4710	VOL/VOL
FIELD CAPACITY	=	0.3420	VOL/VOL
WILTING POINT	=	0.2100	VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.2066	VOL/VOL
EFFECTIVE SAT. HYD. COND.	=	0.419999997000E-04	CM/SEC

LAYER 3

TYPE 2 - LATERAL DRAINAGE LAYER
MATERIAL TEXTURE NUMBER 20

THICKNESS	=	0.20	INCHES
POROSITY	=	0.8500	VOL/VOL
FIELD CAPACITY	=	0.0100	VOL/VOL
WILTING POINT	=	0.0050	VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.1968	VOL/VOL
EFFECTIVE SAT. HYD. COND.	=	10.0000000000	CM/SEC
SLOPE	=	3.00	PERCENT
DRAINAGE LENGTH	=	200.0	FEET

LAYER 4

TYPE 4 - FLEXIBLE MEMBRANE LINER
MATERIAL TEXTURE NUMBER 36

THICKNESS	=	0.04	INCHES
POROSITY	=	0.0000	VOL/VOL
FIELD CAPACITY	=	0.0000	VOL/VOL
WILTING POINT	=	0.0000	VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.0000	VOL/VOL
EFFECTIVE SAT. HYD. COND.	=	0.399999993000E-12	CM/SEC
FML PINHOLE DENSITY	=	1.00	HOLES/ACRE
FML INSTALLATION DEFECTS	=	2.00	HOLES/ACRE
FML PLACEMENT QUALITY	=	3 - GOOD	

LAYER 5

TYPE 3 - BARRIER SOIL LINER
MATERIAL TEXTURE NUMBER 52

THICKNESS	=	0.24	INCHES

POROSITY	=	0.7500 VOL/VOL
FIELD CAPACITY	=	0.7470 VOL/VOL
WILTING POINT	=	0.4000 VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.7500 VOL/VOL
EFFECTIVE SAT. HYD. COND.	=	0.499999997000E-08 CM/SEC

GENERAL DESIGN AND EVAPORATIVE ZONE DATA

NOTE: SCS RUNOFF CURVE NUMBER WAS COMPUTED FROM DEFAULT SOIL DATA BASE USING SOIL TEXTURE # 9 WITH A FAIR STAND OF GRASS, A SURFACE SLOPE OF 3.% AND A SLOPE LENGTH OF 200. FEET.

SCS RUNOFF CURVE NUMBER	=	82.20	
FRACTION OF AREA ALLOWING RUNOFF	=	100.0	PERCENT
AREA PROJECTED ON HORIZONTAL PLANE	=	1.000	ACRES
EVAPORATIVE ZONE DEPTH	=	18.2	INCHES
INITIAL WATER IN EVAPORATIVE ZONE	=	3.414	INCHES
UPPER LIMIT OF EVAPORATIVE STORAGE	=	8.825	INCHES
LOWER LIMIT OF EVAPORATIVE STORAGE	=	3.331	INCHES
INITIAL SNOW WATER	=	0.000	INCHES
INITIAL WATER IN LAYER MATERIALS	=	3.594	INCHES
TOTAL INITIAL WATER	=	3.594	INCHES
TOTAL SUBSURFACE INFLOW	=	0.00	INCHES/YEAR

EVAPOTRANSPIRATION AND WEATHER DATA

NOTE: EVAPOTRANSPIRATION DATA WAS OBTAINED FROM HONOLULU HAWAII

STATION LATITUDE	=	21.33	DEGREES
MAXIMUM LEAF AREA INDEX	=	2.00	
START OF GROWING SEASON (JULIAN DATE)	=	0	
END OF GROWING SEASON (JULIAN DATE)	=	367	
EVAPORATIVE ZONE DEPTH	=	18.2	INCHES
AVERAGE ANNUAL WIND SPEED	=	3.47	MPH
AVERAGE 1ST QUARTER RELATIVE HUMIDITY	=	72.00	%
AVERAGE 2ND QUARTER RELATIVE HUMIDITY	=	66.00	%
AVERAGE 3RD QUARTER RELATIVE HUMIDITY	=	66.00	%
AVERAGE 4TH QUARTER RELATIVE HUMIDITY	=	70.00	%

NOTE: PRECIPITATION DATA FOR KEKAHA/SITE HAWAII WAS ENTERED FROM AN ASCII DATA FILE.

NOTE: TEMPERATURE DATA FOR KEKAHA/SITE HAWAII

WAS ENTERED FROM AN ASCII DATA FILE.

NOTE: SOLAR RADIATION DATA WAS SYNTHETICALLY GENERATED USING
COEFFICIENTS FOR HONOLULU HAWAII
AND STATION LATITUDE = 21.33 DEGREES

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ANNUAL TOTALS FOR YEAR 2001

	INCHES	CU. FEET	PERCENT
PRECIPITATION	9.41	34158.316	100.00
RUNOFF	0.365	1325.132	3.88
EVAPOTRANSPIRATION	9.046	32835.191	96.13
DRAINAGE COLLECTED FROM LAYER 3	0.0000	0.000	0.00
PERC./LEAKAGE THROUGH LAYER 5	0.000000	0.000	0.00
AVG. HEAD ON TOP OF LAYER 4	0.0000		
CHANGE IN WATER STORAGE	-0.001	-2.023	-0.01
SOIL WATER AT START OF YEAR	3.594	13045.956	
SOIL WATER AT END OF YEAR	3.593	13043.934	
SNOW WATER AT START OF YEAR	0.000	0.000	0.00
SNOW WATER AT END OF YEAR	0.000	0.000	0.00
ANNUAL WATER BUDGET BALANCE	0.0000	0.015	0.00

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ANNUAL TOTALS FOR YEAR 2002

	INCHES	CU. FEET	PERCENT
PRECIPITATION	6.03	21888.908	100.00
RUNOFF	0.000	0.000	0.00
EVAPOTRANSPIRATION	6.112	22187.967	101.37

DRAINAGE COLLECTED FROM LAYER 3	0.0000	0.000	0.00
PERC./LEAKAGE THROUGH LAYER 5	0.000000	0.000	0.00
AVG. HEAD ON TOP OF LAYER 4	0.0000		
CHANGE IN WATER STORAGE	-0.082	-299.058	-1.37
SOIL WATER AT START OF YEAR	3.593	13043.934	
SOIL WATER AT END OF YEAR	3.511	12744.876	
SNOW WATER AT START OF YEAR	0.000	0.000	0.00
SNOW WATER AT END OF YEAR	0.000	0.000	0.00
ANNUAL WATER BUDGET BALANCE	0.0000	-0.002	0.00

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ANNUAL TOTALS FOR YEAR 2003

	INCHES	CU. FEET	PERCENT
PRECIPITATION	13.37	48533.125	100.00
RUNOFF	0.004	13.176	0.03
EVAPOTRANSPIRATION	12.464	45244.805	93.22
DRAINAGE COLLECTED FROM LAYER 3	0.0000	0.000	0.00
PERC./LEAKAGE THROUGH LAYER 5	0.000000	0.000	0.00
AVG. HEAD ON TOP OF LAYER 4	0.0000		
CHANGE IN WATER STORAGE	0.902	3275.111	6.75
SOIL WATER AT START OF YEAR	3.511	12744.876	
SOIL WATER AT END OF YEAR	4.413	16019.986	
SNOW WATER AT START OF YEAR	0.000	0.000	0.00
SNOW WATER AT END OF YEAR	0.000	0.000	0.00
ANNUAL WATER BUDGET BALANCE	0.0000	0.034	0.00

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ANNUAL TOTALS FOR YEAR 2004

	INCHES	CU. FEET	PERCENT
PRECIPITATION	23.64	85813.227	100.00
RUNOFF	1.699	6166.105	7.19
EVAPOTRANSPIRATION	16.340	59313.301	69.12
DRAINAGE COLLECTED FROM LAYER 3	2.3715	8608.397	10.03
PERC./LEAKAGE THROUGH LAYER 5	0.000003	0.009	0.00
AVG. HEAD ON TOP OF LAYER 4	0.0117		
CHANGE IN WATER STORAGE	3.230	11725.398	13.66
SOIL WATER AT START OF YEAR	4.413	16019.986	
SOIL WATER AT END OF YEAR	7.643	27745.385	
SNOW WATER AT START OF YEAR	0.000	0.000	0.00
SNOW WATER AT END OF YEAR	0.000	0.000	0.00
ANNUAL WATER BUDGET BALANCE	0.0000	0.015	0.00

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ANNUAL TOTALS FOR YEAR 2005

	INCHES	CU. FEET	PERCENT
PRECIPITATION	15.52	56337.633	100.00
RUNOFF	2.583	9376.111	16.64
EVAPOTRANSPIRATION	10.433	37871.871	67.22
DRAINAGE COLLECTED FROM LAYER 3	6.6364	24090.125	42.76
PERC./LEAKAGE THROUGH LAYER 5	0.000007	0.025	0.00
AVG. HEAD ON TOP OF LAYER 4	0.0327		
CHANGE IN WATER STORAGE	-4.132	-15000.510	-26.63
SOIL WATER AT START OF YEAR	7.643	27745.385	
SOIL WATER AT END OF YEAR	3.511	12744.876	

SNOW WATER AT START OF YEAR	0.000	0.000	0.00
SNOW WATER AT END OF YEAR	0.000	0.000	0.00
ANNUAL WATER BUDGET BALANCE	0.0000	0.007	0.00

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ANNUAL TOTALS FOR YEAR 2006

	INCHES	CU. FEET	PERCENT
PRECIPITATION	34.76	126178.836	100.00
RUNOFF	4.935	17915.826	14.20
EVAPOTRANSPIRATION	19.167	69576.062	55.14
DRAINAGE COLLECTED FROM LAYER 3	10.6575	38686.891	30.66
PERC./LEAKAGE THROUGH LAYER 5	0.000011	0.041	0.00
AVG. HEAD ON TOP OF LAYER 4	0.0520		
CHANGE IN WATER STORAGE	0.000	0.000	0.00
SOIL WATER AT START OF YEAR	3.511	12744.876	
SOIL WATER AT END OF YEAR	3.511	12744.876	
SNOW WATER AT START OF YEAR	0.000	0.000	0.00
SNOW WATER AT END OF YEAR	0.000	0.000	0.00
ANNUAL WATER BUDGET BALANCE	0.0000	0.013	0.00

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ANNUAL TOTALS FOR YEAR 2007

	INCHES	CU. FEET	PERCENT
PRECIPITATION	23.58	85595.414	100.00
RUNOFF	2.830	10273.613	12.00
EVAPOTRANSPIRATION	14.101	51187.535	59.80

DRAINAGE COLLECTED FROM LAYER 3	5.8811	21348.545	24.94
PERC./LEAKAGE THROUGH LAYER 5	0.000006	0.023	0.00
AVG. HEAD ON TOP OF LAYER 4	0.0288		
CHANGE IN WATER STORAGE	0.767	2785.679	3.25
SOIL WATER AT START OF YEAR	3.511	12744.876	
SOIL WATER AT END OF YEAR	4.278	15530.556	
SNOW WATER AT START OF YEAR	0.000	0.000	0.00
SNOW WATER AT END OF YEAR	0.000	0.000	0.00
ANNUAL WATER BUDGET BALANCE	0.0000	0.017	0.00

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ANNUAL TOTALS FOR YEAR 2008

	INCHES	CU. FEET	PERCENT
PRECIPITATION	28.54	103600.219	100.00
RUNOFF	9.007	32694.285	31.56
EVAPOTRANSPIRATION	13.622	49446.723	47.73
DRAINAGE COLLECTED FROM LAYER 3	4.8139	17474.564	16.87
PERC./LEAKAGE THROUGH LAYER 5	0.000005	0.019	0.00
AVG. HEAD ON TOP OF LAYER 4	0.0234		
CHANGE IN WATER STORAGE	1.098	3984.601	3.85
SOIL WATER AT START OF YEAR	4.278	15530.556	
SOIL WATER AT END OF YEAR	5.376	19515.156	
SNOW WATER AT START OF YEAR	0.000	0.000	0.00
SNOW WATER AT END OF YEAR	0.000	0.000	0.00
ANNUAL WATER BUDGET BALANCE	0.0000	0.024	0.00

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ANNUAL TOTALS FOR YEAR 2009

	INCHES	CU. FEET	PERCENT
PRECIPITATION	13.56	49222.816	100.00
RUNOFF	0.588	2136.029	4.34
EVAPOTRANSPIRATION	14.495	52617.121	106.90
DRAINAGE COLLECTED FROM LAYER 3	0.0126	45.607	0.09
PERC./LEAKAGE THROUGH LAYER 5	0.000000	0.000	0.00
AVG. HEAD ON TOP OF LAYER 4	0.0001		
CHANGE IN WATER STORAGE	-1.536	-5575.955	-11.33
SOIL WATER AT START OF YEAR	5.376	19515.156	
SOIL WATER AT END OF YEAR	3.840	13939.201	
SNOW WATER AT START OF YEAR	0.000	0.000	0.00
SNOW WATER AT END OF YEAR	0.000	0.000	0.00
ANNUAL WATER BUDGET BALANCE	0.0000	0.014	0.00

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ANNUAL TOTALS FOR YEAR 2010

	INCHES	CU. FEET	PERCENT
PRECIPITATION	20.98	76157.414	100.00
RUNOFF	3.066	11128.597	14.61
EVAPOTRANSPIRATION	11.540	41890.117	55.00
DRAINAGE COLLECTED FROM LAYER 3	3.9776	14438.622	18.96
PERC./LEAKAGE THROUGH LAYER 5	0.000004	0.015	0.00
AVG. HEAD ON TOP OF LAYER 4	0.0194		
CHANGE IN WATER STORAGE	2.397	8700.039	11.42
SOIL WATER AT START OF YEAR	3.840	13939.201	

SOIL WATER AT END OF YEAR	6.237	22639.240	
SNOW WATER AT START OF YEAR	0.000	0.000	0.00
SNOW WATER AT END OF YEAR	0.000	0.000	0.00
ANNUAL WATER BUDGET BALANCE	0.0000	0.023	0.00

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AVERAGE MONTHLY VALUES IN INCHES FOR YEARS 2001 THROUGH 2010

	JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
PRECIPITATION						
TOTALS	2.17	1.55	3.20	0.84	0.45	0.73
	0.28	1.03	0.97	1.08	2.15	4.48
STD. DEVIATIONS	2.80	1.80	6.51	0.68	0.39	0.46
	0.25	1.33	0.63	1.60	2.10	5.94
RUNOFF						
TOTALS	0.258	0.126	0.469	0.001	0.000	0.000
	0.000	0.059	0.000	0.043	0.169	1.383
STD. DEVIATIONS	0.817	0.293	1.419	0.001	0.000	0.000
	0.000	0.185	0.000	0.136	0.477	2.802
EVAPOTRANSPIRATION						
TOTALS	1.344	1.575	1.761	1.239	0.437	0.728
	0.296	0.930	1.015	0.807	1.296	1.306
STD. DEVIATIONS	0.867	0.789	1.405	1.084	0.403	0.422
	0.233	1.139	0.610	0.999	0.633	1.100
LATERAL DRAINAGE COLLECTED FROM LAYER 3						
TOTALS	0.5937	0.1169	1.0072	0.0725	0.0000	0.0000
	0.0000	0.0000	0.0000	0.0428	0.0768	1.5251
STD. DEVIATIONS	1.8731	0.2537	2.9888	0.2294	0.0000	0.0000
	0.0000	0.0000	0.0000	0.1352	0.2429	2.1790
PERCOLATION/LEAKAGE THROUGH LAYER 5						
TOTALS	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000

STD. DEVIATIONS	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000

AVERAGES OF MONTHLY AVERAGED DAILY HEADS (INCHES)

DAILY AVERAGE HEAD ON TOP OF LAYER 4

AVERAGES	0.0347	0.0075	0.0589	0.0044	0.0000	0.0000
	0.0000	0.0000	0.0000	0.0025	0.0046	0.0891
STD. DEVIATIONS	0.1095	0.0163	0.1747	0.0139	0.0000	0.0000
	0.0000	0.0000	0.0000	0.0079	0.0147	0.1274

AVERAGE ANNUAL TOTALS & (STD. DEVIATIONS) FOR YEARS 2001 THROUGH 2010

	INCHES	CU. FEET	PERCENT
PRECIPITATION	18.94 (8.940)	68748.6	100.00
RUNOFF	2.508 (2.7897)	9102.89	13.241
EVAPOTRANSPIRATION	12.732 (3.7176)	46217.07	67.226
LATERAL DRAINAGE COLLECTED FROM LAYER 3	3.43506 (3.63372)	12469.274	18.13750
PERCOLATION/LEAKAGE THROUGH LAYER 5	0.00000 (0.00000)	0.013	0.00002
AVERAGE HEAD ON TOP OF LAYER 4	0.017 (0.018)		
CHANGE IN WATER STORAGE	0.264 (2.0421)	959.33	1.395

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PEAK DAILY VALUES FOR YEARS 2001 THROUGH 2010

	(INCHES)	(CU. FT.)
PRECIPITATION	10.69	38804.699
RUNOFF	8.091	29369.3809
DRAINAGE COLLECTED FROM LAYER 3	1.23090	4468.16846
PERCOLATION/LEAKAGE THROUGH LAYER 5	0.000001	0.00499
AVERAGE HEAD ON TOP OF LAYER 4	2.230	
MAXIMUM HEAD ON TOP OF LAYER 4	3.903	
LOCATION OF MAXIMUM HEAD IN LAYER 3 (DISTANCE FROM DRAIN)	24.9 FEET	
SNOW WATER	0.00	0.0000

MAXIMUM VEG. SOIL WATER (VOL/VOL)	0.4533
MINIMUM VEG. SOIL WATER (VOL/VOL)	0.1831

*** Maximum heads are computed using McEnroe's equations. ***

Reference: Maximum Saturated Depth over Landfill Liner
by Bruce M. McEnroe, University of Kansas
ASCE Journal of Environmental Engineering
Vol. 119, No. 2, March 1993, pp. 262-270.

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FINAL WATER STORAGE AT END OF YEAR 2010

LAYER	(INCHES)	(VOL/VOL)
1	1.5793	0.2632
2	4.4124	0.3677
3	0.0650	0.3299
4	0.0000	0.0000
5	0.1800	0.7500
SNOW WATER	0.000	

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Appendix E
Gas Collection System Calculations

Appendix E
Gas Collection System Calculations

Appendix E.1
Gas Generation Analysis

Appendix E.2
Gas Collection Pipe Flow Capacity Analysis

Appendix E.3
Zone of Influence Calculations

Appendix E.4
Control Device Sizing

Appendix E.5
Gas System Design Drawings

Appendix E.1
Gas Generation Analysis


LandGEM - Version 3.02
US EPA Office of Research and Development
CLEAN AIR TECHNOLOGY CENTER
EPA
LandGEM
Landfill Gas Emissions Model
Version 3.02
U.S. Environmental Protection Agency
Office of Research and Development
National Risk Management Research Laboratory (NRMRL)
and
Clean Air Technology Center (CATC)
Research Triangle Park, North Carolina
May 2005

Input Review

LANDFILL CHARACTERISTICS

Landfill Open Year	**Before 1985**	
Waste Design Capacity	**3,626,222**	*short tons*
MODEL PARAMETERS		
Methane Generation Rate, k	**0.020**	*year^{-1}*
Potential Methane Generation Capacity, L_o	**100**	*m^3/Mg*
Methane Content	**50**	*% by volume*

GASES / POLLUTANTS SELECTED

Gas / Pollutant #1: **Total landfill gas**
Gas / Pollutant #2: **Methane**
Gas / Pollutant #3: **Carbon dioxide**

Waste Acceptance Rates

	Waste Accepted		Waste-In-Place	
Year	(Mg/year)	(short tons/year)	(Mg)	(short tons)
1985	35,533	39,086	0	0
1986	72,959	80,255	35,533	39,086
1987	72,959	80,255	108,492	119,341
1988	72,959	80,255	181,451	199,596
1989	72,959	80,255	254,410	279,851
1990	72,959	80,255	327,369	360,106
1991	72,959	80,255	400,328	440,361
1992	72,959	80,255	473,287	520,616
1993	100,492	110,541	546,246	600,871
1994	78,035	85,838	646,738	711,412
1995	114,275	125,703	724,773	797,250
1996	196,973	216,670	839,048	922,953
1997	84,825	93,307	1,036,021	1,139,623
1998	58,409	64,250	1,120,845	1,232,930
1999	61,445	67,590	1,179,255	1,297,180
2000	66,207	72,828	1,240,700	1,364,770
2001	70,147	77,162	1,306,907	1,437,598
2002	67,908	74,699	1,377,055	1,514,760
2003	73,693	81,062	1,444,962	1,589,459
2004	78,605	86,465	1,518,656	1,670,521
2005	81,054	89,160	1,597,260	1,756,986
2006	82,043	90,247	1,678,314	1,846,146
2007	81,922	90,114	1,760,357	1,936,393
2008	79,307	87,237	1,842,279	2,026,507
2009	71,343	78,477	1,921,586	2,113,744
2010	63,404	69,744	1,992,928	2,192,221
2011	64,543	70,997	2,056,332	2,261,965
2012	66,279	72,907	2,120,875	2,332,962
2013	67,754	74,529	2,187,153	2,405,869
2014	72,568	79,824	2,254,907	2,480,398
2015	74,545	82,000	2,327,475	2,560,222
2016	74,545	82,000	2,402,020	2,642,222
2017	74,545	82,000	2,476,566	2,724,222
2018	74,545	82,000	2,551,111	2,806,222
2019	74,545	82,000	2,625,657	2,888,222
2020	74,545	82,000	2,700,202	2,970,222
2021	74,545	82,000	2,774,748	3,052,222
2022	74,545	82,000	2,849,293	3,134,222
2023	74,545	82,000	2,923,839	3,216,222
2024	74,545	82,000	2,998,384	3,298,222
2025	74,545	82,000	3,072,929	3,380,222
2026	74,545	82,000	3,147,475	3,462,222
2027	74,545	82,000	3,222,020	3,544,222
2028	0	0	3,296,566	3,626,222

Results

Year	Total landfill gas		
	(Mg/year)	(m^3/year)	(Av ft^3/min)
1985	0	0	0
1986	1.759E+02	1.409E+05	9.464E+00
1987	5.336E+02	4.273E+05	2.871E+01
1988	8.842E+02	7.081E+05	4.757E+01
1989	1.228E+03	9.833E+05	6.607E+01
1990	1.565E+03	1.253E+06	8.419E+01
1991	1.895E+03	1.517E+06	1.020E+02
1992	2.219E+03	1.777E+06	1.194E+02
1993	2.536E+03	2.031E+06	1.364E+02
1994	2.983E+03	2.389E+06	1.605E+02
1995	3.310E+03	2.651E+06	1.781E+02
1996	3.811E+03	3.051E+06	2.050E+02
1997	4.710E+03	3.772E+06	2.534E+02
1998	5.037E+03	4.033E+06	2.710E+02
1999	5.226E+03	4.185E+06	2.812E+02
2000	5.427E+03	4.346E+06	2.920E+02
2001	5.647E+03	4.522E+06	3.038E+02
2002	5.883E+03	4.711E+06	3.165E+02
2003	6.103E+03	4.887E+06	3.283E+02
2004	6.347E+03	5.082E+06	3.415E+02
2005	6.610E+03	5.293E+06	3.556E+02
2006	6.880E+03	5.510E+06	3.702E+02
2007	7.150E+03	5.726E+06	3.847E+02
2008	7.414E+03	5.937E+06	3.989E+02
2009	7.660E+03	6.134E+06	4.121E+02
2010	7.862E+03	6.295E+06	4.230E+02
2011	8.020E+03	6.422E+06	4.315E+02
2012	8.181E+03	6.551E+06	4.401E+02
2013	8.347E+03	6.684E+06	4.491E+02
2014	8.517E+03	6.820E+06	4.582E+02
2015	8.707E+03	6.973E+06	4.685E+02
2016	8.904E+03	7.130E+06	4.791E+02
2017	9.097E+03	7.284E+06	4.894E+02
2018	9.286E+03	7.436E+06	4.996E+02
2019	9.471E+03	7.584E+06	5.096E+02
2020	9.652E+03	7.729E+06	5.193E+02
2021	9.830E+03	7.872E+06	5.289E+02
2022	1.000E+04	8.011E+06	5.383E+02
2023	1.018E+04	8.148E+06	5.475E+02
2024	1.034E+04	8.282E+06	5.565E+02
2025	1.051E+04	8.414E+06	5.653E+02
2026	1.067E+04	8.543E+06	5.740E+02
2027	1.083E+04	8.669E+06	5.825E+02

Year	Total landfill gas		
	(Mg/year)	(m^3/year)	(Av ft^3/min)
2028	1.098E+04	8.793E+06	5.908E+02
2029	1.076E+04	8.619E+06	5.791E+02
2030	1.055E+04	8.448E+06	5.676E+02
2031	1.034E+04	8.281E+06	5.564E+02
2032	1.014E+04	8.117E+06	5.454E+02
2033	9.936E+03	7.956E+06	5.346E+02
2034	9.739E+03	7.799E+06	5.240E+02
2035	9.546E+03	7.644E+06	5.136E+02
2036	9.357E+03	7.493E+06	5.034E+02
2037	9.172E+03	7.345E+06	4.935E+02
2038	8.990E+03	7.199E+06	4.837E+02
2039	8.812E+03	7.057E+06	4.741E+02
2040	8.638E+03	6.917E+06	4.647E+02
2041	8.467E+03	6.780E+06	4.555E+02
2042	8.299E+03	6.646E+06	4.465E+02
2043	8.135E+03	6.514E+06	4.377E+02
2044	7.974E+03	6.385E+06	4.290E+02
2045	7.816E+03	6.259E+06	4.205E+02
2046	7.661E+03	6.135E+06	4.122E+02
2047	7.509E+03	6.013E+06	4.040E+02
2048	7.361E+03	5.894E+06	3.960E+02
2049	7.215E+03	5.777E+06	3.882E+02
2050	7.072E+03	5.663E+06	3.805E+02

Appendix E.2
Gas Collection Pipe Flow Capacity Analysis

Header Pipe Sizing

The proposed GCCS will utilize a header piping system. The system piping was evaluated to establish the pipe sizing that will appropriately convey the maximum expected gas flow [40 CFR §60.752(b)(2)(ii)(A)(1)] which would correspond to the maximum capacity of the flare and blower system (590 scfm – Appendix E.1-2: Year 2028).

Header Slope

The header line is typically sloped at a 3% grade when installed within refuse to provide for gravity drainage of liquid gas condensate, and to minimize blockages resulting from differential settlement of the landfill. Header pipe placed outside of refuse limits will be sloped a minimum of 1%.

Header Pipe Sizing

Three design criteria have been established for calculating the minimum acceptable size for the header pipe diameter:

1. The velocity of gas should not exceed 30 feet per second within the pipe.
2. The velocity of the gas should not exceed 20 feet per second within the pipe when there is countercurrent gas/liquid flow.
3. The maximum allowable pressure drop within the pipe should not be greater than one inch water column per 100 feet of header, or 0.01 inch water column per unit foot of header pipe.

Flow conditions within any segment of header line should not consistently exceed either the pressure loss or the velocity limitations. Under sizing the header pipe can cause excessive pressure losses throughout the system, which reduces gas collection efficiency.

The KYGas® model was used to evaluate system pressure drop and this model was development by the University of Kentucky for performing water and gas distribution flow analyses. The program uses a 2-dimensional model depicting the geometry of the piping system. Once the 2-dimension layout of the system has been entered into the model, the user enters the physical properties of the gas, plus other site-specific parameters for the size and type of pipe, gas flow requirements, and operating pressure conditions to calculate the system gas velocities and pressure distribution.

KYGas® utilizes the Ideal Gas Law for pressure-temperature-density relationships and the Darcy-Weisbach equation for head losses related to uncompressible flow. The program operates under the assumption that all flow in the piping system is steady, one-dimensional, isothermal flow for an ideal gas.

Header Pipe Sizing Calculations

Seg.	Est. Flow Rate (scfm)	Pipe I.D. (in)	Loss per unit length (WC/FT)	Total System Press. Loss (WC)
Perimeter Header	590	11.25	0.0003	0.91

Seg.	Velocity (FT/SEC)	Actual Flow Rate (ft^3/hr)	Spec. Grav.
Perimeter Header	18.34	694	0.98

The blower connected to the perimeter header system will generate a minimum of 40 inches water column vacuum (“wc) which leaves 39 inches available at each well; more than sufficient to extract gas at a sufficient rate in accordance with the NSPS. A valve at each wellhead will allow for regulation of the available vacuum to minimize air intrusion.

Header Construction

The header pipe for the GCCS will be high density polyethylene (HDPE) pipe. HDPE pipe is ideal due to its compatibility with landfill gas and waste, its flexibility (if settlement occurs), its long term stability and its excellent chemical resistance. For buried header, the pipe is set in a trench, and is surrounded by compatible bedding media.

NSPS Compliance

The header system as described in this section will meet the following requirements listed in 40 CFR §60.759:

- Gas system expandability & accessibility – Blind flanges have been incorporated into the system in order to allow for future gas system expansions. In addition HDPE pipe can be cut and new components fused in place to expand the system easily.
- Corrosion resistance – HDPE and PVC are resistant to corrosion from landfill gas
- Fill settlement – The flexibility of the material and the modulus of elasticity of both HDPE and PVC are sufficient to prevent collapse due to settlement
- Ability to withstand planned overburden or traffic loads – The flexibility of the material and the modulus of elasticity of both HDPE and PVC are sufficient to prevent collapse due to settlement under anticipated operating conditions.

Control valves will be located on the collection header piping. The valves can be used to manually shut off the applied vacuum to a particular section of header pipe. This allows portions of the system to be isolated for monitoring and maintenance purposes.

Appendix E.3
Zone of Influence Calculations

Discussion of the Darcy Radius of Influence for Landfill Gas Extraction Systems

Purpose: To present a design procedure for determination of gas extraction well locations and relative placement/spacing.

Method: Utilization of an individual gas extraction well's Darcy zone of influence to determine optimal well spacing sufficient to induce vacuum uniformly throughout the waste disposal area.

Objective: As a standard design method, landfill gas extraction well spacing by means of the Darcy zone of influence provides a reasonable effective extraction area coverage over the waste disposal area, with minimum overlap or open spaces. Placement of gas extraction wells on side slopes should be minimized to reduce air intrusion.

Definition: The zone of influence (ZOI) is the radial distance from an extraction well from which the migration direction of landfill gas will be influenced by an application of vacuum. Since gas is influenced by convection forces (pressure gradient), the zone of influence is established where the measured pressure/vacuum at extreme zone (r_1) of influence is zero.

Darcy Zone of Influence for Radial Compressible Fluid Flow

Discussion: Darcy equation, for radial fluid flow

$$v = \left(\frac{g_C \, K}{\mu} \right) \left(\frac{dP}{dr} \right) \quad equation(1)$$

Where:

g_c	=	acceleration of gravity constant = 32.2 ($lb_M \cdot ft/lb_F \cdot sec^2$)
v	=	apparent flow velocity in (ft/sec) units
μ	=	absolute viscosity of the flowing fluid (landfill gas) in ($lb_M/ft \cdot sec$) units
K	=	absolute permeability of the porous media (refuse) in (ft^2) units
dP	=	pressure gradient in the direction of radial flow in (lb_F/ft^2) units
dr	=	radial distance gradient in (ft) units

Definition: Permeability is defined as a measure of a porous media's ability to transmit fluids.

Assumptions necessary to develop the basic flow equations:

(1) steady-state flow conditions exist.

(2) the pore space of the refuse is 100 percent saturated with the flowing fluid (landfill gas).

(3) the viscosity of the flowing fluid is constant.

(4) isothermal conditions in the refuse prevail.

(5) flow is laminar, horizontal, and linear since refuse grain size is relatively small and the velocity of the fluid flow is low.

Please refer to the ideal radial flow system diagram (Figure 1). With these assumptions in mind, let

$$v = \frac{q}{A}$$

Where:

v	=	the apparent velocity of the flowing fluid (gas)
q	=	volumetric rate of fluid (gas) flow
A	=	total cross-sectional area perpendicular to flow direction
	=	$2\pi r h_s$
h_s	=	total extraction well length of slotted pipe

Substitute in equation (1):

$$q / A = \left(\frac{g_C K}{\mu} \right) \left(\frac{dP}{dr} \right) \quad equation(2)$$

with $A = 2\pi r h_s$ and rearranging

$$q = \left(\frac{2\pi r h_S g_C K}{\mu} \right) \left(\frac{dP}{dr} \right) \quad equation(3)$$

Since landfill gas is a compressible fluid, its viscosity and flow characteristics must be corrected to standard conditions.

When a flowing fluid is compressible, then q is not constant, but is a function of pressure and temperature $f(P,T)$. An expression for the standard flow rate of a gas (q_s) is obtained from Charles' law, assuming ideal gas behavior at standard conditions:

$$\frac{P_1 q_1}{T_1} = \frac{P_2 q_2}{T_2} = constant = \frac{P_S q_S}{T_S} \quad _ \quad at\ standard\ conditions$$

Substitution in equation (3):

$$\frac{P_S q_S}{T_S} = \frac{Pq}{T} = \left(\frac{2\pi r h_S g_C K}{\mu T} \right) \left(\frac{PdP}{dr} \right) = constant$$

Where:

T_s	=	standard temperature = 60(degF) = 520(degR) constant
P_s	=	standard pressure = 14.7 (psia) = 2,116.8 (lb_F/ft^2) constant
T	=	flowing temperature of the fluid (landfill gas)

Therefore:

$$\frac{P_S\, q_S}{T_S} = \left(\frac{2\pi\, r\, h_S\, g_C\, K}{\mu\, T} \right) \left(\frac{P dP}{dr} \right) \quad equation(4)$$

let q_s = standard volumetric rate of fluid flow

$$q_S = (dG / dt)\, V\rho = (dG / dt)\, \pi r^2\, h_T\, \rho$$

Where:

(dG/dt)	=	landfill gas generation rate
V	=	volume of well influence, assuming uniform cylindrical geometry
	=	$\pi r^2 h_T$
ρ	=	density of refuse; assume ρ = 1,300 (lb_M/yd^3) = 48.15 (lb_M/ft^3)
h_T	=	total extraction well length (total well depth)

This approach assumes that all conditions are uniform, and that all gas generated at radius r_1 is extracted. Actually, only a fraction of the gas generated at some distance "r" from the well would be extracted, and this fraction would decrease as the radius increases.

Please refer to the ideal radial flow system diagram (Figure 1).

$$\frac{P_S\ (\ dG/dt\ )\ \pi r^2\ h_T\ \rho}{T_S} = \left(\ \frac{2\pi\, r h_S\ g_C K}{\mu T}\ \right)\left(\ \frac{PdP}{dr}\ \right) \quad equation(\ 5\ )$$

Substitution in equation (4):

Simplification, separation of variables, and insertion of system limits in equation (5):

$$\int_{r_0}^{r_1} rdr = \frac{2\, g_C\ KT_S(\ h_S\ /\ h_T\ )}{P_S(\ dG\ /\ dT\ )\ \rho\mu T} \int_{P_0}^{P_1} PdP$$

Where:

r_o = radius of the extraction well pipe

r_1 = the Darcy zone of influence

Which when integrated:

$$\frac{(\ r_1^2 - r_0^2\ )}{2} = \left[\ \frac{g_C\ K\, T_S(\ h_S / h_T\ )}{P_S(\ dG\ /\ dt\ )\rho\mu T}\ x\ (\ P_1^2 - P_0^2\ )\ \right]$$

$$r_1 = \left[\ \frac{2\, g_C\ K\, T_S(\ h_S\ /\ h_T\ )}{P_S(\ dG\ /\ dt\ )\rho\mu T}\ (\ P_1^2 - P_0^2\ ) + r_0^2\ \right]^{1/2} \quad equation(\ 6\ )$$

Solving for zone of influence *(r_1)*:

This is the Darcy zone of influence equation.

Since a concentric cylindrical surface at distances r_1 and r_0 are assumed, perpendicular gas flow across the surface at r_1 must be much greater than that across the surface at r_0 and since $r_0 <<< r_1$, then r_0 is negligible and:

$$r_1 = \left[\frac{2 g_C K T_S (h_S / h_T)}{P_S (dG / dt) \rho \mu T} (P_1^2 - P_0^2) \right]^{1/2} \qquad equation (7)$$

The maximum vacuum that can be applied in a gas extraction well is usually dependent on the length of solid pipe section specified. The relationship is that as the length of solid pipe section increases, the potential of air intrusion through the cover or side slopes decreases, therefore allowing more vacuum to be applied to the gas extraction well to maximize its effective zone of well influence. The average reasonable applied vacuum at the wellhead *(P_o)* for an active gas extraction system must be anticipated by the designer to calculate the Darcy zone of influence.

The following table is a guideline of reasonable applied vacuum values to be utilized in equation (7):

Length of Solid Pipe (ft)	Applied Vacuum (in. W.C.)	Applied Vacuum (lb/ft) absolute
15	0.25	2,115.5
20	0.75	2,112.9
25	2.0	2,106.4
30	2.5	2,103.8
35	3.0	2,101.2
40	3.5	2,098.61

Note:

ft	foot/feet
in. w.c.	inches water column pressure
lb/ft	pounds per foot

The following calculation demonstrates how the Darcy zone of influence can be determined for a conceptual gas extraction well location plan.

Assumptions:

Landfill gas composition based on EPA model:

percent methane (CH_4) = 50 %

percent carbon dioxide (CO_2) = 50 %

Total = 100 %

Average flowing landfill gas temperature *(T)* = 100(deg F) = 560(deg R)

Gas generation rate *(dG/dt)* = $k*L_o * 2 = 0.02*100*2 = 4$ m^3/Mg-yr = 0.065(ft^3/lb_M-yr)

or *(dG/dt)* = 2.06×10^{-9} (ft^3/lb_M-sec)

k and L_o are based on EPA's AP-42 emissions factors for dry sites.

Average reasonable applied vacuum at the wellhead *(P_o)* for an active gas extraction system with a 20-foot length of solid pipe:

P_o = 1.51 (inches of water column)

= 0.0547 (psig)

= 2,108.9 (lb_F/ft^2) absolute

Conversion: 1.0 (psig) = 27.7 (inches of water column)

Average reasonable absolute permeability of refuse *(K)* based on HELP model for determining leachate heads within a landfill.

K = 2.681×10^{-11}(ft^2)

Typical gas absolute viscosity at standard temperature conditions (60°F)

Absolute Viscosity Reference Values

methane (CH_4) = 7.1×10^{-6} (lb_M/ft.sec)

carbon dioxide (CO_2) = 9.8×10^{-6} (lb_M/ft.sec)

Standard landfill gas viscosity *(μ)* at 60°F:

$\mu = (0.5)(7.1 \times 10^{-6}) + (0.50)(9.8 \times 10^{-6})$

$\mu = 8.45 \times 10^{-6}$ (lb_M/ft.sec)

Determine the ratio of slotted pipe to total pipe section for typical gas extraction wells as specified by the designer.

Calculated average ratio value *(h_s / h_T)* = 0.59 for Kekaha Landfill, approximately two-thirds slotted length per total length.

Constants utilized in the Darcy zone of well influence, equation (7):

g_c = acceleration of gravity constant = 32.2 ($lb_{M.}ft / lb_{F.}sec^2$)

T_s = standard temperature = 520 (°R)

P_s = standard pressure = 2,116.8 (lb_F / ft^2)

ρ = density of refuse = 48.15 (lb_M / ft^3)

P_1 = pressure/vacuum at extreme zone *(r_1)* of influence convention pressure gradient

P_1 = 0 (inches of water column)

P_1 = 0 (psig) = 14.7 (psia) absolute

P_1 = 2,116.8 (lb_F / ft^2) absolute

$$r_1 = \left[\frac{(2 \times 32.2)(2.681 \times 10^{-11})(520)(0.59)[(2{,}116.8)^2 - (2{,}108.9)^2]}{(2{,}116.8)(2.06 \times 10^{-9})(48.15)(8.45 \times 10^{-6})(560)} \right]^{1/2}$$

Note that $P_1 = P_s$ = 2,116.8 (lb_F / ft^2) absolute atmospheric pressure.

Substitute in equation (7) to derive the Darcy zone of influence for a typical gas extraction well.

Therefore: r_1 = 133 (ft) = zone of well influence.

Figure 1 shows the geometry of a typical zone of influence for a landfill gas well. Table A-1 shows the calculated ZOI for all of the extraction wells. The maximum ZOI was determined to be 164 feet. The design,

$$r_1 = \left[1.78 \times 10^4 (ft^2)\right]^{1/2}$$

however, is limited to 150 feet to provide conservative spacing of the extraction wells.

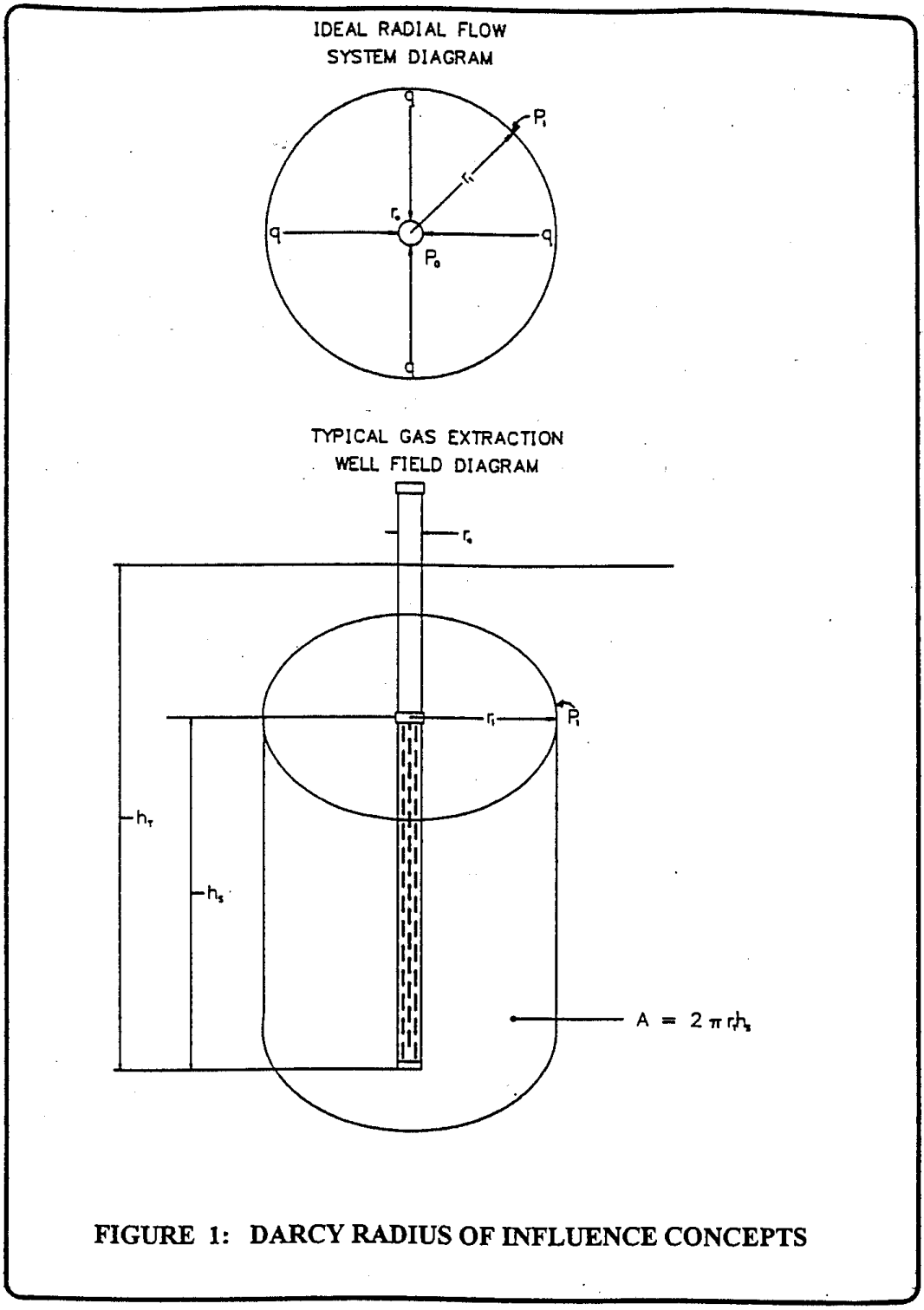


FIGURE 1: DARCY RADIUS OF INFLUENCE CONCEPTS

General Assumptions for Radius of Influence Calculation

The careful formulation of assumptions is critical and requires some knowledge of the landfill's characteristics.

Gas Generation Rate:

Landfill gas is the by-product of the anaerobic decomposition of organic material disposed of in a landfill, by methanogenic (methane producing) bacteria. Landfill gas production is assumed to have a first order reaction rate and is dependent upon the following:

- age of the landfill
- types of waste received
- location (i.e., climate and precipitation)
- moisture conditions within the refuse
- landfill cover materials and thicknesses

EPA's AP-42 "k" factor value of 0.02 $year^{-1}$ will be used. To convert this to English units for use in the ROI program, the k value must be multiplied by the theoretical yield. The theoretical yield from AP-42 is assumed to be 100 m^3 methane/Mg waste, which converts to 3.2 ft^3/lb waste. The gas generation rate is therefore 0.064 ft^3 LFG/lb refuse/year.

Permeability Factor:

Permeability is defined as a measure of the ability of a porous media to transmit fluids. While the permeability of refuse within a landfill can vary greatly, it is assumed to be a constant for ease of calculation. A reasonable absolute permeability value for refuse is 2.68×10^{-10} (ft^2) based on various research studies at landfills. This number was calculated by applying Darcy's Law for Linear Compressible Fluid Flow to the movement of landfill gas through refuse and assuming the following:

- Steady state flow conditions exist.
- The pore space of the refuse is 100 percent saturated with the flowing fluid (landfill gas).
- The viscosity of the flowing fluid is constant.
- Isothermal conditions in the refuse prevail.
- Flow is laminar, horizontal, and linear since refuse grain size is relatively small and the velocity of fluid flow is low.

Refuse Density:

Refuse density is a function of the types of waste received and the degree of compaction at the landfill site. A refuse density of 1,300 lbs/yd^3 will be used for the calculations for the facility based on average historical densities measured at the site.

Gas Temperature:

The temperatures within a landfill can influence the movement of landfill gas in two ways. First, since landfill gas is a compressible fluid, its viscosity and flow characteristics must be corrected to standard temperature and pressure conditions prior to using the Darcy Equation for radial fluid flow.

Secondly, a landfill's interior temperature can affect the rate at which landfill gas is generated since different types of bacteria are present at different temperatures. Methanogens (or methane producing bacteria) that generate landfill gas at temperatures below 110°F are known as mesophilic bacteria, while those that generate gas at temperatures in excess of 110°F are called thermophilic

bacteria. Although both types of bacteria produce approximately the same quality of gas, the gas generation rate is optimized in the thermophilic range.

Average Cover Depth:

The average thickness of final cover is subtracted from the refuse depth available for gas production. Soil is inert and will not contribute to the generation of landfill gas.

Maximum Radius of Influence:

The maximum ZOI was determined to be 164 feet. The design, however, is limited to 150 feet to provide conservative spacing of the extraction wells.

Average Overlap Factor:

It is desirable to achieve a certain degree of overlap of the circular ROIs. Since the calculations are theoretical to begin with, the overlap provides a factor of safety to the gas control system design. If field conditions prevent gas from moving towards a particular well, an overlap helps ensure that the gas can travel to more than one collection point.

The target range of overlap values is typically 10 to 15 percent, unless the designer requires a denser well spacing. The overlap value used for the facility's proposed well spacing is approximately 10%.

Appendix E.4
Control Device Sizing

Gas Mover Equipment Sizing

Per 40 CFR 60.752(b)(2)(ii)(A)(1), the active gas extraction system must be designed to handle the maximum expected gas flow rate from the entire area of the landfill that warrants control over the intended use period of the gas control system equipment. 40 CFR 60.752(b)(2)(ii)(A)(3) requires that gas be collected at a sufficient extraction rate. As mentioned above, the design is based on all of the projected gas volumes.

Since the blower is responsible for providing the vacuum that actually extracts the gas from the well field and moves it through the system, the sizing of the blower is crucial. General design criteria and the method for determining the required blower size are discussed in the following section.

General Design Criteria

Flow Volumes:

Since gas flow volumes will vary over the life of the gas extraction system the blower provides a uniform source of vacuum over a wide range of flow rates. Minimum system flows are those expected when only the initial phases of the system have been installed. Maximum flows will occur after the entire gas system is in place.

Pressure Requirements

The blower must be capable of supplying sufficient negative pressure to overcome pressure drops and resistance through piping and equipment at the calculated maximum gas flow rate, as well as supplying sufficient positive pressure for delivery to the flare for combustion.

The design blower will accommodate the maximum modeled flow rate of 590 cfm, and provide 40" w.c. of available vacuum.

Control Device Sizing

The control device must meet the requirements of 40 CFR §60.752(b)(2)(iii). The control device must be capable of combusting a wide range of flow volumes.

The proposed flare will be located at a flare station on the northeast corner of the site near the office building.

General Design Criteria

The flare at Kekaha Landfill will be designed to reduce the concentration of NMOCs present in the LFG delivered to the flare by at least 98 percent (by weight) or reduce outlet NMOC concentrations to less than 20 parts per million by volume (ppmv).

Per 40 CFR §60.752(b)(2)(iii)(B)(2), the LFG flare shall operate within the performance ranges established during the source performance tests and is operated in such a manner as to meet the emission requirements of the NSPS.

The flare capacity will meet the minimum 590 cfm flow rate, which is the maximum modeled flow rate for the site, and will be more than sufficient to handle current modeled gas flows.

Appendix E.5
Gas System Design Drawings

See Drawings 6 and 13 for Gas System Design

Appendix F
Geosynthetic Material Properties

Appendix F
Geosynthetic Material Properties

Appendix F.1
Geosynthetic Clay Liner

Appendix F.2
60-mil HDPE Textured Geomembrane

Appendix F.3
Nonwoven Cushion Geotextile

Appendix F.4
Nonwoven Separator Geotextile

Appendix F.5
40-mil Textured LLDPE Geomembrane

Appendix F.6
Geocomposite Drainage Layer

Appendix F.7
Interface Friction Angle Testing

Appendix F.1
Geosynthetic Clay Liner

Appendix F-1

Geosynthetic Clay Liner (GCL)

Property	ASTM Test Method	Value	Test Frequency
Clay (as received prior to manufacturing GCL)			
Swell index (ml/2g)	D5890	24	50 tonnes
Fluid Loss (ml)(1)	D5891	18	50 tonnes
Geotextiles (as received prior to manufacturing GCL)			
Cap fabric (nonwoven), mass per unit area (oz/yd^2)(2)	D5261	5.8	25,000 yd^2
Carrier fabric (nonwoven composite), mass per unit area (oz/yd^2)(2)	D5261	5.9	25,000 yd^2
GCL (as manufactured)			
Mass of GCL (lb/ft^2)(3)	D5993	0.81	5,000 yd^2
Mass of bentonite (lb/ft^2)(3)	D5993	0.75	5,000 yd^2
Tensile strength, MD (lb/in.)	D6768	23	25,000 yd^2
Peel strength (lb/in.)	D6496	2.1	5,000 yd^2
Permeability(1) (cm/sec)	D5887	$5x10^{-9}$	30,000 yd^2
Needle-punched reinforcement	---	Required	---

Notes:
(1) These values are maximum (all others are minimum).
(2) Carrier fabric for nonwoven reinforced GCL must contain a scrim component of mass >2.9 oz/yd^2 for dimensional stability.
(3) Mass of GCL and bentonite is measured after oven drying for stated test method.

Material properties based upon Geosynthetic Research Institute GRI –GCL3, Standard Specification for "Test Methods, Required Properties, and Testing Frequencies of Geosynthetic Clay Liners (GCls)", Revision #2 July 26, 2010.

Appendix F.2
60-mil HDPE Textured Geomembrane

Appendix F-2

60-mil HDPE Textured Geomembrane

Property	ASTM Test Method	Value	Test Frequency
Textured Geomembrane, textured both sides **			
Thickness, min. ave. (mils) Lowest individual for 8 out of 10 values (mils) Lowest individual for any of the 10 values (mils)	D5994	57 54 51	Per roll
Asperity height[5], min. ave. (mils)	D 7466	10	Every 2nd roll[6]
Density, min. (g/cm^3)	D1505/D 792	0.940	200,000 lbs
Tensile Properties[1], min. ave. Yield strength (lb/in.) Break strength (lb/in.) Yield elongation (%) Break elongation (%)	D6693, Type IV	 126 90 12 100	20,000 lbs
Tear resistance, min. ave. (lbs)	D1004	42	45,000 lbs
Puncture Resistance, min. ave. (lbs)	D4833	90	45,000 lbs
Stress crack resistance[2] (hrs)	D5397 (App.)	300	Per GRI-GM 10
Carbon black content (range) (%)	D4218[3]	2.0 to 3.0	20,000 lbs
Carbon black dispersion	D5596	Note 4	45,000 lbs
Oxidative induction time	Per current GRI GM13 at time of manufacture		
Oven aging at 85 degrees			
UV Resistance			

Notes:
(1) Machine direction (MD) and cross machine direction (XMD) average values should be basis of 5 test specimens each direction.
Yield elongation is calculated using gage length of 1.3 inches.
Break elongation is calculated using gage length of 2.0 inches.
(2) P-NCTL test is not appropriate for testing geomembranes with textured or irregular rough surfaces. Tests should be conducted on smooth edges of textured rolls or on smooth sheets made from the same formulation as being used for the textured sheet materials. The yield stress used to calculate the applied load for the SP-NCTL test should be the manufacturer's mean value via MQC testing.
(3) Other methods such as D 1603 (tube furnace) or D 6370 (TGA) are acceptable if an appropriate correlation to D 4218 (muffle furnace) can be established.
(4) Carbon black dispersion (only near spherical agglomerates) for 10 different views: 9 in Categories 1 or 2 and 1 in Category 3.
(5) Of 10 readings; 8 out of 10 must be $\geq$ 7 mils, and lowest reading must be $\geq$ 5 mils; also see Note 4.
(6) Alternate the measurement side for double-sided textured sheet.

Material properties based upon Geosynthetic Research Institute GRI –GM 13, Standard Specification for "Test Methods, Test Properties and Testing Frequency for High Density Polyethylene (HDPE) Smooth and Textured Geomembranes", Revision 10, dated April 11, 2011.

Appendix F.3
Nonwoven Cushion Geotextile

Appendix F-3

Nonwoven Cushion Geotextile

Property	ASTM Test Method	Value
Mass per unit area (oz/yd^2)	D5261	16
Grab tensile strength (lbs)	D4632	370
Grab tensile elongation (%)	D4632	50
Trapezoidal tear strength (lbs)	D4533	145
Puncture (pin) strength (lbs)	D4833	170
UV Resistance	D4355	70
Notes: (1) All values are minimum average roll value (MARV) except for UV resistance which is a minimum value. (2) UV Resistance evaluation to be on a 2.0 inch strip tensile specimen after 500 hours exposure.		

Material properties based upon Geosynthetic Research Institute GRI –GT12(a), Standard Specification for "Test Methods and Properties for Nonwoven Geotextiles Used as Protection (or Cushioning) Materials", dated February 18, 2002.

Appendix F.4
Nonwoven Separator Geotextile

Appendix F-4

Nonwoven Separator Geotextile

Property	ASTM Test Method	Value
Mass per unit area (oz/yd^2)	D5261	6
Apparent Opening Size (US Sieve)	D4751	70
Permittivity (sec^{-1})	D4491	1.5
Permeability (cm/sec)	D4491	0.30
Geotextile Structure	Nonwoven, needle-punched	
Notes: (1) All values are minimum average roll value (MARV) except for Apparent Opening Size which is a maximum average roll value.		

Appendix F.5
40-mil Textured LLDPE Geomembrane

Appendix F-5

40-mil Textured LLDPE Geomembrane

Property	ASTM Test Method	Value	Test Frequency
Textured Geomembrane, textured both sides (final cover)			
Thickness, min. ave. (mils) Lowest individual for 8 out of 10 values (mils) Lowest individual for any of the 10 values (mils)	D5994	38 36 34	Per roll
Asperity height[1], min. ave. (mils)	D7466	10	Every 2nd roll[2]
Density, min. (g/cm^3)	D1505/D 792	0.939	200,000 lbs
Tensile Properties[3], min. ave. Break strength (lb/in.) Break elongation (%)	D6693, Type IV	 60 250	 20,000 lbs
2% modulus, max. (lb/in.)	D5323	2400	Per formulation
Tear resistance, min. ave. (lbs)	D1004	22	45,000 lbs
Puncture Resistance, min. ave. (lbs)	D4833	44	45,000 lbs
Axi-Symmetric break resistance strain, min. (%)	D5617	30	Per formulation
Carbon black content (range) (%)	D4218[4]	2.0 to 3.0	20,000 lbs
Carbon black dispersion	D5596	Note 5	45,000 lbs
Oxidative induction time	Per current GRI GM17 at time of manufacture		
Oven aging at 85 degrees			
UV Resistance			

Notes:
(1) Of 10 readings; 8 out of 10 must be $\geq$ 7 mils, and lowest reading must be $\geq$ 5 mils; also see Note 6.
(2) Alternate the measurement side for double-sided textured sheet.
(3) Machine direction (MD) and cross machine direction (XMD) average values should be basis of 5 test specimens each direction. Break elongation is calculated using gage length of 2.0 inches at 2.0 in/minute.
(4) Other methods such as D 1603 (tube furnace) or D 6370 (TGA) are acceptable if an appropriate correlation to D 4218 (muffle furnace) can be established.
(5) Carbon black dispersion (only near spherical agglomerates) for 10 different views: 9 in Categories 1 or 2 and 1 in Category 3.
(6) Not recommended since the high temperature of the Std-OIT test produces an unrealistic result for some of the antioxidants in the UV exposed samples.

Material properties based upon Geosynthetic Research Institute GRI –GM 17, Standard Specification for "Test Methods, Test Properties and Testing Frequency for Linear Low Density Polyethylene (LLDPE) Smooth and Textured Geomembranes", Revision 8, dated October 3, 2011.

Appendix F.6
Geocomposite Drainage Layer

Appendix F-6

Geocomposite Drainage Layer

Property[1]	ASTM Test Method	Value	Test Frequency
HDPE Geonet core (prior to lamination)			
Thickness, min. (mils)	D5199	200	50,000 ft^2
Density, min (g/cm^3)	D1505	0.940	50,000 ft^2
Tensile strength (MD), (lb/in.)	D5035	45	50,000 ft^2
Carbon black content (%)	D1603 (modified) / ASTM D4218	2.0	50,000 ft^2
Nonwoven Geotextile[2] (prior to lamination)			
Mass per unit area (oz/yd^2)	D5621	8	90,000 ft^2
Grab tensile strength (lbs)	D4632	220	90,000 ft^2
Puncture strength (lbs)	D4833	120	90,000 ft^2
AOS, max. ave. (US sieve)	D4751	80	540,000 ft^2
Permittivity (sec^{-1})	D4491	1.3	540,000 ft^2
Flow rate (gpm/ft^2)	D4491	75	540,000 ft^2
UV resistance (% retained)	D4355 (after 500 hrs)	70	Per formulation
Geocomposite (after assembly)			
Transmissivity[3], min. (m^2/sec)	D4716	$1x10^{-4}$	540,000 ft^2
Ply adhesion (lb/in.)	D7005	1.0	50,000 ft^2

Notes:
(1) All values are minimum average roll value (MARV) unless otherwise noted.
(2) Geocomposite to consist of a nonwoven geotextile laminated (heat-bonded) to both sides of the geonet core.
(3) Transmissivity of geocomposite to be used in final cover to be determined by using proposed final cover soils at representative loadings, gradient, and seat time determined by Engineer.

Appendix F.7
Interface Friction Angle Testing

Appendix F-7

Interface Friction Angle Testing

Interface	Test Condition	ASTM Test Method	Minimum Friction Angle[1]
Cell 2 Base Liner System			
1. Operation Layer to Separation Nonwoven Geotextile	Direct Contact	D5321	33.0 degrees[(4)]
2. Separation Nonwoven Geotextile to Drainage Layer	Direct Contact	D5321	33.0 degrees[(4)]
3. Drainage Layer to Cushion Nonwoven Geotextile to Textured HDPE Geomembrane	Floating	D5321	19.5 degrees[(4)]
4. Textured HDPE Geomembrane to Nonwoven Geotextile (Top) of GCL	Direct Contact	D6243	19.5 degrees[(4)]
5. Nonwoven geotextile (Bottom) of GCL to Textured HDPE Geomembrane	Direct Contact	D6243	19.5 degrees[(4)]
6. Textured HDPE Geomembrane to Nonwoven Geotextile (Cushion)	Direct Contact	D5321	19.5 degrees[(4)]
7. Nonwoven Geotextile (Cushion) to Subbase Soils (multiple tests for different subbase material types)	Direct Contact	D5321	19.5 degrees[(4)]

Interface	Test Condition	ASTM Test Method	Minimum Friction Angle[1]
Cell 2 Final Cover System and Phase 1 Final Cover Geomembrane Extension[5]			
1. Rooting Zone to Nonwoven Geotextile (geocomposite – Cell 2 or cushion geotextile – Phase 1)	Direct Contact	D5321	23.1 degrees
2. Nonwoven Geotextile (geocomposite – Cell 2 or cushion geotextile - Phase 1) to Textured LLDPE Geomembrane	Direct Contact	D5321	23.1 degrees
3. Textured LLDPE Geomembrane to Nonwoven Geotextile (Top) of GCL	Direct Contact	D6243	23.1 degrees
4. Nonwoven geotextile (Bottom) of GCL to Textured LLDPE Geomembrane	Direct Contact	D6243	23.1 degrees
5. Textured LLDPE Geomembrane to Grading Layer (Cell 2 only)	Direct Contact	D5321	23.1 degrees
6. Textured LLDPE Geomembrane to Inplace low permeable soil (Phase 1 only)	Direct Contact	D5321	23.1 degrees

Notes:

1. Minimum friction angles are presented solely as a friction angle without apparent adhesion. Laboratory test results which provide an adhesion factor and a lower resulting friction angle may be approved by the Design Engineer for the specific application.
2. Interface friction angle testing to be completed prior to acceptance of geosynthetic and soil materials. Any changes in material, interface friction angle testing should be retested.
3. Coordinate test loads, seating parameters, shear rates, and wet/dry interface requirements with Design Engineer prior to testing to confirm current test methods, procedures, and practices.
4. Friction angle based drainage layer stability analysis to place drainage media up longest base sideslope. Lower friction angle may be acceptable as long as friction angle meets or exceeds the minimum friction angle for buildout stability and the sideslope length for drainage media placement is reduced to reflect the lower friction angle.
5. Interfaces 1, 2, and 6 apply to the Phase 1 final cover geomembrane extension. Interfaces 1, 2, 3, 4, and 5 apply to Cell 2.

Appendix G
Point of Compliance Assessment

Point of Compliance Assessment Update Kekaha Landfill Phase II, Cell 2 Expansion Kekaha, Kaua‘i, Hawai‘i

Prepared for

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December 2015

POINT OF COMPLIANCE ASSESSMENT UPDATE
KEKAHA LANDFILL
PHASE II, CELL 2 EXPANSION
KEKAHA, KAUA'I, HAWAI'I

DECEMBER 2015

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1.0 INTRODUCTION

The County of Kaua‘i currently owns and operates a landfill facility located near the town of Kekaha in the coastal area on the southwest side of the island of Kaua‘i. The facility, as permitted, currently has two noncontiguous landfill phases, Phase I and Phase II. The County of Kaua‘i is proposing an expansion of the Phase II landfill to obtain additional airspace while potential future landfill sites are evaluated and eventually permitted. The most recent permitted lateral expansion of the Phase II landfill consisted of a new waste disposal area of approximately 6.4 acres located to the west end of the Phase II, designated Cell 1.

The proposed expansion will include the construction of an additional engineered waste disposal area, designated Cell 2. Cell 2 is to be located immediately adjacent to the south side of the existing Phase II, and will overlay both the north face of existing Phase I and the south face of the existing Phase II. Construction of Cell 2 will provide approximately 6.6 acres of additional waste filling area and create a continuous Subtitle D base liner system that underlies all of the Phase II landfill, including Cell 1 and Cell 2.

Currently, the leachate extracted from Phase II and Cell 1 is collected and managed in an evaporation pond north of Phase II (which was relocated and constructed as part of the Cell 1 expansion). Leachate from the Cell 2 area will be collected and managed similarly to the current Phase II and Cell 1 areas. The evaporation pond was designed with sufficient capacity to handle leachate from all of the proposed Phase II landfill.

This Point of Compliance Assessment (POCA) update is to consider the proposed Cell 2 in the POCA. A POCA was previously prepared in 1993 by Harding Lawson Associates (HLA) in their report titled, “Point of Compliance Assessment, Kekaha Landfill Phase II, Kekaha, Kauai, Hawaii,” prepared for the County of Kauai for Phase II based upon the Phase II existing liner system. AECOM (previously Earth Tech) prepared a POCA update in September 2008 in a report titled “Point of Compliance Assessment Update, Kekaha Sanitary Landfill Phase II Lateral Expansion” to incorporate the Cell 1 lateral expansion. This update additionally incorporates estimated leakage from Cell 2 lateral expansion liner and sumps as well as the most recent leachate testing data from June 2015.

1.1 Existing Phase II and Cell 1 Base Liner System

The Phase II landfill facility constructed in 1993 is bounded by Kaumuali‘i Highway to the northeast, an unpaved access road and agriculture to the southeast, aquaculture to the northwest, and the Phase I area to the southwest. The landfill base liner elevations and leachate collection system of Phase II varies from about 12 feet above msl to about 7 feet above msl and drains towards the northeast. The existing Phase II landfill sideslopes are 3.5:1 (horizontal:vertical), with a 3% topslope. Phase II liner area encompasses 32 acres and is subdivided into 14 lined subcells (each about 2.3 acres). Phase II was expanded to the northwest with approval of Cell 1. Cell 1 consists of 4 subcells with 3 subcells draining to the northwest and one to the northeast.

The existing base liner system of Phase II consists of (bottom to top):

- Granular subbase.
- Geosynthetic Clay Liner (GCL) consisting of a smooth 20-mil high-density polyethylene (HDPE) geomembrane with an adhered bentonite powder layer on one side (Gundseal). The GCL was installed with the smooth side in direct contact with the granular subbase and the bentonite component side up.
- Single-sided textured 60-mil HDPE geomembrane (textured side down) against bentonite component of the GCL on the sideslope.
- Smooth 60-mil HDPE geomembrane against bentonite component of the GCL on the base.
- Sand drainage layer (base and sideslope).

The existing base liner system for Cell 1 of Phase II consists of (bottom to top):

- Prepared subbase grade
- Subbase 60-mil HDPE geomembrane, textured on both sides of geomembrane
- GCL (lower component)
- 60-mil HDPE geomembrane (upper component), textured on both sides of geomembrane
- Nonwoven cushion geotextile
- 12-inch granular drainage layer (leachate collection)
- Nonwoven separator geotextile
- 24-inch operations layer

1.1.1 Phase II Point of Compliance Assessment (1993)

HLA of Aiea, Hawaii completed a Point of Compliance Assessment to demonstrate equivalency of a 2-foot recompacted clay liner to a GCL while meeting compliance requirements for chemical concentrations in the groundwater at the point of compliance. Using the USEPA Hydrologic Evaluation of Landfill Performance (HELP) model Version 2 and practices at that time, HLA determined that a GCL with a maximum permeability of $1x10^{-9}$ cm/sec performed as an equivalent or better barrier layer to 2 feet of recompacted clay liner with a $1x10^{-7}$ cm/sec maximum permeability.

Performance standards calculated by HLA used the HELP model results for the GCL liner system which estimated 11 ft^3 of leachate per year per acre would infiltrate through the base liner. With an area of 32 acres for Phase II, this would equal approximately 350 ft^3/year. Table 2 - Typical Landfill Leachate Concentrations and Estimated Concentrations at the Point of Compliance, of the HLA POCA report determined estimated chemical concentrations in groundwater are at least one order of magnitude below corresponding maximum contaminant levels (MCLs).

1.1.2 Cell 1 Point of Compliance Assessment Update (2008)

AECOM (previously Earth Tech) completed a POCA update to demonstrate equivalency of a 2-foot recompacted clay liner to a GCL and to determine additional impacts, if any, to the groundwater as previously determined in 1993 by HLA. The report used HELP model Version 3.07 and hand calculations to estimate the infiltration through the liner for the Cell 1 area, a Cell 1 sump, and the leachate evaporation lagoon.

The GCL used for Cell 1 is different than the existing Phase II base liner. The GCL for Cell 1 is a composite product consisting of a bentonite powder/granules encapsulated between a carrier woven geotextile and a cover nonwoven geotextile. The resulting composite of the bentonite and geotextiles are then needle punched for additional strength. GCLs are widely accepted substitutes for the lower component of a composite liner system with the incorporation a prepared subbase surface for the GCL to be installed upon. The maximum permeability of this GCL was modeled as $5.0x10^{-9}$ cm/sec, typical for GCLs available at the time and currently.

The report determined that the Cell 1 liner performance is equivalent or better than a 2-foot recompacted clay liner. The 2008 POCA Update also estimated the leakage through the Cell 1 liner to be 1.0 to 2.1 ft^3/yr, and that it will not have a significant impact to the MCLs as presented in the HLA POCA.

1.2 Cell 2 Expansion Base Liner System

The base liner system for the Phase II Cell 2 expansion will be consistent with the approved base liner of Cell 1 of Phase II. The Cell 2 base liner is expected to consist of (bottom to top):

- Prepared subbase grade
- Nonwoven cushion geotextile
- Subbase 60-mil HDPE geomembrane, textured on both sides of geomembrane
- GCL (lower component)
- 60-mil HDPE geomembrane (upper component), textured on both sides of geomembrane
- Nonwoven cushion geotextile

- 12-inch granular drainage layer (leachate collection)
- Nonwoven separator geotextile
- 24-inch operations layer

2.0 POINT OF COMPLIANCE ASSESSMENT

To perform the POCA, estimated maximum chemical concentrations immediately downgradient of the landfill were calculated for various chemicals, based on actual (2015) constituent concentrations, estimated quantity of leachate leaking out of the landfill, and the estimated groundwater flow.

2.1 Leachate Leakage Through the Base Liner

Leachate leakage quantities through the leachate evaporation lagoon, landfill liners, and sump areas were estimated as discussed below.

2.1.1 Leachate Evaporation Lagoon Leakage

Leakage through the leachate evaporation lagoon liner was discussed in Section 3.2 of the September 2008 Point of Compliance Assessment Update. The 2008 update considered conservative conditions that estimated the maximum leakage through the 2-acre evaporation lagoon liner to be 21.0 ft^3/acre-year. The Cell 2 Expansion does not modify the leachate evaporation lagoon, therefore, the 2008 leakage estimate calculation is still valid.

2.1.2 Landfill Base Liner Leachate Leakage

For the landfill, the quantity of leachate leakage through the base liner is estimated using the USEPA HELP Model Version 3.07, dated November 1, 1997. HELP was developed by the USACE Waterways Experiment Station for the USEPA to evaluate water movement through landfills. The program is used to calculate the amount of evapotranspiration, surface water run-off, percolation through installed covers, leachate generation and leachate head that may be expected from a wide variety of landfill designs, and is based upon various climatologic and soil data.

Leachate leakage through the base liner is estimated for the Phase II, Cell 1, and Cell 2 areas using current weather data and site conditions consistent with the June 2015 POCA memo prepared by AECOM. Model inputs and leakage estimate results are discussed in more detail in Attachment 1, Phase II, Cell 1 and Cell 2 areas.

HELP requires weather data to be input for evapotranspiration, precipitation, temperature, and solar radiation. Evapotranspiration data includes weather station latitude, evaporative zone depth, maximum leaf area index, growing season start and end dates, wind speed, and relative humidity. When possible, site specific weather data was utilized.

HELP also requires soil and design data for general landfill information, layer design (cover and liner components), and the run-off curve number.

- General landfill information includes the waste footprint area, percentage of the landfill where run-off is possible, method of initializing moisture contents, and initial snow water storage.
- Layer data includes layer type, layer thickness, soil characteristics, and initial soil water content. The following four layer types can be used: barrier soil liner, geomembrane liner, lateral drainage, and vertical percolation.
- Soil characteristics which must be input include porosity, field capacity, wilting point, and saturated hydraulic conductivity. The user can input each soil characteristic, or use default characteristics which are provided for 42 different types of soils/materials.
- For lateral drainage layers, the user must input the maximum drainage length and drain slope.
- For geomembranes, the user must input data regarding assumed manufacturing and installation defects (holes per acre) and the quality of the liner placement.
- Soil types and thickness used in modeling the landfill cross-section are based on engineering design.

The soil information input into HELP uses a combination of default and user defined data to describe a

typical cross-section through the landfill. The data input to describe the typical cross-section includes the landfill base liner profile from the top to the bottom of the liner system.

2.1.2.1 HELP Model Layers

For the analysis of the landfill using HELP, each component of the landfill cross-section is modeled as one of the four layer types used in HELP (vertical percolation, lateral drainage, barrier soil liner, or geomembrane liner). The classification of layers is based on the flow of liquid through the layers. HELP model layer classifications and corresponding design layers are presented in Attachment A. HELP user's guide for Version 3 (Schroeder, 1994) provides the following descriptions:

- Barrier soil liner: This layer is a soil layer intended to restrict drainage (percolation/ leakage). Liquid only has the potential to flow vertically through this layer when there is a positive head, or standing liquid, on top of the layer.
- Geomembrane liner: This layer is a low permeability synthetic membrane that reduces the amount of vertical liquid movement. Vertical movement of liquid through this layer can occur primarily through assumed defects in the geomembrane when there is a positive head on top of the layer.
- Lateral drainage layer: This layer can be soil or a geosynthetic material that is located directly above a liner that allows the horizontal (lateral) as well as vertical movement of liquids. HELP assumes the lateral movement of liquid in this layer to be unrestricted, meaning that there is a discharge from a lateral drainage layer.
- Vertical percolation layer: This layer can be soil or a geosynthetic material that provides moisture storage and allows the vertical movement of liquid (precipitation and leachate).

2.1.2.2 HELP Model Landfill Conditions

At any given time during landfill construction and waste filling, the landfill will be under one of three conditions. The three conditions are active areas (open), areas with intermediate cover (intermediate), and areas with final cover in place (closed). The primary difference between the three conditions is the cover material over the waste. The proposed base liner system is the same for each condition while the thickness of daily, intermediate and final cover varies. For purposes of this evaluation, the landfill was modeled with intermediate cover in place for Phase II and Cell 1 with an area of Cell 1 modeled as the active working face where no rainfall will run-off, but all will become leachate. Conservatively, for Cell 2, 50 percent of the landfill area is modeled with intermediate cover while 50 percent is modeled with no run-off and all rainfall becoming leachate.

Consistent with the HAL POCA, and the Earth Tech/AECOM POCA update, the HELP program determined initial moisture content. A 1-acre unit area is modeled to simplify unit comparisons.

2.1.2.3 HELP Model Geomembrane Manufacturing and Installation

Controlling factors in the HELP model include the defects per acre from manufacturing defects (pinholes) and placement quality of the geomembrane. Pinhole density is assumed to account for possible manufacturing defects during geomembrane production. The Help user guide for HELP Version 3.07 states pinhole density for a typical geomembrane is 0.5 to 1 pinhole per acre. For conservative analysis, a pinhole density of 1 was assumed.

Installation placement quality is dependent upon installation techniques, well prepared subbase, and design requirements through project specifications and the use of qualified installers. Project specifications for the expansion specify these requirements and will be enforced during liner installation. Based upon this, placement quality of "good" was applied and reflects the HELP user guide definition of "…assumes good field installation with a well prepared, smooth soil surface and wrinkle control".

With an installation quality of "good", the number of installation defects recommended by the HELP user guide is 1 to 4 installation defects per acre. For this analysis, each base liner system was analyzed with 2 defects per acre.

2.1.1.4 HELP Model Leachate Flow Length

Leachate head, or head on the base, is the depth of leachate on top of the base liner system. Leachate head buildup is dependent on several factors, including drainage distance, drainage slope, permeability of granular drainage layer material and the leachate generation rate.

Drainage distance is the distance leachate travels on the composite liner through the granular drainage layer to reach the leachate collection pipes. The leachate drainage slope represents the slope of the base liner towards the leachate collection pipe. To be conservative, the maximum slope length and shallowest base liner slope are used in the analysis.

2.1.3 Sump Area Leakage

By design and function, the Cell 2 sump areas will have more leachate head than areas of the landfill floor, and therefore, more potential leachate leakage. The Phase II leachate collection system gravity drains to manholes outside the limits of waste, therefore, there will be minimal leachate head increases at the discharge points of Phase II. The Cell 1 sumps also gravity drain to outside the limits of waste and include factory fabricated HDPE flatstock, which is thicker and more resistant to damage than a geomembrane liner. Therefore, no additional leakage is expected from Cell 1 sumps. Typical pump operation in Cell 2 will maintain less than 16-inches of leachate head in the sump which results in less than 10-inches of leachate head on the cell floor. To be conservative a constant leachate head of 16 inches is assumed for the leachate leakage calculations in the Cell 2 sump areas.

Two empirical formulas for leakage rates through a geomembrane by Giroud and Bonaparte (1989) and Giroud (1989) were used to estimate leakage in the Cell 2 sump areas for a scenario of 2 installation defects per acre and 1 manufacturing defect per acre (similar to HELP modeling). An estimated leachate leakage of 2.60 ft^3/acre-year was determined for each sump area as shown in Attachment 2. The leachate head is conservatively assumed for the sump areas and the entire wetted area of the Cell 2 floor to be a total of 0.12 acres for all four sump areas combined.

2.1.4 Leachate Leakage Quantities

Leachate leakage is summarized in the following table:

Table 1: Leachate Leakage Summary

Location Description	Leachate Leakage Rate (ft^3/acre-yr)	Area (acre)	Leachate Leakage (ft^3/yr)	See Backup Data In
2014 Phase II	0.184	32	5.888	Attachment 1-A-1
2014 Cell 1	0.005	6.4	0.032	Attachment 1-A-2
Cell 2	0.024	6.6	0.158	Attachment 1-A-3
Cell 2 Sumps	2.60	0.12	0.312	Attachment 2
Leachate Evaporation Lagoon	21.0	2	42.0	September 2008 POC
		Total Leachate Leakage	48.39	--

The total leachate leakage for Phase II, Cell 1, Cell 2, the Cell 2 sumps, and the leachate evaporation lagoon is conservatively estimated to be 48.39 ft^3/yr. The Cell 1, Cell 2, and Cell 2 sump leachate leakage estimates are additionally conservative in that the subbase geomembrane is not included in the analysis, only the primary geomembrane and GCL are considered.

The total leachate leakage estimated in the HAL POCA was 350 ft^3/yr.

2.2 Groundwater Flow

The volumetric flow rate of groundwater in the aquifer beneath the landfill is needed to determine the chemical concentrations at the point of compliance. The volumetric flow rate of groundwater in the aquifer beneath the landfill can be calculated (consistent with the original POC assessment) as follows:

$$Q_a = KIBW$$

Where,

K = hydraulic conductivity (ft/day)
I = hydraulic gradient (feet/foot)
B = aquifer thickness (ft)
W = width of affected area perpendicular to the groundwater flow direction (ft)

The HLA POCA assumed the hydraulic conductivity to be the same as permeability (15.6 ft/day). A 1995 aquifer test measured the hydraulic conductivity of the caprock aquifer underyling the site to be approximately 162 ft/day. This higher estimate of hydraulic conductivity would result in a lower chemical concentration at the point of compliance because the leaked leachate would be more than 10 times as diluted. Therefore, to be conservative, the 15 ft/day hydraulic conductivity is used. Recent groundwater level data (i.e. February 2014, May 2014, August 2014, and November 2014), reflect a variable groundwater flow direction to the west, south-southeast with average hydraulic gradient of 0.0008 ft/ft, and aquifer thickness was set equal to the value used in the original POCA, 10 feet. The width of affected area perpendicular to groundwater flow as measured diagonally from the west corner of Cell 2 to the east corner of Phase II is approximately 1,900 feet. Accordingly:

$$Q_a = (15.6 \text{ ft/day})(0.0008 \text{ ft/ft})(10 \text{ ft})(1{,}900 \text{ ft})$$
$$= 237.12 \text{ ft}^3\text{/day}$$
$$= 86{,}549 \text{ ft}^3\text{/year}$$

2.3 Chemical Concentrations

Chemical concentrations at the point of compliance were calculated for each chemical of concern noted in the original HAL POCA. The concentration of an individual chemical in groundwater immediately downgradient of the landfill was calculated using the same equation as in the original POC assessment:

$$C_{gw} = ((Q_pC_p) + (Q_aC_a))/(Q_p + Q_a)$$

Where,

C_{gw} = chemical concentration in groundwater (mg/L)
Q_p = volumetric flow rate of infiltration through the affected area into the aquifer (ft^3/day); considered to be the leakage from the landfill liner
C_p = chemical concentration in the infiltration (mg/L)
Q_a = volumetric flow rate of groundwater in the aquifer directly beneath the affected area (ft^3/day)
C_a = initial or background concentration of chemical in aquifer (mg/L)

From Section 2.1.4 above, the leachate leakage rate, Q_p is estimated to be 48.39 ft^3/yr. From Section 2.2 above, the groundwater flow rate, Q_a, is estimated to be 86,549 ft^3/yr. Assuming that background concentrations for the chemicals of concern are negligible (again consistent with the original POC assessment):

$$C_{gw} = [(48.39 \text{ ft}^3\text{/yr } (Cp)+(86{,}549 \text{ ft}^3\text{/yr}) (0 \text{ mg/L})] /[(48.39 \text{ ft}^3\text{/yr}) + (86{,}549 \text{ ft}^3\text{/yr})]$$
$$= 5.59 \times 10^{-4} C_p$$

Chemical concentrations at the point of compliance (C_{gw}) were calculated for each chemical of concern for which leachate concentration data are available, assuming C_p was equal to the maximum leachate concentrations for samples collected from the Wet Well-1, Wet Well-2, and Wet Well-3 sumps in

Table 2
Estimated Leachate Concentrations at the Point of Compliance

Chemical	Maximum Contaminant Level mg/L	Leachate Concentration[1] mg/L	Estimated Concentration at Point of Compliance[2] mg/L
1,1,1-Trichloroethane	0.2	ND	--
1,1-Dichloroethylene	0.007	Not Available	--
1,2-Dichloroethane	0.005	ND (0.01*)	5.59E-06
1,4-Dichlorobenzene	0.075	0.006	3.35E-06
2, 4-Dichlorophenoxy Acetic Acid	0.1	0.0018	1.01E-06
2,4,5-Trichlorophenoxy Acetic Acid	0.01	ND (<0.005*)	2.80E-06
Arsenic	0.05	0.15	8.39E-04
Barium	1	ND	--
Benzene	0.005	0.0016	8.94E-07
Cadmium	0.01	ND	--
Carbon Tetrachloride	0.005	ND	--
Chromium	0.05	0.025	1.40E-05
Endrin	0.0002	ND (<0.005*)	2.80E-06
Fluoride	4	Not Available	--
Lead	0.05	ND	--
Lindane	0.004	Not Available (<0.005*)	2.80E-06
Mercury	0.002	ND	--
Methoxychlor	0.1	Not Available (<0.005*)	2.80E-06
Nitrate-Nitrite	10	2.6	1.45E-03
Selenium	0.01	0.00077	4.30E-07
Silver	0.05	ND	--
Toxaphene	0.005	Not Available (<0.005*)	2.80E-06
Trichloroethylene	0.005	Not Available (<0.069*)	3.86E-05
Vinyl Chloride	0.002	ND	--

Notes:

1. Leachate concentration based on most recent available 2014 site data, November 2014, unless otherwise indicated.
2. Estimated concentration at point of compliance based on a concentration factor of 5.59E-04.

3 Leachate leakage estimate is cumulative for Phase II, Cell 1 and Cell 2 linear areas and Cell 2 sump areas.

J Estimated result is less than the reporting limit but greater than or equal to the method detection limit.

ND Not detected

* Indicates values based on typical landfills as presented in the original POC assessment.

P The RPD between the primary and confirmation column/detector is >40%. The lower value has been reported.

RPD Relative percent difference

November 2014. For chemicals of concern with no site specific data available, consistent with the original POC, values for typical landfill leachate were used. In this way estimated concentrations at the point of compliance have been updated based on the current leachate leakage analysis, confirmation of the hydrogeologic characteristics, and chemical characteristics of site leachate. The results of these calculations are summarized in Table 2.

3.0 CONCLUSIONS

This Point of Compliance Assessment update incorporates conservative assumptions, consistent with the original HLA POCA. Additional conservative assumptions include modeling only the upper geomembrane and GCL while disregarding the lower geomembrane. Because this POCA update shows the estimated concentrations are below the MCLs at the point of compliance, the liner system meets the performance standard for municipal solid waste units and confirms the validity of the alternative liner design.

Appendix G

Attachment 1

Phase II, Cell 1 and Cell 2 Liner Leakage Estimates

Client	County of Kaua'i	**Subject**	Liner Leakage Estimates	**Prepared By** NKW	**Date** 12/15/15
Project	Kekaha Landfill Phase II Lateral Expansion			**Reviewed By** DPW	**Date** 12/16/15
				Approved By NKW	**Date** 12/16/15

LEACHATE GENERATION ANALYSIS

Objective

Estimate the quantity of leachate leakage through the landfill and composite liner in the Phase II, Cell 1 and Cell 2 liner areas for Kekaha Landfill using the USEPA Hydraulic Evaluation of Landfill Performance Version 3.07 (HELP) computer modeling program.

Criteria, Assumptions and Methodology

1. This analysis was performed to estimate leachate leakage rates for Phase II, Cell 1 and Cell 2.

2. The analysis includes three HELP model runs, one for Phase II, one for the Cell 1 area, and one for the Cell 2 area. Output is attached in Attachment 1-A for Cell 1, Attachment 1-B for Phase II, and Attachment 1-C for Cell 2.

3. Intermediate conditions are modeled. Intermediate conditions refers to the situation of 12 inches of temporary cover (without final cover).

HELP Model Version 3.07 Input:

1. Solar radiation data can be default data, user input or synthetically generated by the HELP model. Synthetically generated solar radiation data for Honolulu, Hawaii was used to simulate site weather conditions.

2. Temperature, precipitation, and average wind speed data were obtained from site records from January 2010 through December 2014. Missing data are viewed by the model as a "0". There are no missing data from 2010, 2011, and 2012. For 2013, temperature, precipitation, and average wind speed data were not available from site records for June 11, 2013, through June 30, 2013. For 2014, temperature, precipitation, and average wind speed data were not available from site records for May 9 through June 23 or for August 12 through September 1, or for December 18 through December 21. Data to fill-in for the missing site data were obtained from Weather Underground for the Barking Sands Airport, the closest weather station to the Kekaha Landfill, located about 4 miles away north and west along the Kaua'i coast.

3. Geomembrane liner pinhole density and size were assumed to account for possible manufacturing defects during geomembrane production. The HELP Model User's Guide for Version 3.07 states that the pinhole density for a typical geomembrane is 0.5 to 1 pinhole per acre. One pinhole per acre was assumed to be present.

4. The placement quality for the geomembrane liner was assumed to be "good". A placement quality of "good" according to the HELP Model User's Guide "assumes good field installation with well prepared, smooth soil surface and geomembrane wrinkle control". The HELP Model User's Guide suggests 1 to 4 installation defects for an installation quality of "good". To be conservative, 2 installation defects per acre were assumed for the geomembrane liner.

5. It is assumed that intermediate cover is in place in the Phase II area and 100% of the area will allow rainwater runoff.

6. It is assumed that intermediate cover is in place in the Cell 1 area and 100% of the area will allow rainwater runoff.

7. It is assumed that intermediate cover is in place in the Cell 2 area and that the working face of the landfill is approximately 100 feet by 200 feet. This approximately 0.46-acre area consists of 7.0% of the 6.6-acre Cell 2 area. Therefore, all runoff from this 0.46-acre area will be treated as leachate. The area of Cell 2 allowing rainwater runoff is, therefore, 93.0%.

8. The curve number was calculated by the HELP model based on the soil type and intermediate cover slopes.

    a. For the Phase II area, the intermediate cover slopes are assumed to follow final cover slopes. To be conservative a 320-foot, 3% slope is used. The slope and length are based on the flatter top areas of the landfill as shown on Drawing No. 1, "Kekaha Landfill Second Vertical Expansion Final Grading Plan" prepared by AECOM dated June 2004.
    b. For the Cell 1 area, grades are assumed to be at a 23% slope, with a maximum slope length of 220 feet. The slope and length are assumed based on March 6, 2015 topography.
    c. For the Cell 2 area, grades are assumed to be at a 28% slope with a maximum slope length of 150 feet.

9. The initial moisture content of the GCL was set equal to porosity to show the GCL as saturated (hydrated). The initial moisture content of the other soil layers was conservatively assumed to be at field capacity. Initial moisture contents of waste layers are discussed below.

10. Based on a spring 2000 waste characterization study, the moisture content of the incoming waste was found to be 32.7%. Actual waste on site is assumed to have a higher moisture content for various reasons including rain during filling operations.

11. To reflect site conditions of waste arriving on site below field capacity and, with time reaching field capacity, the waste mass was divided into separate layers separated by soil layers representing daily cover. The moisture content of each waste layer decreases from field capacity for the deepest layer to 86% of field capacity for the most recently placed waste. The initial moisture content of each waste layer is included in Tables 1 and 2 in this calculation.

12. Soil layer properties (porosity, field capacity, wilting point, saturated hydraulic conductivity) were generally modeled using HELP model default soil characteristics. The exception is the drainage layer. The drainage layer was assumed to have a hydraulic conductivity of 6.0×10^{-3} cm/sec to be consistent with local sand used in the drainage layer. See Appendix D of Reference 1. Initial soil and waste layer moisture contents are discussed above.

13. The longest flow path and shallowest slope along the base liner was input, resulting in the highest potential leachate head estimate.

    a. For the Phase II liner, the longest flow path is 70.7 feet and the slope will be 0.7 percent. This information is based on a 100-foot distance between leachate collection pipes, 0.5% slope toward pipes, and 0.5% slope on the pipe.
    b. For the Cell 1 liner, the longest flow path is 240 feet and the slope will be 2.8 percent. This is based on the Phase II Lateral Expansion construction drawings prepared for the Cell 1 base liner by AECOM dated April 2009. This information is based on 2.0% slope toward the collection pipe, and a slope of 2.0% on the pipe.
    c. For the Cell 2 liner, the longest flow path is 190 feet and the slope will be 2.8% based on 2.0% slope toward the collection pipe, and a slope of 2.0% along the pipe.

14. A 1-acre design area was used for modeling purposes to compute unit quantities.

15. Assume each subcell of Phase II of Kekaha Landfill is 2.3 acres (Reference 1). The entire Phase II footprint is 32 acres. The Cell 1 area is 6.4 acres.

16. Intermediate Condition Inputs (see also Tables 1 and 2 in this calculation):

- For the Phase II area: 68 feet (estimated) of waste and daily cover, based on waste height as of March 6, 2015 aerial topography. Maximum top of waste and intermediate cover grades vary from approximately 72 to 84 feet in elevation and the base grades vary from approximate elevations 8 to 12 with a 2-foot drainage layer.
- For the Cell 1 area: 51 feet (estimated) of waste and daily cover, based on waste height as of March 6, 2015 aerial topography. Maximum top of waste and daily cover grades vary from approximately 70 to 77 feet in elevation and the base grades vary from approximate elevations 11 to 17 with a 2-foot operations layer and a 1-foot drainage layer.
- For the Cell 2 area, 105 feet of waste and daily cover based on full height of waste.
- Bare ground conditions
- Evaporative zone depth = 10 in. (recommended by HELP Model for bare ground for Honolulu, Hawaii)
- Maximum leaf area index = 0 (recommended by HELP Model for bare ground)
- Fraction of area allowing runoff = 100 percent for the Phase II area, and 92.8 percent for the Cell 1 area.
- Runoff curve number:
  Phase II = 95.1 (HELP model calculated, Soil No. 12, 3% slope, 320-foot slope length)
  Cell 1 = 95.4 (HELP model calculated, Soil No. 12, 23% slope, 220-foot slope length)
  Cell 2 = 95.50 (HELP model calculated, Soil No. 12, 28% slope, 150-foot slope length).
- Length of model run = 5 years (solar radiation data was synthetically generated for 5 years)

Help Model General Layout

Table 1: Phase II HELP Model Layout

(Layer Number) Layer Description	Flow	Thickness	Saturated Hydraulic Conductivity	Soil Type/ Texture Number
(1)vertical percolation	↓	12 inches	4.2×10^{-5} cm/sec	Intermediate cover/#12
(2) vertical percolation	↓	84 inches	1.0×10^{-3} cm/sec	waste/#18, Init. Moisture = 90% of field capacity (0.2628)
(3) vertical percolation	↓	12 inches	4.2×10^{-5} cm/sec	Daily cover/#12
(4) vertical percolation	↓	108 inches	1.0×10^{-3} cm/sec	waste/#18, Init. Moisture = 98% of field capacity (0.2862)
(5)vertical percolation	↓	12 inches	4.2×10^{-5} cm/sec	Daily cover/#12
(6)vertical percolation	↓	108 inches	1.0×10^{-3} cm/sec	waste/#18, Init. Moisture = 100% of field capacity (0.2920)
(7)vertical percolation	↓	12 inches	4.2×10^{-5} cm/sec	Daily cover/#12
(8)vertical percolation	↓	108 inches	1.0×10^{-3} cm/sec	waste/#18, Init. Moisture = 100% of field capacity (0.292)
(9)vertical percolation	↓	12 inches	4.2×10^{-5} cm/sec	Daily cover/#12
(10)vertical percolation	↓	108 inches	1.0×10^{-3} cm/sec	waste/#18, Init. Moisture = 100% of field capacity (0.2920)
(11)vertical percolation	↓	12 inches	4.2×10^{-5} cm/sec	Daily cover/#12
(12)vertical percolation	↓	108 inches	1.0×10^{-3} cm/sec	waste/#18, Init. Moisture = 100% of field capacity (0.2920)
(13)vertical percolation	↓	12 inches	4.2×10^{-5} cm/sec	Daily cover/#12
(14)vertical percolation	↓	108 inches	1.0×10^{-3} cm/sec	waste/#18, Init. Moisture = 100% of field capacity (0.2920)
(15)lateral drainage	→	24 inches	6.0×10^{-3} cm/sec	sand drainage layer/ #61
(16)60-mil geomembrane Liner	↓	0.06 inch	2.0×10^{-13} cm/sec	geomembrane/#35
(17)Geosynthetic Clay Layer	↓	0.24 inches	3.0×10^{-9} cm/sec	Bentonite mat/#17

Table 2: Cell 1 HELP Model Layout

(Layer Number) Layer Description	Flow	Thickness	Saturated Hydraulic Conductivity	Soil Type/ Texture Number
(1)vertical percolation	↓	12 inches	4.2×10^{-5} cm/sec	Intermediate cover/#12
(2)vertical percolation	↓	12 inches	1.0×10^{-3} cm/sec	waste/#18, Init. Moisture = 88% of field capacity (0.2570)
(3)vertical percolation	↓	12 inches	4.2×10^{-5} cm/sec	Daily cover/#12
(4)vertical percolation	↓	108 inches	1.0×10^{-3} cm/sec	waste/#18, Init. Moisture = 92% of field capacity (0.2686)
(5)vertical percolation	↓	12 inches	4.2×10^{-5} cm/sec	Daily cover/#12
(6) vertical percolation	↓	108 inches	1.0×10^{-3} cm/sec	waste/#18, Init. Moisture = 98% of field capacity (0.2862)
(7) vertical percolation	↓	12 inches	4.2×10^{-5} cm/sec	Daily cover/#12
(8) vertical percolation	↓	108 inches	1.0×10^{-3} cm/sec	waste/#18, Init. Moisture = 100% of field capacity (0.2920)
(9) vertical percolation	↓	12 inches	4.2×10^{-5} cm/sec	Daily cover/#12
(10) vertical percolation	↓	108 inches	1.0×10^{-3} cm/sec	waste/#18, Init. Moisture = 100% of field capacity (0.2920)
(11) vertical percolation	↓	12 inches	4.2×10^{-5} cm/sec	Daily cover/#12
(12) vertical percolation	↓	108 inches	1.0×10^{-3} cm/sec	waste/#18, Init. Moisture = 100% of field capacity (0.2920)
(13)vertical percolation	↓	24 inches	6.0×10^{-3} cm/sec	operations layer/ #61
(14)lateral drainage	→	12 inches	3.0×10^{-1} cm/sec	gravel/ #21
(15)60-mil geomembrane Liner	↓	0.06 inch	2.0×10^{-13} cm/sec	geomembrane/#35
(16)Geosynthetic Clay Layer	↓	0.24 inches	3.0×10^{-9} cm/sec	Bentonite mat/#17

Table 3: Active Condition – Cell 2 HELP Model Layout

(Layer Number) Layer Description	Flow	Thickness	Saturated Hydraulic Conductivity	Soil Type/ Texture Number
(1) vertical percolation	↓	12 inches	4.2×10^{-5} cm/sec	Intermediate cover/#12
(2) vertical percolation	↓	1,248 inches	1.0×10^{-3} cm/sec	waste/#18
(3) vertical percolation	↓	24 inches	6.0×10^{-3} cm/sec	Operations layer/#61
(4) lateral drainage	→	12 inches	3.0×10^{-1} cm/sec	Gravel/#21
(5) 60-mil geomembrane liner	↓	0.06 inch	2.0×10^{-13} cm/sec	Geomembrane/#35
(6) geosynthetic clay layer	↓	0.24 inches	5.0×10^{-9} cm/sec	Bentonite mat/#62

Table 4: Soil Texture Properties

Soil Texture	Soil Classification		Comments, Properties, and Uses
	USDA	USCS	
61	FS	SW	Operations layer for Cell 1, sand drainage layer for Phase II. Based on HELP model default soil no. 2 with the hydraulic conductivity changed from 5.8×10^{-3} cm/sec to 6×10^{-3} cm/sec.
35	---	---	HELP model default parameters for High Density Polyethylene (HDPE) geomembrane.
21	---	---	Gravel drainage layer. HELP model default soil parameter for gravel, hydraulic conductivity of 3.0×10^{-1} cm/sec
18	---	---	HELP model default parameter for municipal solid waste.
17	---	---	HELP model default parameter for a bentonite mat (0.6 cm/ 0.24 in. thick).
12	SiCL	CL	Used for intermediate cover. Help model default parameters.

Note: All soil properties are defaults of the HELP Model, Version 3.07 unless otherwise designated.

Calculations

Calculations performed by the HELP Model are included in the output located in the following pages. See Table 5 for the modeling results based on average annual leakage values for each area. The HELP Model presents the average annual leakage rates in cubic feet per acre per year.

Table 5: Intermediate Condition Calculated Results From HELP Analysis Output

Parameter	Phase II Average Annual Value	Cell 1 Average Annual Value	Cell 2 Average Annual Value
Leachate Leakage Rate (ft^3/acre-yr)	0.184 inches	0.005 inches	0.024 inches
Leachate Leakage (ft^3/yr)	0.031 inches	5.89 inches	0.158 inches

NOTE: Refer to the attached HELP model output (HELP model output file names "KEKPhII.out", "KEK14A.out", "KEK14B.out" for Cell 1 and Cell 2, respectively).

References

1. "Engineering Report, Kekaha Landfill Phase II, Kekaha, Kauai, Hawaii," prepared by Harding Lawson Associates, dated August 5, 1993.
2. "Engineering Report, Kekaha Landfill Phase II, Second Vertical Expansion to 120' MSL, Kekaha, Kauai, Hawaii," prepared by Earth Tech, dated August 2013.
3. The previous HELP model calculation completed in June 2015.

Attachment 1 Sub-Attachments

1-A	Phase II HELP Model Output File, KEK14PhII.OUT
1-B	Cell 1 HELP Model Output File, KEK14A.OUT
1-C	Cell 2 HELP Model Output File, KEK14B.OUT

Attachment 1-A

Phase II HELP Model Output File, KEK14PhII.OUT

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**                                                                  **
**              HYDROLOGIC EVALUATION OF LANDFILL PERFORMANCE        **
**                HELP MODEL VERSION 3.07  (1 NOVEMBER 1997)        **
**                  DEVELOPED BY ENVIRONMENTAL LABORATORY           **
**                    USAE WATERWAYS EXPERIMENT STATION             **
**              FOR USEPA RISK REDUCTION ENGINEERING LABORATORY      **
**                                                                  **
**                                                                  **
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PRECIPITATION DATA FILE:    C:\KEKAHA14\KEK14.D4
TEMPERATURE DATA FILE:      C:\KEKAHA14\KEK14.D7
SOLAR RADIATION DATA FILE:  C:\KEKAHA14\KEK14.D13
EVAPOTRANSPIRATION DATA:    C:\KEKAHA14\KEK14.D11
SOIL AND DESIGN DATA FILE:  C:\KEKAHA14\KEK14B.D10
OUTPUT DATA FILE:           C:\KEKAHA14\KEKPhII.OUT

TIME:  11: 4     DATE:  12/ 8/2015

**********************************************************************

     TITLE:  Kekaha Landfill Phase II- Intmd. Cond. 5 yr run, w/2014 Data

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     NOTE:  INITIAL MOISTURE CONTENT OF THE LAYERS AND SNOW WATER
              WERE SPECIFIED BY THE USER.

                                LAYER  1
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                  TYPE 1 - VERTICAL PERCOLATION LAYER
                      MATERIAL TEXTURE NUMBER  12
          THICKNESS                    =     12.00   INCHES
          POROSITY                     =      0.4710 VOL/VOL
          FIELD CAPACITY               =      0.3420 VOL/VOL
          WILTING POINT                =      0.2100 VOL/VOL
          INITIAL SOIL WATER CONTENT   =      0.3420 VOL/VOL
          EFFECTIVE SAT. HYD. COND.    =  0.419999997000E-04 CM/SEC
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LAYER 2

TYPE 1 - VERTICAL PERCOLATION LAYER
MATERIAL TEXTURE NUMBER 18

THICKNESS	=	84.00	INCHES
POROSITY	=	0.6710	VOL/VOL
FIELD CAPACITY	=	0.2920	VOL/VOL
WILTING POINT	=	0.0770	VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.2628	VOL/VOL
EFFECTIVE SAT. HYD. COND.	=	0.100000005000E-02	CM/SEC

LAYER 3

TYPE 1 - VERTICAL PERCOLATION LAYER
MATERIAL TEXTURE NUMBER 12

THICKNESS	=	12.00	INCHES
POROSITY	=	0.4710	VOL/VOL
FIELD CAPACITY	=	0.3420	VOL/VOL
WILTING POINT	=	0.2100	VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.3420	VOL/VOL
EFFECTIVE SAT. HYD. COND.	=	0.419999997000E-04	CM/SEC

LAYER 4

TYPE 1 - VERTICAL PERCOLATION LAYER
MATERIAL TEXTURE NUMBER 18

THICKNESS	=	108.00	INCHES
POROSITY	=	0.6710	VOL/VOL
FIELD CAPACITY	=	0.2920	VOL/VOL
WILTING POINT	=	0.0770	VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.2862	VOL/VOL
EFFECTIVE SAT. HYD. COND.	=	0.100000005000E-02	CM/SEC

LAYER 5

TYPE 1 - VERTICAL PERCOLATION LAYER
MATERIAL TEXTURE NUMBER 12

THICKNESS	=	12.00	INCHES

POROSITY	=	0.4710 VOL/VOL
FIELD CAPACITY	=	0.3420 VOL/VOL
WILTING POINT	=	0.2100 VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.3420 VOL/VOL
EFFECTIVE SAT. HYD. COND.	=	0.419999997000E-04 CM/SEC

LAYER 6

TYPE 1 - VERTICAL PERCOLATION LAYER
MATERIAL TEXTURE NUMBER 18

THICKNESS	=	108.00 INCHES
POROSITY	=	0.6710 VOL/VOL
FIELD CAPACITY	=	0.2920 VOL/VOL
WILTING POINT	=	0.0770 VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.2920 VOL/VOL
EFFECTIVE SAT. HYD. COND.	=	0.100000005000E-02 CM/SEC

LAYER 7

TYPE 1 - VERTICAL PERCOLATION LAYER
MATERIAL TEXTURE NUMBER 12

THICKNESS	=	12.00 INCHES
POROSITY	=	0.4710 VOL/VOL
FIELD CAPACITY	=	0.3420 VOL/VOL
WILTING POINT	=	0.2100 VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.3420 VOL/VOL
EFFECTIVE SAT. HYD. COND.	=	0.419999997000E-04 CM/SEC

LAYER 8

TYPE 1 - VERTICAL PERCOLATION LAYER
MATERIAL TEXTURE NUMBER 18

THICKNESS	=	108.00 INCHES
POROSITY	=	0.6710 VOL/VOL
FIELD CAPACITY	=	0.2920 VOL/VOL
WILTING POINT	=	0.0770 VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.2920 VOL/VOL
EFFECTIVE SAT. HYD. COND.	=	0.100000005000E-02 CM/SEC

LAYER 9

TYPE 1 - VERTICAL PERCOLATION LAYER
MATERIAL TEXTURE NUMBER 12

THICKNESS	=	12.00	INCHES
POROSITY	=	0.4710	VOL/VOL
FIELD CAPACITY	=	0.3420	VOL/VOL
WILTING POINT	=	0.2100	VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.3420	VOL/VOL
EFFECTIVE SAT. HYD. COND.	=	0.419999997000E-04	CM/SEC

LAYER 10

TYPE 1 - VERTICAL PERCOLATION LAYER
MATERIAL TEXTURE NUMBER 18

THICKNESS	=	108.00	INCHES
POROSITY	=	0.6710	VOL/VOL
FIELD CAPACITY	=	0.2920	VOL/VOL
WILTING POINT	=	0.0770	VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.2920	VOL/VOL
EFFECTIVE SAT. HYD. COND.	=	0.100000005000E-02	CM/SEC

LAYER 11

TYPE 1 - VERTICAL PERCOLATION LAYER
MATERIAL TEXTURE NUMBER 12

THICKNESS	=	12.00	INCHES
POROSITY	=	0.4710	VOL/VOL
FIELD CAPACITY	=	0.3420	VOL/VOL
WILTING POINT	=	0.2100	VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.3420	VOL/VOL
EFFECTIVE SAT. HYD. COND.	=	0.419999997000E-04	CM/SEC

LAYER 12

TYPE 1 - VERTICAL PERCOLATION LAYER
MATERIAL TEXTURE NUMBER 18

THICKNESS	=	108.00	INCHES
POROSITY	=	0.6710	VOL/VOL
FIELD CAPACITY	=	0.2920	VOL/VOL
WILTING POINT	=	0.0770	VOL/VOL

INITIAL SOIL WATER CONTENT	=	0.2920	VOL/VOL
EFFECTIVE SAT. HYD. COND.	=	0.100000005000E-02	CM/SEC

LAYER 13

TYPE 1 - VERTICAL PERCOLATION LAYER
MATERIAL TEXTURE NUMBER 12

THICKNESS	=	12.00	INCHES
POROSITY	=	0.4710	VOL/VOL
FIELD CAPACITY	=	0.3420	VOL/VOL
WILTING POINT	=	0.2100	VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.3420	VOL/VOL
EFFECTIVE SAT. HYD. COND.	=	0.419999997000E-04	CM/SEC

LAYER 14

TYPE 1 - VERTICAL PERCOLATION LAYER
MATERIAL TEXTURE NUMBER 18

THICKNESS	=	108.00	INCHES
POROSITY	=	0.6710	VOL/VOL
FIELD CAPACITY	=	0.2920	VOL/VOL
WILTING POINT	=	0.0770	VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.2920	VOL/VOL
EFFECTIVE SAT. HYD. COND.	=	0.100000005000E-02	CM/SEC

LAYER 15

TYPE 2 - LATERAL DRAINAGE LAYER
MATERIAL TEXTURE NUMBER 61

THICKNESS	=	24.00	INCHES
POROSITY	=	0.4370	VOL/VOL
FIELD CAPACITY	=	0.0620	VOL/VOL
WILTING POINT	=	0.0240	VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.0620	VOL/VOL
EFFECTIVE SAT. HYD. COND.	=	0.600000005000E-02	CM/SEC
SLOPE	=	0.70	PERCENT
DRAINAGE LENGTH	=	70.7	FEET

LAYER 16

TYPE 4 - FLEXIBLE MEMBRANE LINER
MATERIAL TEXTURE NUMBER 35

THICKNESS	=	0.06	INCHES
POROSITY	=	0.0000	VOL/VOL
FIELD CAPACITY	=	0.0000	VOL/VOL
WILTING POINT	=	0.0000	VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.0000	VOL/VOL
EFFECTIVE SAT. HYD. COND.	=	0.199999996000E-12	CM/SEC
FML PINHOLE DENSITY	=	1.00	HOLES/ACRE
FML INSTALLATION DEFECTS	=	2.00	HOLES/ACRE
FML PLACEMENT QUALITY	=	3 - GOOD	

LAYER 17

TYPE 3 - BARRIER SOIL LINER
MATERIAL TEXTURE NUMBER 17

THICKNESS	=	0.24	INCHES
POROSITY	=	0.7500	VOL/VOL
FIELD CAPACITY	=	0.7470	VOL/VOL
WILTING POINT	=	0.4000	VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.7500	VOL/VOL
EFFECTIVE SAT. HYD. COND.	=	0.300000003000E-08	CM/SEC

GENERAL DESIGN AND EVAPORATIVE ZONE DATA

NOTE: SCS RUNOFF CURVE NUMBER WAS COMPUTED FROM DEFAULT SOIL DATA BASE USING SOIL TEXTURE #12 WITH BARE GROUND CONDITIONS, A SURFACE SLOPE OF 3.% AND A SLOPE LENGTH OF 320. FEET.

SCS RUNOFF CURVE NUMBER	=	95.10	
FRACTION OF AREA ALLOWING RUNOFF	=	100.0	PERCENT
AREA PROJECTED ON HORIZONTAL PLANE	=	1.000	ACRES
EVAPORATIVE ZONE DEPTH	=	10.0	INCHES
INITIAL WATER IN EVAPORATIVE ZONE	=	3.420	INCHES
UPPER LIMIT OF EVAPORATIVE STORAGE	=	4.710	INCHES
LOWER LIMIT OF EVAPORATIVE STORAGE	=	2.100	INCHES
INITIAL SNOW WATER	=	0.000	INCHES
INITIAL WATER IN LAYER MATERIALS	=	241.061	INCHES
TOTAL INITIAL WATER	=	241.061	INCHES
TOTAL SUBSURFACE INFLOW	=	0.00	INCHES/YEAR

EVAPOTRANSPIRATION AND WEATHER DATA

NOTE: EVAPOTRANSPIRATION DATA WAS OBTAINED FROM
HONOLULU HAWAII

STATION LATITUDE	=	21.33	DEGREES
MAXIMUM LEAF AREA INDEX	=	0.00	
START OF GROWING SEASON (JULIAN DATE)	=	0	
END OF GROWING SEASON (JULIAN DATE)	=	367	
EVAPORATIVE ZONE DEPTH	=	10.0	INCHES
AVERAGE ANNUAL WIND SPEED	=	2.95	MPH
AVERAGE 1ST QUARTER RELATIVE HUMIDITY	=	72.00	%
AVERAGE 2ND QUARTER RELATIVE HUMIDITY	=	66.00	%
AVERAGE 3RD QUARTER RELATIVE HUMIDITY	=	66.00	%
AVERAGE 4TH QUARTER RELATIVE HUMIDITY	=	70.00	%

NOTE: PRECIPITATION DATA FOR KEKAHA/SITE HAWAII
WAS ENTERED FROM AN ASCII DATA FILE.

NOTE: TEMPERATURE DATA FOR KEKAHA/SITE HAWAII
WAS ENTERED FROM AN ASCII DATA FILE.

NOTE: SOLAR RADIATION DATA WAS SYNTHETICALLY GENERATED USING
COEFFICIENTS FOR HONOLULU HAWAII
AND STATION LATITUDE = 21.33 DEGREES

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AVERAGE MONTHLY VALUES IN INCHES FOR YEARS 2010 THROUGH 2014

	JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
PRECIPITATION						
TOTALS	2.54	3.13	1.32	1.34	2.11	0.65
	0.19	0.21	0.54	1.41	0.87	4.38
STD. DEVIATIONS	0.93	2.58	1.11	1.25	1.70	0.52
	0.15	0.09	0.39	1.22	0.64	4.68

RUNOFF

TOTALS	0.558	1.154	0.363	0.155	0.687	0.065
	0.001	0.000	0.050	0.239	0.206	2.381
STD. DEVIATIONS	0.461	1.252	0.651	0.185	0.836	0.120
	0.001	0.000	0.107	0.357	0.198	2.924

EVAPOTRANSPIRATION

TOTALS	1.966	1.883	1.160	0.825	1.233	0.617
	0.348	0.249	0.388	0.780	0.466	1.572
STD. DEVIATIONS	0.967	1.098	0.880	0.937	1.300	0.316
	0.080	0.077	0.479	0.669	0.398	1.305

LATERAL DRAINAGE COLLECTED FROM LAYER 15

TOTALS	0.0206	0.0172	0.0235	0.0523	0.0835	0.0774
	0.0796	0.0722	0.0606	0.0532	0.0426	0.0379
STD. DEVIATIONS	0.0290	0.0237	0.0321	0.0726	0.1278	0.1100
	0.0877	0.0721	0.0584	0.0504	0.0398	0.0355

PERCOLATION/LEAKAGE THROUGH LAYER 17

TOTALS	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
STD. DEVIATIONS	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000

AVERAGES OF MONTHLY AVERAGED DAILY HEADS (INCHES)

DAILY AVERAGE HEAD ON TOP OF LAYER 16

AVERAGES	0.1975	0.1820	0.2249	0.5059	0.7548	0.7374
	0.7503	0.6889	0.5992	0.5095	0.4212	0.3633
STD. DEVIATIONS	0.2774	0.2510	0.3079	0.6997	1.1347	1.0343
	0.8184	0.6863	0.5778	0.4827	0.3937	0.3405

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AVERAGE ANNUAL TOTALS & (STD. DEVIATIONS) FOR YEARS 2010 THROUGH 2014

****

PEAK DAILY VALUES FOR YEARS 2010 THROUGH 2014

	(INCHES)	(CU. FT.)
PRECIPITATION	5.27	19130.100
RUNOFF	4.237	15379.3652
DRAINAGE COLLECTED FROM LAYER 15	0.00967	35.09277
PERCOLATION/LEAKAGE THROUGH LAYER 17	0.000001	0.00297
AVERAGE HEAD ON TOP OF LAYER 16	2.625	
MAXIMUM HEAD ON TOP OF LAYER 16	3.164	
LOCATION OF MAXIMUM HEAD IN LAYER 15 (DISTANCE FROM DRAIN)	31.7 FEET	
SNOW WATER	0.00	0.0000

MAXIMUM VEG. SOIL WATER (VOL/VOL)	0.3923
MINIMUM VEG. SOIL WATER (VOL/VOL)	0.2100

*** Maximum heads are computed using McEnroe's equations. ***

Reference: Maximum Saturated Depth over Landfill Liner
by Bruce M. McEnroe, University of Kansas
ASCE Journal of Environmental Engineering
Vol. 119, No. 2, March 1993, pp. 262-270.

****

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FINAL WATER STORAGE AT END OF YEAR 2014

LAYER	(INCHES)	(VOL/VOL)
1	3.5376	0.2948
2	24.5280	0.2920
3	4.1040	0.3420
4	31.5360	0.2920
5	4.1040	0.3420
6	31.5360	0.2920
7	4.1040	0.3420
8	31.5360	0.2920
9	4.1040	0.3420
10	31.5360	0.2920
11	4.1700	0.3475
12	31.5360	0.2920
13	4.3200	0.3600
14	31.5360	0.2920
15	2.2707	0.0946
16	0.0000	0.0000
17	0.1800	0.7500
SNOW WATER	0.000	

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Attachment 1-B

Cell 1 HELP Model Output File, KEK14A.OUT

```
**********************************************************************
**********************************************************************
**                                                                  **
**                                                                  **
**              HYDROLOGIC EVALUATION OF LANDFILL PERFORMANCE        **
**                HELP MODEL VERSION 3.07  (1 NOVEMBER 1997)        **
**                DEVELOPED BY ENVIRONMENTAL LABORATORY             **
**                USAE WATERWAYS EXPERIMENT STATION                 **
**             FOR USEPA RISK REDUCTION ENGINEERING LABORATORY       **
**                                                                  **
**                                                                  **
**********************************************************************
**********************************************************************

PRECIPITATION DATA FILE:     C:\KEKAHA14\KEK14.D4
TEMPERATURE DATA FILE:       C:\KEKAHA14\KEK14.D7
SOLAR RADIATION DATA FILE:   C:\KEKAHA14\KEK14.D13
EVAPOTRANSPIRATION DATA:     C:\KEKAHA14\KEK14.D11
SOIL AND DESIGN DATA FILE:   C:\KEKAHA14\KEK14A.D10
OUTPUT DATA FILE:            C:\KEKAHA14\KEK14A.OUT

TIME:   9:49     DATE:   5/26/2015

**********************************************************************

     TITLE:  Kekaha Landfill Cell 1- Intmd. Cond. 5 yr run, w/2014 Data

**********************************************************************

     NOTE:  INITIAL MOISTURE CONTENT OF THE LAYERS AND SNOW WATER
              WERE SPECIFIED BY THE USER.


                                LAYER  1
                                --------

                  TYPE 1 - VERTICAL PERCOLATION LAYER
                      MATERIAL TEXTURE NUMBER  12
          THICKNESS                    =     12.00   INCHES
          POROSITY                     =      0.4710 VOL/VOL
          FIELD CAPACITY               =      0.3420 VOL/VOL
          WILTING POINT                =      0.2100 VOL/VOL
          INITIAL SOIL WATER CONTENT   =      0.3420 VOL/VOL
          EFFECTIVE SAT. HYD. COND.    =  0.419999997000E-04 CM/SEC
```

LAYER 2

TYPE 1 - VERTICAL PERCOLATION LAYER
MATERIAL TEXTURE NUMBER 18

THICKNESS	=	12.00	INCHES
POROSITY	=	0.6710	VOL/VOL
FIELD CAPACITY	=	0.2920	VOL/VOL
WILTING POINT	=	0.0770	VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.2570	VOL/VOL
EFFECTIVE SAT. HYD. COND.	=	0.100000005000E-02	CM/SEC

LAYER 3

TYPE 1 - VERTICAL PERCOLATION LAYER
MATERIAL TEXTURE NUMBER 12

THICKNESS	=	12.00	INCHES
POROSITY	=	0.4710	VOL/VOL
FIELD CAPACITY	=	0.3420	VOL/VOL
WILTING POINT	=	0.2100	VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.3420	VOL/VOL
EFFECTIVE SAT. HYD. COND.	=	0.419999997000E-04	CM/SEC

LAYER 4

TYPE 1 - VERTICAL PERCOLATION LAYER
MATERIAL TEXTURE NUMBER 18

THICKNESS	=	108.00	INCHES
POROSITY	=	0.6710	VOL/VOL
FIELD CAPACITY	=	0.2920	VOL/VOL
WILTING POINT	=	0.0770	VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.2686	VOL/VOL
EFFECTIVE SAT. HYD. COND.	=	0.100000005000E-02	CM/SEC

LAYER 5

TYPE 1 - VERTICAL PERCOLATION LAYER
MATERIAL TEXTURE NUMBER 12

THICKNESS	=	12.00	INCHES
POROSITY	=	0.4710	VOL/VOL
FIELD CAPACITY	=	0.3420	VOL/VOL
WILTING POINT	=	0.2100	VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.3420	VOL/VOL
EFFECTIVE SAT. HYD. COND.	=	0.419999997000E-04	CM/SEC

LAYER 6

TYPE 1 - VERTICAL PERCOLATION LAYER
MATERIAL TEXTURE NUMBER 18

THICKNESS	=	108.00	INCHES
POROSITY	=	0.6710	VOL/VOL
FIELD CAPACITY	=	0.2920	VOL/VOL
WILTING POINT	=	0.0770	VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.2862	VOL/VOL
EFFECTIVE SAT. HYD. COND.	=	0.100000005000E-02	CM/SEC

LAYER 7

TYPE 1 - VERTICAL PERCOLATION LAYER
MATERIAL TEXTURE NUMBER 12

THICKNESS	=	12.00	INCHES
POROSITY	=	0.4710	VOL/VOL
FIELD CAPACITY	=	0.3420	VOL/VOL
WILTING POINT	=	0.2100	VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.3420	VOL/VOL
EFFECTIVE SAT. HYD. COND.	=	0.419999997000E-04	CM/SEC

LAYER 8

TYPE 1 - VERTICAL PERCOLATION LAYER
MATERIAL TEXTURE NUMBER 18

THICKNESS	=	108.00	INCHES
POROSITY	=	0.6710	VOL/VOL
FIELD CAPACITY	=	0.2920	VOL/VOL
WILTING POINT	=	0.0770	VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.2920	VOL/VOL
EFFECTIVE SAT. HYD. COND.	=	0.100000005000E-02	CM/SEC

LAYER 9

TYPE 1 - VERTICAL PERCOLATION LAYER
MATERIAL TEXTURE NUMBER 12

THICKNESS	=	12.00	INCHES
POROSITY	=	0.4710	VOL/VOL
FIELD CAPACITY	=	0.3420	VOL/VOL
WILTING POINT	=	0.2100	VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.3420	VOL/VOL
EFFECTIVE SAT. HYD. COND.	=	0.419999997000E-04	CM/SEC

LAYER 10

TYPE 1 - VERTICAL PERCOLATION LAYER
MATERIAL TEXTURE NUMBER 18

THICKNESS	=	108.00	INCHES
POROSITY	=	0.6710	VOL/VOL
FIELD CAPACITY	=	0.2920	VOL/VOL
WILTING POINT	=	0.0770	VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.2920	VOL/VOL
EFFECTIVE SAT. HYD. COND.	=	0.100000005000E-02	CM/SEC

LAYER 11

TYPE 1 - VERTICAL PERCOLATION LAYER
MATERIAL TEXTURE NUMBER 12

THICKNESS	=	12.00	INCHES
POROSITY	=	0.4710	VOL/VOL
FIELD CAPACITY	=	0.3420	VOL/VOL
WILTING POINT	=	0.2100	VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.3420	VOL/VOL
EFFECTIVE SAT. HYD. COND.	=	0.419999997000E-04	CM/SEC

LAYER 12

TYPE 1 - VERTICAL PERCOLATION LAYER
MATERIAL TEXTURE NUMBER 18

THICKNESS	=	108.00	INCHES
POROSITY	=	0.6710	VOL/VOL
FIELD CAPACITY	=	0.2920	VOL/VOL
WILTING POINT	=	0.0770	VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.2920	VOL/VOL
EFFECTIVE SAT. HYD. COND.	=	0.100000005000E-02	CM/SEC

LAYER 13

TYPE 1 - VERTICAL PERCOLATION LAYER
MATERIAL TEXTURE NUMBER 61

THICKNESS	=	24.00	INCHES
POROSITY	=	0.4370	VOL/VOL
FIELD CAPACITY	=	0.0620	VOL/VOL
WILTING POINT	=	0.0240	VOL/VOL

INITIAL SOIL WATER CONTENT	=	0.0620	VOL/VOL
EFFECTIVE SAT. HYD. COND.	=	0.600000005000E-02	CM/SEC

LAYER 14

TYPE 2 - LATERAL DRAINAGE LAYER
MATERIAL TEXTURE NUMBER 21

THICKNESS	=	12.00	INCHES
POROSITY	=	0.3970	VOL/VOL
FIELD CAPACITY	=	0.0320	VOL/VOL
WILTING POINT	=	0.0130	VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.0320	VOL/VOL
EFFECTIVE SAT. HYD. COND.	=	0.300000012000	CM/SEC
SLOPE	=	2.80	PERCENT
DRAINAGE LENGTH	=	240.0	FEET

LAYER 15

TYPE 4 - FLEXIBLE MEMBRANE LINER
MATERIAL TEXTURE NUMBER 35

THICKNESS	=	0.06	INCHES
POROSITY	=	0.0000	VOL/VOL
FIELD CAPACITY	=	0.0000	VOL/VOL
WILTING POINT	=	0.0000	VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.0000	VOL/VOL
EFFECTIVE SAT. HYD. COND.	=	0.199999996000E-12	CM/SEC
FML PINHOLE DENSITY	=	1.00	HOLES/ACRE
FML INSTALLATION DEFECTS	=	2.00	HOLES/ACRE
FML PLACEMENT QUALITY	=	3 - GOOD	

LAYER 16

TYPE 3 - BARRIER SOIL LINER
MATERIAL TEXTURE NUMBER 17

THICKNESS	=	0.24	INCHES
POROSITY	=	0.7500	VOL/VOL
FIELD CAPACITY	=	0.7470	VOL/VOL
WILTING POINT	=	0.4000	VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.7500	VOL/VOL
EFFECTIVE SAT. HYD. COND.	=	0.300000003000E-08	CM/SEC

GENERAL DESIGN AND EVAPORATIVE ZONE DATA

NOTE: SCS RUNOFF CURVE NUMBER WAS COMPUTED FROM DEFAULT SOIL DATA BASE USING SOIL TEXTURE #12 WITH BARE GROUND CONDITIONS, A SURFACE SLOPE OF 23.% AND A SLOPE LENGTH OF 220. FEET.

SCS RUNOFF CURVE NUMBER	=	95.40	
FRACTION OF AREA ALLOWING RUNOFF	=	100.0	PERCENT
AREA PROJECTED ON HORIZONTAL PLANE	=	1.000	ACRES
EVAPORATIVE ZONE DEPTH	=	10.0	INCHES
INITIAL WATER IN EVAPORATIVE ZONE	=	3.420	INCHES
UPPER LIMIT OF EVAPORATIVE STORAGE	=	4.710	INCHES
LOWER LIMIT OF EVAPORATIVE STORAGE	=	2.100	INCHES
INITIAL SNOW WATER	=	0.000	INCHES
INITIAL WATER IN LAYER MATERIALS	=	184.286	INCHES
TOTAL INITIAL WATER	=	184.286	INCHES
TOTAL SUBSURFACE INFLOW	=	0.00	INCHES/YEAR

EVAPOTRANSPIRATION AND WEATHER DATA

NOTE: EVAPOTRANSPIRATION DATA WAS OBTAINED FROM HONOLULU HAWAII

STATION LATITUDE	=	21.33	DEGREES
MAXIMUM LEAF AREA INDEX	=	0.00	
START OF GROWING SEASON (JULIAN DATE)	=	0	
END OF GROWING SEASON (JULIAN DATE)	=	367	
EVAPORATIVE ZONE DEPTH	=	10.0	INCHES
AVERAGE ANNUAL WIND SPEED	=	2.95	MPH
AVERAGE 1ST QUARTER RELATIVE HUMIDITY	=	72.00	%
AVERAGE 2ND QUARTER RELATIVE HUMIDITY	=	66.00	%
AVERAGE 3RD QUARTER RELATIVE HUMIDITY	=	66.00	%
AVERAGE 4TH QUARTER RELATIVE HUMIDITY	=	70.00	%

NOTE: PRECIPITATION DATA FOR KEKAHA/SITE HAWAII WAS ENTERED FROM AN ASCII DATA FILE.

NOTE: TEMPERATURE DATA FOR KEKAHA/SITE HAWAII WAS ENTERED FROM AN ASCII DATA FILE.

NOTE: SOLAR RADIATION DATA WAS SYNTHETICALLY GENERATED USING COEFFICIENTS FOR HONOLULU HAWAII AND STATION LATITUDE = 21.33 DEGREES

SOIL WATER AT END OF YEAR	187.996	682427.125	
SNOW WATER AT START OF YEAR	0.000	0.000	0.00
SNOW WATER AT END OF YEAR	0.000	0.000	0.00
ANNUAL WATER BUDGET BALANCE	0.0000	0.083	0.00

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MONTHLY TOTALS (IN INCHES) FOR YEAR 2014

	JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
PRECIPITATION	2.52	5.70	0.53	0.03	3.87	1.21
	0.09	0.08	0.58	3.02	0.11	1.01
RUNOFF	0.447	2.193	0.012	0.000	2.032	0.298
	0.000	0.000	0.009	0.890	0.000	0.127
EVAPOTRANSPIRATION	1.597	3.145	1.080	0.030	1.027	1.016
	0.453	0.280	0.085	1.547	0.631	0.246
LATERAL DRAINAGE COLLECTED FROM LAYER 14	0.0221	0.0127	0.0627	0.1358	0.0984	0.2031
	0.1088	0.1124	0.0818	0.0769	0.0575	0.0569
PERCOLATION/LEAKAGE THROUGH LAYER 16	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000

MONTHLY SUMMARIES FOR DAILY HEADS (INCHES)

AVERAGE DAILY HEAD ON TOP OF LAYER 15	0.004	0.002	0.010	0.023	0.016	0.034
	0.018	0.018	0.014	0.013	0.010	0.009
STD. DEVIATION OF DAILY HEAD ON TOP OF LAYER 15	0.003	0.003	0.008	0.006	0.017	0.007
	0.008	0.003	0.000	0.002	0.002	0.002

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ANNUAL TOTALS FOR YEAR 2014

	INCHES	CU. FEET	PERCENT
PRECIPITATION	18.75	68062.531	100.00
RUNOFF	6.009	21813.488	32.05
EVAPOTRANSPIRATION	11.136	40423.395	59.39
DRAINAGE COLLECTED FROM LAYER 14	1.0294	3736.555	5.49
PERC./LEAKAGE THROUGH LAYER 16	0.000003	0.012	0.00
AVG. HEAD ON TOP OF LAYER 15	0.0142		
CHANGE IN WATER STORAGE	0.576	2089.067	3.07
SOIL WATER AT START OF YEAR	187.996	682427.125	
SOIL WATER AT END OF YEAR	188.572	684516.187	
SNOW WATER AT START OF YEAR	0.000	0.000	0.00
SNOW WATER AT END OF YEAR	0.000	0.000	0.00
ANNUAL WATER BUDGET BALANCE	0.0000	0.010	0.00

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AVERAGE MONTHLY VALUES IN INCHES FOR YEARS 2010 THROUGH 2014

	JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
PRECIPITATION						
TOTALS	2.54	3.13	1.32	1.34	2.11	0.65
	0.19	0.21	0.54	1.41	0.87	4.38
STD. DEVIATIONS	0.93	2.58	1.11	1.25	1.70	0.52
	0.15	0.09	0.39	1.22	0.64	4.68
RUNOFF						
TOTALS	0.626	1.189	0.382	0.178	0.723	0.074
	0.001	0.000	0.044	0.263	0.220	2.479
STD. DEVIATIONS	0.482	1.258	0.682	0.208	0.865	0.128
	0.002	0.000	0.093	0.378	0.211	3.003
EVAPOTRANSPIRATION						

TOTALS	1.838	1.950	1.134	0.836	1.175	0.623
	0.470	0.260	0.191	0.659	0.527	1.585
STD. DEVIATIONS	1.016	1.057	0.869	0.946	1.184	0.327
	0.230	0.094	0.066	0.591	0.464	1.269

LATERAL DRAINAGE COLLECTED FROM LAYER 14

TOTALS	0.0046	0.0027	0.0129	0.0882	0.0560	0.0658
	0.0405	0.0366	0.0272	0.0251	0.0197	0.0183
STD. DEVIATIONS	0.0098	0.0056	0.0278	0.1346	0.0822	0.0941
	0.0557	0.0522	0.0385	0.0358	0.0276	0.0260

PERCOLATION/LEAKAGE THROUGH LAYER 16

TOTALS	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
STD. DEVIATIONS	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000

AVERAGES OF MONTHLY AVERAGED DAILY HEADS (INCHES)

DAILY AVERAGE HEAD ON TOP OF LAYER 15

AVERAGES	0.0007	0.0005	0.0021	0.0148	0.0091	0.0111
	0.0066	0.0060	0.0046	0.0041	0.0033	0.0030
STD. DEVIATIONS	0.0016	0.0010	0.0045	0.0226	0.0134	0.0158
	0.0091	0.0085	0.0065	0.0058	0.0046	0.0042

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AVERAGE ANNUAL TOTALS & (STD. DEVIATIONS) FOR YEARS 2010 THROUGH 2014

	INCHES	CU. FEET	PERCENT
PRECIPITATION	18.68 (2.164)	67815.7	100.00
RUNOFF	6.180 (1.5184)	22431.75	33.078
EVAPOTRANSPIRATION	11.248 (1.9883)	40829.43	60.206
LATERAL DRAINAGE COLLECTED FROM LAYER 14	0.39755 (0.54450)	1443.097	2.12797
PERCOLATION/LEAKAGE THROUGH LAYER 16	0.00000 (0.00000)	0.005	0.00001

AVERAGE HEAD ON TOP OF LAYER 15	0.005	(0.008)		
CHANGE IN WATER STORAGE	0.857	(0.7254)	3111.37	4.588

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PEAK DAILY VALUES FOR YEARS 2010 THROUGH 2014

	(INCHES)	(CU. FT.)
PRECIPITATION	5.27	19130.100
RUNOFF	4.344	15768.2871
DRAINAGE COLLECTED FROM LAYER 14	0.01255	45.55249
PERCOLATION/LEAKAGE THROUGH LAYER 16	0.000000	0.00006
AVERAGE HEAD ON TOP OF LAYER 15	0.063	
MAXIMUM HEAD ON TOP OF LAYER 15	0.126	
LOCATION OF MAXIMUM HEAD IN LAYER 14 (DISTANCE FROM DRAIN)	1.3 FEET	
SNOW WATER	0.00	0.0000

MAXIMUM VEG. SOIL WATER (VOL/VOL)	0.3861
MINIMUM VEG. SOIL WATER (VOL/VOL)	0.2100

*** Maximum heads are computed using McEnroe's equations. ***

Reference: Maximum Saturated Depth over Landfill Liner
by Bruce M. McEnroe, University of Kansas
ASCE Journal of Environmental Engineering
Vol. 119, No. 2, March 1993, pp. 262-270.

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FINAL WATER STORAGE AT END OF YEAR 2014

LAYER	(INCHES)	(VOL/VOL)
1	3.5243	0.2937
2	3.5040	0.2920
3	4.1040	0.3420
4	31.5360	0.2920
5	4.1040	0.3420
6	31.5360	0.2920
7	4.1040	0.3420
8	31.5360	0.2920
9	4.1040	0.3420
10	31.5360	0.2920
11	4.1789	0.3482
12	31.5360	0.2920
13	2.7014	0.1126
14	0.3873	0.0323
15	0.0000	0.0000
16	0.1800	0.7500
SNOW WATER	0.000	

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Attachment 1-C

Cell 2 HELP Model Output File, KEK14B.OUT

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**********************************************************************
**********************************************************************
**                                                                  **
**                                                                  **
**              HYDROLOGIC EVALUATION OF LANDFILL PERFORMANCE        **
**                HELP MODEL VERSION 3.07  (1 NOVEMBER 1997)        **
**                DEVELOPED BY ENVIRONMENTAL LABORATORY             **
**                USAE WATERWAYS EXPERIMENT STATION                 **
**            FOR USEPA RISK REDUCTION ENGINEERING LABORATORY       **
**                                                                  **
**                                                                  **
**********************************************************************
**********************************************************************

PRECIPITATION DATA FILE:    C:\KEKAHA14\KEK14.D4
TEMPERATURE DATA FILE:      C:\KEKAHA14\KEK14.D7
SOLAR RADIATION DATA FILE:  C:\KEKAHA14\KEK14.D13
EVAPOTRANSPIRATION DATA:    C:\KEKAHA14\KEK14.D11
SOIL AND DESIGN DATA FILE:  C:\KEKAHA14\CELL2A.D10
OUTPUT DATA FILE:           C:\KEKAHA14\KEK14B.OUT

TIME:  10:33     DATE:  12/ 8/2015

**********************************************************************

     TITLE:  Kekaha Landfill Phase II- Cell 2, Active Condition

**********************************************************************

     NOTE:  INITIAL MOISTURE CONTENT OF THE LAYERS AND SNOW WATER WERE
            COMPUTED AS NEARLY STEADY-STATE VALUES BY THE PROGRAM.


                                LAYER  1
                                --------

                TYPE 1 - VERTICAL PERCOLATION LAYER
                    MATERIAL TEXTURE NUMBER  12
          THICKNESS                    =     12.00   INCHES
          POROSITY                     =      0.4710 VOL/VOL
          FIELD CAPACITY               =      0.3420 VOL/VOL
          WILTING POINT                =      0.2100 VOL/VOL
          INITIAL SOIL WATER CONTENT   =      0.3064 VOL/VOL
          EFFECTIVE SAT. HYD. COND.    =  0.419999997000E-04 CM/SEC
```

LAYER 2

TYPE 1 - VERTICAL PERCOLATION LAYER
MATERIAL TEXTURE NUMBER 18

THICKNESS	=	1248.00	INCHES
POROSITY	=	0.6710	VOL/VOL
FIELD CAPACITY	=	0.2920	VOL/VOL
WILTING POINT	=	0.0770	VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.2944	VOL/VOL
EFFECTIVE SAT. HYD. COND.	=	0.100000005000E-02	CM/SEC

LAYER 3

TYPE 1 - VERTICAL PERCOLATION LAYER
MATERIAL TEXTURE NUMBER 61

THICKNESS	=	24.00	INCHES
POROSITY	=	0.4370	VOL/VOL
FIELD CAPACITY	=	0.0620	VOL/VOL
WILTING POINT	=	0.0240	VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.1197	VOL/VOL
EFFECTIVE SAT. HYD. COND.	=	0.600000005000E-02	CM/SEC

LAYER 4

TYPE 2 - LATERAL DRAINAGE LAYER
MATERIAL TEXTURE NUMBER 21

THICKNESS	=	12.00	INCHES
POROSITY	=	0.3970	VOL/VOL
FIELD CAPACITY	=	0.0320	VOL/VOL
WILTING POINT	=	0.0130	VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.0320	VOL/VOL
EFFECTIVE SAT. HYD. COND.	=	0.300000012000	CM/SEC
SLOPE	=	2.80	PERCENT
DRAINAGE LENGTH	=	190.0	FEET

LAYER 5

TYPE 4 - FLEXIBLE MEMBRANE LINER
MATERIAL TEXTURE NUMBER 35

THICKNESS	=	0.06	INCHES
POROSITY	=	0.0000	VOL/VOL
FIELD CAPACITY	=	0.0000	VOL/VOL
WILTING POINT	=	0.0000	VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.0000	VOL/VOL
EFFECTIVE SAT. HYD. COND.	=	0.199999996000E-12	CM/SEC
FML PINHOLE DENSITY	=	1.00	HOLES/ACRE
FML INSTALLATION DEFECTS	=	2.00	HOLES/ACRE
FML PLACEMENT QUALITY	=	3 - GOOD	

LAYER 6

TYPE 3 - BARRIER SOIL LINER
MATERIAL TEXTURE NUMBER 62

THICKNESS	=	0.24	INCHES
POROSITY	=	0.7500	VOL/VOL
FIELD CAPACITY	=	0.7470	VOL/VOL
WILTING POINT	=	0.4000	VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.7500	VOL/VOL
EFFECTIVE SAT. HYD. COND.	=	0.499999997000E-08	CM/SEC

GENERAL DESIGN AND EVAPORATIVE ZONE DATA

NOTE: SCS RUNOFF CURVE NUMBER WAS COMPUTED FROM DEFAULT SOIL DATA BASE USING SOIL TEXTURE #12 WITH BARE GROUND CONDITIONS, A SURFACE SLOPE OF 28.% AND A SLOPE LENGTH OF 150. FEET.

SCS RUNOFF CURVE NUMBER	=	95.50	
FRACTION OF AREA ALLOWING RUNOFF	=	50.0	PERCENT
AREA PROJECTED ON HORIZONTAL PLANE	=	1.000	ACRES
EVAPORATIVE ZONE DEPTH	=	10.0	INCHES
INITIAL WATER IN EVAPORATIVE ZONE	=	2.957	INCHES
UPPER LIMIT OF EVAPORATIVE STORAGE	=	4.710	INCHES
LOWER LIMIT OF EVAPORATIVE STORAGE	=	2.100	INCHES
INITIAL SNOW WATER	=	0.000	INCHES
INITIAL WATER IN LAYER MATERIALS	=	374.486	INCHES
TOTAL INITIAL WATER	=	374.486	INCHES
TOTAL SUBSURFACE INFLOW	=	0.00	INCHES/YEAR

EVAPOTRANSPIRATION AND WEATHER DATA

NOTE: EVAPOTRANSPIRATION DATA WAS OBTAINED FROM
HONOLULU HAWAII

STATION LATITUDE	=	21.33	DEGREES
MAXIMUM LEAF AREA INDEX	=	0.00	
START OF GROWING SEASON (JULIAN DATE)	=	0	
END OF GROWING SEASON (JULIAN DATE)	=	367	
EVAPORATIVE ZONE DEPTH	=	10.0	INCHES
AVERAGE ANNUAL WIND SPEED	=	2.95	MPH
AVERAGE 1ST QUARTER RELATIVE HUMIDITY	=	72.00	%
AVERAGE 2ND QUARTER RELATIVE HUMIDITY	=	66.00	%
AVERAGE 3RD QUARTER RELATIVE HUMIDITY	=	66.00	%
AVERAGE 4TH QUARTER RELATIVE HUMIDITY	=	70.00	%

NOTE: PRECIPITATION DATA FOR KEKAHA/SITE HAWAII
WAS ENTERED FROM AN ASCII DATA FILE.

NOTE: TEMPERATURE DATA FOR KEKAHA/SITE HAWAII
WAS ENTERED FROM AN ASCII DATA FILE.

NOTE: SOLAR RADIATION DATA WAS SYNTHETICALLY GENERATED USING
COEFFICIENTS FOR HONOLULU HAWAII
AND STATION LATITUDE = 21.33 DEGREES

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AVERAGE MONTHLY VALUES IN INCHES FOR YEARS 2010 THROUGH 2014

	JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
PRECIPITATION						
TOTALS	2.54	3.13	1.32	1.34	2.11	0.65
	0.19	0.21	0.54	1.41	0.87	4.38
STD. DEVIATIONS	0.93	2.58	1.11	1.25	1.70	0.52
	0.15	0.09	0.39	1.22	0.64	4.68
RUNOFF						

TOTALS	0.306	0.661	0.222	0.092	0.433	0.034
	0.001	0.000	0.023	0.131	0.111	1.436
STD. DEVIATIONS	0.252	0.727	0.401	0.107	0.558	0.061
	0.001	0.000	0.048	0.185	0.106	1.867

EVAPOTRANSPIRATION

TOTALS	2.016	2.027	1.205	0.894	1.348	0.765
	0.367	0.186	0.206	0.667	0.672	1.794
STD. DEVIATIONS	0.885	1.048	0.932	1.019	1.226	0.546
	0.303	0.094	0.067	0.597	0.482	1.549

LATERAL DRAINAGE COLLECTED FROM LAYER 4

TOTALS	0.9350	0.9137	0.5027	0.3487	0.1414	0.2560
	0.1440	0.0959	0.0687	0.0586	0.1189	0.1495
STD. DEVIATIONS	0.5748	1.0993	0.3375	0.3082	0.0650	0.3328
	0.0736	0.0335	0.0183	0.0139	0.1833	0.1591

PERCOLATION/LEAKAGE THROUGH LAYER 6

TOTALS	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
STD. DEVIATIONS	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000

AVERAGES OF MONTHLY AVERAGED DAILY HEADS (INCHES)

DAILY AVERAGE HEAD ON TOP OF LAYER 5

AVERAGES	0.1204	0.1303	0.0647	0.0464	0.0182	0.0341
	0.0186	0.0124	0.0091	0.0075	0.0158	0.0193
STD. DEVIATIONS	0.0740	0.1568	0.0435	0.0410	0.0084	0.0443
	0.0095	0.0043	0.0024	0.0018	0.0244	0.0205

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AVERAGE ANNUAL TOTALS & (STD. DEVIATIONS) FOR YEARS 2010 THROUGH 2014

	INCHES	CU. FEET	PERCENT

PRECIPITATION	18.68 (2.164)	67815.7	100.00
RUNOFF	3.450 (1.0473)	12524.66	18.469
EVAPOTRANSPIRATION	12.146 (1.9020)	44091.46	65.017
LATERAL DRAINAGE COLLECTED FROM LAYER 4	3.73312 (1.96604)	13551.234	19.98245
PERCOLATION/LEAKAGE THROUGH LAYER 6	0.00001 (0.00000)	0.024	0.00004
AVERAGE HEAD ON TOP OF LAYER 5	0.041 (0.022)		
CHANGE IN WATER STORAGE	-0.648 (2.2698)	-2351.72	-3.468

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PEAK DAILY VALUES FOR YEARS 2010 THROUGH 2014

	(INCHES)	(CU. FT.)
PRECIPITATION	5.27	19130.100
RUNOFF	2.892	10497.7168
DRAINAGE COLLECTED FROM LAYER 4	0.13639	495.09766
PERCOLATION/LEAKAGE THROUGH LAYER 6	0.000000	0.00064
AVERAGE HEAD ON TOP OF LAYER 5	0.545	
MAXIMUM HEAD ON TOP OF LAYER 5	1.031	
LOCATION OF MAXIMUM HEAD IN LAYER 4 (DISTANCE FROM DRAIN)	10.0 FEET	
SNOW WATER	0.00	0.0000

MAXIMUM VEG. SOIL WATER (VOL/VOL)	0.4229
MINIMUM VEG. SOIL WATER (VOL/VOL)	0.2100

*** Maximum heads are computed using McEnroe's equations. ***

Reference: Maximum Saturated Depth over Landfill Liner
by Bruce M. McEnroe, University of Kansas
ASCE Journal of Environmental Engineering
Vol. 119, No. 2, March 1993, pp. 262-270.

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FINAL WATER STORAGE AT END OF YEAR 2014

LAYER	(INCHES)	(VOL/VOL)
1	3.5835	0.2986
2	364.4160	0.2920
3	2.6753	0.1115
4	0.3920	0.0327
5	0.0000	0.0000
6	0.1800	0.7500
SNOW WATER	0.000	

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Appendix G

Attachment 2

Cell 2 Sump Liner Leakage Analysis

Client County of Kaua'i Subject Cell 2 Sump Leakage Analysis Prepared By NKW Date 12/15/15
Project Kekaha Landfill Cell 2, Phase II Reviewed By DFP Date 12/16/15
Approved By NKW Date 12/16/15

CELL 2 SUMP LEACHATE LEAKAGE ANALYSIS

Objective

Estimate the quantity of leachate leakage through the composite liner in the Cell 2 sump areas.

Design Criteria and Assumptions

1. The base liner system for Cell 2 consists of the following components:
   - Subbase 60-mil HDPE geomembrane
   - GCL (lower component)
   - 60-mil HDPE geomembrane (upper component

2. To be conservative this analysis does not consider the subbase 60-mil HDPE geomembrane. Only the upper geomembrane and GCL are considered.

3. Leachate head of 16-inches is assumed for the sump. This leachate head level in the sump corresponds to 10 inches of head on the landfill floor. Conservatively use the head in the sump over the entire wetted area.

4. As discussed in Appendix D-6, Leachate Pump Sizing Cell 2 Sumps, the wetted area with up to 16 inches of head in the sump is 1,300 sf.

5. Consistent with the HELP model analysis of Cell 2, assume "good" geomembrane placement quality. A placement quality of "good" according to the HELP Model User's Guide "assumes good field installation with well prepared, smooth soil surface and geomembrane wrinkle control". The base liner evaluation as assumed 1 pinhole per acre for manufacturing defects and 2 defects per acre for installation defects. All defects are assumed to be circular in shape.

Calculations

Defect Area

As noted in the assumptions, the HELP Model liner evaluation assumed both manufactured and installation defects per acre of geomembrane. Manufactured defects are noted as pinholes and based upon the HELP model are 1 mm in diameter. The modeling assumed 1 pinhole per acre.

The installation defects in the HELP Model are modeled to be 1 cm^2 and in the completed liner evaluation 2 defects per acre were assumed. For this analysis, installation defects are assumed to be circular.

The assumed area of each sump is 1,300 sf, therefore, the entire area of the four sumps is 5,200 sf or 0.12 acres (1,300 sf x 4 sumps). This calculation conservatively assumes 16 inches of head over the entire area.

Method of Analysis

As discussed in Reference 2, flow or leakage through a geomembrane can occur as a result of defects and permeation through the geomembrane. For defects, the flow rate is much less for a composite liner (a geomembrane and a low permeable soil layer) than in instances where a geomembrane is installed over high permeability soils.

If there is a defect in the geomembrane, the leachate would flow through the defect and then laterally between the geomembrane and the low permeable soil for some distance and then into and through the low permeable soil. Installation quality is a main factor governing flow through a composite liner as well as the geomembrane and the low permeable soil must have good contact, wrinkles in the geomembrane must be minimized, and a smooth compacted subsurface. The size of the defect, leachate head on the geomembrane, and permeability of the soil underlying the geomembrane are factors also affecting the flow rate through a geomembrane.

Two empirical equations have been derived by analytical studies and model tests by Giroud and Bonaparte (1989) and Giroud (1989) for the rate of leakage through a geomembrane with a circular hole. These are:

For "good" contact conditions:

$$Q = 0.21\left(a^{0.1}\right)\left(h^{0.9}\right)\left(k^{0.74}\right)$$

For "poor" contact conditions:

$$Q = 1.15\left(a^{0.1}\right)\left(h^{0.9}\right)\left(k^{0.74}\right)$$

Where, for both equations:
Q = leakage rate through a geomembrane hole, m^3/s
a = area of circular hole in geomembrane, m^2
h = liquid head on top of geomembrane, m
k = permeability of the low permeability soil component of the composite liner, m/s

These equations are not dimensionally homogeneous and are to be used only in the units listed above.

Note "h" (liquid head) under hydrostatic conditions is equal to the liquid depth such as the case with liquid in and around the sump. If the liquid is unconfined and flowing, then "h" will need to be calculated based upon slope angle and depth of liquid.

Solving the equation for "good" contact (as construction quality assurance methods will be employed during installation of the composite liner) and the assumed defects per acre we obtain the following table for low permeable soils underlying the geomembrane of $5x10^{-9}$ cm/sec for a geosynthetic clay liner:

CALCULATION SHEET

AECOM

Project No. 60197394

Client County of Kaua'i Subject Cell 2 Sump Prepared By NKW Date 12/15/15

Project Kekaha Landfill Leakage Analysis Reviewed By DFP Date 12/16/15

Cell 2, Phase II Approved By NKW Date 12/16/15

Geomembrane Leakage Based Upon Number of Defects, Low Permeable Soils, And Good Placement

Manufacturing Defect - Pinhole									
Area, a		Head, h		Permeability, k		No. of Defects	Leakage, Q*		
(mm^2)	(m^2)	(in)	(m)	(cm/s)	(m/sec)	(per acre)	(m^3/s)	(cf/s)	(cf/day)
1	0.000001	16	0.4064	5.00E-09	5.00E-11	1	5.59E-10	1.97E-08	1.71E-03
Installation Defect									
Area, a		Head, h		Permeability, k		No. of Defects	Leakage, Q*		
(cm^2)	(m^2)	(in)	(m)	(cm/s)	(m/sec)	(per acre)	(m^3/s)	(cf/s)	(cf/day)
1	0.0001	16	0.4064	5.00E-09	5.00E-11	2	1.77E-09	6.26E-08	5.41E-03

Denotes cells used in calculation.

* Note: For installation defects, leakage, Q, is multiplied by the total number of defects so the total leakage is reported.

Conclusions

The above table depicts that assuming the presence of 1 manufacturing defect and 2 installation defects present in the geomembrane under the wetted area of the base for 16-inches of leachate head, results in an estimated leakage of 0.00712 cf (1.71×10^{-3} cf + 5.41×10^{-3} cf) per day over an assumed area of 0.12 acres.

If the head was maintained for a year, the estimated leakage annually is estimated at 2.60 cf/yr over an assumed area of 0.12 acres.

The 16 inches of leachate head is expected to be only realized in the deepest portions of the sump areas, but is conservatively assumed for the entire wetted area.

References

1. Qian, Xuede; Koerner, Robert M.; and Gray, Donald H., "Geotechnical Aspects of Landfill Design and Construction", pages 122 to 124.