Prepared for:

Waste Management of Hawaii, Inc.
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ENGINEERING REPORT FOR LANDFILL EXPANSION

WAIMANALO GULCH LANDFILL
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APPENDICES

1 INTRODUCTION

1.1 Location

Waimanalo Gulch Sanitary Landfill (WGSL) is located approximately 15 miles northwest of the Honolulu International Airport on the west side of the island of Oahu, Hawaii (Figure 1). The landfill complex begins at the north side of Farrington Highway and continues approximately 0.75 miles northward (i.e., up canyon) into Waimanalo Gulch (Figure 2). WGSL is located in a region of Oahu that is relatively dry compared with the rest of the island due to the "rain-shadow" effect of the Waianae Mountain Range, east of the site. Long-term average annual rainfall is approximately 15 in. in the vicinity of the landfill

1.2 Background

WGSL was designed, permitted, and constructed in the late 1980s as a repository for waste originating on Oahu. The landfill is owned by the City and County of Honolulu and is operated by Waste Management of Hawaii, Inc. (WMH). The WGSL property is approximately 200 acres in size, with the landfill and support structures occupying 100.9 acres of this area. The landfill consists of two disposal units: a 16-acre ash monofill and a 62.9-acre municipal solid waste (MSW) landfill. .

The landfill has been designed to meet the provisions of Chapter 342H, *Hawaii Revised Statutes*, and Title 11, Chapter 58.1, *Hawaii Administrative Rules* that incorporate Title 40 (Protection of Environment), Part 258 (Criteria for Municipal Solid Waste Landfills) in the Code of Federal Regulations (CFR) (also known as Subtitle D).

The MSW received at the landfill consists of approximately 72% residential waste, 25% special waste, and 3% contaminated soil (Geosyntec, 2007a). These wastes are non-hazardous. The landfill and source transfer stations have Hazardous Waste Exclusion Programs (HWEP) that prevent unacceptable wastes from reaching the landfill.

1.3 Development Sequence

The MSW cells at WGSL have generally been developed sequentially, from MSW Cell 1 to MSW Cell 11, and then MSW Expansion Cells E1 to E4 (Figures 2 and 3). Waste placement in Cells 1 to 11 began in 1989 and continues in areas being brought to final grade. Cells E1 to E4 were constructed from 2003 to 2007. WMH projects that the currently permitted landfill will reach capacity in mid-2010.

Site development plans consisting of proposed base grades, final grades, liner designs, and leachate collection systems were prepared at various times by SEC Donohue, Inc. (1992), RUST Environment & Infrastructure (1994), and Geosyntec Consultants (2003, and 2007a).

Figure 2 shows the currently-permitted final fill plan for the site (Geosyntec, 2007a). The plan features waste fill slopes that are generally equal to or flatter than 3:1 (horizontal to vertical [H:V]); the earthfill slopes are 2H:1V or flatter, and the maximum elevation reached is 513 ft-mean sea level (msl) in the MSW area and 270 ft-msl in the ash area. Figure 2 shows the soil buttresses/toe berms required to maintain stability of the permitted grades. It also shows the proposed public drop off and landfill gas-to-energy facilities.

Figures 3 and 4 show the existing cells and base liner systems, respectively. The landfill currently has eight cells where ash is disposed (Ash Cells 1 through 8) and 17 cells where MSW is disposed (MSW Cells 1 through 3; MSW Cells 4A, 4B, and 4C; MSW Cells 5 through 11; and MSW Cells E1 through E4). .

Leachate from Cells 1 through 11 is collected in the leachate collection and removal system (LCRS) sump located in Cell 4B (Figure 3). This cell was constructed in 1992. Leachate from Cells E1 to E4 is collected in the LCRS sump in Cell E1 (Figure 3). This cell was constructed in 2003. Given the relatively dry climate at the site and the fairly steep bottom slopes of the landfill (5 to 10%), it is expected that the only areas of sustained leachate head in the landfill will be at, or in the near the vicinity of these two sumps.

1.4 Expansion Plan

Figure 5 shows the preferred expansion fill plan, which expands the landfill to the north in cells E5 through E11. The expansion (E5 through E11) adds approximately 36.9 acres to the overall currently permitted footprint for MSW disposal. Fill slopes are equal to or flatter than 3:1 (horizontal to vertical) and the maximum elevation reached is approximately 800 ft- msl.

The limits of each expansion cell (i.e., E5 through E11) shown on Figure 5 are approximate at this time; the actual cell limits will be developed based on waste flows and may be modified based on the actual waste stream[1] (i.e., ash versus MSW). If ash cells are added, the sump arrangement may be changed, if required by the HDOH to separate leachate from the ash and MSW cells. The overall expansion limit will not change.

The expansion area will be accessed using the existing access road that runs over the ash cells and along the west side of the currently permitted landfill. The access road over the filled areas E5 through E11 will be moved as operations progress and the road alignment adjusted accordingly. The access road will be paved with an all-weather surface such as crushed concrete, crushed asphalt or rock.

Surface water design of the west side drainage features was performed by GEI Consultants (GEI) for the preferred expansion; Geosyntec performed surface water design for flows originating from the landfill and for run on from areas adjacent to the east side of the landfill.

Figure 6 shows the alternative expansion plan. The main differences between the preferred expansion and alternative expansion are: (i) a new ash cell (AE-1) adjacent to existing ash cells 1 through 4; and (ii) a new access road that shifts the traffic from the western portion of the site to the eastern portion of the site. The alternative expansion plan would require a significant re-routing of current traffic flow patterns at the site and very specific timing for development of the ash cell. It would also potentially increase visual impacts and therefore, it a less preferred alternative at this time.

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[1] Depending on the ratio of MSW to ash received at the landfill, an ash cell may need to be constructed later in the northern portion of the Expansion area. A change to the operating permit will be submitted for approval by the HDOH.

This Expansion Design Report presents information for the development of future cells at the landfill. The Report discusses the following:

- The landfill base lining and LCRS meets the regulatory requirements of RCRA Subtitle D (40 CFR Part 258);
- The design meets state of practice slope stability criteria at final build-out conditions and is based on industry-accepted MSW, ash, and base and side slope liner interface shear strength properties;
- The design meets the vertical separation requirements for the overhead power lines over the site.
- Details on the final cover system for expansion cells[2]; and
- Details on the overall surface water control design for the site.

[2] If approved by the HDOH, the final cover system may include compacted ash in its configuration.

2 GEOLOGY

In 2000, Geosyntec contracted the services of Geolabs, Inc. (Geolabs) of Honolulu, Hawaii to perform geotechnical and geological investigations of the proposed expansion area at WGSL. Geolabs reviewed local geologic maps, performed a field walk and visually observed the cut slopes, performed a seismic refraction survey, and prepared a report (Geolabs, 2000). Appendix A contains the full report; a summary of findings is presented below.

The Island of Oahu, covering approximately 604 square miles of land area, was formed by the merging of basaltic lava flows from the Wai'anae and Ko'olau shield volcanoes. The Wai'anae Mountains contain the oldest basalt-rock formations on the island. The WGSL is located within one of a series of parallel trending gulches (Waimanalo Gulch) that drain from the upper reaches of the southwest portion of the Wai'anae Mountains downward toward the southwest facing coastline. Waimanalo Gulch is located on the arid portion of Leeward Oahu and contains an ephemeral stream, which remains dry except during rainfall events.

The subgrade at WGSL consists of alternating layers of relatively dense lava flow with more fractured and porous clinker seams. The layers dip gradually downward toward the coastline. Very little alluvial/colluvial soil deposits were observed. However, a 3- to 5-ft thick dark brown clayey soil deposit was noted at the ground surface along the top of the exposed hill slopes.

The regional groundwater level for this portion of the island is lower than the elevations of the project site. Groundwater seepage has not been observed in the exposed cuts. However, for the most recent expansion, Geolabs noted that some minor groundwater seepage could be anticipated immediately following rainfall due to percolation. This seepage will not impact stability nor should it imact construction activities.

Additional geologic reconnaissance was performed in 2006 at Waimanalo Gulch (Mink& Yuen and Knight Enterprises for Golder Associates [2006], included in Geosyntec 2007b). The reconnaissance was performed to map existing geologic features related to groundwater flow for the currently permitted landfill.

3. SEISMIC HAZARD EVALUATION

The Island of Oahu is an emergent portion of several huge basaltic shield volcanoes that rise from the ocean floor. Some earthquakes that affect Oahu are related to the injection of magma into the volcanic edifice, whereas others may be due to gravitational collapse of the flanks of the volcano (Yeats et al., 1997). In general, the earthquakes that impact Oahu are relatively shallow crustal events.

Geosyntec evaluated the seismic hazard at WGSL using the most recent United States Geologic Survey (USGS) probabilistic seismic hazard maps for the State of Hawaii (Klein et al., 1998). As required by the State of Hawaii regulations, Geosyntec considered seismic motions with a 2 percent probability of being exceeded in 50 years (Note: this is equivalent to 10 percent probability of exceedance in 250 years). Geosyntec established the design earthquake for the site to have a moment magnitude (M_w) 7.0 with expected bedrock free-field peak horizontal ground acceleration (PHGA) of 0.25g. A more detailed discussion of site seismicity is presented in Appendix B.

To select representative accelerograms for use in design, Geosyntec developed a target acceleration spectrum envelope for the design earthquake with 5 percent damping at periods of 0.2 seconds and 1.0 second based on Klein, et al. (1998). The database of acceleration time histories for the western United States was screened to find events that were similar in magnitude, tectonic environment, and PHGA. The acceleration response spectra of the candidate accelerograms were then plotted against the target acceleration response spectrum to select the representative accelerograms for use in the design analyses. Geosyntec selected the following three accelerograms from the catalog of shallow crustal earthquakes in the western United States for WGSL:

- The Cholame Shandon Array No. 5 (355 degrees) accelerogram, recorded during the 27 June 1966, Mw 6.3 Parkfield, California earthquake. The estimated distance between the strike-slip fault rupture plane and the recording station is 5.6 miles (9 km). The recorded PHGA was 0.36g. This motion was selected to represent the local low-magnitude event.

- The University of Hawaii (344 degrees) accelerogram, recorded during the 29 November 1975, Mw 7.2 Island of Hawaii earthquake. The earthquake occurred at a depth of 3.1 miles (5 km). The estimated distance between the fault rupture plane and the recording station is 27 miles (43 km). The recorded PHGA was 0.17g. This motion was selected to represent the far-field, high magnitude event.

- The Big Bear Lake – Civic Center Grounds (360 degree) accelerogram, recorded during the 28 June 1992, Mw 6.7 Big Bear, California earthquake. The estimated distance between the strike-slip fault rupture plane and the recording station is 7 miles (11 km). The recorded PHGA was 0.57g. This motion was selected to represent both the local and distant design events due its large magnitude and short site-to-source distance.

The acceleration response spectra of the three motions scaled to a PHGA of 0.25g match and exceed the target acceleration response spectrum in the period range of 0.1 to 1.0 seconds, which is the range of interest for seismic design at the site. The three accelerograms were scaled to 0.25g and used in the seismic site response analysis at the landfill. Appendix B includes the seismic hazard evaluation for the site.

More recently, WHM has agreed with the HDOH that,

> *"In the event of an earthquake having a magnitude 5.0 or greater that originates from a source within a 100-kilometer (60-mile) radius from the site, or an earthquake having a magnitude 7.0 or greater originating anywhere within the major Hawaiian Islands (the triggering event), the facility shall not accept and dispose of waste until a professional engineer registered in the State of Hawaii certifies the integrity and functionality of the landfill and its associated environmental controls, including, but not limited to, the lining system, leachate collection and control system, and surface water management system. In the event of an earthquake having a magnitude between 5.0 and 7.0 (a magnitude less than the triggering event) outside the 100-kilometer (60-mile) radius, the operator or site engineer shall make an immediate assessment to determine if the site should be temporarily shut down."*

4. WASTE STREAM, SOIL EXCAVATION, AND SOIL USAGE

Table 4.1 below presents the preliminary volume estimates for the preferred expansion master plan. Appendix C shows the grading plans over the life of the preferred expansion.

Table 4.1 Preliminary Volume Estimates

Preferred Expansion Quantities (Cells E-5 through E-11 only)	
Expansion Footprint	36.9 acres
Footprint of Landfill after Expansion	115.8 acres
Total Expansion Airspace Increase[3]	8,118,505 cy[4]
Soil Usage[5,6]	
Cell Excavation	3,026,250 cy
Fill[7]	131,804 cy
Soil Buttress	507,655 cy
Excavated material used as part of the Liner System (4 feet thick on floor and 3 feet thick on side slopes)	208,000 cy
Excavated material used a part of the final cover material (4 feet over 36.2 acres)	233,611 cy
Estimated Daily Cover Soil	1,617,937 cy
Surplus (any excess material remaining following closure will be graded,	327,243 cy

[3] Available gross volume above currently-permitted grading plan for <u>expansion area only</u> (Cells E-5 through E-11).
[4] cy = cubic yard.
[5] A positive number means soil available onsite; negative number means soil usage or soil required.
[6] Shrinkage and swell factors not included in soil usage calculations.
[7] To meet grade at certain locations; <u>excludes</u> the soil buttress.

landscaped and left in place in the stockpile area shown on Figure 7)[8,9]	

[8] A soil surplus results. The soil balance quantities for the whole site are shown in Appendix C.
[9] The remaining onsite stockpile area will be re-graded so that surface water is managed as designed (See Section 7 of this report).

5. LINER AND FINAL COVER SYSTEMS

5.1 Liner Systems

The liner system proposed for the expansion, including both ash and MSW cells, as shown on Figure 8, will consist of (from the bottom to the top):

- Prepared subgrade;
- Soil cushion layer;
- 40-mil-thick backing HDPE geomembrane (textured on both sides);
- Geosynthetic clay liner (GCL);
- 60-mil-thick primary HDPE geomembrane (textured on both sides);
- Cushion geotextile;
- 1 foot of gravel (maximum size of 1 inch)[10];
- Filter geotextile; and
- 2 feet of Operations layer (maximum size of 2 inches within 6 inches of the geotextile and a maximum of 6 inches in the upper 18 inches).

To collect leachate in the relatively flat areas of the expansion, perforated HDPE collection pipes will be placed within the drainage layer. The leachate will drain down the cells toward a new lined sump furnished with a riser pipe; the sump will be located adjacent to existing MSW Cells 11 and E-4. If ash is placed in the expansion, the sump configuration may be modified and the number of sumps increased. Any such changes will be submitted to the HDOH for review and approval prior to cell construction. Figure 9 shows the locations of the LCRS collection pipe and sumps for the preferred

[10] The gravel layer extends 10 feet vertically beyond the toe of the slope but is not required on the side slopes. Therefore, the filter geotextile will not be included and the operations layer will be in contact with the cushion geotextile.

expansion. Figure 10 shows the locations of the LCRS collection pipe and sumps for the alternative expansion.

Appendix D includes the infiltration analyses. Geosyntec used the HELP model (USEPA) to estimate leachate generation and maximum leachate head above the liner. The infiltration analyses assume that a final cover has not been constructed; this results in the highest infiltration, which in turn, results in the highest expected leachate head. The estimated maximum head above the composite liner is less than or equal to 12 inches and meets the regulatory requirements.

5.2 Final Cover

Figure 11 shows the final cover system for the expansion area. The final cover configuration will depend on the inclination of the final cover slopes.

6. LANDFILL DEVELOPMENT PLANS

6.1 Proposed Fill Grades

Figure 5 shows the final grading plan for the preferred expansion. Figure 6 shows the final grading plan for the alternative expansion, which assumes that a new ash cell[11] is constructed southeast of the E-1 berm and adjacent to the existing ash cells.

6.2 Proposed Excavation Grades

Figure 12 shows the base grading plan for the preferred expansion. Figure 13 shows the base grading plan for the alternative expansion (i.e., assuming a new ash cell is constructed southeast of E-1 and adjacent to the existing ash cells).

As the landfill expansion is developed, the excavated slopes in the expansion area will be inspected during and after excavation to observe the condition of the exposed rock materials. Based on these observations, the slope geometry may be modified from 1.5H:1V. Within one month following the completion of cell excavation and before lining is placed, all cut slopes over 10 feet high will be inspected and mapped by either a registered engineer or geologist trained in rock slope design.

6.3 Slope Stability Analyses

6.3.1 Introduction

The expansion area abuts the existing permitted areas; therefore, the current report evaluates cross sections through the expansion areas and cross sections through the existing permitted landfill that may be affected by the expansion. Geosyntec (2003 and 2007a) discusses stability analyses of the existing permitted landfill.

Figure 14 shows the cross section lines through the landfill. The same cross section lines apply to Figure 13. The slope stability analyses for both the preferred plan and the alternative plan are identical because the fill grades are the same everywhere, *except* in the area of the proposed ash cell located southeast of the existing ash cells. For the

[11] Depending on the ratio of MSW to ash received at the landfill, an additional ash cell may also need to be constructed at a later date in the northern portion of the Expansion area.

alternative plan, additional cross sections were evaluated for stability. Appendix E includes the stability analyses assumptions and results.

Two-dimensional slope stability analyses were performed using the software program SLOPE/W. The program employs a user-directed search routine to determine the potential critical shear surface with the minimum factor of safety. SLOPE/W was used to automatically search for critical shear surfaces through the waste and along the base lining system. Slope stability was evaluated using the limit equilibrium procedures based on the Spencer (1967) method of slices. Spencer's method satisfies all conditions of force and moment equilibrium.

To estimate the seismic stability of the landfill using deformation analyses, Geosyntec also estimated the yield acceleration (k_y or the horizontal coefficient that results in a factor of safety of 1.0). These k_y values were then incorporated into the seismic deformation analysis discussed in Section 6.4.

The slope stability during interim filling conditions will be evaluated as part of the final design for each new cell. The slope stability (infinite slope) for the final cover will be presented as part of the updated closure documents for the site.

6.3.2 Groundwater Level

The groundwater level is over 100 feet below the base of proposed excavations within the underlying bedrock ;. For the previous cells E1 through E4, a subdrain system was constructed along the base of the cells to intercept and convey any seepage if it occurred. No seepage was observed during construction of these cells. The subdrain system consists of a HDPE perforated pipe encapsulated in a gravel-filled trench. A similar system will be constructed for the expansion cells if continuous seepage is encountered to facilitate cell construction; minor seeps that dry out within one day after a rainstorm will not require subdrain installation. Even if present these seeps will have no impact on slope stability.

6.3.3 Material Properties

The assumed material properties used in the stability analysis and the sources of information for these properties are summarized in Appendix E and tabulated on Table 6.1 below. Geosyntec recommends that WMH verify the shear strengths of the site-specific materials, such as the material required for the soil buttresses, and the interface shear strengths of the base and side slope liners before constructing each cell within the expansion area as part of the Construction Quality Assurance program.

- Liner System. Various liner designs were used to construct different waste disposal cells over time (i.e., ash cells 1 through 8 and MSW cells 1 through 11). For previously constructed areas, Geosyntec used material properties assumed in Geosyntec (2003, 2007a); for the expansion liner system, Geosyntec assumed properties based on laboratory testing completed during prior cell construction The liner interface strengths for the new cells (Ash and MSW) will be verified, prior to construction, with direct shear tests conducted under peer reviewed methods and under the general guidance of ASTM D5321 and D6243. .

- Soil Buttresses. Geosyntec assumed shear strength of 38 degrees, no cohesion, and a unit weight of 135 pcf.

- Ash Waste. Shear strength and in-place unit weight tests have been performed on the ash material disposed at the site (Harding-Lawson, 1993). Additional tests performed by Mountain Edge Environmental (2004) show that the strength for the ash assumed by Geosyntec in previous evaluations (Geosyntec 2003, 2007a) for the site as conservative.

- MSW. Shear strength and unit weights are based on work completed by Kavazanjian et al. (1995).

- Subgrade. Based on the Geolabs geologic investigation, the weathered basalt underlying the site was assumed competent and stronger than the over lying materials; therefore, no slip surfaces extended below the liner system. No zones of liquefiable soils were identified at the site by Harding-Lawson (1993).

Table 6.1 Material Properties for Slope Stability Analyses

Material	Unit Weight (pcf)	Cohesion (psf)	Friction Angle	Reference
Municipal Solid Waste (MSW)	70	500 0 (@ $\sigma_n \geq$ 770 psf)	0° 33°	Fassett (1993) and Kavazanjian, et al. (1995)
Incinerator Ash	90	600	30°	Harding-Lawson (1993)
Soil Buttress	135	0	38°	NAVFAC (1982)
Liner System for *Ash* Cells 1, 2, 3, 4: Geonet/HDPE Geomembrane/Compacted Clay Liner Interfaces	N/A	0	10°	Sharma and Lewis (1994) and Geosyntec files
Liner System for *Ash* Cells 5, 6, 7, 8: Geonet/HDPE Geomembrane/GCL Interfaces	N/A	0 750 (@ $\sigma_n \geq$ 5,653 psf)	10° 2.5°	Sharma and Lewis (1994) and Geosyntec files
Liner System for *MSW* Cells 1, 2, 3, 4A: Compacted Clay Liner	120	0 2,800	20° 0°	Static Seismic, Duncan et al. (1989)
Liner System for *MSW* Cells 4B and 4C: Non-woven Geotextile/HDPE Geomembrane/Compacted Clay Liner Interfaces	N/A	0	9°	Sharma and Lewis (1994) and Geosyntec files
Liner System for *MSW* Cells 4C Remainder, 5, 6, 7, 8, 9, 10, 11: Non-woven Geotextile/HDPE Geomembrane/GCL Interfaces[1]	N/A	0 750 (@ $\sigma_n \geq$ 6,540 psf)	9° 2.5°	Sharma and Lewis (1994) and Geosyntec files
Liner System for Cells E1, E2, E3, and E4: Non-woven Geotextile/HDPE Geomembrane/GCL Interfaces	N/A	0 (@ $\sigma_n \leq$ 2,500 psf) 750 (@ $\sigma_n \geq$ 2,500 psf)	20° 2.5°	Sharma and Lewis (1994) and Geosyntec files
Expansion Lining System (Encapsulated GCL and non-woven geotextile/textured geomembrane)	N/A	0	14°	Geosyntec files

6.3.4 Results

The generally accepted static factor of safety for long term loading conditions is 1.5 (Duncan, 1992). A static factor of safety greater than or equal to 1.5 was achieved for the final grading plan shown on Figure 5. The static factors of safety for each cross section analyzed are tabulated below. The seismic deformation analysis for the landfill is discussed in the next subsection. Appendix E includes the slope stability analyses.

Table 6.2 Slope Stability Analyses Results

Cross Section	Static Factor of Safety	k_y
A	1.6	0.07g
A2	1.5	0.045g
6	2.2	0.30g
41	1.5	0.11g
42	1.9	0.16g
43	2.5	0.14g
45	1.9	0.22g
47	1.8	0.08g

6.4 Seismic Deformation Analysis

6.4.1 General

As required by Subtitle D (40 CFR Part 258), a seismic deformation analysis was performed to assess the performance of the landfill under a design earthquake. The design earthquake used in the seismic deformation analysis were developed as part of the seismic hazard evaluation (see Section 3).

6.4.2 Methodology

The seismic site response of the landfill was simulated and evaluated using the equivalent-linear, seismic response program SHAKE91, a one-dimensional, frequency domain, equivalent-linear elastic response model developed by Schnabel et. al. (1972) and Idriss and Sun (1992).

The waste/base liner/bedrock profile at the site was modeled using the procedures outlined in Appendix F. The three earthquake time histories discussed in Section 3 were selected to represent the design ground motions at the site.

The shear wave velocity of the MSW was assumed to vary with depth. The average unit weight of the MSW including daily and intermediate cover was assumed to be 70 pcf to be consistent with the slope stability analyses. The MSW curves for shear strain modulus ratio of (G/G_{max}) versus cyclic shear strain, and damping ratio (λ) versus cyclic shear strain, were based on work performed by Matasovic and Kavazanjian (1998).

The incinerator ash waste was assumed to behave as sandy silt (PI = 0) material in the analysis. The shear wave velocity of the ash was assumed to vary with depth based on procedures developed by Seed et al. (1984) for medium dense sands. The average unit weight of the ash was assumed to 90 pcf to be consistent with the slope stability analyses. The curves for shear strain modulus ratio of (G/G_{max}) versus cyclic shear strain, and damping ratio (λ) versus cyclic shear strain, were based on work for PI = 0 soils performed by Vucetic and Dobry (1991). See Appendix F for more details.

For each seismic ground motion time history (see Appendix F), the SHAKE91 model was used to generate shear stress time histories at the level of the base liner. The landfill was modeled by selecting four material columns:

- 50-foot high ash column to represent the new ash disposal area in the alternative expansion;
- 90-foot high ash column to represent other areas of ash disposal;
- 150-foot high and 200-foot high MSW columns to represent the MSW cells (existing and expansion).

6.4.3 Seismic Deformation Results

Seismic stability was evaluated in terms of acceptable levels of seismic deformation. The *RCRA Subtitle D (258) Seismic Design Guidance for Municipal Solid Waste Landfill Facilities* (U.S. EPA, 1995) notes that permanent acceptable seismic displacements of 6 to 12-inches are typically used in practice for the design of liner systems; this has also been presented in Seed and Bonaparte (1992). Geosyntec's results indicate that the maximum seismic displacement of is 6 inches or less. These values fall within the acceptable seismic displacements (i.e., deformations).

Geosyntec estimated seismic deformations using the procedures developed by Newmark (1965). In this approach, the stress time histories computed by SHAKE91 at the level of the potential failure surfaces were divided by the overlying mass and double-integrated using the computer software YSLIP_PM (Yan et al., 1997) at different levels of yield acceleration to generate graphs of yield acceleration versus seismic deformation.

The estimated seismic deformation based on the modified Newmark method is less than or equal to 6 inches for the expansion disposal areas for MSW and ash. Appendix F presents the seismic deformation analyses; the yield accelerations are tabulated on Table 6.2.

7. SURFACE WATER DRAINAGE

7.1 Existing Conditions

WGSL is located on the arid portion of leeward Oahu within the Waimanalo Gulch watershed. The watershed is very elongated with elevations ranging from near sea level at the terminus to over 2,000 feet at the upper end. The mean annual rainfall in the basin is approximately 20 to 30 inches with the greatest amounts falling at the upper elevations (Questa Engineering, 2001).

An ephemeral stream in Waimanalo Gulch discharges at the northeastern limit of the landfill. The stream is generally dry except during rainfall events (Geolabs, 2000). Currently, a concrete drainage channel captures surface water runoff from the western rock slopes surrounding the landfill and from up canyon areas. Two temporary 48-inch diameter corrugated metal pipes capture and direct the flow around the western stability berm. Surface water runoff from the eastern rock slopes is directed into drainage ditches along the edge of the eastern perimeter of the landfill and carried via two HDPE pipes to the sedimentation basin. The collected surface water including flow from both the east and west sides of the landfill along with upstream runoff flows into a sedimentation pond near the facility entrance. Flow from the sedimentation pond drains through three large culverts beneath Farrington Highway and eventually discharges into the ocean.

7.2 Design Criteria

Subtitle D requires that the surface water control features at landfills be designed to control both run-on and runoff from the 24-hour, 25-year storm. The 24-hour, 25-year storm at WGSL is 9.2 inches based on information presented by the State of Hawaii (1984). To evaluate the performance of the sedimentation/detention pond and estimate runoff for the landfill grading plan, Geosyntec followed design criteria for water quality presented in the *County of Honolulu Drainage Standards* (City and County of Honolulu, 2000), and used the Soil Conservation Service (SCS) Method.

7.3 Proposed Improvements

Two surface water control systems are proposed for WGSL as described below.

- Western Bypass Channel. GEI Consultants (GEI) prepared the Western Bypass Channel (off-site stormwater conveyance) for the upper canyon and western areas flows adjacent to the Landfill (Appendix G). This system will capture the upper watershed's flows and route them around the landfill so that they do not mix with the surface water runoff from the landfill. The water will flow via large diameter pipes and/or a concrete-lined channels and discharge at a point located south of WGSL's existing sedimentation basin.

- Onsite Stormwater Management System. This system has two components that capture: (i) flows from the western side of the landfill (primarily landfill runoff with minor amounts of run on); and (ii) flows from the eastern side (both landfill runoff and run on). Both flows will discharge to the existing sedimentation basin via drainage pipes and inlets along the edge of the landfill and open channels and downchutes on the landfill. At each collection point, a drop inlet will be constructed and the water will flow into the main conveyance pipe or channel. Collected runoff will flow into the existing surface water sedimentation basin.

Figure 15 shows the alignment for the landfill's eastern and western systems and general details for the pipe and collection channels for the preferred expansion. For the alternative expansion (i.e., new ash cell), the area to the southeast is modified; however, the overall surface water flow patterns remain the same as for the preferred expansion. Appendix H includes the runoff volume calculations for both the western and the eastern landfill drainage systems for both expansion plans. Pipes and channels will be sized when construction drawings for the expansion need to be prepared.

Geosyntec evaluated the existing sedimentation basin configuration (i.e., emergency spillway, etc.) for flood control and for water quality requirements in the *County of Honolulu Drainage Standards* (City and County of Honolulu, 2000). Geosyntec also used information presented by Shimabukuro et al. (1986) and EarthTech (2006) (e.g., dimensions, riser elevations, infiltration trenches, spillway, etc.). Geosyntec's evaluation concludes that the existing basin meets the requirements for flood control and for water quality provided upstream run-on is diverted around the basin.

7.4 Surface Water during Operations

During operations, surface water will be controlled by temporary pipes and ditches that will be moved as necessary to address stockpiles, active fill areas, the extent of each cell, and fill sequencing. Since the size of each cell may vary depending on the waste stream at the time, surface water details will be designed as part of preparing the construction drawing package for each cell.

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