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CLOSURE PLAN

CAPITOL DISPOSAL LANDFILL
Juneau, Alaska

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1. INTRODUCTION

1.1 Terms of Reference

This Closure Plan for the Capitol Disposal Landfill (formerly known as the Channel Landfill) in Juneau, Alaska, has been prepared by GeoSyntec Consultants (GeoSyntec) on behalf of Waste Management of Alaska, Inc. (WMI), the landfill's owner and operator. The closure plan has been prepared in order to meet requirements of Chapter 60, Title 18 of the Alaska Administrative Code (AAC) and Part 258 of Title 40 of the Code of Federal Regulations (CFR).

The current Solid Waste Facility Permit (SWFP) #8511-BA016, issued by the Alaska Department of Environmental Conservation - Southeast Regional Office (ADEC) on 16 March 1995, expires on 1 March 2000. Therefore, as required by the Alaska regulation, this closure plan for the Capitol Disposal Landfill has been prepared to support the permit renewal application.

Tom Boardman, P.E., and Krzysztof S. Jesionek, P.E., prepared this report. Dr. Neven Matasovic, P.E., and Dr. Hari D. Sharma, P.E., G.E., reviewed it in conformance with GeoSyntec's review policy.

1.2 Applicable State and Federal Regulations

This report was written after consulting applicable state regulations contained in Chapter 60, Title 18 of the AAC, and enforced by ADEC. In general, these requirements are referred to as "Title 18" regulations in this closure plan. The applicable federal regulations, which fall under Subtitle D of the Resource Conservation and Recovery Act (RCRA) and contained in Part 258, Title 40 of the CFR, are referred to herein as "Subtitle D" regulations. When they are cited, specific regulatory requirements are shown in *italics*.

The Alaska regulatory program for MSW landfills has been approved by the United States Environmental Protection Agency (USEPA) as being in conformance with Subtitle D. Therefore, ADEC has the authority to administer the requirements of Subtitle D, and approve performance-based final cover designs for MSW landfills without federal review. The Alaska regulatory criteria for final closure plans relevant to the Capitol Disposal Landfill are included in Sections 210 (Permit Application) and 395 (Closure Standards for a Class I or Class II MSWLF) of Chapter 60 (Solid Waste Management) of Title 18. ADEC's classifies the Capitol Disposal Landfill as a Class I MSW landfill.

2. GENERAL SITE INFORMATION

2.1 Site Location

The Capitol Disposal Landfill is located along the Gastineau Channel, between Egan Drive (also known as New Glacier Highway, State Hwy 7) and the Old Glacier Highway, approximately 5 miles (8 km) northwest of downtown Juneau, Alaska (Figure 1). The site is located in Section 34 of Township 41S, Range 67E, Copper River Meridian. The address is 5600 Tonsgard Court.

Lemon and Vanderbilt Creeks pass around the landfill to the north and south, respectively. As the site is located in a tidally influenced lowlands area and within the 100-year flood plain of Lemon Creek, a system of earthen dikes and berms protects the landfill from tidal inundation and flooding from adjacent creeks (Figure 2).

2.2 Site History

Official landfill operations began approximately in 1963 [Sweat-Edwards/Emcon, Inc. (Emcon), 1991]. Before creation of the landfill, disposal of waste was reportedly widespread and uncontrolled throughout the entire Lemon Creek region, [Levine-Fricke-Recon (LFR), 1998].

The unlined landfill has received a variety of wastes during its history. As the primary solid waste disposal facility for the Juneau area, municipal solid waste (MSW) was accepted and disposed across the majority of the site. The MSW was either spread in thin lifts or backfilled into sand/gravel excavations. Construction debris, asbestos, scrap metal, and various other materials were also disposed [Emcon, 1991]. Historic waste disposal locations have been summarized based on reviews of aerial photographs and discussions with previous landfill employees [LFR, 1998].

Sand and gravel has been excavated from a number of locations, including approximately 5 acres (2 ha) in the southern half, which were excavated to a depth of 30 ft (9 m). As ground water is relatively shallow (3 to 5 ft [1 to 1.5 m] below original ground surface), the resulting sand/gravel excavations turned into large ponds. These ponds have typically been backfilled with soils, trees, stumps, and woody debris from offsite construction projects. This backfill has been described as "mud and stumps" in previous reports [Emcon, 1991; LFR, 1998]. The large surface-water pond along the western perimeter of the site is the result of sand and gravel mining operations by the neighboring property owner.

Ownership of the landfill was transferred to Channel Landfill, Inc. in 1977. In 1985, two waste incinerators were installed at the landfill to reduce the volume of MSW disposal. Ash from the incinerators has been disposed in a number of unlined locations across the site while glass, asbestos, stumps, and MSW that exceeded the capacity of the two incinerators was disposed at the landfill [Emcon, 1991; LFR, 1998]. Scrap metal, white metals, and autos were, and continue to be, temporarily stored at the landfill before being barged to a recycling yard in Seattle, Washington.

2.3 Current Site Operations

WMI purchased the landfill in 1998 and changed the name to Capitol Disposal Landfill. The landfill Solid Waste Facilities Permit (SWFP) [ADEC, 1995] allows on-site disposal of the following materials:

- MSW ash;
- mud and stumps;
- asbestos and associated construction debris;
- demolition debris and unburnables; and
- MSW and other putrescibles exceeding the capacity of the incinerators.

The Capitol Disposal Landfill continues to operate twin modular MSW incinerators rated at 36 tons per day of MSW (approximately 3,000 pounds per hour for each unit). The incinerators run 24 hours a day, 365 days a year, except for maintenance down time.

During the summer months, MSW exceeding the capacity of the incinerators is loaded into shipping containers and sent via barge and rail to WMI landfills in Oregon. WMI is planning to construct a new lined cell for on-site disposal of some of this waste. The currently proposed location is an approximately 3-acre (1.2-ha) area in the southern portion of the site (Figure 2) in an area not currently covered with waste. However, final decisions on the location and construction of the new MSW cell will be based on ongoing results of settlement, liquefaction, and slope stability studies. These studies are discussed in Section 3 of this report.

Several private businesses operate in the northern portion of the site. W.R. Tonsgard Logging & Lumber, Inc., operates a sawmill and has several lumber piles in the northwest corner. A gravel and rock crushing yard run by Secon, Inc., operates in the center of this area. Arrow Refuse, Inc., a local waste collection service, has a maintenance shop on the western portion. Alaska Auto Towing temporarily impounds vehicles on the eastern edge. Stripped and totaled automobiles are stockpiled in this area pending shipment to Seattle for scrap metal recycling. It is GeoSyntec's understanding that these businesses will leave the site when their leases expire.

A site plan, which shows the current layout of the landfill facilities, property boundaries, and landfill monitoring stations, is shown on Figure 2.

2.4 Surrounding Land Uses

Various industrial and commercial businesses occupy the area surrounding the landfill. Some of the larger neighboring businesses include Alaska Light and Power, and Juneau Ready Mix (concrete batch plant). The site characterization report [LFR, 1998] identifies the potential environmental impacts of the surrounding properties on the closure of the landfill.

2.5 Subsurface Conditions

The Capitol Disposal Landfill is located on a relatively flat area within a glacially formed fiord at the base of a steep (35 to 45 degrees), 3,500-ft (1,070-m) mean sea level (msl) high mountain range. Surficial deposits have filled the deep bedrock-walled fiord, containing the Gastineau Channel and the project site, and provide the valley with a relatively flat floor. Seismic data indicate that the mountainsides continue downward at a steep slope, and create a bedrock trough at depth. The original floor of the valley is believed to be as much as 600 ft (180 m) below the floor of the present channel

[LFR, 1998]. A detailed description of the regional and site geology is presented in the site characterization report [LFR, 1998].

Documented field investigations to determine the subsurface conditions at the landfill include the following:

- five (5) soil borings, with four converted to monitoring wells, performed in 1991 [Emcon, 1991];
- a geophysical survey performed in 1998 [LFR, 1998];
- eight (8) soil borings, with six (6) converted to monitoring wells [LFR, 1998]; and
- three (3) geotechnical soil borings (Section 2.9 of this report).

Results from these investigations indicate that the site is underlain by loose silty sands to poorly graded sands with silt (SP-SM) and soft sandy silts (ML). The loose sands have been encountered to depths of up to 50 ft (15 m) below original ground surface. Equivalent clean sand SPT blowcounts $(N_1)_{60\text{-}CS}$ have been found to range from 9 to 28 for loose to medium dense sands.

2.6 Hydrogeology

A detailed description of the regional and site hydrogeology is presented in the site characterization report [LFR, 1998]. Ground water beneath the site fluctuates significantly due to tidal effects. During a three-day period of extreme tidal variation (6–9 September 1998), low tide of -3 ft (-900 mm) and high tide of +19 ft (6 m) were observed. Ground-water elevations were measured continuously using pressure transducers lowered into selected on-site ground-water monitoring wells. The average ground-water elevation over this period ranged from 13.7 to 17.6 ft (4.2 to 5.4 m) above mean lower low water (mllw). The average water level in the neighboring surface-water pond was 13.1 ft (4 m) mllw. The water levels at high tide in the monitoring wells nearest to the proposed location for the MSW cell (i.e., wells MW-5 and MW-9, Figure 2) ranged from 15.7 to 15.8 ft (4.8 m) mllw [LFR, 1998].

The large surface-water pond bordering the south and west edges of the site, is the result of sand and gravel mining excavations by the neighboring property owner to depths of over 30 ft (9 m) below ground surface. A culvert with a tide gate (i.e., one-way drainage) is in place at the western end of the pond. This culvert tends to stabilize the water level in the pond at approximately 13-ft (4 m) mllw.

The net ground-water flow at the site is to the southwest. Several slug tests, performed in 1998 to measure the in-situ hydraulic conductivity of the water-bearing soils beneath the site, indicate values ranging from 1.1×10^{-2} to 1.4×10^{-6} cm/s, with an average value of 4.1×10^{-3} cm/s [LFR, 1998]. The fine-grained soils, encountered around the surface-water pond, appear to restrict the movement of ground water through this area. Previous investigators have suggested that this "barrier condition" can be attributed to removal of coarse-grained sediments during the previous sand and gravel mining operations and subsequent backfilling with mud and stumps.

2.7 Climate

Local precipitation (i.e., snow and rainfall combined) varies from 55 in. (1,400 mm) at the Juneau Airport, to 80 in. (2,000 mm) in downtown Juneau. The reported frost depth for the area is approximately 36 in. (910 mm). The highest and lowest average monthly precipitation typically occurs in the fall and spring, respectively. Estimates on the annual potential evapotranspiration in the Juneau area range from 17.8 to 21.9 in. (450 to 550 mm) [LFR, 1998].

2.8 Existing Monitoring and Control Systems

This section of the report describes the existing environmental monitoring and control systems at the landfill for ground water, surface water, leachate, and landfill gas.

2.8.1 Ground-Water Monitoring

A total of ten (10) ground-water monitoring wells have been installed at the site. Well locations are shown on Figure 2. Wells MW-1, MW-3, and MW-4, installed through MSW, were recently abandoned under the supervision of GeoSyntec with a cement grout pumped through a tremmie pipe (Appendix A).

2.8.2 Surface-Water Monitoring and Control

The surface-water management system was designed to comply with Section 60.225(c), Title 18 of the AAC[1], and is within accepted standards of practice. GeoSyntec evaluated the system [GeoSyntec, 1999a, b] and proposed modifications that are scheduled for construction during the summer of 1999.

2.8.3 Leachate Management

As the current footprint of the landfill is unlined, leachate control, sampling, and management is not required by the site permit [ADEC, 1995]. Once the proposed new MSW cell (Figure 2) is constructed, featuring a composite lining system that meets Subtitle D and state requirements, leachate will be collected and disposed in accordance with applicable regulations and operating permit requirements.

2.8.4 Landfill Gas Management

The SWFP [ADEC, 1995] does not require monitoring or collection and extraction of landfill gas (LFG) at the site. However, WMI plans to install a passive LFG collection system within the closure cover. As the majority of refuse is incinerated, relatively low LFG generation rates are anticipated.

2.9 <u>1999 Geotechnical Field Exploration and Laboratory Testing Program</u>

2.9.1 Field Exploratory Program

In April 1999, GeoSyntec performed a geotechnical field investigation to collect subsurface data for engineering analyses. The field program consisted of drilling three (3) soil borings, excavating eight (8) test pits, and abandoning three (3) monitoring wells. Wink Geo-Tech International of Juneau, Alaska, was subcontracted to perform the field work. Soil borings were drilled with a track mounted, Mobile B-47 drill rig with 8.5-in. (215-mm) outer diameter hollow stem augers. In an effort to

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[1] Section 60.225(c), Title 18 of the AAC requires that "*...the owner or operator of a solid waste disposal facility shall construct and maintain a control system that will prevent run-on from flowing onto the active portion of the facility. The control system must be capable of handling the peak discharge from a 25-year storm.*"

minimize sands flowing up the augers, the hollow stem augers were filled with water before pulling the auger plug.

GeoSyntec collected soil samples using the following techniques:

- standard penetration test (SPT) sampler (1.4-in. [35-mm] inner diameter - no room for liners);
- AW rods; and
- mechanical "free fall" hammer: 140 lb (63.5 kg), 30 in. (760 mm) drop; and
- hollow stem augers were filled with water prior to pulling the auger plug.

Soil borings GT99-1 and GT99-2, both within the landfill's footprint, were advanced to a depth of 50 ft (15 m). An additional soil boring (GT99-3) was advanced to a depth of 30 ft (9 m) through a soil berm along the perimeter of the landfill. Appendix B contains the soil boring logs for the investigation.

To evaluate the extent of waste in the area of the proposed new MSW cell, eight (8) test pits were excavated to depths of approximately 3 ft (900 mm) along the north and northeast perimeters (Figure 2). These test pits were excavated at locations that were accessible during the time of drilling. With the exception of one test pit in the northeast corner that unearthed crushed barrels, metal, and wood debris, each test pit excavation encountered saturated mud/stumps to depths of 2 to 3 ft (600 to 900 mm). GeoSyntec also collected two ash samples from an ash disposal area located at the site's northwest corner. Prior to initial laboratory testing, the ash was screened to 1-in. (25-mm) minus to remove bigger pieces of glass, paper, metal, etc.

2.9.2 Laboratory Testing Program

2.9.2.1 Soil Samples

GeoSyntec performed laboratory tests on six (6) soil samples collected during the field investigation. The samples[2] were tested for grain size distribution (ASTM D 422) and classified using the Unified Soil Classification System (USCS). Soils beneath the

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2 i.e., samples S-4 (GT99-3), S-5 (GT99-3), S-7 (GT99-2), S-9 (GT99-2), S-8 (GT99-1) and S-9 (GT99-1)

footprint of the landfill were classified as a silty sand (SM) to poorly graded sand (SP), while soils within the southwest perimeter berm were classified as a well-graded sand with silt and gravel (SW-SM). The percent fines and soil classification information was incorporated into the engineering analyses. Copies of the laboratory reports are included in Appendix C.

2.9.2.2 MSW Incinerator Ash Samples

Initial laboratory testing performed by GeoSyntec Geoenvironmental Laboratory (GEL) for the ash samples included performance of the following tests:

- gradation (ASTM D 422);
- moisture content (ASTM D 2216);
- modified Proctor compaction (ASTM D 1557); and
- Atterberg limits (ASTM D 4318).

Results from these initial tests indicate that the ash can be classified as a "well-graded sand with silt and gravel" (SW-SM) using the USCS. The non-plastic ash samples contained approximately 9% and 11% fines (i.e., particles smaller than #200 sieve or 0.075 mm). The in-place moisture content ranged from approximately 28% to 29%. The maximum dry density (ASTM D1557) ranged from 94 to 97.5 pcf (1,500 to 1,560 kg/m^3), with an optimum moisture content of 19% and 20%, respectively.

A composite of the two ash samples was tested for compaction and hydraulic conductivity. The maximum dry density was found to be 100 pcf (1,600 kg/m^3) with an optimum moisture content of 17% (ASTM D1557). Before testing for hydraulic conductivity, the composite ash sample was screened to 3/8-in. (9.5-mm) minus to accommodate the 2.8-in. (70-mm) diameter testing apparatus. The sample was compacted to a dry density of 96.5 pounds per square foot (psf) (i.e., 96.5% relative compaction), at a moisture content of 20.6% (i.e., optimum +3.6%). When permeated under an effective confining pressure of 5 psi (35 kPa), the hydraulic conductivity of the compacted ash sample was found to be 8.5 x 10^{-5} cm/s (ASTM D 5084).

Further laboratory testing on the composite ash sample included the shear strength tests listed below. Similar to the hydraulic conductivity testing, the ash was screened to

3/8-in. (9.5-mm) minus to accommodate the 2.8-in. (70-mm) diameter testing apparatus before testing:

- two (2) unconsolidated-undrained (UU) triaxial compression tests (ASTM D 2850); and
- one (1) interface shear strength test (ASTM D 5321) between a composite ash sample and a 60-mil (1.5-mm) thick single-sided textured HDPE geomembrane (textured side against ash) manufactured by the National Seal Company.

The undrained shear strength of the composite ash sample was found to range from 6,200 to 6,900 psf (297 to 330 kPa) when compacted to a dry density of 86.6 pcf at a moisture content of 28.8% (i.e., moisture content of ash after disposal), and a dry density of 90.5 pcf at a moisture content of 16.9% (i.e., optimum moisture content), respectively. The unconsolidated undrained shear strength tests were performed under a confining pressure of 20 psi (i.e., 30 ft of ash at a moist unit weight of 115 pcf).

The large displacement interfacial friction angle, between the compacted ash and the textured geomembrane, was found to be approximately 28 degrees across the anticipated final cover normal stress range of 100 to 300 psf (5 to 14 kPa). Before shearing, the ash was compacted to a dry density of 87 pcf at a moisture content of 28%. Copies of the laboratory test results are included in Appendix C.

3. CLOSURE PLAN

3.1 Introduction

In acccordance with federal and state requirements, WMI will implement closure activities to reduce the impacts of the landfill on the environment and to ensure the long-term integrity of the facility following closure. Closure will be implemented progressively, i.e., final cover systems will be provided for areas after they reach final grade elevations. If conditions change at the site, WMI will update this closure plan accordingly.

Closure activities include the following:

- final grading;
- placement of final cover;
- construction of the LFG gas management systems in the Subtitle D cell, as appropriate;
- construction of final surface-water management system controls;
- removal of structures, as appropriate;
- continued water quality monitoring; and
- establishment of final site security and access.

3.2 Previously Closed Areas

As part of landfill development and operations, the area around the incinerators and office building has been covered with an asphalt cap (Figures 2 and 3). Paving materials (such as asphalt, concrete, etc.) have been used in construction of final covers at MSW landfills provided that subsurface materials provide stable foundation support and the paving material can be maintained [Repa, 1987; GeoSyntec 1994; Jesionek et al, 1995]. Due to very high resistance to erosion and low hydraulic conductivity of the paving materials used at the site, the asphalt cap meets performance-based requirements for both an infiltration layer, by achieving "... *an equivalent reduction in infiltration as the infiltration layer specified...*", and for an erosion layer "... *that provides equivalent protection from wind and water erosion as the erosion layer specified...*" by the prescriptive requirements of Section 395, Title 18 of the AAC.

3.3 **Fill Sequencing Plan**

Based on 1998 waste disposal records, the Capitol Disposal Landfill accepts approximately 22 tons of ash and 20 tons of construction and demolition (C&D) waste per day [Tarr, 1999]. These figures correspond to approximately 14 yd^3 (11 m^3) of in-place ash and approximately 25 yd^3 (18 m^3) of in-place C&D waste per day (assuming unit weights of 115 pcf [1,840 kg/m^3] and 60 pcf [960 kg/m^3] respectively). MSW exceeding the capacity of the incinerators is currently shipped by barge to landfills in Oregon. Once built, the proposed lined MSW cell will take limited quantities of this MSW.

In addition to constructing the new MSW cell, WMI plans to place the types of accepted waste (i.e., ash, C&D and MSW) in three different areas of the landfill. As shown on Figure 3, incinerator ash will generally be placed in the northern portion of the landfill, and C&D and MSW will be placed in the southern portion. Therefore, three different working faces may be used for waste disposal. Figure 3 presents the initial stages of the fill sequencing plan. These stages are briefly described below.

During the initial filling stage (Stage Ash-1 on Figure 3), incinerator ash will be placed in the center of the site to redirect surface-water runoff toward drainage ditches and a permanent surface-water outlet at the northern edge of the site (SWS-1). After drainage is established, ash filling will move to the northwest corner of the site (Stage Ash-2) to redirect drainage toward the same outlet. These two ash filling sequences correspond to the proposed Surface-Water Management System (SWMS) design [GeoSyntec, 1999b].

The next ash filling stage (Stage Ash-3 on Figure 3) will begin in the northwest corner and where the landfill will be brought to final grade. The ash will be placed in lifts 6 to 8 ft (2 to 2.5 m) high and 100 ft (30 m) wide. All waste slopes will be approximately 4H:1V and the top surface of a lift will be sloped approximately 3% to 5% toward surface-water collection ditches and outlets. Areas, allowed to drain toward collection ditches and outlets, will be covered with either soil or reinforced plastic sheet. WMI proposes to partially close the landfill in this area as final grades are reached.

C&D waste, including unsold scrap metal, tires, and other unburnables, will continue to be disposed in the southern strip, i.e., between the landfill boundary and the lined MSW cell. C&D waste will be placed in the strip area in lifts approximately 6 to 8 ft (2 to 2.5 m) high.

MSW that exceeds the capacity of the incinerators, will either be placed within the new MSW cell starting from the area of the leachate collection sump toward the east/southeast portion of the cell or barged to off-site landfills. The MSW will be placed in approximately 2-ft (600-mm) thick compacted lifts with 4H:1V side-slopes. Waste placement will be coordinated to minimize contact water and leachate volume.

As work leases expire within the next 2 years for the variety of private companies operating on site, their business operations will be removed providing more space for waste disposal.

3.4 Final Grading Plan

The proposed final grading plan at closure is shown on Figure 4. Figure 5 shows final cover configuration and details. Figure 6 shows cross-sections through the closed landfill. Both Subtitle D and state regulations do not specifically address final grading. However, in order to meet final cover slope stability requirements, the landfill will have side-slopes of 4H:1V (i.e., 25%) in its final configuration. As designed, the highest elevation of the closed landfill will be approximately 120 ft (37 m) above mllw. Top deck areas will be graded at approximately 3% (minimum) toward surface-water drainage ditches along the crest of the slopes.

The landfill slopes in the area of the ash disposal will be constructed with a single bench at an elevation of approximately 80 ft (24 m) mllw. The bench will be approximately 20 ft (6 m) wide and will be sloped at 4% toward the side-slope and 1% to 2% along the bench alignment. The bench will provide for erosion protection of the slopes as well as maintenance access around the site.

The final surface-water management system components will be constructed during final grading. At a minimum, these components will include: (i) final cover bench drainage ditches; (ii) perimeter road drainage ditches; and (iii) downchutes.

Final grading will include construction of the foundation layer for the final cover to provide a stable base for the overlying final cover components. For further slope protection and erosion control, the final site will be hydroseeded and irrigated as required during the germination period. The vegetative cover will be a mixture of native grasses and plants compatible with the expected weather conditions at the site.

3.5 Closure Area

The largest area of the landfill ever likely to require a final cover is estimated to be 38.3 acres (15.3 ha).[3] This total area includes a correction for the 4:1 (H:V) side slopes and does not include the previously closed portion of the landfill (Section 3.2). WMI has proposed to partially close the landfill in the area of ash filling as final grades are reached (Section 3.3).

3.6 Site Capacity

Since the base grades of the landfill were not surveyed prior to excavation and waste placement, and complete waste disposal records do not exist, it is difficult to estimate the total volume of waste currently in-place at the landfill. Consequently, the maximum inventory of waste ever likely to be on site over the active life of the facility is also difficult to estimate. However, based on a review of site history information [Emcon, 1991; LFR, 1998], and the most current topographic map of the landfill area, dated 22 April 1998, GeoSyntec has assumed that waste materials were placed from an elevation of 15 ft (4.5 m) mllw to an average elevation of 30 ft (9 m) mllw across the majority of the landfill. In addition, 5 acres (2 ha) of the southern half of the site were excavated to an elevation of –15 ft (–4.5 m) mllw, and backfilled with mud/stumps to an elevation of 15 ft (4.5 m) mllw. Based on these assumptions, approximately

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[3] Section 60.395(c)(2) of Title 18 requires that the closure plan include "... *an estimate of the largest area of the MSWLF ever likely to require a final cover...*"

968,000 yd^3 (740,000 m^3) of waste has been placed at the Capitol Disposal Landfill as of 22 April 1998 (i.e., the day of the most recent topographic map).[4]

The remaining total airspace at the Capitol Disposal Landfill has been estimated to be approximately 1,797,000 yd^3 (1,374,000 m^3) by comparing the most recent topographic map and the proposed final grading plan (Figure 4). Subtracting the estimated volume taken by the proposed final cover, access roads, and daily soil cover, the remaining waste airspace at the Capitol Disposal Landfill is approximately 1,712,000 yd^3 (1,310,000 m^3). Therefore, the landfill will contain approximately 2,680,000 yd^3 (2,050,000 m^3) of waste at completion of the active life of the facility.

3.7 Site Life Projection

Current waste disposal rates were used to project the Capitol Disposal Landfill's site life. Assuming that approximately 1,800 yd^3 (i.e., permitted 1,600 tons per year at 65 pcf) of MSW, 5,110 yd^3 of ash, and 9,125 yd^3 of C&D waste will be accepted per year in the future, the site life projection for the remaining waste airspace at the Capitol Disposal Landfill (i.e., approximately 1,712,000 yd^3) is approximately 108 years.

3.8 Description of Final Cover System

3.8.1 General

The final cover system for the Capitol Disposal Landfill will meet or exceed minimum regulatory requirements[5] and will perform the following functions [Jesionek et al., 1995]:

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[4] Section 60.395(c)(3) of Title 18 requires that the closure plan includes "*... an estimate of the maximum inventory of wastes ever likely to be onsite over the active life of the facility...*"

[5] Section 258.60(b) of Subtitle D states that "*The Director of an approved State may approve an alternative final cover design that includes:*

(1) An infiltration layer that achieves an equivalent reduction in infiltration as the infiltration layer specified in paragraph (a)(1) of this section, and

(2) An erosion layer that provides equivalent protection from wind and water erosion as the erosion layer specified in paragraph (a)(2) of this section." (continued on the next page)

- separate the waste from the environment;
- provide appropriate slopes to promote surface water run-off; control erosion by conveying run-off at non-scouring flow rates;
- minimize rain and snow infiltration into the waste; and
- control and contain landfill gas.

The proposed final cover system, over the specific landfill disposal areas, will consist of layers as described below (from top to bottom) (Figure 5).

Unlined Ash Disposal Areas

- 12-in. (300-mm) thick soil vegetative (erosion) layer;
- geocomposite drainage layer;
- 60-mil HDPE geomembrane, with double-sided texturing (as a hydraulic barrier or infiltration layer); and
- 6-in. (150-mm) thick screened (to ½-in. [12-mm] minus prior to placement) and compacted ash foundation layer.

Unlined C & D Waste Disposal Areas

- 12-in. (300-mm) thick soil vegetative (erosion) layer;
- geocomposite drainage layer, where required;
- 60-mil HDPE geomembrane, with double-sided texturing (as a hydraulic barrier or infiltration layer); and

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The Alaska regulations allow for alternatives to the prescriptive final cover design described above. Section 395(b) of Title 18 states that "*. . .(b) The department will approve an alternative final cover design if the applicant demonstrates that the proposed design will protect human health and the environment and that it includes an:*

(1) infiltration layer that achieves an equivalent reduction in infiltration as the infiltration layer specificied in (a) of this section; and

(2) erosion layer that provides equivalent protection from wind and water erosion as the erosion layer specified in (a)(3) of this section."

- 24-in. (600-mm) thick compacted ash foundation layer, with the upper 6 in. (150 mm) screened to ½-in. (12-mm) minus prior to placement.

The final cover system over the future lined MSW cell has a similar composite hydraulic barrier layer to the proposed Subtitle D base liner for the MSW cell. The proposed final cover system over the lined MSW area will consist of the following (from top to bottom) (Figure 5):

- 12-in. (300-mm) thick soil vegetative (erosion) layer;
- geocomposite drainage layer, where required;
- 60-mil HDPE geomembrane, with double-sided texturing (as the upper component of the composite hydraulic barrier);
- internally-reinforced, non-woven geotextile carrier type geosynthetic clay liner (GCL) (as the lower component of the composite hydraulic barrier); and
- 24-in. (600-mm) thick compacted ash foundation layer, with the upper 6-in. (150 mm) screened to ½-in. (12-mm) minus prior to placement.

3.8.2 Foundation Layer

The foundation layer provides a stable and smooth working surface on which the overlying hydraulic barrier can be placed. The foundation layer is typically composed of soil, suitable waste materials (such as contaminated non-hazardous soil, ash, municipal biosolids), geosynthetics, or a combination of these materials provided they exhibit acceptable engineering properties. Where soil and compaction characteristics are considered satisfactory, the existing interim cover soil may also serve as the foundation layer material. However, the use of existing interim cover as a foundation layer may require some reworking of the soil, such as stripping, scarification, and recompaction to obtain the desired properties.

Based on laboratory testing performed by GeoSyntec, incinerator ash from the Capitol Disposal Landfill appears to be an acceptable foundation layer material. However, before placement, the upper 6-in. (150-mm) of the ash foundation layer will be screened to remove all particles larger than ½-in. (12-mm). All glass, metal, or any other angular pieces that could potentially damage the overlying geosynthetic hydraulic barrier will be removed as well. The ash will be placed in loose lifts of 8-in. (200-mm) thick or less, and compacted in-place. The upper surface should be graded smooth to provide a stable surface for placement of the overlying hydraulic barrier.

Due to the variability of the maximum dry unit weight (ASTM D 1557) of the ash material, a minimum relative compaction criteria will not be used during the placement and construction testing of the foundation layer. Rather, the minimum number of equipment passes per the compaction criteria associated with the equipment type will be specified before construction, and incorporated into the construction testing procedures.

3.8.3 Hydraulic Barrier Layer

3.8.3.1 Introduction

The function of the hydraulic barrier layer within the final cover system is to minimize rainfall and snow melt infiltration into, and gas migration out of, the underlying landfill. Both the federal (Subtitle D) and state (Title 18) regulations require an earthen hydraulic barrier (infiltration layer) with a minimum thickness of 18-in. (450-mm), that has "*... a hydraulic conductivity less than or equal to the hydraulic conductivity of any bottom liner system or natural subsoils present, or a hydraulic conductivity no greater than* 1×10^{-5} *cm/s, whichever is less.*"

Two different hydraulic barrier layers are proposed for the unlined and lined areas of the landfill. While both barriers do not meet the minimum thickness and earthen material requirements, they both achieve an equivalent or better reduction in infiltration as the prescriptive barrier. In addition, as the foundation layer is not required by the prescriptive regulations, the proposed 24-in. (600-mm) or 6-in. (150-mm) thick layer of compacted ash will provide an additional component to the infiltration layer.

3.8.3.2 Unlined Areas

Where waste is already in-place in unlined areas of the landfill, the prescriptive hydraulic barrier is an 18-in. (450-mm) thick earthen layer with a hydraulic conductivity of 1×10^{-5} cm/s or less. WMI proposes to use an alternative hydraulic barrier consisting of a double-sided textured 60-mil (1.5-mm) thick HDPE geomembrane since the ash does not meet the hydraulic conductivity requirement and fine grained, low permeability soils are not readily available in the Juneau area. The geomembrane is a more effective hydraulic barrier than the prescriptive soil barrier layer. Installation of the HDPE geomembrane over the compacted ash foundation layer results in a "composite effect," minimzing surface-water infiltration into the underlying waste.[6] The double-sided texturing of the material also helps to address final cover stability requirements (Section 3.9.4).

The use of a geomembrane requires careful preparation of the surface on which it is placed to limit the potential for damage during and after installation and to enhance its performance. Figure 5 provides design details for installation of the geomembrane.

3.8.3.3 Lined Areas

The proposed hydraulic barrier over the future lined MSW cell will incorporate a composite hydraulic barrier consisting of an internally-reinforced geosynthetic clay liner (GCL) overlain by a double-sided textured 60-mil (1.5-mm) HDPE geomembrane. The GCL will be comprised of an approximately ¼-in. (6-mm) thick layer of dry sodium bentonite encapsulated between two non-woven geotextiles. The GCL will be reinforced through needle-punched stitching. The sodium bentonite component of a GCL can typically achieve a hydraulic conductivity of 1×10^{-9} to 5×10^{-9} cm/s for the normal stress range expected within the final cover [Koerner and Daniel, 1995]. The GCL and HDPE geomembrane will result in a "composite effect" to help minimize surface-water infiltration into the underlying waste.

3.8.4 Drainage Layer

The function of the drainage layer is to intercept water entering the cover system and convey it to appropriate surface-water control features such as drainage ditches and downchutes. While not required by the regulations, a drainage layer helps maintain a lower water head over the hydraulic barrier during storm events. This results in a reduced potential for infiltration into the wastes below, and improves the slope stability of the cover system. The drainage layer will be placed only in the side-slope areas.

The proposed drainage layer for both cover designs (i.e., for unlined and lined areas) consists of a geosynthetic drainage net sandwiched between two layers of non-woven 8 oz/yd^2 (270 g/m^2) filter geotextile (i.e., geocomposite drainage layer). While the average frost depth at the site is approximately 36 in. (900 mm), the

6 Based on the results of a site specific Hydrologic Evaluation of Landfill Peformance (HELP) model (Section 3.9.6).

geocomposite drainage layer should drain any free water down the geomembrane lined side slopes before the cold winter months. During the spring melt, water percolating from the melting vegetative layer should enter the drainage layer and flow down the slopes toward the drainage collection systems along the toe of the slopes after the entire vegetative/erosion layer has thawed. Seepage pressures exerted in the erosion layer may cause some localized movements, however, these will be addressed during periodic maintenance.

The primary parameter in selecting drainage layer material is transmissivity. HELP model results show that a geocomposite drainage layer, with an equivalent hydraulic conductivity (k) of 10 cm/s or greater, should be used within the final cover system. The upper filter geotextile of the geocomposite drainage layer provides a separation between the geosynthetic drainage layer and the overlying soil vegetative layer. Its primary purpose is to reduce the migration of particles that could clog the drainage layer. The purpose of the lower geotextile is to improve the interfacial shear strength properties between the underlying textured geomembrane and the geocomposite drainage layer.

3.8.5 Erosion Layer

The purpose of the erosion or vegetative layer is to minimize wind and water erosion, minimize percolation of water into the underlying elements of the final cover system, maximize evapotranspiration, provide resistance to animal or plant intrusion into the cover component, and improve aesthetics. The proposed erosion layer for both cover designs (i.e., for lined and unlined areas) will consist of a minimum 12-in. (300-mm) thick layer of organic, silty soils that should promote vegetative growth. Compaction requirements will be dependent on material specific shear strength testing and the type of vegetation selected for the final design. The final cover's vegetation plan will specify hardy, indigenous plants with root zone depths less than 12 in. (300 mm), and minimal maintenance requirements.

3.9 Engineering Analyses

Engineering analyses performed as part of the final cover system design included the following:

- settlement analyses;
- seismic hazard evaluation;
- liquefaction analyses;
- static and seismic slope stability analyses for the final cover system and the waste mass at the final grading; and
- surface-water infiltration analyses.

3.9.1 Settlement Analyses

Settlement will be a function of: (i) immediate and long term settlement of the subgrade soils due to the placement of the overlying wastes and final cover system; (ii) settlement and decomposition of the waste materials themselves; and (iii) potential liquefaction induced settlement (see Section 3.9.3). The effect of settlement on the grading of the final cover will be monitored as part of the post-closure monitoring program.

3.9.1.1 Subgrade - Immediate Settlement

The Capitol Disposal Landfill is likely underlain by loose to medium dense, saturated silty sands and sandy silts. These coarse grained, freely draining soils will settle during the placement of waste and the final cover system (i.e., minimal long-term settlement of the native subgrade soils). The magnitude of this settlement should be small relative to the settlements caused by MSW decomposition and C&D waste compression.

3.9.1.2 Subgrade - Long Term Settlement

The southern half of the site was excavated to a depth of approximately 30 ft (9 m) during sand and gravel mining operations and then backfilled with soils, trees, stumps, and woody debris from offsite construction projects. The currently proposed location for the new MSW cell is over this backfilled area. Due to the loosely placed, saturated, organic nature of the "mud and stumps" fill, it is highly likely that some initial compression of the fill will take place, but the majority of the settlement should occur over the long term due to the decomposition of the organics and consolidation of silty/clayey fill. WMI proposes to construct a settlement test plot to gauge the magnitude of the potential short and long term settlement and will evaluate the results to estimate future long-term settlement and the time frame in which the settlement will occur.

3.9.1.3 Waste Settlement

As with subgrade soils, there are both immediate and long term components to waste settlement. While the ash is relatively inert and should not be subject to long-term decomposition and settlement, the ash material may be underlain by up to 15 ft (5 m) of MSW spread across some areas in the northern portion of the site. This MSW will likely continue to settle.

Sowers [1968] noted that MSW typically settles from 10% to 30% under its own weight in the first few years after its placement. Othman et al. [1995] note that the magnitude and rate of MSW settlement is controlled by many complex factors such as: (i) mechanical compression due to self weight and surface loads; (ii) movement of smaller waste into larger voids; (iii) physicochemical changes such corrosion and oxidation; and (iv) biochemical decomposition under aerobic and anaerobic conditions. The first two factors are short-term settlement mechanisms and have likely occurred already or will occur as the waste is being placed in the future. The last two settlement mechanisms are long-term, and more difficult to quantify. As the goal of the final cover system is to minimize surface-water infiltration into the waste below, maintaining a minimum 3% to 5% grade along the top deck of the closed landfill is an important part of the postclosure maintenance plan. In addition, final cover settlement will be monitored as part of the postclosure activities. Should settlement measurements indicate that the minimum grade is not being met, the surface will be regraded to meet the minimum required 3% to 5% grade.

3.9.2 Seismic Hazard Evaluation

GeoSyntec performed a seismic hazard evaluation to evaluate the design earthquake with a 10% probability of exceedance in 250 years. The design earthquake was established as a moment magnitude (M_w) 8.5 event on the southeast segment of the Denali fault, which is approximately 28 miles (45 km) southwest of the site. Ground motions from this event are characterized by peak horizontal ground acceleration (PHGA) in hypothetical bedrock outcrops at the geometric center of the site equal to

0.25 g, with a corresponding significant duration of strong shaking of approximately 56 seconds. A copy of the seismic hazard evaluation report is included in Appendix D.

3.9.3 Liquefaction Evaluation

3.9.3.1 General

Based on the 1999 geotechnical investigations, it appears that a loose to medium dense, silty sand layer extends from the ground surface (i.e., elevation of about 20 ft [6 m] mllw) to a depth of approximately 50 ft (15 m) (i.e., elevation of -30 ft [-9 m] mllw). As ground water occurs at an elevation of approximately +15 ft (4.6 m) mllw, the loose sand is saturated as well. The recently deposited soils consist of fluvial sands and gravels overlying marine deposited silty sands. Based on these conditions and the site's seismic setting, GeoSyntec performed a liquefaction evaluation. Samples from the three borings drilled by GeoSyntec in 1999 (Section 2.9) were tested using techniques conducive to performing a standard liquefaction analysis.

The liquefaction evaluation used the procedures outlined in "Proceeding of the NCEER Workshop on Evaluation of Liquefaction Resistance of Soils" [Youd and Idriss, 1997]. The design earthquake was established as a moment magnitude (M_w) 8.5 event characterized by peak horizontal ground acceleration in hypothetical bedrock at the geometric center of the site equal to 0.25g. Following the procedures outlined in Subtitle D seismic design guidance for MSW landfills [USEPA, 1995], the bedrock acceleration was amplified to 0.30g at the ground surface due to the site-specific effects of the deep, cohesionless soils. This "free field" horizontal acceleration of 0.30 g was conservatively used in the liquefaction analysis.

Corrected blow counts $(N_1)_{60}$ cs for boring GT99-1 ranged from approximately 8 to 16 (10% to 35% fines); for boring GT99-2 they ranged from approximately 9 to 18 (2% to 16% fines); and for boring GT99-3 they ranged from approximately 14 to 28 (8% to 10% fines and 13% to 26% gravel). Based on the results of the liquefaction analysis, it appears that the loose, saturated, silty sands underlying the landfill will liquefy in the event that the design earthquake occurs. The liquefaction analyses are included in Appendix E. The liquefaction may impact landfill slope stability, which has been addressed in Section 3.9.5. In addition, liquefied subgrade soils experience post-liquefaction vertical settlements, which is presented in the following sub-section.

3.9.3.2 Liquefaction Induced Vertical Settlement

In the event that the subgrade soils liquefy during seismic motions at the landfill, some degree of vertical settlement is expected as excess pore water pressures are dissipated. Using the procedures outlined by Tokimatsu and Seed [1987], the liquefaction induced vertical settlement of the 20 to 45-ft (6 to 14-m) thick layer of silty sand underlying the site was estimated to range from 6 to 11 in. (150 to 280 mm). Considering that the landfill is unlined, such movements should not disrupt the current waste disposal activities. However, if there are impacts to grading within closed sections of the landfill, these will be addressed as part of the postclosure maintenance program.

3.9.4 Final Cover Slope Stability

3.9.4.1 General

The slope stability of the proposed final cover system was analyzed under static and seismic loading conditions. Static stability was evaluated in terms of the acceptable factor of safety against slope failure for long-term, drained conditions. The factor of safety is defined as the ratio of available shear strength (i.e., resisting forces) to mobilized shear strength (i.e., driving forces). The generally accepted static factor of safety for the long-term loading condition is 1.5 [Duncan, 1992]. During or soon after a heavy rain storm and spring thaw, the generally accepted short-term static factor of safety is 1.2.

Seismic stability was evaluated in terms of acceptable levels of seismic deformation. For this analysis, the acceptable level of permanent seismic displacement within the final cover system was assumed to be 12 in. (300 mm) or less [Seed and Bonaparte, 1992].

3.9.4.2 Material Properties

Table 1 lists the material properties and sources of information used in the final cover stability analyses. Shear strength parameters are based on a normal stress range of 100 to 300 psf (5 to 15 kPa) at large displacement.

3.9.4.3 Methods

Since the cover length to thickness ratio is very large, an infinite slope analysis approach was followed. The general equation for calculating factor of safety (FS) for an infinite slope of soil thickness (z) above a weak (failure) plane, as proposed by Matasovic [1991], is given by the following equation (similar equations with special cases are also presented in Lambe and Whitman [1969] and Sharma and Lewis [1994]:

$$FS = (c/(\gamma Z\cos^2\beta) + \tan\phi\,[1-\gamma_w\,(Z - d_w)\,/\,\gamma Z] - k_s \tan\beta \tan\phi) \div (k_s + \tan\beta)$$

where:

FS - factor of safety
c and ϕ - soil or soil/geosynthetic interface strength parameters along the failure plane;
β - slope angle;
d_w - depth of water below ground surface;
γ - unit weight of materials;
γ_w - unit weight of water; and
k_s - the seismic coefficient.

Static and seismic stability analyses were performed for 3H:1V, 3.5H:1V and 4H:1V slopes with drainage layers. The 12-in. (300-mm) thick vegetative layer, with a unit weight of 120 lb/ft^3 (1,900 kg/m^3), was assumed the driving mass. Under the low normal stresses within the cover system, the interfacial strength between the textured HDPE and the upper, non-woven geotextile within the geosynthetic clay liner (GCL) was assumed more critical than the internal shear strength of the reinforced GGL. However, the critical interface for the cover stability analysis was assumed to be between the textured HDPE geomembrane and the compacted ash foundation layer with an interfacial friction angle of 28 degrees and no cohesion (for static loading-long term and seismic loading). For static loading-short term drainage, the peak interfacial strength of 28 degrees and cohesion of 7 psf was used in the analysis.

Table 1
MATERIAL PROPERTIES
FINAL COVER SLOPE STABILITY ANALYSES
Capital Disposal Landfill

Material	Unit Weight (pcf)	Shear Strength		Source of Data
		Cohesion (psf)	Friction Angle	
Vegetative Layer (silty sand [SM])	120	0	34°	Duncan et al. [1989]
Vegetative Layer/Geocomposite Drainage Layer interface	NA	0	32°	GeoSyntec [1998a]
Geocomposite Drainage Layer/Textured HDPE interface	NA	0	30°	GeoSyntec [1996; 1998b]
Textured HDPE/Ash interface				GeoSyntec (Appendix C)
(peak strength)	NA	7 psf	28°	
(large displacement)	NA	0	28°	
Textured HDPE/Geosynthetic Clay Liner interface (non-woven geotextile)	NA	0	29°	GeoSyntec [1999c]

Notes:

1) Shear strength parameters are based on a normal stress range of 100 to 300 psf (5 to 15 kPa) at large displacement.

2) NA – Not applicable.

The seismic slope stability was analyzed by including an inertial (i.e., pseudo-static) horizontal acceleration or seismic coefficient (k_s), measured as a percentage of gravity. The design earthquake was established as a moment magnitude (M_w) 8.5 event characterized by peak horizontal ground acceleration in hypothetical bedrock outcrops at the geometric center of the site equal to 0.25g. Following the procedures outlined in Subtitle D Seismic Design Guidance for MSW Landfills [USEPA, 1995], the bedrock acceleration was amplified to 0.36g at the top of the landfill due to the site specific effects of the deep, cohesionless soils and waste mass. Therefore, the effective peak horizontal acceleration of the final cover system was assumed to be k_{max} = 0.36g. The seismic stability analysis was performed to establish the "yield acceleration" (k_y) resulting in a factor of safety of 1.0. The ratio of k_y to k_{max} was plotted against the design magnitude earthquake to estimate permanent seismic displacements using the modified Newmark [1965] method and charts developed by Makdisi and Seed [1978].

3.9.4.4 Final Cover Slope Stability Results

Based on the results of the slope stability analysis, the proposed final cover will have a static factor of safety greater than 1.5. The proposed final cover is calculated to have less than 12 in. (300 mm) of displacement during the design seismic event when constructed at a 4H:1V slope with a drainage layer. The results of the analysis are summarized in Table 2 and presented in Appendix F.

Table 2
FINAL COVER SLOPE STABILITY RESULTS
Capital Disposal Landfill

Side Slope	Loading Case	Factor of Safety	k_y	Estimated Seismic Displacement[1]
3H:1V	Static	1.6	---	---
	Static (short term)	1.0	---	---
	Seismic (k_{max}=0.36g)	---	0.17g	18 to 40-in. (450 to 1000-mm)
3.5H:1V	Static	1.9	---	---
	Static (short term)	1.11	---	---
	Seismic (k_{max}=0.36g)	---	0.22g	4 to 15-in. (100 to 380-mm)
4H:1V	Static	2.1	---	---
	Static (short term)	1.26	---	---
	Seismic (k_{max}=0.36g)	---	0.25g	2 to 6-in. (50 to 150-mm)

Note:

1) Reported seismic displacements are upper and lower bound values per Makdisi & Seed [1978].

3.9.5 Landfill Slope Stability at Final Grading

3.9.5.1 General

Slope stability was analyzed under static and seismic loading conditions for the proposed final grading plan shown on Figure 4. Two cross-sections were evaluated through the incinerator ash waste to an elevation of approximately +120 ft (37 m) mllw (Sections A-A and B-B). One cross section was evaluated through the C&D waste to an elevation of approximately +80-ft (24 m) (Section C-C). The cross section locations are shown in Appendix G. The goal of the landfill slope stability analysis was to evaluate if the proposed final grading plan shown on Figure 4 was stable in terms of acceptable factors of safety and permanent seismic displacements.

3.9.5.2 Material Properties

Table 3 lists the material properties and sources of information used in the landfill slope stability analyses.

Table 3
MATERIAL PROPERTIES
LANDFILL SLOPE STABILITY ANALYSIS
AT FINAL GRADING
Capital Disposal Landfill

Material	Unit Weight (pcf)	Shear Strength Cohesion (psf)	Shear Strength Friction Angle	Source of Data
MSW	65	0	32°	Kavazanjian et al. [1995]
Ash	115	6,000	0°	GeoSyntec (Appendix C)
C&D Waste	60	0	32°	assumed
Silty Sand	120	0	34°	Duncan et al. [1989]
Liquefied Silty Sand	120	500	0°	Seed et al. [1989]
Mud/Stumps	100 100	1,000 0	0° 26°	assumed Duncan et al. [1989]
Silt	115	0	32°	Duncan et al. [1989]

3.9.5.3 Methods

Two-dimensional slope stability analyses were performed for total stress conditions, using the commercial software package, UTEXAS3. The program was used to search for critical circular and block shear surfaces through the waste and soil subgrade. The slope stability was evaluated using limit equilibrium procedures with the Spencer [1967], Lowe [1960], and Bishop [1955] method of slices. Bishop's method satisfies moment and vertical force equilibrium; Lowe's method satisfies vertical and horizontal force equilibrium; and Spencer's method satisfies all conditions of equilibrium.

Seismic slope stability analyses were performed by including an inertial (i.e., pseudo-static) horizontal acceleration or seismic loading coefficient-measured as a percentage of gravity-occurring during the design seismic event. The design earthquake was established as a moment magnitude (M_w) 8.5 event characterized by a PHGA in hypothetical bedrock outcrops at the geometric center of the site equal to 0.25g. Following the procedures outlined in the Subtitle D Seismic Design Guidance for MSW Landfills [USEPA, 1995], the bedrock acceleration was amplified to 0.36g at the top of the landfill due to the site specific effects of the deep, cohesionless soils and waste mass.

An effective PHGA of the overall potential sliding mass was then developed. This maximum average acceleration value for a potential sliding mass extending to a specific depth (k_{max}), represents the cumulative effect of the non-uniform acceleration profile within the sliding mass overlying the shear surface. For this analysis, k_{max} was conservatively estimated using the Makdisi and Seed [1978] relationship, as 60% of the maximum acceleration at the top of the landfill (U_{max} = 0.36g) during the design seismic event. Therefore, k_{max} was assumed to be 0.22g. The seismic stability analysis was performed to establish the "yield acceleration" (k_y) resulting in a factor of safety of 1.0. The ratio of k_y to k_{max} was plotted against the design magnitude earthquake to estimate permanent seismic displacements using the modified Newmark [1965] method and charts developed by Makdisi and Seed [1978].

Seismic stability was evaluated in terms of acceptable levels of seismic deformation. Seed and Bonaparte [1992] report that permanent displacements of up to 12 in. (300 mm) are typically used in practice for the design of geosynthetic liner systems. The acceptable level of permanent seismic displacement within the landfill at final grading was assumed to be 12 in. (300 mm) or less.

Based on information provided by WMI, GeoSyntec has assumed that the surface-water pond will be filled with mud/stumps backfill to an elevation of approximately 20 ft (6 m) mllw. Slope stability was not evaluated in the southern portion of the landfill where the new MSW cell is proposed as the final waste elevation in this area will depend on the results from the proposed settlement test plot. A final waste elevation of +80-ft (+24-m) mllw was assumed for the final grading plan in this area.

3.9.5.4 Slope Stability Results

Three separate cross sections were evaluated in the stability analysis. Section A-A represents a typical section through the ash and soil subgrade toward the perimeter of the landfill. Section B-B represents a typical section through the ash and soil subgrade toward the surface pond. Section C-C represents a typical section through the C&D waste and soil subgrade toward the perimeter of the site.

Based on the results of the landfill slope stability analysis, the proposed final grading plan of 4H:1V slopes to an elevation of +120 ft (+37 m) mllw in the area of ash placement, and +80 ft (+24 m) mllw in the area of C&D waste placement, will achieve a factor of safety greater than 1.5 under static loading conditions for the existing soil conditions. As the yield acceleration was calculated to be higher than the assumed seismic loading coefficient, permanent seismic displacements are expected to be minimal during the design seismic event for non-liquefied subsurface soils. In the event of subsurface soils liquefying during a seismic event, the static factor of safety was found to be greater than 1.1, indicating a low probability of post-liquefaction "flow failure." The critical static and seismic factors of safety for cross-section C-C were found for shear surfaces through the 20-ft (6-m) high perimeter sand berm, while the critical post-liquefaction shear surface was through the C&D waste mass. The results are summarized in Table 4 and in Appendix G.

Table 4
LANDFILL SLOPE STABILITY RESULTS AT FINAL GRADING
Capital Disposal Landfill

Cross-Section	Loading Case	Factor of Safety	k_y
Section A-A	Static	3.4	---
	Seismic (k_{max}= 0.22g)	NA	0.34g
	Post Liquefaction, Static	1.35	---
Section B-B	Static	3.2	---
	Seismic (k_{max}= 0.22g)	NA	0.34g
	Post Liquefaction, Static	1.32	---
Section C-C	Static	1.8	---
	Seismic (k_{max}= 0.22g)	NA	0.30g
	Post Liquefaction, Static	1.19	---

Note:

1) Factors of safety and k_y were calculated under the assumption that the surface water pond will be filled with mud/stumps backfill to an elevation of 20 ft (6 m) mllw.

Post-Liquefaction Stability and Deformations

The potential for liquefaction-induced flow was evaluated by performing a static stability analysis with residual, undrained shear strengths within the sand layer underlying the landfill. When using conservative residual shear strengths, a static factor of safety of 1.1 or greater is generally considered acceptable to limit the potential for post-earthquake flow failure when evaluating whether an area is unstable or not. [Marcuson et al., 1990; U.S. EPA, 1995]. This approach is consistent with Alaska regulations, which require that an "*...owner or operator shall consider, at a minimum, ...known onsite or local soil conditions that more likely than not will result in differential settling or ground failure under static loading conditions during an earthquake...*"

The post-liquefaction static factor of safety was greater than 1.1 for each of the cross sections evaluated. While the results of the slope stability analysis indicate an acceptable factor of safety against post-earthquake flow failure, some degree of permanent seismic displacements (lateral as well as vertical settlements) should be expected in the event that the landfill is impacted by seismic ground motions. Considering that the site is unlined, such movements may not disrupt landfill operations. Both the operations plan and the post-closure maintenance plan address rectifying post-earthquake deformations.

3.9.6 Surface-Water Infiltration Analysis

3.9.6.1 General

As discussed in Section 3.8, two alternative final cover designs are proposed, i.e., one for the lined and another for the unlined areas (Figure 5). The goal of this infiltration analysis is to demonstrate that both proposed alternative designs meet federal and state regulatory requirements.[7]

3.9.6.2 Permeability Requirement

Measurements of the in-situ hydraulic conductivity (permeability) of the water-bearing soils beneath the site indicate values ranging from 1.1×10^{-2} to 1.4×10^{-6} cm/s, with an average value of 4.1×10^{-3} cm/s [LFR, 1998]. As the proposed final cover system over the unlined areas of the site will incorporate a relatively impermeable HDPE geomembrane (hydraulic conductivity of approximately 2×10^{-13} cm/s), the permeability requirement will be met.

The proposed base lining system for the future MSW cell will incorporate a composite hydraulic barrier consisting of a HDPE geomembrane overlying a geosynthetic clay liner (GCL). To meet the permeability requirement, the final cover system for the future lined area will incorporate the same hydraulic barrier (i.e., GCL overlain by HDPE geomembrane).

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7 Federal (Subtitle D) regulations require that the final cover system "*... has a permeability less than or equal to the permeability of any bottom liner system or natural subsoils present, or a permeability no greater than 1×10^{-5} cm/sec, whichever is less...*".

The state (Title 18) regulations require that the final cover system includes "*... an infiltration layer that contains a minimum of 18 inches of earthen material with a permeability no greater than 1×10^{-5} cm/sec; and an erosion layer that contains a minimum of six inches of earthen material capable of sustaining native plant growth ...*" Both state and federal regulations allow for an alternative final cover design if the proposed design will "*protect public health and the environment*" and includes an infiltration layer that achieves an "equivalent reduction in infiltration" as the specified infiltration layer.

3.9.6.3 Infiltration Requirement

Water balance analyses were performed using the HELP[8] computer model [USEPA, 1984a, 1984b, 1996] to evaluate the prescriptive versus alternative (i.e., performance-based) final cover designs in terms of infiltration through the final cover system and into the underlying waste. Peak hydraulic heads and inflow versus outflow rates were also checked in the analysis.

Climatological data used in the HELP model include daily precipitation, daily mean temperature, daily solar radiation, maximum leaf area index, growing season, and evaporative zone depth. Where possible, synthetic climate data generated by the HELP model was scaled per monthly averages for the Juneau area.

The water balance analyses incorporated soil properties such as saturated hydraulic conductivity, porosity, field capacity, initial soil water content, and wilting point. Default soil values for the HELP model were used in the analysis. The initial soil water content was conservatively assumed to equal to the field capacity of the soil. The erosion layer was assumed to be a silty sand (i.e., classified as SM in the USCS). The prescriptive 18-in. (450-mm) thick hydraulic barrier was assumed to be a silt (ML) with a hydraulic conductivity of 1×10^{-5} cm/s, while the foundation layer was assumed to be compacted incinerator ash (default material type in HELP model). Soil input data for the HELP model are included in Appendix H.

The alternative final cover design incorporates a geocomposite drainage layer and a HDPE geomembrane. Both materials have default values within the HELP model. The geomembrane was assumed to have standard manufacturing defects (i.e., 1 hole/acre), excellent/good installation (i.e., 1 hole/acre), and good intimate contact.

Additional assumptions incorporated into the HELP model included the following:

- "fair to poor" grass vegetation conditions on the final cover surface;
- the evaporative zone depth of 8 in. (200 mm);

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[8] The HELP model is a computer program that incorporates a quasi-two-dimensional water balance method into computation of water infiltration and leachate generation for MSL landfills. The program contains provisions for evaluating daily run-off, evapotranspiration, surface-water infiltration, water heads, and liquid migration from MSW landfills. Input parameters include climatological data, soil data, and final cover design data.

- maximum leaf area index of 2 for fair stand of grass; and
- 5% top deck slope with drainage distance of 175 ft (53 m) (i.e, from top deck centerline to edge of 4H:1V slope).

3.9.6.4 Results

Unlined Areas (Ash and C&D Areas)

The peak daily surface-water infiltration rate through the proposed alternative cover design for the unlined areas was found to be approximately 500 times less than the rate through the prescriptive cover. The peak daily hydraulic head over the alternative final cover was found to be 1 in. (25 mm) versus 6 in. (150 mm) for the prescriptive cover. Appendix H contains a detailed presentation of input data and the summary results of the HELP model calculations.

Lined Areas (MSW Cell)

The peak hydraulic head over the composite liner in the proposed MSW cell was found to be less than 12 in. (300 mm) during the 30-year post-closure period, thereby meeting the requirements of Subtitle D and Title 18 of the AAC.

3.10 <u>Final Cover Construction</u>

Section 60.395(c)(1) of Title 18 requires that the closure plan provides "*...a description of ...the methods and procedures to be used to install the cover.*" General final cover construction methods are described in this section.

3.10.1 Construction Methods

The final cover will be constructed in stages as areas of the active landfill are brought to final design grades. These partially closed sections of the landfill will be overlapped to provide a continuous final cover. Construction of the final cover will generally occur during the drier months of the year. Construction of the final cover and the landfill gas collection and surface-water control systems will be closely coordinated.

3.10.1.1 Foundation Layer

The foundation layer will be built with incinerator ash in lifts that will not exceed 6 in. (150 mm) in compacted thickness. The upper 6 in. (150 mm) of the foundation layer will be screened to ½-in (12-mm) minus before placement. The surface upon which the geosynthetics (i.e., flexible geomembrane or GCL) will be deployed will be examined by the site engineer and geosynthetics installer to identify any potentially damaging conditions before installation.

3.10.1.2 Barrier Layer

The barrier layer will consist of a 60-mil (1.5-mm) thick flexible geomembrane (over the unlined waste disposal areas of the landfill) or a composite hydraulic barrier consisting of a 60-mil (1.5-mm) thick flexible geomembrane overlying a geosynthetic clay liner (over the lined MSW cell). All geosynthetic materials will be installed by a qualified geosynthetics installer meeting the experience requirements outlined in the project specifications.

Geosynthetic Clay Liner

Detailed procedures for the geosynthetic clay liner installation will be provided in the construction specifications and final construction quality assurance plan. At a minimum, the procedures will address the following items:

- ensuring that the subgrade (i.e., top of the foundation layer) does not contain debris, roots, or sharp objects that could damage the geosynthetic cover components;
- construction of an anchor trench at the top of sloped areas;
- placement of panels with the long dimension parallel to the slopes;
- overlap of geosynthetic clay liner panels in accordance with the manufacturer's recommendations; and
- repair of damaged areas using a geosynthetic clay liner patch extending at least 12 in. (300 mm) outside the damaged area.

The geosynthetic clay liner will be placed over the MSW cell area of the landfill only. The geosynthetic clay liner will not be placed during rainy conditions (i.e., to prevent hydration of the bentonite). After placement, the geosynthetic clay liner will be immediately covered by the geomembrane to protect it from moisture. In accordance with standard installation procedures, the contractor will unwrap and install only as much material as can be covered by the geomembrane the same day.

Although laboratory tests have shown that geosynthetic clay liners can "self-heal" small punctures, a relatively smooth subgrade prevents damage to the material. WMI also proposes to implement a stringent construction quality assurance monitoring plan and to limit the types of equipment allowed over the area until after placement of the erosion layer.

Geomembrane

Following geosynthetic clay liner installation (in the MSW cell area) or foundation preparation (in the unlined area), the placement of the 60-mil (1.5-mm) double-sided textured, geomembrane will commence. The construction procedures employed will include, at a minimum, the following:

- approval of geomembrane subgrade by construction quality assurance personnel and the geomembrane installer;
- deployment of HDPE panels following an approved panel layout submitted by installer;
- seaming and welding of panels under observation of construction quality assurance personnel;
- both non-destructive and destructive conformance testing of welds and seams in accordance with the construction quality assurance (CQA) plan;
- repair and retesting of any failing seams; and
- surveying by a licensed land surveyor of seam and repair locations for preparation of an as-built panel layout drawing.

Double-sided textured HDPE geomembrane will arrive at the site in rolls and will be deployed by unrolling the material from the top of the slope. The rolls are typically suspended from a front-end loader or tractor using an A-frame assembly to facilitate the unrolling process. Adjacent HDPE geomembrane panels will be overlapped at least 2 in. (50 mm) and seamed. Fusion seams will be used wherever possible; otherwise, extrusion seams will be used. Seams will be aligned in a down slope direction wherever possible. The details of HDPE installation will be described in the construction specifications and final CQA plan.

3.10.1.3 Drainage Layer

WMI proposes to install a drainage layer over the barrier layer in the side-slope areas of the final cover system. The drainage layer will consists of geotextile filters and a HDPE geonet. The geocomposite will be secured in an anchor trench before deployment downslope.

The geocomposite will be deployed on a downslope with minimum overlaps. In general, installation will conform to the manufacturer's recommendations unless the specifications dictate requirements that are more stringent. Details of deployment will be described in the construction specifications and final CQA plan.

3.10.1.4 Erosion Layer

The soil erosion layer will be placed over the drainage layer of the final cover system to a minimum 12-in. (300-mm) thickness. The material will be placed over the entire closure area by equipment with appropriately low-bearing pressures which will be outlined in the construction specifications.

3.10.2 Construction Quality Assurance

To ensure that proper construction techniques and procedures are used and to verify that the materials and installation techniques used meet the project and regulatory agency specifications, a CQA program will be instituted before placement of the final cover. The final CQA plan will be prepared once the construction drawings and technical specifications for closure have been prepared.

After completing a final cover, a construction quality assurance report will be prepared and submitted to ADEC for approval. The final construction quality assurance report will be signed and sealed by a State of Alaska registered civil engineer. A draft CQA plan is presented in Appendix I.

3.11 Surface-Water Management at Closure

3.11.1 General

This section describes the conceptual surface-water management system after the final cover system has been placed. The landfill will be closed in stages as filling proceeds. The purpose of the system is to control flows on the landfill surface after the final cover has been placed, prevent inundation and ponding on the landfill final cover, minimize erosion of final cover, and minimize erosion, slope failure, washout, and over-topping of associated surface-water conveyances.[9]

3.11.2 Surface-Water Control Components

3.11.2.1 General

Following are the anticipated components of the surface- water control system:

- drainage ditches;
- downchutes;
- conduits; and
- perimeter road ditch.

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[9] State of Alaska regulatory requirements for precipitation and drainage controls are contained in Sections 60.225(b)(2) of Title 18, which states that "... *if the department determines that a control system for stormwater run-off is necessary to prevent the landfill from contributing to siltation or flooding problems in nearby surface water bodies construct and maintain a control system capable of containing and controlling the run-off from a 24-hour, 25-year storm...*"

Further, Sections 60.225(b)(3) of Title 18 requires that "... *the owner or operator of a solid waste disposal facility shall construct and maintain a control system that will prevent run-on from flowing onto the active portion of the facility. The control system must be capable of handling the peak discharge from a 25-year storm.*"

In the final landfill configuration, surface water on the cover will flow onto drainage ditches along the perimeter of the top deck areas and benches, or to perimeter road drainage ditches located along the base of the landfill. Flow from the upper ditches will be routed into downchutes. All flow will be routed through conduits into existing off-site drainage areas. The locations for the proposed drainage features are shown on Figure 4.

3.11.2.2 Benches and Bench Ditches

A single bench will be installed at an approximate elevation of 80 ft (24.4 m) mllw around the final cover to shorten the surface-water runoff drainage path and to intercept the flow before it attains erosive velocities. The bench will collect side-slope drainage and will provide access for maintenance and repair. The proposed bench is approximately 20-ft (6-m) wide and will be sloped approximately 4% toward the side-slope and at least 2% along the bench alignment.

Rainfall on the side-slopes will be collected at the inside edge of the bench and will be directed to inlet structures located at low points. The bench will be horizontally sloped to drain inward to the toe of the slope and laterally along the toe to the inlet structures of the downchutes. Bench ditches will be lined with erosion control matting, or an equivalent, to prevent erosion. The ditches will be v-shaped, generally with 2H:1V side-slopes, and at depths that will be determined during the preparation of the final closure design.

3.11.2.3 Downchutes

The downchutes will convey storm flow from the top deck and bench of the landfill to the perimeter drainage ditch along the base of the landfill. An appropriate number and size of downchutes, determined during the construction-level design of the final cover system, will be installed within the closure areas. The downchutes will consist of rock-lined drainage ditches running down the exterior face of the side-slopes. The complete exposure of the downchutes facilitates visual inspection for leakage and damage.

3.11.2.4 Perimeter Road Ditch

The perimeter road ditch will collect drainage from the side-slopes and provide access for maintenance and repair around the landfill. Rainfall on the side-slopes will be collected at the inside edge of the perimeter ditch and will be directed along the access road to inlet structures located at low points. The road will be graded to drain toward the toe of the slope and laterally along the toe to the inlet structures of the conduits (pipes). Ditches will be lined with erosion control matting, or an equivalent, to prevent erosion. The ditches will be v-shaped with depths and slopes to be determined during the construction-level design.

3.11.2.5 Conduits

The conduits will be used to convey flows from the perimeter road drainage ditch onto the off-site areas. Corrugated HDPE pipe with smooth interior, or equivalent, will be placed and sloped to drain. The conduits will be sized during construction level design.

3.11.2.6 Vegetation on Final Cover

Re-grading before vegetation is established presents the greatest potential for erosion of the slopes. In order to minimize erosion and off-site transport of sediment, permanent vegetative cover will be established as soon as possible on the slopes. The vegetative cover's functions will include:

- reduction of rainfall impact;
- reduction of surface-water velocity;
- promotion of infiltration;
- trapping of sediment; and
- retention of soil.

In order to protect the slopes before vegetation is established, straw or wood fiber-type mulch will be applied at the time of seeding.

3.11.2.7 Energy Dissipaters

As needed, energy dissipaters, including rip-rap and concrete aprons, will be constructed at the outlets of downchutes and culverts to reduce flow velocities.

3.11.2.8 Temporary Sediment Barriers

If necessary, straw bales and filter (silt) fences will be installed over the closure area and in its vicinity to trap sediment. Straw bales will also be placed, as appropriate, in channels, bench ditches, and at culvert and downchute inlets.

3.12 Environmental Monitoring and Control Systems at Closure

Environmental monitoring and control systems at landfill closure will include the following:

- ground-water monitoring system;
- surface-water monitoring system;
- leachate management system; and
- landfill gas management system.

3.12.1 Ground-Water Monitoring System

The current ground-water quality monitoring system consists of six wells; two upgradient wells (MW-6 and MW-10) and four point of compliance wells (MW-2, MW-5, MW-7, and MW-8) [LFR, 1999]. Throughout the closure period, ground-water quality monitoring will be conducted in accordance with the ground-water monitoring and reporting program [LFR, 1999], SFWP [ADEC, 1995], Subtitle D, and their revisions. Inspection and maintenance of the components will also be carried out throughout the closure period.

3.12.2 Surface-Water Management System

The landfill will be closed in phases as filling proceeds. The surface-water management system will operate throughout the closure period and will be periodically inspected and maintained. Drainage ditches, downchutes, conduits, temporary diversion dikes, straw bale barriers, and temporary and permanent seeding will control stormwater run-off and erosion. The ditches will intercept and route off-site non-contact water from the developed areas and adjacent properties. The ditches will be periodically cleaned to reduce siltation.

3.12.3 Leachate Management System

The leachate collection and removal system (LCRS) within the proposed new MSW cell will be operating at the time of landfill closure, and will continue to operate throughout the closure period for this cell. Leachate from the LCRS will be pumped into an on-site storage tank, or if feasible, directly connected to the sewer system and pumped into the Publicly Owned Treatment Works (POTW) facility for treatment and disposal. Accordingly, operational activities at closure may include monitoring leachate head in the leachate sumps to ensure that the LCRS is working properly.

3.12.4 Landfill Gas Management System

The rate of landfill gas (LFG) production is affected by the suitability of the landfill environment for methanogenic-producing biological activity. These environmental factors include the percentage of burned versus unburned waste, waste composition, pH, toxic chemicals, waste compaction, and moisture. Because of the wet climate in southeast Alaska, the waste tends to have high moisture content, and decomposes faster. In recognition of this, WMI will assess the need for a LFG extraction system at the time of closure. Maintenance and monitoring activities of the LFG extraction system will also be carried out, as required, during the postclosure period.[10]

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[10] Section 60.350 of Title 18 of the AAC requires that "*The owner or operator of a Class I... MSWL shall ensure that the concentration of methane gas generated by the facility does not exceed*

(1) 25 percent of the lower explosive limit for methane in facility structures, excluding gas control or recovery system componenets; and

(2) the lower explosive limit for methane at the facility property boundary."

3.13 Additional Closure Activities

3.13.1 Decommissioning of Environmental Control Systems

As part of closure activities, components of environmental control systems that have come in contact with leachate or landfill gas will be dismantled, cleaned and reused, or appropriately disposed. Should the decommissioning of ground-water monitoring wells become necessary, ADEC will be notified and appropriate procedures employed to decommission the wells.

3.13.2 Survey Monuments

There are a number of permanent markers and/or survey monuments already installed along or near the landfill property boundary (Figure 2). These monuments will be used at the time of landfill closure to establish location and elevation of wastes, containment structures, monitoring facilities, and other pertinent objects at the site.[11]

3.13.3 Initial Survey and Map

After closure, WMI will conduct an aerial photographic survey, or used an alternative surveying technique, to create a map depicting the date of closure and the as-closed topography of the area. The survey will be conducted to develop a 2-ft (600-mm) contour map at a scale of 1 in. = 200 ft (1:2400).

3.13.4 Site Security and Access

As part of closure activities, the site will be fenced to restrict access. Locked gates will be periodically monitored by security personnel and entry will only be permitted to authorized personnel. Appropriate signage will also be posted.

3.13.5 Structure Removal

The incinerator building and its waste hauling company will stay on site after the closure of the landfill. All other remaining structures, not required for postclosure maintenance activities or postclosure land uses, will be dismantled and removed from the site as landfill operations are terminated.

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[11] Section 60.210(b)(14) of Title 18 requires that "*... a closure plan... must include... the location of any proposed permanent markers or survey monuments...*"

3.14 Schedule for Closure

Areas within the landfill will be closed once final grades are reached throughout the facility's active life. This staged approach minimizes surface-water infiltration through the waste. As discussed in Section 3.7, the site life projection is approximately 108 years, putting the final landfill closure date in the year 2107.[12] WMI will notify the ADEC of the closure of any discrete unit, before the onset of closure activities, and will provide the agency with construction drawings and specifications[13].

Upon completion of closure activities, a detailed description of the closed site will be filed with ADEC.[14]

3.15 Cost Estimate for Closure

The cost of final closure is primarily a function of the construction of the final cover, revegetation, landfill gas monitoring and control, the surface-water management system, and security measures. installation. The final closure cost estimate includes the following assumptions:

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[12] Section 60.245(a) of Title 18 states that "*Except where a different time is specified... or is otherwise approved by the department, the owner or operator of a landfill shall place the final cover on those waste management areas that have reached final elevation within 90 days after the last waste placement.*"

Section 60.395(c)(4) of Title 18 requires that the closure plan includes "*... a schedule for completing all activities necessary to satisfy the closure standards in this section.*"

[13] Section 60.395(e) of Title 18 requires that "*Before beginning closure of a Class I... MSWLF..., the owner shall submit written notification to the department of the intent to close the MSWLF.*"

[14] Federal regulations, related to the schedule of closure activities, are contained in Section 258.60 of Subtitle D. Section 258.60(f) requires that "*The owner or operator must begin closure activities of each MSWLF unit no later than 30 days after the date on which the MSWLF unit receives the known final receipt of waste...*" Further, Section 258.60(g) requires "*The owner or operator of all MSWLF units must complete closure activities of each MSWLF unit in accordance with the closure plan within 180 days following the beginning of closure...*"

State regulations, related to the schedule of closure activities, are contained in Section 60.395 of Title 18. Section 60.395(f) requires that "*... the owner or operator shall begin closure activities of a Class I... MSWLF no later than 30 days after the date on which the MSWLF stops accepting waste...*" Further, Section 60.395(g) requires "*The owner or operator of a Class I... MSWLF shall complete closure activities of the MSWLF in accordance with the closure plan and within 180 days after beginning closure...*"

- a maximum "disturbed" area of land containing waste of 38.3 acres (15.4 ha) will require grading and final cover placement; and

- some environmental monitoring systems will already be completed and in-place as part of the final closure.

The closure cost estimate includes construction of the following final cover system:

- a minimum 6-in. (150-mm) thick foundation layer;

- a 60-mil (1.5-mm) thick double-sided textured, HDPE geomembrane over the foundation layer in all areas except the MSW cell. In the area above the proposed MSW cell (approximately 3 acres [1.2 ha]), the geomembrane will be underlain by a GCL over the foundation layer;

- a double-sided geocomposite drainage layer (on side-slopes only); and

- a minimum 12-in. (300-mm) thick erosion layer;

- final cover surface-water management system downchutes, ditches, conduits, and other structures; and

- final on-landfill components of the passive gas extraction system (in the proposed MSW cell only).

Table 5 presents a preliminary summary of the closure cost estimate in 1999 dollars.[15] The preliminary cost estimates are based on quotes from suppliers, vendor information, prior experience, and conventional cost-estimating guides. Actual costs may vary, as the details of the closure plan are finalized and because of factors beyond the reasonable control of WMI, including market conditions, construction conditions, material availability, labor relations, and other unforeseeable events. The estimate will be amended if the anticipated closure date changes, or if there are changes to the

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[15] Subtitle D requirements, pertaining to financial assurance for closure, are included in Section 258.71(a). They require that *"The owner or operator must have a detailed written estimate, in current dollars, of the cost of hiring a third party to close the largest area of all MSWLF units ever requiring final cover... at any time during the active life in accordance with the closure plan...*

(1) The cost estimate must equal the cost of closing the largest area of all MSWLF unit ever requiring a final cover at any time during the active life..."

approved financial mechanism. At a minimum, the cost estimate will be annually adjusted for inflation as required by Section 258.71 (a)(2), Subpart G of Subtitle D.

3.16 Postclosure Land Use

Given the industrial activities occurring in the immediate vicinity of the landfill, it is possible that the closed landfill could serve as an operating area for some of these businesses. Any use of the site, whether industrial, or conversion to an open-space park, will not disrupt the ground-water monitoring, surface water, leachate, and other control features at the site.

Table 5
PRELIMINARY COST ESTIMATE FOR CLOSURE
Capitol Disposal Landfill

Item	Estimated Cost (1999 $)
Final Cover	4,305,000
Revegetation/Hydroseeding	77,000
Surface-Water Drainage System	60,000
Passive Landfill Gas Trenches	27,000
Design/CQA Services	300,000
Total Estimated Closure Cost	**$4,769,000**

Note:

1) See Appendix J for Detailed Cost Estimate

3.17 Maintenance of Closure Plan

The most recently approved closure plan will be maintained at the Capitol Disposal Landfill located at 5600 Tonsgard Court, Juneau, Alaska, 99801, telephone number

(907) 780-6545, as long as the facility is open. Once the landfill is closed, the documentation will be moved to WMI area headquarters in San Rafael, California, at 155 North Redwood Drive, San Rafael, California, 94903, telephone number (415) 479-3700.

3.18 Recording

Upon completion of closure activities at the landfill, WMI will record a notation on a deed and submit written notification to ADEC along with a detailed description of the closed site in accordance the agency's requirements.[16]

3.19 Change of Ownership

In the event that WMI sells the Capitol Disposal Landfill to another party, WMI will notify the new owner of the existence of the closure standards and of the signed compliance conditions and agreements. WMI will notify the ADEC of the change in title within 30 days.

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16 Section 60.395(h) of Title 18 of the AAC requires that "*After closure of a Class I... MSWLF, the owner or operator shall submit written notification to the department that a certification verifying completion of closure in accordance with the closure plan has been placed in the operating record. The certification must be signed and sealed by a registered engineer or must be approved by the department.*"

Section 60.395(i) of Title 18 of the AAC requires that "*After closure of a Class I... MSWLF, the owner or operator shall record a notation on the deed to the landfill facility property, or some other instrucment that is normally examined during a title search, and submit written notification to the department that the notation has been recorded and that a copy has been placed in the operating record. The notation on the deed must, in perpetuity, notify any potential purchaser of the property that*

(1) the land was used as a MSWLF; and

(2) use of the land is restricted..."

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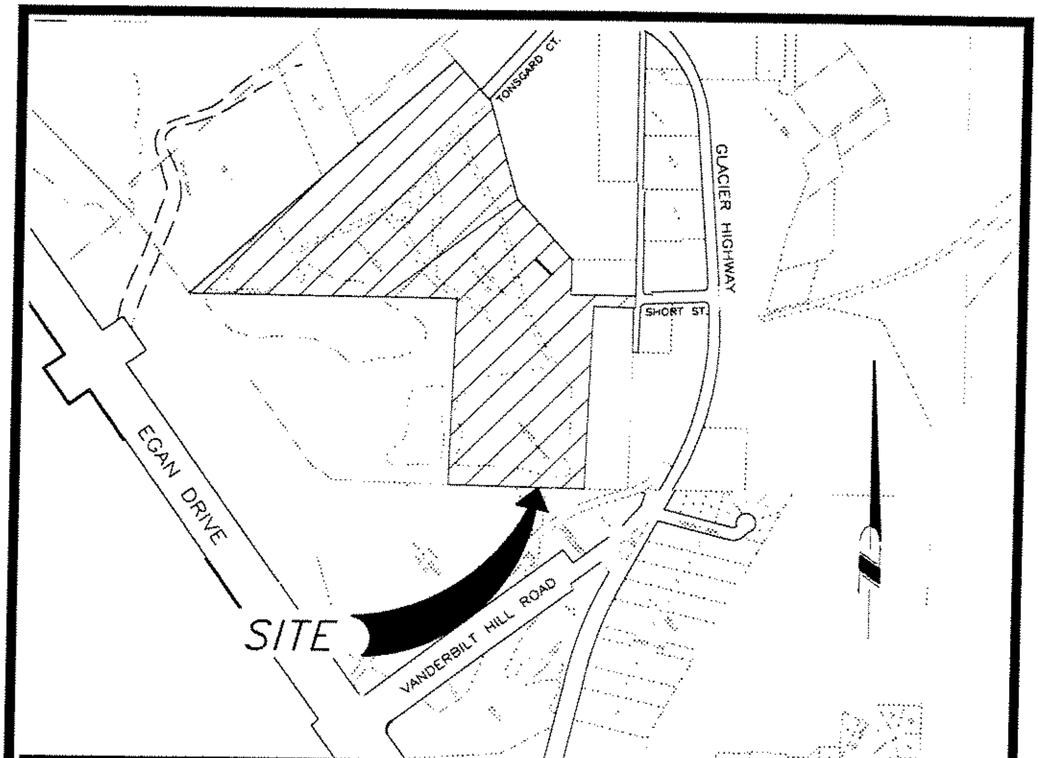
LIMITATIONS

This report was prepared in general accordance with the accepted standard of practice that existed at the time the project was performed. It should be recognized that definition and evaluation of environmental conditions is a difficult and inexact art. The soil conditions only represent the location and type(s) of tests performed. Judgments leading to conclusions and recommendations are generally made with an incomplete knowledge of the conditions present. GeoSyntec has prepared this report for the WMI's exclusive use for this particular project and in accordance with generally accepted engineering practices within the area at the time of our investigation. No other representations, expressed or implied, and no warranty or guarantee is included or intended.

This report may be used only by WMI and only for the purposes stated, within a reasonable time from its issuance. Land use, site conditions (both on-site and off-site) or other factors may change over time, and additional work may be required with the passage of time. Any party other than WMI who wishes to use this report shall notify GeoSyntec of the intended use. Based on the intended use of the report, GeoSyntec may require that additional work be performed and that an updated report be issued. Non-compliance with any of these requirements by WMI or anyone else will release GeoSyntec from any liability resulting from the use of this report by any unauthorized party.

FIGURES

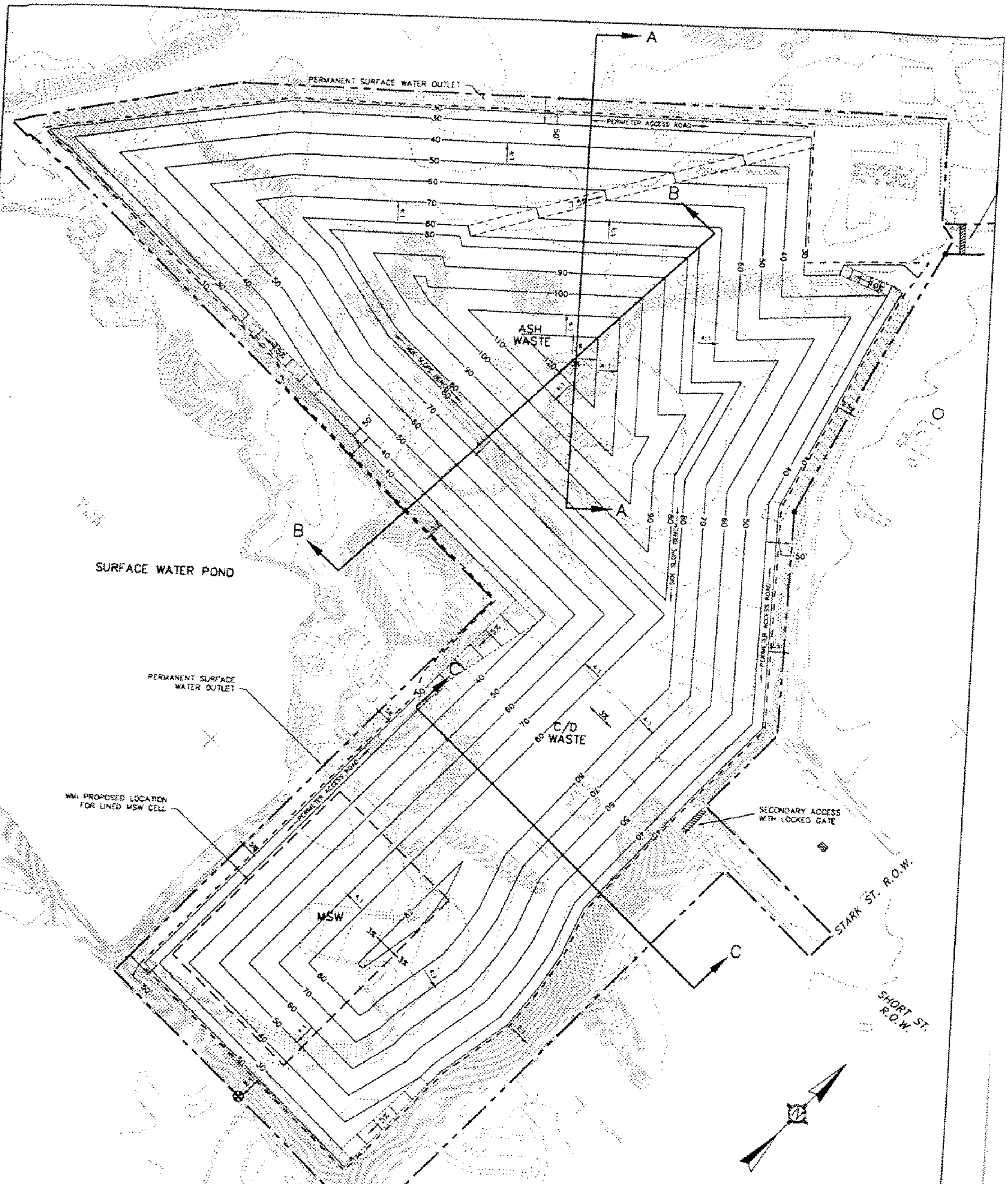


VICINITY MAP

SOURCE: CITY MAPPING
GRAPHIC SCALE
SCALE IN FEET
0 400 800 1200

GEOSYNTEC CONSULTANTS
WALNUT CREEK, CALIFORNIA

FIGURE NO.	1
PROJECT NO.	WS0230
DOCUMENT NO.	
FILE NO.	0230f1



CLOSURE PLAN
STABILITY CROSS SECTIONS
SCALE: 1"=250'

GEOSYNTEC CONSULTANTS
WALNUT CREEK, CALIFORNIA

FIGURE NO.	
PROJECT NO.	WS0230-07
DOCUMENT NO.	
FILE NO.	