PART III, ATTACHMENT 3

WASTE MANAGEMENT UNIT DESIGN

Temple Recycling & Disposal Facility

Temple, Bell County, Texas

TCEQ Permit MSW-692B

Owner/Site Operator/Permittee:

City of Temple

City of Temple
201 N. Main
Temple, Texas 76501

Operator:

WASTE MANAGEMENT

Waste Management of Texas
9708 Giles Lane
Austin, Texas 78781

STATE OF TEXAS
12/5/16
CHARLES G. DOMINGUEZ
83247
LICENSED
PROFESSIONAL ENGINEER

GOLDER ASSOCIATES INC.
Professional Engineering Firm
Registration Number F-2578

INTENDED FOR PERMITTING PURPOSES ONLY

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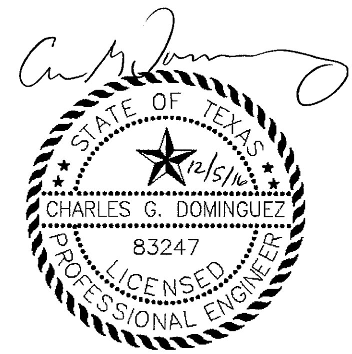

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1.0 INTRODUCTION

The Temple Recycling and Disposal Facility is an existing 269 acre Type I municipal solid waste (MSW) facility owned by the City of Temple, Texas ("City") and operated by Waste Management of Texas, Inc. ("WMTX") under Permit No. MSW-692A. The Temple Recycling and Disposal Facility site entrance is located at 706 Landfill Road, approximately 0.25 miles east of the intersection of Loop 363 and Little Flock Road in Bell County, Texas.

By way of this application, the City of Temple proposes to add 191 acres and remove 17 acres to the permitted area of the facility, for a total permitted area of 443 acres (proposed permit MSW-692B).

The facility has been designed to safeguard the health, welfare, and physical property of the people and the environment through various design considerations, which include volume and site life calculations, geotechnical analyses, liner design, leachate management, and other operational considerations. The following sections describe in detail these specific design components in accordance with 30 TAC §330.63(d)(4), and applicable sections of 30 TAC, Chapter 330, Subchapter H "Liner System Design and Operations."

2.0 OPERATIONAL CONSIDERATION

2.1 All-Weather Operations – §330.63(d)(4)(A)

All-weather site access roads consisting of compacted gravel, crushed stone, asphalt, concrete, or other appropriate road building material that can accommodate the traffic and weather conditions at the site will be provided from the facility to Little Flock Road, the public road used to access the facility, and within the facility to the unloading area(s) designated for wet-weather operation. The tracking of mud and trash onto public roadways from the site will be minimized. Truck traffic leaving the site will exit via Landfill Road, a paved road, which will help clean off the excess mud before reaching the public roadway. The site may also utilize the existing on-site wheel wash facility for trucks exiting the site.

Tracked mud and associated debris at the access to the facility on the public roadway will be removed at least once per day on days when mud and associated debris are being tracked onto the public roadway. A sweeper and/or dozer bucket may be used to clean the public roadway and on-site access roads, as needed. On-site access roads will be inspected on a daily basis. Mud will be removed from on-site roads on a daily basis during periods of rain to prevent tracking onto roads outside the facility.

Dust from on-site and other access roads will be controlled on an as-needed basis to avoid becoming a nuisance to surrounding areas. The on-site water truck will be equipped and used for dust control. Sources of water for this process may be a municipal water supply, the perimeter ditches, water collected in on-site stormwater ponds, and/or outside sources.

On-site and access roadways will be maintained on a regular basis by grading and placing additional road materials to continuously provide access to the unloading area(s).

2.2 Landfill Operational Method – §330.63(d)(4)(B), (C), and (E)

The Temple Recycling and Disposal Facility will continue to utilize an above- and below-grade area fill disposal method. The pattern of waste disposal will be governed by the area fill disposal method. Landfilling will occur below- and above-grade, depending on the status of development. The final grades are shown on Figure III-3-1. New landfill cells (Tract 5) will be developed adjacent to existing filled areas and waste placement operations will commence below-grade. The grades for undeveloped landfill cells are shown on Figure III-3-2. Construction of the proposed expansion area will begin with Cell 1, in the northwestern portion of the expansion area, and conclude with Cell 17, in the northeastern corner of the expansion. Following construction of the expansion area, the currently permitted Tract 1C, Cell 1, will be developed.

The final expanded facility will consist of a single 239-acre waste management unit, filled to the final grades shown on Figure III-3-1. Final cover placement will generally follow the sequence of development as shown in Part II, Figures II-7.1 through II-7.5, and will be ongoing as the site is developed. The landfill will be closed according to the closure plan provided in Part III, Attachment 7, Closure Plan.

Part II, Figures II-7.1 through II-7.5, Operational Fill Sequence drawings, represent the proposed interim phased development of the facility. These figures identify the proposed cells, the general sequence of excavating and filling operations, groundwater well and gas monitor probe locations, structure locations, and fencing. Waste disposal operations are projected to occur within each cell from the low end to the high end; however, this sequence may be altered due to variations in weather, types of waste received, potential safety considerations, types of equipment utilized, and unforeseen operational considerations.

Cross-sections, both longitudinal and latitudinal, through various locations of the landfill are included as Figures III-3-3.1 through III-3-3.4, in accordance with 30 TAC §330.63(d)(4)(E). The cross-sections include, as applicable, the top of the proposed fill (top of the final cover), maximum elevation of the proposed fill, top of the wastes, existing ground, bottom of the excavations, sideslopes of fill areas, gas vents or wells, groundwater monitoring wells, soil boring information, and the initial and static levels of water encountered during the boring programs. There are no levees, existing or proposed, at this facility. As shown on these cross-sections, the maximum elevation of final cover is approximately 835 feet and the maximum elevation of waste is 832 feet. The elevation of the deepest excavation is 515 feet and it occurs at the sump in Tract 5, Cell 10.

3.0 SOLID WASTE DATA – §330.63(d)(4)(D)

3.1 Estimated Rate of Solid Waste Disposal & Site Life

The current permitted disposal capacity of the Temple Recycling and Disposal Facility is approximately 20.29 million cubic yards of airspace (solid waste, plus required daily and intermediate cover). This permit amendment application proposes revisions to the facility design, resulting in a total permitted capacity of approximately 55.54 million cubic yards of airspace available for disposal operations.

As of March 2015, the total remaining airspace at the facility was approximately 7.76 million cubic yards. The proposed expansion will increase the remaining airspace to approximately 43.06 million cubic yards.

WMTX anticipates that in 2015, the landfill will receive approximately 420,000 tons of waste. As economic conditions and available landfill disposal capacity change, and as the population of the region grows, the rate of waste disposal will likely increase. WMTX anticipates that the facility will have a site life of approximately 58 years, at which time the rate of waste disposal will reach approximately 740,000 tons per year. The estimated disposal rate may change during the life of the facility and should not be considered as a limit on the rate that waste may be received by the facility. Detailed volume and site life calculations are included in Appendix III-3A.

4.0 GEOTECHNICAL ANALYSES

This section of the design report presents the results of the geotechnical field and laboratory studies and engineering analyses performed in connection with the expansion of the Temple Recycling and Disposal Facility. The geotechnical characteristics of the site are summarized herein and are based on recent geotechnical investigations performed by Golder Associates (Golder) and on information from previous geotechnical investigations of the site.

Engineering analyses performed include: settlement analysis and stability analyses of excavated slopes, protective cover, interior waste slopes, and the final-filled landfill. These calculations, along with the geotechnical properties of the subsurface described in Section 4.2 of this report, demonstrate that the soils at the site location are suitable for the intended purpose. Descriptions of the engineering properties of the subsurface and the analyses performed are presented in the following sections. Calculations performed as part of the engineering evaluation are included as individual appendices of this report.

4.1 Geotechnical Investigations

The following previous investigations were prepared for the site in support of previous permitting activities:

- 1979 – Twenty-eight soil borings drilled to characterize the original site. Borings were generally advanced to a depth of 40 feet below ground surface (ft-bgs) and covered the entire western portion of the site (Trinity Engineering).
- 1992/1993 – Twenty-six borings and piezometers were drilled to augment the Trinity Engineering characterization of the original site (Jones & Neuse).
- 1993 – Twelve soil borings were advanced to confirm previous investigations (Rust).
- 1994 – Twenty-two soil borings and piezometers were installed to investigate the site hydrogeology in order to develop the site groundwater monitoring system (Rust).
- 1996 – Ten gas monitoring probes were installed around the site perimeter to implement the gas management plan (Rust).
- 1996 – Fifteen new monitoring wells (including replacement well MW-5R) and one new piezometer were added to the monitoring well network. Only 13 borings are new as two of the new monitoring wells are converted piezometers (Rust).
- 1998 – One soil boring was drilled to confirm site stratigraphy (EarthTech [Rust]).
- 2010 – Eight new monitoring wells were installed to expand the previous groundwater monitoring network (Tetra Tech).

The investigations performed by Trinity Engineering Inc. ("Trinity"), Jones & Neuse ("J & N"), Rust, EarthTech, and Tetra Tech characterized the western portion of the site where the currently permitted disposal cells are located. The Tetra Tech investigation installed additional monitoring wells to augment the existing groundwater monitoring plan. Figure III-4-11 in Part III, Attachment 4, the Geology Report shows the locations of the previous borings and monitoring wells and Table III-4-3 in the Geology Report presents the coordinates and elevations of the previously completed borings at the site. A total of 122 borings have previously been advanced at the site.

The previously completed investigations were supplemented by additional borings in the proposed expansion area. The number and depth of additional borings were determined to meet the requirements of 30 TAC §330.63(e)(4)(A) and (B) as described in the boring plan that was approved by the TCEQ. Coordinates and elevations of the borings advanced at the proposed expansion are presented in Table III-3-1 below. Geotechnical analyses included in this report are primarily based on the data collected through the more recent investigations performed by Golder.

Table III-3-1: Coordinates and Elevations of Borings Advanced at the Proposed Expansion

Boring	Northing	Easting	Ground Elevation	Depth	Bottom Elevation
GA-01	526015.8	2947432	614.9	145	469.9
GA-02	523899.9	2950212	601.0	130	471.0
GA-03	526292.8	2947891	614.0	120	494.0
GA-04	526612	2947503	596.9	105	491.9
GA-05	525983.9	2950251	555.7	85	470.7
GA-06	525163.7	2951860	550.5	80	470.5
GA-07	525063.7	2950834	593.1	125	468.1
GA-08	524671.4	2950828	594.4	125	469.4
GA-09	524430.8	2949593	585.2	115	470.2
GA-10	523539.9	2949986	593.3	125	468.3
GA-11	523370.2	2951392	580.0	110	470.0
GA-12	522816.5	2950289	570.1	100	470.1
GA-13	523569	2949270	575.2	105	470.2
GA-14	521850.4	2950642	553.5	60	493.5
GA-15	522256.5	2951080	562.4	93	469.4
GA-16	522662.5	2948776	560.3	90	470.3
GA-17	525247.6	2950132	578.2	110	468.2
GA-18	524465.1	2950260	597.3	103	494.3
GA-19	523996.2	2951615	579.1	110	469.1
GA-20	523359.8	2950551	588.2	95	493.2
GA-21	523002.7	2949612	566.5	70	496.5
GA-22	525705.8	2950797	557.1	73	484.1
GA-23	524605.2	2951837	564.6	68	496.6
GA-24	523949.4	2950751	599.7	105	494.7
GA-25	522924.2	2951295	575.4	80	495.4
GA-26	522517.5	2949749	560.0	65	495.0
GA-27	525404.7	2949722	571.3	41	530.3
GA-28	524772.4	2951431	571.8	38	533.8

Boring	Northing	Easting	Ground Elevation	Depth	Bottom Elevation
GA-29	525162.2	2950476	589.8	46	543.8
GA-30	524829.3	2951052	588.4	44	544.4
GA-31	524816.3	2950518	600.6	56	544.6
GA-32	524786.5	2949727	583.3	39	544.3
GA-33	524513.8	2950596	601.0	52	549.0
GA-34	524326.8	2951031	594.8	47	547.8
GA-35	524259.1	2950567	601.9	59	542.9
GA-36	524326.1	2949895	587.2	46	541.2
GA-37	524090.5	2950151	595.3	49	546.3
GA-38	524062.8	2949852	586.7	41	545.7
GA-39	523566.6	2950476	595.2	61	534.2
GA-40	523767.4	2949707	581.8	39	542.8
GA-41	523352.8	2948905	570.0	40	530.0
GA-42	524062.6	2951236	590.3	60	530.3

4.2 Geotechnical Summary

The borings performed by Golder were advanced through the clay materials with either hollow-stem augers or rotary drilling with HQ coring equipment in rock. A Golder engineer logged the boreholes. The rock samples were obtained as 2.25-inch diameter core samples. The soil at the site is predominantly stiff clay, and soil samples were obtained using thin walled Shelby tubes. All borings were plugged in accordance with 16 TAC §76.72 and §76.104 and seven were completed as piezometers to provide groundwater elevation data (GA-4, GA-14, GA-22, GA-23, GA-24, GA-25, and GA-26). Unconfined compressive strength was measured using the pocket penetrometer.

4.2.1 Laboratory Tests

Laboratory testing was performed on selected samples in accordance with commonly accepted methods and practices. Water content determination was performed in accordance with ASTM D2216; Atterberg limits were performed in accordance with ASTM D4318; grain size analyses were performed using ASTM D422 and D1140; the unit weight was determined using ASTM D7263; specific gravity was determined using ASTM D854, the permeability along the vertical axis of undisturbed samples was determined in accordance with ASTM D5084 Method F (constant volume – falling head), using tap water as the permeant; and the permeability along the horizontal axis of undisturbed samples was determined in accordance with ASTM D5084 Method D (constant rate of flow), using tap water as the permeant. Shear strength testing consisted of unconsolidated-undrained (UU) triaxial compression tests in accordance with ASTM D2850.

Unconfined compressive strength (UCS) was measured based on ASTM D2938. Consolidation testing was performed in accordance with ASTM D2435.

Several samples were collected, including Shelby tube, split-spoon, and core samples. All samples were observed to determine the stratigraphy, while a total of 28 soil samples were used for laboratory testing. A summary of the soil samples and their corresponding tests is given in Table III-3-2. A description of the individual strata identified is summarized in Section 4.2.2 and the corresponding soil properties in Section 4.2.3. Collectively, 16 samples from Stratum I, 5 samples from Stratum II, and 7 samples from Stratum III were tested. These strata form the bottom and side of the proposed excavation, as well as the 30 feet below the lowest elevation of the proposed excavation.

Table III-3-2: Soil Laboratory Testing

BORING/PIT NUMBER	SAMPLE DEPTH (ft)	STRATUM	Water Content	Atterberg Limits	Sieve (3" - #200)	- #200 Sieve (Passing)	Unit Weight	Specific Gravity	Triaxial U/U	Consolidation (ILC)	Permeability	Unconfined Compression
		Test Method	D2216	D4318	D422	D 140	D 263 Method B	D854	D 850	D 435	D5084	D2938
GA-1	6–8	I	x	x		x					x	
GA-1	8–10	I	x	x		x						
GA-2	8–10	III								x		
GA-2	50–52	III	x				x				x	x
GA-2	53–55	III	x				x					
GA-2	56–58	III									x	
GA-3	4–6	I	x	x		x						
GA-3	8–10	I	x	x		x		x				
GA-3	54–56	III	x	x								x
GA-5	23–25	II	x	x		x						
GA-6	8–10	I	x	x		x			x			
GA-6	18–19	II	x	x		x						
GA-7	8–10	I									x	
GA-8	4–6	I	x	x		x						
GA-8	13–15	I	x	x		x						
GA-8	50–52	III									x	
GA-9	53–55	III									x	
GA-10	8–10	II									x	
GA-10	18–20	II	x	x		x						
GA-10	38–40	II	x	x	x							
GA-15	8–10	I									x	
GA-18	6–8	I	x	x	x							
GA-21	6–8	I									x	
GA-21	13–15	I	x	x		x		x		x		
GA-24	0–2	I	x	x		x						
GA-24	13–15	I	x	x		x						
GA-25	13–14	I	x	x		x						
GA-26	2–4	I	x	x		x			x			

4.2.2 Site Stratigraphy

The site stratigraphy has been illustrated through a series of seven cross-sections, as shown on Figures III-4-13.1 through III-4-13.7. These cross-sections utilize previous borings at the site in conjunction with new borings installed in 2014 and 2015 by Golder. The results of the subsurface investigations show that the site is underlain by three distinct strata, namely (in order from ground surface down):

- Stratum I: Stiff to hard, dark brown to tan, low plasticity clay, with high plasticity clay with organic content comprising the top of the stratum in some areas.
- Stratum II: Weathered, extremely weak to weak, tan and light gray, with orange mottling, claystone.
- Stratum III: Slightly weathered to fresh (unweathered), massive, weak to strong, light gray claystone.

Stratum I low plasticity clay with pockets of high plasticity clay and organic content is the product of Stratum II claystone weathering. The interface between Stratum I and II was not always easily defined because of the gradual transition from residual soil to rock. Also, multiple criteria were considered in determining the top of Stratum III, which included the change of rock type, change in color, and SPT N-values, change from completely/highly weathered, fissile claystone to slightly weathered/unweathered, massive claystone.

4.2.3 Soil Properties

The properties of the predominant strata at the site are summarized as follows:

4.2.3.1 Stratum I

This stratum is described as hard, dark brown, tan or gray (with frequent orange mottling), high plasticity clay. The thickness of Stratum I ranges from 0 to 28 feet. Table III-3-3 summarizes the properties of Stratum I.

Table III-3-3: Properties of Stratum I

	Minimum Value	Maximum Value	Average	Number of Tests	Test Method
Water Content	12.8	30.2	19.4	12	ASTM D2216
Liquid Limit	49	73	58	13	ASTM D4318
Plastic Limit	15	22	18	13	ASTM D4318
Plasticity Index	34	51	40	13	ASTM D4318
Liquidity Index	-0.117	0.185	0.029	13	ASTM D4318
Undrained Strength (tsf)	0.9	3.9	2.4	2	ASTM D2850
Vertical Permeability (cm/s)	4.80×10^{-8}	1.63×10^{-7}	1.1×10^{-7}	3	ASTM D5084
Horizontal Permeability (cm/s)	3.91×10^{-8}	—	—	1	ASTM D5084

4.2.3.2 Stratum II

Stratum II consists of completely weathered to moderately weathered, fissile and friable, gray to light gray, extremely weak to weak claystone. Fossilized shells and pyrite nodules were identified in some samples. The Rock Quality Designation (RQD) was generally greater than 50 percent. The top of Stratum II was found between approximately elevation 517 feet and 601 feet, with a thickness ranging between 0 and 49 feet. The average top of the layer is approximately at elevation 563 feet. Table III-3-4 summarizes the properties of Stratum II.

Table III-3-4: Properties of Stratum II

	Minimum Value	Maximum Value	Average	Number of Tests	Test Method
Water Content	9.7	16.8	13.8	4	ASTM D2216
Liquid Limit	44	76	58	4	ASTM D4318
Plastic Limit	16	27	19	4	ASTM D4318
Plasticity Index	28	49	39	4	ASTM D4318
Liquidity Index	-0.208	-0.043	-0.128	4	ASTM D4318
Vertical Permeability (cm/s)	1.57×10^{-8}	-	-	1	ASTM D5084
Horizontal Permeability (cm/s)	8.30×10^{-8}	6.40×10^{-6}	9.08×10^{-7}	12	ASTM D4044

4.2.3.3 Stratum III

Stratum III is slightly weathered to fresh, massive, light gray, weak to strong, claystone. Rock cores were generally free of joints and discontinuities, excepting few locations. The RQD was generally greater than 80 percent and often 100 percent, as shown on the borehole logs in Appendix III-4B of the Geology Report. The top of Stratum III was found between approximately elevation 506 feet and 565 feet. The average top of the stratum is approximately 533 feet. The bottom of this stratum was not identified. Table III-3-5 summarizes the results of properties from the tested samples in this stratum.

Table III-3-5: Properties of Stratum III

	Minimum Value	Maximum Value	Average	Number of Tests	Test Method
Unconfined Compressive Strength (tsf)	81.1	92.5	88.2	2	ASTM D2938
Vertical Permeability (cm/s)	2.0×10^{-9}	3.3×10^{-8}	1.69×10^{-8}	3	ASTM D5084
Horizontal Permeability (cm/s)	2.29×10^{-9}	—	—	1	ASTM D5084

4.3 Engineering Analyses

Analyses were performed to assess the performance of the landfill with respect to settlement and slope stability. Each of these analyses is described in detail in the following sections. The locations of the cross-sections analyzed are shown on Figure III-3-4.

4.3.1 Settlement Analysis

Facility floor settlement may only occur in Strata I and II because the claystone in Stratum III is considered to be incompressible. Review of the excavation plan indicates that much of Strata I and II will be removed prior to construction of the liner system and that much of the Temple Recycling and Disposal Facility floor will be founded on a thin layer of remaining Stratum II overlying incompressible Stratum III claystone. For this analysis, settlement critical cross-sections are cut through two sections of the Temple Recycling and Disposal Facility with the thickest section of Stratum II, which will remain as part of the Temple Recycling and Disposal Facility floor after excavation. Intermittent points along the critical cross-sections are analyzed for settlement and post-settlement quantities with which to define slopes. The cross-section locations are referred to as Line A, located in Tract 5, Cell 14 and Line B, located between Tract 5, Cells 12 and 13. Both lines begin at the facility perimeter and progress toward the facility center where the proposed final elevation is highest.

The settlement analyses indicate that the maximum total settlement of Stratum II is approximately 0.3 feet. The post-settlement floor grades maintain positive drainage and allow the leachate to drain towards the leachate collection system under the conditions analyzed. The results of these analyses are presented in Appendix III-3B.

4.3.2 Stability Analysis

Slope stability analyses were performed using limit equilibrium methods to assess the stability of the proposed landfill. In particular, stability of the proposed excavated landfill sideslopes, stability of the protective cover on landfill sideslopes, stability of the interior waste slopes, overall stability of the final filled landfill, and stability of the final cover system were evaluated.

In general, the analyses consist of the following:

- Characterization of the critical cross-section (e.g., the geometry, geology, geosynthetic interfaces, and groundwater conditions)
- Selection of appropriate strength parameters
- Analysis under anticipated critical conditions

The analyses are summarized in the following sections.

4.3.2.1 Stability Analysis of Excavated Slopes

A stability analysis was performed to consider potential failure surfaces after excavation to build the waste cells. Based on a review of the design grades, the longest proposed excavated slope occurs within Tract 5, Cell 16 and 17. The cross-section in this area is labeled Section A in Figure III-3-4 and consists of a 3H:1V slope from approximate elevation of 593 feet to the toe at an approximate elevation of 541 feet .

A conservatively generalized subsurface stratigraphy was developed for this analysis based on available laboratory test data and field data from boring logs. The subsurface stratigraphy has been developed using three layers from top to bottom: a residual clay layer, a weathered claystone layer, and an unweathered claystone layer. The analysis was conducted for both total stress and effective stress conditions. The stratigraphy and strength data for the in situ soils were developed from borings and laboratory testing described above.

Groundwater level was conservatively assumed to be approximately 10 ft-bgs, based on the available piezometer data. Within the excavation, the phreatic surface was conservatively assumed to correspond to the excavation grade.

SLIDE Version 6.030, an integrated slope stability analysis program for personal computers, was used for the analysis. The slope geometry for the critical section of the landfill was input into the program along with the unit weight and strength parameters. Potential failure surfaces were analyzed and the minimum factor of safety was computed. The calculations are presented in Appendix III-3C-1.

The factor of safety against instability for the slope analyzed is 8.0 for the total stress condition and 4.5 for the effective stress condition. These values indicate the slopes will be stable and are therefore adequate. The factors of safety will also increase as waste is placed within the landfill cells.

4.3.2.2 Stability of Protective Cover on the Cell Sideslopes

An analysis of the stability of the cell sideslope liner system was performed using an infinite slope analysis. Based on a review of the literature and unpublished data on similar materials under similar loading conditions, the critical interface shear strength within the sideslope liner system was estimated to be 24 degrees. The maximum length of slope is 200 feet with a 3H:1V slope, as shown on Figure III-3-4.

The analyses, included as Appendix III-3C-2, indicate that, provided the geocomposite drainage layer is adequate to convey drainage without building up pore water pressures, the factor of safety is found to be at least 1.34, and likely to be higher due to the use of conservative parameters in the analysis. This factor of safety is considered adequate and will increase as waste is placed within the cell.

Additional analyses (included in Appendix III-3D) were performed demonstrating that a standard double-sided geocomposite drainage layer will have a transmissivity adequate to convey water infiltrating through the protective cover over the maximum sideslope length.

4.3.2.3 Stability of the Interior Waste Slopes

Interior waste slope stability analyses were performed using the limit equilibrium slope stability method to determine the factor of safety against sliding along the liner. Based on a review of the floor grades and filling sequence, it was identified that the liner in Tract 5 Cell 9 is the most critical case, where the filling and floor slope occur in the same direction with no buttress effect from existing waste or the floor gradient. A review of the literature and data on the liner system components under similar loading conditions indicate that the weakest interfaces in the liner system occur between the geonet and the smooth geomembrane along the cell floor and between the textured geomembrane and the geocomposite along the sideslopes of the cell. These interfaces were used in the analysis.

The analyses were performed using SLIDE. Two possible waste filling slopes were considered: 1) continuous 3H:1V temporary waste slopes with no benches; and 2) continuous 4H:1V temporary waste slopes with no benches.

The strength parameters were either conservatively chosen from published studies or based on test results for similar conditions, and the slope geometries correspond to the least stable conditions. Considering a minimum factor of safety of 1.3, temporary waste slopes at 3H:1V can be raised to a maximum height of 160 feet without benches. Slopes exceeding this height should be independently evaluated for stability. Temporary waste slopes at 4H:1V can be raised to over 300 feet of slope without reaching a minimum factor of safety of 1.3.

In addition, the operational fill sequence was reviewed and the most critical waste slope, considering the maximum height of exposed waste and floor grades, was also analyzed. Section A on Detail 4 of Figure III-3-4 shows the analyzed section with the highest exposed slope that occurs during Operation Fill Sequence, Phase 5 (from Figure II-7.5). The weakest interfaces in the liner system occur between the geonet and the smooth geomembrane along the cell floor, between the textured geomembrane and the geocomposite along the sideslopes of the cell, and between the smooth geomembrane and geosynthetic clay liner (GCL) on the overliner. Using these interfaces, a minimum factor of safety of 1.3 was obtained for this waste slope indicating adequate stability. The calculations are presented in Appendix III-3C-3.

4.3.2.4 Stability of Final Filled Configuration

Final filled configuration stability analyses were performed using limit equilibrium methods to determine the factors of safety against sliding or failure. Based on a review of the design grades, the reasonable worst-case configuration was assumed to consist of a section along Tract 5, Cell 2 and Tract 5, Cell 6, having

3H:1V excavation sideslopes and 4H:1V final cover slopes to a crest elevation of approximately 820 feet, with a maximum fill elevation of about 835 feet above mean sea level (ft-msl). This cross-section is labeled Section C in Figure III-3-4. The calculations are presented in Appendix III-3C-4. Using strength parameters that are conservatively estimated or based on test results for similar conditions, and the reasonable worst case configuration, the analysis indicates that the final-filled configuration will be stable with minimum factors of safety of 1.52 for block sliding and 2.82 for circular failure.

4.3.2.5 Stability of Final Cover System

A stability analysis of the final cover liner system was performed using an infinite slope analysis to estimate the potential for sliding to occur following closure of the landfill cells. A worst-case section, consisting of a 1,300-foot long, 25-percent slope was analyzed. Based on a review of the literature and unpublished data on similar materials under similar loading conditions, the critical interface shear strength within the final cover liner system was estimated to be 21 degrees.

The analyses are included in Appendix III-3C-5 and indicate that, provided the geocomposite drainage layer is adequate to convey drainage without building up pore water pressures in the geocomposite, the factor of safety against sliding will be approximately 1.54.

Additional analyses (included in Appendix III-3D) were performed to determine the geocomposite drainage layer transmissivity required to adequately convey surface water infiltration over the maximum final cover slope length. If the minimum measured transmissivity value reported in Appendix III-3D is not met, the maximum flow length must be reduced (i.e., the geocomposite drainage layer must be “daylighted”) in direct proportion to the ratio of the actual measured transmissivity and the required measured transmissivity. A detail depicting “daylighting” is included as detail 2 in Attachment 7, Closure Plan, Figure III-7-2.1, Final Cover Details.

4.3.2.6 Summary

The results of the stability analyses indicate that the proposed slopes are stable under the conditions analyzed. For each condition analyzed, the minimum calculated factor of safety exceeds the recommended factor of safety.

5.0 LINER DESIGN

The Temple Recycling and Disposal Facility has multiple liner system designs that have been utilized or are proposed for future disposal cell development. The following sections discuss these designs as well as the Liner Quality Control Plan (LQCP), which specifies construction methods for soil and geosynthetic composite liner systems.

5.1 Disposal Cell Liner System Design

One recompacted clay lined cell (referred to herein as the pre-Subtitle D cell) was developed in the southeast corner of Tract 3. This cell has a 3-foot thick compacted clay liner with no leachate collection layer. The design and analysis of the overliner system is discussed in Section 7.

The liner systems for the existing and proposed Subtitle D cells consist of 2 feet of compacted low-hydraulic conductivity soil, a 60-mil high-density polyethylene (HDPE) geomembrane liner, granular or geosynthetic drainage layers at the base of the landfill, and geotextiles for liner protection and fines filtration and drainage on slopes, and 2 feet of protective cover soil. The locations of the pre-Subtitle D and Subtitle D liner systems are shown on Figure III-3-2. Liner details are included on Figures III-3-5.1 through III-3-5.4. Cross-sections through the perimeter of the horizontal expansion area (Tract 5) and along the connection between the existing permitted landfill and Tract 5 are shown on Figures III-3-9.1 and III-3-9.2, respectively.

The landfill excavation extends below the seasonal high water table. Toe drains and a geocomposite underdrain along the sideslopes will be installed to control groundwater. The underdrain will be maintained and operated until sufficient ballast is in place to resist the uplift pressures below the liner system. The underdrain analyses are included in Appendix III-3F-3. The underdrain system layout and details are shown on Figures III-3-6 and III-3-5.4, respectively.

5.2 Liner Quality Control Plan

The composite liner system for the remaining unconstructed cells will be installed, and all construction will be executed in accordance with the Liner Quality Control Plan (LQCP), as required by §330.63(d)(4)(G) and §330.339. The LQCP is included in Appendix III-3F. Field sampling and testing will be performed by a qualified professional experienced in geotechnical engineering and/or engineering geology or a qualified engineering technician under his/her direct supervision, in accordance with the provisions of the Texas Engineering Practice Act, and other state laws and regulations. Prior to the disposal of solid waste in any new disposal area, a Soil Liner Evaluation Report (SLER) and a Geomembrane Liner Evaluation Report (GLER) will be submitted to TCEQ for review and approval in accordance with 30 TAC §330.341.

6.0 LEACHATE MANAGEMENT

This section of the design report presents the design and operation of the leachate management system for the Temple Recycling and Disposal Facility. Leachate, gas condensate, and contaminated water are produced in the normal course of landfill operations. Leachate is defined in 30 TAC §330.3 as a liquid that has passed through or emerged from solid waste and contains soluble, suspended, or miscible materials removed from such waste. Gas condensate is defined as the liquid generated as a result of any gas recovery process at a MSW facility. Contaminated water is defined as water that has come into contact with waste, leachate, or gas condensate. The following sections describe the management of leachate, gas condensate, and contaminated water generated at the facility and the design of the facility leachate collection system.

The pre-Subtitle D cell was constructed without a leachate collection layer. All of the other existing cells were constructed with a leachate collection consisting of either a granular drainage layer (Cells 1 and 2 of Tract 3) or a geonet drainage layer (all subsequent cells).

In accordance with the requirements set forth in 30 TAC §330.333, the leachate collection and associated leachate removal systems shall be:

- Constructed of materials that are chemically resistant to the leachate expected to be generated
- Of sufficient strength and thickness to prevent collapse under the pressures exerted by overlying wastes, waste cover materials, and by equipment used at the facility
- Designed and operated to function through the scheduled closure and post-closure period of the facility

Additionally, 30 TAC §330.333 requires that the following factors be considered:

- Estimated rate of leachate removal
- Capacity of sumps
- Pipe material and strength
- Pipe network spacing and grading
- Collection sump materials and strength
- Drainage media specifications and performance
- Demonstration that pipes and perforations will be resistant to clogging and can be cleaned

In accordance with 30 TAC §330.207(a), all liquids resulting from the operation of solid waste facilities will be disposed of in a manner that will not cause surface water or groundwater pollution. Leachate, gas condensate, contaminated surface water, and contaminated groundwater will not be discharged into waters of the state or nation, including wetlands, in violation of any requirements of 30 TAC §330.15(h), §26.121 Texas Water Code, the Clean Water Act, and the National Pollutant Discharge Elimination System

(NPDES) requirements. 30 TAC §330.177 states that leachate and gas condensate may be recirculated in a MSW landfill unit with a composite liner system and a leachate collection system.

6.1 Contaminated Water Management and Minimization

Minimization of leachate and contaminated water will be achieved primarily by implementing best management practices (BMPs) to minimize rainfall run-off contacting waste at the working face and to minimize the amount of water passing through or otherwise emitted from waste. Practices to be utilized to minimize leachate and contaminated water will include landfill construction methods, surface water management practices, and cover practices.

6.1.1 Landfill Construction

During cell construction and continued site development, rainfall from outside of the active fill areas will not be allowed to commingle with leachate, gas condensate, contaminated surface water, or contaminated groundwater to the extent practicable. Facility operations will incorporate the use of temporary or permanent berms, culverts, pumps, pipes, and hoses, in conjunction with grading of areas outside the excavation areas, to manage surface water and lessen the potential for incidental contact.

Uncontaminated stormwater run-off will be directed on the landfill by grading an area to collect and channel stormwater run-off through temporary holding areas and/or sumps. Stormwater will then be pumped, as required, to the perimeter drainage system, which is routed to the facility's surface water detention ponds. Staged development of disposal areas in a number of separate phases will further minimize the potential for generation of contaminated water.

6.1.2 Surface Water Management

Active areas of the facility will be graded to minimize surface water run-on into and through the working face, which will be kept as small as practical. Temporary diversion ditches and berms will be constructed and maintained upslope of the active area to direct potential run-on away from the working face. Uncontaminated water will be diverted to the perimeter drainage system and/or will be collected by containment berms, culverts, sumps, or low areas, and pumped to the perimeter drainage system via temporary pumps, hoses, and pipes. Waste placement within disposal cells typically progresses from lower to higher elevations. Composite liners in new disposal cells will incorporate controls such as berms and rain flaps upslope from the active waste disposal area to further segregate uncontaminated rainfall from leachate in parts of the cell that have not yet received waste. These management practices require that only the stormwater, which falls directly into the uncovered active area or areas that have received daily cover, be considered contaminated, and therefore must be collected for treatment.

Other stormwater controls, such as run-on and run-off berms located upgradient and downgradient of the working face, respectively, will be used to further minimize stormwater that would be classified as contaminated. The contaminated stormwater run-off berms will be used to collect stormwater downslope of the working face and the collected stormwater will be removed via portable pumps, hoses, and pipes and managed separately from leachate. Contaminated stormwater will not be recirculated.

Run-on and run-off controls will be utilized in active disposal areas to minimize the potential for stormwater contact with waste and subsequent contamination. The facility expansion will continue to implement these controls for the active working faces of the active disposal areas. The working face of each active disposal area will be encompassed by a run-on berm (top berm) and a run-off berm (toe berm) to segregate and minimize potentially contaminated stormwater and to divert non-contact stormwater. The containment berms will be designed to accommodate the 25-year, 24-hour storm, the equivalent of an 8-inch rainfall event. The top berm is designed to accommodate upstream watersheds that flow towards the working face, and divert the collected uncontaminated stormwater around the working area for discharge through a permitted stormwater outfall. The toe berm is designed to accommodate storage of stormwater that has potentially contacted the open working face or daily cover areas. The berm height requirements and design configurations are detailed in Part III, Attachment 2.

During the ongoing daily operations of the active disposal areas, the facility will implement the active area berm design requirements detailed in Part III, Attachment 2. As a result of progressive disposal and filling operations, ongoing berm extension/construction may be required to accommodate adequate stormwater run-on diversion (top berm) and proper containment of run-off contact waters (toe berm). The daily disposal operations will include an evaluation of the existing containment berms' capability to manage stormwater run-on and run-off. The Temple Recycling and Disposal Facility will manage the active working face in accordance with the Part III, Attachment 2 to maintain adequate top and toe berm heights to ensure proper surface water protection due to a 25-year, 24-hour storm event.

6.1.3 Cover Practices

As landfill operations progress, an intermediate layer of soil at least 1-foot thick (includes 6 inches of new cover over previously placed daily cover soil) will be placed over areas proposed to be inactive for a period greater than 180 days and that are not filled to final grade. Vegetation will be established to promote evapotranspiration, limit erosion, and reduce the amount of infiltration. Eroded slopes will be repaired and ponding on the landfill will be mitigated.

6.2 Leachate Management System

The leachate management system is designed and operated to collect and remove leachate from the top of the liner at the floor of the landfill cell, maintain leachate head levels at or below 30 cm above the composite liner, channel leachate to designated collection sumps, effectively manage the leachate through recovery and recirculation into the waste mass, and reduce leachate through evaporation in collection ponds or disposal. The system is designed to eliminate potential migration of landfill leachate into the environment and to meet the requirements of 30 TAC §330.333, namely:

- Constructed of materials that are chemically resistant to the leachate expected to be generated
- Of sufficient strength and thickness to prevent collapse under the pressures exerted by overlying wastes, waste cover materials, and by any equipment used at the facility
- Designed to function through the scheduled closure and post-closure period of the facility

The leachate collection system at the facility consists of several different design configurations, depending on the time the disposal unit was constructed and the type of lining system in place. The Subtitle D cells have composite liners, leachate collection layers, protective cover, sumps, and risers. The leachate collection and removal system description provided in Section 6.2.1 applies to the Subtitle D-lined areas. The locations of Subtitle D cells are shown on Figure III-3-2.

The pre-Subtitle D cell within the permitted waste area consists of a 3-foot thick compacted clay liner with no leachate collection system.

Liner systems and leachate collection and removal systems constructed for Subtitle D cells, in accordance with 30 TAC §330.331 and §330.333, are composed of the following elements:

- Compacted soil liner
- Geomembrane liner
- Leachate collection and removal system
    - Leachate drainage layer
    - Leachate collection pipes
    - Sumps, pumps, and risers
    - Leachate transfer equipment
    - Leachate storage facilities

The liner/leachate collection systems for the existing and proposed Subtitle D cells consist of 2 feet of compacted low-hydraulic conductivity soil, a 60-mil HDPE geomembrane liner, granular or geosynthetic drainage layers along the base and side slopes of the landfill, geotextiles for liner protection and fines filtration and drainage on sideslopes, 2 feet of protective cover soil, perforated collection pipes encased in drainage material, and leachate collection sumps. The liners are constructed on slopes designed to promote positive drainage to perforated collection pipes, where it is directed to the cell sumps for removal.

The geotextiles used for geomembrane liner protection, for filtration of fines around the leachate collection pipes and sumps, and which comprise the geotextile portion of the geocomposite for the drainage layer, utilize 100 percent continuous-filament polyester or polypropylene. Extensive testing, including EPA 9090 for chemical resistance, has demonstrated that polyester and polypropylene are relatively inert to a wide range of chemical classes encountered in soil and typical leachates. All other components of the leachate collection system (i.e., the geonet in the geocomposite and the pipes) consist of HDPE, which has also been demonstrated to be inert to typical leachates.

6.2.1 Leachate Collection System Design and Operation

6.2.1.1 Leachate Drainage Layer

The basegrades of the liner system and leachate drainage layer of future cells follow the excavation grades, which were set at or near the Stratum II/III interface (i.e., the top of the unweathered rock). As a result, the slopes vary within and between cells. The drainage layer on the landfill floor will consist of single-sided geocomposite or individual geonet and geotextile layers. The sideslope geosynthetic drainage layer will consist of a double-sided geocomposite layer.

The leachate collection and removal system (LCRS) on the cell floor area is designed to limit the maximum leachate depth on the bottom liner to less than 30 cm, in accordance with 30 TAC §330.331(a)(2). The allowance is a design standard that may be exceeded for relatively short periods of time during the active life of the unit, as acknowledged by the USEPA.

A demonstration of the LCRS performance, included in Appendix III-3E-1, was conducted as follows.

- The HELP Model (Hydraulic Evaluation of Landfill Performance, US Army Corps of Engineers, Waterways Experiment Station, Version 3.07, November, 1997) was used to determine the leachate production rate (impingement rate) for various conditions during the life of the landfill.
- The transmissivity needed to maintain a hydraulic head less than the thickness of the geocomposite drainage layer was determined for various combinations of slope and slope lengths for each condition modeled in HELP.

To limit leachate ponding on the protective cover, the drainage material surrounding the leachate collection system pipes will extend through the protective cover forming chimney drains, as shown on the Liner and Leachate Collection System Details, Figure III-3-5.4.

Design requirements for the geocomposite drainage layer are provided in Appendix III-3E-1 (cell floor) and Appendix III-3D-1 (sideslopes). The required geocomposite transmissivities are 8.42×10^{-4} m^2/sec and 3.90×10^{-5} m^2/sec on the cell floor and on the sideslopes, respectively. Results indicate that the required geocomposite transmissivity for the cell floor is higher than the sideslopes. The leachate flow from the overliner area is not considered for cell floor and sideslopes in Tract 5 Cell 1, because the overliner leachate will be directed into the leachate collection pipes through pipes from the overliner area, and the geocomposite layer within Tract 5, Cell 1 will not receive any of this leachate.

6.2.1.2 Leachate Collection Pipes

Perforated 6-inch HDPE leachate collection pipes will be installed in trenches in each cell at a minimum grade of 1 percent to remove leachate from the drainage layer. The leachate collection pipes discharge into sumps located near the base grade low points of each cell, at the toe of the sideslope, as indicated in Figure III-3-2. The long-term landfill settlement analysis, as presented in Appendix III-3B, indicates that the minimum post-settlement grade along the leachate collection pipe would be 0.7 percent.

The leachate collection pipes in all future disposal cells will consist of 6-inch SDR 11 or 17 HDPE pipes. No portion of the leachate piping system is designed to penetrate the composite liner.

The leachate collection system includes details designed to ensure that the system is resistant to clogging, as specified in Figures III-3-5.1 through III-3-5.6. The leachate collection pipes will be encased in drainage material. Additionally, the drainage material surrounding the pipes will be wrapped in a geotextile filter.

Cleanouts will be provided adjacent to the riser pipes to allow access to the leachate collection pipe. The existing and planned cleanouts are and will be constructed of 6-inch diameter non-perforated HDPE pipe joined to the perforated collection pipe in the sump. The 6-inch pipe size allows sufficient cross-sectional area for effective cleaning. Leachate system maintenance is discussed in Section 6.2.1.9.

The leachate collection pipes and cleanout access pipes were designed for long-term performance and were evaluated for flow capacity and stability. The results of the HELP modeling were used to assess the ability of the proposed pipes to convey the maximum anticipated leachate generated in the landfill. Stability analyses of the pipes were performed and considered wall buckling, wall crushing, and ring deflection. These calculations are provided in Appendix III-3E-2.

6.2.1.3 Leachate Collection Sumps

Leachate entering the drainage layer and collection pipes is subsequently discharged into collection sumps located at the perimeter of the landfill, where it is pumped to temporary holding tanks or to the leachate evaporation ponds. Sump inverts are approximately 3 feet below the leachate collection pipe invert to allow accumulation of leachate. The capacity of the sumps was calculated to be approximately 3,454 gallons, as provided in Appendix III-3E-3. Under worst-case conditions (peak daily leachate generation rate with 10 feet of waste in place over the largest cell) and assuming no pumping discharge, the sump is anticipated to be filled in approximately 0.8 hours.

The sumps will be constructed to dimensions and will use the materials indicated on Figure III-3-5.2. The sump floor is lined with compacted low hydraulic conductivity soil overlain by a 60-mil HDPE liner and then by geonet or geocomposite. The lined sump is filled with drainage material encased in nonwoven geotextile.

6.2.1.4 Leachate Pump and Riser System

Sump riser pipes are located along the disposal area perimeter to provide a means of removing leachate from the leachate collection sumps. Risers consist of 18-inch diameter SDR 11 HDPE pipe, of which the lower portion of the riser within the sump is perforated by 3/8-inch diameter holes that allow leachate to flow to the pumps. When repairs or pump replacement is required, this diameter riser pipe allows for easy access to the submersible pumps.

Leachate removal from sumps is typically performed using an automatically operated pumping system. Pump types and the method of activation used may change throughout the life of the facility as equipment is updated. The activation elevation may vary depending upon actual pump operating life and type of pump(s) used, with a goal of maintaining less than 30 centimeters (cm) (12 inches) of head on the liner system. Pump maintenance or repair will be performed expeditiously to ensure that leachate head levels do not exceed the 30-cm regulatory compliance level.

The current pumps are set such that leachate is typically conveyed via pipes into the leachate evaporation ponds or directly into the leachate recirculation network. For future cells, the HELP model predicted that the leachate generation rate is anticipated to vary from a high of approximately 1,000 cubic feet/acre/day (7,500 gal/acre/day) and average approximately 47,000 cubic feet/acre/year (352,000 gal/acre/year). For the largest proposed cell, this translates to approximately 12,000 gal/day for the most critical condition (open condition).

6.2.1.5 Leachate Transfer

Leachate may be transferred from the leachate collection system using three methods: 1) via piping to a recirculation network in the landfill; 2) via tanker to a recirculation area or transported off-site; or 3) via piping to an evaporation pond then to a sanitary sewer system. All leachate removed from the site is documented in the site's operating record.

6.2.1.6 Leachate Storage

There are two existing leachate ponds that have been approved by the TCEQ:

- Pond A: A modular steel tank constructed of a reinforced steel frame with a geomembrane liner underlain by a clay layer is currently in use. For operating purposes, this tank is called Pond A. Pond A has a capacity of 270,000 gallons, excluding freeboard, and was authorized by a permit modification approved by TCEQ in early 2002.
- Pond B: On November 17, 2004, TCEQ approved a permit modification to the facility's Leachate and Contaminated Water Plan authorizing the construction of up to two additional in-ground ponds labeled Pond A and Pond B. Only one of these in-ground ponds has been constructed to date. For operating purposes, the pond constructed is called Pond B. In accordance with 30 TAC §330.207(b), Pond B was constructed with a two-foot thick clay liner overlain by 60-mil HDPE liner and one foot of freeboard for the 25-year, 24-hour rainfall event will be maintained.

Both leachate storage and evaporation ponds are constructed with geomembrane and compacted clay liners. Both ponds have open tops and one foot of freeboard will be maintained to prevent overtopping from a 25-year, 24-hour rainfall event. Leachate storage time in these ponds will depend on evaporation and leachate recirculation volumes. The locations of the leachate ponds are shown on Part III, Attachment 1, Figure III-1-2. Construction and design details of these leachate ponds are provided in Part II, Appendix IIF.

Leachate from these ponds is either evaporated, recirculated or discharged to the City of Temple sanitary sewer system.

6.2.1.7 Leachate Treatment and Disposal

Leachate that is not recirculated will be managed as discussed in Sections 6.2.1.5 and 6.2.1.6 above. Disposal of leachate, as required, will be done at an appropriately authorized facility.

6.2.1.8 Leachate Recirculation

In disposal cells containing a standard Subtitle D liner system (i.e., a compacted clay/geomembrane composite) and leachate collection system, leachate and gas condensate may be recirculated back into the waste. Leachate recirculation may consist of spray application during dry conditions using a portable tank(s) at the active face, injecting leachate through a perforated pipe or well buried in the refuse, or discharging leachate in an area excavated into waste and backfilled with highly permeable material (e.g., gravel or tire chips). Leachate will not be recirculated to the active face during rainy or wet periods. The automated leachate pumps will prevent leachate depth greater than 30 cm from accumulating over the liner and will be in operation during leachate recirculation activities. The application of leachate will be in a manner so as to not cause leachate accumulation, ponding, or other operational problems. The Temple Recycling and Disposal Facility personnel will ensure that recirculating will not result in vectors, odor, or other nuisance conditions.

6.2.1.9 Leachate Collection System Maintenance

Maintenance performed on the leachate collection system includes repairing leachate pumps as necessary with a goal of maintaining less than 30 cm of head on the liner under design conditions. Leachate collection lines may need periodic cleaning or flushing to dislodge biological mass or fines that may have clogged the pipe perforations. This cleaning will typically be performed by an outside contractor with the necessary equipment sufficient to flush the length of leachate collection lines. The longest leachate collection pipe is about 1,550 feet long. Based on current industry practice and prior experience, pipes of this length can be cleaned. Repairing or replacing submersible leachate pumps, periodically cleaning the leachate header pipes, and monitoring the system will ensure that the leachate collection system will function throughout the closure and post-closure period.

6.3 Leachate Collection System Recordkeeping and Documentation

Records will be retained in accordance with Part IV, the Site Operating Plan, to include records of the unit design documentation (as-builts) for the placement of the leachate and gas condensate at the facility. As-built drawings of the leachate collection system, as indicated in the approved Site Development Plan, will be maintained at the landfill or other approved location.

7.0 OVERLINER SYSTEM DESIGN

A portion of the proposed expansion will be located over a pre-Subtitle D area. In accordance with 30 TAC §330.331(a), it will be necessary to install a composite liner and a leachate collection system over the existing waste and under the new waste. The associated design for the vertical expansion over the unlined area is referred to as the "overliner" design in this permit amendment application.

The proposed liner and leachate collection system for the overliner area consists of, from top to bottom:

- 2-foot protective cover soil
- Double-sided geocomposite drainage layer
- 60-mil linear low-density polyethylene (LLDPE) geomembrane liner, textured on both sides
- Geosynthetic clay liner (GCL)

In addition, a grading layer may be placed on top of the existing intermediate cover over the existing waste prior to constructing the overliner system to provide a smooth subgrade for GCL placement.

The overliner grading plan and liner system details are shown on Figures III-3-7 and III-3-8, respectively.

The overliner design will include three grade zones:

- A 12 percent slope at the toe (tie-in between overliner and liner for Tract 5, Cells 1, 16, and 17).
- Followed by a 3H:1V slope up to about elevation 680 feet (which is the maximum waste elevation along the slope in 2015, per site topographic surveys).
- Followed by a 15 percent slope to the crest and limits of the overliner. This zone corresponds to waste placement after 2015, but prior to the overliner construction, and below the maximum elevation authorized under TCEQ Permit MSW-692A.

The overliner will extend at least 25 feet beyond the vertical projection of the limits of existing floor liner systems for constructed Tract 3, Cell 1 and Tract 4, Cells 2B and 3A.

The installation procedures for the overliner system are described in the "Overliner System Quality Control Plan," included as Appendix III-3H to this document.

The following sections present the engineering analyses performed to support the overliner design and include: a "point of compliance" demonstration, stability, settlement, liner overliner system strain analysis (mechanical response), leachate collection system design, and gas collection and control system considerations.

7.1 Point of Compliance Demonstration

The overliner system was evaluated to demonstrate that the proposed alternate liner will provide a level of groundwater protection that is greater than or equal to the level of protection provided by a composite liner system. The evaluation presented in Appendix III-3G-3 indicates that substituting the clay component with a GCL in the overliner liner system will provide a greater or equivalent level of groundwater protection at the Temple Recycling and Disposal Facility. In addition, fate and transport modeling performed on the alternate liner system demonstrates that the maximum contaminant levels detailed in 30 TAC §330.331(a)(1) will not be exceeded at the point of compliance as a result of leakage through the overliner.

7.2 Settlement Analysis

The purpose of the overliner system settlement analysis is to:

- Show the that positive drainage is maintained for the overliner system leachate collection layer
- Verify that the strain induced on the overliner system components due to settlement is within acceptable limits

Settlement for the overliner system will primarily be the result of compression of the underlying old waste and, to a lesser extent, consolidation of the foundation soil layers due to increased loads from the new waste and final cover placement.

Old waste settlement consists of two components: 1) time-dependent secondary compression (or creep), and 2) primary settlement caused by the stress increase from new waste and final cover. Secondary compression within the foundation material will be very small; therefore, only consolidation settlement was evaluated below the landfill.

Settlement below the overliner was estimated and used to determine the post-settlement grades of the overliner. As shown in the settlement calculations included in Appendix III-3G-1, the minimum post-settlement grade in the direction of leachate flow is approximately 8 percent; therefore, positive drainage will remain at the end of the 30-year post-closure period.

The proposed overliner system was analyzed to determine induced tensile strain due to differential settlement of existing waste and the formation of a localized depression beneath the liner system. The tensile strain was first analyzed using the settlement results discussed above. The length of liner between two points after waste settlement were calculated and compared with that prior to settlement. Results showed that the proposed liner system will be mainly under "compression," i.e., liner length "shortened," and a very limited length of the upper portion will experience a maximum tensile strain of 2.4 percent.

A review of the literature and data on the allowable strain of liner components indicated that the GCL is expected to have the minimum allowable tensile strain of 10 percent. The allowable strains for the geonet, geotextile, and geomembrane are 5 percent, 5 percent, and 10 percent, respectively. Therefore, the calculated strain is less than the minimum allowable strain of the liner system components.

An evaluation of strain in the overliner system components due to localized depressions (subsidence) near the surface of the old waste was performed, and is included as Appendix III-3G-2. A parametric analysis, comparing the diameter of the subsidence area and depth at its center to the allowable strain of the overliner components, indicates that the ratio of depth to diameter is approximately 0.14 for 5 percent strain and 0.20 for 10 percent strain.

Depressions of this magnitude would only be expected if voids or highly compressible material are present immediately below the overliner. To reduce the potential for subsidence below the overliner system, the existing waste will be surcharged by placing at least 20 feet of soil for a minimum 3-month period. The surcharge will tend to collapse voids and compress the underlying material.

7.3 Stability Analysis

7.3.1 Stability of Protective Cover on the Overliner

An analysis of the stability of the overliner liner system was performed using an infinite slope analysis. Based on a review of the literature and unpublished data on similar materials under similar loading conditions, the critical interface shear strength within the liner system was estimated to be 24 degrees. The worst-case maximum length of slope is 1,000-ft long with a 3H:1V slope.

The analysis included as Appendix III-3G-5 indicates that provided the geocomposite drainage layer is adequate to convey drainage without building up pore water pressures in the geocomposite, the factor of safety is found to be at least 1.34, and likely to be higher due to the use of conservative parameters in the limit equilibrium analysis. This factor of safety is considered adequate and will increase as waste is placed within the cell.

Additional analyses (included in Appendix III-3G-4) were performed demonstrating that a standard double-sided geocomposite drainage layer will have a transmissivity adequate to convey water infiltrating through the protective cover over the maximum overliner slope length.

7.3.2 Overliner Leachate Collection System

The leachate collected from the overliner area will be diverted to Tract 5, Cell 1 through a leachate pipe installed during Cell 1 construction. The final overliner grade in the direction of leachate flow (post-settlement) is 8 percent at minimum. Such liner grades ensure positive leachate drainage.

The overliner leachate collection and removal system is designed to limit the maximum leachate depth over the liner to less than the thickness of the geocomposite drainage layer, which meets the 30-cm criterion stipulated in 30 TAC §330.331(a)(2). The leachate collection system was designed considering the leachate flow from the overliner area. The longest leachate flow path including the overliner area was used in the analysis and associated drainage capacity evaluations.

The required transmissivity of the geocomposite drainage layer on the overliner is 6.1 x 10^{-5} m^2/sec at a load equal to or greater than 10,000 psf and a minimum gradient of 33 percent. The calculation is included in Appendix III-3G-4.

7.3.3 Gas Collection and Control System Considerations

The construction of the overliner system will interfere with existing gas wells EW-13R, EW-12R, EW-46R, EW-11R, EW-10R, EW-9R, EW-8R, and EW-61. Prior to constructing the overliner system, these existing wells will be abandoned. The wells will be cut and capped below the ground surface and any laterals to these wells will be cut and capped to remove the wells from the vacuum system. A discussion of the procedure and a typical abandonment detail is included in Part III, Attachment 6. These wells will either be replaced with new wells or horizontals will be constructed underneath the overliner area prior to construction, as necessary to maintain gas control in this area.

8.0 MANAGEMENT OF GAS CONDENSATE AND MONITORING WELL WATER

8.1 Gas Collection System Condensate

Condensate from the gas collection system will be collected in collection sumps located on the landfill footprint as part of the Gas Collection and Control System (GCCS). In addition, on-site leachate storage systems may be used and combined with leachate for on-site management, recirculation, or off-site disposal. Gas condensate may be recirculated into Subtitle D-lined areas in accordance with Section 6.2.1.8 of this plan and/or disposed of using procedures similar to those discussed in Sections 6.2.1.5 through 6.2.1.7.

8.2 Monitoring Well Water

Water collected in the process of sampling the groundwater monitor wells will be handled in accordance with the Groundwater Sampling and Analysis Plan in Appendix III-5B.