PART III, ATTACHMENT 3
WASTE MANAGEMENT UNIT DESIGN

Hawthorn Park Recycling & Disposal Facility

Houston, Harris County, Texas

TCEQ Permit MSW-2185A

Owner/Site Operator/Permittee:

WASTE MANAGEMENT

USA Waste of Texas Landfills, Inc.
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STATE OF TEXAS
2/12/21
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INTENDED FOR PERMITTING PURPOSES ONLY

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1.0 INTRODUCTION

The Hawthorn Park Recycling and Disposal Facility (referred to hereinafter as "Hawthorn Park RDF," or "facility," "landfill," or "site") is an existing 171.6-acre Type IV municipal solid waste (MSW) facility owned and operated by USA Waste of Texas Landfills, Inc. (USA Waste) under TCEQ Permit No. MSW-2185. The facility is located at 10550 Tanner Road, approximately 500 feet east of Beltway 8 (Sam Houston Parkway), north of Tanner Road in Houston, Harris County, Texas.

By way of this Permit Amendment Application (PAA), USA Waste proposes to add approximately 38.6 acres to the permitted area of the facility, for a total permitted area of 210.2 acres under this permit amendment application MSW-2185A.

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, 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.

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 inert aggregate materials that can accommodate the traffic and weather conditions at the site will be provided from the facility to Tanner Road, the public road used to access the facility, and within the facility to the unloading area(s) designated for wet-weather operation. Truck traffic leaving the site will enter and exit via a paved access road, which will help clean off excess mud before reaching the public roadway.

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 daily. Mud will be removed from on-site roads daily 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 the adjacent detention pond, and/or outside sources.

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

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

The 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.

Initial waste placement under the proposed expansion (Permit No. MSW-2185A) will commence above-grade in the eastern portion of the facility over the currently permitted East Block waste unit. Subsequent waste placement will be generally eastward, with a combination of landfilling above existing filled areas and the development of new landfill cells that are part of the expansion area. The grades for these new undeveloped landfill cells are shown on Figure III-3-1. Details of the proposed sequence of landfilling are shown on Part II, Figures II-7.1 through II-7.5.

In accordance with 30 TAC §330.63(d)(4)(F), construction and design details of the compacted perimeter berms that are proposed in conjunction with aboveground (aerial-fill) waste disposal areas are shown on the fill cross-sections on Figures III-3-3.2 through III-3-3.4 and on the liner details on Figure III-3-5.

The final expanded facility will consist of a single 210.2-acre waste management unit, filled to the final design grades shown on Figure III-3-2. The landfill will be closed according to the Closure Plan in Part III, Attachment 7.

The operational fill sequence drawings in Part II, Figures II-7.1 through II-7.5, represent the proposed interim phased development of the facility. These figures identify the proposed 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 as outlined in these figures; 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, side slopes 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 227 feet above mean sea level (ft-msl) and the maximum elevation of waste is 225 ft-msl. The elevation of the deepest excavation is 40 ft-msl, and it occurs in the existing permitted area (West Block) where was has previously been placed.

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 Hawthorn Park RDF is approximately 3,475,000 cubic yards of airspace (solid waste, plus required daily and intermediate cover). This PAA proposes revisions to the facility design, resulting in a total permitted capacity of approximately 19,510,000 cubic yards of airspace available for disposal operations.

The total remaining airspace at the facility is approximately 71,724 cubic yards. The proposed expansion will increase the remaining airspace by approximately 16,034,766 cubic yards.

If this PAA is approved, USA Waste anticipates that, in 2022, the landfill will receive approximately 150,000 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. USA Waste anticipates that the facility will have a site life of approximately 46.3 years, at which time the rate of waste disposal will reach approximately 340,000 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 Hawthorn Park RDF. The geotechnical characteristics of the site are summarized herein and are based on recent geotechnical investigations performed by Biggs & Mathews Environmental, Inc. (BME) and on information from previous geotechnical investigations of the site.

Engineering analyses performed include the settlement analysis and stability analyses of excavated slopes, interior waste slopes, and final-filled configuration. These calculations, along with the geotechnical properties of the subsurface described in Section 4.4 of this report, demonstrate that the soils at the site 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 Hawthorn Park RDF has been characterized by previous investigations using a total of approximately 180 soil borings, monitoring wells, and piezometers. Eleven (11) additional borings (all greater than 30 feet below the elevation of deepest excavation (EDE) of 40 feet) were installed in 2019 to comply with 30 TAC §330.63(e)(4)(B) and gather geotechnical data within the future expansion area.

The previous site investigations are discussed in Section 4.2 of the Geology Report in Part III, Attachment 4, and summarized below:

- 1977 – Four soil borings drilled by Southwestern Laboratories for subsurface investigation purposes. Borings generally advanced to a depth of approximately 60 feet and covered the area to the southwest of the Center Block (Permit No. MSW-1135).
- 1981/1982 – Twenty-one soil borings drilled by McBride-Ratcliff and Associates (MRA) were installed to investigate the soil properties and determine the bottom liner thickness and sidewall composition within the southern portion of Sector 1 located in the East Block.
- 1982 – Twelve soil borings drilled by Aviles Engineering. Borings generally covered the entire West Block.
- 1983 – Two soil borings drilled by MRA. Borings installed along the eastern boundary of the West Block.
- 1983/1984 – Fifteen soil borings drilled by Western Contractor Services, Inc. (WCS). Borings generally covered the entire West Block.
- 1985/1986 – Eighteen soil borings drilled by MRA. Borings generally covered the central and northern portion of Center Block. Borings were not installed in the southern portion of Center Block.
- 1987 – Nine additional soil borings drilled by MRA. Four borings covered the southern portion of Center Block, and five borings covered the East Block.
- 1992 – Thirty-two soil borings drilled by MRA. Borings generally covered the entire Center Block and East Block and the detention area.

- 2019 – Eleven soil borings drilled by BME. Four borings were installed within and along the current permit boundary and seven borings were installed within the expansion areas.

4.2 Geotechnical Summary

The site stratigraphy has been divided into four distinct layers, Layers I through IV, ranging from clays and sandy clays to sand.

4.2.1 Laboratory Tests

Laboratory testing was performed on selected samples in accordance with commonly accepted methods and practices. For the most recent investigation, water content determination was performed in accordance with ASTM D2216; Atterberg limits were performed in accordance with ASTM D4318; grain size analyses were performed in accordance with ASTM D422 and D1140; and the permeability along both the vertical and horizontal axes on undisturbed samples was determined in accordance with ASTM D5084 Method F (constant volume – falling head), using tap water as the permeant (one permeability orientation per sample chosen for testing). Shear strength testing consisted of consolidated-undrained (CU) triaxial compression tests in accordance with ASTM D4767. Consolidation testing was performed in accordance with ASTM D2435.

During the various investigations, samples were collected, including Shelby tube, split-spoon, and core samples. A summary of the soil samples from the 2019 investigation and their corresponding tests is given in Table III-3-1 – Summary of Soil Laboratory Testing.

Laboratory test results from the BME (2019) investigation are included in Appendix III-3E.

Table III-3-1 – Summary of Soil Laboratory Testing – 2019 Investigation

Boring/Pit Number	Sample Depth (ft)		Water Content	Atterberg Limits	- #200 Sieve (Passing)	Consolidated Undrained Triaxial Compression (CU)	Consolidation (ILC)	Permeability
		Test Method	D2216	D4318	D1140	D4767	D2435	D5084
		Layer						
BME-2 / U7 Horizontal	12-14	I	x					x
BME-3/U12	22-24	I	x	x	x		x	
BME-5 / U4 Horizontal	6-8	I	x					x
BME-6 / U7 Horizontal	12-14	I	x					x
BME-6 / U13	24-27	I	x	x	x		x	
BME-6 / U32	60-64	III	x	x	x		x	
BME-7 / U7	12-14	I	x	x	x	x		
BME-7/U13	24-26	I	x					x
BME-8 / U6	10-12	I	x					x
BME-9 / U5	8-10	I	x					x
BME-9/U19	36-38	III	x					x
BME-10 / U13	24-26	I	x					x
BME-11 / U5 Horizontal	8-10	I	x					x
BME-11 / U4	6-8	I	x	x	x	x		

4.2.2 Site Stratigraphy

The site stratigraphy has been illustrated through a series of four cross-sections, as shown in the Part III, Attachment 4, Appendix III-4C, Figures ATTIII-4C-2 through ATTIII-4C-5. These cross-sections utilize previous borings at the site in conjunction with new borings installed in 2019 by BME. The results of the subsurface investigation and previous studies show that the site is underlain by four distinct strata, namely (in order from ground surface down):

- Layer I: Clays and sandy clays with minor amounts of sand and silt
- Layer II: Channel fill deposits consisting predominantly of fine sands with gradations to silty fine sands and silts and is considered the uppermost groundwater-bearing unit of the site
- Layer III: Primarily low permeability clay with sand and silt forming internal transmissive zones
- Layer IV: Fine sand with some interbedded clays

4.2.3 Soil Properties

The properties of the predominant strata at the site are summarized in the subchapters below.

4.2.3.1 Layer I

Layer I consists predominantly of clays and sandy clays with minor amount of sand and silt. Layer I is found from ground surface to approximately 22 to 31 feet below ground surface (ft bgs). Due to this layer's high clay content it serves as an effective confining bed to the underlying transmissive unit. Table III-3-2 below summarizes the properties of Layer I determined from laboratory testing.

Table III-3-2 – Properties of Layer I

	Minimum Value	Maximum Value	Average	Number of Tests	Test Method
Water Content (%)	8	31	18.2	149	ASTM D2216
Liquid Limit	27	81	44.4	152	ASTM D4318
Plastic Limit	11	30	18	152	ASTM D4318
Plasticity Index	7	53	26.4	152	ASTM D4318
Liquidity Index	0	2.1	0.3	152	ASTM D4318
Consolidation	See results in Appendix III-3E				
Vertical Permeability (cm/s)	1.3E-08	1.3E-08	1.3E-08	1	ASTM D5084
Horizontal Permeability (cm/s)	3.2E-08	5.5E-08	4.4E-08	2	ASTM D5084
Permeability (cm/s) (Undefined Orientation)	1.1E-08	7.0E-08	3.2E-08	3	Varies

4.2.3.2 Layer II

Layer II is composed of channel fill deposits consisting dominantly of fine sands, with gradations to silty fine sands and silts. Layer II is generally found 20 ft bgs and has an average thickness of 30 ft. Table III-3-3 below summarizes the properties of Layer II determined from laboratory testing.

Table III-3-3: Properties of Layer II

	Minimum Value	Maximum Value	Average	Number of Tests	Test Method
Water Content (%)	11	25	19.7	24	ASTM D2216
Liquid Limit	19	67	29.2	22	ASTM D4318
Plastic Limit	12	26	17.4	22	ASTM D4318
Plasticity Index	5	41	11.8	22	ASTM D4318
Liquidity Index	0	1.2	0.4	22	ASTM D4318
Vertical Permeability (cm/s)	6.0E-08	9.2E-07	4.9E-07	2	ASTM D5084
Horizontal Permeability (cm/s)	2.6E-08	5.1E-08	3.9E-08	2	ASTM D5084
Permeability (cm/s) (Undefined Orientation)	6.0E-08	7.8E-08	6.9E-08	2	Varies

4.2.3.3 Layer III

Layer III is found approximately 59 to 100 ft bgs and is correlatable across the site. Layer III consists primarily of clays, sandy clays, and silty clays. The layer is primarily a zone of low permeability; however, due to presence of some sand and silt, internal transmissive zones are present. Table III-3-4 below summarizes the results of properties from the tested samples in this stratum.

Table III-3-4: Properties of Layer III

	Minimum Value	Maximum Value	Average	Number of Tests	Test Method
Water Content (%)	13.3	42	22.9	127	ASTM D2216
Liquid Limit	25	86	48.6	129	ASTM D4318
Plastic Limit	13	29	20.5	129	ASTM D4318
Plasticity Index	7	59	28.1	129	ASTM D4318
Liquidity Index	0	0.6	0.1	128	ASTM D4318
Consolidation Test	See results in Appendix III-3E				
Vertical Permeability (cm/s)	8.5E-09	8.2E-08	4.5E-08	2	ASTM D5084
Horizontal Permeability (cm/s)	7.9E-09	3.4E-08	2.3E-08	3	ASTM D5084
Permeability (cm/s) (Undefined Orientation)	7.9E-09	9.1E-08	4.5E-08	5	Varies

4.2.3.4 Layer IV

Layer IV consists of fine sands and varying amounts of silt. Layer IV is found at depths greater than 100 ft bgs and has an approximate average thickness of 25 ft.

4.3 Unit Engineering Analysis

Analyses were performed to assess the performance of the landfill with respect to settlement and slope stability, and to provide for temporary dewatering during construction. Each of these analyses is described in detail in the following sections.

4.3.1 Settlement Analysis

Maximum facility floor settlement will occur where the compressible materials beneath the liner are thickest and the change in overburden stress is the greatest. The location that provides the thickest section of compressible material (Layer I and III clays) and the greatest change in overburden stresses occurs at the approximate location of Cross-Section A-A' on Figure II-3-4, located in-between the West Block and Center Block area on a north-south direction.

The settlement analysis indicates that the minimum settlement at the center of waste and toe of slope is approximately 1.95 feet and 0.53feet, respectively, which results in a maximum differential settlement of 1.42 feet and a negligible strain in the compacted clay liner (0.0003%). Because the predicted strain of the clay liner is significantly less than the allowable strain of 0.1%, the differential settlement will not be detrimental to the clay liner.

The results of the settlement analysis are presented in Appendix III-3B.

4.3.2 Stability Analysis

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. Based on the U.S. Corps of Engineers' "Design and Construction of Levees" manual (EM 1110-2-1913), the recommended factors of safety are 1.3 for short-term and 1.5 for long-term conditions. Short-term conditions include excavated slopes and interior waste slopes. The final-filled configuration represents long-term conditions.

Slope stability analyses were performed using limit equilibrium methods to assess the stability of the proposed landfill. In particular, the stability of the proposed excavated landfill side slopes, interior waste slopes, and the final-filled waste configuration of the landfill were evaluated.

In general, the analyses consist of the following:

- Characterization of critical cross-section (e.g., the geometry, geology, and groundwater conditions);
- Selection of appropriate strength parameters; and
- 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 of excavated slopes was performed to evaluate potential failure surfaces for excavation of waste management units. The critical section for excavation, Section B-B' on Figure III-3-4, which occurs in the future expansion area located in-between the Center Block and East Block, is a 3H:1V slope from an approximate elevation of 108 ft-msl to the toe at design elevation of 76 ft-msl.

Subsurface stratigraphy previously developed by BME were used to determine the soil strata in the future expansion area. The soil strength parameters were obtained from boring log information and from laboratory soil testing data. A groundwater level of approximately 96 ft-msl was assumed based on the subsurface stratigraphy data along the analyzed cross-section. Within the excavation, the groundwater level is conservatively assumed to correspond to the excavation grade.

The stability analysis of excavated slopes was performed using SLIDE2, a 2-dimensional limit-equilibrium analysis software, developed by Rocscience. Circular failure surfaces were evaluated. The minimum factor of safety for the cross-section and failure mode was calculated using the General Limit Equilibrium / Morgenstern-Price and Spencer Methods.

The results from the methods providing the least factor of safety is presented in Appendix III-3C. The minimum factors of safety are 1.42 for drained and 1.31 for undrained conditions. These values exceed the minimum allowable factor of safety for excavated slopes of 1.3, which indicates that the excavation slopes will be stable under the analyzed design configuration.

4.3.2.2 Stability of the Interior Waste Slopes

A stability analysis of interior waste slopes was performed to determine the factor of safety against failing through or sliding along the proposed liner system. The most critical interior waste slope cross-section is dependent on operations procedures. For this analysis, it was conservatively assumed that a 3H:1V temporary waste slope will exist from near the maximum height of the landfill to a point on the floor in the northern portion of the expansion area between the West and Center Blocks. This cross-section is shown on Figure III-3-4 as Section C-C'.

The stability analysis of interior waste slopes was performed using SLIDE2, a 2-dimensional limit-equilibrium analysis software, developed by Rocscience. Circular and block failure surfaces were evaluated. The minimum factor of safety for the cross-section and failure mode was calculated using the General Limit Equilibrium / Morgenstern-Price and Spencer Methods.

The results from the methods providing the least factor of safety is presented in Appendix III-3C. The minimum factor of safety for circular and block failures modes are 2.24 and 1.94, respectively. These values exceed the minimum allowable factor of safety for interior waste slope of 1.3, which indicates that the interior waste slopes will be stable under the analyzed design configuration.

4.3.2.3 Stability of Final Filled Configuration

A stability analysis of the final-filled configuration was performed to determine the factor of safety against sliding and the global stability for the final-filled configuration. The most critical final-filled configuration cross-section occurs where waste slope length is greatest and where the perimeter berm side slope is shortest. Based on a review of the design grades, the expansion area between the West and Center Blocks was determined to contain the critical section. This cross-section is shown on Figure III-3-4 as Section D-D'.

The stability analysis of the final-filled configuration was performed using SLIDE2, a 2-dimensional limit-equilibrium analysis software, developed by Rocscience. Circular and block failure surfaces were evaluated. The minimum factor of safety for the cross-section and failure was calculated using the General Limit Equilibrium / Morgenstern-Price and Spence Methods.

The results from the methods providing the least factor of safety are presented in Appendix III-3C, Figures III-3C3-1 and III-3C3-2. The minimum factors of safety are 2.75 and 2.45 for circular and block failure modes, respectively. These values exceed the minimum allowable factor of safety for final-filled waste slope of 1.5, which indicates that the final-filled configuration will be stable under the analyzed design configuration.

4.3.3 Temporary Dewatering

To facilitate construction of the below-grade disposal cells in the expansion area, depressurization wells will be used to lower groundwater potentiometric levels in Layer II. Calculations to support the design of these depressurization wells are included in Appendix III-3D-2.

5.0 LINER DESIGN

The Hawthorn Park RDF will implement a liner design meeting the requirements of §330.331(d)(2) for future cell development in the proposed expansion area. The following discusses this design as well as the Liner Quality Control Plan (LQCP), which specifies construction methods for the soil liner system.

5.1 Disposal Cell Liner System Design

The liner system configuration for the proposed cells is comprised as follows (from top of liner system to bottom):

- 1-foot thick protective cover
- 3-foot thick re-compacted clay liner

Liner details are included in Figure III-3-5. Cross-sections through the expansion areas are shown in Figures III-3.1 through III-3.4.

The proposed future landfill excavation generally extends below the seasonal-high groundwater table. Existing and proposed depressurization wells will be utilized as a short-term depressurization and dewatering system within and along the future excavation areas where the potentiometric surface is above the design-excavation configuration. The short-term depressurization and dewatering system will pump excess groundwater out of Layers I and II at a calculated flowrate that will provide a minimum factor of safety of 1.2. The detailed calculations are included as Appendix III-3D-2.

Long-term excavation stability will be achieved by the weight (ballast) of the soil liner, protective cover, and waste above the liner. Example calculations are included as Appendix III-3D-3.

5.2 Liner Quality Control Plan

The 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-3D. 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) will be submitted to TCEQ for review and approval in accordance with 30 TAC §330.341.