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9 May 2014

Mr. David Dippel, P.G.
MC-124
Texas Commission on Environmental Quality
MSW Permits Section, Waste Permits Division
P.O. Box 13087
Austin, Texas 78711-3087

Subject: Response to Second Notice of Deficiency (NOD)
Fairbanks Landfill – Harris County
Municipal Solid Waste (MSW) – Permit Application No. 1565B
Major Permit Amendment Application – Notice of Deficiency (NOD)
Tracking Nos. 17465613 & 17978769; CN602560930/RN100218544

Dear Mr. Dippel:

On behalf of USA Waste of Texas Landfills, Inc., Geosyntec Consultants (Geosyntec) has prepared this letter in response to the notice of deficiency (NOD) comments on the above-referenced permit amendment request transmitted in a 3 April 2014 letter from the Texas Commission on Environmental Quality (TCEQ) to USA Waste of Texas Landfills, Inc.

Pages 1 and 9 of the Part I Form, which includes the applicant's certification statement for this submittal, are provided at the end of this letter.

RESPONSE TO COMMENTS

TCEQ's comments are presented below in italicized type, with responses immediately following the comments in regular type. Additionally, the resulting replacement pages to the permit amendment application are enclosed with this letter to replace the previously submitted versions of the applicable pages. These revisions have an updated date reflecting the revision. A working copy is also attached to this submittal that uses an underline/strikethrough format, in order to mark the revised text, to highlight the revision and facilitate TCEQ's review.

Attachment A: Technical and Rule Compliance Comments

Appendix IIA–Maps and Drawings

Comment A1: Regarding the response to Comment A19 - Section 2 was revised to include brief discussions for C1, C2, and C3. Please clarify whether there are any other culverts existing or planned for the site (it was noticed that a drainage feature "SW-Culv" is listed in Table 2B-1-7); if yes, please revise Section 2 and other relevant portions of the application as necessary.

Response to Comment A1: It is believed that the above comment pertains to Section 3, not Section 2 (more specifically, Section 3.2). To clarify, the only existing culvert is the "outfall pipe" of the existing storm water pond, and the only proposed culverts are the three (3) culverts mentioned in Section 3.2. Section 3.2 of the Drainage Report (Part III, Attachment 2) has been revised for clarity, to note that the feature called "SW-Culv" in the HEC-HMS Model (Attachment 2B) refers to Culvert C3 (i.e., the proposed outlet pipe of the South Pond).

Comment A2: Regarding the response to Comment A20 – Please confirm that if the landfill is closed at any point prior to the scheduled closure, all scheduled surface drainage features (except for the ones on the landfill slopes: top and sides) will have been constructed in accordance with the design/installation schedule, and there will be no need to revise/modify the drainage systems; otherwise please revise the Closure Cost Estimates to properly account for the additional costs related to the surface drainage systems (please include brief discussions of the related activities).

Response to Comment A2: As requested, we have considered the above comment, and have confirmed that at any given point in time of landfill development, the scheduled perimeter surface drainage features will have already been constructed in accordance with the design/installation schedule discussed in Section 8 of the Drainage Report (Part III, Attachment 2). In particular, see the third paragraph on Page 2-24 of the Drainage Report. Accordingly, no further revisions have been made.

Attachment 2A–Surface Water Management System Drawings

Comment A3: Regarding the response to Comment A24 - The revised Drawing 2-7 does not appear to have the revisions stated in the response letter. Please revise the drawing as requested.

Response to Comment A3: Drawing 2-7 inadvertently omitted the previously requested structural fill material and construction requirements. Accordingly, Drawing 2-7 has been revised to include this information.

Attachment 2E–On-site Design – Culverts and Perimeter Drainage Channels

Comment A4: Regarding the response to Comments A33 - Please revise Drawing 2-5 in Attachment 2A and other relevant drawings, if necessary, to reflect the revisions made to Table 2E-7.

Response to Comment A4: Table 2E-7 indicates that the inlet to Culvert C1 has a peak headwater elevation of 110.7 ft, MSL during the 25-year, 24-hour storm event. A review of Drawing 2-5 revealed that the water surface is correctly positioned at that elevation at the inlet of Culvert C1. Therefore, no changes are necessary; the information on Drawing 2-5 matches that on Table 2E-7.

Attachment 3A–Waste Management Unit Design

Comment A5: *Regarding the response to Comments A46 - Please revise Table I-1 of Part I and Table II-1 of Part II and, if applicable, the text portions in Part I and Part II to specify that the maximum final cover elevation will be 250.5 feet above mean sea level (a note may be added to explain the final cover design conditions that will result in the final cover elevation of 250.00 feet above mean sea level).*

Response to Comment A5: Part I Report Table I-1 and Part II Report Table II-1 have been revised as requested. Additionally, Part III Report Table III-1 and the Part III Report narrative have also been revised to be consistent. In these instances, it was decided not to add the optional note discussed in the above comment since these portions of the permit application narrative reports are general in nature. Instead, notes of explanation regarding the final cover design conditions and resulting maximum elevation are provided on the relevant drawings.

Comment A6: *Regarding the response to Comments A47 – A detail in Drawing 3-12 refers to Note 3, but the list of notes does not include Note 3. Please revise Drawing 3-12 to include Note 3; please also ensure that Note 3 contains or refers to the information mentioned in the NOD response.*

Response to Comment A6: Drawing 3-12 inadvertently omitted Note 3, which was intended to provide a cross-reference to the perimeter ditch designations and sizing design. Accordingly, Drawing 3-12 has been revised by adding Note 3 with the requested information.

Attachment 3D.4.2 – Ballast Uplift Calculations

Comment A7: *Regarding the response to Comments A61 – Please provide copies of published literature that delineate the principle and the application of the second term in the calculation equation for R_N.*

Response to Comment A7: For reference, the second term in the calculation of the resisting force refers to the uplift resistance provided due to the lateral earth pressure of the clay liner. It is also noted that the magnitude of this resisting force is very small and has a very minor contribution towards the overall uplift resistance (i.e., negligible effect on the calculation results). However, the inclusion of this component of the uplift resistance is consistent with geotechnical engineering principles, and as requested, Attachment 3D.4.2 has been revised to include copies of relevant pages of a widely used geotechnical engineering textbook that defines and explains this term.

Attachment 5–Groundwater Monitoring Plan

The following comment is provided by Mr. David Dippel, P.G.

Comment A8: *Regarding the response to Comments A66 – Please revise the permit application to reflect the applicants response in the first NOD letter. Also, please provide a reference* *to design and construction specifications for the placement of fill material in areas adjacent to landfill cell development where Layer II Sand had been excavated.*

Response to Comment A8: [Note that this response was prepared by the geologist-of-record, Mr. Michael Snyder, P.G., of the firm Biggs & Mathews.] Appropriate language related to the identification of the uppermost aquifer that was previously used in the response letter has been added to Section 6.3 of Attachment 4 and to Section 1.1 of Attachment 5.

In addition, a reference to the constructed fill details has been added to the sections identified above. The specifications for compacted fill are provided in Section 5 of the Liner Quality Control Plan (Part III, Attachment 3C). There are also some details and cross sections that identify where fill will be placed. The compacted fill areas are shown on the drawings in Attachment 3A (Drawings 3-6 through 3-11).

Attachment 7–Closure Plan

Comment A9: Regarding the response to Comments A70 - The revised Section 3.2.1 does not appear to include the revisions described in the first NOD response letter. Please revise the application to address Comment A70.

Response to Comment A9: Section 3.2.1 of the Closure Plan (Part III, Attachment 7) inadvertently omitted the requested information on the specified maximum hydraulic conductivity, and has been revised as requested to include this information.

Attachment 7B–Final Cover Quality Control Plan

Comment A10: Regarding the response to Comments A81 - Please revise Table 7B-3 to specify a testing frequency of 1 per 3 acres per lift. Please revise the application as appropriate.

Response to Comment A10: Table 7B-3 has been revised as requested. The notes to Table 7B-3 have also been revised to be consistent, for additional clarity, and to remove several minor reduncancies.

Attachment 9–Cost Estimates for Closure and Post-closure Care

Comment A11: Regarding the response to Comments A84 - Please refer to comment on NOD response for A2 in this Attachment, and revise Attachment 9 as necessary.

Response to Comment A11: Please see our response to Comment A2 in this letter. As discussed, no revisions to Attachment 9 (the Closure Cost Estimate) are necessary.

Attachment B: Additional Comments

Attachment 2A–Surface Water Management System Drawings

Comment B1: Regarding the response to Comments B8 - Please revise the application at pertinent locations to include provisions to mitigate the areas where the noted discrepancies occur, to ensure that the channel bottom grade is consistent with the design (it was noted that the grade lines showed the actual elevations of the proposed and existing channel bottom). Please revise the application as necessary.

Response to Comment B1: Note 3 has been added to Drawing 2-5 to address this comment. The added note indicates that areas of the perimeter channel with discrepancies will be mitigated at the time of channel construction/upgrade so that the channel grade is made consistent with the design, and that discrepancies are also identified and mitigated during ongoing inspections/maintenance.

Attachment 2E–On-site Design – Culverts and Perimeter Drainage Channels

Comment B2: Regarding the response to Comments B24 - Please specify the units on the horizontal axis for the hydrographs that are included in the application following Tables 2B-1-7 and 2B-1-8 (please explain how the horizontal axis correlates to the times in the tables). It was also noticed that Table 2B-1-7 lists peak times for the drainage features; please explain the significant disparity between the peak times for the drainage areas/the perimeter channels and the peak times for Pond NE (the time of peak Pond-NE is about 11 hours 38 minutes behind the peak times for the drainage areas and perimeter channels that discharge into the ponds). Please revise the application as necessary.

Response to Comment B2: The horizontal axis for the hydrographs (produced by the HEC-HMS model output) is in the units of days. Although the HEC-HMS output format does not provide axis labels, Geosyntec has manually-added labels to the horizontal axis of the hydrographs as requested. Tables 2B-1-7 and 2B-1-8 are also output produced by HEC-HMS and express the time of peak as a hypothetical date and time of day (DD-MM-YYYY, HR:MIN). The "start of run" on the top of the tables indicates the date and time when the hypothetical storm event begins, and the "end of run" at the top of the tables indicates the date and time of the end of the modeling simulation. Converting units, the time to peak is the same (i.e., the tables match the HEC-HMS output). For example, Table 2B-1-7 indicates a time of peak of "Pond_S" (i.e., the South Pond) of 01Jan2012, 12:50. This matches the time of peak on the hydrograph on Figure 2B-1-6 (i.e., just over half way through Day 1 of the modeling simulation).

With respect to the disparity in peak times: From Table 2B-1-7, the time to peak from "Pond_NE" (i.e., the Northeast Pond) is at 23:41 of Day 1, and the time to peak for "Pond_S" (the South Pond) is at 12:50 of Day 1. This is roughly a 12 hour delay as noted in the above comment. There are a couple reasons for this. The first is that the South Pond receives more flow volume, which along with the stage-storage relationship of this pond, causes the water surface elevation to rise more quickly. This causes the South

Pond to have a higher peak elevation than the Northeast Pond. When modeled in HEC-HMS, this means that the Northeast Pond will not begin to discharge until the elevation in the South Pond becomes smaller. This can be seen in Figures 2B-1-5 and 2B-1-6. The yellow dashed line is water surface elevation, and it can be seen that the Northeast Pond increases to roughly elevation 102.15 ft, MSL just before midnight of Day 1 before it begins to decrease. For the South Pond, the water surface elevation rises to over elevation 105.2 ft, MSL at an earlier time, but then decreases to 102.15 ft, MSL just before midnight on Day 1, at which time this allows the Northeast Pond to discharge. This timing explains the resulting peak time in the Northeast Pond occurring approximately 12 hours after the peak time in the South Pond.

For the foregoing reasons, the information in the application has been checked and the HEC-HMS model simulations are confirmed to be behaving as expected, and accordingly no revisions have been made other than the aforementioned labeling of the horizontal axis of the hydrographs.

Attachment 3A–Waste Management Unit Design

Comment B3: Regarding the response to Comments B36 - Please revise at least one drawing to better show the existing and planned perimeter access roads (the legends provided in the current drawings, such as Drawing 3-2, do not allow easy/positive discernment of the perimeter access roads).

Response to Comment B3: Drawing 3-2 has been revised as requested, to shade the perimeter access road.

Attachment 3D.1– Geotechnical Report

Comment B4: Regarding the response to Comments B47 - Please revise Section 4.1 to define the terms of cover layers and operational covers as used in the first four bullets in Section 4.1.

Response to Comment B4: Section 4.1 of Attachment 3D.1 has been revised as requested.

Attachment 3D.2 – Slope Stability Analysis

Comment B5: Regarding the response to Comments B50 - Please revise the application at pertinent locations to specify that the soil strength properties used in the stability analysis will be verified during the liner and final cover construction processes; and, if the verifications reveal that the soils have different strength properties, the stability analysis will be re-evaluated to make sure that the results are still acceptable or new stability analysis will be performed using the new soil strength properties.

Response to Comment B5: Table 3C-2 of the Liner Quality Control Plan (LQCP) has been revised to address this comment, by including the requirement for pre-construction laboratory strength testing to verify that the recompacted clay liner material achieves the minimum required shear strength properties.

It is also noted that the slope stability analyses previously presented in Attachment 3D.2 made reasonable and conservative assumptions for the shear strength of the recompacted clay liner based on values reported in technical literature for compacted soil material with consistent properties to those specified for this facility. The slope stability results indicate that the liner or cover shear strengths are not drivers in the analysis (i.e., much lower strengths than assumed could be tolerated, and would still produce acceptable factors of safety against sliding). An additional slope stability analysis has been performed, and Attachment 3D.2 has been revised accordingly, to back-calculate the minimum recompacted clay liner shear strengths that would produce acceptable factors of safety. Because the back-calculated minimum necessary strength of the final cover is extremely low compared to the expected strength (required final cover strength is only 5% of the expected final cover strength), verification strength testing of the final cover is not warranted; and for this reason the Final Cover Quality Control Plan (FCQCP) has not been revised. The back-calculated minimum necessary strength of the liner is also quite low compared to the expected strength (about 40% of the expected value); however, verification testing is proposed as part of the pre-construction process to confirm that the minimum strength is attained, and the LQCP has been revised accordingly.

ADDITIONAL REQUESTED CHANGES

The following additional changes are requested:

- Part II, Appendix IID (Airports and Aviation Information). Additional information has been received from the Federal Aviation Administration (FAA). As a follow up to the First NOD Comment A6, a request for an obstruction evaluation was made to FAA. The FAA's response has now been received, and it is requested to add this information to the permit amendment application to the end of Appendix IID. The information documents the FAA's "Determinations of No Hazard" findings for the proposed expansion (i.e., the results of the obstruction evaluation).
- Part III, Attachment 6 (Landfill Gas Management Plan). It is proposed to make very minor revisions to Sections 4.2 and 5. The revision to Section 4.2 is to remove extraneous wording that could result in a potential inconsistency with the rest of the permit amendment application regarding the location where the gas monitoring records may be kept. The revision to Section 5 is to remove redundant notifications to TCEQ, and to be consistent with current practices of notifying the TCEQ Region.
- Part IV (Site Operating Plan). It is proposed to make very minor revisions to Sections 1.1 and 18.2. The revision to Section 1.1 is to remove extraneous wording that could result in a potential inconsistency with the rest of the permit amendment application regarding the location where the SOP may be kept. The revision to Section 18.2 is to make the statement regarding litter pickup consistent with the information in the cross-referenced Section 11.

PART I FORM AND CERTIFICATION STATEMENT

As mentioned, Pages 1 and 9 of the Part I Form are being submitted with this response. Page 9, the Signature Page, provides the certification statement signed by the applicant's responsible official.

CLOSURE

One original and three (3) copies of this submittal are being provided to the TCEQ MSW Permits Section in Austin. An electronic copy of this submittal has also been posted to the internet at the same URL as the initial posting of the application. Additionally, a copy of this submittal is being placed in the Fairbanks Branch Library for public viewing, to accompany the initial application already placed in that library. Geosyntec trusts that the above responses to TCEQ's comments provide the necessary information requested by TCEQ to complete their technical review of the permit amendment application. If you have any questions regarding the information presented in this letter, please do not hesitate to contact the undersigned by telephone at (512) 451-4003, or by E-mail at sgraves@geosyntec.com.

Sincerely,

Scott M. Graves, P.E.
Associate, Geosyntec Consultants, Inc.

Copy to: Mr. Charles Rivette, P.E., USA Waste of Texas Landfills, Inc.
Mr. Steve Jacobs, USA Waste of Texas Landfills, Inc.

REDLINE/STRIKETHOUGH PAGES

To facilitate TCEQ's review, the attached pages present a "redline/strikethough" version of the proposed text revisions to the permit amendment application. Note that due to re-pagination of the redline/strikethrough version, the page numbers may not match the final page numbers in the "clean" (replacement page) version.

For convenience, divider tabs are provided to indicate which portion of the application the revisions pertain. The designation "ST" on the tabs is an abbreviation for "strikethrough", and is intended to help identify the tabs that contain the redline/strikethrough versions, as opposed to the "clean" replacement pages provided subsequently with this response.

Prepared for:

USA Waste of Texas Landfills, Inc.

PERMIT AMENDMENT APPLICATION

PART I – SITE AND APPLICANT INFORMATION

SUPPLEMENTAL TECHNICAL REPORT

FAIRBANKS LANDFILL
MSW PERMIT NO. 1565B
HOUSTON, HARRIS COUNTY, TEXAS

Prepared by:

Geosyntec consultants

Texas Board of Professional Engineers Firm Registration No. F-1182

3600 Bee Caves Road, Suite 101

Austin, Texas 78746

(512) 451-4003

SEALED FOR THIS PART I SUPPLEMENTAL TECHNICAL REPORT, AND FOR PERMITTING PURPOSES ONLY.

WITHIN EACH APPENDIX, ITEMS THAT REQUIRE A SIGNATURE AND SEAL BY A LICENSED PROFESSIONAL (E.G., ENGINEER, SURVEYOR, OR GEOSCIENTIST) ARE SIGNED, SEALED, AND DATED, AS APPROPRIATE, BY THE RESPONSIBLE PROFESSIONAL.

Submitted August 2013

Revised March 2014

Revised May 2014

3.2.3 Site Layout and Proposed Changes

As mentioned, a Site Plan presenting the extent of the current facility and proposed expansion is presented in Appendix IA as Drawing IA-6. Inspection of Drawing IA-6 shows that the permit boundary and landfill footprint is proposed to increase towards the east and south. The northern and western limits of the landfill have been constructed, and no changes these existing waste limits are proposed. A minor reduction in the permit boundary is proposed on the west side of the site, to eliminate a small area where facility operations have not occurred and will not occur. No changes are proposed to the existing site entrance/exit location. Table I-1, presented below, summarizes the current permit conditions and the proposed changes.

TABLE I-1

SUMMARY OF CURRENT PERMIT AND PROPOSED EXPANSION - FAIRBANKS LANDFILL

Item	Units	Current Condition (Permit 1565A)	Increase due to Expansion	New Condition (Permit 1565B)
Permit Boundary Area	(acres)	118.1	70.9	188.95
Waste Disposal Footprint Area	(acres)	80.0	57.3	137.3
Buffer/Other Area	(acres)	38.1	13.6	51.7
Buffer/Other Area as a Percentage of Permit Boundary	(percent)	32.3%	19.1%	27.3%
Total Waste Disposal Capacity	(cubic yards)	8,326,000	17,886,000	26,212,000
Remaining Capacity as of 26 March 2012 Aerial Flyover	(cubic yards)	98,000	17,886,000	17,984,000
Projected Remaining Site Life	(years)	0.3	26.7	27.0
Maximum Elevation of Final Cover	(ft, msl)	154.0	96.5~~0~~	250.5~~0~~
Elevation of Deepest Excavation	(ft, msl)	51.0	No Change	51.0

Drawing IA-6 shows that for this proposed expansion, the two existing waste disposal units will be joined together to form one combined landfill footprint. The entire combined landfill footprint will have a contiguous, tied-in liner meeting the regulatory-prescribed design criteria for a Type IV landfill facility. Details of the liner system design are presented in Part III of the Permit Amendment Application.

Table I-1 indicates that of the 188.95-acre permit boundary, the waste footprint of the landfill will occupy approximately 137.3 acres, and the remaining area of about 52 acres will be used as buffers and other site features (e.g., perimeter access road, surface water ponds, main access road

Prepared for:

USA Waste of Texas Landfills, Inc.

PERMIT AMENDMENT APPLICATION

PART II – EXISTING CONDITIONS SUMMARY AND CHARACTER OF THE FACILITY AND SURROUNDING LAND

SUPPLEMENTAL TECHNICAL REPORT

FAIRBANKS LANDFILL
MSW PERMIT NO. 1565B
HOUSTON, HARRIS COUNTY, TEXAS

Prepared by:

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Submitted August 2013
Revised March 2014
Revised May 2014

will not occur. No changes are proposed to the existing site entrance/exit location. Table II-1, presented below, summarizes the current permit conditions and the proposed changes.

TABLE II-1

SUMMARY OF CURRENT PERMIT AND PROPOSED EXPANSION - FAIRBANKS LANDFILL

Item	Units	Current Condition (Permit 1565A)	Increase due to Expansion	New Condition (Permit 1565B)
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Projected Remaining Site Life	(years)	0.3	26.7	27.0
Maximum Elevation of Final Cover	(ft, msl)	154.0	96.5~~0~~	250.5~~0~~
Elevation of Deepest Excavation	(ft, msl)	51.0	No Change	51.0

As Drawing IIA-10 indicates, the two existing waste disposal units will be joined together to form one combined landfill footprint for this proposed expansion. The entire combined landfill footprint will have a contiguous, tied-in liner meeting the regulatory-prescribed design criteria for a Type IV landfill facility. Details of the liner system design are presented in Part III of the Permit Amendment Application.

Table II-1 indicates that of the 188.95-acre permit boundary, the waste footprint of the landfill will occupy approximately 137.3 acres, and the remaining area of about 52 acres will be used as buffers and other site features (e.g., perimeter access road, surface water ponds, main access road with scales and scale-house/office, etc.). The distance from the permit boundary to all solid waste unloading, storage, disposal, or processing operations will exceed a minimum buffer distance of 50 feet (see Drawing IIA-10).

As shown on Drawing IIA-10, the existing pipeline easement that crosses the site in a southwest-northeast orientation will be relocated to be adjacent to the southern and eastern permit boundaries, and the existing easement and associated pipelines will be abandoned. Easements and right-of-ways are discussed further in Section 14.1.1 of this report.

Prepared for:

USA Waste of Texas Landfills, Inc.

PERMIT AMENDMENT APPLICATION

PART III – SITE DEVELOPMENT PLAN

NARRATIVE REPORT

FAIRBANKS LANDFILL
MSW PERMIT NO. 1565B
HOUSTON, HARRIS COUNTY, TEXAS

Prepared by:

Geosyntec consultants

Texas Board of Professional Engineers Firm Registration No. F-1182
3600 Bee Caves Road, Suite 101
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(512) 451-4003

SEALED FOR THIS PART III NARRATIVE REPORT, AND FOR PERMITTING PURPOSES ONLY.

WITHIN EACH ATTACHMENT, ITEMS THAT REQUIRE A SIGNATURE AND SEAL BY A LICENSED PROFESSIONAL (E.G., ENGINEER, SURVEYOR, OR GEOSCIENTIST) ARE SIGNED, SEALED, AND DATED, AS APPROPRIATE, BY THE RESPONSIBLE PROFESSIONAL.

Submitted August 2013
Revised March 2014
Revised May 2014

TABLE III-1

SUMMARY OF CURRENT PERMIT AND PROPOSED EXPANSION - FAIRBANKS LANDFILL

Item	Units	Current Condition (Permit 1565A)	Increase due to Expansion	New Condition (Permit 1565B)
Permit Boundary Area	(acres)	118.1	70.9	188.95
Waste Disposal Footprint Area	(acres)	80.0	57.3	137.3
Buffer/Other Area	(acres)	38.1	13.6	51.7
Buffer/Other Area as a Percentage of Permit Boundary	(percent)	32.3%	19.1%	27.3%
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Maximum Elevation of Final Cover	(ft, msl)	154.0	96.5~~0~~	250.5~~0~~
Elevation of Deepest Excavation	(ft, msl)	51.0	No Change	51.0

As indicated on Attachment 3, Drawing 3-1, the two existing waste disposal units will be joined together to form one combined landfill footprint as part of the expansion. The entire combined landfill footprint will have a contiguous tied-in liner (see Attachment 3, Drawing 3-3) meeting the regulatory-prescribed design criteria for a Type IV landfill facility. Details of the liner system design are discussed subsequently in Section 4 of this report.

Table III-1 indicates that of the proposed 188.95-acre permit boundary, the waste footprint of the landfill will occupy approximately 137.3 acres, and the remaining area of about 52 acres will be used as buffers and other site features (e.g., perimeter access road, surface water ponds, main access road with scales and scale-house/office, etc.).

For Permit MSW-1565B, the filling pattern for waste disposal will start by continuing to fill the existing northern landfill area to higher elevations as the geometry allows for this expansion. Construction of new landfill sectors and subsequent waste filling in those sectors will then progress in the numerical sequence of sectors identified on Attachment 3, Drawing 3-1. More detailed phasing plans showing the excavation and filling sequences was previously presented in a series of drawings in Part II, Appendix IIA of this Permit Amendment Application.

As previously discussed in Part II of the Permit Amendment Application (Section 14.1.1 of the Part II narrative report), there is an existing pipeline easement that crosses the site in a sequence of excavation and filling at various points in time during upcoming landfill development.

The excavation side slopes will be configured at 3 horizontal:1 vertical (3H:1V) down to the cell floor, which is generally flat. The final aerial fill side slopes (i.e., above-grade final slopes) will be configured at 4H:1V slopes (i.e., a 25% grade) up to a landfill top deck area sloped upward at three (3) percent to a ridgeline, as shown on Drawing 3-3. The final cover system will be installed incrementally with the landfill development progression as fill areas reach their maximum final waste grade elevations.

4.5 Landfill Depth and Height Statistics

The elevation of deepest excavation is 51 feet above mean sea level (ft, MSL). The maximum elevation of waste is 248 ft, MSL. The maximum elevation of the final cover is 250.5 ft, MSL.

4.6 Estimated Rate of Solid Waste Deposition and Site Life

The landfill volume, estimated rate of solid waste deposition, and the resulting site life estimate is presented in Attachment 3B. For reference, a description of the waste characteristics, anticipated facility service area, and a five-year projection of the estimated maximum annual waste acceptance rate is presented in the "waste acceptance plan" in Part II of the Permit Amendment Application as required by 30 TAC §330.61(b).

4.7 Landfill Cross Sections

A series of landfill cross sections is provided in Attachment 3A (see Drawings 3-6 through 3-10). These cross sections have been selected to pass through key site features so as to accurately depict the existing and proposed depths of all fill areas within the site. The sections show the top of the perimeter berm; top of the proposed fill (top of the final cover); maximum elevation of proposed waste fill; top of the wastes; existing ground; bottom of the excavations; side slopes of trenches and fill areas; gas monitoring probes; groundwater monitoring wells, plus the initial and static levels of any water encountered. The cross-sections also show the logs of soil borings that pass near the profile. The 100-year flood elevation in Rolling Fork Creek is identified on the sections that pass through the west side of the site next to the creek.

4.8 Landfill Construction Design Details

Landfill construction design details are also presented in Attachment 3A (see Drawings 3-11 and 3-12), to accompany the previously mentioned cross section. The cross sections call-out the

ATTACHMENT 2

FACILITY SURFACE WATER DRAINAGE REPORT

Prepared for:
USA Waste of Texas Landfills, Inc.

PERMIT AMENDMENT APPLICATION

PART III – SITE DEVELOPMENT PLAN ATTACHMENT 2

FACILITY SURFACE WATER DRAINAGE REPORT

FAIRBANKS LANDFILL
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Submitted August 2013
Revised March 2014
Revised May 2014

-to the landfill slopes (i.e., down-slope) will collect the runoff from the top deck and sideslopes and convey this runoff to the landfill perimeter at the toe of the cover system sideslopes. These downchute channels will be lined with an articulated concrete block (ACB) material, or equal, to resist hydraulic forces from the water flowing in these channels.

Perimeter Channel. The western and northern sides of the landfill are existing, and include perimeter channels to convey runoff from drainage terraces and downchutes, and any contributing sheet flow, around the landfill and into surface water ponds. The proposed expansion will continue to route runoff from the western and northern sides of the landfill in this manner, using the same alignment and slopes as the existing perimeter channels. Due to the additional drainage areas contributing to these perimeter channels, they will need to convey larger peak flows than the existing perimeter channels and therefore in some cases will be widened to provide the additional capacity requirements. The perimeter drainage channels around the west and north sides of the site have a single high-point (see Drawing 2-4), approximately mid-way along the northern side of the site. One side of the channel high-point will convey flow eastward, into the Northeast Surface Water Pond. The other side of the channel high-point will convey flow westward and then southward around the landfill perimeter and into a culvert that flows into the South Surface Water Pond.

Culverts. There are three culverts proposed (see Drawing 2-1). Culvert C1 is a box culvert that will receive flow from the perimeter channel on the west side of the landfill, and will convey water into the South Pond. Culvert C2 is a pipe culvert located on the eastern portion of the site beneath the main facility access road that will hydraulically connect the South Pond and the Northeast Pond. Culvert C3 (labeled as "SW-Culv" in the HEC-HMS Model in Attachment 2B) is a pipe culvert that serves as the outfall discharge point from the South Pond into Rolling Fork Creek, on the southwestern side of the site.

Surface Water Ponds. Two surface water ponds are proposed (see Drawing 2-1): a Northeast Surface Water Pond; and a South Surface Water Pond. It is noted that the term "surface water pond" is used because the ponds are intended to provide a detention function (controlling the rate of surface water release from the site), as well as provide a sediment control/water quality function.

The two surface water ponds will be hydraulically connected by the aforementioned Culvert C1, a 24-inch corrugated metal pipe situated beneath the site access road.. As mentioned, the perimeter channel along the western and northern sides of the site will convey runoff into these ponds. Additionally, runoff collected by the drainage terraces and downchutes on the eastern and southern portions of the landfill will convey flow into these ponds. At the eastern end of the perimeter channel where it enters the Northeast Surface Water Pond, a grouted riprap apron will be used for erosion protection. At the southwestern end of the perimeter channel, a culvert (C1) will be used to connect the perimeter channel to the South Surface Water Pond (and will also have erosion protection). Where the downchutes flow directly into the ponds, the ACB-lined (or

ATTACHMENT 2A

SURFACE WATER MANAGEMENT SYSTEM DRAWINGS

LIST OF DRAWINGS		
Drawing No.	Title	Date
2-1	Facility Surface Water Management Plan	March 2014
2-2	Pre-Development Plan With Drainage Patterns	March 2014
2-3	Post-Development Plan With Drainage Patterns	August 2013
2-4	Perimeter Drainage Channel Plan With Stationing	August 2013
2-5	Perimeter Drainage Channel Profile	~~August~~ May 201~~3~~4
2-6	Surface Water Ponds – Plan View	March 2014
2-7	Surface Water Management System Details I	~~March~~ May 2014
2-8	Surface Water Management System Details II	August 2013
2-9	Surface Water Management System Details III	August 2013
2-10	Surface Water Management System Details IV	March 2014
2-11	Culvert Sections	March 2014
2-12	Rolling Fork Creek and Landfill Perimeter Berm Section	March 2014

ATTACHMENT 3A

LANDFILL DESIGN DRAWINGS

LIST OF DRAWINGS		
Drawing No.	Title	Drawing Date (latest revision)
3-1	Facility Layout Plan	August 2013
3-2	Overall Base Grading Plan	~~August~~ May 2014~~3~~
3-3	Overall Final Cover Grading Plan	March 2014
3-4	Landfill Entrance Plan	August 2013
3-5	Landfill Cross-Section Location Map	August 2013
3-6	Landfill Cross-Section A-A'	March 2014
3-7	Landfill Cross-Section B-B'	March 2014
3-8	Landfill Cross-Section C-C'	March 2014
3-9	Landfill Cross-Section D-D'	March 2014
3-10	Landfill Cross-Section E-E'	March 2014
3-11	General Landfill Construction Design Details I	March 2014
3-12	General Landfill Construction Design Details II	~~March~~ May 2014

Prepared for:
USA Waste of Texas Landfills, Inc.

PERMIT AMENDMENT APPLICATION

PART III – SITE DEVELOPMENT PLAN
SUB-ATTACHMENT 3C

LINER QUALITY CONTROL PLAN

FAIRBANKS LANDFILL
PERMIT NO. MSW-1565B
HOUSTON, HARRIS COUNTY, TEXAS

Prepared by:
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Submitted August 2013
Revised March 2014
Revised May 2014

TABLE 3C-1
MATERIAL SPECIFICATIONS FOR RECOMPACTED CLAY LINER

PROPERTY	QUALIFIER	UNITS	SPECIFIED VALUES	TEST METHOD[1]
Maximum Particle Size[2]	Maximum	Inch	1	ASTM D 422
Percent Passing #200 Sieve	Minimum	Percent	30	ASTM D 422
Liquid Limit	Minimum	Percent	30	ASTM D 4318
Plasticity Index	Minimum	Percent	15	ASTM D 4318
Hydraulic Conductivity	Maximum	cm/s	1×10^{-7}	ASTM D 5084[3]
Triaxial Compressive Strength (cohesion)	Minimum	psf	650	ASTM D 2850

Notes:

(1) CQA testing frequencies are provided in Tables 3C-2 and 3C-3 of this LQCP.
(2) Recompacted clay liner material must also not contain rocks or stones that total more than 10% by weight.
(3) Refer to Table 3C-2 for additional hydraulic conductivity testing requirements.

TABLE 3C-2
PRE-CONSTRUCTION TESTING REQUIREMENTS FOR RECOMPACTED CLAY LINER

TEST	METHOD	MINIMUM FREQUENCY OF TESTING[1]	PASSING CRITERIA
Particle Size (Sieve) Analysis	ASTM D 422	1 per source	See Table 3C-1
Atterberg Limits	ASTM D 4318	1 per source	See Table 3C-1
Natural (as-received) Moisture Content	ASTM D 2216	1 per source	None
Standard Compaction	ASTM D 698, if "light" weight compactor to be used[2]	1 per source (select either Standard or Modified Compaction Test based on weight of compactor to be used)	None
Modified Compaction	ASTM D 1557, if "heavy" weight compactor to be used[2]		None
Remolded Hydraulic Conductivity	ASTM D 5084[3]	1 per moisture/density relationship	≤ 1 x 10^{-7} cm/s
Remolded Triaxial Compression Strength (UU, single point)	ASTM D 2850	1 per source	≥ 650 psf

Notes:

(1) The testing frequency of one per source refers to a relatively consistent and distinguishable soil type at a borrow source location based on visual observations and field classification procedures. If the same borrow source is utilized for the soil supply of more than one liner area project, results from previous pre-construction tests may continue to be used.

(2) Compaction test method shall be selected to be representative of the type of compaction equipment planned for use by the Contractor. For reference, CAT 815 series compactors or equivalent are considered "light" weight equipment, representative of Standard Compaction Tests, and CAT 825 series compactors or equivalent are considered "heavy" weight equipment, representative of Modified Compaction Tests.

(3) Hydraulic conductivity testing shall be performed using tap water or a 0.05N solution of $CaSO_4$. Use effective stress of 20 psi. Distilled or deionized water shall not be used. The permeant should be deaired. All hydraulic conductivity test data shall be submitted with the SLER.

(4) Perform remolded hydraulic conductivity and triaxial compression tests as appropriate for the type of compaction equipment planned for use, on either: (i) a remolded sample that is compacted greater than or equal to 95% of the maximum dry density and at the optimum moisture content as determined from the Standard Proctor test; or (ii) a remolded sample that is compacted greater than or equal to 90% of the maximum dry density and at one percentage point dry of optimum as determined from the Modified Proctor test. Alternatively, a higher relative compaction or moisture content can be used in pre-construction testing; however, these higher values will then be the minimum required values for the recompacted clay liner.

(5) Additional hydraulic conductivity tests may be performed during the preconstruction testing program if authorized by the Owner, in order to develop a more detailed, alternative APZ that may broaden the range of allowable moisture-density target compaction criteria or define allowable conditions for use of soil blends. See Section 2.3.2.2 of this LQCP for a discussion of this approach.

Prepared for:

USA Waste of Texas Landfills, Inc.

PERMIT AMENDMENT APPLICATION
PART III – SITE DEVELOPMENT PLAN
ATTACHMENT 3D.1

GEOTECHNICAL REPORT

FAIRBANKS LANDFILL
MSW PERMIT NO. 1565B
HOUSTON, HARRIS COUNTY, TEXAS

Prepared by:

Geosyntec consultants

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Submitted August 2013

Revised March 2014

Revised May 2014

4. CONCLUSIONS

4.1 Overview of Geotechnical Findings

The findings of the 2012 geotechnical investigation of the landfill expansion area are generally consistent with previous investigations, resulting in a site-wide characterization of the geotechnical site characteristics. The resulting findings are summarized below.

- Stratum I soils (generally Clay) are suitable for use as low-permeability liner and final cover layers, vegetative layer, operational (i.e. weekly, intermediate) cover, and general (i.e., ~~structural~~compacted) fill. However, it is noted that Stratum I soils have been largely removed from the site.

- Fill Soils (generally Clay) encountered during the investigation are suitable for use as low-permeability liner and final cover layers, vegetative layer, operational (i.e. weekly, intermediate) cover, and general (i.e., ~~structural~~compacted) fill.

- Stratum II soils (Sand) are suitable for use as protective cover and operational (i.e. weekly, intermediate) cover. However, it is noted that Stratum II soils have been largely removed from the site through previous sand-pit operations.

- Stratum III soils (Clay) are suitable for use as low-permeability liner and final cover layers, operational (i.e. weekly, intermediate) cover, and general (i.e., ~~structural~~compacted) fill.

- Stratum IV soils (Sand) are interpreted to be below the elevation of deepest excavation (EDE) planned for the facility. Therefore, they are not expected to be encountered during landfill development. However, in terms of their geotechnical properties, they are suitable for use as protective cover and operational (i.e. weekly, intermediate) cover.

- Stratum V soils (Clay) are much deeper beneath the site, well below the EDE. Therefore, they are not expected to be encountered during landfill development. However, in terms of their geotechnical properties, they would be suitable for the same uses as Stratum III.

- With respect to the in-situ characteristics of the soils as they relate to constructability, permeability, slope stability, and settlement, all of the strata and soils encountered appear to provide suitable characteristics for adequate performance (as supported by the geotechnical design calculations presented elsewhere in Attachment 3D).

Additional discussion is presented below in the remainder of this report to further describe the rationale for the above findings.

ATTACHMENT 3D.2

SLOPE STABILITY ANALYSIS

SLIDE computer output and figures illustrating each of the shear surface scenarios are presented in Appendix 2.

6.5 Back-calculated Strengths for Compacted Clay Liner and Final Cover

It is noted that with respect to clay liner and cover strengths, the slope stability analyses previously discussed use assumed strength properties that are expected to be reasonable for the liner and final cover using conservatively selected strengths based on values reported in technical literature for the types of soil expected to be used. However, to provide a recommendation for a minimum strength, the minimum strength of the liner or final cover can be back-calculated until the desired target minimum calculated factor of safety is achieved.

These back-analyses were performed use the same cross sections as discussed previously in this report. The shear strength of the compacted clay liner and/or the compacted clay final cover was varied iteratively for each scenario until the lowest allowable strength is identified that produces a calculated factor of safety greater than or equal to the target minimum calculated factor of safety. Table 6 presents the results of these back-analyses.

Table 6
SUMMARY OF SLIDE RESULTS FOR BACK-CALCULATED LINER/COVER STRENGTHS

Back-Calculated Liner and Final Cover System Strength Properties Shear Surface Scenario	Back-Calculated Minimum Strength Required
Veneer Stability	
South slope. Veneer stability of liner system using undrained strengths. Undrained strength, Su, of liner was reduced until FS = 1.25.	174 psf
South slope. Veneer stability of final cover using drained strengths. Drained friction angle assumed to be 0 degrees. Drained cohesion of final cover reduced until FS = 1.5.	82 psf
East slope. Veneer stability of liner system using undrained strengths. Undrained strength, Su, of liner was reduced until FS = 1.25.	170 psf
East slope. Veneer stability of final cover using drained strengths. Drained friction angle assumed to be 0 degrees. Drained cohesion of final cover reduced until FS = 1.5.	82 psf
Interim Slope	
Block-type shear surface - Seated in the liner system and through Waste. Undrained strength, Su, of liner was reduced until FS = 1.25.	375 psf
South Slope	
Block-type shear surface – Seated in the liner system and through Waste. Drained friction angle assumed to be 0 degrees. Drained cohesion of liner reduced until FS = 1.5.	650 psf
East Slope	
Block-type shear surface – Seated in the liner system and through Waste. Drained friction angle assumed to be 0 degrees. Drained cohesion of liner reduced until FS = 1.5.	490 psf

Inspection of the above table reveals the minimum strength of the liner and/or cover system needed to attain adequate calculated factors of safety for the various scenarios analyzed. The above table further reveals that the highest required shear strength is for a long-term scenario of sliding through the liner system, which requires a cohesion of 650 psf.

7 SUMMARY AND CONCLUSIONS

Based on the analyses presented herein, the following conclusions are drawn.

- Critical cross sections were selected for analysis, and various sliding modes were considered.
- Soil and waste properties were selected based on conservative interpretations of site specific lab results or correlations from published technical literature.

A1.1 Estimating the Drained Friction Angle of Stratum II

Stratum II is a sand layer. A drained friction angle, ϕ', of 33° for Stratum II was used for slope stability analyses. Standard penetration test (SPT) blow counts from the most recent as well as the previous subsurface investigation were used in estimating ϕ'. The following table applies blow count and overburden corrections to obtain $(N_1)_{60}$ values. Terzaghi et al.'s (1996) SPT correlation was then used to approximate the drained friction angle (Figure 8). This method results in an average friction angle of about 35°. Geosyntec slightly reduced the assumed Stratum II friction angle to ϕ'=33°, to add further conservatism to the analysis.

Table 76
STRATUM II FRICTION ANGLE CALCULATION

Borehole	Elevation	Blow Counts, N	N_{60}	Effective Stress, σ'_v	Overburden Correction, C_N	$(N_1)_{60}$	Consistency	Friction Angle
(-)	ft amsl	(-)	Use C_E=0.75	psf	$C_N = (p_a/\sigma'_v)^m \leq 2$, m=0.5 Liao and Whitman (1986)	(-)	(-)	Terzahgi (1996) ϕ' (deg)
BME-1	83.8	25	19	2336	0.95	18	Med. Dense	34
BME-1	81.8	25	19	2588	0.90	17	Med. Dense	34
BME-1	79.8	33	25	2840	0.86	21	Med. Dense	35
BME-1	77.8	49	37	3092	0.83	30	Dense	38
BME-1	75.8	24	18	3344	0.80	14	Med. Dense	33
BME-1	73.8	22	17	3596	0.77	13	Med. Dense	32
BME-1	71.8	25	19	3848	0.74	14	Med. Dense	33
BME-1	69.8	47	35	4100	0.72	25	Med. Dense	36
BME-1	67.8	18	14	4352	0.70	9	Loose	31
BME-1	65.8	29	22	4604	0.68	15	Med. Dense	33
BME-1	63.8	67	50	4856	0.66	33	Dense	39
BME-1	61.8	41	31	5108	0.64	20	Med. Dense	35
BME-2	91.4	39	29	1820	1.08	32	Dense	38
BME-2	90.4	35	26	2072	1.01	27	Med. Dense	37
BME-2	89.4	34	26	2324	0.95	24	Med. Dense	36
BME-2	88.4	44	33	2576	0.91	30	Med. Dense	38
BME-2	87.4	32	24	2828	0.87	21	Med. Dense	35
BME-2	86.4	26	20	3080	0.83	16	Med. Dense	33
BME-2	85.4	28	21	3332	0.80	17	Med. Dense	33
BME-2	84.4	25	19	3584	0.77	14	Med. Dense	33
BME-2	83.4	41	31	3836	0.74	23	Med. Dense	36
BME-2	82.4	41	31	4088	0.72	22	Med. Dense	35

A1.2 Estimating the Drained Friction Angle of Stratum IV

Stratum IV is a sand layer. Blow counts from the most recent subsurface investigation were used to approximate the drained friction angle of Stratum IV. The following table applies blow count and overburden corrections to obtain $(N_1)_{60}$ values. Terzaghi et al.'s (1996) SPT correlation was then used to approximate the drained friction angle (Figure 8). The average drained friction angle, ϕ' = 35° derived from the correlation presented below was used for stability analyses.

Table 8~~7~~
STRATUM IV FRICTION ANGLE CALCULATION

Borehole	Elevation	Blow Counts, N	N_{60}	Effective Stress, σ'_v	Overburden Correction, C_N	$(N_1)_{60}$	Consistency	Friction Angle
(-)	ft amsl	(-)	Use C_E=0.75	psf	$C_N = (p_a/\sigma'_v)^m \leq 2$, m=0.5 Liao and Whitman (1986)	(-)	(-)	Terzaghi (1996) ϕ' (deg)
BME-1	28.8	62	47	9390	0.47	22	Med. Dense	36
BME-1	23.8	62	47	10070	0.46	21	Med. Dense	35
BME-1	18.8	100	75	10750	0.44	33	Dense	39
BME-1	13.8	100	75	11430	0.43	32	Dense	38
BME-3	37.4	26	20	6902	0.55	11	Med. Dense	31
BME-3	32.4	33	25	7582	0.53	13	Med. Dense	32
BME-3	27.4	47	35	8262	0.51	18	Med. Dense	34
BME-3	22.4	54	41	8942	0.49	20	Med. Dense	35
BME-4	39.2	9	7	6240	0.58	4	Very Loose	28
BME-4	34.2	13	10	6920	0.55	5	Loose	29
BME-4	29.2	38	29	7600	0.53	15	Med. Dense	33
BME-4	24.2	45	34	8280	0.51	17	Med. Dense	33
BME-4	19.2	45	34	8960	0.49	16	Med. Dense	33
BME-4	14.2	21	16	9640	0.47	7	Loose	30
BME-4	9.2	26	20	10320	0.45	9	Loose	30
BME-4	4.2	48	36	11000	0.44	16	Med. Dense	33
BME-5	29.1	34	26	6902	0.55	14	Med. Dense	33
BME-5	24.1	41	31	7582	0.53	16	Med. Dense	33
BME-5	19.1	78	59	8262	0.51	30	Med. Dense	38
BME-5	14.1	77	58	8942	0.49	28	Med. Dense	37
BME-5	9.1	78	59	9622	0.47	27	Med. Dense	37
BME-5	4.1	78	59	10302	0.45	27	Med. Dense	37
BME-5	-0.9	77	58	10982	0.44	25	Med. Dense	37
BME-5	-5.9	100	75	11662	0.43	32	Dense	38
BME-5	-10.9	100	75	12342	0.41	31	Dense	38
BME-6	39.2	36	27	5970	0.60	16	Med. Dense	33
BME-6	37.2	56	42	6242	0.58	24	Med. Dense	36
BME-6	32.2	100	75	6922	0.55	41	Dense	40
BME-6	27.2	100	75	7602	0.53	40	Dense	40
BME-6	22.2	47	35	8282	0.51	18	Med. Dense	34
BME-6	17.2	100	75	8962	0.49	36	Dense	39
BME-6	7.2	98	74	10277	0.45	33	Dense	39

A1.4 Estimating the Strength of Compacted Clay for the Cover/Liner and Constructed Fill

The following table is derived from Duncan et al. (1989) for drained and undrained strengths of compacted clay. In the slope stability analyses, drained strengths used for the cover, liner, and constructed fill were c' = 250 psf and ϕ' = 25°. The undrained shear strength assumed was 1600 psf (i.e., the cohesion (c), and with a friction angle (ϕ) of zero).

Table 9~~8~~
SHEAR STRENGTH PROPERTIES OF COMPACTED SOILS
(DUNCAN ET AL. 1989)

USCS Symbol	Soil Type	Typical Strength Characteristics			
		Drained		Undrained	
		c' (psf)	ϕ' (deg)	c (psf)	ϕ (deg)
CL	Inorganic clays of low to medium plasticity	285	28 ± 2	2100 ± 320	1-3
CH	Inorganic clay of high plasticity	245 ± 120	19 ± 5	1800 ± 980	0-2

Analysis #7a

Project Summary

- File Name: SouthSlope 2014-04-10 Undrained Block Liner System Veneer Backcalculated Su.slim
- Slide Modeler Version: 6.027
- Project Title: SLIDE - An Interactive Slope Stability Program
- Date Created: 11/12/2012, 11:59:43 AM

General Settings

- Units of Measurement: Imperial Units
- Time Units: days
- Permeability Units: feet/second
- Failure Direction: Left to Right
- Data Output: Standard
- Maximum Material Properties: 20
- Maximum Support Properties: 20

Analysis Options

Analysis Methods Used

- Spencer

- Number of slices: 30
- Tolerance: 0.005
- Maximum number of iterations: 50
- Check malpha < 0.2: Yes
- Initial trial value of FS: 1
- Steffensen Iteration: Yes

Groundwater Analysis

- Groundwater Method: Water Surfaces
- Pore Fluid Unit Weight: 62.4 lbs/ft3
- Advanced Groundwater Method: None

Surface Options

- Surface Type: Non-Circular Block Search
- Number of Surfaces: 5000
- Pseudo-Random Surfaces: Enabled
- Convex Surfaces Only: Disabled
- Left Projection Angle (Start Angle): 135
- Left Projection Angle (End Angle): 135
- Right Projection Angle (Start Angle): 45
- Right Projection Angle (End Angle): 45
- Minimum Elevation: Not Defined
- Minimum Depth: Not Defined

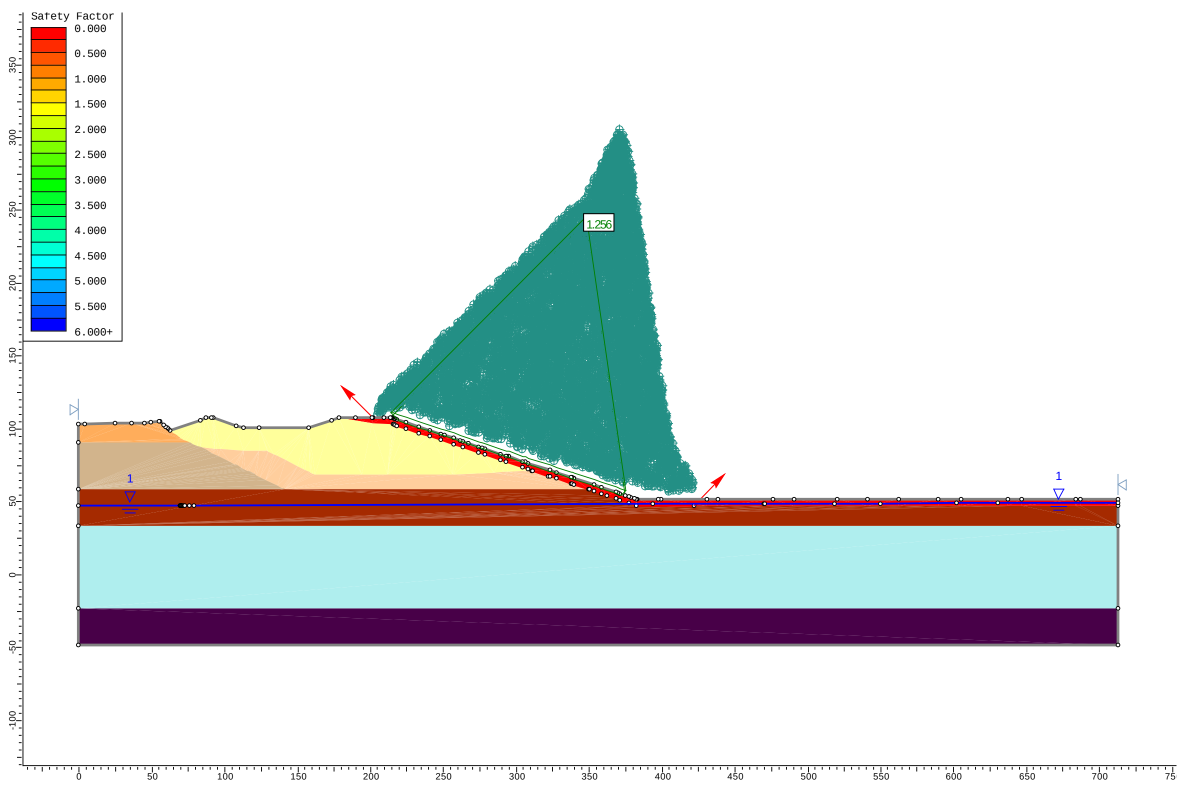
Material Properties

Property	Cover/Liner	Fill Layer	Layer I: Clay	Layer II: Sand	Layer III: Clay	Layer IV: Sand/Silt	Layer V: Clay	Constructed Fill
Color								
Strength Type	Undrained	Undrained	Undrained	Mohr-Coulomb	Undrained	Mohr-Coulomb	Undrained	Undrained
Unit Weight [lbs/ft3]	120	130	134	126	130	136	123	120
Cohesion [psf]				0		0		
Friction Angle [deg]				33		35		
Cohesion Type	174	2074	2304		1490		1500	1600
Water Surface	None	None	None	None	None	Piezometric Line 1	None	None
Hu Value						1		
Ru Value	0	0	0	0	0		0	0

Global Minimums

Method: spencer

- FS: 1.255880
- Axis Location: 347.615, 245.674
- Left Slip Surface Endpoint: 213.680, 111.261
- Right Slip Surface Endpoint: 374.785, 57.878
- Resisting Moment=5.21411e+006 lb-ft
- Driving Moment=4.15177e+006 lb-ft
- Resisting Horizontal Force=28032.2 lb
- Driving Horizontal Force=22320.8 lb
- Total Slice Area=660.091 ft2



Analysis #8a

Project Summary

- File Name: EastSlope 2014-04-10 Undrained Block Liner System Veneer Backcalculated Su.slim
- Slide Modeler Version: 6.027
- Project Title: SLIDE - An Interactive Slope Stability Program
- Date Created: 11/12/2012, 3:14:56 PM

General Settings

- Units of Measurement: Imperial Units
- Time Units: days
- Permeability Units: feet/second
- Failure Direction: Right to Left
- Data Output: Standard
- Maximum Material Properties: 20
- Maximum Support Properties: 20

Analysis Options

Analysis Methods Used

- Spencer

- Number of slices: 30
- Tolerance: 0.005
- Maximum number of iterations: 50
- Check malpha < 0.2: Yes
- Initial trial value of FS: 1
- Steffensen Iteration: Yes

Groundwater Analysis

- Groundwater Method: Water Surfaces
- Pore Fluid Unit Weight: 62.4 lbs/ft3
- Advanced Groundwater Method: None

Surface Options

- Surface Type: Non-Circular Block Search
- Number of Surfaces: 5000
- Pseudo-Random Surfaces: Enabled
- Convex Surfaces Only: Disabled
- Left Projection Angle (Start Angle): 135
- Left Projection Angle (End Angle): 135
- Right Projection Angle (Start Angle): 45
- Right Projection Angle (End Angle): 45
- Minimum Elevation: Not Defined
- Minimum Depth: Not Defined

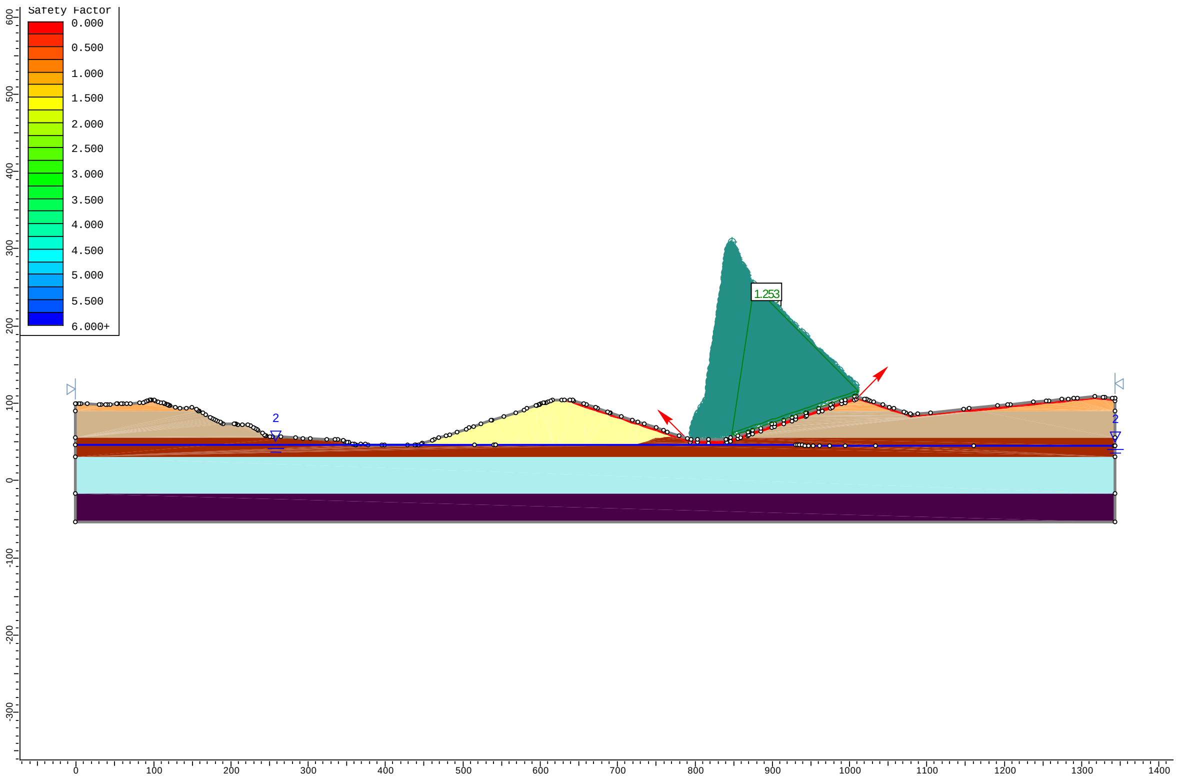
Material Properties

Property	**Cover/Liner**	**Layer I: Clay**	**Layer II: Sand**	**Layer III: Clay**	**Layer IV: Sand/Silt**	**Layer V: Clay**	**Constructed Fill**
Color							
Strength Type	Undrained	Undrained	Mohr-Coulomb	Undrained	Mohr-Coulomb	Undrained	Undrained
Unit Weight [lbs/ft3]	120	134	126	130	136	123	120
Cohesion [psf]			0		0		
Friction Angle [deg]			33		35		
Cohesion Type	170	2304		1490		1500	1600
Water Surface	None	None	None	None	Piezometric Line 2	None	None
Hu Value					1		
Ru Value	0	0	0	0		0	0

Global Minimums

Method: spencer

- FS: 1.252530
- Axis Location: 876.543, 252.243
- Left Slip Surface Endpoint: 847.860, 61.969
- Right Slip Surface Endpoint: 1011.552, 115.132
- Resisting Moment=5.20477e+006 lb-ft
- Driving Moment=4.15541e+006 lb-ft
- Resisting Horizontal Force=27827.6 lb
- Driving Horizontal Force=22217.1 lb
- Total Slice Area=660.378 ft2



Analysis #9a

Project Summary

- File Name: SouthSlope 2014-04-10 Drained Block Final Cover System Veneer Backcalculated c, phi=0.slim
- Slide Modeler Version: 6.027
- Project Title: SLIDE - An Interactive Slope Stability Program
- Date Created: 11/12/2012, 11:59:43 AM

General Settings

- Units of Measurement: Imperial Units
- Time Units: days
- Permeability Units: feet/second
- Failure Direction: Right to Left
- Data Output: Standard
- Maximum Material Properties: 20
- Maximum Support Properties: 20

Analysis Options

Analysis Methods Used

  - Spencer

- Number of slices: 30
- Tolerance: 0.005
- Maximum number of iterations: 50
- Check malpha < 0.2: Yes
- Initial trial value of FS: 1
- Steffensen Iteration: Yes

Groundwater Analysis

- Groundwater Method: Water Surfaces
- Pore Fluid Unit Weight: 62.4 lbs/ft3
- Advanced Groundwater Method: None

Surface Options

- Surface Type: Non-Circular Block Search
- Number of Surfaces: 5000
- Pseudo-Random Surfaces: Enabled
- Convex Surfaces Only: Disabled
- Left Projection Angle (Start Angle): 135
- Left Projection Angle (End Angle): 135
- Right Projection Angle (Start Angle): 45
- Right Projection Angle (End Angle): 45
- Minimum Elevation: Not Defined
- Minimum Depth: Not Defined

Material Properties

Property	**Waste**	**Cover/Liner**	**Fill Layer**	**Layer I: Clay**	**Layer II: Sand**	**Layer III: Clay**	**Layer IV: Sand/Silt**	**Layer V: Clay**
Color								
Strength Type	Shear Normal function	Mohr-Coulomb	Shear Normal function	Mohr-Coulomb	Mohr-Coulomb	Shear Normal function	Mohr-Coulomb	Mohr-Coulomb
Unit Weight [lbs/ft3]	90	120	130	134	126	130	136	123
Cohesion [psf]		82		40	0		0	0
Friction Angle [deg]		0		35	33		35	18
Water Surface	None	None	None	None	None	None	Piezometric Line 1	None
Hu Value							1	
Ru Value	0	0	0	0	0	0		0

Shear Normal Functions

- Name: Waste Conservative

Normal (psf)	**Shear (psf)**
0	501

772	501
62656	40690

- Name: Layer III Conservative Drained

Normal (psf)	Shear (psf)
0	0
1619	1052
43200	16598

- Name: Fill Layer Conservative Drained

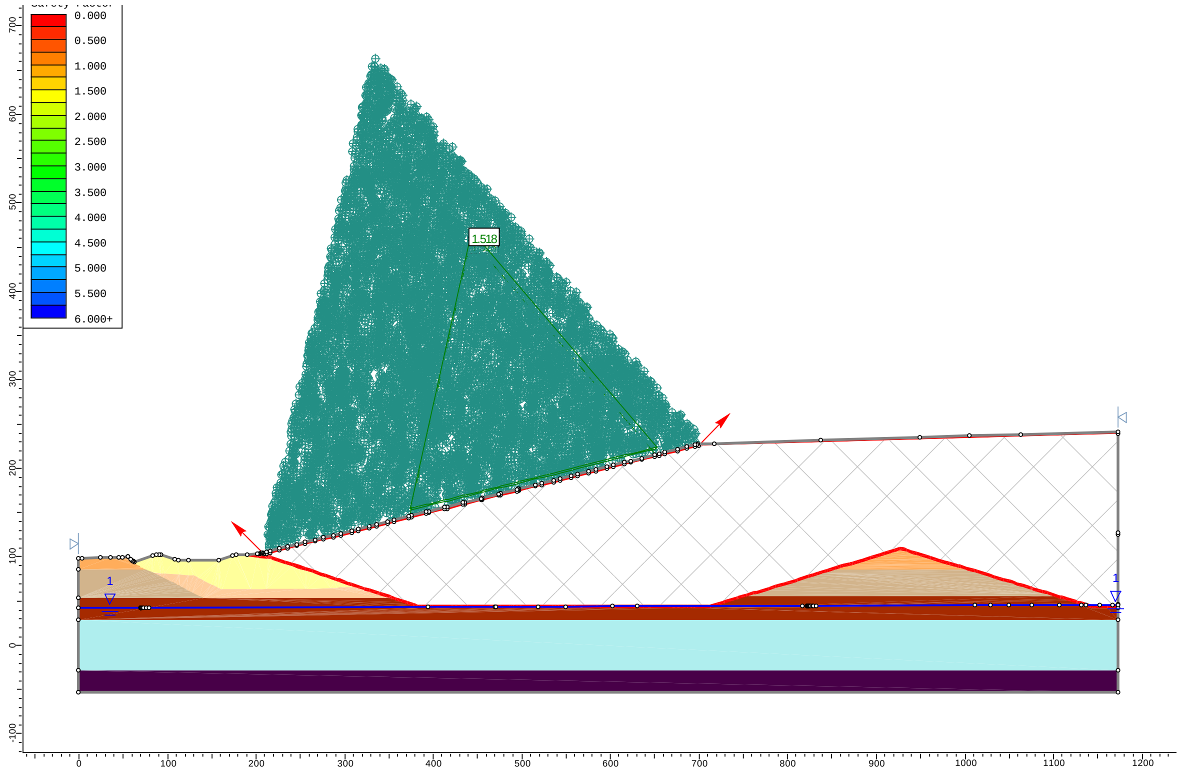
Normal (psf)	Shear (psf)
0	0
12525	4150
43200	11458

Property	Constructed Fill
Color	
Strength Type	Mohr-Coulomb
Unit Weight [lbs/ft3]	120
Cohesion [psf]	250
Friction Angle [deg]	25
Water Surface	None
Ru Value	0

Global Minimums

Method: spencer

- FS: 1.518340
- Axis Location: 443.318, 468.313
- Left Slip Surface Endpoint: 373.559, 154.397
- Right Slip Surface Endpoint: 652.596, 224.156
- Resisting Moment=6.86305e+006 lb-ft
- Driving Moment=4.52011e+006 lb-ft
- Resisting Horizontal Force=22881 lb
- Driving Horizontal Force=15069.8 lb
- Total Slice Area=543.24 ft2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0.500
1.000
1.500
2.000
2.500
3.000
3.500
4.000
4.500
5.000
5.500
6.000+
1.518
1
1
700
600
500
400
300
200
100
0
-100
0
100
200
300
400
500
600
700
800
900
1000
1100
1200

Analysis #10a

Project Summary

- File Name: EastSlope 2014-04-10 Drained Block Final Cover System Veneer Backcalculated c, phi=0.slim
- Slide Modeler Version: 6.027
- Project Title: SLIDE - An Interactive Slope Stability Program
- Date Created: 11/12/2012, 3:14:56 PM

General Settings

- Units of Measurement: Imperial Units
- Time Units: days
- Permeability Units: feet/second
- Failure Direction: Right to Left
- Data Output: Standard
- Maximum Material Properties: 20
- Maximum Support Properties: 20

Analysis Options

Analysis Methods Used

- Spencer

- Number of slices: 30
- Tolerance: 0.005
- Maximum number of iterations: 50
- Check malpha < 0.2: Yes
- Initial trial value of FS: 1
- Steffensen Iteration: Yes

Groundwater Analysis

- Groundwater Method: Water Surfaces
- Pore Fluid Unit Weight: 62.4 lbs/ft3
- Advanced Groundwater Method: None

Surface Options

- Surface Type: Non-Circular Block Search
- Number of Surfaces: 5000
- Pseudo-Random Surfaces: Enabled
- Convex Surfaces Only: Disabled
- Left Projection Angle (Start Angle): 135
- Left Projection Angle (End Angle): 135
- Right Projection Angle (Start Angle): 45
- Right Projection Angle (End Angle): 45
- Minimum Elevation: Not Defined
- Minimum Depth: Not Defined

Material Properties

Property	**Waste**	**Cover/Liner**	**Layer I: Clay**	**Layer II: Sand**	**Layer III: Clay**	**Layer IV: Sand/Silt**	**Layer V: Clay**	**Constructed Fill**
Color								
Strength Type	Shear Normal function	Mohr-Coulomb	Mohr-Coulomb	Mohr-Coulomb	Shear Normal function	Mohr-Coulomb	Mohr-Coulomb	Mohr-Coulomb
Unit Weight [lbs/ft3]	90	120	134	126	130	136	123	120
Cohesion [psf]		82	40	0		0	0	250
Friction Angle [deg]		0	35	33		35	18	25
Water Surface	None	None	None	None	None	Piezometric Line 2	None	None
Hu Value						1		
Ru Value	0	0	0	0	0		0	0

Shear Normal Functions

- Name: Waste Conservative

Normal (psf)	**Shear (psf)**
0	501

772	501
62656	40690

- Name: Layer III Conservative Drained

Normal (psf)	Shear (psf)
0	0
1619	1052
43200	16598

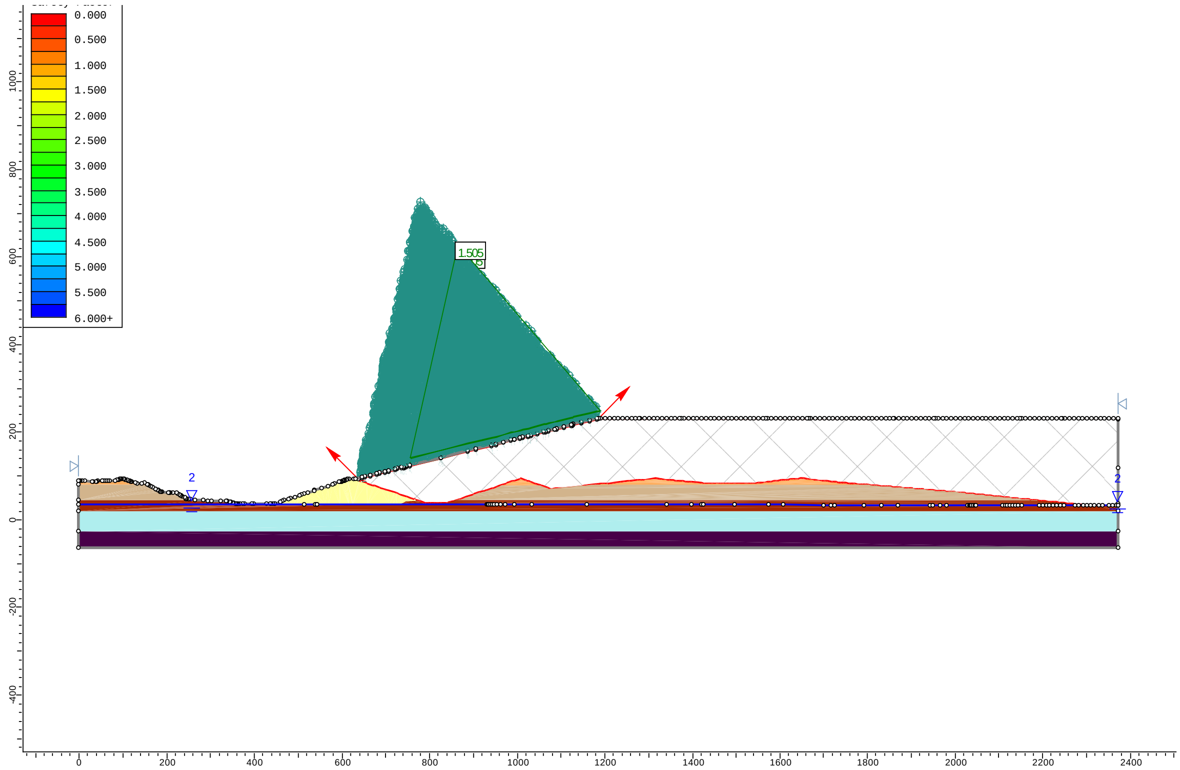
- Name: Fill Layer Conservative Drained

Normal (psf)	Shear (psf)
0	0
12525	4150
43200	11458

Global Minimums

Method: spencer

- FS: 1.504870
- Axis Location: 865.143, 627.803
- Left Slip Surface Endpoint: 756.661, 142.107
- Right Slip Surface Endpoint: 1188.611, 249.600
- Resisting Moment=1.63492e+007 lb-ft
- Driving Moment=1.08642e+007 lb-ft
- Resisting Horizontal Force=35419.9 lb
- Driving Horizontal Force=23536.8 lb
- Total Slice Area=843.806 ft2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0.500
1.000
1.500
2.000
2.500
3.000
3.500
4.000
4.500
5.000
5.500
6.000+
1.505

Analysis #13a

Project Summary

- File Name: InterimSlope 2014-04-10 Undrained Block Liner Backcalculated Su.slim
- Slide Modeler Version: 6.027
- Project Title: SLIDE - An Interactive Slope Stability Program
- Date Created: 11/12/2012, 1:08:13 PM

General Settings

- Units of Measurement: Imperial Units
- Time Units: days
- Permeability Units: feet/second
- Failure Direction: Right to Left
- Data Output: Standard
- Maximum Material Properties: 20
- Maximum Support Properties: 20

Analysis Options

Analysis Methods Used

  - Spencer

- Number of slices: 30
- Tolerance: 0.005
- Maximum number of iterations: 50
- Check malpha < 0.2: Yes
- Initial trial value of FS: 1
- Steffensen Iteration: Yes

Groundwater Analysis

- Groundwater Method: Water Surfaces
- Pore Fluid Unit Weight: 62.4 lbs/ft3
- Advanced Groundwater Method: None

Surface Options

- Surface Type: Non-Circular Block Search
- Number of Surfaces: 5000
- Pseudo-Random Surfaces: Enabled
- Convex Surfaces Only: Disabled
- Left Projection Angle (Start Angle): 135
- Left Projection Angle (End Angle): 135
- Right Projection Angle (Start Angle): 45
- Right Projection Angle (End Angle): 45
- Minimum Elevation: Not Defined
- Minimum Depth: Not Defined

Material Properties

Property	**Waste**	**Cover/Liner**	**Fill Layer**	**Layer I: Clay**	**Layer II: Sand**	**Layer III: Clay**	**Layer IV: Sand/Silt**	**Layer V: Clay**
Color								
Strength Type	Shear Normal function	Undrained	Undrained	Undrained	Mohr-Coulomb	Undrained	Mohr-Coulomb	Undrained
Unit Weight [lbs/ft3]	90	120	130	134	126	130	136	123
Cohesion [psf]					0		0	
Friction Angle [deg]					33		35	
Cohesion Type		375	2074	2304		1490		1500
Water Surface	None	None	None	None	None	None	Piezometric Line 1	None
Hu Value							1	
Ru Value	0	0	0	0	0	0		0

Shear Normal Functions

- Name: Waste Conservative

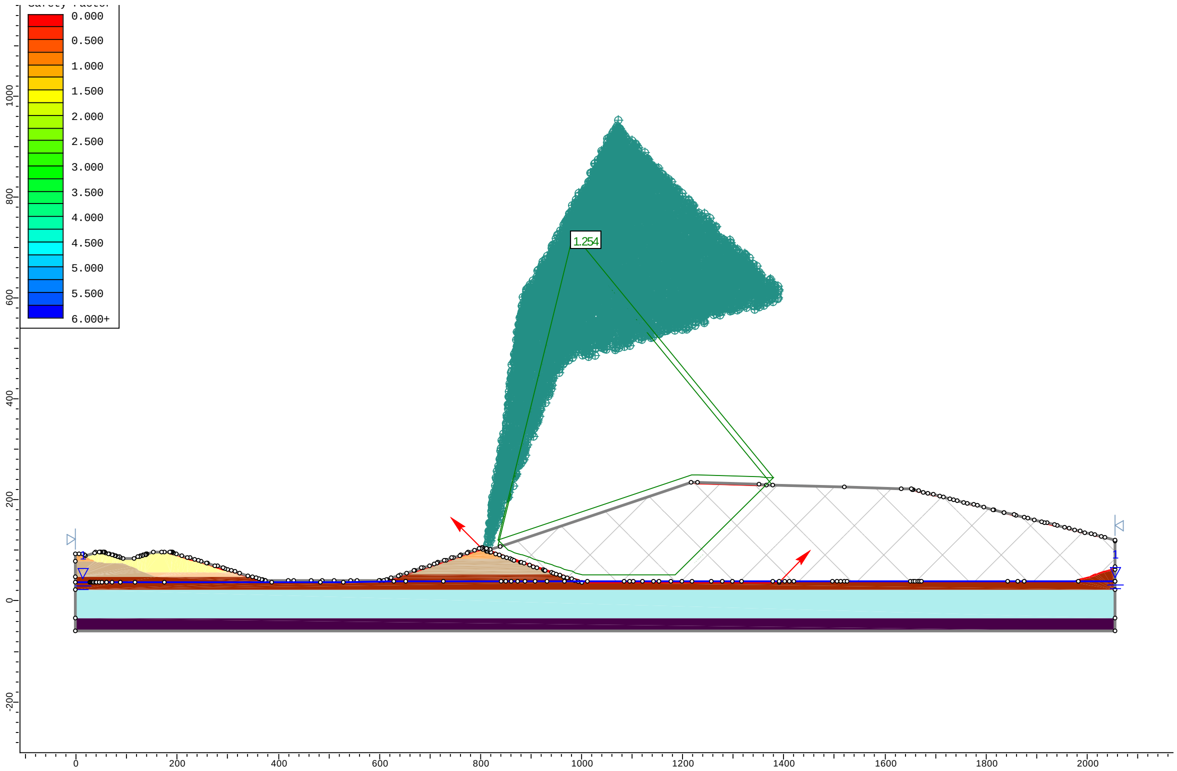
Normal (psf)	Shear (psf)
0	501
772	501
62656	40690

Property	Constructed Fill
Color	
Strength Type	Undrained
Unit Weight [lbs/ft3]	120
Cohesion Type	1600
Water Surface	None
Ru Value	0

Global Minimums

Method: spencer

- FS: 1.253510
- Axis Location: 982.578, 727.205
- Left Slip Surface Endpoint: 833.719, 120.771
- Right Slip Surface Endpoint: 1378.410, 244.257
- Resisting Moment=7.05809e+008 lb-ft
- Driving Moment=5.63069e+008 lb-ft
- Resisting Horizontal Force=838417 lb
- Driving Horizontal Force=668858 lb
- Total Slice Area=59263.8 ft2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0.500
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2.000
2.500
3.000
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4.000
4.500
5.000
5.500
6.000+
1.254
1000
800
600
400
200
0
-200
0
200
400
600
800
1000
1200
1400
1600
1800
2000

Analysis #18a

Project Summary

- File Name: SouthSlope 2014-04-10 Drained Block Liner Backcalculated c, phi=0.slim
- Slide Modeler Version: 6.027
- Project Title: SLIDE - An Interactive Slope Stability Program
- Date Created: 11/12/2012, 11:59:43 AM

General Settings

- Units of Measurement: Imperial Units
- Time Units: days
- Permeability Units: feet/second
- Failure Direction: Right to Left
- Data Output: Standard
- Maximum Material Properties: 20
- Maximum Support Properties: 20

Analysis Options

Analysis Methods Used

  - Spencer

- Number of slices: 30
- Tolerance: 0.005
- Maximum number of iterations: 50
- Check malpha < 0.2: Yes
- Initial trial value of FS: 1
- Steffensen Iteration: Yes

Groundwater Analysis

- Groundwater Method: Water Surfaces
- Pore Fluid Unit Weight: 62.4 lbs/ft3
- Advanced Groundwater Method: None

Surface Options

- Surface Type: Non-Circular Block Search
- Number of Surfaces: 5000
- Pseudo-Random Surfaces: Enabled
- Convex Surfaces Only: Disabled
- Left Projection Angle (Start Angle): 135
- Left Projection Angle (End Angle): 135
- Right Projection Angle (Start Angle): 45
- Right Projection Angle (End Angle): 45
- Minimum Elevation: Not Defined
- Minimum Depth: Not Defined

Material Properties

Property	**Waste**	**Cover/Liner**	**Fill Layer**	**Layer I: Clay**	**Layer II: Sand**	**Layer III: Clay**	**Layer IV: Sand/Silt**	**Layer V: Clay**
Color								
Strength Type	Shear Normal function	Mohr-Coulomb	Shear Normal function	Mohr-Coulomb	Mohr-Coulomb	Shear Normal function	Mohr-Coulomb	Mohr-Coulomb
Unit Weight [lbs/ft3]	90	120	130	134	126	130	136	123
Cohesion [psf]		650		40	0		0	0
Friction Angle [deg]		0		35	33		35	18
Water Surface	None	None	None	None	None	None	Piezometric Line 1	None
Hu Value							1	
Ru Value	0	0	0	0	0	0		0

Shear Normal Functions

- Name: Waste Conservative

Normal (psf)	**Shear (psf)**
0	501

772	501
62656	40690

- Name: Layer III Conservative Drained

Normal (psf)	**Shear (psf)**
0	0
1619	1052
43200	16598

- Name: Fill Layer Conservative Drained

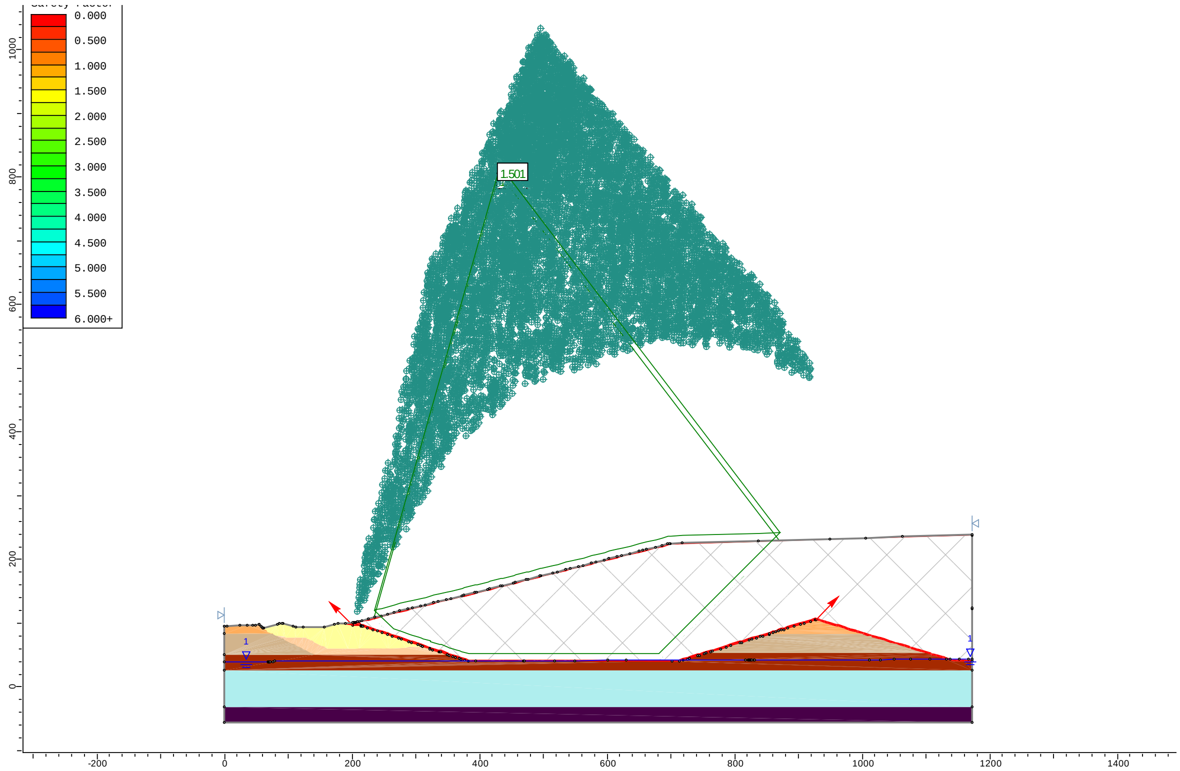
Normal (psf)	**Shear (psf)**
0	0
12525	4150
43200	11458

Property	**Constructed Fill**
Color	
Strength Type	Mohr-Coulomb
Unit Weight [lbs/ft3]	120
Cohesion [psf]	250
Friction Angle [deg]	25
Water Surface	None
Ru Value	0

Global Minimums

Method: spencer

- FS: 1.500760
- Axis Location: 432.038, 816.334
- Left Slip Surface Endpoint: 235.256, 119.821
- Right Slip Surface Endpoint: 871.179, 241.001
- Resisting Moment=9.25873e+008 lb-ft
- Driving Moment=6.16938e+008 lb-ft
- Resisting Horizontal Force=986941 lb
- Driving Horizontal Force=657629 lb
- Total Slice Area=69100.1 ft2


0.000
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5.500
6.000+
1.501
1000
800
600
400
200
0
-200
0
200
400
600
800
1000
1200
1400

Analysis #23a

Project Summary

- File Name: EastSlope 2014-04-10 Drained Block Liner Backcalculated c, phi=0.slim
- Slide Modeler Version: 6.027
- Project Title: SLIDE - An Interactive Slope Stability Program
- Date Created: 11/12/2012, 3:14:56 PM

General Settings

- Units of Measurement: Imperial Units
- Time Units: days
- Permeability Units: feet/second
- Failure Direction: Right to Left
- Data Output: Standard
- Maximum Material Properties: 20
- Maximum Support Properties: 20

Analysis Options

Analysis Methods Used

  - Spencer

- Number of slices: 30
- Tolerance: 0.005
- Maximum number of iterations: 50
- Check malpha < 0.2: Yes
- Initial trial value of FS: 1
- Steffensen Iteration: Yes

Groundwater Analysis

- Groundwater Method: Water Surfaces
- Pore Fluid Unit Weight: 62.4 lbs/ft3
- Advanced Groundwater Method: None

Surface Options

- Surface Type: Non-Circular Block Search
- Number of Surfaces: 5000
- Pseudo-Random Surfaces: Enabled
- Convex Surfaces Only: Disabled
- Left Projection Angle (Start Angle): 135
- Left Projection Angle (End Angle): 135
- Right Projection Angle (Start Angle): 45
- Right Projection Angle (End Angle): 45
- Minimum Elevation: Not Defined
- Minimum Depth: Not Defined

Material Properties

Property	**Waste**	**Cover/Liner**	**Layer I: Clay**	**Layer II: Sand**	**Layer III: Clay**	**Layer IV: Sand/Silt**	**Layer V: Clay**	**Constructed Fill**
Color								
Strength Type	Shear Normal function	Mohr-Coulomb	Mohr-Coulomb	Mohr-Coulomb	Shear Normal function	Mohr-Coulomb	Mohr-Coulomb	Mohr-Coulomb
Unit Weight [lbs/ft3]	90	120	134	126	130	136	123	120
Cohesion [psf]		490	40	0		0	0	250
Friction Angle [deg]		0	35	33		35	18	25
Water Surface	None	None	None	None	None	Piezometric Line 2	None	None
Hu Value						1		
Ru Value	0	0	0	0	0		0	0

Shear Normal Functions

- Name: Waste Conservative

Normal (psf)	**Shear (psf)**
0	501

772	501
62656	40690

- Name: Layer III Conservative Drained

Normal (psf)	Shear (psf)
0	0
1619	1052
43200	16598

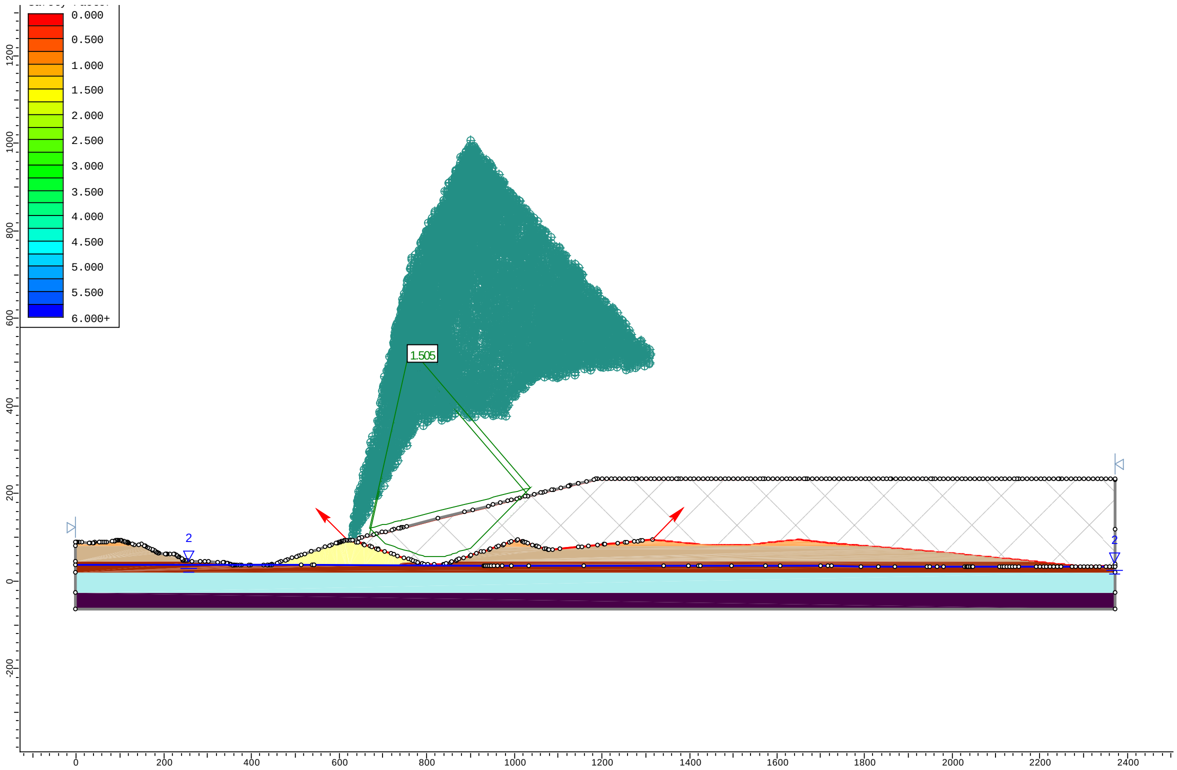
- Name: Fill Layer Conservative Drained

Normal (psf)	Shear (psf)
0	0
12525	4150
43200	11458

Global Minimums

Method: spencer

- FS: 1.504840
- Axis Location: 761.964, 532.629
- Left Slip Surface Endpoint: 670.345, 120.537
- Right Slip Surface Endpoint: 1036.667, 212.079
- Resisting Moment=2.38546e+008 lb-ft
- Driving Moment=1.58519e+008 lb-ft
- Resisting Horizontal Force=430971 lb
- Driving Horizontal Force=286389 lb
- Total Slice Area=24938.5 ft2


0.000
0.500
1.000
1.500
2.000
2.500
3.000
3.500
4.000
4.500
5.000
5.500
6.000+
1.505

ATTACHMENT 3D.4.2

BALLAST UPLIFT CALCULATION

BALLAST UPLIFT CALCULATION
FAIRBANKS LANDFILL

FOR PERMIT PURPOSES ONLY;
CALCULATION PAGES 1
THROUGH 131

GEOSYNTEC CONSULTANTS, INC.
TX ENG. FIRM REGISTRATION NO. F-1182

1. INTRODUCTION

The purpose of this calculation package is to calculate the thickness of ballast required to resist uplift pressures on the liner system due to the presence of perched groundwater within Stratum II.

2. METHODOLOGY

The Texas Commission on Environmental Quality (TCEQ) recommends a minimum factor of safety (FS_{min}) against liner system uplift of 1.2 if no ballast is required or if soils are used as ballast. Alternatively, if waste is selected as ballast, the required long-term FS_{min} is 1.5. The required thickness of ballast on the liner system to achieve these FS_{min} values can be calculated by the following steps:

- Select critical points for evaluation of a cell (i.e., sector) (based on local groundwater conditions with respect to landfill base and/or side slope elevations), top of liner, and critical subsurface strata. Evaluate the elevations of the seasonal high groundwater table (SHGT) (synonymous with the "historical high" groundwater levels). Or, use observed groundwater levels if conditions are intermittent and not represented by a continuous water table.

- Select the required long-term factor of safety against uplift (1.2 or 1.5) depending on the ballast material.

- Calculate the maximum hydrostatic uplift force, U_N, acting normal to the liner (see free body diagram in Figure 3D.4.2-1) at each point:

$$U_N = \gamma_w \times H_{wt}$$

where: γ_w = unit weight of water; H_{wt} = vertical distance from the liner to the seasonal high groundwater table.

- Evaluate the unit weight of the ballast materials (soil and/or waste):

When possible, the total unit weight of the soil ballast layers should be verified by laboratory or field data. If these data are not available, the following unit weights may be used:

Waste - total unit weight of the waste used in uplift stability calculations For municipal solid waste, TCEQ requires in 30 TAC §330.337(h)(2) that the unit weight of waste used as ballast material be selected as 1,200 pounds per cubic yard, or 44 pounds per cubic foot. Since this landfill is a Type IV and will not have MSW, but rather will have a construction and demolition (C&D) type of waste, it is likely that the waste will be even ~~more dense~~denser (heavy). However, for conservatism, 44 pounds per cubic foot will be used as the unit weight of waste in these calculations.

Protective Cover - Assume loose dumped unit weight of protective cover soil as 70% of the typical in-situ unit weight. If material is lightly compacted during placement, 80% of the typical in-situ or standard Proctor maximum unit weight may be used. From these guidelines and the anticipated light compaction during placement (e.g., dozer), a value of 90 pounds per cubic foot was selected for the unit weight of protective cover material.

Compacted Clay Liner – The recompacted clay liner material will be compacted to 95% dry density. A value of 115.6 pcf was selected for computing the resistance to uplift by the compacted clay liner. Note that this value is slightly lower than the value used in the slope stability analyses in Attachment 3D.2. The lower unit weight selected here is conservative in terms of this uplift calculation.

- Calculate the resisting force, R_N, provided by recompacted clay liner and protective cover soils acting normal to the liner (see free body diagram in Figure 3D.4.2-1) at each point:

$$R_N = R_V \times \cos\beta + R_H \times \sin\beta = \Sigma(\gamma_i \times T_i) \times \cos\beta + \Sigma(K_o \times \gamma_i \times T_i) \times \sin\beta$$

where: R_V = vertical resisting force; R_H = horizontal resisting force; γ_i = total unit weight of the i^{th} ballast component above the liner; T_i = vertical thickness of the i^{th} ballast component above the liner; K_0 = coefficient of static earth pressure provided by the liner (as shown in Figure 3D.4.2-2 (Holtz and Kovacs, 1981)); and β = the slope of the liner system. It is noted that the lateral earth pressure from the liner and protective soil at the calculation point provide the resisting force against uplift.

- Calculate the provided FS without ballast at each point:

$$FS = R_N / U_N = [\Sigma(\gamma_i \times T_i) \times \cos\beta + \Sigma(K_o \times \gamma_i \times T_i) \times \sin\beta] / (\gamma_w \times H_{wt})$$

If the provided FS is greater than or equal to FS_{min}, then no ballast is required. If FS is less than the FS_{min}, then ballast is required.

- If ballast is required, calculate the required thickness, T_i, of the ballast materials assuming that only the vertical pressure of the ballast contributes to the additional resistance against uplift:

$$\Sigma(\gamma_i T_i) * \cos\beta = ((FS_{min} * U_N) - R_N)$$

3. CALCULATIONS

The following section presents the calculations to evaluate the required thickness of ballast to resist uplift for two potential ballast materials: Waste-as-Ballast (Case I) and Soil-as-Ballast (Case II). Geologic cross sections were developed for the site and are provided in the Geology Report (Part III, Attachment 4), which give an indication of where the water-bearing zone that will encounter the sidewall liner system in places is located (i.e., Stratum II). The base liner system grading plan and final cover grading plan are presented in Part III, Attachment 3, Drawings 3-2 and 3-3, respectively. Finally, a map of the historical high groundwater elevations in Stratum II is presented in the Liner Quality Control Plan (LCQP) in Attachment 3C. The historical high groundwater elevations in Stratum II are used to calculate the uplift forces.

The liner in the southwestern part of landfill (Sector R) is selected as the critical case for design as this location has the highest elevation difference between the SHGT and the liner base grades. A representative typical/idealized cross section of the landfill liner at this worst-case location is provided in Figure 3D.4.2-1 3 of this calculation package. As shown in Figure 3D.4.2-13, the bottom of Stratum II (perched water bearing stratum) is located at elevation 60 ft MSL, and will encounter the liner sidewall. Therefore, hydrostatic uplift was evaluated along the 3 horizontal: 1 vertical (3H:1V) liner side slope a few feet above the Stratum II and Stratum III interface. At this location, the historical high groundwater table elevation is 86 ft MSL. The height of the water table in this area is calculated as:

$$H_{wt} = 86\ ft\ MSL - 60\ ft\ MSL = 26\ ft$$

The uplift force on the 3H:1V liner side slopes (β = 18.43°) is computed as:

$$U = (H_{wt} \times \gamma_w) = (26\ ft \times 62.4\ pcf) = 1622.4\ psf$$

The pre-ballast resisting force is evaluated based on resistance available from a 3-ft thick compacted clay liner with 1-ft of protective cover. The resisting force is a combination of horizontal and vertical components and computed as follows:

$$R_N = \Sigma(\gamma_i \times T_i) \times \cos\beta + \Sigma(K_0 \times \gamma_i \times T_i) \times \sin\beta$$

R_N = [(115.6 pcf × 3 ft + 90.0 pcf × 1 ft) × cos(18.43°)] +[1-sin(18°)]× (115.6 pcf × 3 ft + 90.0 pcf × 1 ft) × sin(18.43°)

R_N = 509.8 psf

where, the coefficient of static earth pressure (K_0) is defined as 1-sinϕ; and ϕ was selected as 18° for recompacted clay liner for the purposes of this computation.

The calculated factor of safety without ballast is:

$FS = R_N / U_N$ = 509.8 psf /1622.4 psf = 0.31

Therefore, ballast will be required to resist calculated uplift pressures from Stratum II along the liner of this evaluated location in Sector R. If waste is selected as the ballast material (Case I), sufficient ballast should be placed to achieve a FS_{min} equal to 1.5 against uplift. The thickness of waste to be used as ballast (T_{wb}) material is calculated as:

$$T_{wb} = ((FS_{min} \times U_N) - R_N) / (\gamma_{wb} \times \cos\beta)$$

$$T_{wb} = ((1.5 \times 1622.4\ psf) - 509.8\ psf) / (44\ pcf \times \cos(18.43°)) = 46.1\ ft$$

Therefore, the required thickness of waste if used as ballast in Sector R where it encounters Stratum II along the sidewall is 47.0 ft (rounded up).

Similarly, if soil material is selected as ballast (Case II), sufficient ballast should be placed to achieve a FS_{min} equal to 1.2 against uplift. The thickness of soil ballast (T_{sb}) is calculated as:

$$T_{sb} = ((FS_{min} \times U_N) - R_N) / (\gamma_{sb} \times \cos\beta)$$

$$T_{sb} = ((1.2 \times 1622.4\ psf) - 509.8\ psf) / (90\ pcf \times \cos(18.43°)) = 16.8\ ft$$

Therefore, the required thickness of soil material if soil is used as ballast in Sector R where it encounters Stratum II along the sidewall is 17.0 ft (rounded up).

4. RESULTS

Design calculations as shown above were conducted for the north portion of Sector Q, the south portion of Sector Q, Sector R, Sector S, and Sector T (i.e., the proposed sectors that have not yet been constructed). The calculations for required thickness of ballast required in each sector are summarized in Table 3D.4.2-1. Since the base (floor) of the landfill liner will be keyed-in to the clayey Stratum III and groundwater is not expected to encounter the floor of the landfill, the computations presented herein are performed at the intersection of Stratum II and the liner side slopes, using the same methodology presented above.

5. CONCLUSIONS

Uplift and ballast computations were performed for various cells at Fairbanks landfill based on the SHGT elevation, the extent of Stratum II, and the landfill base grades. An underdrain system (i.e., pressure relief/dewatering system) will be used to control groundwater prior to sufficient ballast being in-place. When waste placement is high enough, it will serve as ballast to counteract uplift forces on the sidewall. The required thickness of ballast for the applicable landfill sectors is provided in Table 3D.4.2-1. Note that the calculations were performed for two cases - using either soil or waste as ballast - although it is expected that waste will be used as ballast. It is also noted that the required thickness of ballast refers to the ballast necessary to resist uplift forces at the intersection of the base of Stratum II with the sidewall (i.e., at the deepest/worst-case point).

As landfill waste filling progresses, the actual waste thickness will exceed the minimum required thickness of waste-ballast (See Figure 3D.4.2-~~1~~3). This demonstrates that waste may be used as ballast, without the need to be supplemented by additional soil ballast placement.

As discussed in the Liner Quality Control Plan (LQCP), an underdrain will be provided in areas where the liner encounters Stratum II. The underdrain will be operated until the thickness of ballast (waste) placed within each cell reaches the required thickness to resist uplift with an adequate calculated factor of safety. Furthermore, placement of ballast will be documented in a Ballast Evaluation Report (BER) in accordance to the LQCP.

6. REFERENCES

Holtz, R.D. and W.D. Kovacs, (1981). "An Introduction to Geotechnical Engineering", Prentice-Hall, Inc., New Jersey, pp. 225-226, 519.

TABLES

- **Table 3D.4.2-1. Summary of Uplift and Ballast Calculation Results**

Table 3D.4.2-1. Summary of Uplift and Ballast Calculation Results

Cell No.	Base Elev.	Stratum II Elev.	SHGT Elev. [1]	H_{wt}	U_N	R_V	R_H	R_N	FS[2]	T_{wb}[3]	T_{sb}[4]	FS_{wb}[5]	FS_{sb}[6]
Units	ft msl	ft msl	ft msl	ft	psf	psf	psf	psf	-	ft	ft	-	-
Sector Q (north)	70.0	58.0	85.0	15.0	936.0	436.8	301.8	509.8	0.54	22.0	8.0	1.58	1.31
Sector Q (south)	60.0	60.0	79.5	19.5	1216.8	436.8	301.8	509.8	0.42	32.0	12.0	1.58	1.31
Sector R	55.0	60.0	86.0	26.0	1622.4	436.8	301.8	509.8	0.31	47.0	17.0	1.59	1.26
Sector S	55.0	60.0	84.0	24.0	1497.6	436.8	301.8	509.8	0.34	42.0	16.0	1.57	1.30
Sector T	55.0	59.5	81.0	21.5	1341.6	436.8	301.8	509.8	0.38	36.0	13.0	1.56	1.25

Notes:

1. SHGT = Seasonally High Groundwater Table (synonymous with historical high groundwater levels).
2. Factor of Safety without ballast material.
3. Thickness of Waste Ballast (T_{wb}) material (rounded up to nearest 1 ft) above the Stratum II Elevation needed to provide FS_{min} of 1.5. **[Use this column to select the required minimum thickness of waste that would provide sufficient ballast to warrant ceasing operation of the underdrain system at that sector/location – provided that this is confirmed and documented in the requisite BER submittal.]**
4. Thickness of Soil Ballast (T_{sb}) material above the Stratum II Elevation (rounded up to nearest 1 ft) needed to provide FS_{min} of 1.2.
5. Factor of Safety with T_{wb} of waste ballast material.
6. Factor of Safety with T_{sb} of soil ballast material.

FIGURES

- **Figure 3D.4.2-1: Example Free Body Diagram at Liner Side Slope**
- **Figure 3D.4.2-2: Excerpts from Holtz and Kovacs (1981) on Lateral Earth Pressure**
- **Figure 3D.4.2-13. Typical/Idealized Cross-Section**

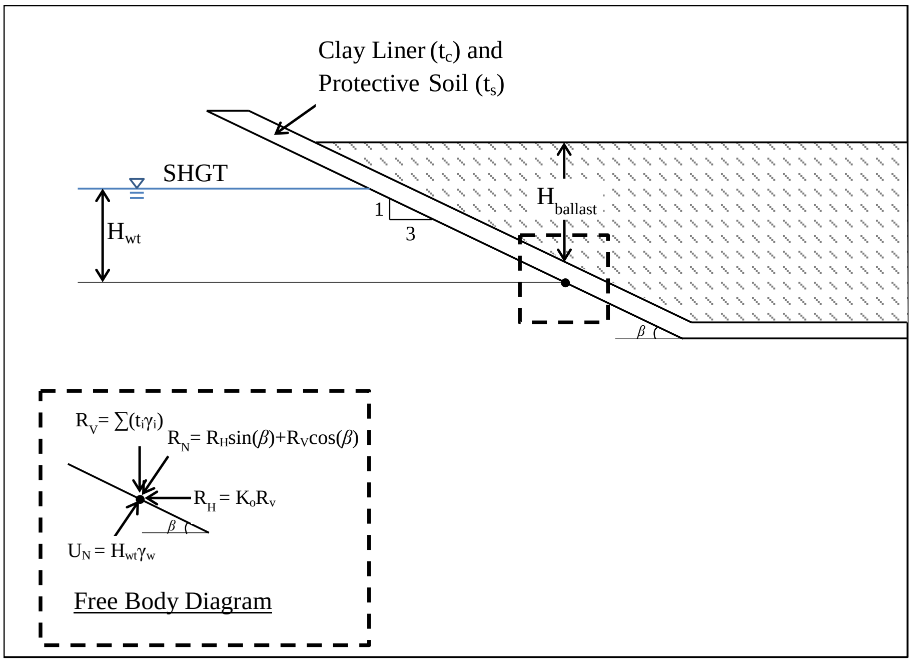


Figure 3D.4.2-1: Example Free Body Diagram at Liner Side Slope

Notes:

1. $H_{ballast}$ is the thickness of ballast (waste or soil) above the calculation point on the liner side slopes.
2. K_o is the coefficient of static earth pressure of the clay liner material and is defined by the equation $K_o = 1-\sin(\phi)$ (Holtz and Kovacs, 1981); where ϕ was selected as 18° for this computation.
3. SHGT = Seasonally High Groundwater Table (synonymous with historical high groundwater levels).

7.6 RELATIONSHIP BETWEEN HORIZONTAL AND VERTICAL STRESSES

You may recall from hydrostatics that the pressure in a liquid is the same in any direction—up, down, sideways, or at any inclination, it doesn't matter. However this is not true in soils. Rarely in natural soil deposits is the horizontal stress in the ground equal exactly to the vertical stress. In other words, the stresses in situ are not necessarily hydrostatic. We can express the ratio of the horizontal to vertical stress in the ground as

$$\sigma_h = K\sigma_v \qquad (7\text{-}18)$$

where K is an *earth pressure coefficient*. Since the ground water table can fluctuate and the total stresses can change, the coefficient K is *not* a constant for a particular soil deposit. However, if we express this ratio in terms of effective stresses, we take care of the problem of a variable water table, or

$$\sigma_h' = K_o\sigma_v' \qquad (7\text{-}19)$$

K_o is a very important coefficient in geotechnical engineering. It is called the *coefficient of lateral earth pressure at rest*. It expresses the stress conditions in the ground in terms of *effective stresses*, and it is independent of the location of the ground water table. Even if the depth changes, K_o will be a constant as long as we are in the same soil layer and the density remains the same. However this coefficient is very sensitive to the geologic and engineering stress history, as well as to the densities of the overlying soil layers (see for example, Massarsch, et al., 1975). The value of K_o is important in stress and analyses, in assessing the shearing resistance of

particular soil layers, and in such geotechnical problems as the design of earth-retaining structures, earth dams and slopes, and many foundation engineering problems.

The K_o in natural soil deposits can be as low as 0.4 or 0.5 for sedimentary soils that have never been preloaded or up to 3.0 or greater for some very heavily preloaded deposits. Typical values of K_o for different geologic conditions are given in Chapter 11.

Figure 3D.4.2-2: Excerpts from Holtz and Kovacs (1981) on Lateral Earth Pressure

The best known equation for estimating K_o was derived by Jáky (1944, 1948), which is a theoretical relationship between K_o and the angle of internal friction ϕ', or

$$K_o = 1 - \sin\phi' \tag{11-6}$$

This relationship, as shown in Fig. 11.14, seems to be an adequate predictor of K_o for normally consolidated sands. Since most of the points lie between 0.35 and 0.5 for these sands, K_o of 0.4 to 0.45 would be a reasonable average value to use for preliminary design purposes.

Figure 3D.4.2-2: Excerpts from Holtz and Kovacs (1981) on Lateral Earth Pressure (Continued)

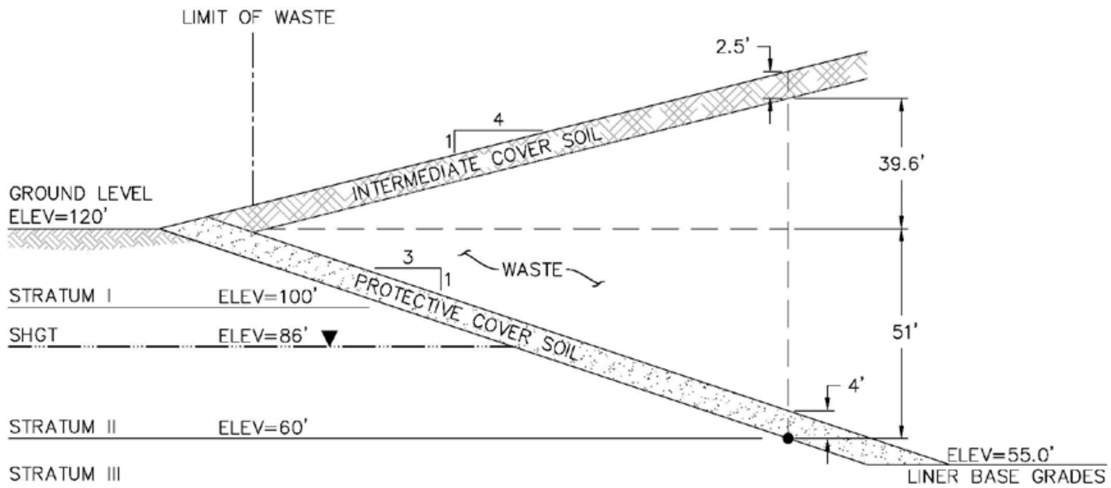


Figure 3D.4.2-~~1~~3: Typical/Idealized Cross-Section

Note: This figure demonstrates that the typical waste filling operation to final permitted waste grades will provide over 90-ft of waste thickness above the critical sidewall location at the base of Stratum II. The calculations indicate that about 47-ft of waste ballast is required to provide a sufficient factor of safety against uplift in Sector R. This shows that through the course of waste filling, sufficient waste will be placed to resist uplift under the calculated conditions.

ATTACHMENT 4

GEOLOGY REPORT

FAIRBANKS LANDFILL
HARRIS COUNTY, TEXAS
TCEQ PERMIT NO. MSW 1565B

PERMIT AMENDMENT APPLICATION

PART III – SITE DEVELOPMENT PLAN
ATTACHMENT 4
GEOLOGY REPORT

Prepared for

USA WASTE OF TEXAS LANDFILLS, INC.
A WASTE MANAGEMENT COMPANY

August 2013

Revised March 2014

Revised May 2014

STATE OF TEXAS
JOHN MICHAEL SNYDER
GEOLOGY
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5-2-2014

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CONTENTS (Continued)

Table 4-9
Hydraulic Conductivity Values
Fairbanks Landfill

Monitoring Well No.	Hydraulic Conductivity (cm/sec)	Rising/Falling Head
Layer II		
MW-1A	1.0×10^{-3}	Rising Head
MW-1A	8.5×10^{-4}	Falling Head
MW-2A	5.6×10^{-4}	Rising Head
MW-2A	6.0×10^{-4}	Rising Head
MW-2A	7.0×10^{-4}	Falling Head
MW-7A	7.5×10^{-4}	Rising Head
MW-7A	1.2×10^{-3}	Rising Head
Geometric Mean Layer II	7.84×10^{-4}	
Layer IV – Uppermost Aquifer		
P-3B	2.6×10^{-4}	Falling Head
P-3B	5.2×10^{-4}	Rising Head
P-4B	3.0×10^{-3}	Falling Head
P-4B	3.3×10^{-3}	Rising Head
P-5B	1.9×10^{-4}	Falling Head
P-5B	1.7×10^{-4}	Rising Head
Geometric Mean Layer IV	5.92×10^{-4}	

6.3 Site Hydrogeology

Since the 1998 permit amendment application for the site, Layer II has been identified as the uppermost groundwater zone rather than the uppermost aquifer. Layer IV has been consistently identified as the uppermost aquifer in groundwater monitoring reports submitted to the TCEQ since that time and is referred to as the uppermost aquifer. The identification of Layer IV as the uppermost aquifer is appropriate because the Layer II sand was historically excavated for sand mining purposes resulting in dewatering of this stratum. This dewatering frequently created dry monitoring wells. In addition, Layer II occurs only in the sidewalls of the facility whereas Layer IV underlies the entirety of the waste fill excavation. Layer IV has been unaltered by excavation activities, and is present beneath the entire excavation, and monitoring wells in this zone routinely have groundwater to be sampled.

Because the Layer II sand has been removed from the Layer II sand at the downgradient south and east sides of the site and replaced by reconstructed clay sidewalls and backfilled material, it would not be possible to obtain representative groundwater samples from Layer II as required by 30 TAC 330.403. Details of the constructed fill can be found in Section 5 of the LQCP (Part III, Attachment 3C and in Attachment 3A, Drawings 3-6 through 3-11).

Furthermore, because the Layer II sand is (or will be) substantially removed from this site and no Layer II sand remains at the downgradient east and south perimeter of the site, Layer IV is the uppermost aquifer at the site.

6.3.1 Layer II Groundwater Zone

As described in Section 4.4, site stratigraphy is divided into five geologic units: Layer I (surficial sand, silt, and clay), Layer II (sand), Layer III (clay), Layer IV (sand) and Layer V (clay). The uppermost groundwater zone at the site is the Layer II sand unit. Water levels measured in site monitoring wells and exploratory borings indicate that groundwater in the upper Layer II sand unit occurs under generally unconfined, water-table conditions and is confined or retarded at its lower limit by the underlying Layer III clay. The thickness of Layer II ranges from approximately 20 to 40 feet and has an average thickness of approximately 35 feet. Open excavations on the site and adjacent properties have been excavated for sand mining and waste filling operations. Over most of the existing site the Layer II sand has been removed. When the excavation for the proposed waste area is complete Layer II will have been removed over most of the site across both the existing site and the proposed expansion area (see Figure 4C-9, 4F-2, and 4F-3). As shown on these site cross sections, Layer II sand will remain in place between the previous waste fill units but will not exist on the south and east perimeters of the site. Groundwater levels in Layer II are affected by natural dewatering related to evaporation in the open excavation. Monitoring wells and piezometers near the open excavations are frequently dry. Figure 4F-1 is a potentiometric surface map constructed from water levels in site piezometers during the site characterization prior to site excavation. Figure 4F-2 shows a potentiometric surface constructed from groundwater monitoring wells and piezometers from a May 2012 water level reading event. Groundwater currently only exists in the limited areas where Layer II still exists. Figure 4F-3 depicts that Layer II sand will have been removed from much of the site when the expansion area is excavated. The Layer II sand has been removed from the east and south perimeter of the site. Natural groundwater flow direction in the Layer II sand is toward the southeast. Because that groundwater flow has been cut off by the excavations, clay liners, clayey fill , and clayey constructed fill groundwater no longer will flow beneath the site, but rather will be diverted around the site to the northeast and southwest parts of the site.

6.3.2 Layer III Confining Unit

The low permeability (predominantly CH clay) and continuity of Layer III enable this thick clay unit to function as the confining unit between the uppermost Layer II sand groundwater zone and the deeper Layer IV sand. The thickness of Layer III clay encountered at the site ranges from approximately 18 to 34 feet and averages approximately 27 feet. The proposed landfill bottom will be excavated in the Layer III clay.

Three Layer III clay piezometers, P-3A, P-E1, and P-5A, were installed in July and August 1997. Water levels measured in these piezometers indicate that groundwater in this clay unit is limited and that the permeability of the clay is low. Piezometer P-E1 in the northwest corner of the site was basically dry until October 1997, with less than one foot of water in the bottom of the well. Water level measurement in P-E1 is 57.80 feet above msl, indicating a very slow recovery rate of less than 0.02 feet per day. P-5A in the southwest corner of the site has had approximately two feet of water column since its installation in August 1997. Because the clay piezometer borings were completed using wet rotary drilling techniques, the small volumes of water observed in these wells are probably artifacts of the drilling fluid. The highest groundwater elevation in P-3A was observed on November 21, 1997, at 63.96 feet above msl. The recovery rate for P-3A is about 0.03 feet per day. The high groundwater elevation (63.96 feet above msl) in the Layer III clay is 35 feet lower than the highest water level in the overlying Layer II groundwater zone and 10 feet higher than the highest water level in the underlying Layer IV groundwater zone.

6.3.3 Layer IV Uppermost Aquifer

The maximum thickness explored was approximately 60 feet; the average thickness of the Layer IV sand is approximately 50 feet.

Three Layer IV sand piezometers, P-3B, P-E2, and P-5B, were installed in July and August 1997. Groundwater elevations measured since August 5, 1997, indicate that water levels are stable in these wells. An additional five Layer IV sand piezometers, P-6 through P-10, were installed in October 1997. Eight monitoring wells (MW-8 through MW-15) were installed in 1997. Two piezometers were installed in the expansion area in 2012 as part of this study. The highest measured water level elevation in Layer IV was observed on July 16, 2009 at 54.62 feet above msl in monitoring well MW-15. This elevation is approximately 10 feet lower than the highest measured elevation in the overlying Layer III clay and approximately 55 feet lower than the highest elevation in the uppermost Layer II groundwater zone, indicating that these three geologic units are not hydraulically connected. Groundwater in the Layer IV sand is confined. Layer IV groundwater elevations are listed in Table 4-6 and Table 4-7.

6.3.4 Layer V Lower Confining Unit

Layer V is a continuous, low permeability clay layer that functions as the lower confining unit to the overlying Layer IV uppermost aquifer. The maximum thickness explored was approximately 30 feet. Laboratory hydraulic conductivity tests on samples of the Layer V clay resulted in hydraulic conductivity values of 4.8×10^{-8} and 2.1×10^{-8} centimeters per second (cm/sec). Layer V is interpreted to be continuous across the site. Its composition and permeability is similar to the Layer III clay, making it an effective lower confining unit.

6.4 Groundwater Flow Direction and Rate

Shallow Layer II groundwater in the site area naturally flows to the southeast. Because extensive excavation activities have removed a large portion of the sands in the uppermost Layer II zone, shallow groundwater flow within the site boundary is toward open excavations. When the waste fill excavations are complete, nearly all of the Layer II sand will be removed and replaced with clay liners, clayey fill materials, and constructed clay sidewalls and thus no groundwater will be able to flow in this sand beneath the site. Layer II groundwater flowing from the upgradient northwest will be cut off and diverted around the site toward the northeast and southwest corners of the site and toward offsite areas.

Groundwater flow in the deeper Layer IV sand unit is to the northwest. Groundwater in Layer IV is confined at its upper limit by the overlying Layer III clay and at its lower limit by the underlying Layer V clay. Groundwater in the deeper Layer IV zone is not hydraulically connected to groundwater in the uppermost Layer II zone.

Travel times across the site were estimated using the formula:

$$v = (k * i) / n_e$$

Where: v = travel velocity
k = hydraulic conductivity of the aquifer
i = hydraulic gradient
n_e = effective porosity

Groundwater velocity for the deeper Layer IV uppermost aquifer was calculated using the geometric mean of the hydraulic conductivity values from slug tests in Layer IV piezometers (see Section 6.2), hydraulic gradients from the potentiometric surface maps on Figures 4F-4 and 4F-15 in Appendix 4F, and an effective porosity of 30 percent for fine sand. Groundwater flow velocity is estimated to flow at approximately *four feet per year* in Layer IV. All input values and calculation to determine groundwater velocity are shown on the groundwater velocity calculation sheet in Appendix F4, Figure 4F-16.

6.5 Arid Exemption

The applicant is not seeking an arid exemption for the landfill unit; therefore, 30 TAC §330.63(e)(6) is not applicable to this application.

ATTACHMENT 5

GROUNDWATER MONITORING PLAN

FAIRBANKS LANDFILL
HARRIS COUNTY, TEXAS
TCEQ PERMIT NO. MSW 1565B

PART III – SITE DEVELOPMENT PLAN
ATTACHMENT 5
GROUNDWATER MONITORING PLAN

Prepared for

USA WASTE OF TEXAS LANDFILLS, INC.
A WASTE MANAGEMENT COMPANY

August 2013

Revised March 2014

Revised May 2014

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5-2-2014

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Biggs & Mathews Environmental, Inc.
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GROUNDWATER MONITORING SYSTEM DESIGN CERTIFICATION

General Site Information

Site: Fairbanks Landfill

Site Location: Harris County, Texas

MSW Permit Application No.: 1565B

Qualified Groundwater Scientist Statement

I, Michael Snyder, am a licensed professional geoscientist in the State of Texas and a qualified groundwater scientist as defined in 30 TAC §330.3. I have reviewed the groundwater monitoring system and supporting data contained herein. In my professional opinion, the groundwater monitoring system is in compliance with the groundwater monitoring requirements specified in 30 TAC §330.401 through §330.421. This system has been designed for specific application to the Fairbanks Landfill (Permit Application No. MSW 1565B). The only warranty made by me in connection with this document is that I have used that degree of care and skill ordinarily exercised under similar conditions by reputable members of my profession, practicing in the same or similar locality. No other warranty, expressed or implied, is intended.

Firm/Address: Biggs and Mathews Environmental, Inc.
1700 Robert Road, Suite 100
Mansfield, Texas 76063

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5-2-2014

Signature: John Michael Snyder, P.G.
No. 595-Texas

Date:

Biggs & Mathews Environmental, Inc.
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1 SITE HYDROGEOLOGY

1.1 Hydrogeologic Units

Since the 1998 permit amendment application for the site, Layer II has been identified as the uppermost groundwater zone rather than the uppermost aquifer. Layer IV has been consistently identified as the uppermost aquifer in groundwater monitoring reports submitted to the TCEQ since that time and is referred to as the uppermost aquifer. The identification of Layer IV as the uppermost aquifer is appropriate because the Layer II sand was historically excavated for sand mining purposes resulting in dewatering of this stratum. This dewatering frequently created dry monitoring wells. In addition, Layer II occurs only in the sidewalls of the facility whereas Layer IV underlies the entirety of the waste fill excavation. Layer IV has been unaltered by excavation activities, and is present beneath the entire excavation, and monitoring wells in this zone routinely have groundwater to be sampled.

Because the Layer II sand has been removed from the Layer II sand at the downgradient south and east sides of the site and replaced by reconstructed clay sidewalls and backfilled material, it would not be possible to obtain representative groundwater samples from Layer II as required by 30 TAC 330.403. Details of the constructed fill can be found in Section 5 of the LQCP (Part III, Attachment 3C and in Attachment 3A, Drawings 3-6 through 3-11).

Furthermore, because the Layer II sand is (or will be) substantially removed from this site and no Layer II sand remains at the downgradient east and south perimeter of the site, Layer IV is the uppermost aquifer at the site.

1.1.1 Layer II Groundwater Zone

As described in Section 4.4, site stratigraphy is divided into five geologic units: Layer I (surficial sand, silt, and clay), Layer II (sand), Layer III (clay), Layer IV (sand) and Layer V (clay). The uppermost groundwater zone at the site is the Layer II sand unit. Water levels measured in site monitoring wells and exploratory borings indicate that groundwater in the upper Layer II sand unit occurs under generally unconfined, water-table conditions and is confined or retarded at its lower limit by the underlying Layer III clay. The original thickness of Layer II ranged from approximately 20 to 40 feet and had an average thickness of approximately 35 feet.

Open excavations on the site and adjacent properties have been excavated for sand mining and waste filling operations. Over most of the existing site the Layer II sand has been removed. When the excavation for the proposed waste area is complete Layer II will have been removed across much of the existing site and the proposed expansion area (see Attachment 4, Figure 4C-9, 4F-2, and 4F-3). Groundwater levels in Layer II are affected by natural dewatering related to evaporation in the open excavation.

Monitoring wells and piezometers near the open excavations are frequently dry. Figure 4F-1 is a potentiometric surface map constructed from water levels in site piezometers during the site characterization prior to site excavation. Figure 4F-2 shows a potentiometric surface constructed from groundwater monitoring wells and piezometers from a May 2012 water level reading event. Groundwater currently only flows in the limited areas where Layer II still exists. Because that groundwater flow has been cut off by the excavations, clay liners, clayey fill, and clayey constructed fill groundwater no longer will flow beneath the site, but rather will be diverted around the site to the northeast and southwest parts of the site. Figure 4F-3 depicts that Layer II will have been removed from the new site when the expansion area is excavated. No Layer II sand will exist at the downgradient east and south perimeter of the site.

1.1.2 Layer III Confining Unit

The low permeability (predominantly CH clay) and continuity of Layer III enable this thick clay unit to function as the confining unit between the uppermost Layer II sand groundwater zone and the deeper Layer IV sand. The thickness of Layer III clay encountered at the site ranges from approximately 18 to 34 feet and averages approximately 27 feet. The proposed landfill bottom will be excavated in the Layer III clay.

Three Layer III clay piezometers, P-3A, P-E1, and P-5A, were installed in July and August 1997. Water levels measured in these piezometers indicate that groundwater in this clay unit is limited and that the permeability of the clay is low. Piezometer P-E1 in the northwest corner of the site was basically dry until October 1997, with less than 1 foot of water in the bottom of the well. Water level measurement in P-E1 is 57.80 feet above msl, indicating a very slow recovery rate of less than 0.02 feet per day. P-5A in the southwest corner of the site has had approximately two feet of water column since its installation in August 1997. Because the clay piezometer borings were completed using wet rotary drilling techniques, the small volumes of water observed in these wells are probably artifacts of the drilling fluid. The highest groundwater elevation in P-3A was observed on November 21, 1997, at 63.96 feet above msl. The recovery rate for P-3A is about 0.03 feet per day. The high groundwater elevation (63.96 feet above msl) in the Layer III clay is 35 feet lower than the highest water level in the overlying Layer II groundwater zone and 10 feet higher than the highest water level in the underlying Layer IV groundwater zone.

1.1.3 Layer IV Uppermost Aquifer

Layer IV consists primarily of sand, silty sand, and gravel. It occurs below the Layer III sands and is the uppermost aquifer. The maximum thickness explored was approximately 60 feet; the average thickness of the Layer IV sand is approximately 50 feet.

Three Layer IV sand piezometers, P-3B, P-E2, and P-5B, were installed in July and August 1997. Groundwater elevations measured since August 5, 1997, indicate that water levels are stable in these wells. An additional five Layer IV sand piezometers, P-6 through P-10, were installed in October 1997. Eight monitoring wells (MW-8 through

MW-15) were installed in 1997. Two piezometers were installed in the expansion area in 2012 as part of this study. The highest measured water level elevation in Layer IV was observed on July 16, 2009 is 54.62 feet above msl in monitoring well MW-15. This elevation is approximately 10 feet lower than the highest measured elevation in the overlying Layer III clay and approximately 55 feet lower than the highest elevation in the uppermost Layer II groundwater zone, indicating that these three geologic units are not hydraulically connected. Groundwater in the Layer IV sand is confined. Layer IV groundwater elevations are listed in Table 4-6 and Table 4-7.

1.1.1 Layer V Lower Confining Unit

Layer V is a continuous, low permeability clay layer that functions as the lower confining unit to the overlying Layer IV groundwater zone. The maximum thickness explored was approximately 30 feet. Laboratory hydraulic conductivity tests on samples of the Layer V clay resulted in hydraulic conductivity values of 4.8×10^{-8} and 2.1×10^{-8} centimeters per second (cm/sec). Layer V is interpreted to be continuous across the site. Its composition and permeability is similar to the Layer III clay, making it an effective lower confining unit.

1.2 Groundwater Flow Direction and Rate

Shallow groundwater in the site area naturally flows to the southeast. Because extensive excavation activities have removed a large portion of the sands in the uppermost Layer II zone, shallow groundwater flow within the site boundary is toward open excavations. Once excavation of the Layer II sand at the south and east downgradient perimeter of the site is complete, lined areas will cut off flow within remaining Layer II. Layer II groundwater flowing from the upgradient northwest will be cut off and diverted around the site toward the northeast and southwest corners of the site and toward offsite areas.

Groundwater flow in the deeper Layer IV sand unit is to the northwest. Groundwater in Layer IV is confined at its upper limit by the overlying Layer III clay and at its lower limit by the underlying Layer V clay. Groundwater in the deeper Layer IV zone is not hydraulically connected to groundwater in the uppermost Layer II zone or the deeper Chicot Aquifer sand unit.

Groundwater velocity for the deeper Layer IV uppermost aquifer was calculated using the geometric mean of the hydraulic conductivity values from slug tests in Layer IV piezometers (see Attachment 4, Section 6.2), hydraulic gradients from the potentiometric surface maps on Figures 4F-4 and 4F-15 in Appendix 4F, and an effective porosity of 30 percent for fine sand. Calculations indicate that groundwater in Layer IV moves approximately four feet per year (see Figure 4F-16).

ATTACHMENT 6

LANDFILL GAS MANAGEMENT PLAN

Prepared for:
USA Waste of Texas Landfills, Inc.

PERMIT AMENDMENT APPLICATION
PART III – SITE DEVELOPMENT PLAN
ATTACHMENT 6

LANDFILL GAS MANAGEMENT PLAN

FAIRBANKS LANDFILL
MSW PERMIT NO. 1565B
HOUSTON, HARRIS COUNTY, TEXAS

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Submitted August 2013
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4. RECORDKEEPING AND REPORTING

4.1 Gas Monitoring Probe Installation Report

A gas monitoring probe installation report shall be prepared upon completion of each gas probe installation project and submitted to TCEQ. The installation report will include the following:

- A figure showing the site plan and gas monitoring probe locations/designations (e.g., copy of Drawing 6-1 of this plan, or similar figure).
- Boring logs for each new gas probe installed, including the drilling date and method, name(s) of the engineer or geologist who logged the hole, and information on the subsurface findings (soil types and depths, groundwater depth, if present, etc.).
- Construction summary logs for each new installed gas probe, providing the surveyed location coordinates of the probe, surveyed elevation of existing ground and top of probe riser casing, and identification of the probe materials, dimensions and depths/elevations, screen type and interval length, extent and types of filter pack, extent and types of annular seal, material and extent of backfill, presence of concrete pad, protective bollards, etc.

As previously discussed, GP-1 through GP-10, are existing gas monitoring probes. Installation information for these existing gas probes is presented in Appendix 6-A of this plan.

When additional gas monitoring probes are installed, their installation records will be submitted to TCEQ as mentioned above, and the records may be added to Appendix 6-A of this plan.

4.2 Quarterly Gas Monitoring Records

Quarterly monitoring records for the gas probes and facility structures will be maintained ~~at the landfill~~ in the facility's Site Operating Record throughout the active life of the facility and during the post-closure period. The monitoring records will be recorded on data sheets similar to the one attached to this document (Appendix 6-B). The exact format of the monitoring form may be modified from the example attached to this document, but the data recorded during each monitoring event will at a minimum include the information identified in Section 3.5 of this plan.

In the event that the maximum allowable landfill gas concentrations set forth in Section 3.1 of this plan are exceeded, the facility must report the results to TCEQ and take other steps required by 30 TAC §330.371(c)(1) through (3), and as described subsequently in Section 5 of this plan.

c. If the initial detection is verified to be an exceedance, the following parties shall be notified of the situation:
   - ~~the TCEQ Executive Director;~~
   - TCEQ Region 12;
   - the local Fire Department and Harris County Public Health and Environmental Services; and
   - neighboring landowners within 500-ft of the exceedance location.

2. Within Seven Days of Verified Exceedance. A record of the methane gas levels detected and a description of the immediate actions taken to protect human health will be placed in the Site Operating Record.

3. Within 60 Days of Verified Exceedance.
   a. A detailed evaluation will be made to determine the potential source and extent of the methane gas migration. A Remediation Plan will be prepared and must be submitted to the TCEQ Executive Director. The Remediation Plan will present the results of the detailed evaluation, along with the remedial measures taken, which may include additional monitoring, source control t (e.g., installation of gas vent(s)) a passive interceptor trench/barrier system, active building ventilation systems,), etc.
   b. The Remediation Plan will incorporate remediation performance monitoring. The remediation performance monitoring will be conducted on a monthly basis at the affected gas monitoring location(s) and will submitted to TCEQ, until methane concentrations in the affected gas monitoring location(s) are below the allowable limits specified at the beginning of this section for six (6) consecutive months.

As allowed by 30 TAC §330.371(d), alternate schedules to those given above may be established by the TCEQ Executive Director.

ATTACHMENT 7

CLOSURE PLAN

Prepared for:
USA Waste of Texas Landfills, Inc.

PERMIT AMENDMENT APPLICATION

PART III – SITE DEVELOPMENT PLAN
ATTACHMENT 7

CLOSURE PLAN

FAIRBANKS LANDFILL
PERMIT NO. MSW-1565B
HOUSTON, HARRIS COUNTY, TEXAS

Prepared by:

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3. DESCRIPTION OF CLOSURE DESIGN AND CLOSURE SEQUENCE

3.1 Introduction

This section describes the design and installation requirements for the landfill final cover system, and discusses the closure sequence.

3.2 Final Cover System Design

The final cover system is designed to meet the requirements of 30 TAC §330.453(a), (b), and (c).

3.2.1 Cross Section and Areas of Installed Final Cover

The proposed final cover system for the facility is shown on an engineering detail on Drawing 3-11 in Attachment 3A (Landfill Design Drawings) of this Permit Amendment Application, and is described as follows (from bottom to top):

- 1.5-ft thick compacted soil layer composed of clayey soil, classified by the Unified Soil Classification System (USCS) as "SC" (sandy clay), "CL" (lean clay), or "CH" (fat clay) and having a coefficient of permeability (i.e., a hydraulic conductivity) no greater than 1×10^{-5} cm/sec (i.e., $k \leq 1 \times 10^{-5}$cm/sec); and
- a 6-inch or 12-inch thick topsoil layer[1] capable of sustaining native plant growth and seeded or sodded immediately after installation.

[1]If the underlying compacted soil layer is classified as SC or CL, the minimum topsoil thickness is 6 inches. If the underlying compacted soil layer is classified as CH, the minimum topsoil thickness is 12 inches.

The material requirements specified for the final cover system are included in the Final Cover Quality Control Plan (FCQCP) provided in Attachment 7B to this Plan. Soils with USCS classifications other than those listed above may be used in the final cover system with prior written approval from the TCEQ Executive Director.

As mentioned in Section 2.2, it is anticipated that the largest area of the landfill that could potentially be open and require final cover is 52.2 acres, as shown on Drawing 7-1 in Attachment 7A of this Plan. Drawing 7-1 also shows that as of March 2014, 30.6 acres have been final capped; and a note on the drawing references the Final Cover System Evaluation Reports (FCSERs) for the two capping events to-date. These 30.6 acres were final capped with the same final cover system as listed above.

Prepared for:
USA Waste of Texas Landfills, Inc.

PERMIT AMENDMENT APPLICATION

PART III – SITE DEVELOPMENT PLAN
ATTACHMENT 7B

FINAL COVER QUALITY CONTROL PLAN

FAIRBANKS LANDFILL
PERMIT NO. MSW-1565B
HOUSTON, HARRIS COUNTY, TEXAS

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TABLE 7B-3
FIELD TESTING AND ONGOING CONFORMANCE TESTING REQUIREMENTS FOR COMPACTED SOIL LAYER

TEST	METHOD	MINIMUM FREQUENCY OF TESTING	PASSING CRITERIA
In-Place Density and In-Place Moisture Content (Nuclear Gauge)	ASTM D 6938	1 per acre per lift	See Section 2.2.2
Particle Size Analysis	ASTM D 422	Sample: 1 per acre through full layer Test: 1 per 3 acres per lift[2]	≤ 1.5 in. max particle size
Atterberg Limits	ASTM D 4318	Sample: 1 per acre through full layer Test: 1 per 3 acres per lift[2]	None
Soil Classification	ASTM D 2487	Sample: 1 per acre through full layer Test: 1 per 3 acres per lift[2]	SC, CL, or CH
Undisturbed Hydraulic Conductivity	ASTM D 5084[1]	Sample: 1 per acre through full layer Test: 1 per 3 acres per lift[2]	$\leq 1 \times 10^{-5}$ cm/s
Layer Thickness Verification	See Section 2.3.7~~Note 2~~	1 per acre~~[2]~~	≥ 1.5 ft thick

Note:

(1) Undisturbed hydraulic conductivity tests shall ~~be obtained using thin-walled push tube sampler (e.g., Shelby tube), and shall~~ be tested using tap water or a 0.05N solution of $CaSO_4$. Use effective stress of 5 psi. Distilled or deionized water shall not be used. The permeant should be deaired. All hydraulic conductivity test data shall be submitted with the FCSER.

(2) The suggested sampling and testing method of the constructed compacted soil is as follows:

- Soil samples for particle size analysis, Atterberg limits, classification, and undisturbed hydraulic conductivity tests shall be obtained using thin-walled push tube sampler (e.g., Shelby tube) ~~testing will be collected by pushing a 3-inch (nominal) diameter thin-walled tube sampler~~ advanced into the constructed materials using a drill rig hydraulic system, a bulldozer, an excavator, or other appropriate equipment.
- ~~The location and the elevation of the sample locations will be determined using surveying methods. The~~ ~~S~~Samples will ~~then~~ be sealed and placed in protective core boxes or similar containers for transport to the independent soils laboratory for testing.
- After obtaining survey/layer thickness information, ~~T~~the sample location boreholes will be backfilled with wetted bentonite or a cement-bentonite grout tremied into the borehole from the bottom to the top of the compacted soil layer.
- The specified ~~laboratory soil testing~~sampling will be performed at a frequency of one ~~one set of tests~~full layer sample per acre, and the tests specimens will be evenly distributed through the lifts, such that the minimum testing frequency indicated above is met on a per lift basis.

An alternate sampling method may be used as approved by the POR, provided that undisturbed Shelby tube samples are obtained for hydraulic conductivity testing, that the above-specified minimum testing frequency is met on a per lift basis, and that the other related requirements of this FCQCP are met. ~~The tube sample~~

~~specimen obtained (or the borehole from which the tube was taken) will be used to measure the thickness of the compacted soil layer to determine if the minimum layer thickness has been provided.~~

Prepared for:

USA Waste of Texas Landfills, Inc.

PERMIT AMENDMENT APPLICATION

PART IV – SITE OPERATING PLAN (SOP)

FAIRBANKS LANDFILL
MSW PERMIT NO. 1565B
HOUSTON, HARRIS COUNTY, TEXAS

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SEALED FOR THIS PART IV SITE OPERATING PLAN, AND FOR PERMITTING PURPOSES ONLY.

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SITE OPERATING PLAN (SOP)

1. INTRODUCTION

1.1 Terms of Reference

The Fairbanks Landfill (hereafter referred to as the "facility" or "site") is a Type IV municipal solid waste (MSW) facility, owned and operated by USA Waste of Texas Landfills, Inc. This Site Operating Plan (SOP) provides general instructions for site management and personnel to operate the facility in a manner consistent with the design of the facility and with the Texas Commission on Environmental Quality's (TCEQ's) rules to protect human health and the environment. This SOP complies with the requirements of 30 TAC Chapter 330 Subchapter D of the TCEQ Municipal Solid Waste Management Regulations (MSWMR) "Operational Standards for Solid Waste Land Disposal Sites" for Type IV landfills.

The specific procedures outlined in this SOP are operational requirements and must be understood, acknowledged, and followed by the site personnel. This SOP will be maintained ~~at the site~~ as part of the Site Operating Record in an easily accessible location to allow the site operating personnel to review the SOP as needed. This SOP will be retained during the active life of the site and throughout the site's post-closure care maintenance period.

References to the terms "Executive Director" or "TCEQ" used in this SOP shall refer to the Executive Director of the TCEQ or the designated representative of the TCEQ. References to information in the permit or "permit application" for this facility shall refer to the most current version of these documents, including any amendments, modifications, or revisions as approved.

The Site Manager has overall responsibility for implementation and adherence to this SOP. Wherever this SOP describes procedures or requirements without naming a specific individual or position responsible for those requirements, the Site Manager shall have primary responsibility for those requirements. Where a specific position is responsible for a particular task, that responsibility is described. Otherwise, the Site Manager may assign any qualified personnel to accomplish the requirements of this SOP.

1.2 Facilities Addressed by this SOP

As mentioned in Section 1.1, this SOP has been prepared to address 30 TAC Chapter 330 Subchapter D for Type IV landfills. Disposal of waste in the landfill is the primary site activity. Additionally, the following recycling areas will be established on-site: (i) a staging area to collect large/heavy/bulky items (e.g., appliances) for recycling or salvaging; (ii) a wood recycling area; and (iii) a construction and demolition (C&D) waste recycling area. This SOP

During dry weather, the operator will control dust by watering site roads using the water truck and/or sweeping the roads. The on-site water source that can be used for this purpose are the surface water ponds.

As mentioned in Section 11 of this SOP, litter and other debris on site roads will be picked up at least once per day each day that the facility is operating and disposed of properly.

18.3 Road Maintenance Frequencies

Internal roads will be inspected at least once every two months for the presence of ruts, soft spots, potholes and drainage to determine the need for regrading. The frequency of road regrading will be dependent on the results of inspections and whether ruts, potholes, or soft spots of sufficient severity are detected. However, at a minimum, road regrading will occur once per year. As directed by the Site Manager or designated alternate, wet weather operations may require more frequent regrading to properly maintain the roads. Roadside ditches or culverts will be maintained as necessary to provide drainage. The on-site fleet of equipment, such as the on-site motor grader, broom, backhoe excavator, and dozers, may be used to provide maintenance, as appropriate.

Road inspections and maintenance/repair activities will be documented by the Site Manager or designated alternate and placed in the Site Operating Record. Minimum information will include date of inspection and/or repairs, name of employee performing work, and the relevant findings/actions.