Protecting, Enhancing, and Restoring Our Environment

May 16, 2018

Ms. Cathy Stepp
Regional Administrator
EPA Region V
77 West Jackson Blvd.
Chicago, IL 60604

Mr. Jack Schinderle
Director, Waste Management and Radiological Protection Division
Michigan Department of Environmental Quality
525 West Allegan Street
Lansing, MI 48933

Subject: **Proposed Permit Modification - Upgrades to MC VI-G Phase 2 Liner Design Revision 1**
Wayne Disposal, Inc.
Belleville, Wayne County, Michigan

Dear Ms. Stepp and Mr. Schinderle:

On behalf of Wayne Disposal, Inc. (WDI), CTI and Associates, Inc. (CTI) is submitting this Revision 1 to the May 3, 2018 Permit Modification Letter Report for your review and approval. The May 3, 2018 letter report details proposed upgrades to the design of the Master Cell VI-G Phase 2 (MC VI-G Phase 2) liner. The purpose of this Revision 1 is to respond to comments WDI has received from the Environmental Protection Agency (EPA) and the Michigan Department of Environmental Quality.

WDI and CTI received comments as follows: Comments from the MDEQ dated May 3, 2018, Comments from the MDEQ dated May 9, 2018, and Comments from the EPA dated May 14, 2018. These comments and responses are included herein as Attachment C, Correspondence Regarding the WDI 2018 Permit Modification, Revision 1. This revised Attachment C replaces the original Attachment C included with the May 3, 2018 Permit Modification Letter Report.

Responses to the comments also resulted in changes to the original Attachments A and B included with the May 3, 2018 Permit Modification Letter Report. Therefore, this Revision 1 also includes Attachment A, Equivalency Information and References, Revision 1 and Attachment B, 2018 Permit Engineering Drawings, Revision D (revising Sheets 22A and 22B). These revised attachments supersede the original Attachments A and B included in the May 3, 2018 Permit Modification Letter Report.

If you have any questions regarding the revisions to the May 3, 2018 submittal, please feel free to contact the undersigned at (248) 486-5100 or tsoong@cticompanies.com.

Sincerely,

CTI and Associates, Inc.

Te-Yang Soong, Ph.D., P.E.
Principal Engineer

Cc: Kerry Durnen, US Ecology
Sylwia Scott, US Ecology
Pete Quackenbush, MDEQ
Lisa Graczyk, EPA

List of Attachments

Proposed Permit Modification Letter Report, May 3, 2018

Attachment A: Equivalency Information and References, Revision 1, May 16, 2018

Attachment B: 2018 Permit Engineering Drawings (under a separate cover), Revision D

Attachment C: Correspondence Regarding the WDI 2018 Permit Modification, Revision 1, May 16, 2018

Attachment D: GCL Manufacturer Specifications, CQA Manual, and Installation Guidelines

May 3, 2018

Ms. Cathy Stepp
Regional Administrator
EPA Region V
77 West Jackson Blvd.
Chicago, IL 60604

Mr. Jack Schinderle
Director, Waste Management and Radiological Protection Division
Michigan Department of Environmental Quality
525 West Allegan Street
Lansing, MI 48933

Subject: **Proposed Permit Modification - Upgrades to MC VI-G Phase 2 Liner Design**
Wayne Disposal, Inc.
Belleville, Wayne County, Michigan

Dear Ms. Stepp and Mr. Schinderle:

On behalf of Wayne Disposal, Inc. (WDI), CTI and Associates, Inc. (CTI) is submitting this Permit Modification Letter Report for your review and approval of proposed upgrades to the design of the Master Cell VI-G Phase 2 (MC VI-G Phase 2) liner. The purpose of this change is to incorporate the numerous advantages of Geosynthetic Clay Liner (GCL).

The following sections of this letter report summarize the analysis methodology, results, and recommendations for the upgrades. Calculations and documents supporting the proposed upgrades and the revised permit engineering drawings are attached.

Introduction

This letter report presents the basis for the proposed liner revisions for MC VI-G Phase 2 at WDI. The proposed upgrades incorporate an alternative GCL-based liner design providing the following benefits compared to the currently approved compacted clay liner (CCL) based design:

- GCL is man-made with superior consistency and reliability
- GCL has superior resistance to freeze-thaw damage and is preferred considering Michigan's climate
- GCL has superior resistance to settlement–induced tensioning
- GCL reduces the need for compaction and is more consistent in achieving the approved grades
- GCL has substantially lower hydraulic conductivity

Although it is WDI's intent to incorporate GCLs in future construction of MC VI-G Phases 3 through 6 and F subcells, this proposed design upgrade pertains only to the construction of MC VI-G Phase 2 subcells to facilitate a prompt and timely review and approval in support of the planned 2018 MC VI-G Phase 2 Subcell G2 construction. **Figure 1** shows a site plan of WDI's Master Cell VI G and F (approved by the MDEQ on May 4, 2012 and EPA on September 27, 2013). The proposed liner system upgrade presented in this letter report pertains to MC VI-G Phase 2 (consisting of Subcells G2 and G3) and is highlighted in **Figure 1**.

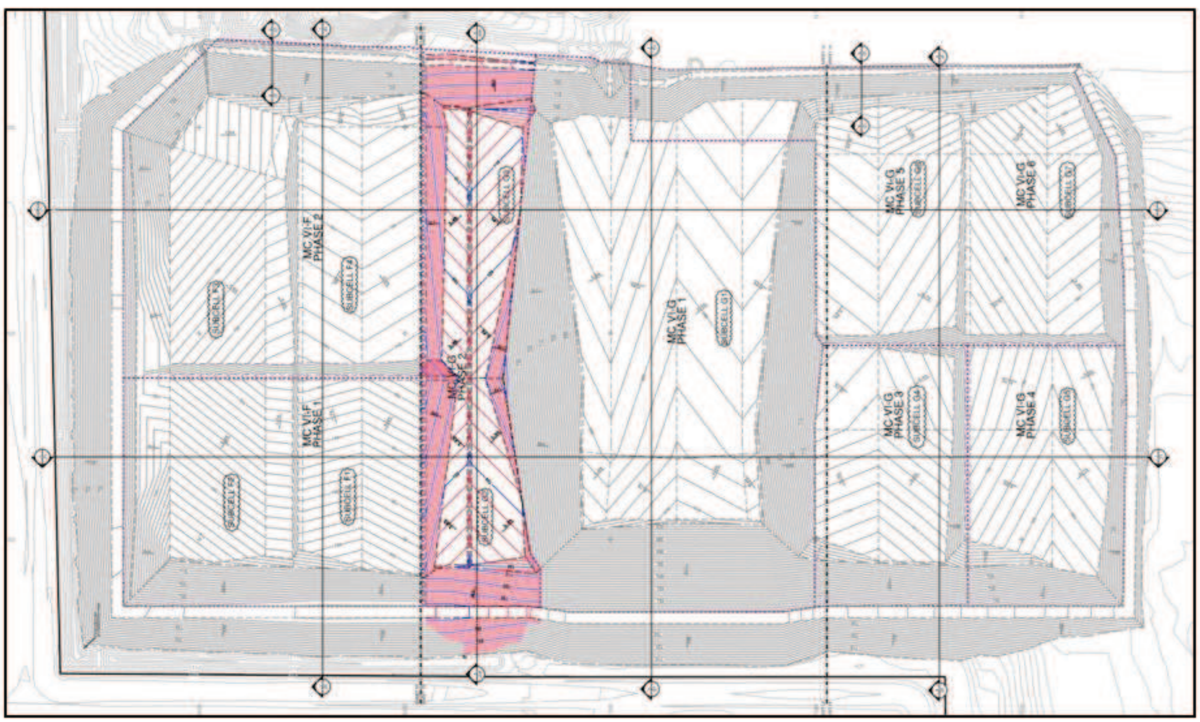


Figure 1. Master Cell VI-G and F Layout

In accordance with Rule 299.9620 (4) of the Michigan Part 111 Administrative Rules, an alternate design may be approved if the owner or operator can demonstrate the design will prevent the migration of any hazardous constituent into the groundwater or surface water at least as effectively as the design requirements specified in the subrule. The following sections discuss how the proposed design satisfies this requirement.

Proposed Liner System

This modification proposes using GCL, in lieu of the currently approved CCL, as an alternative soil component of the liner system for the future construction of Master Cell VI-G Phase 2 subcells. GCL products are factory-manufactured hydraulic barriers consisting of a layer of sodium bentonite supported by geotextiles (woven and/or non-woven) and, in some cases, an additional film of flexible membrane liner (FML) for enhanced barrier performance. These components (sodium bentonite, geotextiles, and FML) are mechanically held together by either needling or chemical adhesive.

Sodium bentonite (the interlayer of GCL) is an effective barrier primarily because it can absorb moisture (i.e., hydrate and swell) producing a dense, uniform layer with extremely low hydraulic conductivity (on the order of 10^{-9} cm/sec). Sodium bentonite's exceptional hydraulic properties make GCL superior to CCL with respect to a steady state of water even though the thickness of GCL is less than CCL.

WDI is proposing to install two layers of GCL (as described in **Attachment A**) immediately beneath the primary HDPE geomembrane liner of MC VI-G Phase 2 subcells. **Figure 2** below shows the proposed liner construction details. Note that the captions of some of the other liner components (e.g., 80-mil HDPE geomembranes, double-sided geocomposite, geogrid, etc.) are omitted in **Figure 2** for clarity and because those components of the liner system are not changing. Please refer to **Attachment B**, 2018 Permit Engineering Drawings, Sheet 22A, for complete liner construction details.

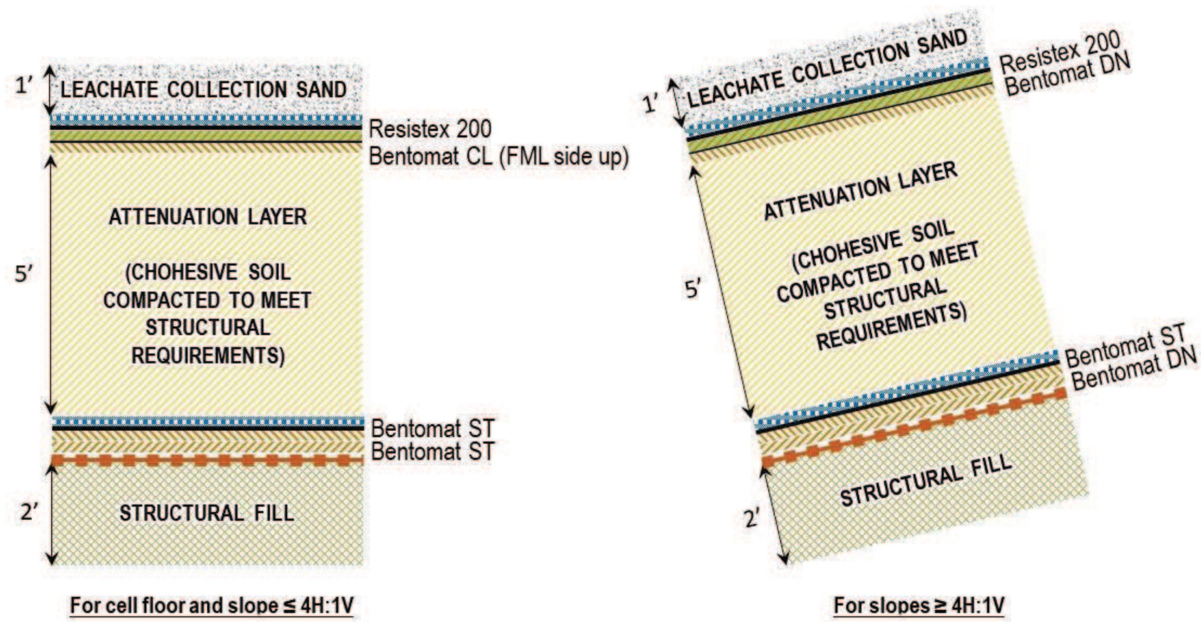


Figure 2. Proposed Liner System in MC VI-G Phase 2

As shown in **Figure 2**, the proposed liner system consists of multiple layers of geosynthetic and earthen materials to optimize the performance of the base liner system. These layers, along with their respective functions, are tabulated in **Table 1** for a direct comparison between the proposed and the permitted base liner systems (in the order from top to bottom).

Table 1. Comparison Between Permitted and Proposed Liner Systems (cell floor from top to bottom)

Component	Permitted System	Proposed System
Primary leachate collection	1' of drainage sand	
	Double-sided drainage geocomposite	
Primary geomembrane liner	80-mil textured HDPE geomembrane	
Primary clay liner	5-ft CCL ($K \leq 1.0 \times 10^{-7}$ cm/s)	Resistex® 200, manufactured by CETCO
		Bentomat® CL, manufactured by CETCO
		5-ft cohesive soil attenuation layer
Secondary leachate collection	Double-sided drainage geocomposite	
Secondary geomembrane liner	80-mil textured HDPE geomembrane	
Secondary clay liner	3-ft CCL ($K \leq 1.0 \times 10^{-7}$ cm/s)	Bentomat® ST, manufactured by CETCO
		Bentomat® ST, manufactured by CETCO
Base reinforcement	Bi-axial geogrid	
Liner subbase	2-ft structural fill	

As indicated in **Table 1**, the main difference between the permitted and the proposed liner systems are the use of GCLs in lieu of CCLs. Other liner components will remain unchanged. Additionally, the only difference between the cell floor and sideslope (slope ≥ 4(H):1(V)) liners is the second GCL layer in the primary liner system (Bentomat® CL) will be replaced with a standard CETCO GCL product (Bentomat® DN) to maximize slope stability. Similarly, the second GCL layer in the secondary liner system (Bentomat® ST) will be replaced with a standard CETCO GCL product (Bentomat® DN) to maximize slope stability. Details of the GCL products proposed to be used in the construction of MC VI-G Phase 2 subcells can be found in **Attachment D** of this report.

Equivalency Demonstration

Federal and Michigan regulations allow alternative liner designs provided "equivalence" can be demonstrated. For this report, the assessment was conducted by the following steps allowing for a technically-sound, effective and project-focused equivalency demonstration.

1. Identify various technical criterion that are relevant to the proposed MC VI-G Phase 2 base liners.
2. Divide the identified criteria into distinct categories to facilitate a direct technical comparison between GCLs (the proposed alternative) and CCLs (the approved design).
3. Identify criteria where technical equivalency between GCLs and CCLs has already been well-studied, demonstrated and documented by the lining industry (e.g., landfills, surface impoundments, mining, water-proofing of hydraulic structures, etc.) and based on past tests and project experiences, to be superior or equivalent to CCL. No additional demonstration effort is needed for these items.
4. Identify criteria which are mainly site-, project-, or product-specific items, and demonstrate equivalency.

As shown in **Table 2**, the following five items are identified and subjected to detailed comparison.

Hydraulic Properties

- Steady state solute flux
- Chemical adsorptive capacity / Solute breakthrough time

Physical/Mechanical Properties

- Stability of slopes
- Bearing capacity

Construction Properties

- Puncture resistance/subgrade condition

Table 2. Generalized Technical Equivalency Assessment for Liners Beneath Landfills

Category	Criterion for Evaluation	Equivalency of GCL to CCL			
		GCL is superior	GCL is equivalent	Equivalency is product-, design-, or site-specific	Category irrelevant to this project
Hydraulic	Steady state water flux	X			Evaluation will focus on site-specific leachate
	Breakthrough time - water	X			Evaluation will focus on site-specific leachate
	Horizontal flow in seams or lifts		X		-
	Horizontal flow beneath geomembranes	X			-
	Steady state solute flux			X	-
	Chemical adsorptive capacity / Solute breakthrough time			X	-
	Permeability to gases	-	-	-	A non-issue when GCL is installed under FML
	Generation of consolidation water	X			-
Physical/ Mechanical	Freeze-thaw behavior	X			-
	Wet-dry behavior	X			-
	Vulnerability to erosion	-	-	-	Erosion is irrelevant in the proposed liner
	Total settlement		X		-
	Differential settlement	X			-
	Stability on slopes			X	-
	Bearing capacity			X	-
Construction	Puncture resistance			X	-
	Ease of placement	X			-
	Speed of construction	X			-
	Availability of material	X			-
	Requirements of water	X			-
	Air pollution concerns	X			-
	Quality assurance considerations		X		-

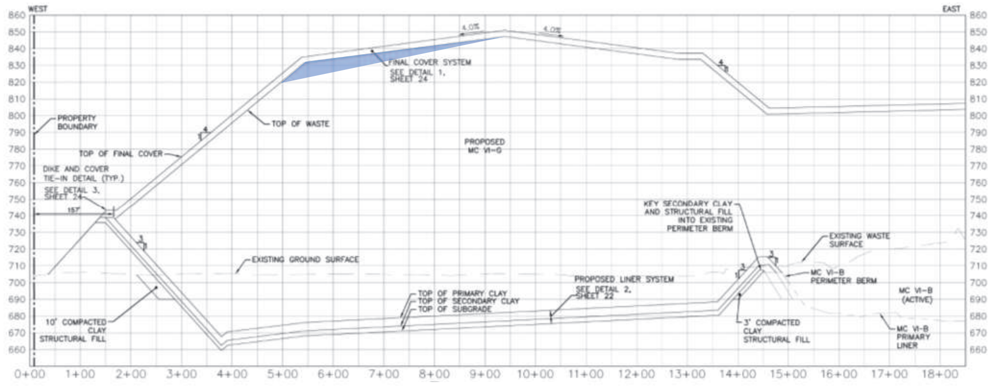
Category of which GCL is superior than CCL	Category of which equivalency is product-, design-, or site-specific
Category of which GCL is equivalent to CCL	Category is irrelevant to this project

WDI successfully demonstrates that the proposed GCL liner system is technically equivalent to the permitted CCL liner system in these criteria in **Attachment A**. Therefore, the proposed GCL liner system will minimize the risk of migration of hazardous constituents into the groundwater or surface water at least as effectively as the CCL design requirements specified in the rule.

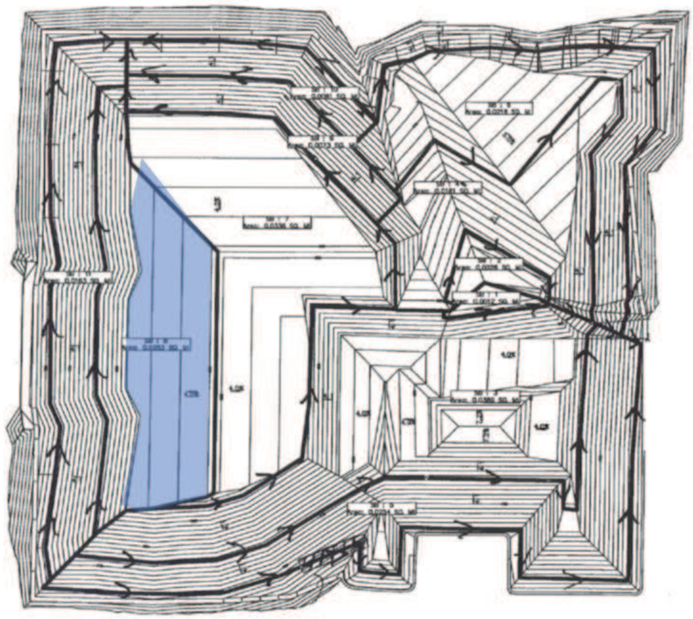
Airspace Balance

The proposed change in liner design, as a result of replacing the 3-ft CCL in the secondary liner with two layers of GCLs, would result in a potential increase of landfill volume of 27,240 cubic yards. To off-set this gain of airspace, the top of waste grading along the western limit of MC VI-G and F were "truncated" to ensure the proposed revision will not expand the landfill volume. The proposed new top of waste grading results in a decrease in landfill volume of 27,361 cubic yards for a net landfill volume loss of 121 cubic yards.

The proposed revisions will not impact the design and performance of the final cover and stormwater management systems. **Figure 3(a)** illustrates the concept of "truncating" the top of waste grade to off-set the volume gained from replacing the 3-ft CCL in the secondary liner with 2 layers of GCL. **Figure 3(b)** illustrates the approximate extent of revisions. Both revisions are highlighted in blue.



(a) E-W Cross Section of MC VI-G Phase 2 – Illustration of Top of Waste Revision



(b) Final Grading of WDI Illustrating the Approximate Extent of the Top of Waste Revision

Figure 3. Modification of Waste Grading to Off-set the Gain in Airspace Due to the Proposed Revision

Permit Drawings

The proposed upgrades to the MC VI-G Phase 2 base liner system will result in some revisions to the permit drawing sheets listed in **Table 3**. A complete set of permit drawings, including both revised and unrevised sheets, is included in **Attachment B** for ease of review and reference.

Table 3. List of Revised Permit Drawings

Sheet	Title
1	Title sheet
5	Construction phasing plan
9	Top of secondary liner grading plan (1 of 3)
10	Top of secondary liner grading plan (2 of 3)
12	Top of primary liner grading plan (1 of 3)
13	Top of primary liner grading plan (2 of 3)
16	Final cover grading plan (1 of 2)
17	Final cover grading plan (2 of 2)
20	Cross section (1 of 3)
20A	Cross section (2 of 3)
21	Cross section (3 of 3)
22A	Liner system details for G2 and G3
22B	Liner system details for G2 and G3
32	Conceptual Gas Venting System

MDEQ/EPA Correspondence

While preparing this 2018 WDI permit modification, discussions regarding this letter report took place between the U.S. EPA, MDEQ, WDI, and CTI. To aid in referencing this correspondence, a list of questions and responses is included in **Attachment C**. The table in **Attachment C** also includes references to the location in this letter report where further information regarding the item discussed can be found.

GCL Manufacturer Specifications, CQA Manual, and Installation Guidelines

The proposed base liner in MC VI-G Phase 2 includes manufacturer and product specific GCL components as shown in **Figure 2** above. These GCL components were selected based on the equivalency demonstration provided in Attachment A. Manufacturer specifications for the GCL products selected for use in the MC VI-G Phase 2 base liner are included in **Attachment D**.

In order to maximize the safety, efficiency, and physical integrity of the selected GCL, the manufacturer's CQA Manual and Installation Guidelines (**Attachment D**) will supersede the GCL section of the existing CQA Plan for the base liner of MC VI-G Phase 2.

If you have any questions regarding this submittal, please feel free to contact the undersigned at (248) 486-5100 or tsoong@cticompanies.com.

Sincerely,

CTI and Associates, Inc.

Te-Yang Soong, Ph.D., P.E.
Principal Engineer

Cc: Kerry Durnen, US Ecology
Sylwia Scott, US Ecology
Pete Quackenbush, MDEQ
Lisa Graczyk, EPA

List of Attachments

Attachment A: Equivalency Information and References

Attachment B: 2018 Permit Engineering Drawings (under a separate cover)

Attachment C: Correspondence Regarding the WDI 2018 Permit Modification

Attachment D: GCL Manufacturer Specifications, CQA Manual, and Installation Guidelines

Attachment A: Equivalency Information and References

Revision 1, May 18, 2018

Proposed Liner System for MC VI-G Phase 2

WDI is proposing to install a polymer-treated GCL (Resistex® 200, manufactured by CETCO) immediately beneath the primary 80-mil HDPE geomembrane liner of MC VI-G Phase 2 to maximize the barrier performance of the liner system. **Figure A-1** shows the proposed liner construction details. Note that the captions of other liner components (e.g., 80-mil HDPE geomembranes, double-sided geocomposite, geogrid, etc.) are omitted in **Figure A-1** for clarity. Please refer to **Attachment B**, 2018 Permit Engineering Drawings, Sheet 22A, for more liner construction details.

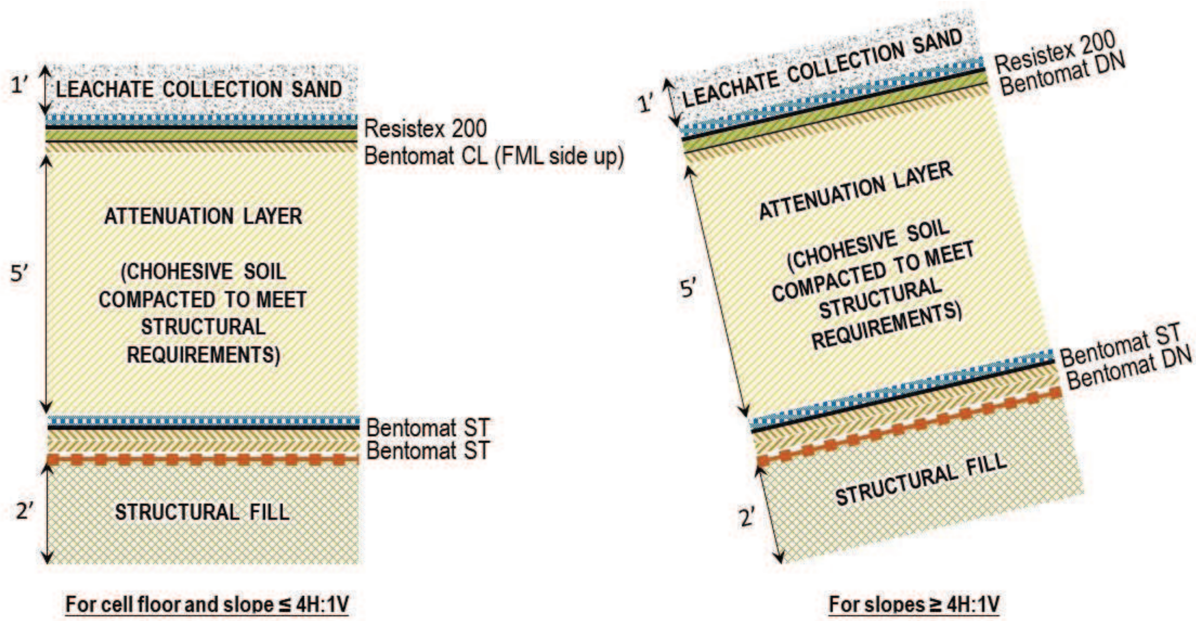


Figure A-1. Proposed MC VI-G Phase 2 Base Liner Construction Detail.

To quantify the equivalency of the proposed liner system including GCL to the permitted liner system including CCL, WDI has provided the GCL manufacturer (CETCO) with site-specific leachate test data for a conservative evaluation of GCL chemical compatibility. CETCO conducted a series of tests in their R&D laboratory on the supplied sample of leachate from WDI.

After 243 hours of permeation, CETCO has measured an average permeability of 1.5×10^{-9} cm/sec with 0.7 pore volumes of leachate passing through the specimen. This means that the bentonite / polymer blend in the Resistex® 200 is hydrating and cutting off flow as designed. For the equivalency demonstration calculations (specifically, the steady-state solute flux) to be presented later, a conservative permeability of 1×10^{-8} cm/sec was used. In other words, an extra adjustment or safety factor of 6.7 was applied for additional conservatism. See **Appendix A-1** for CETCO's chemical evaluation report.

In addition to installing the polymer-treated GCL (Resistex® 200) immediately beneath the primary 80-mil HDPE geomembrane liner on the cell floor, WDI is also proposing to use another specialty GCL, Bentomat® CL, for enhanced protection. Bentomat® CL has an additional FML laminated on one side of the GCL to offer the highest level of hydraulic barrier performance. By installing this product with the FML side "facing up" towards the cell as indicated in **Figure A-1**, Bentomat® CL provides another impervious layer to isolate its own bentonite layer from contacting moisture, if any, that may migrate through the primary HDPE geomembrane liner and the overlain GCL (Resistex® 200).

For sideslopes that are steeper than 4(H):1(V), WDI proposes to replace the FML-laminated GCL (Bentomat® CL) with a standard GCL product (Bentomat® DN) for slope stability purposes. Bentomat® DN consists of two layers of needle-punched, non-woven geotextiles on both sides of the bentonite interlayer. This configuration provides superior sideslope shear resistance. The FML-laminated GCL (Bentomat® DN) to be installed on the cell floor will be extended 5-ft vertically above the toe of the sideslopes for optimized performance.

Technical Equivalency

An equivalency assessment was conducted by the following steps allowing for a technically-sound, effective and project-focused equivalency demonstration.

1. Identify various technical criterion that are relevant to the proposed MC VI-G Phase 2 cell liners.
2. Divide the identified criterion into distinct categories to facilitate a direct technical comparison between GCLs (the proposed alternative) and CCLs (the approved design).
3. Identify criterion where technical equivalency between GCLs and CCLs has already been well-studied, demonstrated and documented by the lining industry (e.g., landfills, surface impoundments, mining, water-proofing of hydraulic structures, etc.), based on past tests and project experiences. No additional demonstration effort is needed for these items.
4. Identify criteria which are mainly site-, project-, or product-specific items, and demonstrate equivalency.

The results of Steps 1, 2 and 3 are summarized in **Table A-1** below. Both the format and content shown in the table is largely adapted from the well-referenced papers by Koerner and Daniel (1993), Bonaparte et. al. (2002), as well as from general liner engineering practice over the past two decades, with some site-specific modifications that are considered appropriate for the construction of the MC VI-G Phase 2 liner.

Table A-1. Generalized Technical Equivalency Assessment for Liners Beneath Landfills

Category	Criterion for Evaluation	Equivalency of GCL to CCL			
		GCL is superior	GCL is equivalent	Equivalency is product-, design-, or site-specific	Category irrelevant to this project
Hydraulic	Steady state water flux	X			Evaluation will focus on site-specific leachate
	Breakthrough time - water	X			Evaluation will focus on site-specific leachate
	Horizontal flow in seams or lifts		X		-
	Horizontal flow beneath geomembranes	X			-
	Steady state solute flux			X	-
	Chemical adsorptive capacity / Solute breakthrough time			X	-
	Permeability to gases	-	-	-	A non-issue when GCL is installed under FML
	Generation of consolidation water	X			-
Physical/ Mechanical	Freeze-thaw behavior	X			-
	Wet-dry behavior	X			-
	Vulnerability to erosion	-	-	-	Erosion is irrelevant in the proposed liner
	Total settlement		X		-
	Differential settlement	X			-
	Stability on slopes			X	-
	Bearing capacity			X	-
Construction	Puncture resistance			X	-
	Ease of placement	X			-
	Speed of construction	X			-
	Availability of material	X			-
	Requirements of water	X			-
	Air pollution concerns	X			-
	Quality assurance considerations		X		-

Category of which GCL is superior than CCL	Category of which equivalency is product-, design-, or site-specific
Category of which GCL is equivalent to CCL	Category is irrelevant to this project

As shown in **Table A-1**, the following five items (criterion) are identified for Step 4 discussed above:

Hydraulic Properties

- Steady state solute flux
- Chemical adsorptive capacity / Solute breakthrough time

Physical/Mechanical Properties

- Stability of slopes
- Bearing capacity

Construction Properties

- Puncture resistance/subgrade condition

These items were subjected to detailed comparison between GCLs and CCLs as presented in the following sections.

Hydraulic Properties

Steady state solute flux

Past testing and experience have shown that sodium bentonite (the interlayer of GCL) is chemically compatible with many common waste streams, including leachate, some petroleum hydrocarbons, deicing fluids, livestock wastes, and dilute sodium cyanide mine waste.

In certain chemical environments, the sodium ions in bentonite can be replaced with cations dissolved in the water that comes in contact with the GCL, a process referred to as cation exchange. This type of exchange reaction can reduce the amount of water that can be held in the interlayer, resulting in decreased swell.

With the design and installation configuration shown in **Figure A-1** in mind, the steady state solute flux equivalency demonstration was prepared and presented in **Tables A-2a** and **A-2b**. Please note that the following assumptions were made in the demonstration for additional conservatism:

1. Comparisons were made as if the 80-mil HDPE primary geomembrane liner does not exist. In other words, GCL's superior swelling capability to "plug" holes or imperfections in the overlying HDPE liner is completely ignored.

2. Considering the evaluation performed by the GCL manufacturer of GCL chemical compatibility with site specific leachate data, the hydraulic conductivity of the upper GCL (Resistex® 200) is assumed at 1 x 10^{-8} cm/sec despite the tested results suggesting a permeability of 1.5 x 10^{-9} cm/sec. As discussed previously, this adjustment serves to conservatively address the concern of chemical compatibility associated with site-specific leachate. This adjustment is extremely conservative since this GCL layer will be completely covered by a layer of 80-mil HDPE geomembrane liner and hydration of GCL by leachate can only take place if there is leachate leakage through liner imperfections. The chance of this assumed scenario (i.e., the entire GCL layer is exposed to leachate with an increased hydraulic conductivity) does not practically exist.

3. Values of head-on-liner used in the evaluation were selected as 12.0 inches (30.5 cm) for the cell floor (per regulation) and 6.0 inches (15.2 cm) for sideslopes steeper than 4(H):1(V). Please note that the head-on-liner over both the floor and the sideslope is calculated as not to exceed 6 inches as shown in the "Maximum head-on-liner calculation" included in **Appendix A-2**. Moreover, while only the standard GCL product (Bentomat® DN) is used in the flux calculation, the calculated maximum head-on-liner will theoretically occur near the toe of the sideslope where the specialty GCL (Bentomat® CL) will be installed. This presents an additional conservative factor of safety.

4. Technically, an "apples-to-apples" comparison of steady state solute flux should be made by comparing flux that comes from the bottom of the 5-ft attenuation layer (in the proposed design case) and from the bottom of the 5-ft CCL layer (in the permitted design case). However, the equivalency evaluation was conservatively conducted by determining the flux that flows through the two layers of GCLs and comes out the bottom of the lower GCL layer (Bentomat® CL). In other words, any flow retardation capacity that could be provided by the underlying 5-ft thick cohesive attenuation layer is completely ignored in this evaluation.

5. Consequent to assumptions 3 and 4 discussed above, the hydraulic gradient (the driving force that causes flow to take place) selected for the proposed liner case is 14 times and 8 times greater than that selected for the permitted liner case for floor and sideslope liners, respectively. This represents another very conservative assumption.

The evaluation of the steady state solute flux criteria is made by dividing the calculated steady state solute flux of the proposed liner (GCL) by the number associated with the permitted liner (CCL). The resulting "ratio", if it is less than or equal to 100%, would indicate that the performance of the proposed liner system is acceptable, and therefore technical equivalency is demonstrated.

Input parameters, assumptions, and results of the steady state solute flux evaluation are presented in **Tables A-2a** and **A-2b** for cell floor and slopes that are steeper than 4(H):1(V), respectively.

Table A-2a. Steady State Solute Flux Equivalency Demonstration
Liner over Cell Floor and Slopes ≤ 4(H):1(V)

Layer	Thickness (cm)	K (cm/sec) (water)	K (cm/sec) (WDI leachate)	Additional adjustment	Adjusted K (cm/sec)	Thickness/ Perm
Resistex 200	0.95	3E-09	1.5E-09	6.7	1.0E-08	47,625,000
Bentomat CL	0.95	5E-10	5E-10	1.0	5E-10	1,905,000,000

Saturated thickness of GCL = 0.375" (or 0.95 cm)

K equivalent	
1E-09	cm/sec

$$k_v = \frac{H}{\left(\frac{H_1}{k_1}\right) + \left(\frac{H_2}{k_2}\right) + \left(\frac{H_3}{k_3}\right) + \ldots\ldots\ldots + \left(\frac{H_n}{k_n}\right)}$$

Demonstration is made by comparing the steady-state flux (Q's) using Darcy's Law Q = kiA (assuming no geomembrane)

Clay Liner	K_{eq} (cm/sec)	head (cm)	thickness (cm)	gradient i	Flux, Q (gal/acre-day)
5-ft of CCL	1E-07	30.48	152.4	1.20	111
Resistex 200 / Bentomat CL	1E-09	30.48	1.91	17.0	15
Conversion: 1.0 $cm^3/sec/cm^2$ =	*9.237E+08*	*gal/acre/day*		**Q_{GCL}/Q_{CCL} =**	**14%**

Table A-2b. Steady State Solute Flux Equivalency Demonstration

Liner on Slopes ≥ 4(H):1(V)

Layer	Thickness (cm)	K (cm/sec) (water)	K (cm/sec) (WDI leachate)	Adjustment factor	Adjusted K (cm/sec)	Thickness/ Perm
Resistex 200	0.95	3E-09	5E-09	2.0	1E-08	158,750,000
Bentomat DN	0.95	5E-09	5E-09	1.0	5E-09	190,500,000

Saturated thickness of GCL = 0.375" (or 0.95 cm)

K equivalent	
5.5E-09	cm/sec

$$k_v = \frac{H}{\left(\frac{H_1}{k_1}\right) + \left(\frac{H_2}{k_2}\right) + \left(\frac{H_3}{k_3}\right) + \dots\dots\dots + \left(\frac{H_n}{k_n}\right)}$$

Demonstration is made by comparing the steady-state flux (Q's) using Darcy's Law Q = kiA (assuming no geomembrane)

Clay Liner	K_{eq} (cm/sec)	head (cm)	thickness (cm)	gradient i	Flux, Q (gal/acre-day)
5-ft of CCL	1E-07	15.2	152.4	1.10	102
Resistex 200 / Bentomat DN	5E-09	15.2	1.91	9.0	45
Conversion: 1.0 $cm^3/sec/cm^2$ =	*9.237E+08*	*gal/acre/day*		**Q_{GCL}/Q_{CCL} =**	**45%**

As shown in **Tables A-2a** and **A-2b**, the steady state solute flux “ratios” are 14% and 45% for the cell floor and sideslope, respectively. Both numbers are significantly less than 100% indicating the performance of the proposed liner system is superior. Therefore, technical equivalency is demonstrated and the proposed liner system is acceptable.

Chemical adsorptive capacity / Solute breakthrough time

Federal and State regulations focus on preventing contamination of groundwater (CFR 40 Part 264.301(b) and Michigan Part 111 R299.9620(4)(a)). Therefore, selecting a point in the subsoil that has the same hydrogeological characteristics and distance to groundwater and using that point as a reference for both liner systems would be an appropriate approach in demonstrating equivalency.

As shown in **Figure A-2**, two models were established according to the concept described above: (a) permitted and constructed MC VI-G Phase 1 liner and (b) proposed MC VI-G Phase 2 liner. As shown in **Figure A-2**, the thickness of in-situ clayey subsoils under the existing waste where the proposed MC VI-G Phase 2 will be constructed, is approximately the same as the combined thickness of MC VI-G Phase 1 CCL liner and its in-situ clayey soil.

This is an important finding since numerical equivalency, in terms of chemical adsorptive capacity and solute breakthrough time, can already be achieved by the 10-ft in-situ clay present in the MC VI-G Phase 2 subsoils since all clayey soils (e.g., CCL or in-situ clay) exhibit a similar diffusion coefficient (Lake and Rowe (2005)).

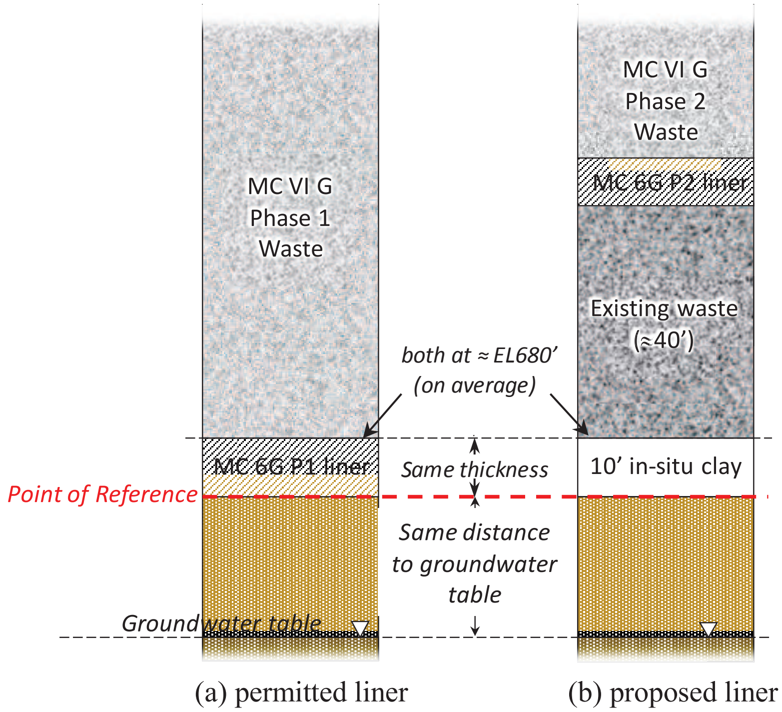


Figure A-2. Conceptual Model for Chemical Adsorptive Capacity and Breakthrough Time Comparison

In addition, as shown in **Figure A-1**, the proposed MC VI-G Phase 2 liner system contains 7-ft of cohesive soil layers (5-ft attenuation layer and 2-ft structural fill). Since the distance between the contaminant source (leachate above the primary liner) and the point of reference is significantly thicker for the proposed MC VI-G Phase 2 compared to MC VI-G Phase 1, the breakthrough time will be significantly increased in the proposed system.

Another factor impacting the breakthrough time is the steady state flux passing through the liner system (higher flux would lead to shorter breakthrough time). Since it has already been demonstrated (see **Tables A-2a** and **A-2b**) that the proposed GCL liner system will significantly reduce the steady state flux, the GCL liner system should also significantly increase the advective breakthrough time.

Additionally, as shown in **Figure A-2b**, approximately 40-ft of existing waste further separates the new waste in MC VI-G Phase 2 from the in-situ clay subsoil and groundwater. This existing waste layer provides additional chemical adsorptive capacity due to the following properties:

- Its anaerobic natural and high sulfide condition could bond heavy metals (Bhattacharyya et. al. (2006) and Robinson and Sum (1980))

- Non-degradable organic and other material provide additional adsorption and/or absorption capabilities for organic contaminants (De Gisi et. al. (2016) and Erses et. al. (2005))
- Additional biological activity reduces the half-life of organic pollutants and reduces potential breakthrough (Christensen et. al. (1994) and Guan et. al. (2014))
- Increases the mass transport distance and further reduces the concentration gradient (Shackelford (2013) and Xie (2015)
- Reduces the "concentration gradient" with the contaminants in the existing waste

Based on the above discussions, the performance of the proposed MC VI-G Phase 2 liner system is superior in the criterion of chemical adsorptive capacity / solute breakthrough time than the reference case (MC VI-G Phase 1 liner system). Therefore, technical equivalency is demonstrated and the proposed liner system is acceptable.

Physical/Mechanical Properties

Stability of slope

The GCL industry has addressed concerns related to GCL interface and internal shear resistance and its potential impact to landfill slope stability with products that will perform satisfactorily in typical landfill cell liner applications. For example, most GCL products are internally-reinforced with needle-punched fibers to ensure that the shear resistance of the bentonite interlayer exceeds standard stability requirements.

To demonstrate that the proposed liner system is technically equivalent to the permitted liner system with respect to slope stability, WDI examined the stability of the proposed liner system on the MC VI-G Phase 2 waste and liner slopes. Specifically, WDI verified that the proposed liner system does not introduce any interface and/or internal shear plane that is more critical than what is in the currently permitted liner system.

To verify stability, WDI referred to the slope stability analyses that were conducted and documented in the Basis of Design Report in the current permit (approved by the MDEQ on May 4, 2012 and EPA on September 27, 2013), where the stability of the sideslope under excavation, stability of the liner system under construction, stability of the waste mass during filling, stability of the final cover, and stability of the long-term final closure were evaluated.

Two findings of the prior investigation that are relevant to this technical equivalency demonstration, both related to interface shear resistance, are identified and listed below:

- As long as the interim waste slope during filling does not exceed an inclination of 3.5(H) to 1(V), a friction angle of 13.8 degrees or higher between any different geosynthetic-to-geosynthetic or geosynthetic-to-soil interfaces will result in satisfactory factor of safety (FS) values of 1.5 or greater.
- As long as a combination of friction and adhesion under an overburden pressure of 1.0 psi is greater than a friction angle of 21.8 degrees, stability of liner systems on slopes not steeper than 3(H) to 1(V) can be ensured.

Historical data and past experiences indicate that these requirements can be readily met by liner systems that utilize GCL products. Nevertheless, WDI will, as part of the CQA requirements, conduct direct shear tests (ASTM D6243) for relevant GCL-related interfaces (e.g., against 80-mil textured HDPE geomembranes, between different GCL products, against cohesive attenuation layer soils, etc.) as well as internal shear strength for different GCL products before approving the products to be used for construction of the MC VI-G Phase 2 liner system.

<u>Bearing capacity</u>

Studies and past experiences have demonstrated that an adequate thickness of cover soil (1 foot or 300 mm) will prevent a decrease in GCL thickness due to construction equipment loading thereby ensuring appropriate GCL bearing capacity. Performance equivalency can be achieved by properly specifying the installation procedure of the GCL and cover soil and a robust CQC/CQA program. A minimum thickness of 1 foot (300 mm) of cover soil is specified as a technical requirement and CQA site personnel will observe/verify/ document that such a requirement is maintained between the equipment tires/tracks and the GCL at all times during the installation process.

For the same reason, the initial (lowest) lift of the attenuation layer will be constructed with a 1-ft lift thickness to ensure GCL in the secondary liner system does not encounter loading from the construction equipment without adequate soil protection.

Attachment D of the Permit Modification Letter Report includes the CQA manual and Installation Guidelines for the GCL.

Construction Properties

Puncture resistance

Liner systems face external puncture risk from debris in overlying waste and internal puncture risk from rocks in soil liner components potentially damaging geosynthetics. In this case there is also puncture risk by debris in the underlying waste in Master Cell IV.

External puncture resistance from overlying waste: The inclusion of GCLs arguably increases the resistance of the primary liner system to punctures from overlying debris by adding additional layers of geosynthetics. But ignoring that improvement as it is not the intended purpose of the GCLs, the primary composite liner is fundamentally unchanged in terms of puncture resistance. The GCL itself is protected from above by the one foot of sand, geocomposite and 80 mil membrane.

Internal puncture resistance: The primary GCL will rest directly on the attenuation layer and the secondary GCL will rest directly on the structural fill. Stones potentially present in the attenuation layer and structural fill will be prevented from puncturing the GCL by a rigorously designed and enforced CQC/CQA program. Technical specifications for the GCL, included in **Attachment D** of the Permit Modification Letter Report, limit any stone particle in the upper most lift of the subgrade soils (i.e., the attenuation layer and structural fill) to be not larger than 1 inch (25 mm) in size. Proof-rolling of the prepared subgrade surface is also required to reduce stone particle protrusion.

External puncture resistance from underlying waste: The GCL will be protected from underlying debris by the structural fill layer. The structural fill layer will be prevented from contacting potentially damaging underlying debris (this first assumes underlying waste will be exposed which may not occur) by a rigorously designed and enforced CQC/CQA program that will include removal of debris that reasonably could penetrate the structural fill and proof-rolling of the surface on which the structural fill layer will be constructed to reduce the potential for protrusion.

Additional subgrade preparation requirements are listed in the CQA Manual and manufacturer's specifications included in **Attachment D** of the Permit Modification Letter Report. The Certifying Engineer's approval of the subgrade must also be obtained prior to GCL installation.

Conclusions

Wayne Disposal, Inc. is proposing the use of GCL in the construction of MC VI-G Phase 2 Subcells G2 and G3. WDI has presented information above demonstrating that the proposed liner system is equivalent or superior to the currently permitted liner system and is capable of preventing the migration of hazardous constituents into the groundwater or surface water at least as effectively as the approved liner system.

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List of Appendices

Appendix A-1 Chemical Compatibility Evaluation Report Provided by CETCO

Appendix A-2 Maximum Head-on-Liner Calculation, Revision 1, May 16, 2018

Appendix A-1: Chemical Compatibility Evaluation Report Provided by CETCO

CETCO 2870 Forbs Avenue Hoffman Estates, IL 60192
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May 1, 2018

Te-Yang Soong, Ph.D., P.E.
CTI and Associates, Inc.
28001 Cabot Drive, Ste. 250
Novi, MI 48377

RE: US Ecology's Wayne Disposal, Inc., Master Cell VI Sub-Cell G Phase 2
Geosynthetic Clay Liner – Tier I Report

Dear Mr. Soong:

The purpose of this letter is to present the results of compatibility testing of the CETCO® CG-50® bentonite used to make our Bentomat® products and the Resistex® geosynthetic clay liner (GCL) for the above mentioned project. This report is being made at the completion of the permeability testing for Resistex® 200 FLW9 GCL. All testing was performed by CETCO®'s in-house GAI-LAP accredited laboratory located in Hoffman Estates, Illinois.

Per your request, CETCO® initiated a geosynthetic clay liner (GCL) chemical compatibility evaluation as outlined in our Technical Reference (TR-345, attached) in April 2018 after receiving a representative sample of leachate. Completion of Tier I and II evaluations (see TR-345) indicated that a standard GCL (Bentomat®) in the presence of the leachate would likely not provide suitable performance as defined by permeability. CETCO®'s Resistex® 200 FLW9 GCL was also evaluated for its Tier II performance and is CETCO®'s recommended product for Tier III testing.

Permeability testing was completed in general accordance with ASTM D6766, Scenario II. For this testing, a cell pressure of 80 pounds per square inch (psi), 77 psi headwater pressure, and 75 psi tailwater pressure were utilized and represent test conditions that CETCO® utilizes in evaluating our GCL products. Permeability testing of the Resistex® 200 FLW9 product was terminated upon your request after 243.0 hours and 0.7 pore volumes of flow through the sample. The final average permeability for the Resistex® 200 FLW9 product was 1.5×10^{-9} cm/sec.

In addition to our Tier I & II results please find enclosed a copy of our Technical Data Sheet and Technical Reference. We appreciate your interest in CETCO® products. Please contact Tom Hauck, CETCO® Technical Sales Manager, at (248) 652-9274 if you have any further questions.

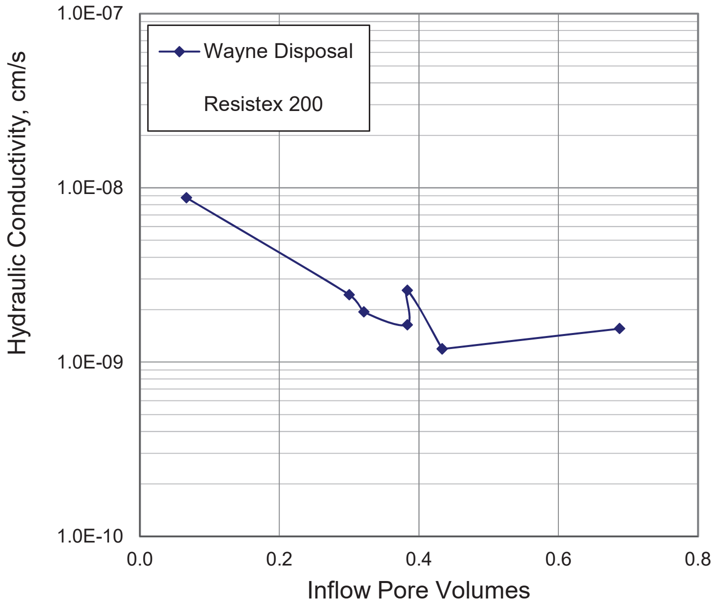
Table 1. Summary of final three measurements for the Resistex® 200 fLW9 product

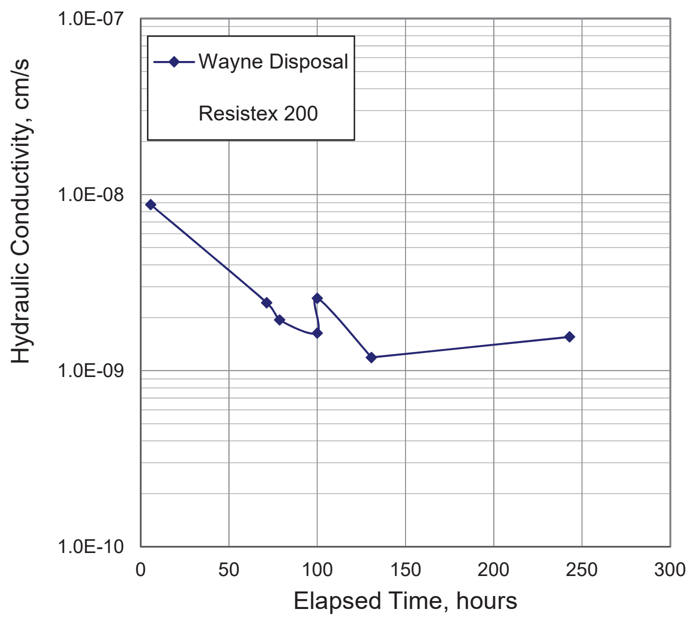
Elapsed Time (hr)	Pore Volumes	Inflow/ Outflow	Permeability (cm/sec)
100.0	0.383	0.96	1.6×10^{-9}
130.7	0.433	0.96	1.2×10^{-9}
243.0	0.688	0.96	1.6×10^{-9}

Very truly yours,

John M. Allen, P.E.
Technical Services Manager
CETCO® Environmental Products

Attachments (3)





Permeability with pore volumes and time for the Resistex® 200 FLW9 GCL using site specific leachate per ASTM D6766, Scenario II, for the US Ecology's Wayne Disposal, Inc., Master Cell VI Sub-Cell G Phase 2

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Analytical Results for the provided leachate for US Ecology's Wayne Disposal, Inc., Master Cell VI Sub-Cell G Phase 2 Project

Leachate Code Number	LT 18-1
Leachate Description	leachate
Leachate Type	leachate
Actual pH	9.250
Actual EC (uS/cm)	48,600
Calculations	**LT 18-1**
ICP Estimated EC (uS/cm) (Snoeyink Jenkins)	43281.45
Ionic Strength Estimated by ICP (mol/L)	0.693
RMD Estimated by ICP (M^0.5)	5.370
Ratio of SO4/Cl	0.190

Cl^-	16400.000
Ag+	0.169
Al^{3+}	
As3+	2.816
B4O5(OH)4	51.462
Ba2+	1.778
Ca^{2+}	47.013
Cd2+	0.189
Cr^{3+}	0.211
Cu2+	0.123
Fe^{+2}	3.859
Hg2+	3.527
K+	2231.718
Mg^{2+}	102.739
Mn^{2+}	1.216
Mo2+	11.253
Na^+	9056.907
Ni^{3+}	1.473
P of PO4-3	10.700
Pb2+	1.359
S	2811.831
Sb+2	0.968
Se2+	0.754
Ti4+	0.124
Zn^{2+}	0.532
Zr4+	0.219
H+(Calculated)	0.000
OH- (Calculated)	0.302

EVALUATING GCL CHEMICAL COMPATIBILITY

Sodium bentonite is an effective barrier primarily because it can absorb water (i.e., hydrate and swell), producing a dense, uniform layer with extremely low hydraulic conductivity, on the order of 10^{-9} cm/sec. Water absorption occurs because of the unique physical structure of bentonite and the complementary presence of sodium ions in the interlayer region between the bentonite platelets. Sodium bentonite's exceptional hydraulic properties allow GCLs to be used in place of much thicker soil layers in composite liner systems.

Sodium bentonite which is hydrated and permeated with relatively "clean" water will perform as an effective barrier indefinitely. In addition, past testing and experience have shown that sodium bentonite is chemically compatible with many common waste streams, including Subtitle D municipal solid waste landfill leachate (TR-101 and TR-254), some petroleum hydrocarbons (TR-103), deicing fluids (TR-109), livestock waste (TR-107), and dilute sodium cyanide mine wastes (TR-105).

In certain chemical environments, the interlayer sodium ions in bentonite can be replaced with cations dissolved in the water that comes in contact with the GCL, a process referred to as ion exchange. This type of exchange reaction can reduce the amount of water that can be held in the interlayer, resulting in decreased swell. The loss of swell usually causes increased porosity and increased GCL hydraulic conductivity. Experience and research have shown that calcium and magnesium are the most common source of compatibility problems for GCLs (Jo et al, 2001, Shackelford et al, 2000, Meer and Benson, 2004, Kolstad et al, 2004/2006). Examples of liquids with potentially high calcium and magnesium concentrations include: leachates from lime-stabilized sludge, soil, or fly ash; extremely hard water; unusually harsh landfill leachates; and acidic drainage from calcareous soil or stone. Other cations (ammonium, potassium, and sodium) may contribute to compatibility problems, but they are generally not as prevalent or as concentrated as calcium (Alther et al, 1985), with the exception of brines and seawater. Even though these highly concentrated solutions do not necessarily contain high levels of calcium, their high ionic strength can reduce the amount of bentonite swelling, resulting in increased GCL hydraulic conductivity.

This reference discusses the tools that can be used by a design engineer to evaluate GCL chemical compatibility with a site-specific leachate or other liquid.

HOW IS GCL CHEMICAL COMPATIBILITY EVALUATED?

Ideally, concentration-based guidelines would be available for determining GCL compatibility with a site-specific waste. Unfortunately, considering the variety and chemical complexity of the liquids that may be evaluated, as well as the many variables that influence chemical compatibility (e.g., prehydration with subgrade moisture [TR-222], confining stress [TR-321], and repeated wet-dry cycling [TR-341]), it is not possible to establish such guidelines. Instead, a three-tiered approach to evaluating GCL chemical compatibility is recommended, as outlined below.

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Tier I

The first tier is a simple review of existing analytical data. The topic of GCL chemical compatibility has been the subject of much study in recent years, with several important references available in the literature. One of these references, Kolstad et al (2004/2006), reported the results of several long-term hydraulic conductivity tests involving GCLs in contact with various multivalent (i.e., containing both sodium and calcium) salt solutions. Based on the results of these tests, the researchers found that a GCL's long-term hydraulic conductivity (as determined by ASTM D6766) can be estimated if the ionic strength (I) and the ratio of monovalent to divalent ions (RMD) in the permeant solution are both known, using the following empirical expression:

$$\frac{\log K_c}{\log K_{DI}} = 0.965 - 0.976 \times I + 0.0797 \times RMD + 0.251 \times I^2 \times RMD$$

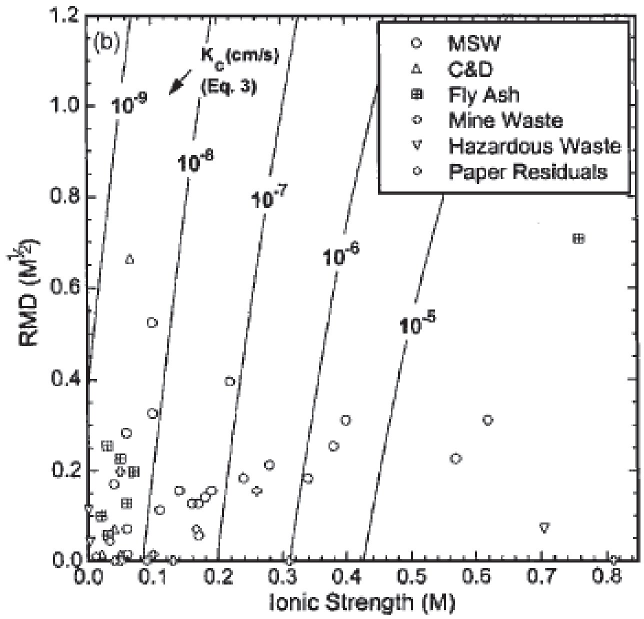
where:

I = ionic strength (M) of the site-specific leachate.

RMD = ratio of monovalent cation concentration to the square root of the divalent cation concentration ($M^{1/2}$) in the site-specific leachate.

K_c = GCL hydraulic conductivity when hydrated and permeated with site-specific leachate (cm/sec).

K_{DI} = GCL hydraulic conductivity with deionized water (cm/sec).



Using this tool, a Tier I compatibility evaluation can be performed if the major ion concentrations (typically, calcium, magnesium, sodium, and potassium) and ionic strength (estimated from either the total dissolved solids [TDS], or electrical conductivity [EC]) of the site leachate are known. For example, using the relationship above and MSW leachate data available in the literature, Kolstad et al. were able to conclude that high hydraulic conductivities (i.e., $>10^{-7}$ cm/sec) are unlikely for GCLs in base liners in many solid waste containment facilities.

In many cases, the Tier I evaluation is sufficient to show that a site-specific leachate should not pose compatibility problems. However, if the analytical data indicate a potential impact to GCL hydraulic performance, or if there is no analytical data available, then it is necessary to proceed to the second tier, involving bentonite "screening" tests, which are described below.

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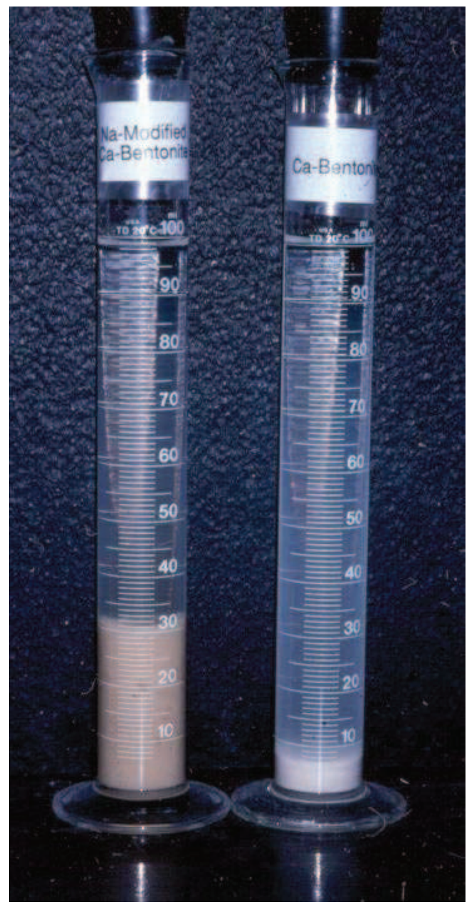
Tier II

The next tier of compatibility testing involves bentonite screening tests, performed in accordance with ASTM Method D6141. These tests are fairly straightforward, and can be performed at one of CETCO's R&D laboratories or at most commercial geosynthetics testing laboratories.

Liquid samples should be obtained very early in the project, such as during the site hydrogeological investigation. It is important that the sample collected is representative of actual site conditions. Synthetic leachate samples may also be considered for use in the compatibility tests. The objective is to create a liquid representative of that which will come in contact with the GCL. At least 1-gallon (4-Liter) of each sample should be submitted for testing. Samples should be accompanied by a chain-of-custody or information form. When a sample is received at the CETCO laboratory, the following screening tests are performed to assess compatibility:



- Fluid Loss (ASTM D5890) – A mixture of sodium bentonite and the site water/leachate is tested for fluid loss, an indicator of the bentonite's sealing ability.
- Swell Index (ASTM D5891) – Two grams of sodium bentonite are added to the site water/leachate and tested for swell index, the volumetric swelling of the bentonite.
- Water quality – The pH and EC of the site water/leachate are measured using bench-top water quality probes. pH will indicate if any strong acids (pH < 2) or bases (pH > 12) are present which might damage the bentonite clay. EC indicates the strength of dissolved salts in the water, which can hamper the swelling and sealing properties of bentonite if present at high concentrations.
- Chemistry – The site water/leachate is analyzed for major dissolved cations using ICP. The analytical results can then be used to perform a Tier I assessment, if one has not already been done.

As part of this testing, fluid loss and free swell tests are also performed on clean, deionized, or "DI" water for comparison to the results obtained with the site water/leachate sample. Sodium bentonite tested with DI water is expected to have a free swell of at least 24 mL/2g and a fluid loss less than 18 mL. Changes in bentonite swell and fluid loss indicate that the constituents dissolved in the site water may have an impact on GCL hydraulic conductivity. However, since it is only a screening tool, there are no specific values for the fluid loss and swell index tests that the clay must meet in order to be considered chemically compatible with the test liquid in question. Differences between the results of the baseline tests and those conducted with the site leachate may warrant further hydraulic testing.

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A major drawback of the D6141 tests is the potential for a false “negative” result, meaning that the bentonite swell index or fluid loss might predict no impact to hydraulic performance, where in reality, there may be a long-term adverse effect. This is primarily a concern with dilute calcium or magnesium solutions, which may slowly affect GCL hydraulic performance over months or years. Short-term (2-day) bentonite screening tests would not be able to capture this type of long-term effect. This is not expected to be a concern with strong calcium or magnesium or high ionic strength solutions, which have been shown to impact GCL hydraulic conductivity almost immediately, and whose effects would therefore be captured by the short-term bentonite screening tests. Another limitation of the bentonite screening tests is their inability to simulate site conditions, such as clean water prehydration, increased confining pressure, and wet/dry cycling. These limitations can be in part addressed by moving to the third tier, a long-term GCL hydraulic conductivity test, discussed below.

Tier III

The third-tier compatibility evaluation consists of an extended GCL hydraulic conductivity test performed in accordance with ASTM D6766. This test method is essentially a hydraulic conductivity test, but instead of permeating the GCL sample with DI water, the site-specific leachate is used. Since leachates can often be hazardous, corrosive, or volatile, the testing laboratory must have permeant interface devices, such as bladder accumulators, to contain the test liquid in a closed chamber, and prevent contamination of the flow measurement and pressure systems, or release of chemicals to the ambient air.

Method D6766 provides some flexibility in specifying the testing conditions so that certain site conditions can be simulated. For example, in situations where the GCL will be deployed on a subgrade soil that is compacted wet of optimum, the GCL will very likely hydrate from the relatively clean moisture in the subgrade (TR-222), long before it comes in contact with the potentially aggressive site leachate. Lee and Shackelford (2005) showed that a GCL which is pre-hydrated with clean water before being exposed to a harsh solution is expected to exhibit a lower hydraulic conductivity than one hydrated directly with the solution. Depending on the expected site conditions, the D6766 test can be specified to pre-hydrate the GCL with either water (Scenario 1) or the site liquid (Scenario 2).

Another site-specific consideration is confining pressure. Certain applications, such as landfill bottom liners and mine heap leach pads, involve up to several hundred feet of waste, resulting in high compressive loads on the liner systems. Although the standard confining pressure for the ASTM D6766 test is 5 psi (representing less than 10 feet of waste), the test method is flexible enough to allow greater confining pressures, thus mimicking conditions in a landfill bottom liner or heap leach pad. Petrov et al (1997) showed that higher confining pressures will decrease bentonite porosity, and tend to decrease GCL permeability. TR-321 shows that higher confining pressures will improve hydraulic conductivity even when the GCL is permeated with aggressive calcium solutions.

ASTM D6766 has two sets of termination criteria: hydraulic and chemical. To meet the hydraulic termination criterion, the ratio of inflow rate to outflow rate from the last three readings must be between 0.75 and 1.25. It normally takes between one week and one month to reach the hydraulic termination criterion. To meet the chemical termination criterion, the test must continue until at least two pore volumes of flow have passed through the sample and chemical equilibrium is established between the effluent and influent. The test method defines chemical equilibrium as effluent electrical conductivity within ±10% of the influent electrical conductivity. This requirement was put in place to ensure that a large enough volume of site liquid passes through the sample to allow slow ion exchange reactions to occur. Two pore volumes can take approximately a month to permeate through the GCL sample. However, reaching chemical equilibrium (effluent EC within 10% of influent EC), may take more than a year of testing, depending on the leachate characteristics.

ASTM D6766 is a very useful tool which provides a fairly conclusive assessment of GCL chemical compatibility with a site-specific leachate. However, the major drawback of the D6766 test is the potentially long period of time required to reach chemical equilibrium. This limitation reinforces the need for upfront compatibility testing early in the project. Clearly, requiring the contractor to perform this testing during the construction phase is not recommended.

WHAT DO THE ASTM D6766 COMPATIBILITY TEST RESULTS MEAN?

ASTM D6766 is currently the state-of-the-practice in the geosynthetics industry for evaluating long-term chemical compatibility of a GCL with a particular site waste stream. An ASTM D6766 test that is properly run until both the hydraulic (inflow and outflow within ±25% over three consecutive readings) and chemical (effluent EC within ±10% of influent EC) termination criteria are achieved, provides a good approximation of the GCL's long-term hydraulic conductivity when exposed to the site leachate. Jo et al (2005) conducted several GCL compatibility tests with weak calcium and magnesium solutions, with some tests running longer than 2.5 years, representing several hundred pore volumes of flow. The intent of this study was to run the tests until complete ion exchange had occurred, which required even stricter chemical equilibrium termination criteria than the D6766 test. The study found that the final GCL hydraulic conductivity values measured after complete ion exchange were fairly close to (within 2 to 13 times) the hydraulic conductivity values determined by ASTM D6766 tests, which took much less time to complete.

The laboratory that performs the chemical compatibility test, whether it is the CETCO R&D laboratory or an independent third-party laboratory, is only reporting the test results under the specified testing conditions, and is not making any guarantees about actual field performance or the suitability of a GCL for a particular project. It is the design engineer's responsibility to incorporate the D6766 results into their design to determine whether the GCL will meet the overall project objectives. Neither the testing laboratory nor the GCL manufacturer can make this determination.

Also, it is important to note that the results of D6766 testing for a particular project are only applicable for that site, for the specific waste stream that is tested, and only for the specific conditions replicated by the test. For instance, D6766 testing performed at high normal loads representative of a landfill bottom liner should not be applied to a situation where the GCL will only be placed under a modest normal load, such as a landfill cover or pond. Similarly, the results of a D6766 test where the GCL was pre-hydrated with clean water should not be applied to sites located in extremely arid climates where little subgrade moisture is expected, unless water will be applied manually to the subgrade prior to deployment. And finally, since D6766 tests are normally performed on continuously hydrated GCL samples, the test results should not be applied to situations where repeated cycles of wetting and drying of the GCL are likely to occur, such as in some GCL-only landfill covers, as desiccation can worsen compatibility effects.

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2. ASTM D 6141, Standard Guide for Screening Clay Portion of Geosynthetic Clay Liner for Chemical Compatibility to Liquids.
3. ASTM D 6766, Standard Test Method for Evaluation of Hydraulic Properties of Geosynthetic Clay Liners Permeated with Potentially Incompatible Liquids.
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6. CETCO TR-105, “Bentomat Compatibility Testing with Dilute Sodium Cyanide”.
7. CETCO TR-107, “GCL Compatibility with Livestock Waste”.
8. CETCO TR-109, “GCL Compatibility with Airport De-Icing Fluid”.
9. CETCO TR-222, “Hydration of GCLs Adjacent to Soil Layers”.
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Appendix A-2: Maximum Head-on-Liner Calculation

Revision 1, May 16, 2018

A-2.1: Maximum Head-on-Liner Calculation for Cell Floor

A-2.2: Maximum Head-on-Liner Calculation for Side Slope

A-2.3: CTI 2012, Head-on-Liner Calculation using Numerical Approach

A-2.4: NTH 2012, Leachate Generation Estimation and Head Calculation

A-2.1: Maximum Head-on-Liner Calculation for Cell Floor

Project Name:	Wayne Disposal, Inc.	Client:	US Ecology
Project Number:	1188070010	Project Manager:	Te-Yang Soong, Ph.D., P.E.
Project Location:	Belleville, Michigan	QA Manager:	Xianda Zhao, Ph.D., P.E.

Calculation Sheet Information

Calculation Medium: ☒ Electronic 2018 Permit Modification
☐ Hard-copy Number of pages (excluding cover sheet): 2

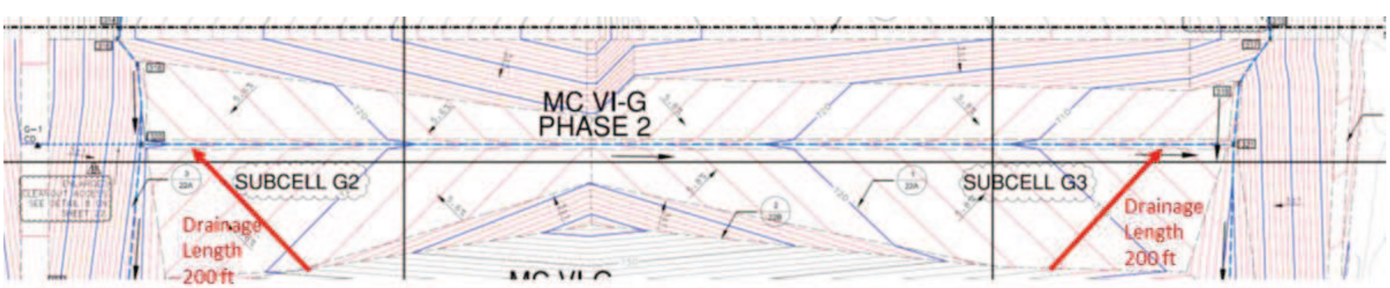
Title of Calculation:	Appendix A-2.1 Maximum Floor Head-on-Liner Calculation R.01
Calculation Originator:	Xianda Zhao
Calculation Contributors:	
Calculation Checker:	Te-Yang Soong

Calculation Objective

1. Determine the maximum leachate head on the cell floor of the primary liner in Master Cell VI-G Phase 2 Subcells G2 and G3

Design Criteria/Design Basis (with Reference to Source of Data)

1. Average daily peak leachate generation rates were obtained from "Leachate Generation Estimation and Head Calculation" (NTH, 2011).
2. A recessed leachate collection trench is proposed. The "free-drain" conditions for the leachate flow on the cell floor are satisfied.
3. The leachate head on liner is determined using the McEnroe Equation with a numerical method.
4. The transmissivity of the drainage Geocomposite is 2.4 x 10^{-4} square meters per second (m^2/sec) (6.1 x 10^{-5} m^2/sec prior applying the reduction factors).
5. Reduction factors of 1.75, 1.5, and 1.5 are selected for creep, chemical clogging and biological clogging, respectively.
6. The hydraulic conductivity of the protective soil over the Geocomposite is 1.0 x 10^{-5} meters per second (m/sec) based on R299.9619.
7. The maximum drainage length of subcells G2 and G3 is 200 ft.
8. The floor slopes of subcells G2 and G3 are 5.6% and 5.8%, respectively.



Results/Conclusions
1. The maximum heads are 2.7 and 1.6 inches on cell floors in subcells G2 and G3, respectively. The proposed design reduced the maximum leachate head on the floors of subcells G2 and G3 compared to the permitted design (5.0 and 5.7 on cell floors in subcells G2 and G3, respectively).

References/Source Documents
1. NTH 2011, WDI Operating License Application Master Cells VI F&G, Volume III Basis of Design Report. 2. Guideline and Manual for Planning and Design in Sewerage Systems., JSWA., 2009. 3. CTI 2012, Head-on-Liner Calculation using Numerical Approach.

Revision Records

Checker comments provided on: ☐ Hard-copy ☒ Electronic File

No.	Revision Identifier (Number or Letter)	Version Type		Originator Initials	Date	Checker Initials	Date
		Draft	Final				
1	Rev. 0	☒	☐	XZ	4/29/2018		
2	Rev. 1	☐	☒	XZ	5/9/2018	TYS	5/9/2018
3		☐	☐				
4		☐	☐				
		☐	☐				
		☐	☐				
		☐	☐				

Approval

☐ The Detail Check has been completed. Any significant issues not resolved between the Checker and Originator have been resolved by the Approver.

Xiude Zhao — 5/9/2018

Originator Signature — Date

5/9/2018

Checker Signature — Date

5/9/2018

Approved Signature — Date

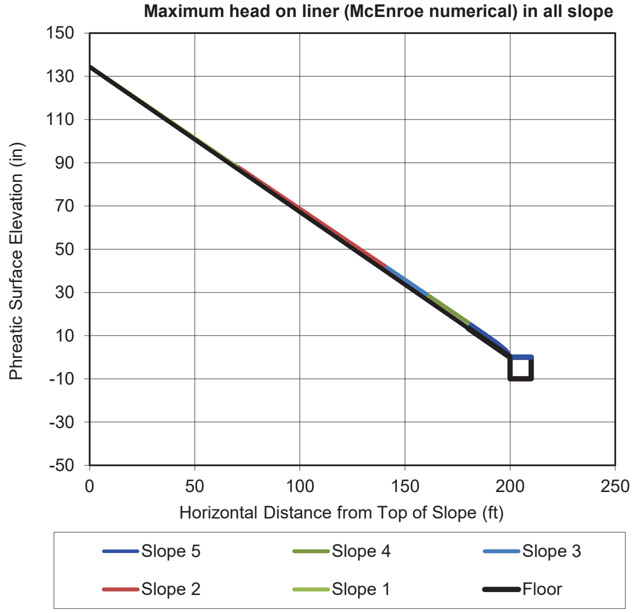
HEAD ON LINER CALCULATIONS

WDI MC6 G Phase 2 G2 - Floor

R.01

			SLOPE 5	SLOPE 4	SLOPE 3	SLOPE 2	SLOPE 1
Prepared by: XZ 5/9/2018							
Reviewed by: TYS 5/9/2018							
Approved by: XZ 5/9/2018	*minimum y (in)*	*0.010*	**Bottom**				**Top**
Slope in the direction of flow	S	ft./ft.	**5.60%**	**5.60%**	**5.60%**	**5.60%**	**5.60%**
Slope angle	α	radians	0.0559	0.0559	0.0559	0.0559	0.0559
Flow length in the direction of flow	L	ft.	**20**	**20**	**20**	**70**	**70**
Rate of vertical inflow per unit area	r	gal/acre/day	**8,960**	**8,960**	**8,960**	**8,960**	**8,960**
Thickness of sand (or protective soil)	t_{sand}	in	**3.0**	**3.0**	**3.0**	**3.0**	**2.0**
		ft.	*0.250*	*0.250*	*0.250*	*0.250*	*0.167*
Permeability of sand (or protective soil)	K_{sand}	cm/sec	**1.00E-03**	**1.00E-03**	**1.00E-03**	**1.00E-03**	**1.00E-03**
Thickness of geonet	t_{geonet}	in.	**0.200**	**0.200**	**0.200**	**0.200**	**0.200**
		ft.	**0.017**	**0.017**	**0.017**	**0.017**	**0.017**
Geonet transmissivity	m2/s	m2/s	**2.40E-04**	**2.40E-04**	**2.40E-04**	**2.40E-04**	**2.40E-04**
Reduction Factor			**3.96**	**3.96**	**3.96**	**3.96**	**3.96**
Permeability of geonet	K_{geonet}	cm/sec	1.19E+00	1.19E+00	1.19E+00	1.19E+00	1.19E+00
Combined (apparent) permeability	$\mathbf{K_{app}}$	**cm/sec**	**1.45E-01**	**1.45E-01**	**1.45E-01**	**1.45E-01**	**2.08E-01**
Leachate Head at Discharge Point	**h at L=0**	in	**0.10**	**2.63**	**2.35**	**2.05**	**1.03**
Step Size	dL	in	0.1	0.1	0.1	0.1	0.1
Unit Width	W	ft	1	1	1	1	1

Maximum head on liner (McEnroe numerical) in each slope	**in**	**2.69**	**2.63**	**2.35**	**2.05**	**1.03**
Maximum head on liner location (McEnroe numerical) in each slope	**ft**	**187.49**	**180.00**	**160.00**	**140.00**	**70.00**
		0.26	*0.32*	*0.73*	*1.31*	*1.38*
Maximum head on liner (McEnroe numerical) in all slope	**in**	**2.69**				



McEnroe 1993 "Maximum Saturated Depth over Landfill Liner"
Journal of Environmental Engineering

For Slope 1

$$y_{i+1} = y_i + \left(\tan\alpha_1 - \frac{r_1 x_i}{k_1 y_i \cos^2\alpha_1} \right)(x_{i+1} - x_i)$$

Fore Slopes 2 - 5

$$y_{i+1} = y_i + \left(\tan\alpha_j - \frac{\sum_{f=1}^{j-1} r_f L_f + r_j \left(x_i - \sum_{f=1}^{j-1} L_f \right)}{k_j y_i \cos^2\alpha_j} \right)(x_{i+1} - x_i)$$

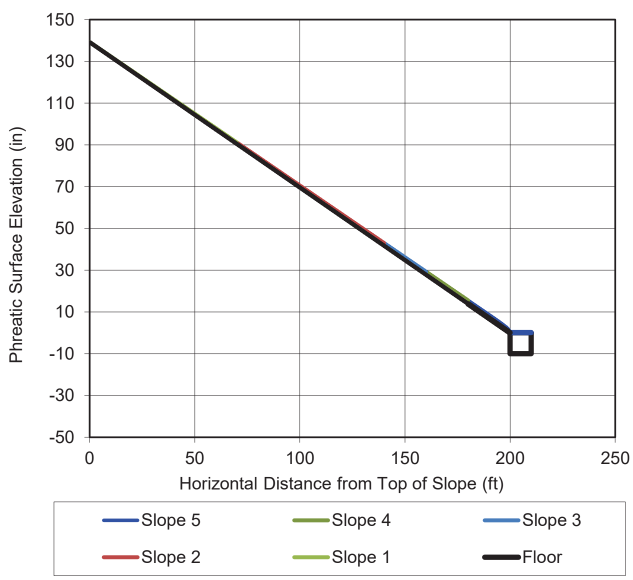
HEAD ON LINER CALCULATIONS

WDI MC6 G Phase 2 G3 - Floor

R.01

			SLOPE 5	SLOPE 4	SLOPE 3	SLOPE 2	SLOPE 1
Prepared by: XZ 5/9/2018							
Reviewed by: TYS 5/9/2018							
Approved by: XZ 5/9/2018	*minimum y (in)*	*0.010*	**Bottom**				**Top**
Slope in the direction of flow	S	ft./ft.	**5.80%**	**5.80%**	**5.80%**	**5.80%**	**5.80%**
Slope angle	α	radians	0.0579	0.0579	0.0579	0.0579	0.0579
Flow length in the direction of flow	L	ft.	**20**	**20**	**20**	**70**	**70**
Rate of vertical inflow per unit area	r	gal/acre/day	**7,874**	**7,874**	**7,874**	**7,874**	**7,874**
Thickness of sand (or protective soil)	t_{sand}	in	**2.0**	**2.0**	**2.0**	**2.0**	**2.0**
		ft.	*0.167*	*0.167*	*0.167*	*0.167*	*0.167*
Permeability of sand (or protective soil)	K_{sand}	cm/sec	**1.00E-03**	**1.00E-03**	**1.00E-03**	**1.00E-03**	**1.00E-03**
Thickness of geonet	t_{geonet}	in.	**0.200**	**0.200**	**0.200**	**0.200**	**0.200**
		ft.	**0.017**	**0.017**	**0.017**	**0.017**	**0.017**
Geonet transmissivity	m2/s	m2/s	**2.40E-04**	**2.40E-04**	**2.40E-04**	**2.40E-04**	**2.40E-04**
Reduction Factor			**3.94**	**3.94**	**3.94**	**3.94**	**3.94**
Permeability of geonet	K_{geonet}	cm/sec	1.20E+00	1.20E+00	1.20E+00	1.20E+00	1.20E+00
Combined (apparent) permeability	$\mathbf{K_{app}}$	**cm/sec**	**2.09E-01**	**2.09E-01**	**2.09E-01**	**2.09E-01**	**2.09E-01**
Leachate Head at Discharge Point	**h at L=0**	in	**0.10**	**1.54**	**1.37**	**1.20**	**0.60**
Step Size	dL	in	0.1	0.1	0.1	0.1	0.1
Unit Width	W	ft	1	1	1	1	1

Maximum head on liner (McEnroe numerical) in each slope	**in**	**1.62**	**1.54**	**1.37**	**1.20**	**0.60**
Maximum head on liner location (McEnroe numerical) in each slope	**ft**	**191.63**	**180.00**	**160.00**	**140.00**	**70.00**
		0.34	*0.43*	*0.69*	*1.00*	*2.56*
Maximum head on liner (McEnroe numerical) in all slope	**in**	**1.62**				



McEnroe 1993 "Maximum Saturated Depth over Landfill Liner"

Journal of Environmental Engineering

For Slope 1

$$y_{i+1} = y_i + \left(\tan \alpha_1 - \frac{r_1 x_i}{k_1 y_i \cos^2 \alpha_1} \right) (x_{i+1} - x_i)$$

Fore Slopes 2 - 5

$$y_{i+1} = y_i + \left(\tan \alpha_j - \frac{\sum_{f=1}^{j-1} r_f L_f + r_j \left(x_i - \sum_{f=1}^{j-1} L_f \right)}{k_j y_i \cos^2 \alpha_j} \right) (x_{i+1} - x_i)$$

A-2.2: Maximum Head-on-Liner Calculation for Side Slope

Project Name:	Wayne Disposal, Inc.	Client:	US Ecology
Project Number:	1188070010	Project Manager:	Te-Yang Soong, Ph.D., P.E.
Project Location:	Belleville, Michigan	QA Manager:	Xianda Zhao, Ph.D., P.E.

Calculation Sheet Information

Calculation Medium: ☒ Electronic 2018 Permit Modification
☐ Hard-copy Number of pages (excluding cover sheet): 4

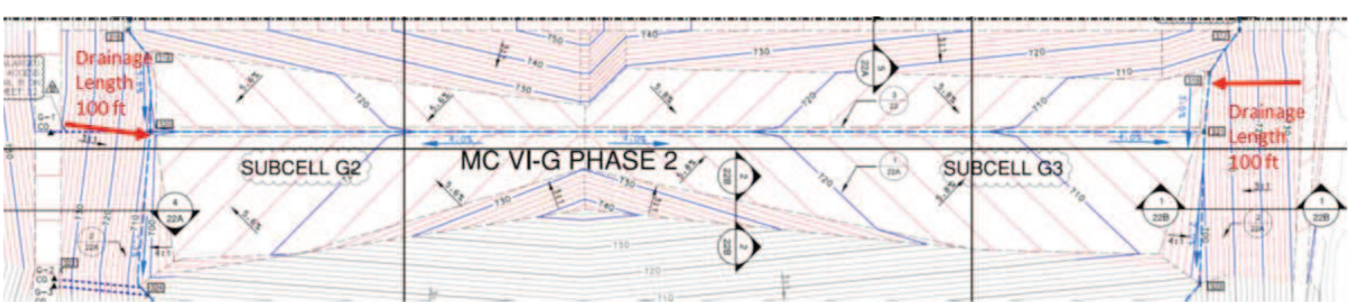
Title of Calculation:	Appendix A-2.2 Maximum Sideslope Head-on-Liner Calculation R.01
Calculation Originator:	Xianda Zhao
Calculation Contributors:	
Calculation Checker:	Te-Yang Soong

Calculation Objective

1. Determine the maximum leachate head on the side slope of the primary liner in Master Cell VI-G Phase 2 Subcells G2 and G3

Design Criteria/Design Basis (with Reference to Source of Data)

1. Average daily peak leachate generation rates were obtained from “Leachate Generation Estimation and Head Calculation” (NTH, 2011).
2. Liquid depth in the leachate collection pipe is determined using Manning’s equation.
3. The leachate head on liner is determined using the McEnroe Equation with a numerical method.
4. The minimum slope of the leachate collection pipe is assumed at 1%.
5. The transmissivity of the drainage Geocomposite is 1.2 x 10^{-4} square meters per second (m^2/sec) (3.0 x 10^{-5} m^2/sec prior applying the reduction factors based on R299.9619).
6. Reduction factors of 1.75, 1.5, and 1.5 are selected for creep, chemical clogging and biological clogging, respectively.
7. The hydraulic conductivity of the protective soil over the Geocomposite is 1.0 x 10^{-5} meters per second (m/sec) based on R299.9619.
8. The acreages of subcells G2 and G3 are 3.05 and 4.28 acres, respectively.



Results/Conclusions

1. A liquid depth of 2 inches was used to determine the flow capacity of the leachate collection pipe in the toe drain. The factors of safety are 3.8 and 3.1 for subcells G2 and G3, respectively. A total leachate depth of 5.18 inches was calculated for the toe drain.
2. Using a starting leachate level of 5.18 inches at the toe of the slope, the head on liner was determined. The maximum head is located at the starting point (toe of the side slope) and at a depth of **5.18** inches.

References/Source Documents

1. NTH 2011, WDI Operating License Application Master Cells VI F&G, Volume III Basis of Design Report.
2. Guideline and Manual for Planning and Design in Sewerage Systems., JSWA., 2009.
3. CTI 2012, Head-on-Liner Calculation using Numerical Approach.

Revision Records

Checker comments provided on: ☐ Hard-copy ☒ Electronic File

No.	**Revision Identifier (Number or Letter)**	**Version Type**		**Originator Initials**	**Date**	**Checker Initials**	**Date**
		Draft	**Final**				
1	Rev. 0	☒	☐	XZ	4/24/2018		
2	Rev. 1	☐	☒	XZ	5/9/2018	TYS	5/9/2018
3		☐	☐				
4		☐	☐				
		☐	☐				
		☐	☐				
		☐	☐				

Approval

☐ The Detail Check has been completed. Any significant issues not resolved between the Checker and Originator have been resolved by the Approver.

Xiunde Zhao	5/9/2018
Originator Signature	Date
[signature]	5/9/2018
Checker Signature	Date
[signature]	5/9/2018
Approved Signature	Date

FLOW CAPACITY CALCULATION FOR SUBCELL G2

Prepared by	***XZ***	***Date***	***4/24/2018***
Reviewed by	***TYS***	***Date***	***4/25/2018***
Approved by	***XZ***	***Date***	***4/25/2018***

Cell Area	**3.05**	ac
Perculation Rate	**0.33**	inch
	8960	gpad
Req. Q	3654	ft3/day
	19.0	gpm

Stone Bed	**2**	**inch**
Pipe Wall Thickness	**1.182**	**inch**
Liquid depth in Pipe	**2**	**inch**
Liquid depth in Trench	**5.18**	**inch**

PARTIAL FULL PIPE		
d (in)	**6.12**	
h (in)	**2**	
S	**0.01**	
n	**0.011**	
r	0.255	ft
h	0.166666667	ft
α	0.353736348	radians
	20.26759978	degree
Θ	2.434119958	radians
	139.4648004	degree
A (ft2)	0.058008961	
P (ft)	0.620700589	
Rh (ft)	0.093457235	
Q (ft3/s)	**0.162**	**FS**
Q (gpm)	**72.625**	**3.8**
Q (gpd)	**104,579.36**	
Q (m3/s)	0.0046	
V (ft/s)	**2.790**	

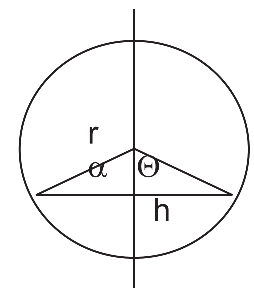
$$\alpha = \sin^{-1}\frac{r-h}{r} \text{ in radians}$$

$$\theta = \pi - 2\alpha$$

$$A = r^2\frac{\theta}{2} - (r-h)\left(\sqrt{2rh - h^2}\right)$$

$$P = r\theta$$

$$R = \frac{A}{P} = \frac{r}{2} - \frac{(r-h)\sqrt{2rh-h^2}}{r\theta}$$



$$Q_{allow} = \frac{1.49}{n} A R_h^{2/3} S^{1/2}$$

1.49 = conversion constant (SI to US)
n = 0.11 for HDPE pipe
A = Flow Area
P=wetted perimeter
Rh=hydraulic radius=area/perimeter

FLOW CAPACITY CALCULATION FOR SUBCELL G3

Prepared by	***XZ***	***Date***	***4/24/2018***
Reviewed by	***TYS***	***Date***	***4/25/2018***
Approved by	***XZ***	***Date***	***4/25/2018***

Cell Area	**4.28**	ac
Perculation Rate	**0.29**	inch
	7874	gpad
Req. Q	4506	ft3/day
	23.4	gpm

Stone Bed	**2**	**inch**
Pipe Wall Thickness	**1.182**	**inch**
Liquid depth in Pipe	**2**	**inch**
Liquid depth in Trench	**5.18**	**inch**

PARTIAL FULL PIPE		
d (in)	**6.12**	
h (in)	**2**	
S	**0.01**	
n	**0.011**	
r	0.255	ft
h	0.166666667	ft
α	0.353736348	radians
	20.26759978	degree
Θ	2.434119958	radians
	139.4648004	degree
A (ft2)	0.058008961	
P (ft)	0.620700589	
Rh (ft)	0.093457235	
Q (ft3/s)	**0.162**	**FS**
Q (gpm)	**72.625**	**3.1**
Q (gpd)	**104,579.36**	
Q (m3/s)	0.0046	
V (ft/s)	**2.790**	

$$\alpha = \sin^{-1}\frac{r-h}{r} \quad \text{in radians}$$

$$\theta = \pi - 2\alpha$$

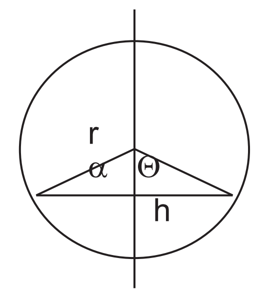
$$A = r^2\frac{\theta}{2} - (r-h)\left(\sqrt{2rh-h^2}\right)$$

$$P = r\theta$$

$$R = \frac{A}{P} = \frac{r}{2} - \frac{(r-h)\sqrt{2rh-h^2}}{r\theta}$$

$$Q_{allow} = \frac{1.49}{n} A R_h^{2/3} S^{1/2}$$

1.49 = conversion constant (SI to US)
n = 0.11 for HDPE pipe
A = Flow Area
P=wetted perimeter
Rh=hydraulic radius=area/perimeter



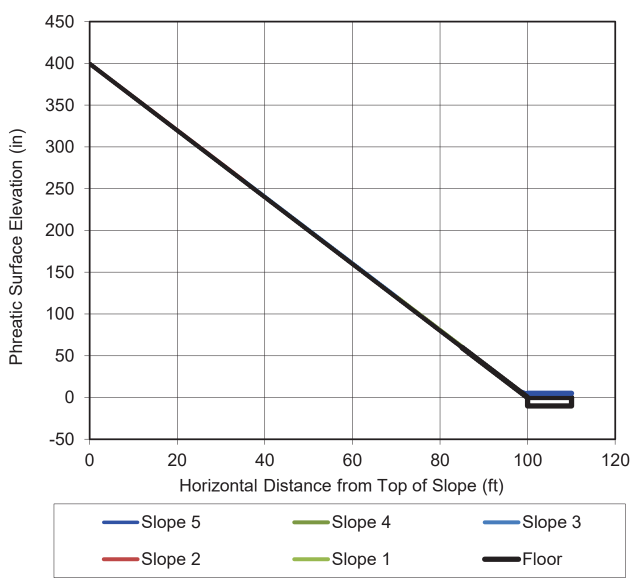
HEAD ON LINER CALCULATIONS

WDI MC6 G Phase 2 G2 - Side Slope

R.01

Prepared by: XZ 5/9/2018							
Reviewed by: TYS 5/9/2018			**SLOPE 5**	**SLOPE 4**	**SLOPE 3**	**SLOPE 2**	**SLOPE 1**
Approved by: XZ 5/9/2018	*minimum y (in)*	*0.010*	**Bottom**				**Top**
Slope in the direction of flow	S	ft./ft.	**33.30%**	**33.30%**	**33.30%**	**33.30%**	**33.30%**
Slope angle	α	radians	0.3215	0.3215	0.3215	0.3215	0.3215
Flow length in the direction of flow	L	ft.	**15**	**15**	**35**	**35**	**0**
Rate of vertical inflow per unit area	r	gal/acre/day	**8,960**	**8,960**	**8,960**	**8,960**	**8,960**
Thickness of sand (or protective soil)	t_{sand}	in	**6.0**	**6.0**	**6.0**	**6.0**	**6.0**
		ft.	*0.500*	*0.500*	*0.500*	*0.500*	*0.500*
Permeability of sand (or protective soil)	K_{sand}	cm/sec	**1.00E-03**	**1.00E-03**	**1.00E-03**	**1.00E-03**	**1.00E-03**
Thickness of geonet	t_{geonet}	in.	**0.200**	**0.200**	**0.200**	**0.200**	**0.000**
		ft.	**0.017**	**0.017**	**0.017**	**0.017**	**0.000**
Geonet transmissivity	m2/s	m2/s	**1.18E-04**	**1.18E-04**	**1.18E-04**	**1.18E-04**	**1.18E-04**
Reduction Factor			**3.94**	**3.94**	**3.94**	**3.94**	**3.94**
Permeability of geonet	K_{geonet}	cm/sec	5.91E-01	5.91E-01	5.91E-01	5.91E-01	0.00E+00
Combined (apparent) permeability	$\mathbf{K_{app}}$	**cm/sec**	**3.84E-02**	**3.84E-02**	**3.84E-02**	**3.84E-02**	**1.00E-03**
Leachate Head at Discharge Point	**h at L=0**	in	**5.18**	**0.86**	**0.71**	**0.35**	**0.01**
Step Size	dL	in	0.1	0.1	0.1	0.1	0.1
Unit Width	W	ft	1	1	1	1	1

Maximum head on liner (McEnroe numerical) in each slope	**in**	**5.18**	**0.86**	**0.71**	**0.35**	**0.01**
Maximum head on liner location (McEnroe numerical) in each slope	**ft**	**100.00**	**85.00**	**70.00**	**35.00**	**0.00**
		1.04	*28.50*	*30.15*	*34.17*	*35.88*
Maximum head on liner (McEnroe numerical) in all slope	**in**	**5.18**				



McEnroe 1993 "Maximum Saturated Depth over Landfill Liner"
Journal of Environmental Engineering

For Slope 1

$$y_{i+1} = y_i + \left(\tan \alpha_1 - \frac{r_1 x_i}{k_1 y_i \cos^2 \alpha_1} \right) (x_{i+1} - x_i)$$

Fore Slopes 2 - 5

$$y_{i+1} = y_i + \left(\tan \alpha_j - \frac{\sum_{f=1}^{j-1} r_f L_f + r_j \left(x_i - \sum_{f=1}^{j-1} L_f \right)}{k_j y_i \cos^2 \alpha_j} \right) (x_{i+1} - x_i)$$

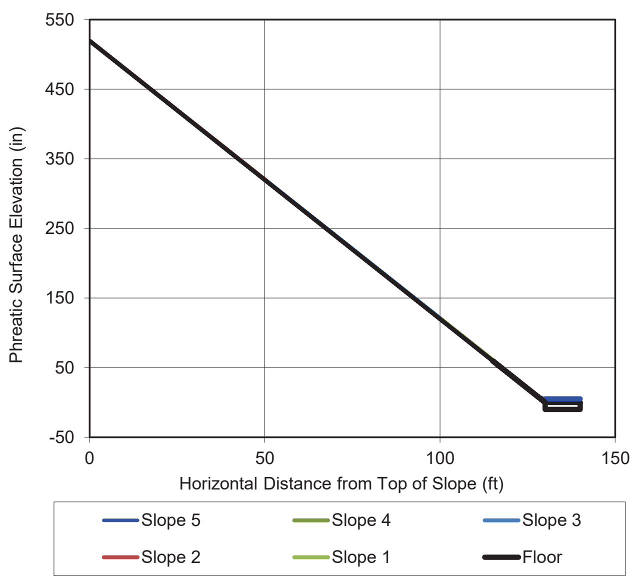
HEAD ON LINER CALCULATIONS

WDI MC6 G Phase 2 G3 - Side Slope

R.01

Prepared by: XZ 5/9/2018							
Reviewed by: TYS 5/9/2018			**SLOPE 5**	**SLOPE 4**	**SLOPE 3**	**SLOPE 2**	**SLOPE 1**
Approved by: XZ 5/9/2018	*minimum y (in)*	*0.010*	**Bottom**				**Top**
Slope in the direction of flow	S	ft./ft.	**33.30%**	**33.30%**	**33.30%**	**33.30%**	**33.30%**
Slope angle	α	radians	0.3215	0.3215	0.3215	0.3215	0.3215
Flow length in the direction of flow	L	ft.	**15**	**15**	**50**	**50**	**0**
Rate of vertical inflow per unit area	r	gal/acre/day	**7,874**	**7,874**	**7,874**	**7,874**	**7,874**
Thickness of sand (or protective soil)	t_{sand}	in	**6.0**	**6.0**	**6.0**	**6.0**	**6.0**
		ft.	*0.500*	*0.500*	*0.500*	*0.500*	*0.500*
Permeability of sand (or protective soil)	K_{sand}	cm/sec	**1.00E-03**	**1.00E-03**	**1.00E-03**	**1.00E-03**	**1.00E-03**
Thickness of geonet	t_{geonet}	in.	**0.200**	**0.200**	**0.200**	**0.200**	**0.000**
		ft.	**0.017**	**0.017**	**0.017**	**0.017**	**0.000**
Geonet transmissivity	m2/s	m2/s	**1.18E-04**	**1.18E-04**	**1.18E-04**	**1.18E-04**	**1.18E-04**
Reduction Factor			**3.94**	**3.94**	**3.94**	**3.94**	**3.94**
Permeability of geonet	K_{geonet}	cm/sec	5.91E-01	5.91E-01	5.91E-01	5.91E-01	0.00E+00
Combined (apparent) permeability	K_{app}	**cm/sec**	**3.84E-02**	**3.84E-02**	**3.84E-02**	**3.84E-02**	**1.00E-03**
Leachate Head at Discharge Point	**h at L=0**	in	**5.18**	**1.02**	**0.89**	**0.45**	**0.01**
Step Size	dL	in	0.1	0.1	0.1	0.1	0.1
Unit Width	W	ft	1	1	1	1	1

Maximum head on liner (McEnroe numerical) in each slope	**in**	**5.18**	**1.02**	**0.89**	**0.45**	**0.01**
Maximum head on liner location (McEnroe numerical) in each slope	**ft**	**130.00**	**115.00**	**100.00**	**50.00**	**0.00**
		1.04	*26.79*	*28.19*	*33.12*	*35.88*
Maximum head on liner (McEnroe numerical) in all slope	**in**	**5.18**				



McEnroe 1993 "Maximum Saturated Depth over Landfill Liner"

Journal of Environmental Engineering

For Slope 1

$$y_{i+1} = y_i + \left(\tan \alpha_1 - \frac{r_1 x_i}{k_1 y_i \cos^2 \alpha_1} \right) (x_{i+1} - x_i)$$

Fore Slopes 2 - 5

$$y_{i+1} = y_i + \left(\tan \alpha_j - \frac{\sum_{f=1}^{j-1} r_f L_f + r_j \left(x_i - \sum_{f=1}^{j-1} L_f \right)}{k_j y_i \cos^2 \alpha_j} \right) (x_{i+1} - x_i)$$

A-2.3: CTI 2012, Head-on-Liner Calculation using Numerical Approach

CALCULATION SHEET

Client: Landfill	Project No.:		
Project: Head-on-Liner Calculation	Calculated By: XZ	Date: 3/6/2012	
Calculation: Head-on-liner calculation using numerical approach	Checked By: TY	Date: 3/9/2012	
	Approved By: KF	Date: 5/30/2012	

HEAD-ON-LINER CALCULATION USING NUMERICAL APPROACH

OBJECTIVE

To determine the maximum saturated leachate depth within leachate drainage media above an impermeable liner using a numerical implementation of the McEnroe (1993) Equations .

DESIGN CRITERIA, ASSUMPTIONS AND METHODOLOGY

The head-on-liner calculation is conducted according to the following procedure:

1. Determine the average transmissivity value of drainage geocomposite using test results obtained under the design normal stress. This value is reduced through the application of several reduction factors as described in following equation (Koerner 2005):

$$\theta_{allow} = \frac{\theta_{test}}{RF_{IN} \times RF_{CR} \times RF_{CC} \times RF_{BC}} \qquad (1)$$

   Where,

   RF_{IN} = reduction factor for intrusion (or elastic deformation)
   RF_{CR} = reduction factor for creep deformation
   RF_{CC} = reduction factor for chemical clogging
   RF_{BC} = reduction factor for biological clogging
   θ_{allow} = allowable transmissivity for the geocomposite, m^2/s
   θ_{test} = tested transmissivity for the geocomposite, m^2/s

2. Determine the combined (apparent) hydraulic conductivity of the drainage layer (geocomposite overlain by a sand layer) using the equation by Qian et al. (2004):

$$k_{combined} = k_{geonet} + \left(k_{sand} - k_{geonet}\right)\frac{t_{sand}^{\;2}}{\left(t_{sand} + t_{geonet}\right)^2} \qquad (2)$$

   where,

   $k_{combined}$ = combined hydraulic conductivity of the saturated drainage layer (cm/s)
   k_{sand} = hydraulic conductivity of sand (cm/s)
   k_{geonet} = hydraulic conductivity of geocomposite (cm/s)
   t_{sand} = thickness of the saturated sand layer (in)
   t_{geonet} = thickness of geocomposite (in)

3. Head-on-liner calculation – McEnroe (1993) Method (valid only for free draining condition)

   A commonly used method for calculating the maximum head-on-liner was developed by McEnroe (1993). McEnroe (1993) developed a differential equation to describe the flow in the drainage layer using the extended Dupuit assumptions. McEnroe also derived an analytical solution from the governing differential equation to determine the maximum head (saturated depth) buildup under free draining conditions. McEnroe's 1993 method (under free draining conditions) is expressed as:

If R<1/4

$$y_{\max} = LS*\left(R - RS + R^2S^2\right)^{1/2} * \{[(1-A-2R)(1+A-2RS)]/[(1+A-2R)(1-A-2RS)]\}^{1/(2A)} \quad (3)$$

If R=1/4

$$y_{\max} = LSR*(1-2RS)/(1-2R)*\exp\{2R*(S-1)/[(1-2RS)(1-2R)]\} \quad (4)$$

If R>=1/4

$$y_{\max} = LS*\left(R - RS + R^2S^2\right)^{1/2} * \exp\{(1/B)*\tan^{-1}[(2RS-1)/B]-(1/B)*\tan^{-1}[(2R-1)/B]\} \quad (5)$$

The parameters "R", "A", and "B" used in the above equations are defined as:

$$R = q/\left(k\sin^2\alpha\right) \quad (6)$$

$$A = (1-4R)^{1/2} \quad (7)$$

$$B = (4R-1)^{1/2} \quad (8)$$

Where:
k = hydraulic conductivity of the saturated drainage layer
L = drainage length
q = leachate infiltration rate
α = slope angle

There are several limitations to the McEnroe (1993) method:

a. The analytical solution requires "free draining conditions".
b. Hydraulic conductivity, leachate infiltration rate, and slope angle must be consistent along the entire drainage length.

4. Head-on-liner calculation –numerical approach

The McEnroe (1993) method is an analytical solution of the differential equation governing flow under free draining conditions. However, this differential equation can be integrated numerically to describe the saturated depth profile based on the boundary conditions. In other words, the governing differential equation can be solved numerically without preconditions such as the free-draining requirement.

The differential equation governing flow along a single drainage length is McEnroe (1993):

$$ky\left(\frac{dy}{dx} - \tan\alpha\right)\cos^2\alpha + rx = 0 \quad (9)$$

where,

k = hydraulic conductivity of the combined saturated drainage layer (cm/s)
y = saturated liquid depth over the liner (cm or in)
x = horizontal coordinate (cm or in)
r = leachate infiltration rate (cm/s)
α = slope angle

Equation 9 can be rearranged into finite difference form:

$$y_{i+1} = y_i + \left(\tan\alpha - \frac{rx_i}{ky_i\cos^2\alpha}\right)dx \qquad (10)$$
$$dx = x_{i+1} - x_i$$

Equation 10 can be numerically integrated using a pre-selected saturated liquid depth (y_L) at the low point of the drainage path, where "x" is equivalent to the maximum drainage length (Figure 1). The procedure will result in a full phreatic surface profile. From this profile the maximum head-on-liner value can be determined.

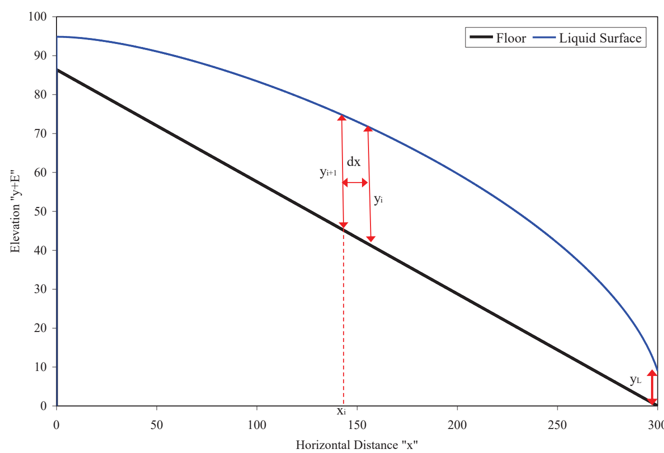


Figure 1. Example of Phreatic Leachate Surface

For a drainage system with multiple slopes (Figure 2), Equation 10 is arranged for each slope segment. Note that dimensions shown in Figure 2 are arbitrarily selected for illustrative purposes.

For slope segment 1: $0 \le x_i \le L_1$

$$y_{i+1} = y_i + \left(\tan\alpha_1 - \frac{r_1 x_i}{k_1 y_i \cos^2\alpha_1}\right)(x_{i+1} - x_i) \qquad \text{Eq. 11}$$

at $x = L_1$: y is equal to the value calculated from segment 2 at the same value of x.

For other slope segments (segment j where j >1): $\sum_{f=1}^{j-1} L_f \le x_i \le \sum_{f=1}^{j} L_f$:

$$y_{i+1} = y_i + \left(\tan\alpha_j - \frac{\sum_{f=1}^{j-1} r_f L_f + r_j \left(x_i - \sum_{f=1}^{j-1} L_f \right)}{k_j y_i \cos^2 \alpha_j} \right) (x_{i+1} - x_i) \qquad \text{Eq. 12}$$

at $x = \sum_{f=1}^{j-1} L_f$: y is equal to the value calculated from segment *j-1* at the same value of x.

Where

k_1 and k_j = combined hydraulic conductivity of the saturated drainage layer in slope segments 1 and *j*, respectively

r_1 and r_j = leachate infiltration rate to slope segments 1 and *j*, respectively

α_1 and α_j = slope angle of slope segments 1 and *j*, respectively

L_1 and L_j = total drainage length of slope segments 1 and *j*, respectively

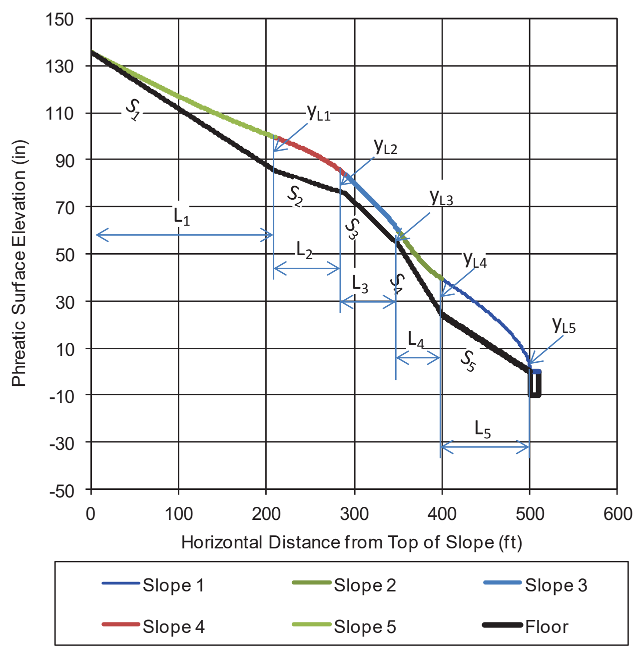


Figure 2. Example of Multiple Phreatic Leachate Surface

VERIFICATION OF THE NUMERICAL MODEL

A spreadsheet (in Microsoft Excel) was developed for the numerical integration of Equations 11 and 12. This spreadsheet included five slope segments. Multiple input parameters can be adjusted independently for each slope segment (Figure 3). To verify the accuracy of the numerical model results, the maximum values of leachate head on liner were calculated using a variety of input parameters and compared to the results estimated using the McEnroe (1993) method. Due to the limitations of the McEnroe (1993) method, constant values of leachate infiltration rate, slope angle, and permeability were applied to all slope segments in the numerical model and the free draining conditions were simulated using the numerical approach by applying a small leachate depth at the lowest point of the slope.

Test 1: Step distance for numerical integration

The maximum head-on-liner values were calculated using both the McEnroe (1993) Method and the numerical approach for six different permeability values (Table 1) and five leachate infiltration rates (Table 2). Four integration step distances (ranging from 0.2 to 3 inches) were used. In both tests, the results from the numerical approach are very close to the results calculated using McEnroe (1993) method. Therefore, the numerical approach was verified. Moreover, the incremental variation in numerical integration step distance (dx) did not significantly impact the results under the trial conditions. To minimize the file size and reduce computation time, an integration step distance of **0.5 inches** is recommended when using the numerical modeling approach.

Table 1. Sensitivity of Numerical Approach to Integration Step Distance for Various Permeability Values.

INPUT PARAMETERS					
		Infiltration Rate (gpad)	Drainage Length (ft)	Slope	Liquid Depth at Lowest Point (in)
Numerical Solution	Slope 1	3,000	140	2.00%	-
	Slope 2	3,000	235	2.00%	-
	Slope 3	3,000	200	2.00%	-
	Slope 4	3,000	75	2.00%	-
	Slope 5	3,000	350	2.00%	1.0
McEnroe 93 Method		3,000	1,000	2.00%	free drain

RESULTS					
Sand k (cm/s)	Ymax (in)				
	McEnroe 93	Numerical			
		dx=0.2 in	dx=0.5 in	dx=1.0 in	dx=3.0 in
0.01	112.35	102.98	112.35	112.38	112.56
0.05	30.64	30.61	30.65	30.65	30.67
0.10	16.63	16.63	16.64	16.64	16.65
0.50	3.70	3.70	3.70	3.70	3.70
1.00	1.89	1.89	1.89	1.89	1.89
5.00*	0.39	0.39	0.39	0.39	0.57

Table 2. Sensitivity of Numerical Approach to Integration Step Distance for Various Permeability Values and Leachate Infiltration Rates

INPUT PARAMETERS				
		Drainage Length (ft)	Slope	Liquid Depth at Lowest Point (in)
Numerical Solution	Slope 1	140	2.00%	-
	Slope 2	235	2.00%	-
	Slope 3	200	2.00%	-
	Slope 4	75	2.00%	-
	Slope 5	350	2.00%	1.0
McEnroe 93 Method		1,000	2.00%	free drain

RESULTS							
			Ymax (in)				
Infiltration Rate r (gpad)	Sand k (cm/s)	r/k*	McEnroe 93	Numerical			
				dx=0.2 in	dx=0.5 in	dx=1.0 in	dx=3.0 in
100	0.01	1.08E-05	6.02	6.02	6.02	6.02	6.03
500	0.01	5.41E-05	26.16	26.16	26.17	26.17	26.19
1,000	0.05	2.17E-05	11.50	11.50	11.50	11.50	11.51
3,000	0.05	6.50E-05	30.64	30.61	30.65	30.65	30.67
5,000	0.05	1.08E-04	47.18	47.18	47.18	47.19	47.23
5,000	0.10	5.41E-05	26.16	26.16	26.17	26.17	26.19
5,000	0.50	1.08E-05	6.02	6.02	6.02	6.02	6.03

Note:
* The ratio of infiltration rate and hydraulic conductivity of the drainage layer will control the maximum leachate depth on the liner (see Eq. 12).

Test 2: Starting leachate depth

In the numerical integration approach, a starting leachate depth at the lowest point (discharge point) of the slopes will be needed to initialize the integration. Four starting leachate depths were used in this test. The maximum head-on-liner values from both the McEnroe (1993) Method and the numerical solution were calculated for four different permeability values (Table 3). The results from the numerical approach are very close to the results calculated using the McEnroe (1993) method with one exception. Under the high permeability condition, the maximum head-on-linear was determined to be 3.70 inches using McEnroe 93 method. The results from numerical approach with a starting leachate depth of 1 inch or less were same as the value calculated from the McEnroe (1993) method. However, if the starting leachate depth was selected as 9 inches, the maximum leachate depth will occur at the starting point. This result indicates that the numerical integration approach can be used to determine the maximum head-on-liner when the "free draining" condition is not satisfied. In most cases, a starting leachate depth of **1.0 inch** can be used to represent the "free draining" condition. Note that under same conditions such as very high value of the ratio between infiltration rate and conductivity (high infiltration rate and low conductivity), the low starting leachate depth may result unstable solutions from the model. If it is occurred, user can adjust the staring value. A stable result can be verified by the trails and demonstrate that the the numerical solution is stable and not unduly affected by the starting leachate depth

Table 3. Sensitivity of Numerical Solution to the Starting Leachate Depth

<table>
<tr><th colspan="6">INPUT PARAMETERS</th></tr>
<tr><th colspan="2"></th><th>Infiltration Rate (gpad)</th><th>Drainage Length (ft)</th><th>Slope</th><th>Liquid Depth at Lowest Point (in)</th></tr>
<tr><th rowspan="5">Numerical Solution</th><td>Slope 1</td><td>3,000</td><td>140</td><td>2.00%</td><td>-</td></tr>
<tr><td>Slope 2</td><td>3,000</td><td>235</td><td>2.00%</td><td>-</td></tr>
<tr><td>Slope 3</td><td>3,000</td><td>200</td><td>2.00%</td><td>-</td></tr>
<tr><td>Slope 4</td><td>3,000</td><td>75</td><td>2.00%</td><td>-</td></tr>
<tr><td>Slope 5</td><td>3,000</td><td>350</td><td>2.00%</td><td>-</td></tr>
<tr><th colspan="2">McEnroe 93 Method</th><td>3,000</td><td>1,000</td><td>2.00%</td><td>free drain</td></tr>
<tr><th colspan="6">RESULTS</th></tr>
<tr><th rowspan="3">Sand
k
(cm/s)</th><th colspan="5">Ymax (in)</th></tr>
<tr><th rowspan="2">McEnroe 93</th><th colspan="4">Numerical dx=0.5 in</th></tr>
<tr><th>Yo=0.1 in</th><th>Yo=0.5 in</th><th>Yo=1 in</th><th>Yo=9 in</th></tr>
<tr><td>0.01</td><td>112.35</td><td>112.83</td><td>112.37</td><td>112.35</td><td>112.45</td></tr>
<tr><td>0.05</td><td>30.64</td><td>30.67</td><td>30.65</td><td>30.65</td><td>30.80</td></tr>
<tr><td>0.10</td><td>16.63</td><td>16.64</td><td>16.64</td><td>16.64</td><td>16.85</td></tr>
<tr><td>0.50</td><td>3.70</td><td>3.70</td><td>3.70</td><td>3.70</td><td>9.00</td></tr>
</table>

Test 3: Add geocomposite layer

To improve the drainage capacity of the drainage layer, a geocomposite layer can be added under the sand drainage layer. The combined hydraulic conductivity can be calculated using Equation 2. Two permeability values for sand with and without geocomposite layer were tested. The results from the numerical approach are very close to the values calculated using McEnroe 93 method (Table 4).

Table 4. Head-on-Liner Calculation with and without Geocomposite Layer

INPUT PARAMETERS					
		Infiltration Rate (gpad)	Drainage Length (ft)	Slope	Liquid Depth at Lowest Point (in)
Numerical Solution	Slope 1	3,000	70	2.00%	-
	Slope 2	3,000	117	2.00%	-
	Slope 3	3,000	100	2.00%	-
	Slope 4	3,000	38	2.00%	-
	Slope 5	3,000	175	2.00%	1.0
McEnroe 93 Method		3,000	500	2.00%	free drain

RESULTS					
				Ymax (in)	
Sand k (cm/s)	Geocomposite	Saturated Depth (inch)	Combined k (cm/s)	McEnroe 93	Numerical dx=0.5 in
0.0100	no	n/e	0.010	112.35	112.35
0.0100	yes	6.4	0.138	6.19	6.20
0.0010	no	n/e	0.001	267.21	266.93
0.0010	yes	7.8	0.108	7.78	7.79

n/e: no effect on the results.

DESIGN EXAMPLES USING THE NUMERICAL APPROACH

Six design examples are presented below to demonstrate the application of the numerical approach to the calculation of the maximum head-on-liner values. Descriptions and results for each example are summarized in Table 5. The detailed input parameters and phreatic surface plot for each example is presented in Figures 4 to 9, respectively. As demonstrated in Table 5, the numerical approach can accomodate multiple design conditions. In all design examples, the head-on-liner value cannot be estimated using the McEnroe (1993) method due to the complexity of the system.

Table 5. Summary of Design Examples

EXAMPLE	DESCRIPTION	Max Head-on-Liner (INCHES)
1	Single slope with different leachate infiltration rates for each slope segment	8.08
2	Five slopes with constant leachate infiltration rate for each slope segment	16.64
3	Five slopes with different leachate infiltration rates for each slope segment	8.08
4	Single slope with constant leachate infiltration rate; Increased flow capacity in bottom two slope segments by installing geocomposite layer	11.73
5	Five slopes with different leachate infiltration rates for each slope segment; High infiltration rate at top of the slope (representing open conditions); Increased flow capacity in bottom two slope segments by installing geocomposite layer	10.48
6	Single slope with constant leachate infiltration rate; Increased flow capacity by installing geocomposite layer in all slope segments; Applied different leachate depths for each slope segment; no trench at lowest point of the slope (no "free drain") and the leachate depth is 9 inches at lowest point (discharge point).	10.74

CONCLUSION

A numerical approach was developed to solve the differential equation governing flow in permeable media above an impermeable barrier presented by McEnroe (1993). This new approach was verified by analyzing multiple different boundary conditions and comparing the results to those calculated using analytical solutions developed by McEnroe (1993). Several design examples were provided to demonstrate the capability of this approach.

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Koerner, R. M. (2005). *Designing with Geosynthetics*, 5th ed. Upper Saddle River, NJ: Prentice Hall.

McEnroe, B. (1993) "Maximum Saturated Depth over Landfill Liner" Journal of Environmental Engineering, Val. 119, No.2, Page 262-270

Qian, X.D., Gray, D.H., and Koerner, R.M. (2004), "Estimation of Maximum Liquid Head over Landfill Barriers," Journal of Geotechnical and Geoenvironmental Engineering, ASCE, 130:5, 488-497

CALCULATION SHEET

Client: Landfill
Project: Head-on-Liner Calculation
Calculation: Head-on-liner calculation using numerical approach

Project No.:
Calculated By: XZ Date: 3/6/2012
Checked By: TY Date: 3/9/2012
Approved By: KF Date: 5/30/2012

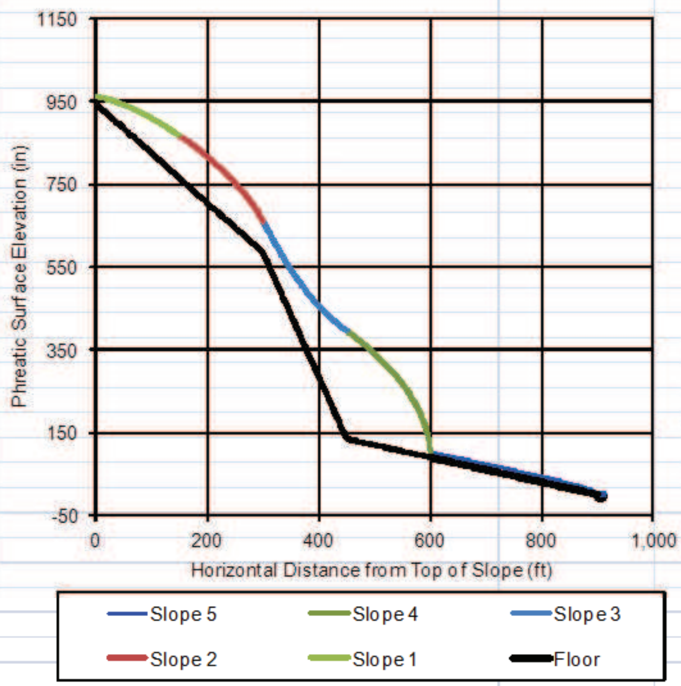
HEAD ON LINER CALCULATIONS

			SLOPE 5 Bottom	SLOPE 4	SLOPE 3	SLOPE 2	SLOPE 1 Top
Slope in the direction of flow	S	ft./ft.	2.50%	2.50%	25.00%	10.00%	10.00%
Slope angle	α	radians	0.0250	0.0250	0.2450	0.0997	0.0997
Flow length in the direction of flow	L	ft.	300	150	150	150	150
Rate of vertical inflow per unit area	r	gal/acre/day	1,000	2,000	3,000	4,000	4,000
Thickness of sand (or protective soil)	t_{sand}	in	10.3	10.0	12.0	12.0	12.0
		ft.	0.858	0.833	1.000	1.000	1.000
Permeability of sand (or protective soil)	K_{sand}	cm/sec	1.00E-03	1.00E-03	1.00E-03	1.00E-03	1.00E-03
Thickness of geonet	t_{geonet}	in.	0.200	0.000	0.000	0.000	0.000
		ft.	0.017	0.000	0.000	0.000	0.000
Geonet transmissivity	m2/s	m2/s	1.00E-03	1.00E-03	1.00E-03	1.00E-03	1.00E-03
Reduction Factor			9.11	9.11	9.11	9.11	9.11
Permeability of geonet	K_{geonet}	cm/sec	2.16E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00
Combined (apparent) permeability	K_{app}	**cm/sec**	8.25E-02	1.00E-03	1.00E-03	1.00E-03	1.00E-03
			[illegible]	1.00E-03	1.00E-03	1.00E-03	1.00E-03
Leachate Head at Discharge Point	h at L=0	in	1.0	12.56	260.82	76.52	102.66
Step Size	dL	in	0.5	0.5	0.5	0.5	0.5
Unit Width	W	ft	1	1	1	1	1

Maximum head on liner (McEnroe numerical) in each slope	in	13.31	260.82	260.82	112.22	102.66
Maximum head on liner location (McEnroe numerical) in each slope	ft	760.75	450.00	450.00	213.79	150.00
		9.04	62910.07	61910.79	10044.19	8218.51
Maximum head on liner (McEnroe numerical) in all slope	in	260.82				

Maximum head on liner (McEnroe 93 with free drain)	in	5.32
	R	0.02
Maximum head on liner (McEnroe 93 with free drain+Superposition)	in	6.32

	Average McEnroe 93	Average
k	2.82E-02	HOL
S	8.75%	(in)
q	2500	11.44



McEnroe 1993 "Maximum Saturated Depth over Landfill Liner"
Journal of Environmental Engineering

For Slope 1

$$y_{i+1} = y_i + \left(\tan \alpha_1 - \frac{r_1 x_i}{k_1 y_i \cos^2 \alpha_1} \right) (x_{i+1} - x_i)$$

Fore Slopes 2 - 5

$$y_{i+1} = y_i + \left(\tan \alpha_j - \frac{\sum_{f=1}^{j-1} r_f L_f + r_j \left(x_i - \sum_{f=1}^{j-1} L_f \right)}{k_j y_i \cos^2 \alpha_j} \right) (x_{i+1} - x_i)$$

Figure 3. Input and Output Sheet in the Head-on-Liner Calculation Spreadsheet

EXAMPLE 1: Variance in Leachate Infiltration Rates

INPUT PARAMETERS (dx=0.5 in)							
		Infiltration Rate (gpad)	Drainage Length (ft)	Slope	Combined k (cm/s)	Liquid Depth at Lowest Point (in)	Max Head-on-Liner (in)
Numerical Solution	Slope 1	3,000	140	2.00%	0.10	-	2.90
	Slope 2	2,000	235	2.00%	0.10	-	5.95
	Slope 3	1,000	200	2.00%	0.10	-	7.18
	Slope 4	500	75	2.00%	0.10	-	7.43
	Slope 5	500	350	2.00%	0.10	1.0	8.08
						OVERALL	8.08
McEnroe 93 Method		3,000	1,000	2.00%	0.10	free drain	16.63

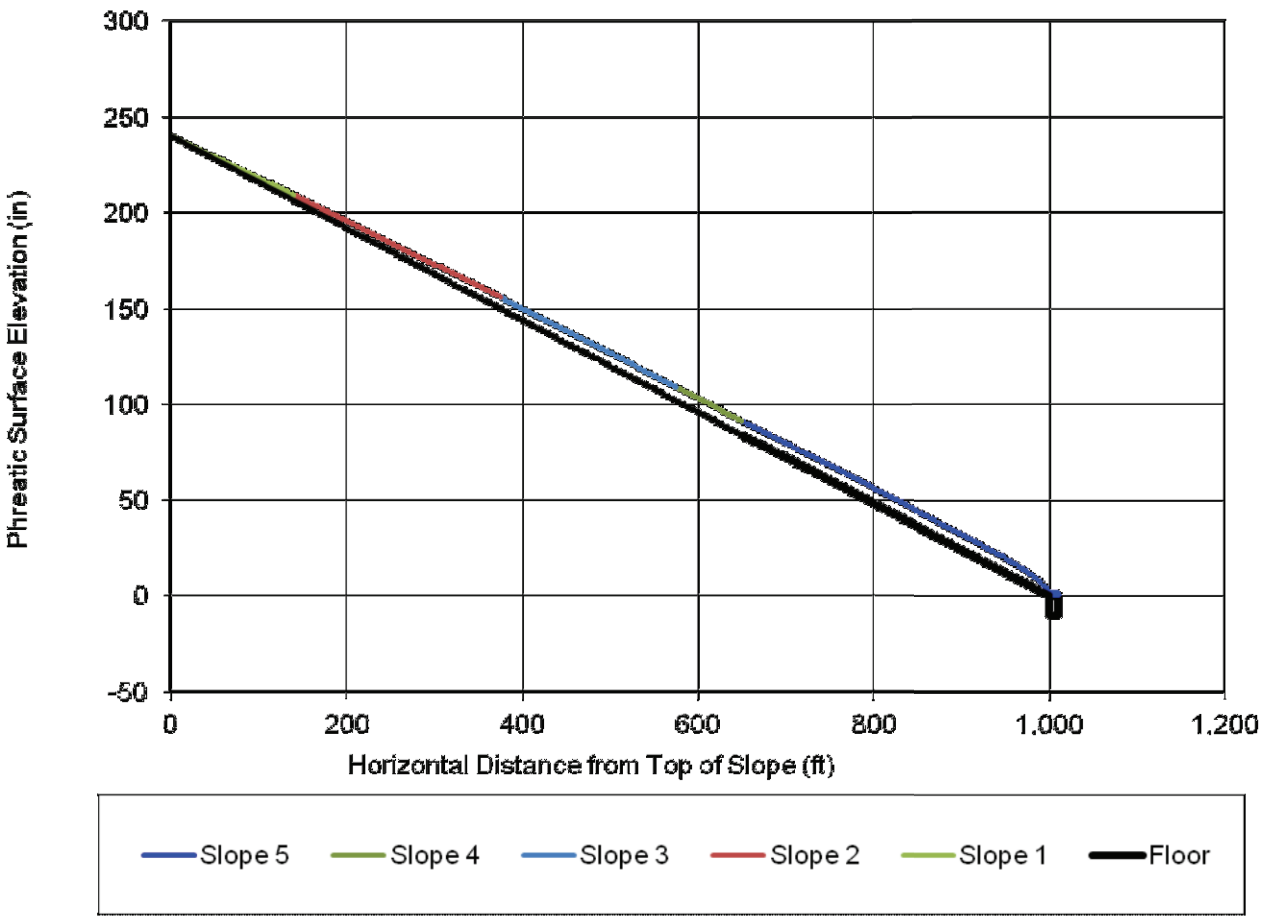


Figure 4. Design Example 1
Variance in Leachate Infiltration Rates

EXAMPLE 2: Variance in Slopes

INPUT PARAMETERS (dx=0.5 in)							
		Infiltration Rate (gpad)	Drainage Length (ft)	Slope	Combined k (cm/s)	Liquid Depth at Lowest Point (in)	Max Head-on-Liner (in)
Numerical Solution	Slope 1	3,000	140	20.00%	0.10	-	0.83
	Slope 2	3,000	235	12.00%	0.10	-	1.48
	Slope 3	3,000	200	10.00%	0.10	-	12.28
	Slope 4	3,000	75	2.00%	0.10	-	13.82
	Slope 5	3,000	350	2.00%	0.10	1.0	16.64
						OVERALL	16.64
McEnroe 93 Method		3,000	1,000	2.00%	0.10	free drain	16.63

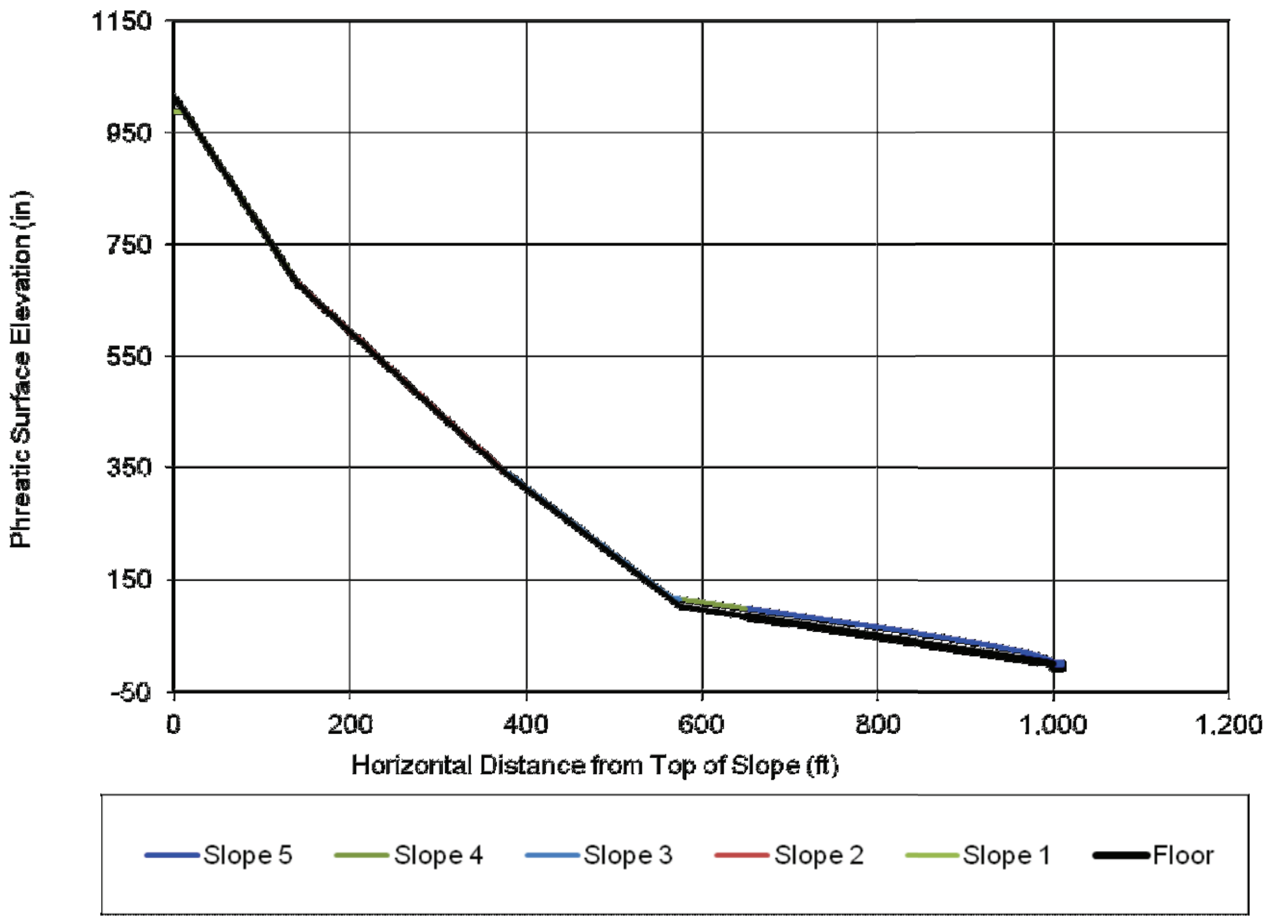


Figure 5. Design Example 2
Variance in Slopes

CALCULATION SHEET
Client: Landfill
Project: Head-on-Liner Calculation
Calculation: Head-on-liner calculation using numerical approach
Project No.:
Calculated By: XZ Date: 3/6/2012
Checked By: TY Date: 3/9/2012
Approved By: KF Date: 5/30/2012

EXAMPLE 3: Variance in Slopes and Leachate Infiltration Rates

INPUT PARAMETERS (dx=0.5 in)							
		Infiltration Rate (gpad)	Drainage Length (ft)	Slope	Combined k (cm/s)	Liquid Depth at Lowest Point (in)	Max Head-on-Liner (in)
Numerical Solution	Slope 1	3,000	140	20.00%	0.10	-	0.83
	Slope 2	2,000	235	12.00%	0.10	-	1.17
	Slope 3	1,000	200	10.00%	0.10	-	7.18
	Slope 4	500	75	2.00%	0.10	-	7.43
	Slope 5	500	350	2.00%	0.10	1.0	8.08
						OVERALL	8.08
McEnroe 93 Method		3,000	1,000	2.00%	0.10	free drain	16.63

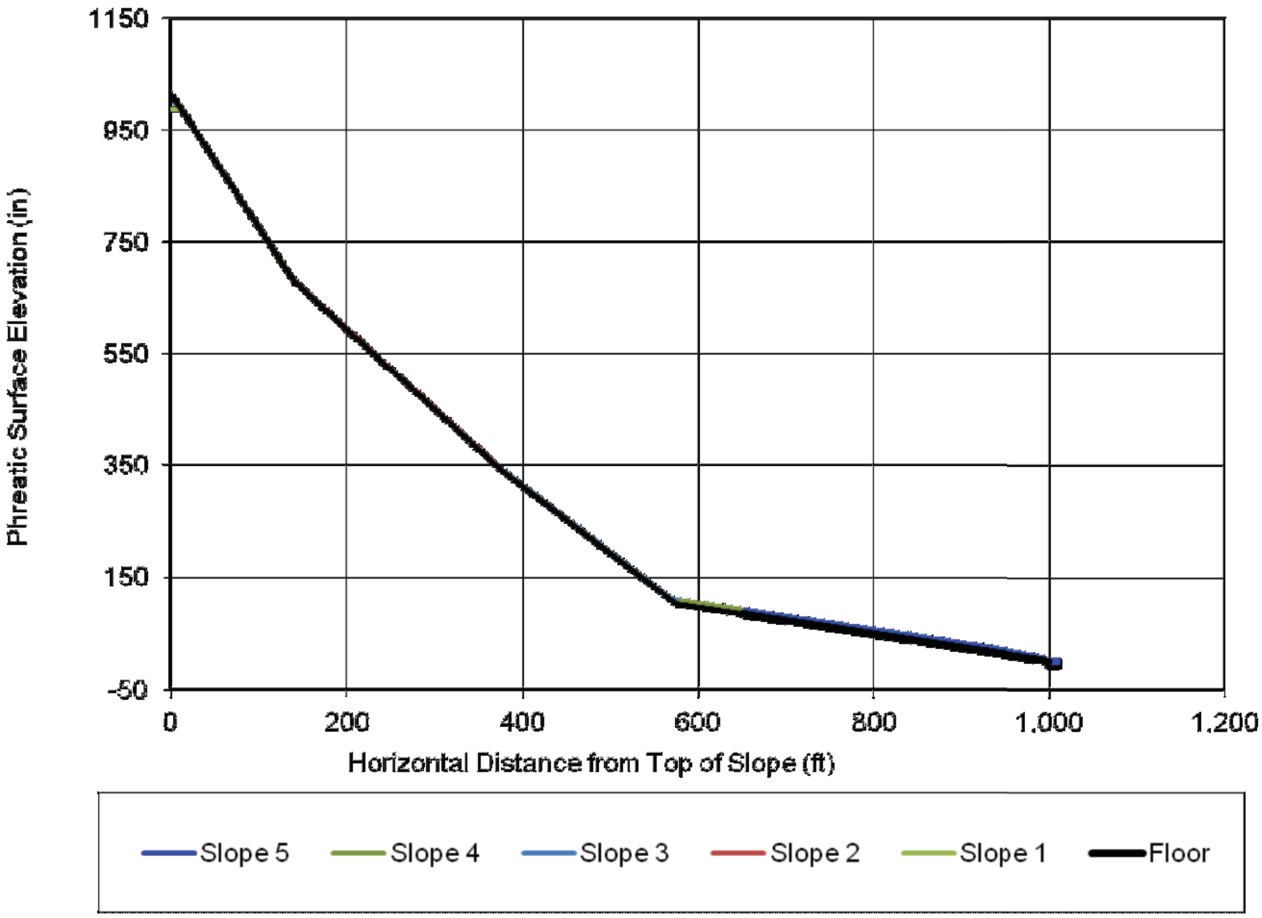


Figure 6. Design Example 3
Variance in Leachate Infiltration Rates and Slopes

CALCULATION SHEET

Client: Landfill		Project No.:	
Project: Head-on-Liner Calculation		Calculated By: XZ	Date: 3/6/2012
Calculation: Head-on-liner calculation using numerical approach		Checked By: TY	Date: 3/9/2012
		Approved By: KF	Date: 5/30/2012

EXAMPLE 4: Variance in Combined Permeabilitiy

INPUT PARAMETERS (dx=0.5 in)									
		Infiltration Rate (gpad)	Drainage Length (ft)	Slope	Combined k (cm/s)	Geonet Layer	Saturated Sand Thickness (in)	Liquid Depth at Lowest Point (in)	Max Head-on-Liner (in)
Numerical Solution	Slope 1	3,000	150	2.50%	0.050	no	-	-	5.30
	Slope 2	3,000	150	2.50%	0.050	no	-	-	10.35
	Slope 3	3,000	150	2.50%	0.050	no	-	-	11.73
	Slope 4	3,000	150	2.50%	0.184	yes	6.00	-	5.84
	Slope 5	3,000	150	2.50%	0.166	yes	7.00	1.0	6.49
								OVERALL	11.73
McEnroe 93 Method		3,000	750	2.50%	0.050			free drain	19.45

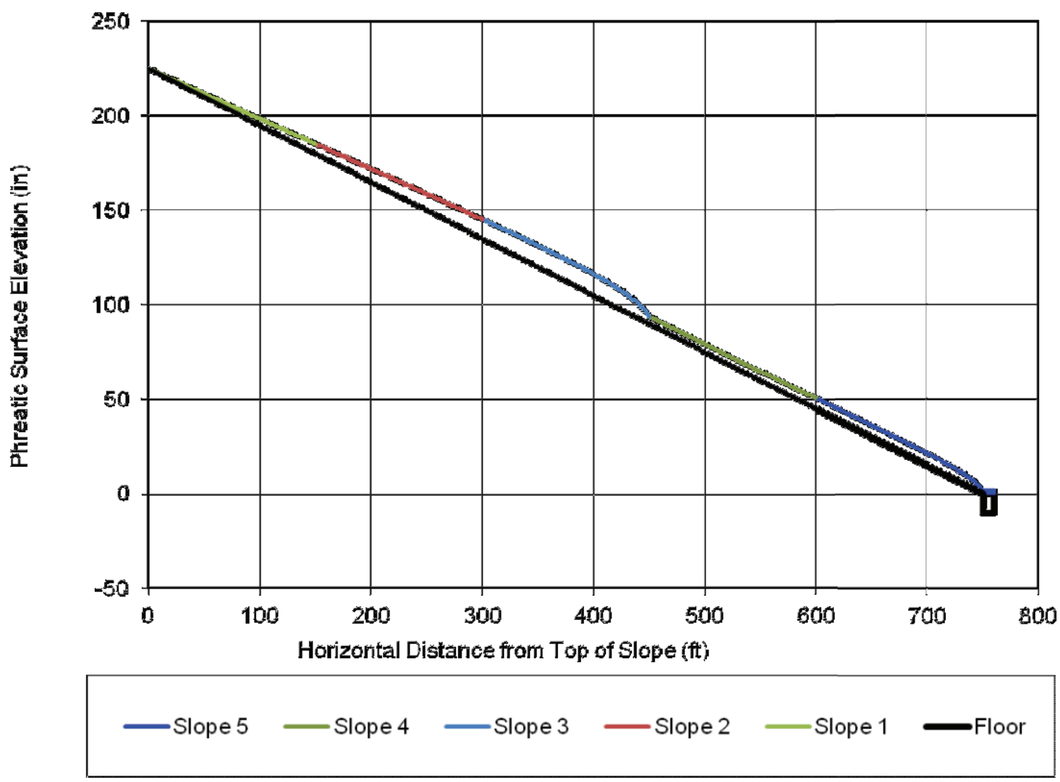


Figure 7. Design Example 4:
Variance in Combined Permeability (using geocomposite)

EXAMPLE 5: Variance in Slopes, Leachate Infiltration Rates, and Combined Permeability

INPUT PARAMETERS (dx=0.5 in)									
		Infiltration Rate (gpad)	Drainage Length (ft)	Slope	Combined k (cm/s)	Geonet Layer	Saturated Sand Thickness (in)	Liquid Depth at Lowest Point (in)	Max Head-on-Liner (in)
Numerical Solution	Slope 1	4,000	150	10.00%	0.020	no	-	-	4.03
	Slope 2	4,000	150	10.00%	0.020	no	-	-	7.39
	Slope 3	3,000	150	25.00%	0.020	no	-	-	8.48
	Slope 4	2,000	150	2.50%	0.103	yes	10.00	-	10.15
	Slope 5	500	300	2.50%	0.101	yes	10.30	1.0	10.48
								OVERALL	10.48
McEnroe 93 Method		4,000	900	2.50%	0.020			free drain	64.99

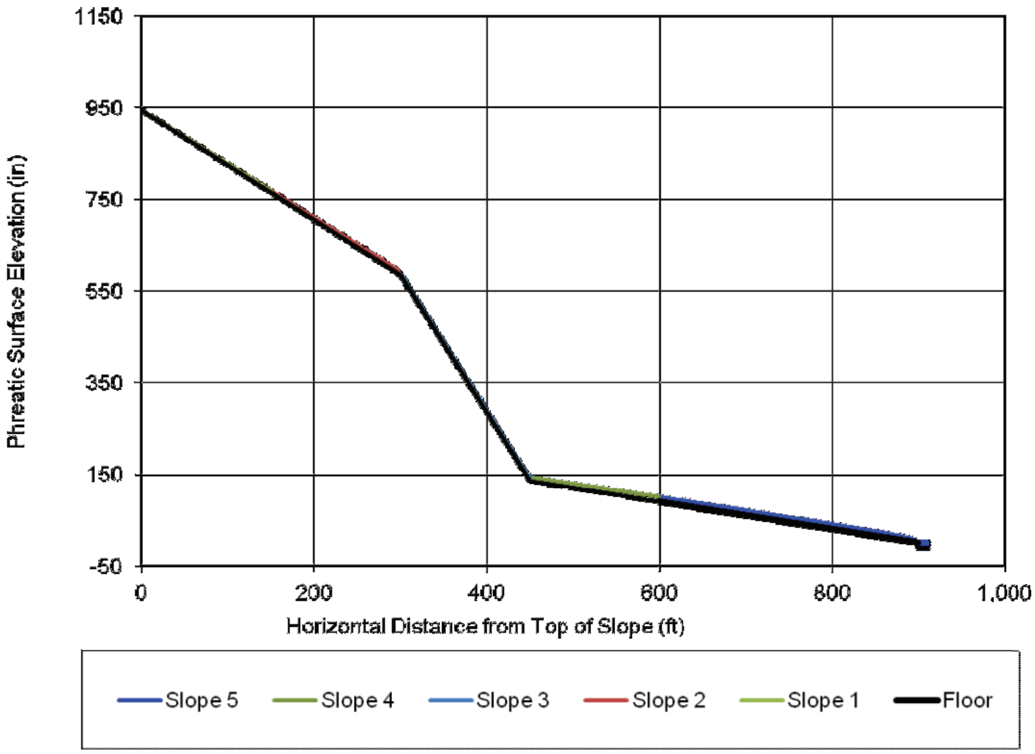


Figure 8. Design Example 5:
Variance in Slopes, Leachate Infiltration Rates, and Combined Permeability

EXAMPLE 6: No Resisted Trench (free drain condition is not satisfied)

INPUT PARAMETERS (dx=0.5 in)									
		Infiltration Rate (gpad)	Drainage Length (ft)	Slope	Combined k (cm/s)	Geonet Layer	Saturated Sand Thickness (in)	Liquid Depth at Lowest Point (in)	Max Head-on-Liner (in)
Numerical Solution	Slope 1	3,000	100	2.00%	0.661	yes	1.00	-	0.96
	Slope 2	3,000	100	2.00%	0.248	yes	3.79	-	3.94
	Slope 3	3,000	100	2.00%	0.120	yes	7.72	-	7.89
	Slope 4	3,000	100	2.00%	0.095	yes	10.22	-	10.39
	Slope 5	3,000	100	2.00%	0.078	yes	10.63	9.0	10.74
								OVERALL	10.74
McEnroe 93 Method		3,000	500	2.00%	0.078			free drain+superposition	19.39

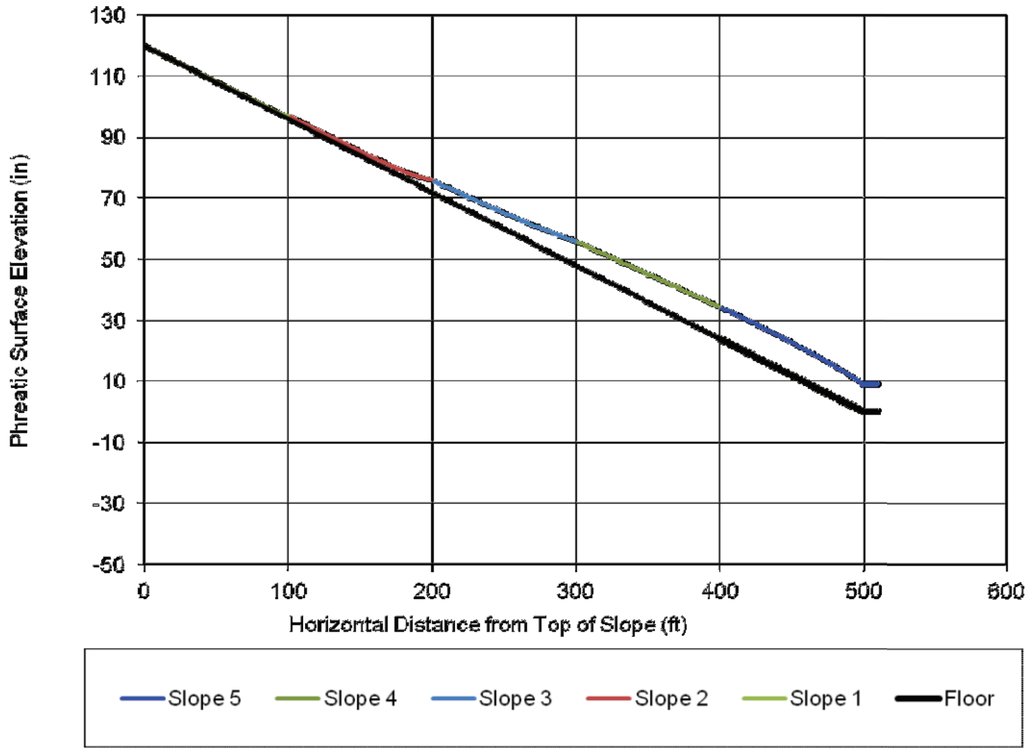


Figure 9. Design Example 6:
Free drain condition is not satisfied

A-2.4: NTH 2012, Leachate Generation Estimation and Head Calculation

NTH Consultants, Ltd.

CALCULATION PACKAGE COVER SHEET

PROJECT NO.: 13-060921-03	**CALCULATION PACKAGE NO.**
PROJECT NAME: WDI MC VI F/G Development	
PREPARED BY: David R. Lutz / CHRIS W. SUTTON	**DATE:** 2/15/11 / REV. 09/14/11
CHECKED BY: Robb B. Moore / CORINA M. TROMMER	**DATE:** 2/22/11 / Rev. 09/14/11
QC BY: Ibraheem Shumar	**DATE:** 3/4/11 - 9/6/11
SUBJECT: Leachate Generation Estimation and Head Calculation	

CONTENTS

	DESCRIPTION
PURPOSE:	To estimate the leachate generation with HELP model and the maximum leachate head on the bottom liner using the Mound Model for the proposed developement.
REFERENCES:	Qian, X. D. Gray, D.H., and Koerner, R.M. (2004). "Estimation of Maximum Liquid Head over Landfill Barrier." J. Geotechnical & Environmental Engineering 130(5), 488-497.
ASSUMPTIONS:	See Attached Calculation Package
INPUT DATA:	See Attached Calculation Package
RELEVANT CPs:	N/A
RESULTS:	See Attached Calculation Package

Job: WDI MC VI-F/G Development	Project No.:13-060921-03	Page 1
Subject: Leachate Generation Estimation and	By: DRL/CWS	Date: 2/15/11 / Rev. 9/14/11
Head Calculation	Chk. by: RBM/CMT	Date: 2/22/11 / Rev. 9/14/11

I. PURPOSE

The purpose of this calculation is to estimate leachate generation and maximum leachate head on the liner system. Leachate generation estimation is completed using the HELP model. The maximum leachate head on the bottom liner is estimate using the mound model for the proposed development.

II. BACKGROUND

The proposed liner system is a double composite liner that consists of the following layers from top down:

1) 12-inch sand layer (with $K \geq 1x10^{-2}$ cm/sec);
2) Double-sided geocomposite;
3) 80-mil textured HDPE primary liner;
4) 5-foot of compacted clay (with $K \leq 1x10^{-7}$ cm/sec);
5) Double-sided geocomposite;
6) 80-mil textured HDPE secondary liner; and
7) 3- foot of compacted clay (with $K \leq 1x10^{-7}$ cm/sec)

The final cover consists of the following components, from the top down: (1) 36-inch vegetative/protective soil layer; (2) Double-sided geocomposite drainage layer; (3) 40-mil HDPE geomembrane liner; and (4) Geosynthetic Clay Liner (GCL).

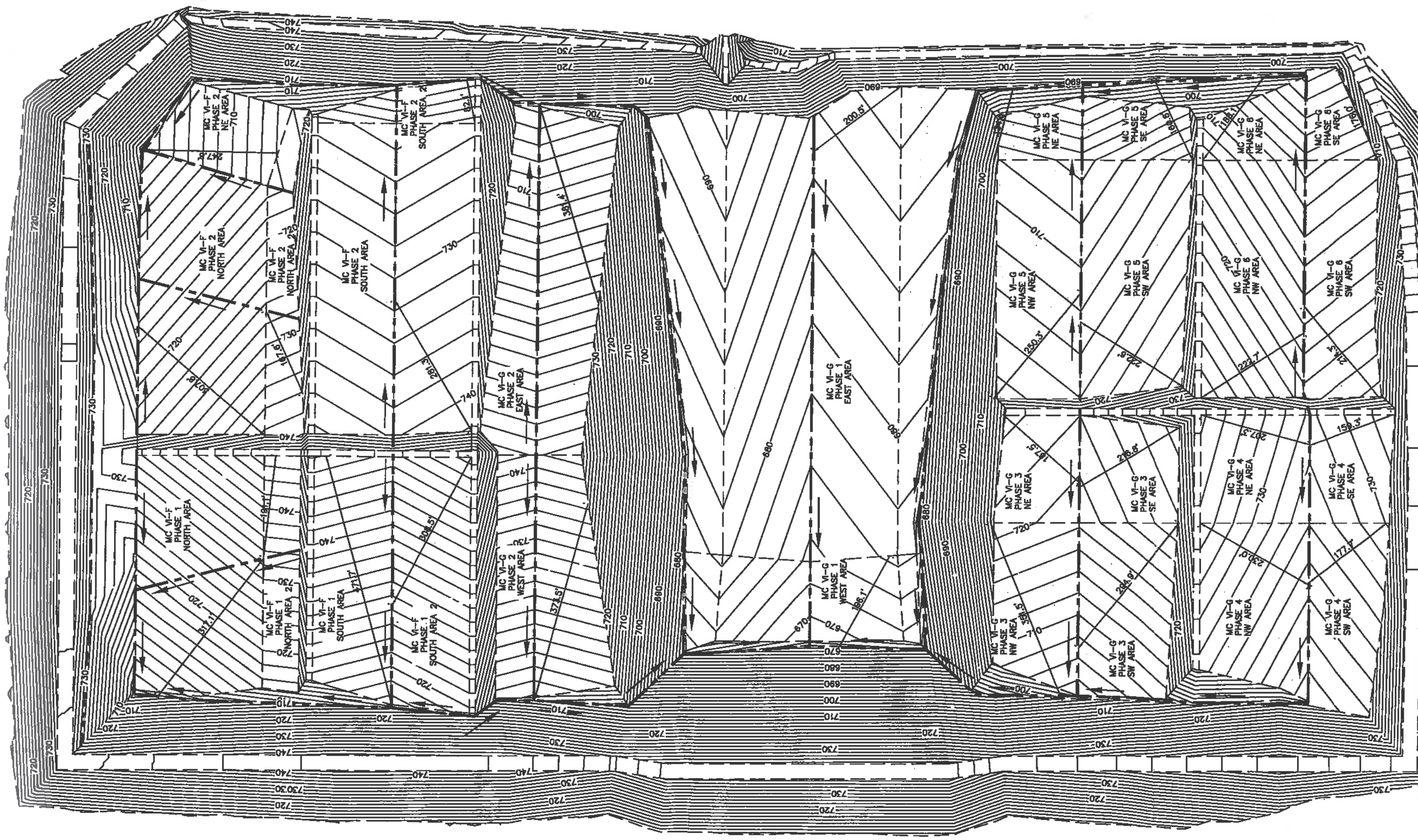
Per Part 111 Rule 299.9619, "a leachate head of no more than 30 centimeters (12 inches) on the liner" is allowed. The two-layer drainage system, including 1-foot sand layer and 200-mil geocomposite, will convey leachate to the leachate collection piping to maintain less than 1-foot of leachate on the liner.

III. APPROACH

The HELP (Hydrologic Evaluation of Landfill Performance) model was used to estimate the leachate generation collected in the leachate collection system. The proposed development area was divided into several areas as shown on page 2. The floor grades, pipe slope grades, and drainage lengths are summarized on Table 1. The leachate generation during filling operations were analyzed for each area. The most critical areas of the development were analyzed for leachate generation during the post-closure system.

The method presented in Reference 1 (Qian, et al, 2004) was used to perform the leachate head mounding calculation. The leachate generation rate estimated with HELP model for each area was used as vertical inflow rate in the mound calculation.

Pg 2
← N
1"=200'
Rev. 07/14/11



j:\13\13060921\library\6 f-g development\mound model 2-2011.dwg

TABLE 1
PROPOSED LANDFILL AREAS

Area	Base Grade (S_1) %	Pipe Grade (S_2) %	Composite Floor Slope %	Drainage Length Perpendicular to Slope (L) ft
MC VI-F Phase 1 South Area	2.0%	7.7%	8.0%	472
MC VI-F Phase 1 South Area 2	4.0%	7.7%	8.7%	309
MC VI-F Phase 1 North Area	9.6%	0.0%	9.6%	317
MC VI-F Phase 1 North Area 2	7.7%	0.0%	7.7%	191
MC VI-F Phase 2 South Area	2.0%	3.4%	3.9%	281
MC VI-F Phase 2 South Area 2	10.9%	0.0%	10.9%	82
MC VI-F Phase 2 NE Area	7.4%	0.0%	7.4%	248
MC VI-F Phase 2 North Area	7.9%	0.0%	7.9%	308
MC VI-F Phase 2 North Area 2	5.6%	0.0%	5.6%	168
MC VI-G Phase 1 East Area	2.0%	1.5%	2.5%	201
MC VI-G Phase 1 West Area	2.0%	3.3%	3.9%	196
MC VI-G Phase 2 West Area	2.0%	7.6%	7.9%	374
MC VI-G Phase 2 East Area	2.0%	6.6%	6.9%	381
MC VI-G Phase 3 NE Area	2.0%	1.8%	2.7%	188
MC VI-G Phase 3 NW Area	2.0%	5.0%	5.4%	352
MC VI-G Phase 3 SE Area	4.0%	1.8%	4.4%	217
MC VI-G Phase 3 SW Area	4.0%	5.0%	6.4%	295
MC VI-G Phase 4 NE Area	5.5%	1.5%	5.7%	207
MC VI-G Phase 4 NW Area	5.5%	3.1%	6.3%	230
MC VI-G Phase 4 SE Area	4.0%	1.5%	4.3%	177
MC VI-G Phase 4 SW Area	4.0%	3.1%	5.1%	208
MC VI-G Phase 5 NE Area	2.0%	8.0%	8.2%	166
MC VI-G Phase 5 NW Area	2.0%	2.5%	3.2%	250
MC VI-G Phase 5 SE Area	4.0%	8.0%	8.9%	191
MC VI-G Phase 5 SW Area	4.0%	2.5%	4.7%	223
MC VI-G Phase 6 NE Area	5.8%	6.1%	8.4%	180
MC VI-G Phase 6 NW Area	5.8%	3.7%	6.9%	223
MC VI-G Phase 6 SE Area	3.0%	6.1%	6.8%	166
MC VI-G Phase 6 SW Area	3.0%	3.7%	4.8%	258

IV. INPUT DATA

The landfill configurations used in HELP model for filling and post closure cases are listed on Table 2. The input data used in the HELP model is summarized in Table 3. In addition to the data shown on Table 1, a geocomposite with hydraulic conductivity of 12.2 mm/sec was used in the leachate head calculation.

TABLE 2
LANDFILL CONFIGURATION FOR HELP MODEL

		Filling Period	Post Closure
Layer No.	Materials	Thickness (in.)	
1	Vegetative/Protective Soil	N/A	36
2	Geocomposite	N/A	0.2
3	HDPE Geomembrane	N/A	0.04
4	GCL	N/A	0.36
5	Waste	120*	2,005/1,064**
6	Sand	12	12
7	Geocomposite	0.20	0.20
8	HDPE Geomembrane	0.08	0.08
9	Compacted Clay Liner	60	60
10	Geocomposite	0.20	0.20
11	HDPE Geomembrane	0.08	0.08
12	Compacted Clay Liner	36	36

* 10 feet of waste for peak leachate generation

** Maximum Final waste thickness for 4 % area. 2/3 of maximum final waste thickness for 25% area.

TABLE 3
INPUT DATA FOR HELP MODEL

	Filling Period	Post Closure Period
Total Area (acre)	1	1
Area Allowing Runoff (%)	0	100
Surface Slope (%)	See Table 1	4 / 25
Surface Slope Length (ft)	See Table 1	375/250
Surface Condition	Bare Ground	Fair Grass
Max. Leaf Area Index	0	4
Evaporative Zone Depth (in.)	6	20
Drainage Length in Liner (ft)	See Table 1	200/ 166
Drainage Slope in Liner (%)	See Table 1	2.5/ 8.2

V. CALCULATIONS

The results of the HELP model estimated leachate generation are summarized in Table 4. The HELP model output files are attached in Appendix A.

A spreadsheet program was used to estimate the leachate head on the liner. The spreadsheet was developed by Qian based on his methodology presented in Reference 1. The spreadsheets are attached in Appendix B. The estimated leachate head for each area is summarized in Table 5. As shown in Table 5, the maximum leachate head over the liner for each area in the development is less than the 30 cm (12 in.) requirement of R299.9619.

TABLE 4
HELP MODEL SUMMARY TABLE

Area	Average Peak Daily Leachate Generation (Active Filling) (in)	Average Annual Leachate Generation (Active Filling) (in)	Average Peak Daily Leachate Generation (Post Closure) (in)	Average Annual Leachate Generation (Post Closure) (in)
MC VI-F Phase 1 South Area	0.27	15.8	Insignificant	Insignificant
MC VI-F Phase 1 South Area 2	0.37			
MC VI-F Phase 1 North Area	0.37			
MC VI-F Phase 1 North Area 2	0.39			
MC VI-F Phase 2 South Area	0.23			
MC VI-F Phase 2 South Area 2	0.34			
MC VI-F Phase 2 NE Area	0.37			
MC VI-F Phase 2 North Area	0.36			
MC VI-F Phase 2 North Area 2	0.38			
MC VI-G Phase 1 East Area	0.21			
MC VI-G Phase 1 West Area	0.32			
MC VI-G Phase 2 West Area	0.33			
MC VI-G Phase 2 East Area	0.29			
MC VI-G Phase 3 NE Area	0.24			
MC VI-G Phase 3 NW Area	0.25			
MC VI-G Phase 3 SE Area	0.31			
MC VI-G Phase 3 SW Area	0.32			
MC VI-G Phase 4 NE Area	0.36			
MC VI-G Phase 4 NW Area	0.36			
MC VI-G Phase 4 SE Area	0.35			
MC VI-G Phase 4 SW Area	0.35			
MC VI-G Phase 5 NE Area	0.36			
MC VI-G Phase 5 NW Area	0.21			
MC VI-G Phase 5 SE Area	0.40			
MC VI-G Phase 5 SW Area	0.32			
MC VI-G Phase 6 NE Area	0.39			
MC VI-G Phase 6 NW Area	0.37			
MC VI-G Phase 6 SE Area	0.38			
MC VI-G Phase 6 SW Area	0.30			

TABLE 5 LEACHATE HEAD CALCULATION SUMMARY	
Area	Maximum Head On Liner (in)
MC VI-F Phase 1 South Area	6.6
MC VI-F Phase 1 South Area 2	1.3
MC VI-F Phase 1 North Area	0.8
MC VI-F Phase 1 North Area 2	0.3
MC VI-F Phase 2 South Area	4.8
MC VI-F Phase 2 South Area 2	0.1
MC VI-F Phase 2 NE Area	0.8
MC VI-F Phase 2 North Area	2.2
MC VI-F Phase 2 North Area 2	0.5
MC VI-G Phase 1 East Area	3.8
MC VI-G Phase 1 West Area	3.8
MC VI-G Phase 2 West Area	5.0
MC VI-G Phase 2 East Area	5.7
MC VI-G Phase 3 NE Area	3.5
MC VI-G Phase 3 NW Area	5.7
MC VI-G Phase 3 SE Area	3.2
MC VI-G Phase 3 SW Area	2.9
MC VI-G Phase 4 NE Area	1.2
MC VI-G Phase 4 NW Area	1.3
MC VI-G Phase 4 SE Area	2.0
MC VI-G Phase 4 SW Area	2.2
MC VI-G Phase 5 NE Area	0.2
MC VI-G Phase 5 NW Area	4.7
MC VI-G Phase 5 SE Area	0.3
MC VI-G Phase 5 SW Area	3.2
MC VI-G Phase 6 NE Area	0.3
MC VI-G Phase 6 NW Area	0.7
MC VI-G Phase 6 SE Area	0.3
MC VI-G Phase 6 SW Area	4.6

Based on R299.9619, a minimum transmissivity (T) of $3.0 * 10^{-5}$ m^2/s for geonet should be utilized. Typical geocomposite has a thickness of 200 mils or 5 mm. As stated earlier, the geocomposite hydraulic conductivity for the proposed development is 12.2 mm/sec. The ultimate hydraulic transmissivity is calculated as:

ultimate hydraulic transmissivity = hydraulic conductivity * thickness

$$= 12.2 \text{ mm/sec} * 5 \text{ mm} * (1 \text{ m}^2 / 1 * 10^6 \text{ mm}^2) = 6.1 * 10^{-5} \text{ m}^2/\text{s} > 3.0 * 10^{-5} \text{ m}^2/\text{s} \ \underline{\text{OK}}.$$

By using the factors of safety of 1.75, 1.5, and 1.5 to account for creep, chemical clogging, and biological clogging, respectively (recommended by Koerner in his book, Design with Geosynthetics), the measured hydraulic transmissivity can be calculated as

$$\text{allowable hydraulic transmissivity} = 6.1*10^{-5}*(1.75*1.5*1.5) = 2.4*10^{-4} \text{ m}^2/\text{s}.$$

Based on NTH's experience with the use of the HELP model to simulate landfill leachate generation, the maximum leachate generation occurs at an approximate waste height of 10 feet. The higher the waste height, the lower the leachate generation rate. Thus, assuming 10 feet of waste and a unit weight of 111 pcf for the waste, the overburden pressure on the drainage layer is:

(111 pcf)(10 ft) = 1,110 psf

VI. RESULTS & CONCLUSIONS

The results of the HELP modeling indicates that for any area of the development, the maximum estimated peak daily and average annual leachate generation rates during active filling is 0.40 in/day and 15.81 in., respectively. Both the peak daily and average annual leachate generation rate during post closure is estimated to be insignificant. The leachate head calculation estimated that the leachate head on the liner is less than 12 inches for each area of the development. Using a 200 mil geocomposite, with a transmissivity greater than or equal to $2.4*10^{-4}$ m^2/s under a gradient of 2.5% and under an overburden pressure of 1,110 psf.