APPENDIX VI-H.3

LANDFILL STABILITY ANALYSIS (STATIC)

Geosyntec consultants

COMPUTATION COVER SHEET

Client: DRPI **Project:** DRPI Landfill Expansion **Project #:** ME1571 **Task #:** 1

TITLE OF COMPUTATIONS SLOPE STABILITY EVALUATION

COMPUTATIONS BY:
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Printed Name and Title: R. David Espinoza, Senior Principal
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APPROVAL NOTES:

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LANDFILL STABILITY ANALYSIS

1. INTRODUCTION

The purpose of this analysis is to evaluate the overall stability of the proposed expansion of the Delaware Recyclable Products, Inc. (DRPI) active landfill facility (DRPI Landfill) located in New Castle County, Delaware. The proposed expansion consists of an increase in the maximum landfill elevation from 130 ft-msl previously permitted in 2005 to 190 ft-msl.

2. SECTIONS FOR ANALYSIS

A plan view of the DRPI Landfill, with the proposed final grading plan, is shown in Figure 1. Four representative cross sections were analyzed.

The selected cross sections were chosen to take into consideration unique features of the landfill geometry (e.g., maximum height, bench locations), foundation stratigraphy (thickness of existing waste, and thickness of the underlying clay and sand layers). Figures 2a through 2d show the geometric and stratigraphic characteristics of each of the sections analyzed.

3. STABILITY CRITERIA

According to the technical manual published by the USEPA entitled, *"Solid Waste Disposal Facility Criteria"* [USEPA, 1993], the minimum recommended factor of safety against slope stability failure for permanent conditions is 1.5. For temporary conditions, the minimum recommended factor of safety against static slope stability failure is 1.25. In general, waste placement is a temporary condition; therefore, the recommended minimum factor of safety during waste placement is 1.25. For long-term conditions, after waste placement has ended, a minimum factor of safety of 1.5 is recommended.

4. LOADING CONDITIONS

4.1 Static Loads

The main static load applied to the foundation soils is the gravity load exerted by the waste placement. This load is estimated based on an average unit weight of waste of 70 pcf. This value is consistent with previous experience [Geosyntec, 1996 and 2000].

4.2 Seismic Loads

According to Section 258.14 of RCRA Subtitle D regulations, 40 CFR Part 258, a seismic impact zone is identified as an area with a ten percent or greater probability that the maximum horizontal acceleration (MHA) in lithified earth material, expressed as a percentage of the earth's gravitational pull (g) will exceed 0.10g in 250 years.

The MHA in bedrock with two percent probability of exceedance in 50 years for the DRPI Landfill site is estimated to be 0.0986g, based on the 2014 USGS Conterminous U.S. unified hazard tool (Figure 3) estimated for the site coordinates (Latitude 39.7145 and Longitude -75.572) [USGS, 2014]. The peak acceleration with two percent probability of exceedance in 50 years is approximately equivalent to the one with ten percent probability of exceedance in 250 years. From the comparison, the site is considered not located within a seismic impact zone. Therefore, seismic loading is not considered.

5. STRATIGRAPHY AND MATERIAL PARAMETERS

5.1 General

The stratigraphy and material parameters assumed for the analysis are consistent with the previous analysis conducted for the 2004 permit application [Geosyntec, 2004]. The basis for selecting the stratigraphy and material parameters is discussed below.

As shown in the cross sections (Figures 2a through 2d), the stratigraphy at the DRPI Landfill includes an upper and a lower clay layer with interbedded sand. These layers are referenced in this calculation as Clay Layer 1, Sand Layer 1, Clay Layer 2, and Sand Layer 2. Generally, stratigraphy and groundwater surface elevations were based on a stratigraphic profile and a groundwater contour map prepared by Blazosky [2004]. The bottom of the existing waste in Cells 1 through 5 is assumed to be approximately elevation 10 ft-msl, which is based upon record drawings for Cells 4 and 5 [GZA, 1998]. The soil parameters were selected based on the Soil Characterization Summary presented as Appendix VI-C to the Engineering Report. Undrained shear strength of the clays is expected to increase due to consolidation under the weight of the overlying waste. The estimation of shear strength increase is presented in Attachment B of this calculation package.

The remainder of this section addresses the parameters of these soil types used in the stability analysis. Table 1 summarizes the soil parameters used in the analysis.

Table 1 – Soil Parameters used for Analysis

Material (1)	Condition	Cohesion (psf)	Friction Angle (°)	Unit Weight (pcf)	Unit Weight Saturated (pcf)
Waste	Intermediate / Long-Term	0	33	70	-
Textured GM/GT(2)	Intermediate / Long-Term	0	30	110	110
Smooth GM/GT(2)	Intermediate / Long-Term	0	9		
Textured GM/GCL(2)	Intermediate / Long-Term	0	18.9		
Clay Layer 1	Intermediate	Varies(3)	0	120	120
	Long-Term	0	21		
Sand Layer 1	Intermediate / Long-Term	0	30	125	125
Clay Layer 2	Intermediate	Varies(3)	0	120	120
	Long-Term	0	21		
Sand Layer 2	Intermediate / Long-Term	0	30	125	125

Notes:
(1) Label used for SLIDE analysis (see Attachment C).
(2) GM = Geomembrane, GT = Geotextile; and GCL = Geosynthetic Clay Liner
(3) See Attachment B for estimation of clay undrained shear strength.

5.2 Liner System

The liner system details for Phases 4, 5 and 6 are shown in Attachment A to this calculation package. The impact of the existing and proposed liner system on global stability is evaluated in this calculation by modeling the liner system as a two foot thick layer with an appropriate interface friction angle. As presented in Attachment A, friction angle values were chosen to represent the weakest interface in the liner systems. Based on the interface shear strength test results provided by TRI/Environmental, Inc. dated 6 July 2007 for Phase VI construction, the most critical interface on the base liner system, which includes the structural fill/60-mil HDPE textured geomembrane/GCL/double-sided geomembrane, has a peak interface friction angle of 18.9° (see Attachment A). The weakest layer in the liner system for other Phases is the interface between geomembrane and geocomposite layer, which is assumed to have a friction angle of 9 degrees for smooth geomembrane on cell base, and 30 degrees for textured geomembrane on cell sideslope(consistent with previous Geosyntec analysis in 2005). The friction angle for each of these cases is included in Table 1.

5.3 Clay Layers 1 and 2

Clay Layers 1 and 2 are encountered above Sand Layers 1 and 2, respectively. Clay Layer 1 consists of a stiff clay to silty clay often described as brownish-gray or gray, while Clay Layer 2 consists of a stiff pinkish red clay [Blazosky, 2003]. For short-term conditions the clay layers are assumed to be undrained. The undrained shear strength of Clay Layer 1 was found to be 1,000 psf, while Clay Layer 2 was found to be 1,400 psf [GZA, 1998], based on unconsolidated undrained triaxial test results. An analysis was conducted to evaluate the undrained shear strength of clay due to the consolidation under the weight of overlying waste. This analysis is presented in Attachment B.

For stability analyses at the end of waste placement both clay layers are assumed to have drained shear strength and are modeled with an effective friction angle of 21 degrees based on previous experience with soils in this area [Geosyntec, 2003]. Both layers are assumed to have an average unit weight of 120 pcf.

5.4 Sand Layers 1 and 2

Sand Layers 1 which is encountered under Clay Layer 1 consists of varicolored, often found to be gray, red, yellow or white, fine silty-sand and sand. Sand Layer 2, encountered under Clay Layer 2, is a medium to fine-grained layer consisting of white, orange and light-brown sands [Blazosky, 2003]. An effective friction angle of 30 degrees and no cohesion are used to model the strength of these layers. The unit weight for these layers was assumed to be 125 pcf. These parameters are based on previous reports [GZA, 1998].

5.5 Waste

Existing and future waste is assumed to have a unit weight of 70 pcf with a friction angle of 33 degrees, based on Geosyntec's past experience.

6. METHOD OF ANALYSIS

The stability of the selected cross sections was evaluated based on limit equilibrium theory using the methods of slices. The computer program SLIDE [Rocscience, 2012] was used to perform the analyses. SLIDE is a 2D slope stability program for evaluating the factor of safety of circular and non-circular (also referred to as block) failure surfaces in soils. The procedure consisted of analyzing numerous potential failure surfaces to find the critical failure surface that renders the minimum factor of safety for the slope. Circular and non-circular failure surfaces can be analyzed. For circular and non-circular failure surfaces, Spencer's

method [Spencer, 1967] is used. In Spencer's method, both force and moment equilibrium are satisfied in each slice and the slope of the inter-slice forces is assumed constant and parallel to each other.

For the slope stability cases analyzed in this calculation, both non-circular and circular failure surfaces were considered. During the analysis, the search boundaries were varied to ensure that the most critical surface was captured during the search. Numerous potential failure surfaces were analyzed to find the critical failure surface that results in the minimum factor of safety for the slope.

Parameters for the circular search for Spencer's method include using 25 slices, a tolerance of 0.005, and a maximum of 50 iterations. These parameters were selected such that the analysis was performed accurately and expediently. For the circular search, a search grid with 25 horizontal increments and 25 vertical increments was used to find the minimum factor of safety. SLIDE provides both the minimum factor of safety and a contour map of the calculated factors of safety. In some cases, when the contour lines that contain the minimum factor of safety were not fully closed, the search grid was expanded horizontally or vertically and the analysis performed again. This iterative process ensured that a global minimum, and not a local minimum, factor of safety was calculated.

A minimum of two search windows were used to analyze non-circular failures for each section. These windows were iteratively placed to obtain the minimum factor of safety. Parameters used in Spencer's method included using 25 slices, a tolerance of 0.005, and a maximum of 50 iterations.

In order to evaluate the influence of a weak interface shear strength (i.e., the liner system), a non-circular search was performed using a polyline that goes through the existing or proposed liner for the long-term intermediate conditions. Using this method, the impact of the liner systems on the global stability was examined. In addition, for this analysis, sections were extended horizontally in both directions to ensure that the critical failure surface was not constrained by geometry. For example, if a critical failure surface was found near the right or left boundary, the boundary was extended, and the analysis performed again.

7. STABILITY RESULTS

Stability analysis results for intermediate and long-term conditions under static conditions for the analyzed cross sections are presented in Attachment C. A summary of the results is presented in Tables 2 and 3. As shown in the tables, the calculated factors of safety are greater than the recommended minimum value of 1.25 (short-term conditions) and 1.5 (long-term conditions).

Table 2 – Factors of Safety for Short-term Conditions

Section	Type of Analysis	Factor of Safety
A-A	Circular	1.61
	Block	1.51
B-B	Circular	1.34
	Block	1.43
C-C	Circular	1.49
	Block	1.35
D-D	Circular	1.44
	Block	1.26

Table 3 – Factors of Safety for Long-term Conditions

Section	Type of Analysis	Factor of Safety
A-A	Circular	2.27
	Block	2.02
	Along Liner	1.92
B-B	Circular	1.70
	Block	2.35
	Along Liner	1.73
C-C	Circular	1.83
	Block	2.33
	Along Liner	1.64
D-D	Circular	1.66
	Block	2.12
	Along Liner	2.24

8. CONCLUSION

The analysis presented in this calculation package shows that waste placement can be performed while maintaining an appropriate factor of safety throughout the entire life of the

landfill. Specifically, the calculated factors of safety are greater than the recommended minimum values.

9. REFERENCES

Blazosky Associates (2002). *"DRPI Industrial Waste Landfill Revised Site Wide Subsurface Monitoring Installation Summary"*, prepared for Waste Management, June 1999, revised June 2002.

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Rocscience, (2012). *"Slide (Version 6.0): A 2D Slope Stability Analysis for Soil and Rock Slopes"*, Rocscience, Toronto, Canada.

Spencer (1967). *"A method of analysis of the stability of embankments assuming parallel inter-slice forces",* Geotechnique, London, England, 17(1), 11-26.

USEPA (1993). *"Solid Waste Disposal Facility Criteria"*, Document No. EPA 530-R-93-017, United States Environmental Protection Agency, November, 1993, Revised April 13, 1998.

USGS (2008). "2008 United States National Seismic Hazard Maps", United States Geological Survey, http://earthquake.usgs.gov/research/hazmaps/products_data/2008/

FIGURES

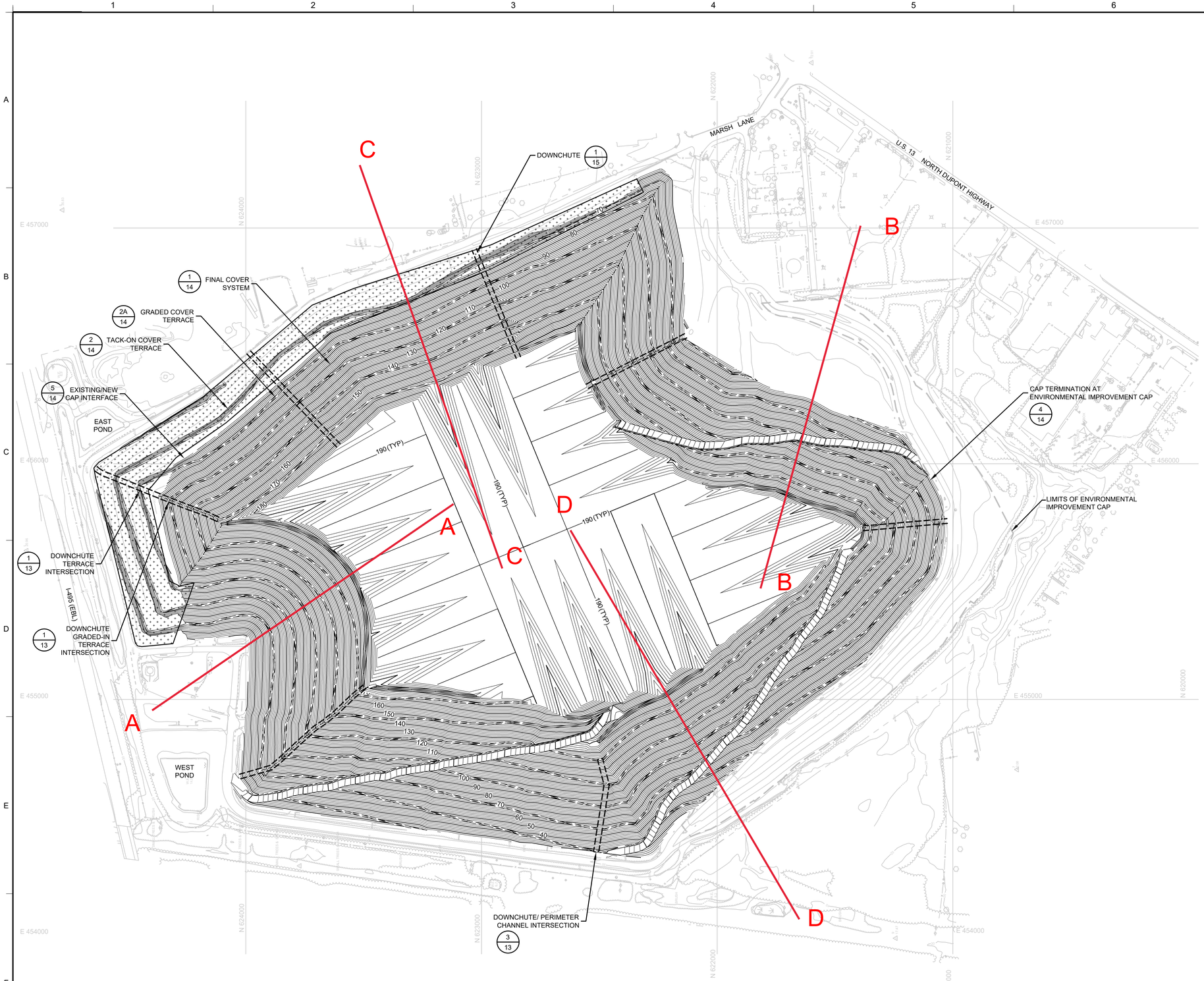
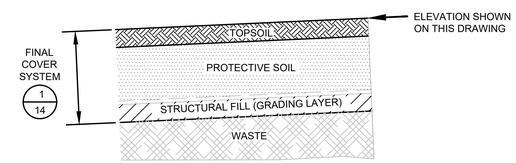


Figure 1. Final Cover Grading Plan and Locatons of Analyzed Cross Sections

NORTH

LEGEND

- PROPERTY BOUNDARY
- EXISTING GRADE CONTOUR (FEET–MSL)
- FINAL COVER GRADE CONTOUR (FEET–MSL)
- EXISTING TREELINE
- DRAINAGE CHANNEL 2/20
- EXISTING ROAD
- ACCESS ROAD
- EXISTING BUILDING
- DOWNCHUTE 1/18
- EROSION MAT LINER (SEE NOTE 8)
- EXISTING CLAY CAP FINAL COVER (SEE NOTE 9)



DETAIL
GRADING REFERENCE KEY
NOT TO SCALE

NOTES:

1. SEE DRAWING 2 FOR GENERAL NOTES.
2. INSTALL FLARED END SECTION AT CULVERT INLET AND OUTLET AS IDENTIFIED.
3. ALL REINFORCED CONCRETE PIPE TO BE CLASS III. INSTALL STRAIGHT HEADWALL AND ENDWALL AT CULVERT INLET AND OUTLET AS IDENTIFIED.
4. TOTAL PROJECT DISTURBED AREA IS APPROXIMATELY 172.2 ACRES.
5. THE DNREC SEDIMENT AND STORMWATER MANAGEMENT PROGRAM, MUST BE NOTIFIED IN WRITING FIVE (5) DAYS PRIOR TO COMMENCING WITH CONSTRUCTION. FAILURE TO DO SO CONSTITUTES A VIOLATION OF THE APPROVED SEDIMENT AND STORMWATER MANAGEMENT PLAN.
6. REVIEW AND/OR APPROVAL OF THE SEDIMENT AND STORMWATER MANAGEMENT PLAN SHALL NOT RELIEVE THE CONTRACTOR FROM ITS RESPONSIBILITY FOR COMPLIANCE WITH THE REQUIREMENTS OF THE SEDIMENT AND STORMWATER REGULATIONS, NOR SHALL IT RELIEVE THE CONTRACTOR FROM ERRORS OR OMISSIONS IN THE APPROVED PLAN.
7. IF THE APPROVED PLAN NEEDS TO BE MODIFIED, ADDITIONAL SEDIMENT AND STORMWATER CONTROL MEASURES MAY BE REQUIRED AS DEEMED NECESSARY BY DNREC.
8. FOLLOWING ANY SOIL DISTURBANCE, PERMANENT OR TEMPORARY STABILIZATION SHALL BE COMPLETED IN ACCORDANCE WITH THE EROSION AND SEDIMENT CONTROL PLAN AS TO THE SURFACE OF ALL PERIMETER SEDIMENT CONTROLS, SOIL STOCKPILES, AND ALL OTHER DISTURBED OR GRADED AREAS ON THE PROJECT SITE.
9. THE EXTENT OF THE EXISTING CLAY COVER IS PROVIDED IN A DRAWING ENTITLED "CAPPING INVENTORY" BY VANDEMARK AND LYNCH, DATED 31 AUGUST 1999.
10. SEE DRAWINGS 19 AND 20 FOR LANDFILL ACCESS ROAD CHANNEL PROFILES.
11. SEE DRAWING 20 FOR DRAINAGE CHANNEL SCHEDULE AND DRAWINGS 22 AND 23 FOR PERIMETER CHANNEL PROFILES.

200 100 0 200 400
SCALE: 1" = 200'

REV	DATE	DESCRIPTION	DRN	APP

Geosyntec consultants
10221 WINCOPIN CIRCLE, 4TH FLOOR
COLUMBIA, MARYLAND 21044 USA
PHONE: 410.381.4333

WASTE MANAGEMENT
DELAWARE RECYCLABLE PRODUCTS, INC.
246 MARSH LANE
NEW CASTLE, DELAWARE 19720
PHONE: 302.655.1360

TITLE: FINAL COVER SYSTEM AND STORMWATER MANAGEMENT SYSTEM PLAN

PROJECT: VERTICAL EXPANSION DESIGN MODIFICATION APPLICATION

SITE: DRPI INDUSTRIAL WASTE LANDFILL NEW CASTLE, DELAWARE

THIS DRAWING MAY NOT BE ISSUED FOR PROJECT TENDER OR CONSTRUCTION, UNLESS SEALED.

SIGNATURE

DATE

DESIGN BY:	KSS	DATE:	JUNE 2018
DRAWN BY:	MF/TJ/BF	PROJECT NO.:	ME1571
CHECKED BY:	KSS	FILE:	1571-013
REVIEWED BY:	CHP	DRAWING NO.:	
APPROVED BY:	CHP		13 OF 28

PERMIT DRAWING

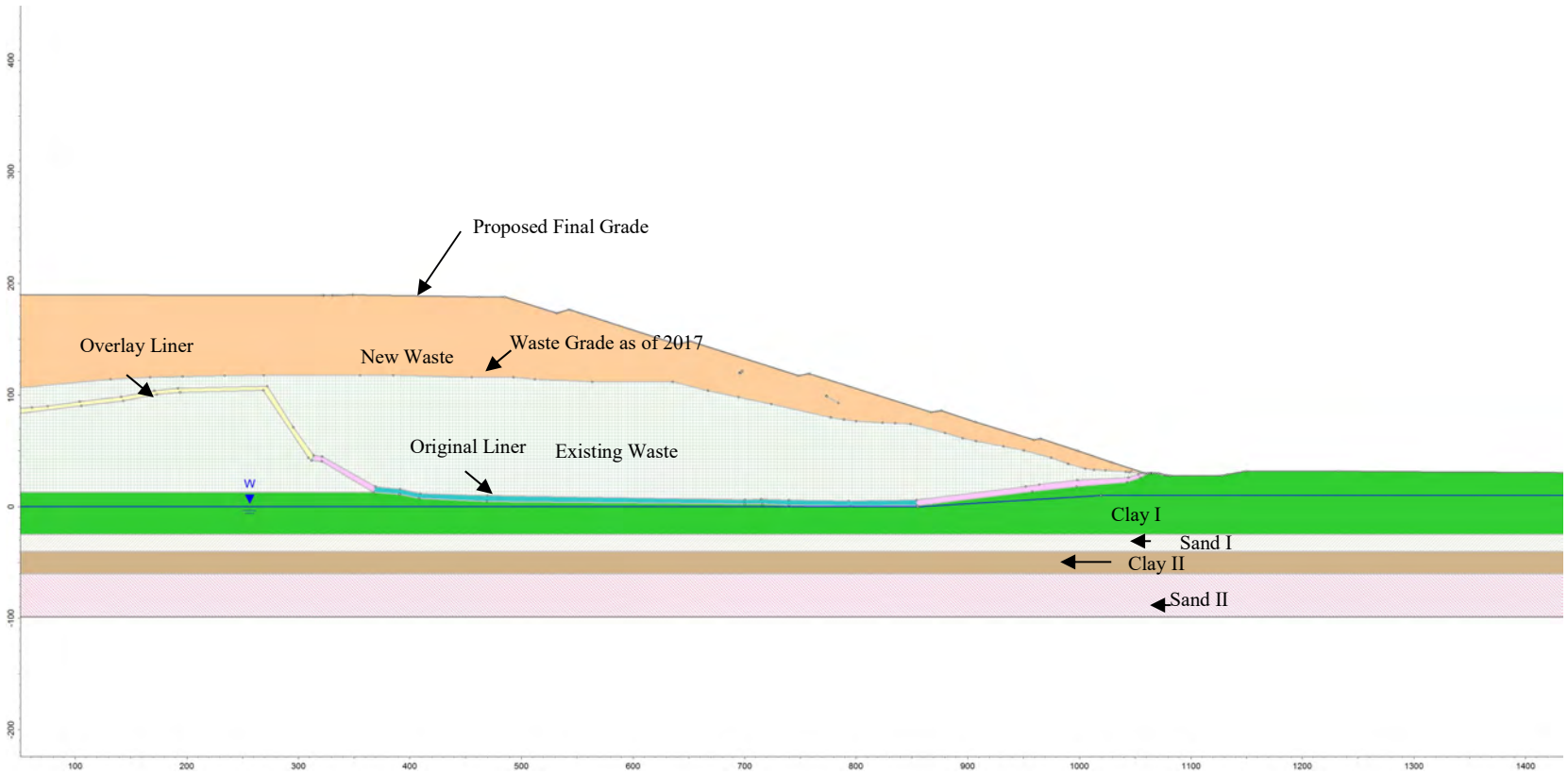


Figure 2a Cross Section A-A

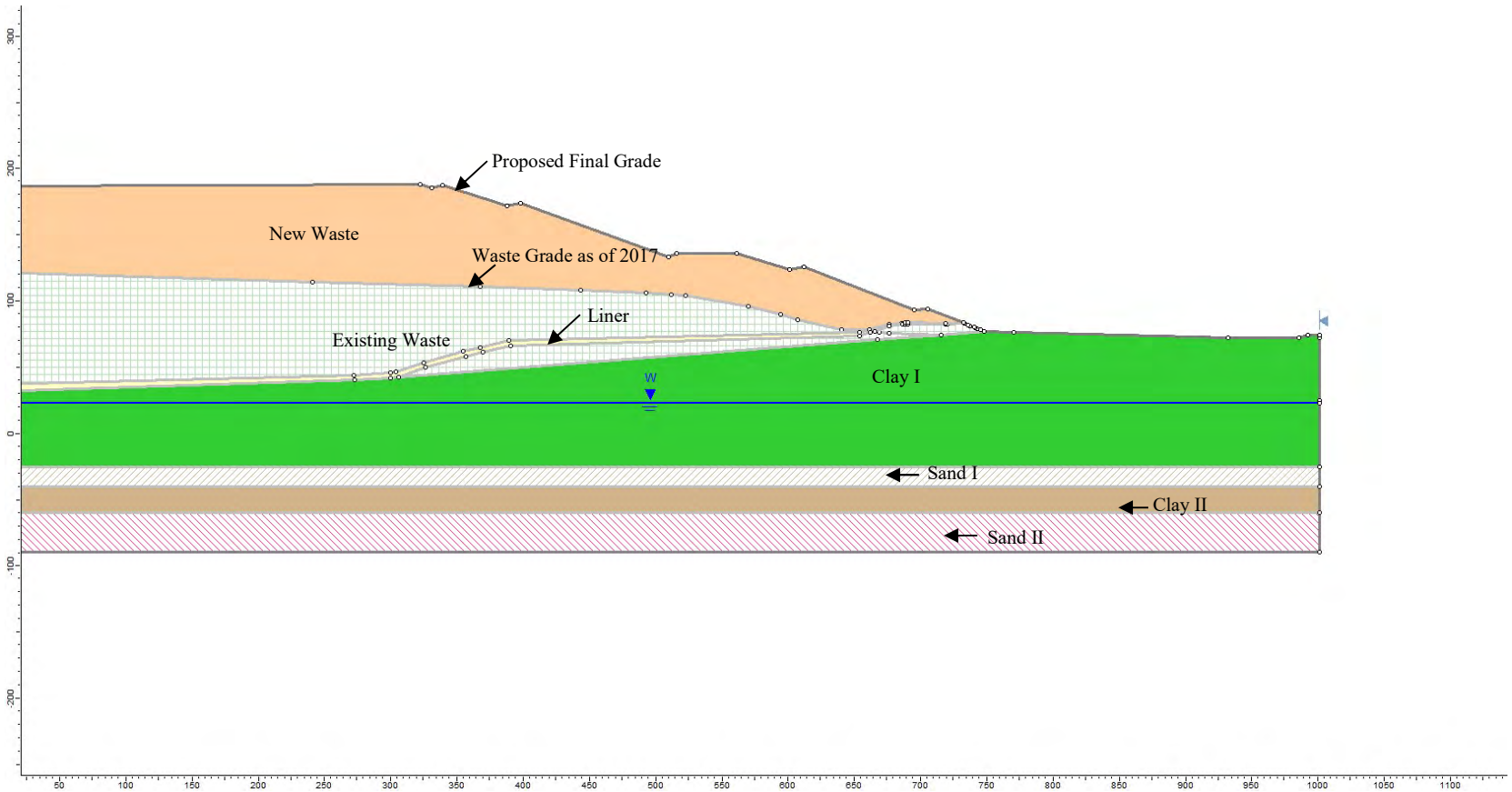


Figure 2b Cross Section B-B

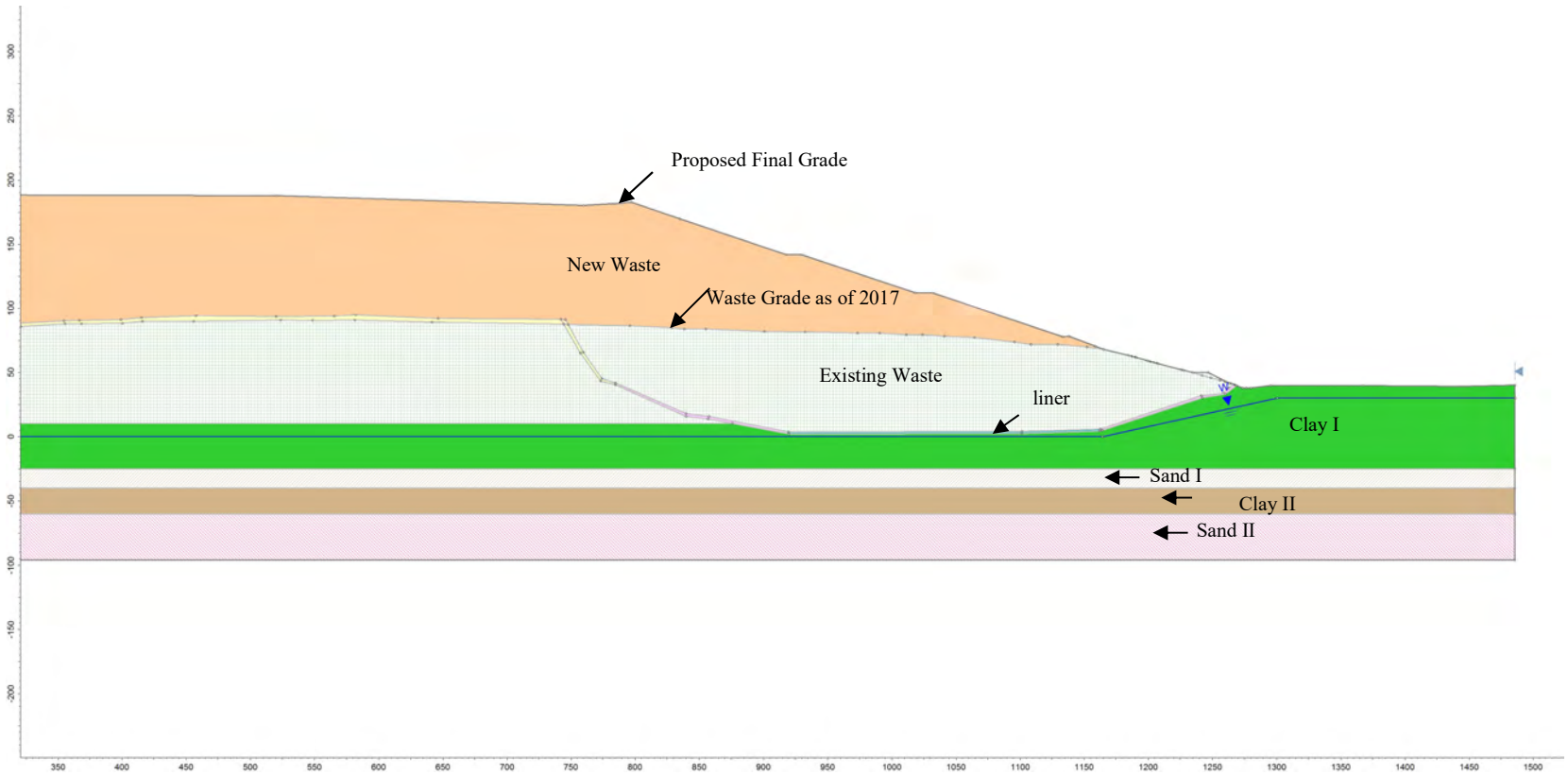


Figure 2c Cross Section C-C

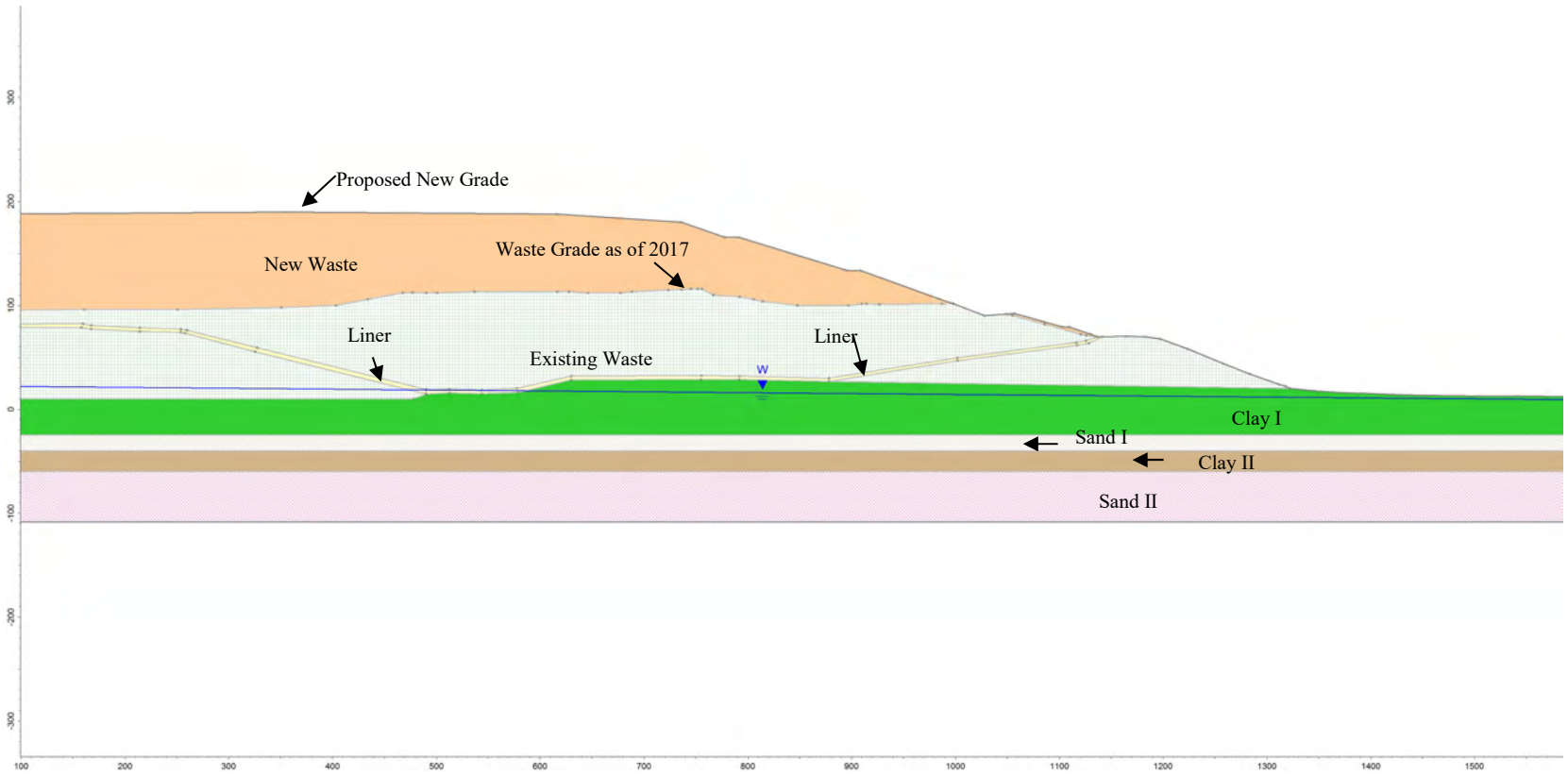
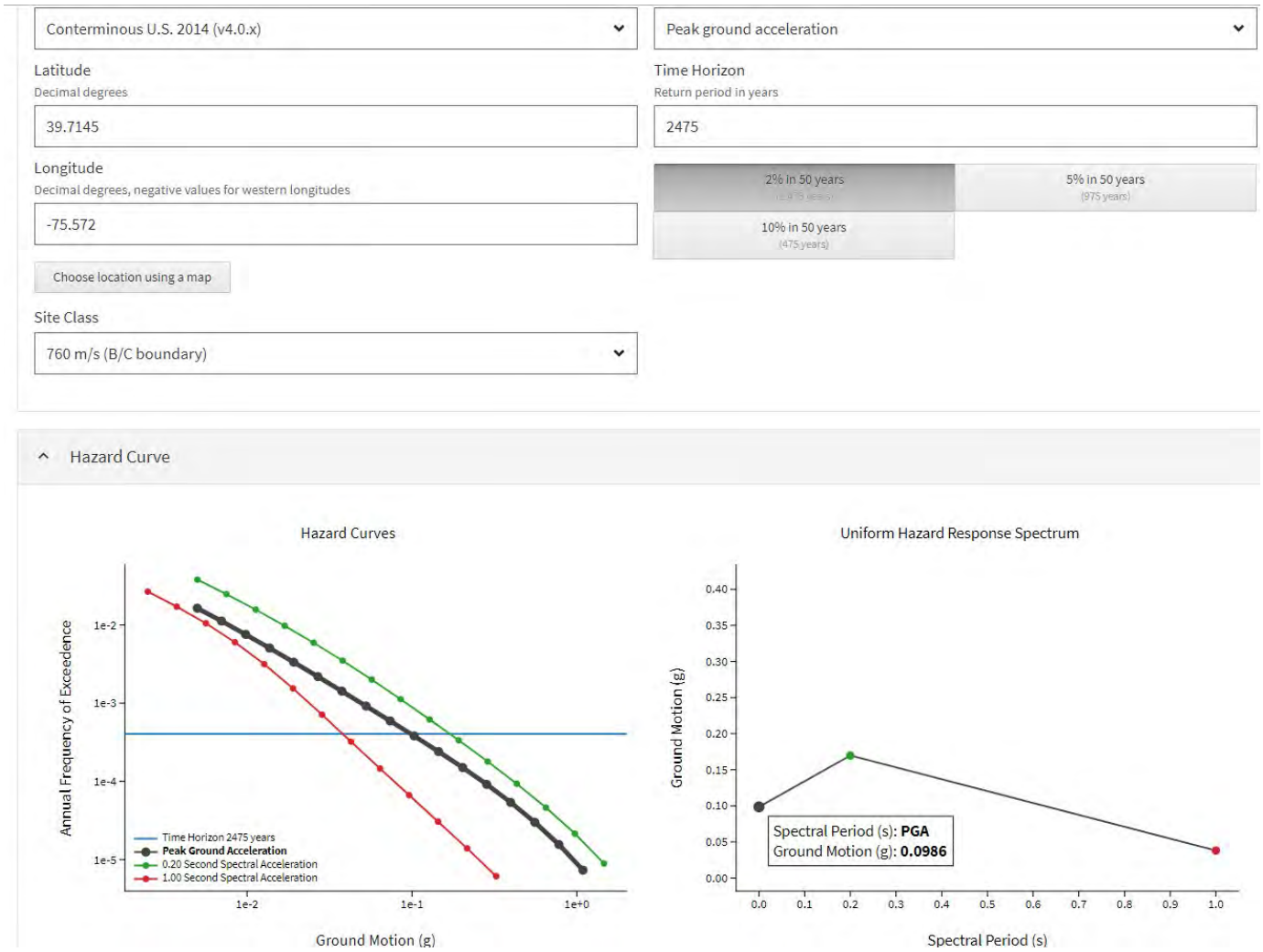


Figure 2d Cross Section D-D

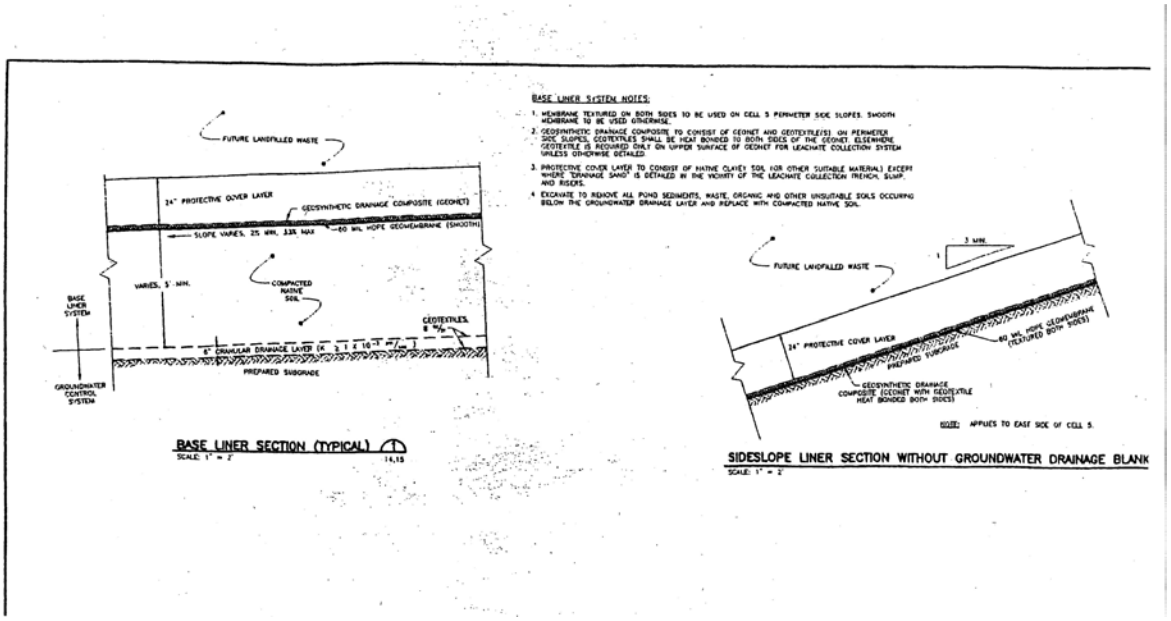


Source: USGS [2008]

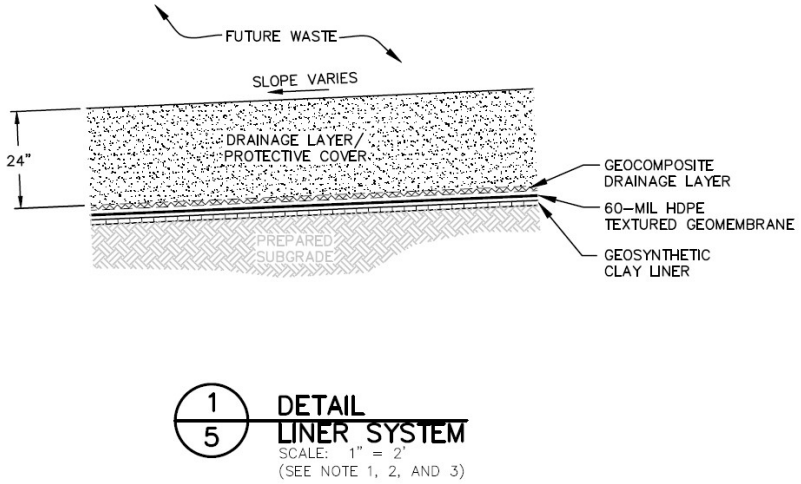
Figure 3 USGS Seismic Hazard Map

ATTACHMENT A

Information on Liner System



Liner System Details for Phase V



Liner System Details for Phase VI

TRI/ENVIRONMENTAL, INC.
A Texas Research International Company

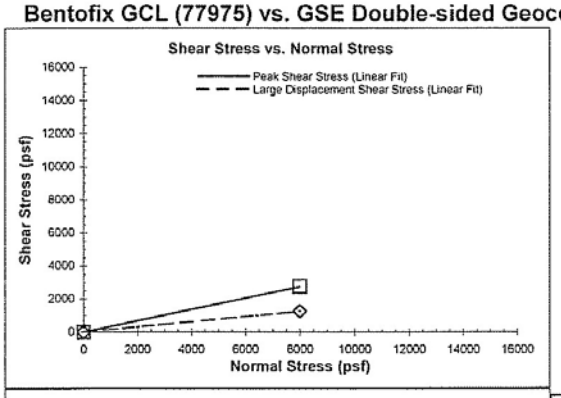
Interface Friction Test Report

Client: **Waste Management, Inc.**
Project: **DRPI**
Test Date: 06/27/07-06/28/06

TRI Log#: E2243-90-10
Test Method: ASTM D 5321

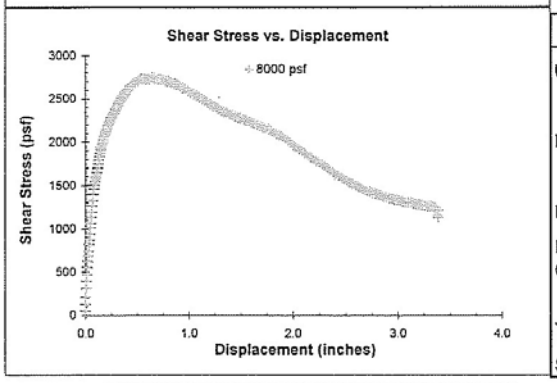
John M. Allen, 07/06/2007
Quality Review/Date

Tested Interface: Structural Fill vs. GSE 60 mil HDPE Textured Geomembrane (103137177) vs Bentofix GCL (77975) vs. GSE Double-sided Geocomposite (131233572) vs. Protective Cover



Test Results

	Peak	Large Displacement (@ 3.4 in.)
Friction Angle (degrees):	18.9	8.8
Y-intercept or Adhesion (psf):	0	0



Test Conditions

Upper Box & Floating Lower Box: GSE 60 mil HDPE Textured Geomembrane overlain by structural fill remolded at 112.4 pcf at 14.1%; Bentofix GCL (scrim side to geocomposite; GSE double-sided geocomposite overlaying protective cover soil remolded at 110.6 pcf at 0%

Box Dimensions: 12"x12"x4"

Interface Conditioning: Interface soaked and loading applied for a minimum of 24 hours prior to shear.

Test Condition: Wet

Shearing Rate: 0.04 inches/minute

Test Data

Specimen No.	1
Bearing Slide Resistance (lbs)	84
Normal Stress (psf)	8000
Corrected Peak Shear Stress (psf)	2746
Corrected Large Displacement Shear Stress (psf)	1244
Peak Secant Angle (degrees)	18.9
Large Displacement Secant Angle (degrees)	8.8

The testing herein is based upon accepted industry practice as well as the test method listed. Test results reported herein do not apply to samples other than those tested. TRI neither accepts responsibility for nor makes claim as to the final use and purpose of the material. TRI observes and maintains client confidentiality. TRI limits reproduction of this report, except in full, without prior approval of TRI.

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Phase VI Liner System Interface Shear Strength Test Results

ATTACHMENT B

ESTIMATION OF UNDRAINED SHEAR STRENGTH INCREASE DUE TO CONSOLIDATION

ATTACHMENT B: ESTIMATION OF UNDRAINED SHEAR STRENGTH INCREASE DUE TO CONSOLIDATION

INTRODUCTION

The clays underlying DRPI Landfill has undergone some varying degrees of consolidation after waste was placed atop. Therefore, the undrained shear strength is expected to increase due to the consolidation process. The purpose of this calculation is to estimate the current degree of consolidation achieved and provide recommended undrained shear strength values for use with the slope stability evaluation.

DEGREE OF CONSOLIDATION ESTIMATE

Based on laboratory test results provided by C&J Associates [1994], the lower bound of the coefficient of consolidation (C_v) for Clay 1 is 6×10^{-4} cm^2/sec (0.056 ft^2/day). The dimensionless time factor (T) for estimating the degree of consolidation is calculated as:

$$T = \frac{C_v t}{H^2}$$

where : t = elapsed time since loading; and H = drainage distance.

The drainage distance is half of the clay layer thickness for double sided drainage. Using the maximum clay layer thickness (60 ft for Clay 1 and 20 ft for Clay 2), H is estimated to be 30 ft for Clay 1, and 10 ft for Clay 2. Based on the waste placement history at the site, the time factor at various parts of the landfill is calculated using the above equations. Assuming that the waste is placed at a constant filling rate, the degree of consolidation achieved currently (as of 2017) is taken from the design chart in Figure B-1. The table below summarized the results:

Phase	I-III	IV	V	VI
Waste placement since (year)	1983	1994	1999	2005
Elapsed time, t (year)	34	23	18	12
Time factor (Clay 1), T	0.77	0.52	0.41	0.27
Average Degree of Consolidation, U (%)	0.65	0.54	0.48	0.40
Time factor (Clay 2), T	6.95	4.70	3.68	2.46
Average Degree of Consolidation, U (%)	>90	>90	>90	>90

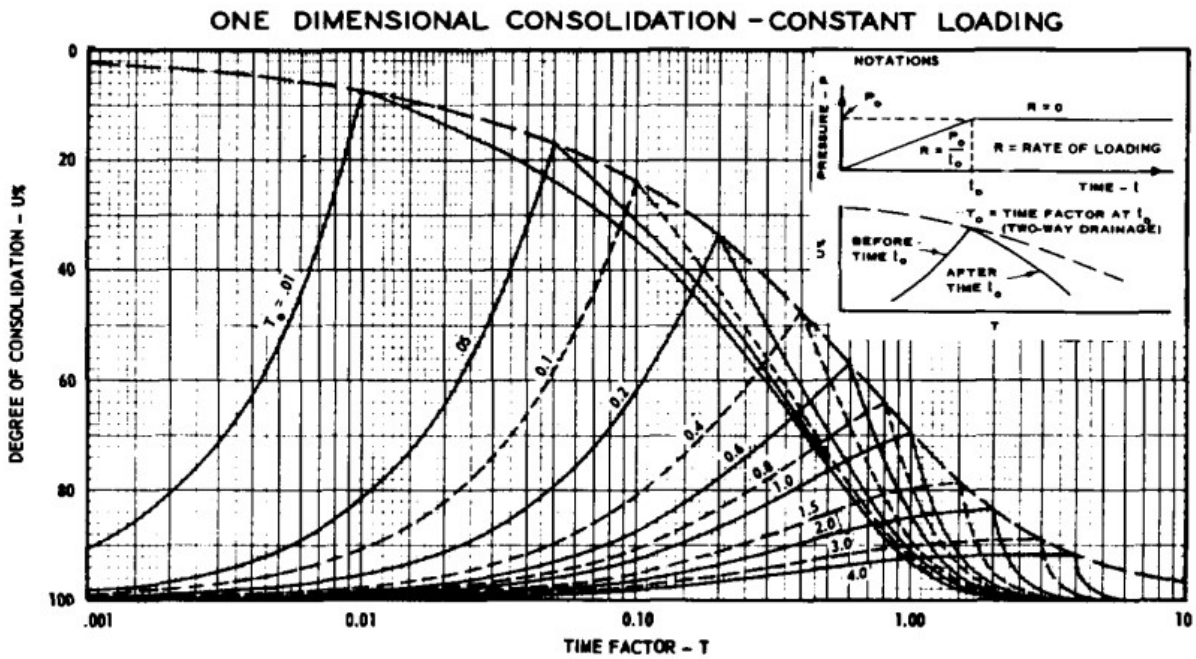

Figure B-1 Degree of Consolidation

UNDRAINED SHEAR STRENGTH INCREASE DUE TO CONSOLIDATION

The undrained shear strength (S_u) increase due to consolidation is estimated using empirical correlation provided by Kulhawy and Mayne [1990]:

$$S_u / \sigma_p = 0.11 + 0.0037PI$$

where: σ_p is the preconsolidation pressure, and PI = plasticity index.

Based on laboratory test results conducted by C&J [1994], the plasticity indices of Clay 1 and 2 are similar, which ranges from 19 to 38 with an average of 26. Thus, the shear strength increase due to consolidation can be calculated as:

$$\Delta S_u = 0.21\Delta\sigma_p$$

The increase of preconsolidation pressure ($\Delta\sigma_p$) is estimated as:

$$\Delta\sigma_p = U(\gamma_{waste} h_{waste} - \gamma_{soil} h_{soil})$$

Where: γ_{waste}=unit weight of overlying waste; h_{waste}=thickness of waste; γ_{soil} = unit weight of removed soil during landfill construction; and h_{soil} = removed soil thickness, U = average degree of consolidation achieved.

In the analysis, the clay is divided into different zones with varying waste thickness. Undrained shear strength of clays in those zones are estimated based on the weight of overlying waste and degree of consolidation completed using the above equations. Before undergoing consolidation, the undrained shear strength of Clay 1 was found to be 1,000 psf, while Clay 2 was found to be 1,400 psf [GZA, 1998], based on unconsolidated undrained triaxial test results. The table below summarizes the undrained shear strength used in the analysis:

Cross Sections		A-A			B-B			C-C			D-D		
Average Waste Thickness (ft)		40	75	100	25	55	75	33	72	80	23	64	85
Removed Soil Thickness (ft)		8	15	15	10	15	20	30	30	30	0	0	0
Overburden Pressure Increase (psf)		1840	3450	5200	550	2050	2850	0	1440	2000	1610	4480	5950
Clay 1	Average Degree of Consolidation (%)	54			27			65			40		
	Undrained shear Strength Increase (psf)	209	391	590	31	116	162	0	197	273	135	376	500
	Undrained Shear Strength (psf)	**1209**	**1391**	**1590**	**1031**	**1116**	**1162**	**1000**	**1197**	**1273**	**1135**	**1376**	**1500**
Clay 2	Average Degree of Consolidation (%)	90			90			90			90		
	Undrained shear Strength Increase (psf)	347	652	983	104	387	539	0	272	378	304	847	1125
	Undrained Shear Strength (psf)	**1747**	**2052**	**2383**	**1504**	**1787**	**1939**	**1400**	**1672**	**1778**	**1704**	**2247**	**2525**

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