Part III
Attachment III-D
Appendix III-D.5

GEOTECHNICAL ANALYSES REPORT

Pescadito Environmental Resource Center
MSW No. 2374
Webb County, Texas

PESCADITO
ENVIRONMENTAL RESOURCE CENTER

Initial Submittal MARCH 2015
Revised SEPTEMBER 2015
Revised August 2016

8-4-2016

STATE OF TEXAS
MICHAEL W. ODEN
67165
REGISTERED
PROFESSIONAL ENGINEER

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Prepared for:

Rancho Viejo Waste Management, LLC
1116 Calle del Norte
Laredo, TX 78041

Prepared by:

CBI

CB&I Environmental & Infrastructure, Inc.
12005 Ford Rd, Suite 600
Dallas, TX 75234

STATE OF TEXAS
JESSE PAUL VARSHO
114074
LICENSED
PROFESSIONAL ENGINEER

9-4-15

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Part III
Attachment III-D
Appendix III-D.5

GEOTECHNICAL ANALYSES REPORT

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114074
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MICHAEL W. ODEN
67165
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Michael W. Oden

STATE OF TEXAS
MICHAEL W. ODEN
67165
REGISTERED
PROFESSIONAL ENGINEER

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APPENDIX III-D.5-7

SLOPE STABILITY ANALYSIS WITH FLEXIBLE MEMBRANE LINER

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1.0 INTRODUCTION

This Final Cover Stability Analysis with Flexible Membrane Liner has been prepared to supplement the information contained in III-D.5-5 "Final Cover Stability Analysis" for the Pescadito Environmental Resource Center and addresses the addition of a Flexible Membrane Liner (FML) and drainage geocomposite at the base of the clay infiltration layer described in the Alternative Final Cover Demonstration Report (III-D.8).

The analysis on the following pages demonstrate that a Final Cover System including the FML and drainage geocomposite proposed for use over cells that contain Class 1 industrial wastes can achieve the desired Factor of Safety of 1.5 and provide a stable cover.

1.1 Problem Statement

Determine the impact of including an FML and drainage geocomposite in the final cover system relative to maintaining a factor of safety of at least 1.5 for static conditions of the landfill final cover system (note the site is not located within a seismic hazard zone)

1.2 References

1. Abramson, L.W., Lee, T.S., Sharma, S., and Boyce, G.M., Slope Stability and Stabilization Methods, 2002 (refer to attached pages).

2. Koerner, G.R. and Narejo, D, Direct Shear Database of Geosynthetic-to-Geosynthetic and Geosynthetic-to-Soil Interfaces. Geosynthetic Research Institute, 2005. (refer to attached pages).

3. Landfill design specifications for layer types and thicknesses provided in the Summary of Geotechnical Design Parameters (contained in Appendix III-D.5-1).

4. Cross-sectional detail of final cover system provided in the Design Drawing set (Appendix III-D.3) and in Appendix III-D.8 contained in this Application.

1.3 Assumptions

- An infinite slope method was conservatively utilized to analyze the final cover stability. This type of analysis is conservative because it does not consider the buttressing that will be provided by the benching of the final cover at each terrace berm.
- The equation used in the analysis of forces for static conditions (Reference No. 1) and factor of safety (FS) against slope failure is given below:
- The inclusion of geosynthetic components in the final cover system created three additional interfaces requiring analysis in addition to the soil component previously analyzed. The interfaces and the resulting FS resulting from that particular interface are given below.

1.4 Calculations

- The equation used in the analysis of forces for static conditions (**Reference No. 1**) and factor of safety (FS) against slope failure is given below:

$$FS = \frac{c' + (h)(\gamma_{sat})(cos^2\beta)(tan\emptyset')}{(\gamma_{sat})(h)(sin\beta)(cos\beta)}$$

Where,

- γ_{sat} = Saturated unit weight of soil
- β = Angle of slope
- h = Thickness of cover soil
- ϕ' = Effective shear strength friction angle of soil
- c' = Effective shear strength cohesion of soil

- The final cover system with FML design includes the following components from top to bottom:

  - 7" Vegetative Cover / Erosion Control Layer
  - 30" Infiltration Layer
  - Double-sided geocomposite drainage layer
  - Textured 40 mil LLDPE Geomembrane Liner

- From the layers listed above the thickness of soil above the geocomposite drainage layer is:

$$[37 in. \times (1\, ft./12 in.)] \div cos 14.04^{\circ} = 3.178\, ft.$$

- The maximum slope of final landform is 4H:1V, therefore β = 14.04 degrees.
- Various site soil materials are assumed to be used for the final cover system. The final cover is assumed to be saturated with a saturated unit weight equal to 132 pcf.

Using h = 3.178 feet, γ_{sat} = 132 pcf, β = 14.04°, the equation reduces to

$$FS = (c'_{psf} + 394.79_{psf} (\tan \phi')) / 98.73_{psf}$$

and the calculated Factor of Safety corresponding to the interfaces are:

soil strength, c' = 720 psf, ϕ' = 13.5° - *Reference 3*

FS = 8.25 >> minimum required 1.5

soil to drainage composite, c' = 19 psf, ϕ' = 29.0° - *Reference 2*

FS = 2.41 >> minimum required 1.5

drainage composite to textured LLDPE, c = 169 psf, ϕ' = 26.0° - *Reference 2*

FS = 3.66 >> minimum required 1.5

soil to textured LLDPE, c = 121 psf, ϕ' = 21.0° - *Reference 2*

FS = 2.76 >> minimum required 1.5

2.0 GLOBAL STABILITY IMPACTS

The current global slope stability analysis, Appendix III-D.5-2 includes a final cover that consists solely of soils. The above calculations all show a higher factor of safety than the minimum factors of safety obtained in the global stability analyses. Therefore, adding an underlying geocomposite and geomembrane into the final cover system will not affect global stability.

3.0 CONCLUSIONS

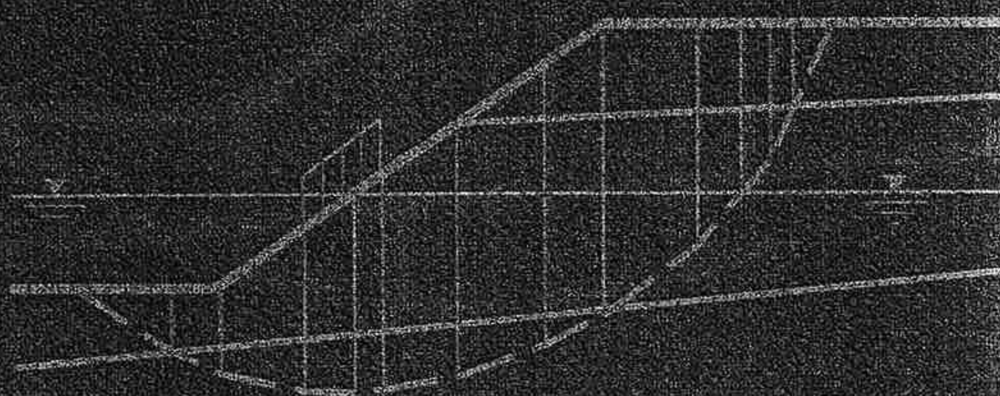
The analysis shows that the final cover system including geosynthetic components described above will be stable on the final landform. It is noted that any combination of shear strength parameters (cohesion and internal friction angle) that provide an equivalent secant friction angle greater than 20.6 degrees at a normal loading of 420 psf (soil final cover thickness times saturated density) will result in a calculated Factor of Safety of 1.5 or greater.

Reference No. 1

Slope Stability Method
(Abramson, et al)

SLOPE STABILITY AND STABILIZATION METHODS

second edition



LEE W. ABRAMSON • THOMAS S. LEE
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Next the normal, N, and driving, T, forces are determined:

$$N = W\cos\beta \qquad \text{and} \qquad T = W\sin\beta \tag{Eq. 6-9}$$

The available frictional strength along the failure plane will depend on ϕ and is given by

$$S = N\tan\phi \tag{Eq. 6-10}$$

Then if we consider the FOS as the ratio of available strength to strength required to maintain stability (limit equilibrium), the FOS will be given by

$$F = \frac{N\tan\phi}{W\sin\beta} = \frac{\tan\phi}{\tan\beta} \tag{Eq. 6-11}$$

The FOS is independent of the slope height and depth, z, and depends only on the angle of internal friction, ϕ, and the angle of the slope, β. Also, at $F = 1$, the maximum slope angle will be limited to the angle of internal friction, ϕ.

6.6.2 Infinite Slope in c–ϕ Soil with Seepage

If a saturated slope, in cohesive c–ϕ soil, has seepage parallel to the slope surface as shown in Figure 6.14, the same limit equilibrium concepts may be applied to determine the FOS, which will now depend on the effective normal force, N'. From Figure 6.14, the pore water force acting on the base of the typical slice will be given by

$$U = (\gamma_w h\cos^2\beta)\frac{b}{\cos\beta} = \gamma_w bh\cos\beta. \tag{Eq. 6-12}$$

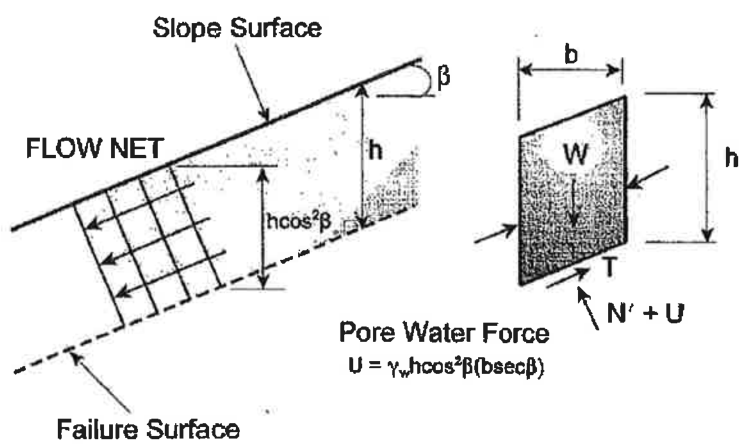

Figure 6.14 Infinite slope failure in c–ϕ soil with parallel seepage.

The available frictional strength along the failure plane will depend on ϕ' and the effective normal force and is given by

$$S = c'b \sec\beta + (N - U)\tan\phi' \tag{Eq. 6-13}$$

So the FOS for this case will be

$$F = \frac{cb \sec\beta + (N - U)\tan\phi'}{W \sin\beta} \tag{Eq. 6-14}$$

If we substitute $W = \gamma_{sat}bh$ into the above expression and rearrange, the FOS will be given by

$$\boxed{F = \frac{c' + h(\gamma_{sat} - \gamma_w)\cos^2(\beta)\tan\phi'}{\gamma_{sat}\, h \sin\beta \cos\beta}} \tag{Eq. 6-15}$$

where $\gamma' = (\gamma_{sat} - \gamma_w)$. For a $c' = 0$ soil, the above expression may be simplified to give

$$F = \frac{\gamma'}{\gamma_{sat}} \times \frac{\tan\phi'}{\tan\beta} \tag{Eq. 6-16}$$

From Equation 6.16 one can see that for a *granular* material, the FOS is still independent of the slope height and depth, h, but is reduced by the factor γ'/γ_{sat}. For typical soils, this reduction will be about 50 percent in comparison to dry slopes.

The above analysis can be generalized if the seepage line is assumed to be located at a height of $(m \times h)$ above the failure surface. In this case, the FOS will be given by

$$F = \frac{c' + h\cos^2\beta[(1-m)\gamma_m + m\gamma']\tan\phi'}{h \sin\beta \cos\beta[(1-m)\gamma_m + m\gamma_{sat}]} \tag{Eq. 6-17}$$

and γ_{sat} and γ_m are the saturated and moist unit weights of the soil below and above the seepage line. The above equation may be readily reformulated to determine the critical depth of the failure surface for any seepage condition and a c'–ϕ' soil.

6.7 PLANAR SURFACE ANALYSIS

Planar failure surfaces usually occur in slopes with a thin layer of soil that has relatively low strength in comparison to the overlying materials. Also, this is the preferred mode of failure for jointed materials that may dip toward proposed excavations.

A planar failure surface can be readily analyzed with a closed-form solution that depends on the slope geometry and the shear strength parameters of the soil along the failure plane. For the slope shown in Figure 6.15, three forces—weight, W, mobilized shear strength, S_m, and the normal reaction, N—need to be determined in order to evaluate the stability.

Reference No. 2

Direct Shear Database
(Koerner and Narejo)

Owl Copy

Geosynthetic Research Institute
475 Kedron Avenue
Folsom, PA 19033-1208 USA
TEL (610) 522-8440
FAX (610) 522-8441

Drexel UNIVERSITY

GRI GEI GII GSI GAI GCI

Direct Shear Database of Geosynthetic-to-Geosynthetic and Geosynthetic-to-Soil Interfaces

by

George R. Koerner, Ph.D., P.E.
Geosynthetic Research Institute
Folsom, PA 19033-1208
gkoerner@dca.net

and

Dhani Narejo, Ph.D.
GSE Lining Technology, Inc.
Houston, TX 77073
dnarejo@gseworld.com

GRI Report #30

June 14, 2005

Appendix Table 1. Summary of interface shear strengths.

Interface 1*	Interface 2*	Peak Strength					Residual Strength				
		Fig. No.	δ (deg)	Ca (kPa)	Points	R^2	Fig. No.	δ (deg)	Ca (kPa)	Points	R^2
HDPE-S	Granular Soil	1a	21	0	162	0.93	1b	17	0	128	0.92
HDPE-S	Cohesive Soil										
	Saturated	1c	11	7	79	0.94	1d	11	0	59	0.95
	Unsaturated	1c	22	0	44	0.93	1d	18	0	32	0.93
HDPE-S	NW-NP GT	1e	11	0	149	0.93	1f	9	0	82	0.96
HDPE-S	Geonet	1g	11	0	196	0.90	1h	9	0	118	0.93
HDPE-S	Geocomposite	1i	15	0	36	0.97	1j	12	0	30	0.93
HDPE-T	Granular Soil	2a	34	0	251	0.98	2b	31	0	239	0.96
HDPE-T	Cohesive Soil										
	Saturated	2c	18	10	167	0.93	2d	16	0	150	0.90
	Unsaturated	2c	19	23	62	0.91	2d	22	0	35	0.93
HDPE-T	NW-NP GT	2e	25	8	254	0.96	2f	17	0	217	0.95
HDPE-T	Geonet	2g	13	0	31	0.99	2h	10	0	27	0.99
HDPE-T	Geocomposite	2i	26	0	168	0.95	2j	15	0	164	0.94
LLDPE-S	Granular Soil	3a	27	0	6	1.00	3b	24	0	9	1.00
LLDPE-S	Cohesive Soil	3c	11	12.4	12	0.94	3d	12	3.7	9	0.93
LLDPE-S	NW-NP GT	3e	10	0	23	0.63	3f	9	0	23	0.49
LLDPE-S	Geonet	3g	11	0	9	0.99	3h	10	0	9	1.00
LLDPE-T	Granular Soil	4a	26	7.7	12	0.95	4b	25	5.2	12	0.95
LLDPE-T	Cohesive Soil	4c	21	5.8	12	1.00	4d	13	7.0	9	0.98
LLDPE-T	NW-NP GT	4e	26	8.1	9	1.00	4f	17	9.5	9	0.96
LLDPE-T	Geonet	4g	15	3.6	6	0.97	4h	11	0	6	0.98
PVC-S	Granular Soil	5a	26	0.4	6	0.99	5b	19	0	6	0.99
PVC-S	Cohesive Soil	5c	22	0.9	11	0.88	5d	15	0	9	0.95
PVC-S	NW-NP GT	5e	20	0	89	0.91	5f	16	0	83	0.74
PVC-S	NW-HB GT	5g	18	0	3	1.00	5h	12	0.1	3	1.00
PVC-S	Woven GT	5i	17	0	6	0.54	5j	7	0	6	0.93
PVC-S	Geonet	5k	18	0.1	3	1.00	5l	16	0.6	3	1.00

Appendix Table 1. (continued)

Interface 1*	Interface 2*	Peak Strength					Residual Strength				
		Fig. No.	δ (deg)	Ca (kPa)	Points	R^2	Fig. No.	δ (deg)	Ca (kPa)	Points	R^2
PVC-F	NW-NP GT	6a	27	0.2	26	0.95	6b	23	0	26	0.95
PVC-F	NW-HB GT	6c	30	0	8	0.97	6d	27	0	8	0.90
PVC-F	Woven GT	6e	15	0	6	0.78	6f	10	0	6	0.76
PVC-F	Geonet	6g	25	0	11	1.00	6h	19	0	11	0.99
PVC-F	Geocomposite	6i	27	1.1	5	1.00	6j	22	4.7	6	1.00
CSPE-R	Granular Soil	7a	36	0	3	1.00	7b	16	0	3	1.00
CSPE-R	Cohesive Soil	7c	31	5.7	6	0.71	7d	18	0	6	0.99
CSPE-R	NW-NP GT	7e	14	0	6	0.97	7f	10	0	6	0.98
CSPE-R	NW-HB GT	7g	21	0	3	1.00	7h	10	0	3	1.00
CSPE-R	Woven GT	7i	11	0	6	0.92	7j	11	0	3	1.00
CSPE-R	Geonet	7k	28	0	9	0.87	7l	16	0	9	0.80
NW-NP GT	Granular Soil	8a	33	0	290	0.97	8b	33	0	117	0.96
NW-HB GT	Granular Soil	8c	28	0	6	0.99	8d	16	0	6	0.91
Woven GT	Granular Soil	8e	32	0	81	0.99	8f	29	0	28	0.98
NW-NP GT	Cohesive Soil	9a	30	5	79	0.96	9b	21	0	28	0.79
NW-HB GT	Cohesive Soil	9c	29	0.9	15	0.71	9d	10	0	15	0.83
Woven GT	Cohesive Soil	9e	29	0	34	0.94	9f	19	0	16	0.86
GCL Reinforced (internal)	N/A	10a	16	38	406	0.85	10b	6	12	182	0.91
GCL (NW-NP GT)	HDPE-T	11a	23	8	180	0.95	11b	13	0	157	0.90
GCL (W-SF GT)	HDPE-T	11c	18	11	196	0.96	11d	12	0	153	0.92
Geonet	NW-NP GT	12a	23	0	52	0.97	12b	16	0	32	0.97
Geocomposite (NW-NP GT)	Granular Soil	13a	27	14	14	0.86	13b	21	8	10	0.92