prepared for

Waste Management of Carolinas, Inc.

9900 Freeman Road
Kernersville, North Carolina 27284

PHASE 2 COVER SYSTEM INVESTIGATION REPORT

PIEDMONT LANDFILL
KERNERSVILLE, NORTH CAROLINA

prepared by

GeoSyntec Consultants

1255 Roberts Boulevard, Suite 200
Kennesaw, Georgia 30144

Project Number: NCP2005.3184

March 2005

NORTH CAROLINA PROFESSIONAL SEAL No. 17097 ENGINEER JOHN F. BEECH
7 Mar 2005

WASTE MANAGEMENT

WASTE MANAGEMENT SOUTHERN GROUP

2859 Paces Ferry Road
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March 17, 2005

NC Department of Environment and Natural Resources
Division of Waste Management – Solid Waste Section
1646 Mail Service Center
Raleigh, North Carolina 27699-1646
Attn: Mr. Geoffrey H. Little

Subject: Submittal of Site Investigation Report
Piedmont Landfill and Recycling Center, Kernersville, North Carolina
Permit Number 34-06

Dear Mr. Little:

Please find enclosed one (1) original and one (1) copy of the Site Investigation Report for the above referenced facility developed in response to the Solid Waste Sections request dated February 11, 2005. This report is being submitted in accordance with the schedule contained in our February 16, 2005, submittal as modified by our request dated March 8, 2005, and approved in your e-mail dated March 9, 2005.

As requested by the Section, this report is intended to identify the cause or causes of the slope failure that occurred in January 2005, assess the condition and integrity of the final cover over the remainder of the site and address other items specifically requested by the Section in subsequent communications.

We trust you find the attached report satisfactory. If you have any questions or require additional information, please contact me at (770) 805-3529.

Waste Management of Carolinas, Inc.

Mark R. Snyder

Mark R. Snyder, P.E.
Project Manager, Closed Sites

enclosure

cc: Jim Coffee, NCDENR, w/o enclosure
March Smith, WMCI, w/o enclosure

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March Smith, WMCI, w/o enclosure

Reviewer Note: Due to Proposed Repair, No Comments issued for Investigative Report.
G. Little 6/15/05

Prepared for:

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PIEDMONT LANDFILL KERNERSVILLE, NORTH CAROLINA

Prepared by:

GEOSYNTEC CONSULTANTS
1255 Roberts Boulevard, Suite 200
Kennesaw, Georgia 30144

Project Number: NCP2005.3184

March 2005

NORTH CAROLINA PROFESSIONAL SEAL No. 17097 ENGINEER JOHN F. BEECH

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1. INTRODUCTION

This report was prepared to document the investigation of a downslope movement of a portion of the final cover system at the Piedmont Landfill in Kernersville, North Carolina. The movement occurred in mid January 2005, approximately two to two and one-half months after construction of the cover system was complete. The remainder of this report is organized as follows:

- Background information on the landfill and the events at the time of the downslope movement are described in Section 2.
- Field investigation activities related to the downslope movement are summarized in Section 3.
- The results of a laboratory testing program conducted as part of the investigation are presented in Section 4.
- Available information on the operation of the gas control system at the landfill is reviewed in Section 5.
- A review of factors affecting stability is presented in Section 6.
- Conclusions and recommendations are made in Section 7.

In addition, a number of specific questions related to the cover system instability were transmitted to Waste Management of Carolinas, Inc. (WM) from the North Carolina Department of Environment and Natural Resources (NCDENR). These questions are addressed in Appendix A of this document.

This report was prepared at the request of WM by Dr. John F. Beech, P.E. of GeoSyntec Consultants, Inc. (GeoSyntec). The report was reviewed by Dr. Rudy Bonaparte, P.E., also of GeoSyntec, in accordance with the internal peer review policy of the firm.

2. BACKGROUND AND CHRONOLOGY OF EVENTS

2.1 Site History

The Piedmont Landfill was originally permitted by WM in 1989, with RUST Environment and Infrastructure (RUST) as the design engineer of record. Waste disposal activities began in late 1989 and were terminated in the summer of 2004. The landfill is fully lined, with a single composite liner and leachate collection system underlying the entire landfill.

Two permit modifications to the final cover design occurred during the life of the facility. In May 1998, Ecologic Associates, Inc. prepared a permit modification that steepened final cover side slopes from 4H:1V (horizontal:vertical) to 3H:1V and provided for substitution of the originally-permitted 40-mil thick textured very low density polyethylene (VLDPE) geomembrane with a 40-mil thick textured linear low density polyethylene (LLDPE) geomembrane. In February and May 2004, GeoSyntec prepared a modification to the cover system design that replaced the then-permitted geomembrane and overlying drainage layer (formerly 40-mil thick textured geomembrane overlain by a double-sided geocomposite drainage layer) with a 50-mil thick LLDPE textured geomembrane with an integrated drainage layer on the top side of the geomembrane and an overlying nonwoven geotextile filter layer. The 2004 technical demonstration and NCDENR approval are included in Appendix B.

2.2 Final Cover System Construction

Construction of the Piedmont Landfill final cover system was completed in three phases over the operating life of the facility. The areas closed during each of the three phases of closure construction are shown on Figure 2-1. Facts regarding each construction phase including the dates, area closed, contractors, material suppliers, and construction quality assurance (CQA) firms are summarized as follows.

- Phase 1 Closure:
  - Date of construction - September 1996 through April 1997;
  - Area of closure - 22 acres;
  - Earthwork contractor - Morgan Corporation;

  - Geosynthetics installer - National Seal Company (NSC);
  - Geosynthetic supplier - NSC (geocomposite, geomembrane);
  - Geosynthetic supplier - Bentofix Technologies (GCL); and
  - CQA firm – RUST.

- Southern and Eastern Sideslope Closure:
  - Date of construction - July 2003 through September 2003;
  - Area of closure - 14.8 acres;
  - Earthwork and geosynthetics contractor - Comanco Environmental Corporation;
  - Geosynthetic supplier - Agru America (geomembrane);
  - Geosynthetic supplier - Tenax (geocomposite);
  - Geosynthetic supplier - CETCO (GCL); and
  - CQA firm - GeoSyntec Consultants.

- Phase 2 Closure:
  - Date of construction - June 2004 through October 2004;
  - Area of closure - 27.1 acres;
  - Earthwork and geosynthetics contractor - Comanco Environmental Corporation;
  - Geosynthetic supplier - Agru America (geomembrane);
  - Geosynthetic supplier - SKAPS Industries (geotextile);

- Geosynthetic supplier - CETCO (GCL); and
- CQA firm - GeoSyntec Consultants.

With specific reference to the Phase 2 closure, construction of the 27-acre Phase 2 final cover system commenced in June and was completed in October 2004. Phase 2 construction commenced about the same time waste placement at the landfill ceased, in order to achieve closure of the landfill as quickly as possible after final grades were reached. The Phase 2 cover system is an exit closure, meaning that it is the last area of the Piedmont Landfill to receive final cover. This exit closure condition is noteworthy because the Piedmont Landfill is the first WM landfill in the southeast to undergo a fully-lined exit closure wherein the entire landfill is enclosed with a geomembrane both below the waste (liner system) and above the waste (final cover system).

2.3 Final Cover System Design

The design components of the final cover system for each of the three closure phases is described below, with the components for each phase listed from the top surface to the bottom of the cover system. Note that the geosynthetic material product names are identified in parenthesis. Detail drawings showing the components of each cover system in cross section are presented in Figure 2-2.

- Phase 1 Closure:
  - 24-in. thick protective cover soil and topsoil layer;
  - 6-oz/sy double-sided geocomposite drainage layer (Texnet);
  - 40-mil thick textured VLDPE geomembrane (CoverSeal);
  - geosynthetic clay liner (Bentofix NS, installed nonwoven side up); and
  - prepared subgrade (10 in. minimum thickness).
- Southern and Eastern Sideslope Closure:
  - 24-in. thick protective cover soil and top soil layer;
  - 6-oz/sy double-sided geocomposite drainage layer (UVB-5065-2);
  - 40-mil textured LLDPE geomembrane (Microspike);

- geosynthetic clay liner (Bentomat SDN, nonwoven both sides); and
- prepared subgrade (10 in. minimum thickness).

- Phase 2 Closure:
  - 24-in thick protective cover soil and topsoil layer;
  - 8-oz/sy nonwoven geotextile (GE180);
  - 50-mil textured LLDPE geomembrane (Microdrain);
  - geosynthetic clay liner (Bentomat ST, installed woven side up); and
  - prepared subgrade.

2.4 Chronology of Events

2.4.1 Overview

On 28 August 2004, during construction of the Phase 2 final cover system, gas-pressure induced uplifting and associated downslope movement of a portion of the cover system occurred. This uplifting and movement, the corresponding investigation conducted at that time, and the completion of cover system construction after the uplifting and movement are documented in the 2004 CQA report prepared by GeoSyntec. The approximate limits of August 2004 cover system movement are illustrated on Figure 2-3. As already mentioned, in January 2005, approximately two and one-half months after the completion of construction, a portion of the cover system underwent downslope movement. The approximate limits of the January 2005 cover system movement are illustrated on Figure 2-4. Brief descriptions of these events are discussed below. As part of the current investigation, WM prepared a chronology of events, related to the August 2004 movement, which is summarized in Section 2.4.2. WM also prepared a chronology of events related to the January 2005 movement which is also summarized in Section 2.4.2.

2.4.2 August 2004 Cover System Movement

On 25 August 2004, immediately after the geomembrane was completely installed and seamed in the Phase 2 area, uplift of the geomembrane occurred in an area over which protective cover soil had not yet been placed. Uplifting of the geomembrane is shown in Photographs 1 and 2 in Appendix C.

In order to control the uplift, the closure construction contractor cut a temporary hole in the exposed geomembrane. Localized temporary uplift of the geomembrane during cap construction is a relatively common occurrence during final cover construction; however, the uplift on 25 August occurred quickly after completion of geomembrane installation and the uplift extended over an area considerably larger than typically observed.

On 28 August 2004, an approximate six acre portion of the Phase 2 final cover system, including the protective cover soil and topsoil layer which had already been placed, underwent uplift and downslope movement. Conditions of the construction area after the movement are documented in Photographs 3 to 8 in Appendix C. The approximate limits of the area that underwent movement are shown in Figure 2-3. This uplift and downslope movement is attributed to the build up of gas pressure under the cover system as a result of inadequate flare capacity compounded by a landfill gas flare-system shutdown on 27 August. The uplift was mitigated when the closure construction contractor excavated the cover soil and cut a hole in the geomembrane to relieve the gas pressure. Temporary passive gas vents were then installed. A detailed investigation was undertaken to verify the integrity of the final cover system immediately after the uplift and movement occurred. The results of the investigation were discussed with NCDENR on 21 September 2004. Based on the investigation, it was concluded that the geomembrane was effectively intact and did not need to be replaced. A repair program was implemented. The investigation and repair program is documented in the Phase 2 final cover system CQA report.

2.4.3 January 2005 Cover System Movement

As already noted, in mid January 2005, an approximate seven acre portion of the Phase 2 final cover system underwent downslope movement in the area shown on Figure 2-4.

Based on observations made by personnel that visited the Piedmont Landfill site, the cover system movement occurred sometime between 5 p.m. on 13 January and 10 a.m. on 20 January, 2005. WM and Joyce Engineering personnel were on site on 13 January 2005, at which time the final cover system was observed to be intact. An independent contractor to WM noticed the failed area sometime between 8 a.m. and 10 a.m. on 20 January when he was on site to perform maintenance on the flare system.

Between 8 p.m. on 13 January to 9 a.m. on 14 January 2004, approximately 1.1 inches of precipitation fell onto the Piedmont Landfill site. This storm event is significantly smaller than the storm event used to design the Phase 2 final cover system.

The flare system for the landfill shut down on 14 January 2005. Based on available information, it is estimated that Flare No. 1 shut down at approximately 2 p.m. This estimate is based on reports by Joyce Engineering that the chart paper was changed out around noon on 14 January and scaling of the flare system chart paper indicates that Flare No. 1 ceased operation 2 to 2.5 hours later. Flare No. 2 is believed to have shut down at approximately 2:10 p.m. This estimate is based on the output from the data logger recording for the blower flow rate and flare temperature.

3. FIELD INVESTIGATION

3.1 Overview

As part of the investigation of the January 2005 cover system movement, several field investigation activities were undertaken. An initial reconnaissance of the area of interest was conducted on 21 January 2005. A follow-up site visit was conducted on 24 January 2005. Subsequent to this second visit, a field sampling program was developed which was undertaken on 2 February 2005. Lastly, a site reconnaissance of the areas closed in 1996/1997 and 2003 was conducted on 1 March 2005. These field investigation activities are briefly described in the remainder of this section of the report.

3.2 21 January Field Investigation

On 20 January 2005, WM notified GeoSyntec of the reported final cover system slope movement at the Piedmont Landfill. GeoSyntec was requested to immediately mobilize to the field to document the nature and extent of the problem. On 21 January 2005, Mr. Nelson Breeden of GeoSyntec arrived on site. GeoSyntec also arranged for Allied Land Surveying Company to be on site to survey features associated with the cover system movement.

Photographic documentation of the area of interest taken on 21 January and 1 March, 2005, are shown in Photographs 9 to 20 of Appendix C. It was observed that the final cover system on the north facing slope that had been constructed in 2004 had moved down the slope. Near the crest of the slope the GCL component of the cover system was exposed (Figure 2-4). This exposed area of GCL was approximately 700 feet in length along the slope and up to a maximum of 60 feet in length down the slope. The limits of exposed GCL shown on Figure 2-4 are based on field survey. Figure 2-4 also shows the alignment of the crest and swales of the drainage benches on the northern slope. The alignment of these features is also based on field survey.

3.3 24 January Field Investigation

On 24 January 2005, Dr. Beech of GeoSyntec visited the Piedmont Landfill site. The purpose of the visit was to observe the area of interest in order to better understand and document the nature of the downslope cover system movement.

The following observations are based on Dr. Beech's visit:

- Sliding occurred between the geomembrane and GCL components of the cover system on the landfill's north slope. Specifically, the movement occurred between the textured underside of the geomembrane and the woven geotextile top side of the GCL.

- There was no evidence of movement of the protective cover soil relative to the underlying geosynthetic components (geomembrane and geotextile) of the cover system. The cover soil appeared to move as a unit with the underlying geosynthetics.

- Within the area of movement, the drainage benches moved down slope with the protective cover soil. There was no movement of the drainage benches relative to the protective cover.

- The protective cover soil mounded at the toe of the access road that crosses the north slope. This mound of soil tended to buttress the cover material further up slope.

- The cover soil was visually observed to have a good stand of vegetation and there were no signs of significant erosion or rills in the cover soil.

- In general, the exposed GCL was in "good condition". However, there were localized areas where the GCL surface was torn. It is suspected that this tearing was caused by the textured surface of the geomembrane grabbing the GCL as it slid. Free bentonite was not observed on the surface of the exposed GCL.

- The area movement in January 2005 coincided to a significant degree with the location of the August 2004 cover system uplift and movement episode. However, the area of January 2005 movement extended about 250 feet further to the west than the August 2004 area, and the August 2004 area extended about 200 feet further to the east than the January 2005 area.

Based on the observations made during the 24 January 2005 site visit, it was apparent that movement had occurred along the geomembrane-GCL interface. Based on this observation, it was decided to develop a sampling and laboratory testing program for the cover system materials of interest. The sampling program is described below and the laboratory testing program is described in Section 4.

3.4 2 February Field Investigation

3.4.1 Overview

On 2 February 2005, GeoSyntec mobilized a field sampling effort to obtain soil and geosynthetic samples and observe and document site conditions. Activities associated with this event included initial planning, sample collection, site observations, and follow up reporting. GeoSyntec contracted with Contaminant Control Inc. (CCI) to provide labor and equipment to excavate the test pits for sample collection. A brief description of the sampling activities undertaken and observations made are presented in the following subsections. Photographs 21 through 50 of Appendix C show the test pits.

3.4.2 Sample Locations

GeoSyntec collected soil and geosynthetic samples from three test pits generally identified and located as follows: (i) TP-1, from an area with no evidence of cover system movement; (ii) TP-2, from an area with evidence of January 2005 cover system movement (outside the area of August 2004 observed cover system movement); and (iii) TP-3, from an area with evidence of cover system movement both in January 2005 and August 2004. Approximate locations of the test pits are illustrated on Figure 3-1. Note that the approximate areas of August 2004 and January 2005 cover system movement are identified in Figure 3-1 for reference.

3.4.3 Sample Collection

Samples were collected from test pits excavated using a small excavator with a smooth edge bucket. Test pits were approximately seven to eight feet square. The protective cover soil was excavated at each test pit location to within two to three inches of the geosynthetics. The remaining protective cover soil was removed by hand shoveling to protect and preserve the geosynthetics. A sample of the protective cover soil consisting of two five-gallon buckets was collected at each test pit location. GeoSyntec personnel brushed the geotextile surface clean using a hand broom, marked an approximate three-foot square sample outline, and marked the sample with an identification number and orientation (i.e. top) prior to cutting the geotextile. After cutting, the geotextile was gently rolled back from one side to make observations and take photographs of the underlying geomembrane. Next, a sample identification number and orientation were marked on the geomembrane sample and a geomembrane sample was cut. The geotextile and geomembrane samples were then removed from the test pit. Photographs were taken and observations were made of the GCL upon removal of the overlying geosynthetics. Next, GeoSyntec personnel marked the GCL with an identification number and orientation, cut the GCL sample, and removed the sample from the test pit. GCL samples were folded in quarters and sealed in plastic bags to preserve moisture content. A prepared subgrade layer sample consisting of two five-gallon buckets was collected at each test pit location.

Upon the completion of sample collection, each test pit location was backfilled and restored. Excavation spoils were placed, compacted, and smoothed with the excavator bucket to fill in soil sample holes in the subgrade soil layer. Next, two sheets of plastic were placed to cover the three-foot square geosynthetic sample location: one was tucked between the GCL and the geomembrane; and the second was placed overlapping and on top of the geotextile. The test pit was then backfilled, compacted, and smoothed with the excavator bucket. The cover system surface was "track walked" with the excavator and the test pit location was marked with two wooden stakes for future identification.

3.4.4 Observations

GeoSyntec personnel made a number of observations during test pit excavation and sampling activities. Observations were reported in a daily field report and documented in photographs presented in Appendix C. General observations for each test pit are summarized in Table 3-1.

3.5 1 March Field Investigation

The field investigation activities conducted on 21 and 24 January, and 2 February, focused on the area of the north slope in which movement was readily apparent. On 1 March, a site reconnaissance was conducted by GeoSyntec personnel (Mr. Breeden) to visually inspect the condition of the Piedmont Landfill final cover system in areas outside of the January 2005 area of movement. This site reconnaissance included the portion of the Phase 2 closure that had not undergone downslope movement and the areas closed in 1996/1997 and 2003. The purpose of this activity was to look for evidence of cover system instability such as cracking at the crest or along the slope, or mounding of soil at the toe of the slope. Documentation from this visit is included in Photographs 51 through 62 in Appendix C. Mr. Breeden did not observe any evidence of cover system instability during his observation of the 1996/1997 and 2003 closure construction areas or the portion of the Phase 2 closure outside of the January 2005 area of movement.

4. LABORATORY TESTING PROGRAM

4.1 Overview

The samples collected from Test Pits 1, 2, and 3 were transported by GeoSyntec personnel to SGI Testing Services, LLC in Norcross, Georgia. In addition, on 20 February 2005, a representative of WM collected a sample of unused geomembrane from a stockpile of excess material from the construction of the Phase 2 final cover system. This sample was also forwarded to SGI for testing. The following tests were performed on the sampled materials:

- moisture content of the GCL, subgrade soil, and protective cover soil;
- asperity height of textured surface of geomembrane; and
- interface shear strength of cover system materials.

The test results are summarized in the remainder of this section. Detailed test results are provided in Appendix D.

4.2 Moisture Content Measurements

Movement in the field was observed to occur along the geomembrane-GCL interface. Both the internal and interface shear strength of GCLs is known to be dependent on the moisture content of the GCL. Therefore, the moisture content of the GCL samples obtained from the test pits was measured. The moisture content of the subgrade soil and cover soil samples was also measured, although that information is of secondary interest to this investigation. The moisture content results are summarized in Table 4-1, and are presented in Appendix D.

Measured GCL moisture contents were found to be in the range of 75 to 85 percent. This range of moisture contents is consistent with moisture content values reported in the literature for GCLs placed on compacted subgrades. This range is also consistent with the moisture content at which the GCL interface testing performed as part of the 24 May 2004 technical demonstration was conducted. The GCL-geomembrane interface testing conducted as part of that demonstration is presented in Appendix B.

4.3 Asperity Height

The degree of geomembrane texturing was qualitatively evaluated by measuring the height of the asperities that form the textured surface of the geomembrane. Asperity heights were measured on samples from material from each test pit and from the unused on-site stockpile. For each sample, ten asperity height measurements were made and an average height value for the sample was calculated. The average asperity height for each sample is given in Table 4-2. It can be seen in the table that the samples from Test Pits 1, 2, and 3 have average asperity heights on the order of 14 mils compared to the 17 mil asperity height for the unused material from the stockpile. Based on these results, as well as the interface shear strength test results presented subsequently, it is hypothesized that the geomembrane installation process resulted in an approximate 20 percent decrease in asperity height. Although the asperity height of the material tested as part of the 24 May 2004 technical demonstration was not measured, the manufacturer specification sheet included in that technical demonstration document calls out a minimum average roll value (MARV) of 16 mils for the asperity height. It is noted that the MARV asperity height will be less than average asperity height. Thus, the average asperity height for manufactured material would be in excess of 16 mils or likely on the order of 17 to 18 mils. The installation effect on asperity height has an influence on the interface shear strengths produced, as evidenced by the results presented below.

4.4 Interface Shear Strength Testing Program

An interface shear strength testing program was conducted to evaluate representative interface shear strengths of the cover system components. The results of the interface shear strength testing program are used to evaluate slope stability in Section 6. The samples collected from Test Pits 1, 2, and 3 and the sample collected from the unused on-site stockpile were used in the testing program. Fresh samples of GCL were obtained from the manufacturer and used in the testing program. Fresh samples were used due to concern that the GCL samples obtained in the field and brought to the laboratory had undergone free swell and were no longer representative of pre-movement field conditions where the GCL had hydrated under the normal stress applied by the overlying protective cover soil and topsoil layers. Interface tests were performed in a 12 in. by 12 in. shear box in general accordance with ASTM D 5321, Standard Test Method for Determining the Coefficient of Soil or Geosynthetic Friction by the Direct Shear Method. Additional testing details are provided in Appendix D. For the tests, the GCL was hydrated under the applied normal stress to a target moisture content of 80 to 100 percent. The sample configuration for the tests, unless otherwise indicated, was as follows:

- protective cover soil;
- 8-oz/sy nonwoven geotextile filter;
- 50-mil thick textured LLDPE geomembrane (AGRU Microdrain) with microspike textured surface down;
- Bentomat ST GCL with woven geotextile up; and
- subgrade formed with site soils.

This sample configuration was sheared in a manner that allowed sliding to occur along the interface with the lowest shear strength. In all tests, sliding occurred between the geomembrane-GCL interface. Detailed results from the interface tests involving the above-described sample configuration are presented in Appendix D; the interface results are summarized in Table 4-3. The following observations are provided with respect to the interface shear strength results:

- The unused sample of geomembrane from the stockpile had similar interface strength characteristics as the geomembrane sample tested as part of the technical demonstration in May 2004. This result implies that the geomembrane material delivered to the Piedmont Landfill site for the Phase 2 closure had "as delivered" interface characteristics conforming to the May 2004 technical demonstration document.

- The interface shear strength for the installed geomembrane from Test Pit TP-1, obtained from an area that had not undergone movement, resulted in lower interface shear strengths than the pre-construction and unused stockpile geomembrane samples. This result suggests that the process of geomembrane installation alone reduced the geomembrane texturing (also called "roughness") in comparison to the amount of texturing on the factory delivered material. This is an unexpected result.

- The interface shear strengths obtained on tests using geomembrane samples from Test Pits TP-2 and TP-3 (i.e., the areas where downslope movement had occurred) produced results that are similar to, and actually slightly higher (stronger) than those obtained from the test on the sample from Test Pit TP-1.

This result implies that both the uplifting and movement that occurred in August 2004, and the downslope movement that occurred in January 2005, did not further reduce the geomembrane texturing beyond the reduction causes by the installation process.

In addition, a test was conducted to simulate the geomembrane-GCL interface conditions associated with the uplift and downslope movement that occurred in August 2004. A fresh sample of the AGRU Microdrain geomembrane obtained from the on-site stockpile was placed against a fresh sample of GCL prehydrated to a target moisture content between 80 and 100 percent. The test was conducted in a manner that shearing was controlled to occur along the geomembrane-GCL interface. The test was conducted to a shear displacement of 3.5 in. After shearing to this displacement, the samples were reset to their original locations, reloaded, and resheared. This sequence of events was meant to simulate interface movement in August 2004, followed by re-application of the cover system normal stresses. The results of the test are summarized in Table 4-4 and the details are provided in Appendix D. The test results produced an approximate 10 percent reduction in peak shear strength between the first and second shearing episodes. The asperity height of the sample was measured after the second shearing episode. The measured asperity height of 16.9 mils is similar to the height of 17.1 mils, presented in Table 4-3, for the sample of untested material from the stockpile. The asperity height test results are presented in Appendix D.

5. REVIEW OF GCCS RECORDS

5.1 Overview

As noted previously, the movement observed in August 2004 was associated with the build up of landfill gas beneath a portion of the Phase 2 final cover system. Photographic documentation of the movement observed in August 2004 is presented in Appendix C. Gas pressure buildup was also identified as a possible contributor to conditions that produced the January 2005 cover system movement. For this reason, gas collection and control system (GCCS) data were reviewed as part of the current investigation. The GCCS for the Piedmont Landfill is briefly described in Section 5.2. An indicator into the efficiency of the operation of the GCCS is the methane concentration at individual extraction wells. This information is reviewed in Section 5.3. Another overall indicator of operation is the measured flow rate at the blowers associated with the flare system. This data is reviewed in Section 5.4.

5.2 Description of Gas Collection and Control System (GCCS)

Landfill gas at the Piedmont Landfill is managed through a GCCS. The gas is collected from 55 vertical gas extraction wells installed throughout the landfill. Sixteen of these wells (identified below) were installed as part of the exit closure. The remainder of the wells were installed at earlier dates. The locations of the gas extraction wells are shown on Figure 5-1. The wells are connected to a header line consisting of either 8-in. or 10-in. diameter HDPE pipe that loops around the landfill. A series of 6-in. diameter laterals connect to this header loop. The header loop is connected to a combustion system consisting of two landfill gas (LFG) flares operating in parallel. Flare No. 1 has a maximum rated capacity of 1100 standard cubic feet per minute (scfm) and was installed in 1996. The associated blower has a rated capacity of 1100 scfm at a water column vacuum of 28 in. Flare No. 2 was installed in December 2004 and has a maximum rated capacity of 950 scfm. The associated blower has a rated capacity of 950 scfm at a water column vacuum of 36 in.

Gas extraction well Nos. 16, 17, 21, 22, 23, 27, 28, 36, 37, 38, 40, 41, 44, 45, 47, and 48 were installed in 2004 as part of the exit closure construction for Phase 2. These wells were placed into service in the summer and fall of 2004 and formal monitoring started in October 2004 following completion of Phase 2 construction. The wells were installed in accordance with the 1998 Landfill Gas Collection and Control Design Plan prepared by Earth Tech. In accordance with that design, well Nos. 38, 40, 41, 44, and

45 were installed with the top of the screen 40 ft below the final waste surface. The approximate waste surfaces in July 2001 and January 2004 are shown in Figure 5-2. Waste placed between these two surfaces is the last waste placed at the landfill prior to exit closure. A cross-section through the landfill is shown in Figure 5-3. This cross-section includes several gas wells and the waste surface in July 2001 and January 2004. As can be observed from the cross-section, the gas wells in the exit closure area are screened below the most recently placed waste, which is expected to have a higher gas generation rate at this time than the older waste in the landfill.

5.3 Measured Methane Concentrations

The percent methane measured in the gas extraction wells on 5 January, 17 February, and 15 March 2005 are plotted on Figures 5-4, 5-5, and 5-6 respectively. From these figures, it can be observed that in January 2005, a number of the wells installed as part of the exit closure had some of the highest measured methane concentrations with a number of the wells in excess of 65 percent. This is a sign that the well field was understressed at the time of the reading and landfill gas was accumulating in the landfill at a rate faster than the rate of gas extraction. In general, these concentrations exceed those recommended by various organizations for optimal stressing of the landfill gas extraction system. For example, the Solid Waste Association of North America (SWANA) indicates that a GCCS should be operated to maintain methane concentrations on the order of 50 to 55 percent. It is noted that the methane concentrations measured in a number of the wells in February and March are approaching the target range. The change in methane concentrations are attributed to ongoing tuning of the well field being conducted by WM.

5.4 Measured Blower Flow Rates

On 5 January 2005, the recorded blower flow rate for Flare No. 1 was 429 scfm. On 13 and 14 January 2005, the recorded blower flow rate for on Flare No. 2 was approximately on the order of 600 scfm. It thus appears that for at least a portion of January 2005, the combined flow rate of the two blowers was only approximately 55 percent of the estimated 2005 gas generation rate of 1869 scfm from the new source performance standards (NSPS) permit application. The observation that the rate at which gas was being withdrawn from the landfill is significantly less in January 2005 than the estimated gas generation rate is supported by the 5 January gas monitoring data referenced above which shows that many of the gas extraction wells in the landfill were understressed at that time. Withdrawal of gas from the landfill at a rate lower then the generation rate can result in the build up of excess gas in the landfill over time. It is believed that the most critical area of the landfill with respect to excess gas buildup and cover system slope stability would be in the exit closure area.

The blowers that are associated with Flare No. 1 and Flare No. 2 were reported to be operating between 850 and 875 scfm and 750 and 800 scfm, respectively, in February 2005. Based on the measured methane concentrations in February, as shown in Figure 5-5, improved operation of the blower system appears to have resulted in a general decrease in measured methane concentrations.

It is important to note that while the methane concentrations and blower system operation may not have been optimized in January, the landfill was in compliance with the site NSPS requirements.

5.5 Potential for Build Up of Positive Pressure

As part to of the evaluation of the GCCS at Piedmont Landfill, on 1 March 2005, WM shut off the vacuum to gas extraction well No. 23 and monitored the change in pressure with time. The resulting measured change in pressure with time is plotted in Figure 5-7. From this plot, it can be observed that the gas pressure relative to atmospheric pressure decreased from a water column vacuum of -9 in. to 0 in. over an approximate one hour period. The pressure then increased to a positive value of 1.5 in. over the next five hours. Gas extraction well No. 23 is surrounded by active wells. The calculated radii of influence of these surrounding wells for a gas generation rate of 0.11 ft^3/lb/year, the 2005 gas generation rate from the NSPS permit application, are plotted in Figure 5-8. As can be seen by review of the figure, due to the close proximity of the radii of influence of the wells surrounding well No. 23, the amount of vacuum applied to the surrounding wells will most likely influence the pressure observed at gas extraction well 23. A reduction in the vacuum in these surrounding wells would likely result in a further increase in the pressure measured at well No. 23. This test result indicates that shutting down the vacuum to a well can result in a build up of positive pressure within a timeframe of several hours.

6. REVIEW OF FACTORS AFFECTING STABILITY

6.1 Overview

The stability of the final cover system in the Phase 2 area of the Piedmont Landfill is evaluated in this section. The objective is to identify the cause or causes of movement observed in January 2005 and to assess the overall stability of the Phase 2 area slope in its current condition. The stability analyses are based on the material and interface properties measured on samples collected from the field and reported in Section 4. The stability analysis of the cover system presented in the 24 May 2004 technical demonstration is reviewed in Section 6.2. Stability of the "as installed" cover system is reviewed in Section 6.3. The impact of the August 2004 movement on stability is addressed in Section 6.4. Factors affecting the stability of the final cover system in January 2005 are addressed in Section 6.5.

6.2 Review of 24 May 2004 Technical Demonstration

As noted in Section 2.1, in May 2004, GeoSyntec, on behalf of WM, prepared a modification request to the final cover system design for review and approval by NCDENR. The modification replaced the then-permitted cover system geomembrane and overlying drainage layer (40-mil thick textured geomembrane overlain by a double-sided geocomposite drainage layer) with a 50-mil thick LLDPE textured geomembrane with an integrated drainage layer on the top side of the geomembrane and an overlying nonwoven geotextile filter layer. The May 2004 technical demonstration for this modification is included in Appendix B. As part of the technical demonstration, laboratory direct shear interface testing was performed to provide input parameters for the slope stability evaluation of the alternate design. The results of these laboratory tests are also included in Appendix B. Based on these results, the steepest slopes of Phase 2 final cover system (3H:1V) are calculated to have a "design stage" factor of safety against sliding of 1.73 based on peak interface shear strengths. The calculated factor of safety based on the measured large-displacement shear strength was 1.24. These slope stability analysis results are presented in Appendix F of this report.

6.3 Installation Affects on Cover System Stability

As part of the field investigation described in Section 3 of this report, samples of final cover system components were collected in Phase 2 areas that had undergone movement and in an area that had not undergone movement. Three test pits were advanced for purposes of evaluating the condition of the cover system components and to obtain samples for laboratory evaluation. The locations of the three test pits are described in Section 3.4.2 and include two areas where the cover system had undergone downslope movement and one area where no movement had been observed. The test pit locations are shown on Figure 3-1 of this report.

As previously described in Section 4.4 of this report, the sample from Test Pit TP-1 (from an area of no movement) was tested to evaluate whether the geomembrane material installed at the site had interface shear strength characteristics similar to those exhibited in the testing presented in the May 2004 design-phase technical demonstration document. In addition, a geomembrane sample from a roll of unused material at the site was also tested. The results of these interface shear tests are summarized in Table 4-3. The test results show that the unused sample of geomembrane from the stockpile had similar interface strength characteristics as the geomembrane sample tested as part of the technical demonstration in May 2004. This result implies that the geomembrane material delivered to the Piedmont Landfill site for the Phase 2 closure had "as delivered" interface characteristics conforming to the August 2004 technical demonstration document.

The interface shear strength for the installed geomembrane from Test Pit TP-1, the area that had not undergone movement, resulted in lower interface shear strengths than the pre-construction and unused stockpile geomembrane samples. This result suggests that the process of geomembrane installation alone reduced the geomembrane texturing (also called "roughness") in comparison to the amount of texturing on the factory delivered material. This is an unexpected result. The asperity height of the texturing on the geomembrane samples from the test pits and the unused sample from the stockpile were also measured as part of the post-movement investigation. The results of these measurements are summarized in Table 4-2. From this table it can be seen that the installed geomembrane samples have texturing with a shorter asperity height than the uninstalled samples. It appears from these results that the geomembrane installation process reduced the asperity height and the associated "degree of texturing" of the LLDPE AGRU Microdrain product by about 20 percent.

Based on these results, the cover system geomembrane experienced an "installation affect" wherein the installed interface shear strengths generated by this material are lower than the interface strengths generated when unused material is tested. The influence of this "installation affect" on the slope stability factor of safety was evaluated using the interface shear strength data presented in Table 4-3. Using this data, the average measured interface shear strengths for the test pit samples, and considering the slow interface shear test results from Table 4-3, a peak secant friction angle of 24.5 degrees and a large-displacement friction angle of 19.6 degrees are calculated. The "as-installed" factor of safety for the steepest slopes at the Piedmont Landfill (3H:1V) is 1.41 for peak strength conditions and 1.11 for large-displacement conditions (see Appendix F for slope stability calculation results).

Based on the foregoing, after installation of the Phase 2 area geomembrane component of the final cover system, the steepest slopes in the area (3H:1V) had a factor of safety based on peak interface shear strength condition of 1.41. Other slopes in the area are slightly flatter (3.25H:1V) and based on the interface shear strength results have an as-installed factor of safety of 1.53. Even flatter slopes will have proportionally higher as-installed factors of safety.

Based on these analysis, it is concluded that there was a reduction in the cover system slope stability factor of safety from that calculated in the 24 May 2005 technical demonstration document, due to an installation-induced change in the degree of texturing of the geomembrane. This installation affect reduced the calculated factor of safety on a 3H:1V slope from a value of 1.73 to a value of 1.41.

6.4 Impact of August 2004 Movement

The investigation into the Phase 2 cover system movements also considered whether or not the geomembrane uplifting and movement that occurred in August 2004 produced displacement and shearing at the geomembrane-GCL interface that exceeded the peak interface shear strength and generated large-displacement interface conditions. An interface shear test designed to evaluate the potential effect of this movement on the peak interface shear strength was undertaken as described in Section 4.4 of this report. The test results produced an approximate 10 percent reduction in peak shear strength between the first and second shearing episodes. The magnitude of this reduction needs to be confirmed by additional analysis. Using this result as a tentative value, the post-August factor of safety for 3H:1V slopes in the area that underwent uplift and movement in August 2004 is calculated to be 1.28. The calculations are presented in Appendix F.

6.5 Potential Condition Affecting Stability in January 2005

Two potential conditions that could have triggered the January 2005 cover system movement are the precipitation event that occurred on the night of 13 January 2005, and the possible build up of gas pressure beneath the final cover system. The potential impacts of these two conditions are evaluated below.

Based on the chronology described in Section 2, the evening before the failure occurred the site was subjected to approximately 1.1 inches of precipitation from approximately 8 pm on 13 January to 9 am on 14 January 2005. Precipitation data and typical precipitation curves are presented in Appendix E. A three hour storm with a return period of one year is expected to yield approximately 1.6 inches of precipitation. The surface water management system is designed for a 24 hour storm with a 25 year return period. This design storm is expected to yield approximately 6.5 inches of precipitation. The observed storm event of 1.1 in 13 hours is significantly less than the storm event for which the surface water management system is designed. A further review of the precipitation data indicates that the site was subjected to a similar storm in December 2004. The fact that precipitation did not cause instability during the earlier event is an indication that the precipitation that occurred on 13 and 14 January is not the cause of the observed movement.

The possibility of a storm induced failure was further ruled out because the post-movement investigation revealed that there were no signs of movement of the protective cover soil component of the final cover system relative to the underlying geosynthetic components. In precipitation/seepage induced slope movements, the protective cover soil will move relative to the underlying uppermost hydraulic barrier layer, in this case the geomembrane. Several studies have shown that seepage-induced stresses in protective cover soil layers have no significant effect on the factor of safety against sliding for interfaces below the geomembrane component of the cover system. For these reasons, it is concluded that precipitation did not induce the downslope movement of the cover system observed in January 2005.

Another possible trigger for the January 2005 movements is the buildup of gas pressure beneath the geomembrane. In order to understand the potential impact of gas pressure buildup on the stability of the final cover system, the pressure required to reduce the cover system slope stability factor of safety to 1.0 was calculated. Based on the foregoing discussion of interface shear strength results, slope areas at 3H:1V that did not undergo movement prior to January 2005 had a calculated pre-failure factor of safety of 1.41, whereas areas that underwent uplift and movement in August 2004 had a calculated factor of safety of 1.28. For these factors of safety, gas pressures equivalent to approximately a 12 in. water column and 8 in. water column are required, respectively, to reduce the factor of safety to a value of 1.0.

The gas pressure buildup in August 2004 exceeded these values as evidenced by the uplifting of areas of the cover system where protective cover soil had already been placed. The buildup of gas pressure is also demonstrated by the plot in Figure 5-7. Furthermore, the relative co-location of the January 2005 cover system movement episode and the August 2004 cover system movement episode imply that the Phase 2 exit closure area of the landfill is more susceptible to the development of gas uplift pressures than any other portion of the landfill. Based on these observations and the GCCS evaluation described in Section 5, it is concluded that gas uplift pressure is the most likely explanation for the reduction of cover system slope stability factor of safety to a value of 1.0 at the time of cover system movement.

7. SUMMARY AND CONCLUSIONS

7.1 Design Phase

This report has focused on a portion of the final cover system for the Piedmont Landfill that underwent downslope movement in January 2005. The area of movement was approximately seven acres in size, and it was contained within a 27-acre area of final cover system, identified as the Phase 2 final cover system, that was constructed between June and October 2004. All of The components of the Phase 2 final cover system consist of, from top to bottom:

- 24-in thick protective cover soil and topsoil layer;
- 8-oz/sy nonwoven geotextile filter layer (GE180);
- 50-mil thick textured LLDPE geomembrane (Microdrain);
- geosynthetic clay liner (Bentomat ST, installed woven side up); and
- prepared subgrade.

The design details for this Phase 2 final cover system were presented to NCDENR as an alternate to the originally-permitted final cover system in a technical demonstration document submitted to NCDENR on 24 May 2004. NCDENR approved the alternate for Phase 2 on 26 May 2004. As part of the technical demonstration, laboratory direct shear interfaces testing was performed to provide input parameters for the slope stability evaluation of the alternate design. The results of these laboratory tests are included in Appendix B of this report. Based on these test results, the steepest slope of Phase 2 final cover system (3H:1V) have a calculated factor of safety against sliding of 1.73 based on peak interface shear strengths. The calculated factor of safety based on the measured large-displacement shear strength is 1.24. These slope stability analysis results are presented in Appendix F of this report. Both of these calculated factors of safety are within the generally-accepted range for landfill final cover systems. The typical minimum acceptable slope stability factor of safety for cover system slopes and peak shears strength conditions is 1.5. The critical interface with respect to slope stability was found to be between the underside of the geomembrane and the top surface of the GCL.

7.2 Construction Phase

Construction of the 27-acre Phase 2 final cover system commenced in June and was completed in October 2004. Phase 2 construction commenced about the same time waste placement at the landfill ceased, in order to achieve closure of the landfill as quickly as possible after final grades were reached. The Phase 2 cover system is an exit closure, meaning that it is the last area of the Piedmont Landfill to receive final cover. This exit closure condition is noteworthy because the Piedmont Landfill is the first WM landfill in the southeast to undergo a fully-lined exit closure wherein the entire landfill is enclosed with a geomembrane both below the waste (liner system) and above the waste (final cover system).

The landfill gas extraction wells in the Phase 2 area were installed shortly before the start of Phase 2 cover system construction. These wells became operational throughout the Phase 2 construction period (June to October) as placement of the final cover soil was completed and the well header pipes were connected to the GCCS. These extraction wells were installed in accordance with the permitted gas extraction system design and, as a result, the tops of the well screens are approximately 40 ft below the top of the waste. As a consequence of these details, only limited gas extraction occurred in the upper portions of the waste profile in the Phase 2 area prior to and after sealing of the area with the final cover geomembrane. In addition at this time the GCCS was serviced by a single blower and flare with insufficient capacity to fully accommodate the additional gas collected by the new Phase 2 wells.

On 26 August 2004, immediately after the geomembrane was completely installed and seamed in the Phase 2 area, uplift of the geomembrane occurred in an area in which protective cover soil had not yet been placed. In order to control the uplift, the closure construction contractor cut a temporary hole in the exposed geomembrane. Localized temporary uplift of the geomembrane during cap construction is a relatively common occurrence during final cover construction; however, the uplift on 26 August occurred very quickly after completion of geomembrane installation and the uplift covered an area considerably larger than typically observed. This substantial uplift in August 2004 is believed to be due to the combined attributes of the design and operation of the GCCS in this area coupled with the exit closure condition as previously described.

On 28 August 2004, approximately six acres of the final cover system, including the protective cover soil and topsoil layer which had already been placed, underwent uplift and downslope movement. This uplift and downslope movement is attributed to the build up of gas pressure under the cover system as a result of a landfill gas flare-system shutdown on 27 August. The uplift was mitigated by excavating the cover soil and puncturing the geomembrane to relieve the gas pressure. Temporary passive gas vents were then installed. A detailed investigation was undertaken to verify the integrity of the final cover system immediately after the uplift and movement occurred. The results of the investigation were discussed with NCDENR on 21 September 2004. Based on the investigation it was concluded that the geomembrane was effectively intact and did not need to be replaced. A repair program was implemented. The investigation and repair program is documented in the Phase 2 final cover system CQA report.

7.3 Movement of January 2005

Approximately seven acres of the Phase 2 cover system underwent movement in January 2005. The area of movement is shown on Figure 2-4 of this report. It is noted that approximately 12 acres of the Phase 2 cover system constructed on slopes similar to those in the failed area did not undergo movement. The movement took the form of movement at the interface between the lower surface of the geomembrane and the upper surface of the GCL components of the final cover system.

Based on observations made by personnel that visited the Piedmont Landfill site, the cover system movement occurred sometime between 5 p.m. on 13 January and 10 a.m. on 20 January, 2005. WM and Joyce Engineering personnel were on site on 13 January 2005, and the final cover system was observed to be intact. An independent contractor to WM noticed the failed area sometime between 8 a.m. and 10 a.m. on 20 January when he was on site to perform maintenance on the flare system.

Approximately 1.1 inches of precipitation occurred from approximately 8 p.m. on 13 January to 9 a.m. on 14 January 2005 This storm event is significantly smaller than the design storm event for the final cover system. For the reasons described in the next subsection, it is believed that this precipitation event did not cause or significantly contribute to triggering the downslope movement of the final cover system.

After August 2004, WM upgraded the capacity of the GCCS by adding a second blower and flare to the system. Both landfill flares shut down on 14 January, 2005. Based on available information, it is estimated that Flare No. 1 shut down at approximately 2 p.m. This estimate is based on reports by Joyce Engineering that the chart paper was changed out at noon on 14 January and scaling of the chart paper indicates that Flare No. 1 ceased operation 2 to 2.5 hours later. Flare No. 2 is believed to have shut down at approximately 2:10 p.m. This estimate is based on output from the data logger recording for the blower flow rate and flare temperature.

Based on the currently available information, it cannot be clearly established whether the cover system slope movement caused the landfill flare system to be shut down, or, conversely whether the flare system shut down, caused a build up in gas pressure beneath the cover system, resulting in the observed instability and movements. It is noted, however, that in either sequence of events, the build-up of gas pressure on the underside of the Phase 2 area (exit closure area) final cover system is believed to have contributed to the observed instability. The reasons for this belief are described below.

7.4 Evaluation of Potential Cause(s) of Movement

Geosynthetic Interface Shear Strengths: As part of the investigation performed after the downslope cover system movements, samples were collected of the final cover system components in Phase 2 areas that had undergone movement and in areas that had not undergone movement. Three test pits were advanced for purposes of evaluating the condition of the cover system components and to obtain samples for laboratory evaluation. The collection of the samples is described in Section 3.4. The location of the three test pits from which the samples were collected are shown on Figure 3-1 of this report. Test Pit TP-1 is in an area that did not undergo movement in either August 2004 or January 2005. Test Pit TP-2 is in an area that only underwent movement in January 2005. Test Pit TP-3 is in an area that underwent movement in both August 2004 and January 2005. Samples of geomembrane, as well as the other cover system geosynthetics and soils, were obtained from each test pit and transported to a qualified soil-geosynthetics testing laboratory (SGI Testing Services, LLC, Norcross, Georgia) for evaluation.

A series of laboratory interface shear strength tests were performed as part of the investigation. In each test, a sample of geomembrane was sheared against the woven geotextile side of a fresh sample of GCL obtained from the GCL manufacturer to obtain an interface shear strength value. The sample from Test Pit TP-1 (from an area of no movement) was tested to evaluate whether the geomembrane material installed at the site had interface shear strength characteristics similar to those exhibited in the testing presented in the May 2004 design-phase technical demonstration document. The samples from Test Pits TP-2 and TP-3 were tested to evaluate the effect of the January 2005 downslope movement on geomembrane interface shear strength characteristics. Fresh GCL samples were used in these tests due to concern that the GCL samples removed from the test pits and brought to the laboratory had undergone free swell and were no longer suitable for testing. In addition to the geomembrane samples from the test pits, a geomembrane sample from a roll of unused material stockpiled at the site was also tested. The results of these interface shear tests are summarized in Table 4-3 of the report. The following observations are provided with respect to the interface shear strength results:

- The unused sample of geomembrane from the stockpile had similar interface strength characteristics as the geomembrane sample tested as part of the technical demonstration in May 2004. This result supports the conclusion that the geomembrane material delivered to the Piedmont Landfill site for the Phase 2 closure had "as delivered" interface characteristics conforming to the August 2004 technical demonstration document.

- The interface shear strength for the installed geomembrane from Test Pit TP-1, obtained from an area that had not undergone movement, resulted in lower interface shear strengths than the pre-construction and unused stockpile geomembrane samples. This result suggests that the process of geomembrane installation alone reduced the geomembrane texturing (also called "roughness") in comparison to the amount of texturing on the factory delivered material. This is an unexpected result. Based on the average measured interface shear strengths for the test pit samples, and considering the slow interface shear test results from Table 4-3, the "as-installed" calculated factor of safety for the steepest Phase 2 slope at the Piedmont Landfill (3H:1V) is 1.41 for peak strength conditions and 1.11 for large displacement conditions (see Appendix F for slope stability calculation results).

- The interface shear strengths obtained on tests using geomembrane samples from Test Pits TP-2 and TP-3 (i.e., the areas where downslope movement had occurred) produced results that are similar to, and actually slightly higher (stronger) than, those obtained from the test on the sample from Test Pit TP-1. This result implies that both the uplifting and movement that occurred in August 2004, and the downslope movement that occurred in January 2005, did not further reduce the geomembrane texturing beyond the reduction caused by the installation process.

The asperity height of the texturing on the geomembrane samples from the test pits and the unused sample from the stockpile were also measured as part of the post-movement investigation. The results of these measurements are summarized in Table 4-2. From this table, it can be seen that the installed geomembrane samples have texturing with a shorter asperity height than the uninstalled samples. It appears from these results that the geomembrane installation process reduced the asperity height and the associated "degree of texturing" of the LLDPE AGRU Microdrain product by about 20 percent.

Based on the foregoing, after installation of the Phase 2 area geomembrane component of the final cover system, the steepest slopes in the area of movement (3H:1V) had a calculated factor of safety based on peak interface shear strengths of 1.41. Other slopes in the area are slightly flatter (3.25H:1V) and based on the interface shear strength results have an as-installed factor of safety of 1.53. Even flatter slopes will have proportionally higher as-installed factors of safety.

Another question that was considered as part of the investigation is whether or not the geomembrane uplifting and movement that occurred in August 2004 produced displacement and shearing at the geomembrane-GCL interface that exceeded the peak interface shear strength and generated large-displacement interface conditions. Downslope movements at the base of the slope in August were up to several feet, based on the size of observed wrinkles. Based on the field observations, this movement occurred when the geomembrane was being uplifted from the slope and thus occurred under normal stress applied by the cover soil. Some of the movement inferred from the size of the wrinkles is actually is believed to be caused by the accumulation of installation wrinkles that are often observed at the base of the slope. An interface shear test designed to evaluate the potential effect of this movement on the peak interface shear strength was undertaken. The test involved first conducting an interface shear test on fresh samples of geomembrane and GCL to a shear displacement of 3.5 in. After shearing to this displacement, the geomembrane and GCL samples were reset to their original positions, reloaded, and resheared. This sequence of events was meant to simulate interface movement in August 2004, followed by re-application of the cover system normal stresses. The test results produced an approximate 10 percent reduction in peak shear strength between the first and second shearing episodes. By using this result as a tentative value for the purposes of this evalutaion, the post-August factor of safety of 3H:1V slopes in the area that underwent uplift and movement in August 2004 is calculated to have been 1.28. The calculated factor of safety reduction (from 1.41 to 1.28) appears reasonable based on the fact that: (i) between August 2004 and January 2005 there was no observation of slope distress, in the form of tension cracks or compression ridges in the area of interest - this observation implies that the factor of safety in the area had to be enough above the limiting value of 1.0 that incipient failure effects were not present (estimated to occur at a factor of safety of 1.1), and (ii) an approximately 250-ft long portion of the area that failed in January 2005 is outside of the area that underwent movement in August 2004. The applicable interface shear strength in this area prior to January 2005 should have been that associated with a geomembrane-GCL interface that had not previously undergone shearing, and this observation clearly points toward another factor providing the trigger for the downslope movement observed in January 2005.

Rainfall Infiltration and Seepage: Another possibility investigated for this report is that the storm event of 13 January 2005 reduced the slope stability factor of safety. This possibility was ruled out, however, because the post-movement investigation revealed that there were no signs of movement of the protective cover soil component of the final cover system relative to the underlying geosynthetic components. In precipitation/seepage induced slope movements, the protective cover soil will move relative to the underlying uppermost hydraulic barrier layer, in this case the geomembrane. Several studies have shown that seepage-induced stresses in protective cover soil layers have no significant effect on the factor of safety against sliding for interfaces below the geomembrane component of the cover system. It is also noted that the site had been subjected to a similar precipitation event in December 2004, but did not undergo movement or exhibit signs of incipient movement at that time. For these reasons, it is concluded that precipitation did not induce the downslope movement of the cover system observed in January 2005.

Buildup of Gas Pressure: Another possible trigger for the January 2005 movement is the buildup of gas pressure beneath the geomembrane. To investigate this possibility, the landfill GCCS operation records were reviewed. On 5 January 2005, the recorded blower flow rate for Flare No. 1 was 429 scfm. On 13 and 14 January 2005, the recorded blower flow rate for Flare No. 2 was approximately 600 scfm. It thus appears that for at least a portion of January 2005, the combined flow rate of the two blowers was only approximately 55 percent of the estimated 2005 gas generation rate from the NSPS permit application of 1869 scfm. The possibility that the rate at which gas was withdrawn from the landfill was significantly less in January 2005 than the gas generation rate is supported by gas monitoring data from 5 January which shows that many of the gas extraction wells in the landfill were under stressed at that time. Withdrawal of gas from the landfill at a rate lower then the generation rate can result in the build up of excess landfill gas in the landfill over time. It is believed that the most critical area of the landfill with respect to excess gas buildup and cover system slope stability would be in the exit closure area, due to the conditions described previously. The fact that the gas uplift in August 2004 occurred in much the same area as the January 2005 movement supports the conclusion that the exit closure area is the most sensitive portion of the landfill with respect to the potential for gas pressure buildup in an understressed extraction environment. The impact of such a build up would be compounded in the event that one or both flares temporally shut down.

In order to understand the potential impact of gas pressure buildup on the stability of the final cover system, the pressure required to reduce the cover system slope stability factor of safety to 1.0 was calculated. Based on the foregoing discussion of interface shear strength results, slope areas at 3H:1V that did not undergo movement prior to January 2005 had a calculated pre-failure factor of safety of 1.42, whereas areas that underwent uplift and movement in August 2004 had a calculated factor of safety of 1.28. For these factors of safety, gas pressures equivalent to approximately a 12 in. water column and 8 in. water column are required, respectively, to reduce the factor of safety to a value of 1.0. The gas pressure buildup in August 2004 exceeded these values as evidenced by the uplifting of areas of the cover system where protective cover soil had already been placed.

Summary: Based on the above information it appears that a number of factors likely contributed to the January 2005 movement of the final cover system, with landfill gas pressure buildup having the most important role.

A reduction in interface strength characteristics from the design interface strength characteristics resulted from the installation process discussed previously. The interface strength properties were possibly further reduced in the area of the August 2004 uplift and movement as a result of this event. Based on the analysis presented in this report, it appears likely that landfill gas was gradually accumulating in the landfill prior to January 2005 because the gas removal rate from the landfill was only approximately 55 percent of the calculated gas generation rate. This gas buildup would have been most pronounced in the exit closure area (i.e., the Phase 2 closure area) for the reasons described previously. A gradual pressure buildup would have led to a gradual reduction in slope stability factor of safety, ultimately producing the observed slope movements. It is noted, however, that evidence of a gradual reduction in slope stability factor of safety that might be associated with a gradual buildup of gas pressure was not observed. Based on this observation, a more rapid gas pressure buildup and slope movement associated with the flare system shutdown on 14 January 2005 cannot be ruled out.

7.5 Evaluation of Areas that Have Not Moved

The primary focus of the investigation described in this report has been on the approximately seven acres of Phase 2 area final cover system that underwent downslope movement in January 2005. In addition, the stability of the remainder of the Piedmont Landfill (approximately 57 acres) that did not undergo movement was also considered. The evaluation of the remainder of the final cover system involved several specific landfill areas, including: the Phase 2 final cover system constructed in 2004 that did not move; the Southern and Eastern Side Slope final cover system constructed in 2003, and the Phase 1 final cover system constructed in 1996 and 1997. This evaluation is predicated on the assumption that positive landfill gas pressures will not develop beneath the cover system.

Approximately 27 acres of the Phase 2 final cover system was constructed in 2004. Of this total area, approximately eight acres of cover system are on relatively flat slopes, typical 15 percent or flatter. The remainder of the 19 acres is comprised of slopes that are on the order of 3H:1V to 3.25H:1V. A significant portion of the Phase 2 slope area outside of the January 2005 area of movement is inclined at 3.25H:1V based on as-built survey data. Using the average interface shear strength parameters for the test pit samples in Table 4-3 under the slow shearing condition, the calculated factor of safety against sliding for a slope inclined at 3.25H:1V is 1.53 for peak conditions and 1.20 for large displacement conditions. As discussed previously, the calculated slope stability factor of safety for a slope inclined at 3H:1V is 1.41 for peak conditions and 1.11 for large displacement conditions. Based on the above calculated results, the Phase 2 final cover system in areas outside the January 2005 movement area is considered to have an adequate factor of safety. Recent visual observations of the Phase 2 areas that did not undergo movement in January 2005 did not show any signs of distress or cracking of the cover soil that would be associated with potential instability of the cover system. It is important to note that since January 2005, WM has progressively improved operation of the Piedmont Landfill gas extraction system and indications are that the system is not presently understressed as was the case in January 2005. This improved condition is important to the stability of the Piedmont Landfill final cover system.

The Southern and Eastern side slope final cover system constructed in 2003 is different from that in the Phase 2 area in that the AGRU geomembrane is in contact with the nonwoven geotextile surface of the GCL. Based on the interface shear strength testing presented in Appendix B this report, a textured geomembrane in contact with the nonwoven geotextile surface of GCL will have a larger interface shear strength than a textured geomembrane in contact with the woven geotextile surface of the GCL. Therefore, the Southern and Eastern side slope final cover system will have a larger factor of safety than that calculated for the Phase 2 final cover system in the areas that did not fail. Also, recent visual observations of this area did not show any signs of distress or cracking of the cover soil that would be associated with potential instability of the cover system.

The Phase 1 final cover system constructed in 1996 and 1997 has been in place for approximately eight years and has not showed signs of distress or movement. In this area, a textured geomembrane is in contact the nonwoven geotextile surface of a GCL. For all the reasons described above (interface shear strengths, recent visual observations, improved gas system operations), the Phase 1 final cover system is judged to be stable with an adequate factor of safety.

TABLES

TABLE 3-1

GENERAL SAMPLE COLLECTION OBSERVATIONS
2 FEBRUARY 2005 SAMPLE COLLECTION EFFORT

	Test Pit 1	**Test Pit 2**	**Test Pit 3**
Descriptive Location	**No Observed Cover System Movement**	**January 2005 Cover System Movement**	**August 2004 and January 2005 Cover System Movement**
Approximate Protective Cover Soil Thickness	2 feet	3 feet	4 feet
Geomembrane	Top surface generally dry	Top surface wet, a few wrinkles, bentonite film on bottom surface	Top surface wet, can observe free water, bentonite film on bottom surface
GCL	Hydrated and soft	Hydrated and soft, thin bentonite film on top of woven geotextile	Hydrated and soft, thin bentonite film on top of woven geotextile
Landfill Gas	Intermittent odor beneath geosynthetics	Geomembrane ballooning, obvious gas pressure release upon cutting geomembrane, landfill gas odor observed	Geomembrane ballooning, obvious gas pressure release upon cutting geomembrane, GCL ballooning observed, landfill gas odor observed

Note: Gas Extraction wells in vicinity of Test Pit 2 and Test Pit 3 were brought back into service approximately one week after sample collection.

TABLE 4-1

MOISTURE CONTENT MEASUREMENTS

Material	Location		
	Test Pit 1	Test Pit 2	Test Pit 3
Protective Cover Soil	15.6 15.7	14.3 12.2	13.6 16.7
Foundation Soil	15.7 12.3	11. 11.7	11.0 10.2
GCL	85.1	86.5	73.0 72.9

1, Moisture content was measured in accordance with ASTM D2216, Standard Test Method for Laboratory Determination of Water (Moisture) Content of Soil and Rock by Mass.
2. Where two values a[...] ontent measurements for t[...]

Foundation Soil TP1
Stated incorrectly
12.3 should read 17.3

TABLE 4-2

ASPERITY HEIGHT OF SAMPLES OF GEOMEMBRANE

Material	Avg. Asperity Height, mil/s
TP-1	14.2
TP-2	14.0
TP-3	13.6
Stockpile Sample	17.1

1, Asperity height measured in general accordance with GRI GM12, Asperity Measurement of Textured Geomembranes.
2. The average values are based on 10 measurements for each sample.

TABLE 4-3

INTERFACE SHEAR STRENGTH TEST RESULTS

Material Source	Test No.	Peak Secant Friction Angle[2], degree	Large-Displacement Friction Angle[2], degree	Test Condition[3]
TP-1	1A	21.8	18.2	fast
TP-1	1B	23.8	18.9	slow
TP-2	2A	22.6	17.4	fast
TP-2	2B	24.4	20.1	slow
TP-3	3A	23.0	16.3	fast
TP-3	3B	25.4	19.9	slow
Stockpile	4A	28.4	20.6	fast
Factory[4]	_	29.0	21.6	fast

1. All test performed in general accordance with ASTM D 5321 at the SGI Testing Services, LLC laboratory in Norcross, Georgia in February and March 2005.
2. Measured at a normal stress of 240 psf.
3. fast: consolidated 0.5 hrs before shearing at 0.04 in./min.
   slow: consolidated 24 hrs before shearing at 0.004 in./min.
4. Results are for testing performed as part of the technical demonstration dated 24 May 2004 (see Appendix B).

TABLE 4-4

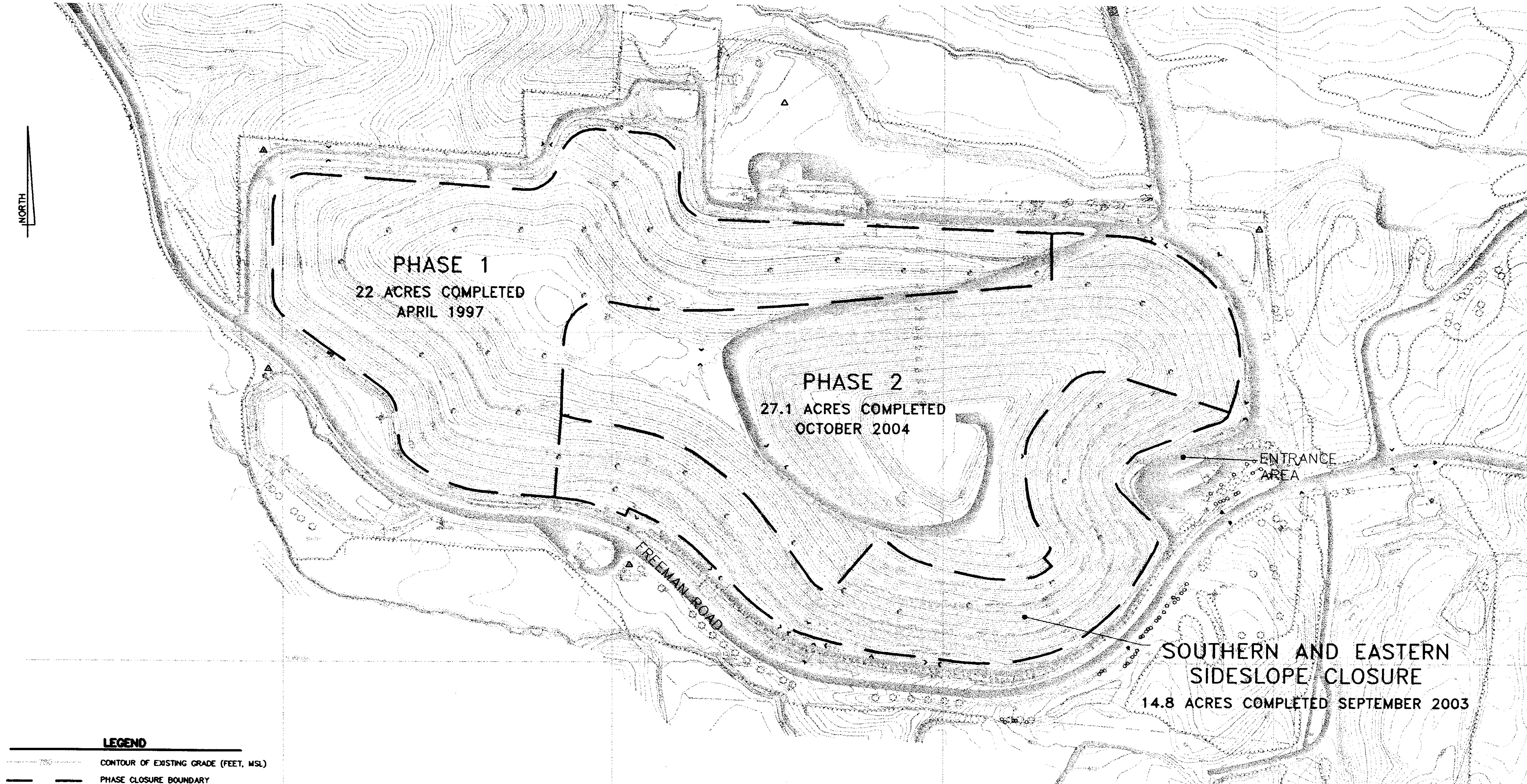
INTERFACE SHEAR STRENGTH TEST RESULTS FOR SIMULATION OF UPLIFT AND MOVEMENT OF COVER SYSTEM

Shear Episode	Peak Secant Friction Angle[2], degrees	Large Displacement Friction Angle[2], Degrees
Initial[4]	27.3	25.0
Second[5]	25.4	23.0

1. All tests performed in general accordance with ASTM D5321 at SGI Testing Services, LLC in Norcross, Georgia in March 2005.
2. Measured at a normal stress of 240 psf.
3. Consolidated for 0.5 hours before shearing at a rate of 0.04 in./min.
4. Sample sheared in the initial test to a displacement of 3.5 in.
5. Sample unloaded, reset, reloaded, and resheared to a displacement of 3.5 in.

FIGURES

INVESTIGATION REPORT
PIEDMONT LANDFILL
CLOSURE COVER PHASING



LEGEND

- CONTOUR OF EXISTING GRADE (FEET, MSL)
- PHASE CLOSURE BOUNDARY
- FENCE LINE
- TREE / BRUSH LINE
- EXISTING ACCESS ROAD
- TREE
- EXISTING GAS WELL
- EXISTING MONITORING WELL

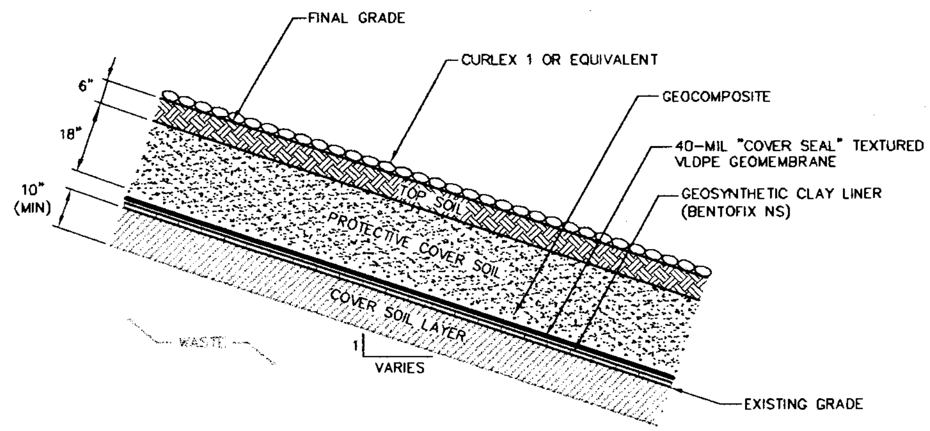
0 300
HORIZONTAL SCALE IN FEET

BASE TOPOGRAPHIC MAP IS FROM AERIAL PHOTOGRAPHY DATED JANUARY 2004

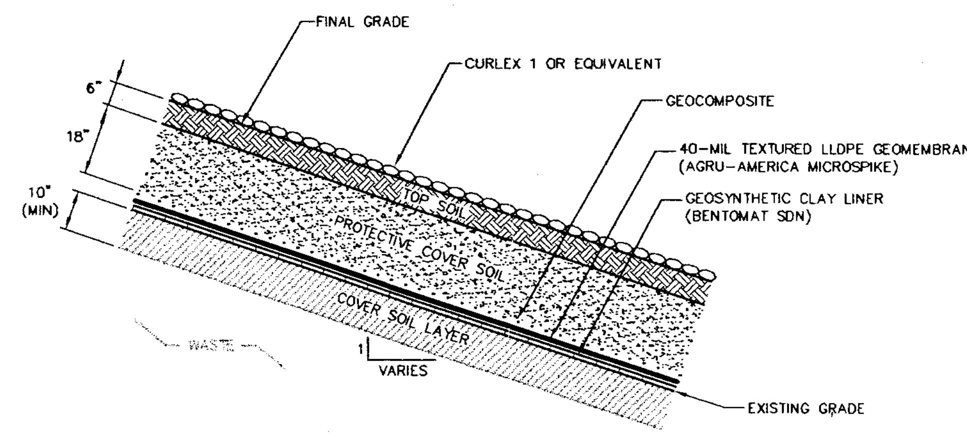
GEOSYNTEC CONSULTANTS
ATLANTA, GA

DATE: 17 MARCH 2005	SCALE: 1" = 300'
PROJECT NO. NCP2005-3184	FILE NO. 3184-F001.DWG
DOCUMENT NO. GA050128	FIGURE NO. 2-1

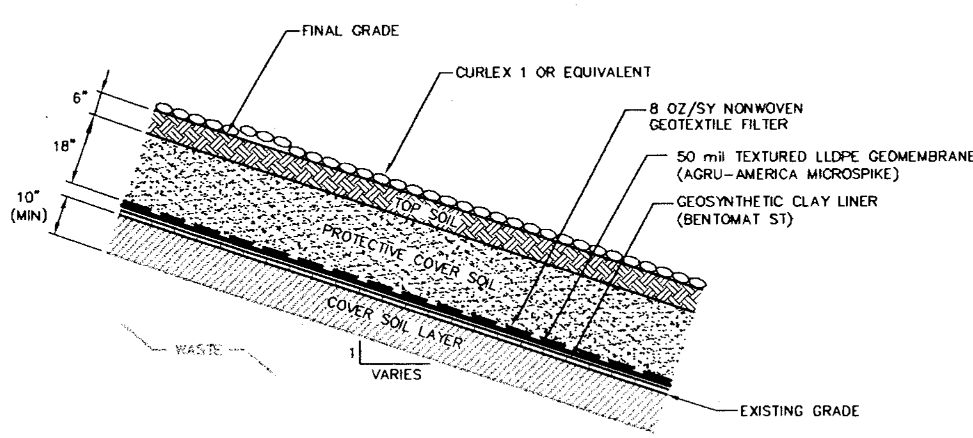
INVESTIGATION REPORT
PIEDMONT LANDFILL
COVER SYSTEM DETAILS



DETAIL
PHASE 1 FINAL CLOSURE (COMPLETED APRIL 1997)
SCALE: 1" = 2'



DETAIL
SOUTHERN AND EASTERN SIDESLOPE (COMPLETED SEPTEMBER 2003)
SCALE: 1" = 2'

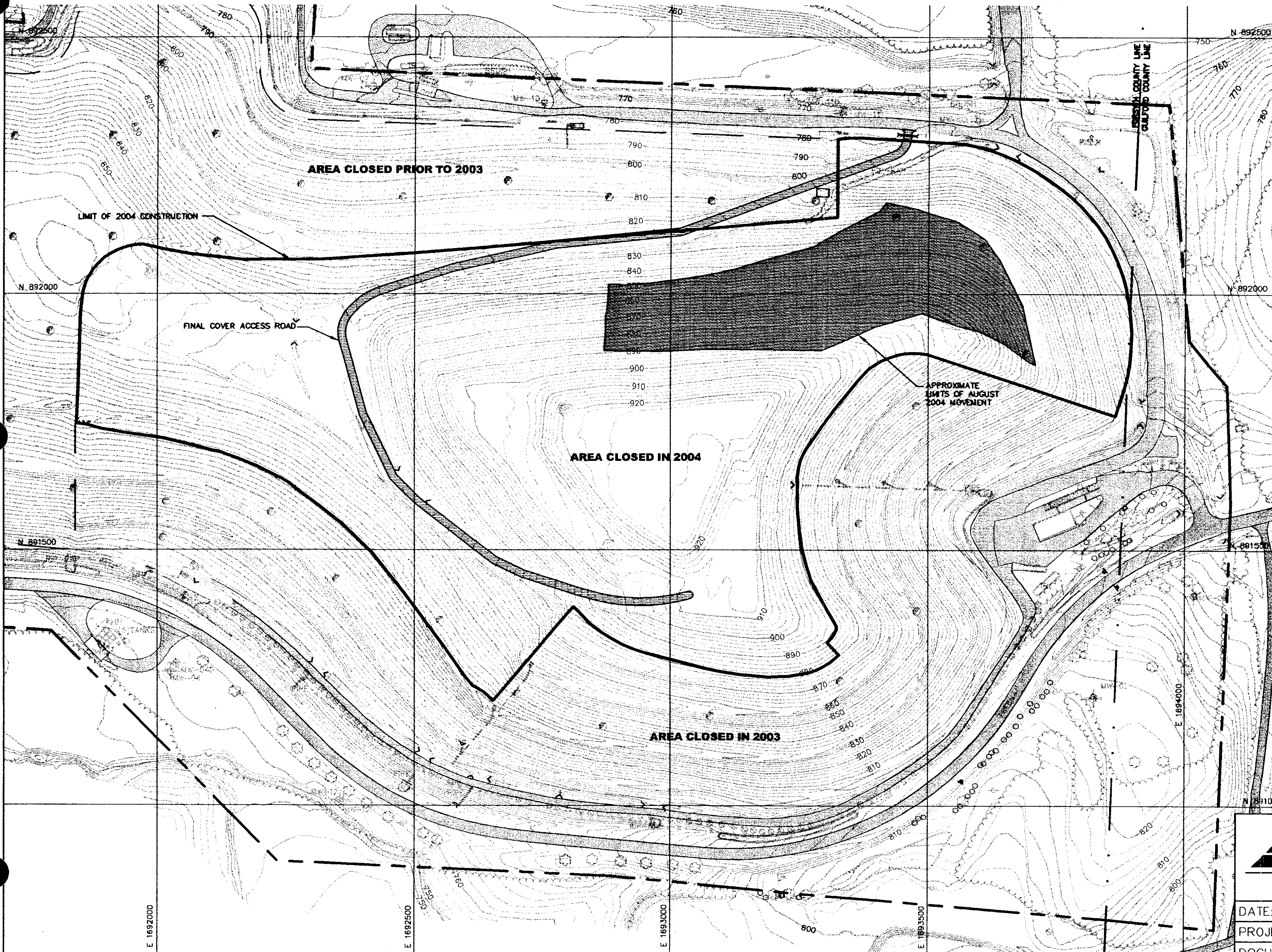


DETAIL
PHASE 2 FINAL CLOSURE (COMPLETED OCTOBER 2004)
SCALE: 1" = 2'

GEOSYNTEC CONSULTANTS
ATLANTA, GA

DATE: 17 MARCH 2005	SCALE: 1" = 2'
PROJECT NO. NCP2005-3184	FILE NO. 3184-F002.DWG
DOCUMENT NO. GA050128	FIGURE NO. 2-2

INVESTIGATION REPORT
PIEDMONT LANDFILL
APPROXIMATE LIMIT OF COVER SYSTEM UPLIFTING AND MOVEMENT – AUGUST 2004



NORTH

LEGEND

780	CONTOUR OF EXISTING GRADE (FEET, MSL)
	COUNTY BOUNDARY
	PHASE CLOSURE BOUNDARY
	FENCE LINE
	TREE / BRUSH LINE
	FINAL COVER ACCESS ROAD
	EXISTING ACCESS ROAD
	APPROXIMATE LIMITS OF CLOSURE AREAS
	TREE
	EXISTING GAS WELL
	EXISTING MONITORING WELL
	LIMIT OF CONSTRUCTION

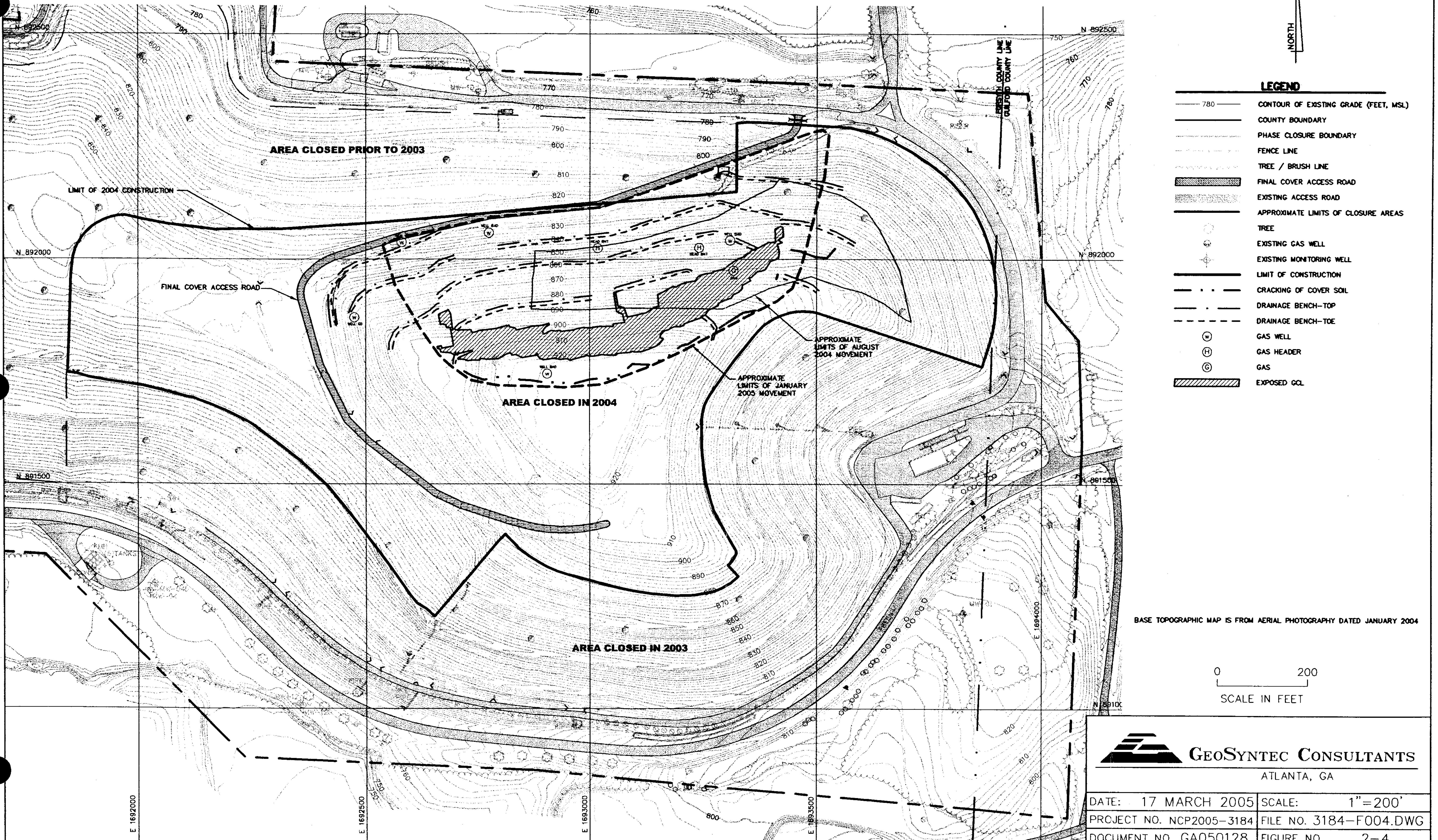
BASE TOPOGRAPHIC MAP IS FROM AERIAL PHOTOGRAPHY DATED JANUARY 2004

0 200
SCALE IN FEET

GEOSYNTEC CONSULTANTS
ATLANTA, GA

DATE: 17 MARCH 2005	SCALE: 1"=200'
PROJECT NO. NCP2005-3184	FILE NO. 3184-F003.DWG
DOCUMENT NO. GA050128	FIGURE NO. 2-3

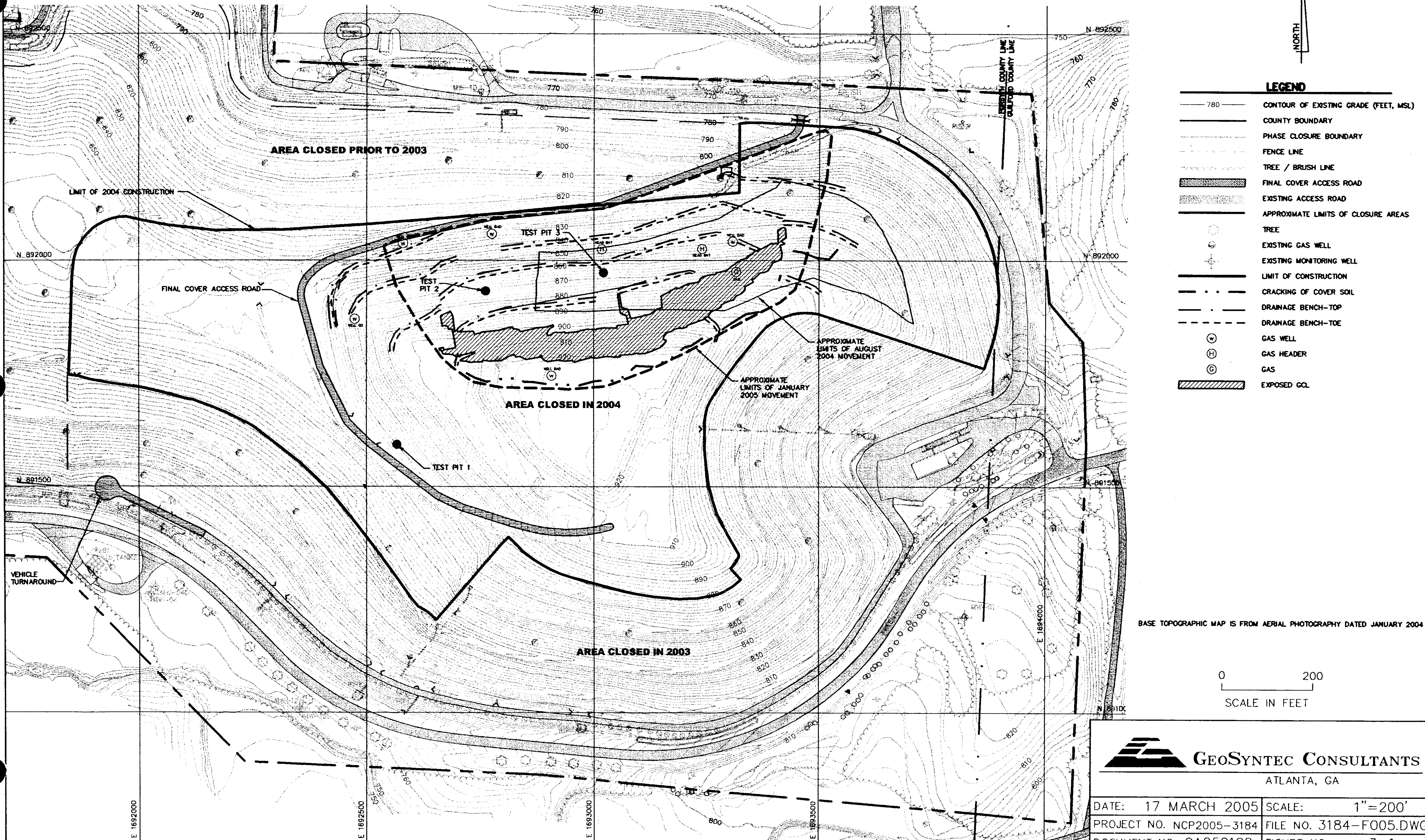
INVESTIGATION REPORT PIEDMONT LANDFILL
APPROXIMATE LIMIT OF COVER SYSTEM MOVEMENT – JANUARY 2005



GEOSYNTEC CONSULTANTS
ATLANTA, GA

DATE: 17 MARCH 2005	SCALE: 1"=200'
PROJECT NO. NCP2005-3184	FILE NO. 3184-F004.DWG
DOCUMENT NO. GA050128	FIGURE NO. 2-4

INVESTIGATION REPORT PIEDMONT LANDFILL
TEST PIT LOCATIONS

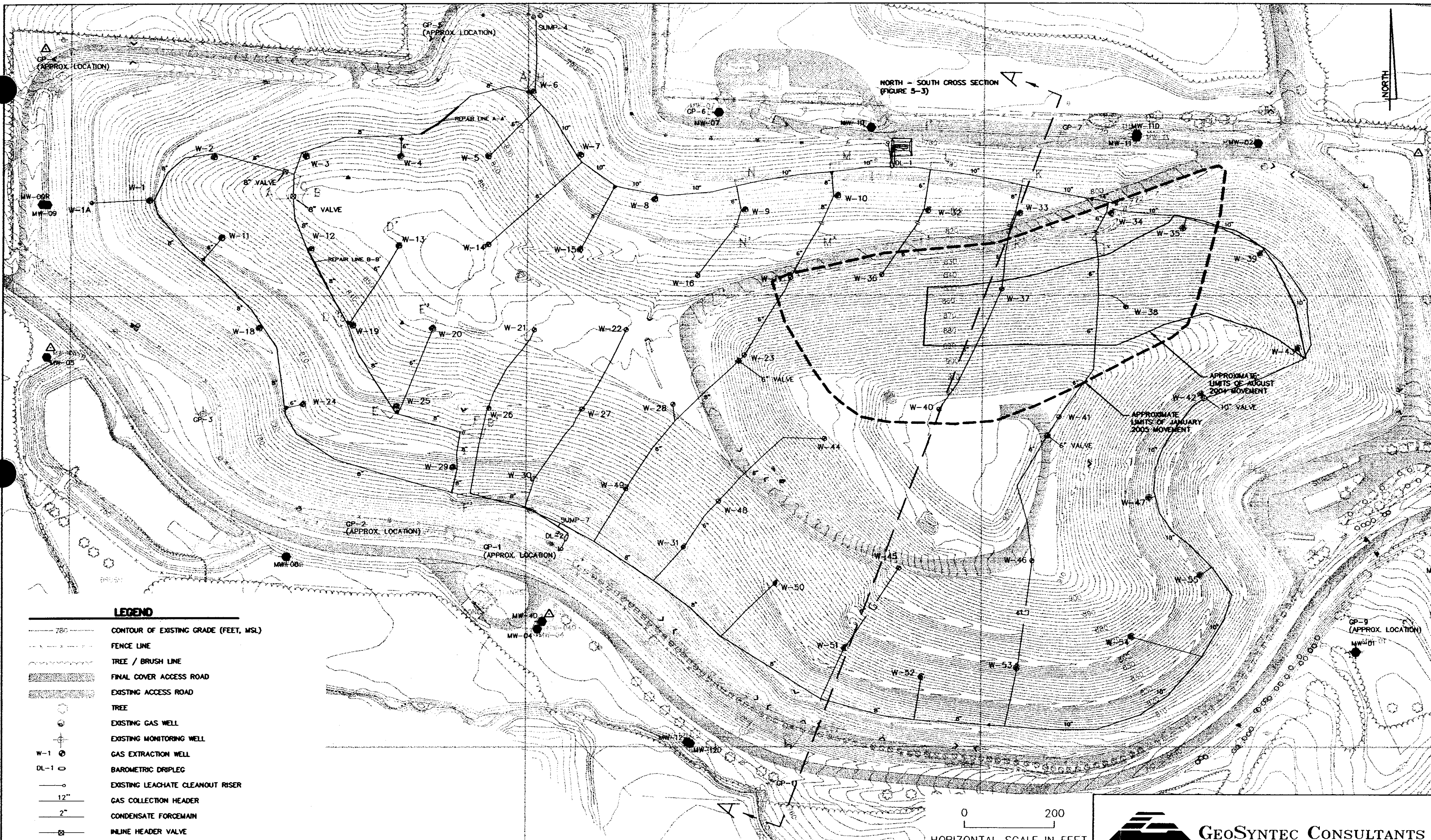


BASE TOPOGRAPHIC MAP IS FROM AERIAL PHOTOGRAPHY DATED JANUARY 2004

0 200
SCALE IN FEET

GEOSYNTEC CONSULTANTS
ATLANTA, GA

DATE: 17 MARCH 2005	SCALE: 1"=200'
PROJECT NO. NCP2005-3184	FILE NO. 3184-F005.DWG
DOCUMENT NO. GA050128	FIGURE NO. 3-1



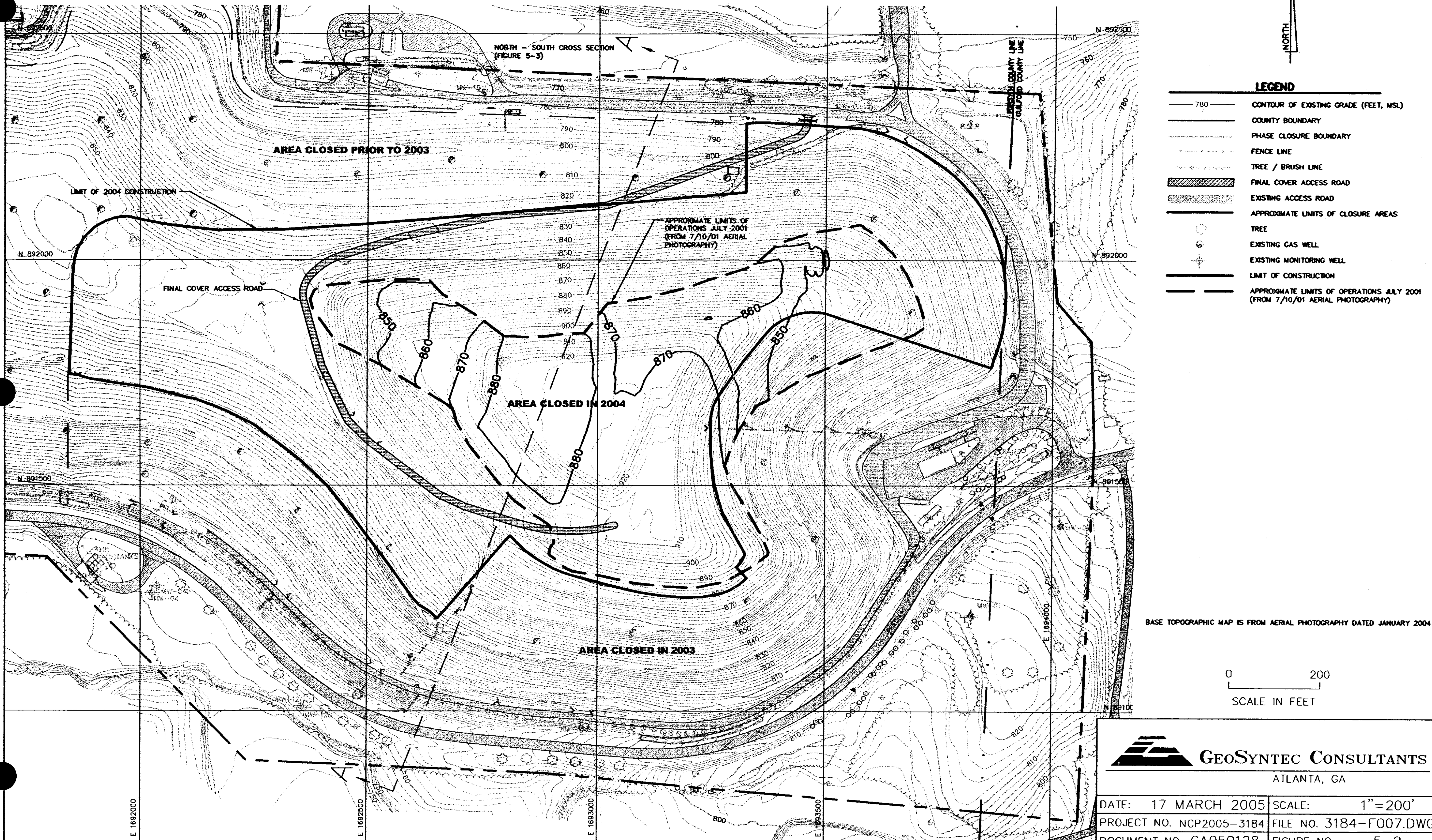
INVESTIGATION REPORT
PIEDMONT LANDFILL
GAS EXTRACTION WELL LOCATIONS

1. BASE TOPOGRAPHIC MAP IS FROM AERIAL PHOTOGRAPHY DATED JANUARY 2004
2. LANDFILL GAS SYSTEM AS-BUILT PROVIDED BY JOYCE ENGINEERING OF GREENSBORO, NC.

GEOSYNTEC CONSULTANTS
ATLANTA, GA

DATE: 17 MARCH 2005	SCALE: 1" = 200'
PROJECT NO. NCP2005-3184	FILE NO. 3184-F006.DWG
DOCUMENT NO. GA050128	FIGURE NO. 5-1

INVESTIGATION REPORT
PIEDMONT LANDFILL
HISTORICAL AREAS OF OPERATION



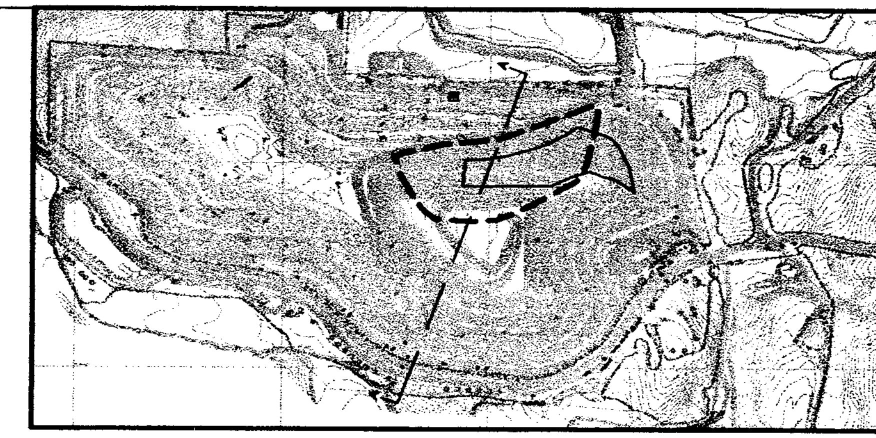
BASE TOPOGRAPHIC MAP IS FROM AERIAL PHOTOGRAPHY DATED JANUARY 2004

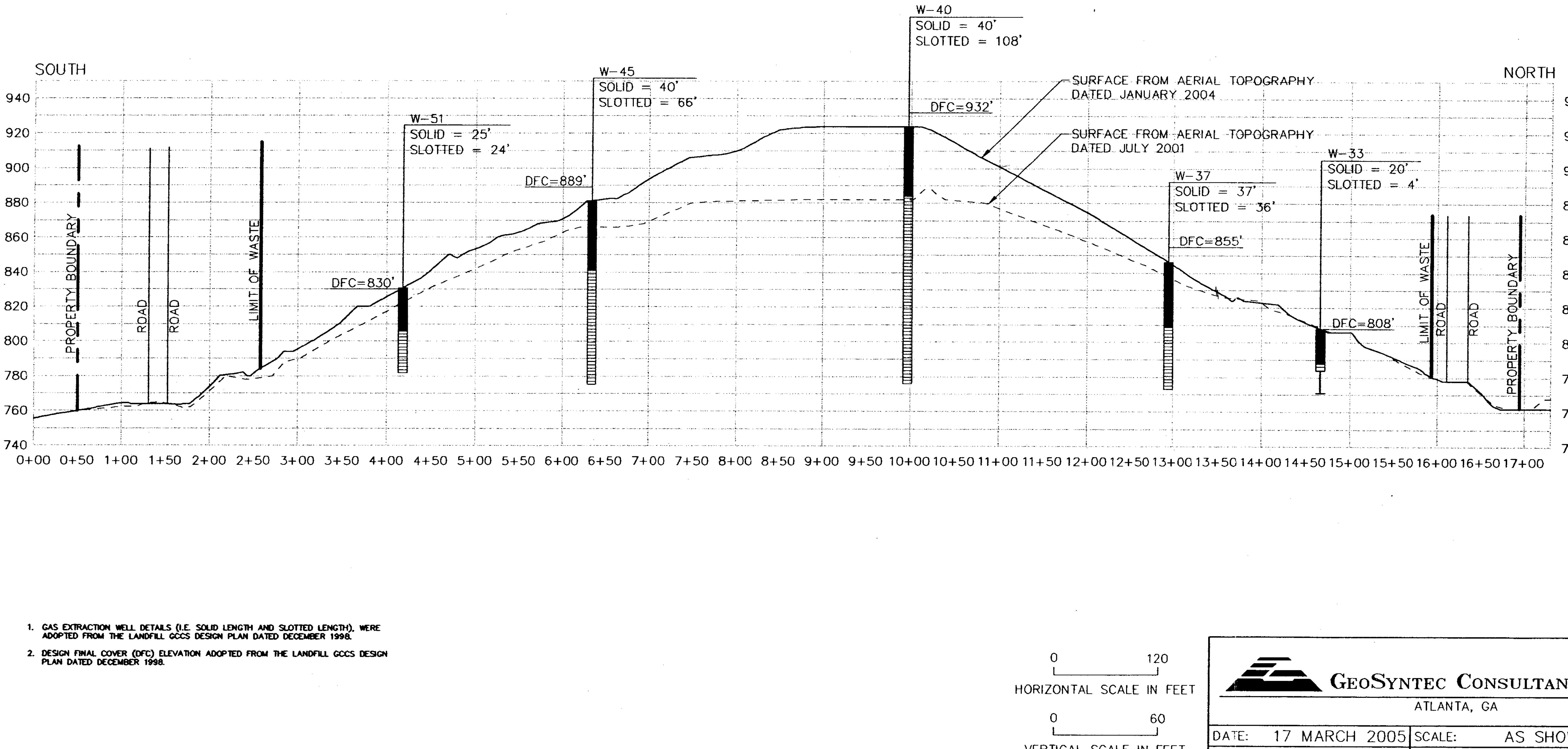
0 200
SCALE IN FEET

GEOSYNTEC CONSULTANTS
ATLANTA, GA

DATE: 17 MARCH 2005	SCALE: 1"=200'
PROJECT NO. NCP2005-3184	FILE NO. 3184-F007.DWG
DOCUMENT NO. GA050128	FIGURE NO. 5-2

INVESTIGATION REPORT
PIEDMONT LANDFILL
NORTH – SOUTH CROSS SECTION A



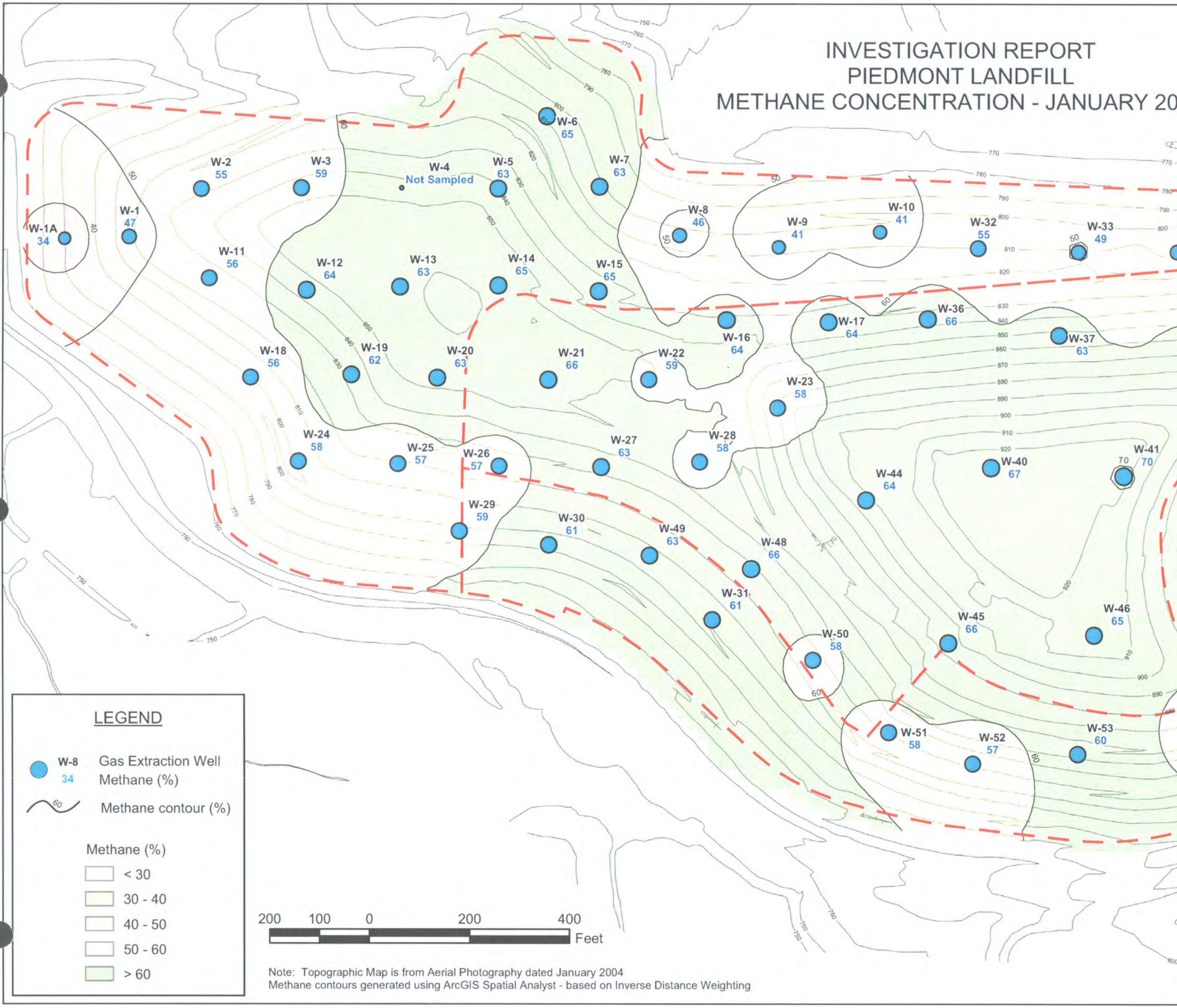


1. GAS EXTRACTION WELL DETAILS (I.E. SOLID LENGTH AND SLOTTED LENGTH), WERE ADOPTED FROM THE LANDFILL GCCS DESIGN PLAN DATED DECEMBER 1998.
2. DESIGN FINAL COVER (DFC) ELEVATION ADOPTED FROM THE LANDFILL GCCS DESIGN PLAN DATED DECEMBER 1998.

GEOSYNTEC CONSULTANTS
ATLANTA, GA

DATE:	17 MARCH 2005	SCALE:	AS SHOWN
PROJECT NO.	NCP2005-3184	FILE NO.	3184-F006.DWG
DOCUMENT NO.	GA050128	FIGURE NO.	5-3

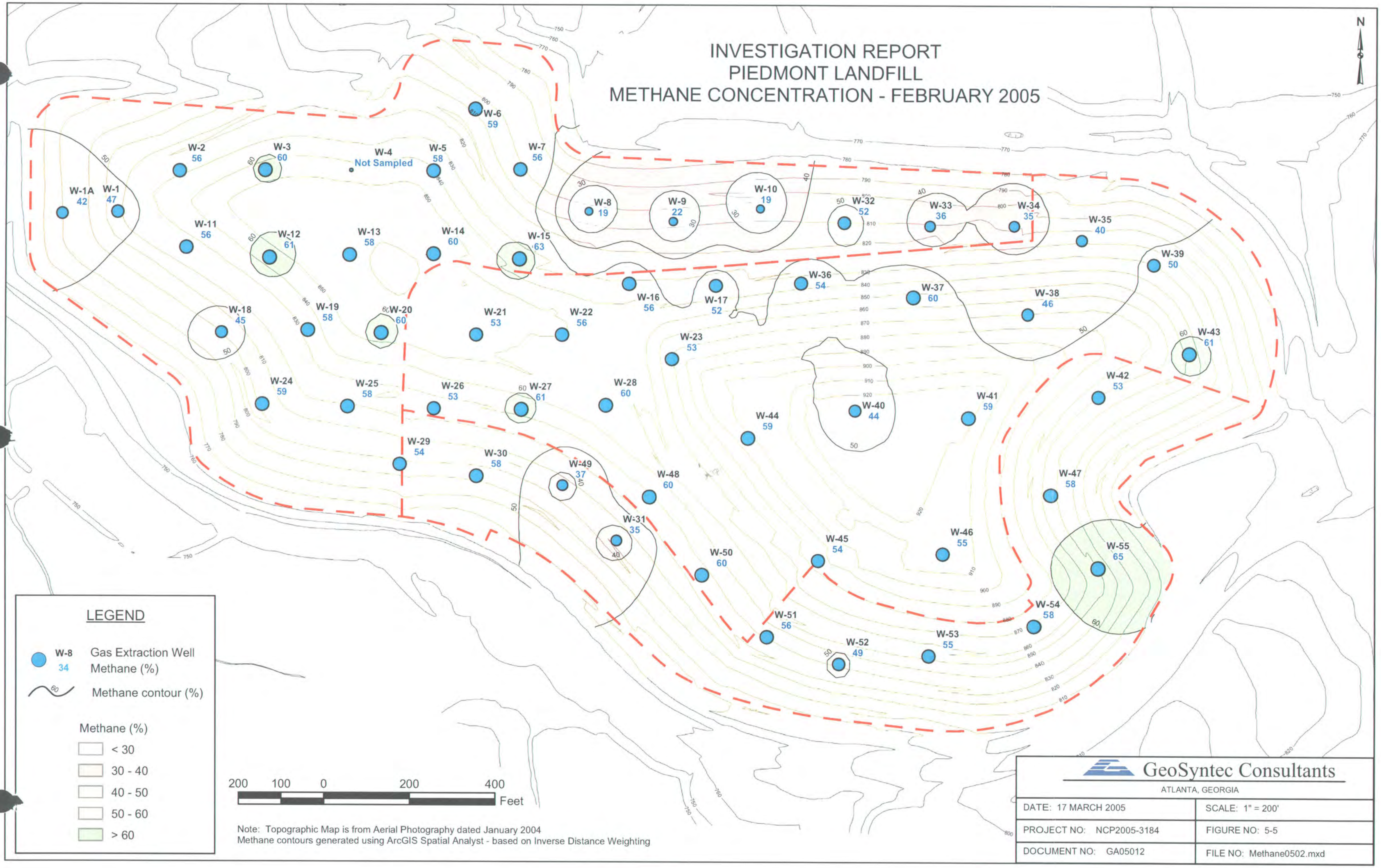
INVESTIGATION REPORT
PIEDMONT LANDFILL
METHANE CONCENTRATION - JANUARY 2005

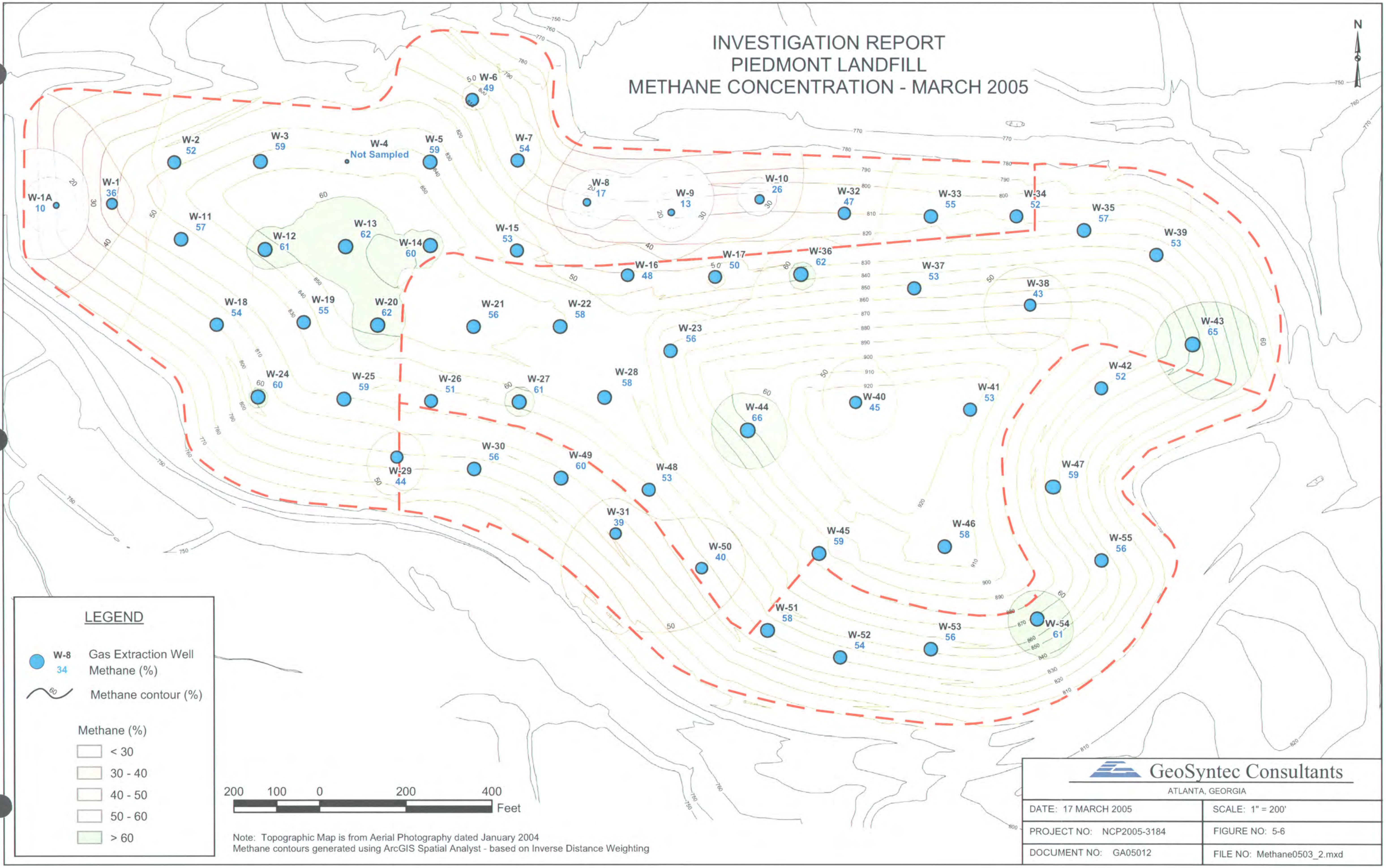


Note: Topographic Map is from Aerial Photography dated January 2004
Methane contours generated using ArcGIS Spatial Analyst - based on Inverse Distance Weighting

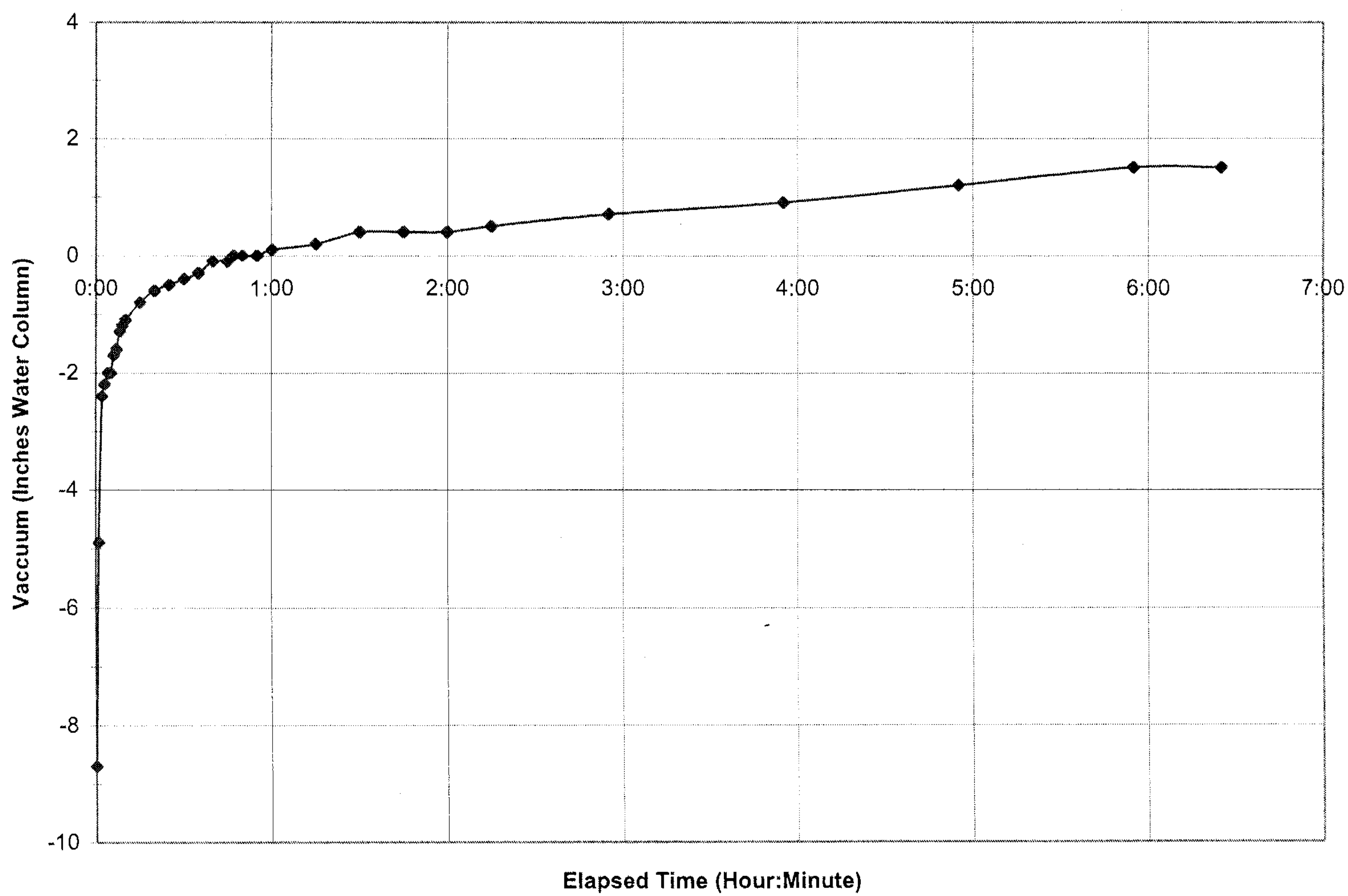
GeoSyntec Consultants
ATLANTA, GEORGIA

DATE: 17 MARCH 2005	SCALE: 1" = 200'
PROJECT NO: NCP2005-3184	FIGURE NO: 5-4
DOCUMENT NO: GA05012	FILE NO: Methane0501.mxd


INVESTIGATION REPORT
PIEDMONT LANDFILL
METHANE CONCENTRATION - FEBRUARY 2005
N
W-1A 42
W-1 47
W-2 56
W-3 60
W-4 Not Sampled
W-5 58
W-6 59
W-7 56
W-8 19
W-9 22
W-10 19
W-11 56
W-12 61
W-13 58
W-14 60
W-15 63
W-16 56
W-17 52
W-18 45
W-19 58
W-20 60
W-21 53
W-22 56
W-23 53
W-24 59
W-25 58
W-26 53
W-27 61
W-28 60
W-29 54
W-30 58
W-31 35
W-32 52
W-33 36
W-34 35
W-35 40
W-36 54
W-37 60
W-38 46
W-39 50
W-40 44
W-41 59
W-42 53
W-43 61
W-44 59
W-45 54
W-46 55
W-47 58
W-48 60
W-49 37
W-50 60
W-51 56
W-52 49
W-53 55
W-54 58
W-55 65
LEGEND
W-8 34 Gas Extraction Well Methane (%)
Methane contour (%)
Methane (%)
< 30
30 - 40
40 - 50
50 - 60
> 60
200 100 0 200 400 Feet
Note: Topographic Map is from Aerial Photography dated January 2004
Methane contours generated using ArcGIS Spatial Analyst - based on Inverse Distance Weighting
GeoSyntec Consultants
ATLANTA, GEORGIA
DATE: 17 MARCH 2005
SCALE: 1" = 200'
PROJECT NO: NCP2005-3184
FIGURE NO: 5-5
DOCUMENT NO: GA05012
FILE NO: Methane0502.mxd

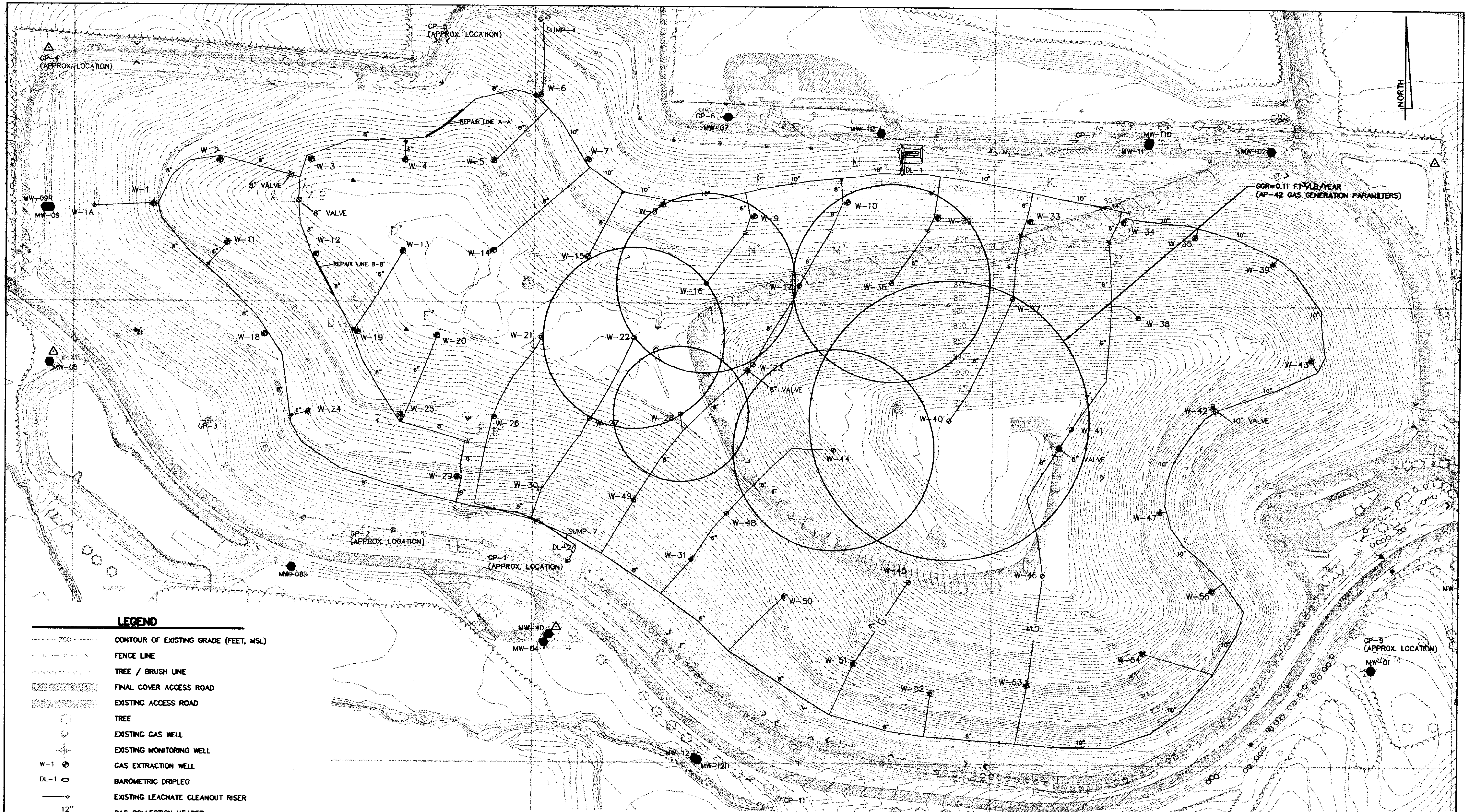

INVESTIGATION REPORT
PIEDMONT LANDFILL
METHANE CONCENTRATION - MARCH 2005
N
W-1A 10
W-1 36
W-2 52
W-3 59
W-4 Not Sampled
W-5 59
W-6 49
W-7 54
W-8 17
W-9 13
W-10 26
W-11 57
W-12 61
W-13 62
W-14 60
W-15 53
W-16 48
W-17 50
W-18 54
W-19 55
W-20 62
W-21 56
W-22 58
W-23 56
W-24 60
W-25 59
W-26 51
W-27 61
W-28 58
W-29 44
W-30 56
W-31 39
W-32 47
W-33 55
W-34 52
W-35 57
W-36 62
W-37 53
W-38 43
W-39 53
W-40 45
W-41 53
W-42 52
W-43 65
W-44 66
W-45 59
W-46 58
W-47 59
W-48 53
W-49 60
W-50 40
W-51 58
W-52 54
W-53 56
W-54 61
W-55 56
LEGEND
W-8 34 Gas Extraction Well Methane (%)
Methane contour (%)
Methane (%)
< 30
30 - 40
40 - 50
50 - 60
> 60
200 100 0 200 400 Feet
Note: Topographic Map is from Aerial Photography dated January 2004
Methane contours generated using ArcGIS Spatial Analyst - based on Inverse Distance Weighting
GeoSyntec Consultants
ATLANTA, GEORGIA
DATE: 17 MARCH 2005
SCALE: 1" = 200'
PROJECT NO: NCP2005-3184
FIGURE NO: 5-6
DOCUMENT NO: GA05012
FILE NO: Methane0503_2.mxd

Pressure Change with Time, Well 23



GEOSYNTEC CONSULTANTS
ATLANTA, GEORGIA

DATE:	17 MARCH 2005	SCALE:	
PROJECT NO.	NCP2005-3184	FILE NO.	GA050128.CDR
DOCUMENT NO.	GA050128	FIGURE NO.	5-7



INVESTIGATION REPORT
PIEDMONT LANDFILL
CALCULATED RADII OF INFLUENCE OF WELLS SURROUNDING GAS EXTRACTION WELL 23

0 200
HORIZONTAL SCALE IN FEET

1. BASE TOPOGRAPHIC MAP IS FROM AERIAL PHOTOGRAPHY DATED JANUARY 2004
2. LANDFILL GAS SYSTEM AS-BUILT PROVIDED BY JOYCE ENGINEERING OF GREENSBORO, NC.

GEOSYNTEC CONSULTANTS
ATLANTA, GA

DATE: 17 MARCH 2005	SCALE: 1" = 200'
PROJECT NO. NCP2005-3184	FILE NO. 3184-F008.DWG
DOCUMENT NO. GA050128	FIGURE NO. 5-8

APPENDIX A

RESPONSE TO NCDENR COMMENTS

RESPONSE TO NCDENR COMMENTS

21 February 2005 comments from Mr. Geof Little of NCDENR to Mr. Mark Snyder of WM.

Comment 1. *Address the integrity of the exposed GCL with respect to weathering and UV exposure. This issue is moot if the proposal is to replace the exposed GCL.*

Response: The exposed GCL will need to be removed prior to repair of the cover system. Therefore, weathering of the GCL does not need to be evaluated.

Comment 2. *The interface direct shear strength between the (i) 50-mil LLDPE Agru Micro-Drain geomembrane and the Bentomat ST GCL, which was used for the most recent closure construction, and (ii) 40-mil LLDPE Agru MicroSpike geomembrane and Bentomat SDN GCL, which was used in the first phase of closure. The shear strength testing should mimic the exact same interfaces as determined in the field (such as the woven or nonwoven upper face of the GCL in contact with the same lower face of the FML). Please note that the March 8, 2004, request for approval of alternate geosynthetic materials in the final cover system from Waste Management Attachment 4 shows interface direct shear testing for 60-mil Agru Super GripNet HDPE geomembrane against TNS E080 geotextile and against clay liner (CL-1).*

Response: A subsequent submittal was made on 24 May 2004 which requested NCDENR approval of Agru LLDPE Structured Geomembranes for use in the Piedmont Landfill final cover system. This request was approved by NCDENR on 26 May 2004. Copies of both documents were provided to NCDENR via e-mail on 22 February 2005 and are included in Appendix B to this report.

Comment 3: *The Slope Stability Calculations contained in Attachment 5 of the request referenced in Item No. 2 above should be reviewed for accuracy with respect to the materials used to construct the final cover system.*

Response: The slope stability calculations in Attachment 5 establish an envelope of strength parameters that need to be met to achieve the target factor of safety. As indicated in our 24 May 2004 request for approval, these calculations are applicable to both the February and May 2004 technical demonstrations.

Comment 4: *The presence of the geotextile drainage layer should be confirmed in all locations of the final cover system.*

Response: A re ... nstruction of the most recei ... ile was installed in all locat

Feb 2004 Tech Demo?
May 2004 Tech Demo?
Calculations??

23 February 20(... DENR to Mr. Mark Snyder of WM.

Comment 1. *The ... es not contain slope stabi ... LLDPE MicroDrain geom*

Response: As nc ... DENR's 21 February 2005 ... ined in Attachment 5 of the

Comment 2. *Also, previous slope stability calculations contained in the February 18, 2004, Request for Alternate Geosynthetic Materials do not account for the weight of the beams, which are placed on top of the liner system and are not benched as an integral part of the liner system.*

Response: The calculated factor of safety for the berms is approximately the same as for the overall cover system stability. This occurs because, even though the soil cover thickness in the vicinity of the berms is thicker than the soil between the berms, both the driving and resting forces affecting stability increase proportionally along a potential slip surface as the soil thickness increases. Proportionally increasing the driving and resisting forces typically does not affect the calculated factor of safety. Also as noted in Section 3.3, movement of the berms relative to the adjacent cover soil was not observed.

Comment 3. *Please review your files for the alternative liner request and approval for the 14.7 acre closure certified in the November 2003 CQA Final Report.*

Response: The 14.7 acre area closed in 2003 was constructed in accordance with the design shown in the June 1997 report by EcoLogical Associates, P.C. entitled "petition for Alternate Cover".

Comment 4. *Please provide final As-Constructed drawings for the final cover system constructed in the past 2 phases including representative cross-sections.*

Response: Limits of the various phases of closure are shown on Figure 2-1 of this report. Representative cross-sections for each phase are shown on Figure 2-2.

APPENDIX B

18 FEBRUARY 2004 AND 24 MAY 2004
TECHNICAL DEMONSTRATIONS

GEOSYNTEC CONSULTANTS

1100 Lake Hearn Drive, Suite 200
Atlanta, Georgia 30342-1523 • USA
Tel. (404) 705-9500 • Fax (404) 705-9400

18 February 2004

Ms. Sherri Coghill
Environmental Engineer
Division of Waste Management
North Carolina Department of Environment and Natural Resources
1646 Mail Service Center
Raleigh, NC 27699

Subject: Request for Approval of Alternate Geosynthetic Materials for the Piedmont Landfill Final Cover System Forsyth County, North Carolina

Dear Ms. Coghill:

On behalf of Waste Management, Inc. (WM), GeoSyntec Consultants (GeoSyntec) is requesting the North Carolina Department of Environment and Natural Resources (NCDENR) approve alternate final cover system geosynthetic materials for the Piedmont Landfill located in Forsyth County, North Carolina. The geosynthetic components of the currently permitted final cover systems consist of a 40-mil thick geomembrane and a geocomposite drainage layer. The configuration of the final cover systems will not be altered, and the functional requirements of the geosynthetic layers will not be changed. It is only requested that an Agru/America, Inc. (A/A) Structured Geomembrane product (a geomembrane and drainage net in one) be used in the final cover system in lieu of using individual geomembrane and geocomposite drainage layer components.

The remainder of this letter is organized to present: (i) technical information about the A/A Structured Geomembrane, including results of laboratory shear strength and hydraulic transmissivity testing; and (ii) results of hydraulic performance and slope stability analyses performed by GeoSyntec to demonstrate suitability of the proposed alternate geosynthetic materials for use in the final cover system of the Piedmont Landfill.

GD3004/GA040081.doc

Ms. Sherri Coghill
18 February 2004
Page 2

DESCRIPTION OF THE PIEDMONT LANDFILL FINAL COVER SYSTEMS

The currently permitted final cover systems for the Piedmont Landfill are described in a July 1996 report titled "Petition for Alternative Final Cover, Piedmont Landfill and Recycling Center", prepared by Rust Environment and Infrastructure (Rust). Figure 1 of this report shows schematics of the permitted final cover systems, and is attached to this letter. As shown in Figure 1, all of the permitted final cover systems include a geomembrane and a geocomposite drainage layer.

NCDENR is requested to approve the use of an A/A Structured Geomembrane and an 8 oz/yd2 geotextile filter as an alternate to the geomembrane and geocomposite drainage layer components of the final cover systems. The A/A Structured Geomembrane is a geomembrane liner and a drainage structure in one product. The resin is extruded homogeneously to provide a 60-mil thick geomembrane with solid stud drainage structures on top and texturing on the bottom. The texturing can be in the form of spikes (as for the Super Gripnet® Liner) or can be typical geomembrane texturing (as for the A/A Drain Liner). The texturing is intended to improve shear strength of the interface between the liner and the underlying barrier layer.

A/A STRUCTURED GEOMEMBRANE TECHNICAL INFORMATION

General Physical Characteristics

Technical information about the A/A Structured Geomembranes (i.e., Super Gripnet® Liner and A/A Drain Liner) is provided in Attachment 1. As can be seen from the product data sheets included in Attachment 1, the liner physical properties are equivalent to other commercially available HDPE geomembrane products. Using a 60-mil thick HDPE A/A Structured Geomembrane is an improvement over the currently permitted 40-mil thick polyethylene geomembrane.

Similar to other commercially available geomembranes, the A/A Structured Geomembrane is delivered to the site in rolls 22.5 ft wide and 197 ft long. Panels are welded together using the same equipment as other HDPE geomembranes, and construction quality assurance (CQA) testing of the seams is performed using the typical procedures and equipment in accordance with the site approved CQA Plan.

Hydraulic Transmissivity Characteristics

GeoSyntec performed hydraulic transmissivity tests on the A/A Structured Geomembrane (i.e., Super Gripnet® Liner) under a normal stress and range of hydraulic gradients and seating times that are applicable to the Piedmont Landfill final cover system. The results of the tests are included as Attachment 2. As shown in Attachment 2, the hydraulic transmissivity of the Super Gripnet® Liner ranged from 6.48×10^{-4} to 1.04×10^{-3} m^2/sec. The A/A Drain Liner should exhibit similar hydraulic transmissivity values as the Super Gripnet® Liner. These transmissivity values are within the range of transmissivities typical of most commercially available geosynthetic drainage layers.

GeoSyntec performed hydraulic analysis for a Piedmont Landfill final cover system that would contain an A/A Structured Geomembrane using the United States Environmental Protection Agency (USEPA) Hydrologic Evaluation of Landfill Performance (HELP) computer model. The hydraulic analysis assumptions and results are presented in Attachment 3. These results show that a final cover system which incorporates an A/A Structured Geomembrane has adequate hydraulic transmissivity to ensure proper drainage of water infiltrating through the overlying cover soil layer, and to limit hydraulic head buildup on the underlying barrier layer.

Shear Strength Characteristics

GeoSyntec performed laboratory shear strength testing to evaluate interfacial shear strength characteristics of a final cover system encompassing an A/A Structured Geomembrane. The test configuration consisted of a Super Gripnet® Liner sandwiched between materials similar to the currently permitted final cover system materials. The failure surface was allowed to occur at the weakest interface between the final cover system materials. The results of the testing are included as Attachment 4 to this letter. Based on these results, the shear strength of the Super Gripnet® Liner is relatively high (more than that represented by a friction angle of 33 degrees and a cohesion of 55 psf).

Ms. Sherri Coghill
18 February 2004
Page 4

GeoSyntec performed static and seismic slope stability analyses to estimate minimum required shear strength properties for the Piedmont Landfill final cover systems. These analyses are included as Attachment 5 to this letter. The results of the analyses show that the minimum required peak and residual shear strengths for the final cover system materials and interfaces are equivalent to friction angles of approximately 26 and 21 degrees, respectively. Based on the above described testing results, the final cover system containing an A/A Structured Geomembrane should be able to meet these minimum shear strength requirements.

CLOSURE

NCDENR is requested to approve the proposed geosynthetic product, Agru/America, Inc. Structured Geomembrane, for use in the Piedmont Landfill final cover system, as it meets the project specifications and performance requirements. Please contact the undersigned if you have any questions or comments related to this letter.

Sincerely,

Majdi Ahmed Othman

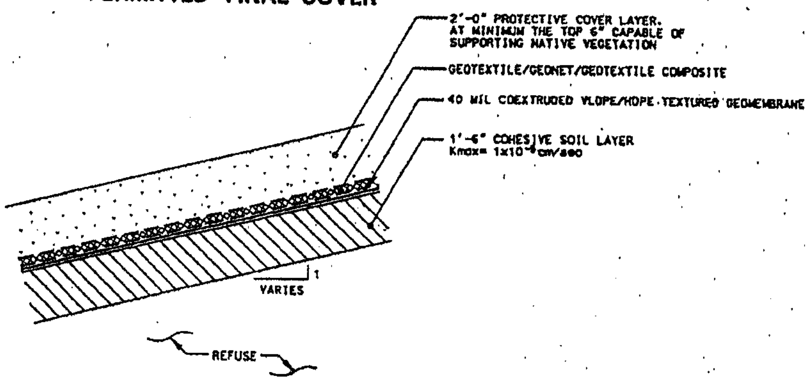
Majdi A. Othman, Ph.D., P.E.
Associate

Attachments

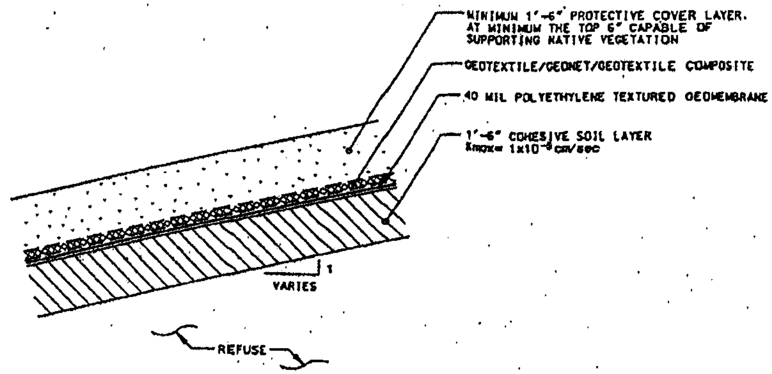
Copy to: Michael Loyd, Waste Management

GD3004/GA040081.doc

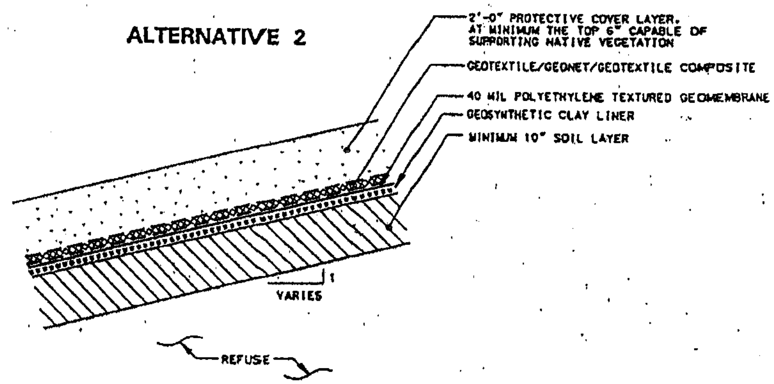
PERMITTED FINAL COVER



ALTERNATIVE 1



ALTERNATIVE 2



RUST ENVIRONMENT & INFRASTRUCTURE

FIGURE 1
FINAL COVER DETAILS

PIEDMONT LANDFILL AND RECYCLING CENTER
KERNERSVILLE, FORSYTH COUNTY, NORTH CAROLINA

7221FGR1.DG

ATTACHMENT 1

AGRU/AMERICA, INC.
STRUCTURED GEOMEMBRANE
TECHNICAL INFORMATION

AGRU/AMERICA, INC.

SUPER GRIPNET® LINER

Drainage + Anchor Structure Liner

Extruded homogeneously to provide a minimum 60 mil (1.5 mm) containment liner with drainage and anchor structure capability.
These multi-structural products enhance steep composite slope designs with maximum interfacial shear resistance and superior drainage capacity.

Features/Benefits

- Superior interfacial friction with all soil types and conditions
- Allows steepest possible slopes

Product Data (Minimum Average Roll Values)

US. Patent - No. 5.258.217

Property	Test Method	Values		
Thickness (mils nominal)	ASTM D751	60	80	100
Melt Flow Index (g/10 minutes)	ASTM D1238 - E	.28	.28	.28
Density (g/cm³ min)	ASTM D792 or D1505	.948	.948	.948
Tensile Strength at Yield (lbs/in. width)	ASTM D638 (Modified)	114	152	190
Tensile Strength at Break (lbs/in. width)	Type I Specimen	126	168	210
Elongation at Yield (%)	Gauge length 2 in. break,	13	13	13
Elongation at Break (%)	1.3 in. yield, 2 ipm	200	200	200
Tear Resistance (lbs)	ASTM D1004 - Die C	40	53	67
Low Temperature Impact (°F max)	ASTM D746	-103	-103	-103
Dimensional Stability (% change max)	ASTM D1204, 1 hr @ 212°F	±2	±2	±2
Environmental Stress Crack (hrs)	ASTM D1693	1500	1500	1500
Puncture Resistance (lbs)	FTMS 101 - C, Method 2065	75	100	125
Carbon Black Content (%)	ASTM D1603	2-3	2-3	2-3
Carbon Black Dispersion	ASTM D3015	A2	A2	A2

Supply Information (Standard Roll Dimensions)

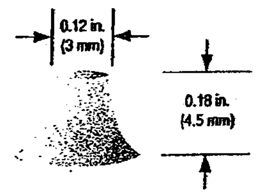
Thickness		Width		Length		Area		Weight	
mil	mm	ft	m	ft	m	ft²	m²	lbs	kg
60	1.5	22.5	6.86	197	60	4,433	412	2,060	934
80	2.0	22.5	6.86	165	50	3,713	343	1,973	895
100	2.5	22.5	6.86	165	50	3,713	343	2,418	1,096

NOTES: *1.) All rolls are supplied with two slings. 2.) All rolls are fitted with a 6 inch ID HDPE core. 3.) Special roll lengths are available on request. 4.) Standard rolls have a diameter of 29 inches (750 mm). 5.) A 40 foot standard container will hold 9 rolls. 6.) A 48 foot flatbed will hold 14 rolls.*

AGRU/AMERICA, INC.

A/A DRAIN LINER®

Designed for Drainage and Anchor Performance

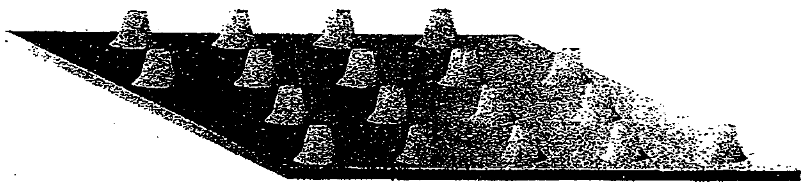


Extruded homogeneously to provide a minimum 60 mil (1.5 mm) containment liner with solid stud drainage structures at 0.8 inch (20 mm) centers.

When used in conjunction with a smooth or textured upper sheet, the composite provides an effective double liner system without the necessity of a geonet.

Features/Benefits

- Excellent interfacial friction with soils
- Excellent hydraulic transmissivity



Product Data (Minimum Average Roll Values)

Property	Test Method	Values		
Thickness (mils nominal)	ASTM D751	60	80	100
Melt Flow Index (g/10 minutes)	ASTM D1238 - E	.28	.28	.28
Density (g/cm^3 min)	ASTM D792 or D1505	.948	.948	.948
Tensile Strength at Yield (lbs/in. width)	ASTM D638 (Modified)	120	160	200
Tensile Strength at Break (lbs/in. width)	Type I Specimen	132	176	220
Elongation at Yield (%)	Gauge length 2 in. break,	13	13	13
Elongation at Break (%)	1.3 in. yield, 2 ipm	200	200	200
Tear Resistance (lbs)	ASTM D1004 - Die C	50	67	83
Low Temperature Impact (°F max)	ASTM D746	-103	-103	-103
Dimensional Stability (% change max)	ASTM D1204, 1 hr @ 212°F	±2	±2	±2
Puncture Resistance (lbs)	FTMS 101 - C, Method 2065	95	126	158
Carbon Black Content (%)	ASTM D1603	2 - 3	2 - 3	2 - 3
Carbon Black Dispersion	ASTM D3015	A2	A2	A2

Supply Information (Standard Roll Dimensions) 465 studs/ft^2 = 5,000 studs/m^2

Thickness		Width		Length		Area		Weight	
mil	mm	ft	m	ft	m	ft^2	m^2	lbs	kg
60	1.5	22.5	6.86	263	80	5,917	550	2,295	1,041
80	2.0	22.5	6.86	230	70	5,175	480	2,489	1,129
100	2.5	22.5	6.86	197	60	4,433	412	2,637	1,196

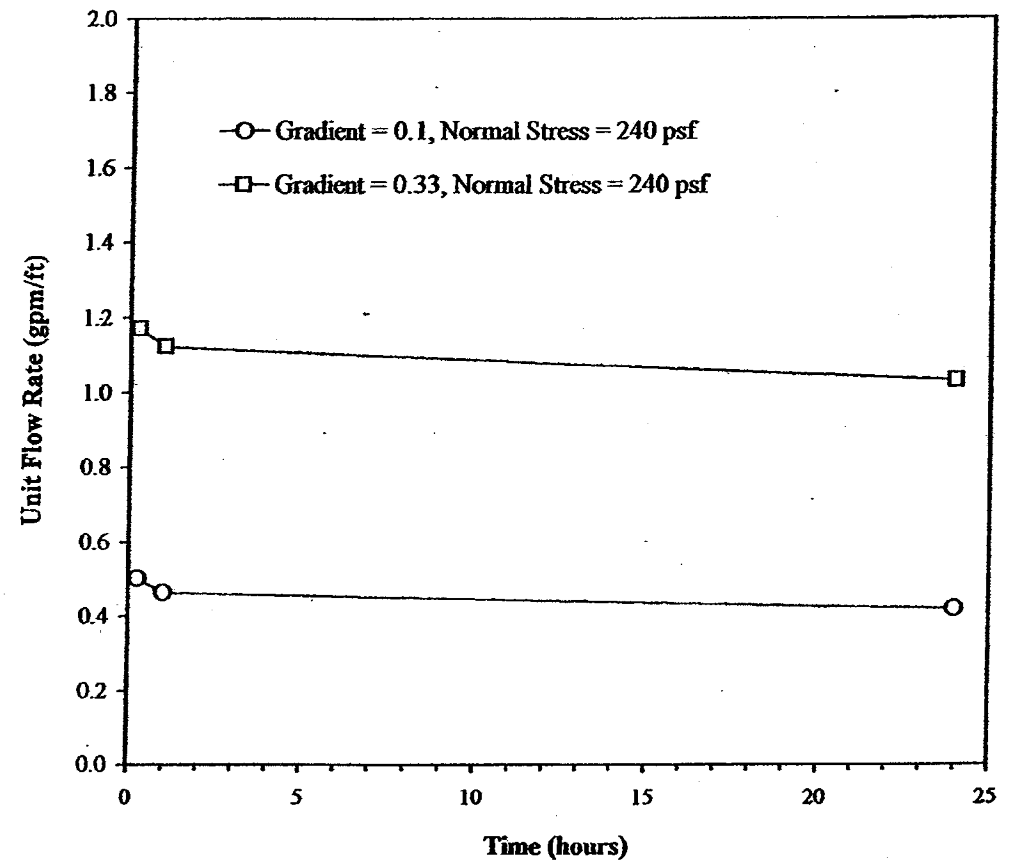
NOTES: 1.) All rolls are supplied with two slings. 2.) All rolls are fitted with a 6 inch ID HDPE core. 3.) Special roll lengths are available on request.) Standard rolls have a diameter of 29 inches (750 mm). 5.) A 40 foot standard container will hold 9 rolls. 6.) A 48 foot flatbed will hold 14 rolls.

ATTACHMENT 2

AGRU/AMERICA, INC.
STRUCTURED GEOMEMBRANE
HYDRAULIC TRANSMISSIVITY LABORATORY
TESTING RESULTS

HYDRAULIC TRANSMISSIVITY TESTING (ASTM D 4716)

Protective Cover Soil/ TNS E080 Geotextile /AGRU SuperGripNet Geomembrane
SGI Lab Sample ID: AL9365,9357,9358



Test No.	Flow Direction	Normal Stress (psf)	Seating Time (hour)	Hydraulic Gradient (-)	Transmissivity (m^2/sec)	Unit Flow Rate (gpm/ft)
1	MD	240	0.25	0.10	1.04E-03	0.50
2	MD	240	1.00	0.10	9.60E-04	0.46
3	MD	240	24.00	0.10	8.71E-04	0.42
4	MD	240	0.25	0.33	7.35E-04	1.17
5	MD	240	1.00	0.33	7.04E-04	1.12
6	MD	240	24.00	0.33	6.48E-04	1.03

Notes:

(1) Test configuration from top to bottom: Protective Cover soil / TNS E080 geotextile / Agru SuperGripNet geomembrane (cylinders up and spikes down) against Neoprene (replacement for Clay liner).

(2) Test Specimen Dimensions: length: 12 in., width = 12.0 in.

DATE REPORTED: 8/23/2002

SGI TESTING SERVICES, LLC

FIGURE NO.	A-2
PROJECT NO.	SGI2064
DOCUMENT NO.	SGI02106
FILE NO.	

ATTACHMENT 3

HYDRAULIC PERFORMANCE ANALYSIS OF FINAL COVER SYSTEM CONTAINING AGRU/AMERICA, INC. STRUCTURED GEOMEMBRANE

Written by:	Tamer Y. Elkady	Date:	04 /2 /18 (YY MM DD)	Reviewed by:	Majdi Othman	Date:	04/ 2 /18 (YY MM DD)
Client:	WMI	Project:	Piedmont Landfill	Project/Proposal No.:	GD3294	Task No:	01

HYDRAULIC PERFORMANCE OF FINAL COVER SYSTEM FOR THE PIEDMONT LANDFILL

PURPOSE

The proposed modification to the Piedmont Landfill final cover system involves replacing the geosynthetics of the final cover system (i.e., geomembrane and geocomposite drainage layer) with an Agru America, Inc. (A/A) Structured Geomembrane (Super Gripnet® Liner or A/A Drain Liner) that acts as a geomembrane and a drainage structure at the same time. The analysis described herein was performed to evaluate the hydraulic performance of the final cover system consisting of an A/A Structured Geomembrane (referenced hereafter as the A/A final cover system).

METHOD OF ANALYSIS

Evaluation of the hydraulic performance of the A/A final cover system was performed using the Hydrologic Evaluation of Landfill Performance (HELP) model, version 3.07, developed for USEPA (Schroeder et al., 1994 a,b). The HELP program is a quasi two dimensional hydrologic model of water movement across, into, through, and out of landfills.

PARAMETERS USED IN THE ANALYSIS

The following input information was used with the HELP program:

Climatic data

The climatic data required by HELP were assumed to be similar to that used in the evaluation of alternative final cover systems prepared by Rust Environment and Infrastructure (Rust, 1996) and approved by the North Carolina Department of Environment and Natural Resources (NCDENR). A brief description of this climatic data is presented below.

- The climate was modeled for Greensboro, North Carolina using a synthetic daily weather generator computer option over a 20-year period.
- The evaporative zone depth was input as the thickness of the final cover system layers that overlay the barrier layer (i.e., 24 inches).

Written by:	Tamer Y. Elkady	Date:	04 /2 /18 (YY MM DD)	Reviewed by:	Majdi Othman	Date:	04/ 2 /18 (YY MM DD)
Client:	WMI	Project:	Piedmont Landfill	Project/Proposal No.:	GD3294	Task No:	01

Final cover data

As indicated earlier, the configuration of the permitted final cover will not be changed except that the geosynthetics of the final cover system (i.e., geomemebrane and geocomposite drainage layer) will be replaced by an A/A Structured Geomembrane. Therefore, components of the A/A final cover system consist, from top to bottom, of:

- a 24-inch thick protective soil layer;
- an 8 oz/yd^2 geotextile filter;
- an A/A Structured Geomembrane consisting of a 60-mil thick high density polyethylene (HDPE) geomembrane with 0.18-inch thick drainage studs on the top; and
- an 18–inch compacted clay barrier with a hydraulic conductivity no greater than 1 x 10^{-5} cm/sec.

Therefore, the following data was used to represent the properties of the different layers for the A/A final cover system:

- The protective soil layer was modeled as soil with the following properties (default material texture # 8):

Thickness:	24 inches
Porosity:	0.463
Field capacity:	0.232
Wilting point:	0.116
Saturated hydraulic conductivity:	4x10^{-4} cm/sec

- The drainage layer of the final cover system was modeled as a material with the following properties:

Thickness:	0.18 inches
Porosity:	0.85
Field capacity:	0.01
Wilting point:	0.005

GeoSyntec Consultants have previously conducted tests on the proposed A/A Structured Geomembrane to evaluate its hydraulic transmissivity under different hydraulic gradients and seating times. Based on these test results, the hydraulic transmissivity for the drainage layer was found to range from approximately 6.5 x 10^{-4} m^2/sec to 1.0 x 10^{-3} m^2/sec. For the purpose of this analysis, data from the hydraulic transmissivity testing was extrapolated to estimate the hydraulic transmissivity of the drainage layer at the end of the post closure period (i.e., after 30 years) as shown in Figure 1. Based on Figure 1, the long-term hydraulic transmissivity of the drainage layer was conservatively assumed to be 4.8 x 10^{-4} m^2/sec. Therefore, the saturated hydraulic conductivity was calculated as follows:

$$\text{Saturated hydraulic conductivity} = \frac{4.8 \times 10^{-4}\ \text{m}^2/\text{sec} \times 10^{4}\ \text{cm}^2/\text{m}^2}{0.18\ \text{in} \times 2.54\ \text{cm/in}} = 10\ \text{cm/sec}$$

- The 60-mil thick high density polyethylene (HDPE) geomembrane was modeled as a geosynthetic material texture # 35, with a hydraulic conductivity of 2 x 10^{-13} cm/sec. The geomembrane was assumed to have one hole per acre with a hole size of 0.16 in^2.

- The compacted clay barrier was modeled as a barrier layer with the following properties:

Thickness:	18 inches
Porosity:	0.419
Field capacity:	0.307
Wilting point:	0.180
Saturated hydraulic conductivity:	1 x 10^{-5} cm/sec

The drainage layer conditions applicable to the final cover system considered a critical slope of 25 percent (i.e., 4H:1V) and a drainage length of 115 ft which is the distance between surface water diversion berms. The final cover system was assumed to be vegetated with good grass and to allow runoff.

RESULTS AND CONCLUSIONS

The HELP Model output file is provided at the end of this calculation package. Based on the analysis results, the maximum head on top of the liner was estimated to be 0.047 inches which is considered small with respect to the thickness of the drainage layer (i.e., 0.18 inch). Therefore, the hydraulic performance of the A/A final cover system is deemed adequate.

REFERENCES

Rust Environment and Infrastructure (1996) *Petition for Alternative Final Cover, Piedmont Landfill and Recycling Center, Kernersville, North Carolina*, prepared for the North Carolina Department of Environment, Health and Natural Recources (NCDEHNR).

Schroeder. P.R., Lloyd, C.M., and Zappi, P.A. (1994a) "The Hydrologic Evaluation of Landfill Performance (HELP) Model, User's Guide for Version 3." U.S. Environmental Protection Agency, Office of Research and Development Washington, D.C., Report No. EPA/600/R094/168a.

Schroeder. P.R., Dozier, T.S., and Zappi, P.A., McEnroe, B.M., Sjostrom, J.W., and Peyton, R.L. (1994b) "The Hydrologic Evaluation of Landfill Performance (HELP) Model, Engineering Documentation for Version 3." U.S. Environmental Protection Agency, Office of Research and Development Washington, D.C., Report No. EPA/600/R-94/168b.

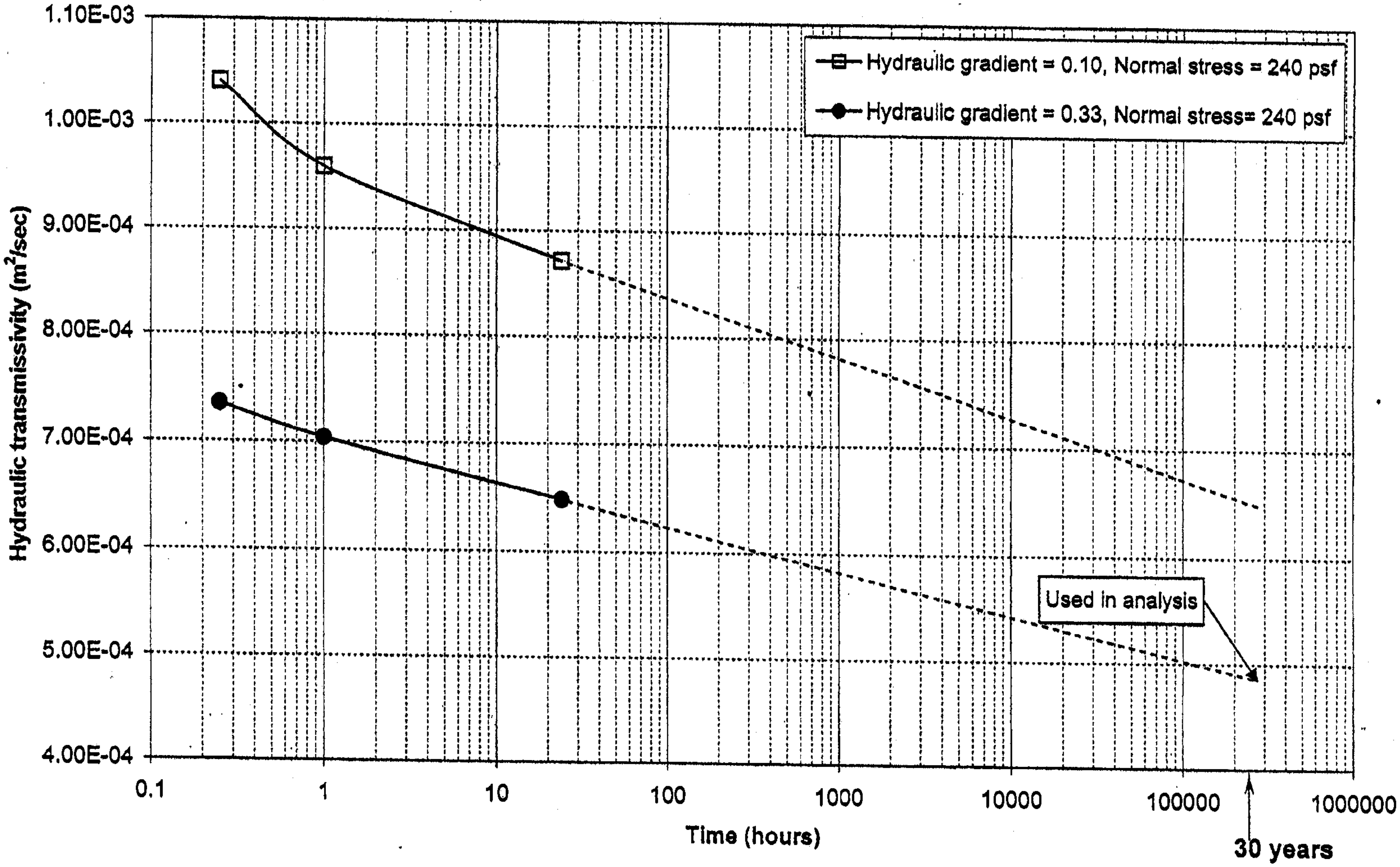


Figure 1: Estimation of the long-term hydraulic transmissivity for the drainage layer of a Super Gripnet® Liner

HELP Model Output File

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0
******************************************************************************
******************************************************************************
**                                                                          **
**                                                                          **
**              HYDROLOGIC EVALUATION OF LANDFILL PERFORMANCE               **
**              HELP MODEL VERSION 3.07  (1 NOVEMBER 1997)                  **
**              DEVELOPED BY ENVIRONMENTAL LABORATORY                       **
**              USAE WATERWAYS EXPERIMENT STATION                           **
**              FOR USEPA RISK REDUCTION ENGINEERING LABORATORY             **
**                                                                          **
**                                                                          **
******************************************************************************
******************************************************************************


PRECIPITATION DATA FILE:      C:\HELP3.07\PIED4.D4
TEMPERATURE DATA FILE:        C:\HELP3.07\PIED7.D7
SOLAR RADIATION DATA FILE:    C:\HELP3.07\PIED13.D13
EVAPOTRANSPIRATION DATA:      C:\HELP3.07\PIED11.D11
SOIL AND DESIGN DATA FILE:    C:\HELP3.07\PIEDLF.D10
OUTPUT DATA FILE:             C:\HELP3.07\PIEDLF.OUT


TIME:  15:42      DATE:  11/11/2003


******************************************************************************

     TITLE:  Piedmont Landfill Final Cover Equivalency Demo

******************************************************************************


     NOTE:  INITIAL MOISTURE CONTENT OF THE LAYERS AND SNOW WATER WERE
            COMPUTED AS NEARLY STEADY-STATE VALUES BY THE PROGRAM.


                              LAYER  1
                              --------

                  TYPE 1 - VERTICAL PERCOLATION LAYER
                      MATERIAL TEXTURE NUMBER  8
         THICKNESS                  =    24.00   INCHES
         POROSITY                   =     0.4630 VOL/VOL
         FIELD CAPACITY             =     0.2320 VOL/VOL
         WILTING POINT              =     0.1160 VOL/VOL
         INITIAL SOIL WATER CONTENT =     0.1962 VOL/VOL
         EFFECTIVE SAT. HYD. COND.  =  0.369999994000E-03 CM/SEC
      NOTE:  SATURATED HYDRAULIC CONDUCTIVITY IS MULTIPLIED BY  4.63
             FOR ROOT CHANNELS IN TOP HALF OF EVAPORATIVE ZONE.


                              LAYER  2
                              --------
```

TYPE 2 - LATERAL DRAINAGE LAYER
MATERIAL TEXTURE NUMBER 20

THICKNESS	=	0.18	INCHES
POROSITY	=	0.8500	VOL/VOL
FIELD CAPACITY	=	0.0100	VOL/VOL
WILTING POINT	=	0.0050	VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.0198	VOL/VOL
EFFECTIVE SAT. HYD. COND.	=	10.0000000000	CM/SEC
SLOPE	=	25.00	PERCENT
DRAINAGE LENGTH	=	115.0	FEET

LAYER 3

TYPE 4 - FLEXIBLE MEMBRANE LINER
MATERIAL TEXTURE NUMBER 35

THICKNESS	=	0.06	INCHES
POROSITY	=	0.0000	VOL/VOL
FIELD CAPACITY	=	0.0000	VOL/VOL
WILTING POINT	=	0.0000	VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.0000	VOL/VOL
EFFECTIVE SAT. HYD. COND.	=	0.199999996000E-12	CM/SEC
FML PINHOLE DENSITY	=	0.00	HOLES/ACRE
FML INSTALLATION DEFECTS	=	1.00	HOLES/ACRE
FML PLACEMENT QUALITY	=	3 - GOOD	

LAYER 4

TYPE 3 - BARRIER SOIL LINER
MATERIAL TEXTURE NUMBER 0

THICKNESS	=	18.00	INCHES
POROSITY	=	0.4190	VOL/VOL
FIELD CAPACITY	=	0.3070	VOL/VOL
WILTING POINT	=	0.1800	VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.4190	VOL/VOL
EFFECTIVE SAT. HYD. COND.	=	0.999999975000E-05	CM/SEC

GENERAL DESIGN AND EVAPORATIVE ZONE DATA

NOTE: SCS RUNOFF CURVE NUMBER WAS COMPUTED FROM DEFAULT SOIL DATA BASE USING SOIL TEXTURE # 8 WITH A GOOD STAND OF GRASS, A SURFACE SLOPE OF 25.% AND A SLOPE LENGTH OF 115. FEET.

SCS RUNOFF CURVE NUMBER	=	75.70	
FRACTION OF AREA ALLOWING RUNOFF	=	100.0	PERCENT
AREA PROJECTED ON HORIZONTAL PLANE	=	1.000	ACRES
EVAPORATIVE ZONE DEPTH	=	24.0	INCHES
INITIAL WATER IN EVAPORATIVE ZONE	=	4.710	INCHES
UPPER LIMIT OF EVAPORATIVE STORAGE	=	11.112	INCHES

PIEDLF

LOWER LIMIT OF EVAPORATIVE STORAGE	=	2.784	INCHES
INITIAL SNOW WATER	=	0.000	INCHES
INITIAL WATER IN LAYER MATERIALS	=	12.255	INCHES
TOTAL INITIAL WATER	=	12.255	INCHES
TOTAL SUBSURFACE INFLOW	=	0.00	INCHES/YEAR

EVAPOTRANSPIRATION AND WEATHER DATA

NOTE: EVAPOTRANSPIRATION DATA WAS OBTAINED FROM
GREENSBORO NORTH CAROLINA

STATION LATITUDE	=	35.13 DEGREES
MAXIMUM LEAF AREA INDEX	=	3.50
START OF GROWING SEASON (JULIAN DATE)	=	90
END OF GROWING SEASON (JULIAN DATE)	=	305
EVAPORATIVE ZONE DEPTH	=	24.0 INCHES
AVERAGE ANNUAL WIND SPEED	=	7.60 MPH
AVERAGE 1ST QUARTER RELATIVE HUMIDITY	=	66.00 %
AVERAGE 2ND QUARTER RELATIVE HUMIDITY	=	68.00 %
AVERAGE 3RD QUARTER RELATIVE HUMIDITY	=	74.00 %
AVERAGE 4TH QUARTER RELATIVE HUMIDITY	=	70.00 %

NOTE: PRECIPITATION DATA WAS SYNTHETICALLY GENERATED USING
COEFFICIENTS FOR GREENSBORO NORTH CAROLINA

NORMAL MEAN MONTHLY PRECIPITATION (INCHES)

JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
3.51	3.37	3.88	3.16	3.37	3.93
4.27	4.19	3.64	3.18	2.59	3.38

NOTE: TEMPERATURE DATA WAS SYNTHETICALLY GENERATED USING
COEFFICIENTS FOR GREENSBORO NORTH CAROLINA

NORMAL MEAN MONTHLY TEMPERATURE (DEGREES FAHRENHEIT)

JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
37.50	39.90	48.00	58.30	66.50	73.50
77.20	76.30	69.90	58.40	48.50	40.20

NOTE: SOLAR RADIATION DATA WAS SYNTHETICALLY GENERATED USING
COEFFICIENTS FOR GREENSBORO NORTH CAROLINA
AND STATION LATITUDE = 35.13 DEGREES

****

PIEDLF

AVERAGE MONTHLY VALUES IN INCHES FOR YEARS 1 THROUGH 20

	JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
PRECIPITATION						
TOTALS	2.70	3.27	4.10	2.81	3.05	3.96
	4.76	5.13	3.82	2.52	2.60	3.80
STD. DEVIATIONS	1.65	1.38	1.92	1.36	1.29	2.53
	1.98	2.38	2.39	1.71	1.91	1.80
RUNOFF						
TOTALS	0.020	0.025	0.087	0.008	0.001	0.036
	0.035	0.052	0.078	0.043	0.043	0.039
STD. DEVIATIONS	0.067	0.062	0.247	0.021	0.005	0.136
	0.102	0.122	0.129	0.090	0.184	0.078
EVAPOTRANSPIRATION						
TOTALS	0.955	1.264	2.299	2.872	3.972	3.347
	4.108	3.626	2.542	1.201	0.899	0.773
STD. DEVIATIONS	0.313	0.375	0.541	0.489	0.964	1.531
	1.469	1.211	1.187	0.394	0.228	0.219
LATERAL DRAINAGE COLLECTED FROM LAYER 2						
TOTALS	1.7864	1.6875	2.1593	0.8184	0.4657	0.6773
	0.6998	0.7899	0.9016	1.0004	0.9212	2.1860
STD. DEVIATIONS	1.6939	0.9179	1.5454	0.7042	0.6396	0.8718
	0.7325	0.9025	0.8272	0.8180	1.1286	1.2995
PERCOLATION/LEAKAGE THROUGH LAYER 4						
TOTALS	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
STD. DEVIATIONS	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000

AVERAGES OF MONTHLY AVERAGED DAILY HEADS (INCHES)

	JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
DAILY AVERAGE HEAD ON TOP OF LAYER 3						
AVERAGES	0.0005	0.0005	0.0006	0.0002	0.0001	0.0002
	0.0002	0.0002	0.0003	0.0003	0.0003	0.0006
STD. DEVIATIONS	0.0005	0.0003	0.0004	0.0002	0.0002	0.0003
	0.0002	0.0003	0.0002	0.0002	0.0003	0.0004

****

****

AVERAGE ANNUAL TOTALS & (STD. DEVIATIONS) FOR YEARS 1 THROUGH 20

	INCHES		CU. FEET	PERCENT
PRECIPITATION	42.52	(7.072)	154363.9	100.00
RUNOFF	0.466	(0.3584)	1691.78	1.096
EVAPOTRANSPIRATION	27.859	(3.6012)	101128.41	65.513
LATERAL DRAINAGE COLLECTED FROM LAYER 2	14.09365	(4.16200)	51159.961	33.14243
PERCOLATION/LEAKAGE THROUGH LAYER 4	0.00001	(0.00000)	0.021	0.00001
AVERAGE HEAD ON TOP OF LAYER 3	0.000	(0.000)		
CHANGE IN WATER STORAGE	0.106	(1.1018)	383.74	0.249

****

****

PEAK DAILY VALUES FOR YEARS 1 THROUGH 20

	(INCHES)	(CU. FT.)
PRECIPITATION	3.76	13648.800
RUNOFF	1.058	3839.1340
DRAINAGE COLLECTED FROM LAYER 2	1.52679	5542.25293
PERCOLATION/LEAKAGE THROUGH LAYER 4	0.000000	0.00128
AVERAGE HEAD ON TOP OF LAYER 3	0.013	
MAXIMUM HEAD ON TOP OF LAYER 3	0.047	
LOCATION OF MAXIMUM HEAD IN LAYER 2 (DISTANCE FROM DRAIN)	0.0 FEET	
SNOW WATER	2.87	10419.2432
MAXIMUM VEG. SOIL WATER (VOL/VOL)	0.3791	
MINIMUM VEG. SOIL WATER (VOL/VOL)	0.1160	

*** Maximum heads are computed using McEnroe's equations. ***

Reference: Maximum Saturated Depth over Landfill Liner
by Bruce M. McEnroe, University of Kansas
ASCE Journal of Environmental Engineering
Vol. 119, No. 2, March 1993, pp. 262-270.

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0
****

FINAL WATER STORAGE AT END OF YEAR 20

LAYER	(INCHES)	(VOL/VOL)
1	6.8257	0.2844
2	0.0018	0.0100
3	0.0000	0.0000
4	7.5420	0.4190
SNOW WATER	0.000	

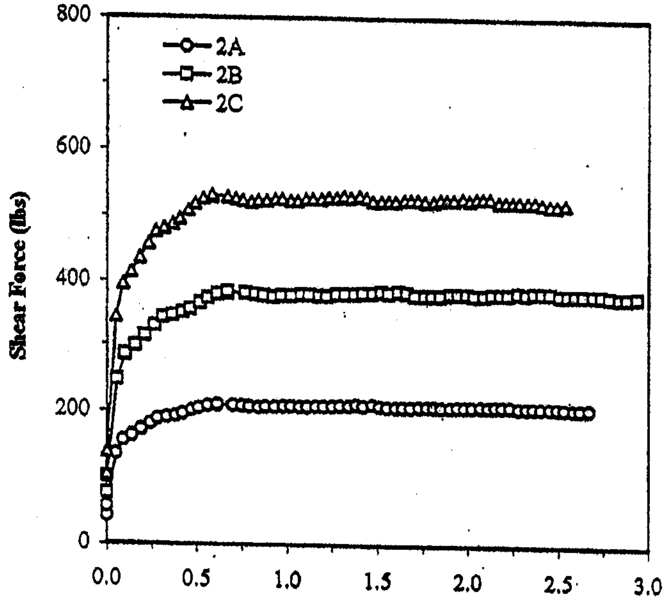
****
****

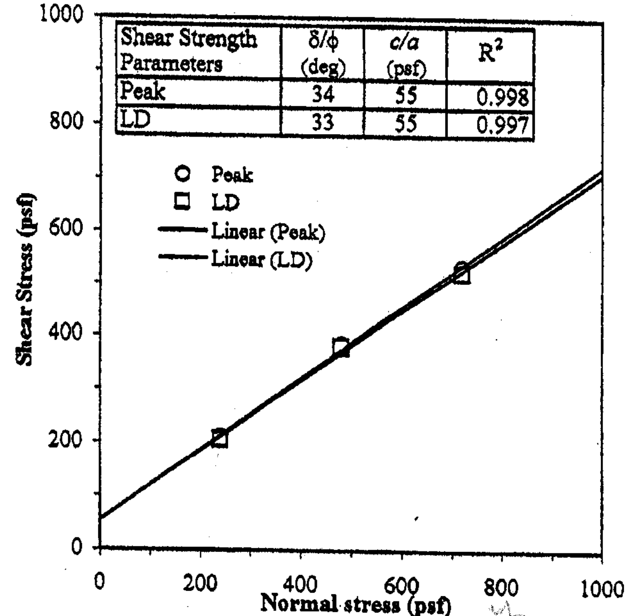
ATTACHMENT 4

RESULTS OF LABORATORY SHEAR STRENGTH TESTING ON FINAL COVER SYSTEM CONTAINING AGRU/AMERICA, INC. STRUCTURED GEOMEMBRANE

INTERFACE DIRECT SHEAR TESTING (ASTM D 5321)

Test Series 2: Protective Soil (PS-1) against TNS E080 geotextile against 60-mil Agru SuperGripNet liner HDPE geomembrane (cylinders up and spikes down) against Clay Liner (CL-1) under wetted and consolidated conditions





Test No.	Shear Box Size (in. x in.)	Normal Stress (psf)	Shear Rate (in./min)	Soaking Stress (psf)	Soaking Time (hour)	Consolidation Stress (psf)	Consolidation Time (hour)	Lower Soil γ_{di} (pcf)	Lower Soil ω_i (%)	Lower Soil ω_f (%)	Upper Soil γ_{di} (pcf)	Upper Soil ω_i (%)	Upper Soil ω_f (%)	GCL ω_i (%)	GCL ω_f (%)	Shear Stress τ_p (psf)	Shear Stress τ_{LD} (psf)	Failure Mode
2A	12 x 12	240	0.040	-	-	240	0.5	105.4	18.0	19.6	TIP	4.4	15.6	-	-	211	207	(1)
2B	12 x 12	480	0.040	-	-	480	0.5	104.7	18.8	19.3	TIP	4.1	11.9	-	-	384	380	(1)
2C	12 x 12	720	0.040	-	-	720	0.5	104.9	18.6	19.0	TIP	4.9	9.3	-	-	533	522	(1)

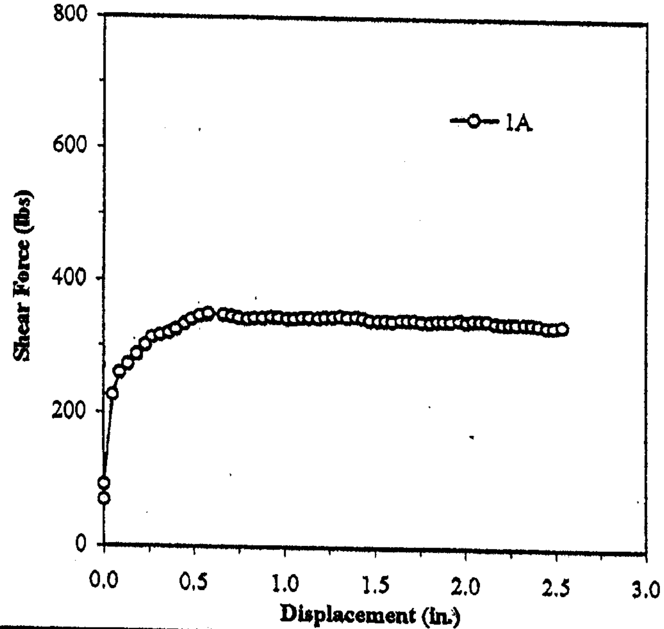
Notes: (1) Sliding (i.e., shear failure) occurred between the protective soil and geotextile during each test.
(2) The reported total-stress parameters of friction angle and adhesion were determined from a best-fit line drawn through the test data. Caution should be exercised in using these strength parameters for applications involving normal stresses outside the range of the stresses covered by the test series. The large-displacement (LD) shear strength was calculated using the shear force measured at the end of the test. TIP refers to the fact that the material was tamped in-place and its dry unit weight was not recorded.

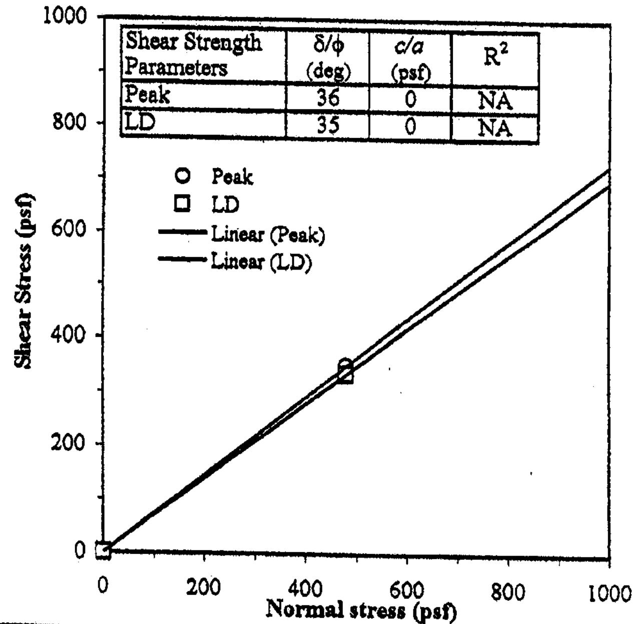
SGI TESTING SERVICES, LLC

DATE OF TEST:	17 July 2002
FIGURE NO.	A-4
PROJECT NO.	SGI2064
DOCUMENT NO.	SGI02106
FILE NO.	

INTERFACE DIRECT SHEAR TESTING (ASTM D 5321)

Test Series 1: Protective Soil (PS-1) against TNS E080 geotextile against 60-mil Agru SuperGripNet liner HDPE geomembrane (cylinders up and spikes down) against Clay Liner (CL-1) under wetted, consolidated, and slow shear conditions





Test No.	Shear Box Size (in. x in.)	Normal Stress (psf)	Shear Rate (in./min)	Soaking Stress (psf)	Soaking Time (hour)	Consolidation Stress (psf)	Consolidation Time (hour)	Lower Soil γ_{di} (pcf)	Lower Soil ω_i (%)	Lower Soil ω_f (%)	Upper Soil γ_{di} (pcf)	Upper Soil ω_i (%)	Upper Soil ω_f (%)	GCL ω_i (%)	GCL ω_f (%)	Shear Stress τ_p (psf)	Shear Stress τ_{LD} (psf)	Failure Mode
1A	12 x 12	480	0.004	-	-	480	24	105.2	18.3	16.9	TIP	4.4	10.2	-	-	350	335	(1)

Notes: (1) Sliding (i.e., shear failure) occurred between the protective soil and geotextile during the test.
(2) The reported total-stress parameters of friction angle and adhesion were determined from a best-fit line drawn through the test data. Caution should be exercised in using these strength parameters for applications involving normal stresses outside the range of the stresses covered by the test series. The large-displacement (LD) shear strength was calculated using the shear force measured at the end of the test. TIP refers to the fact that the material was tamped in-place and its dry unit weight was not recorded.

SGI TESTING SERVICES, LLC

DATE OF TEST:	15 to 16 July 2002
FIGURE NO.	A-3
PROJECT NO.	SGI2064
DOCUMENT NO.	SGI02106
FILE NO.	

ATTACHMENT 5

STATIC AND SEISMIC SLOPE STABILITY ANALYSES OF FINAL COVER SYSTEM CONTAINING AGRU/AMERICA, INC. STRUCTURED GEOMEMBRANE

Written by: Tamer Elkady Date: 04 /2 /18 (YY MM DD) Reviewed by: Majdi Othman Date: 04/ 2 /18 (YY MM DD)

Client: WMI Project: Piedmont LF Project/Proposal No.: GD3294 Task No: 01

STABILITY ANALYSIS OF FINAL COVER SYSTEM FOR THE PIEDMONT LANDFILL

PURPOSE

The purpose of the analyses presented in this calculation package is to evaluate static and seismic stability of the final cover system that includes an Agru America, Inc. (A/A) Structured Geomembrane (i.e., Super Gripnet® Liner or A/A Drain Liner), referred to hereafter as the A/A final cover system.

METHOD OF ANALYSES

Static Stability Analysis:

Static slope stability of a landfill final cover system can be analyzed assuming infinite slope conditions or finite slope conditions. The infinite slope stability analysis method considers a slope of infinite length whereby the driving and resisting forces occur only along or parallel to an interface (i.e., slip plane). The finite slope stability analysis method considers a slope of finite length and additionally takes into account soil strength above a slip plane, primarily as a toe-buttressing effect. Since the final cover slopes at the Piedmont Landfill are relatively short, the finite slope stability analysis method is appropriate.

The finite slope stability factor of safety equation, as formulated by Giroud, et al. [1995], is:

$$FS=\left[\frac{\gamma_t(t-t_w)+\gamma_b t_w}{\gamma_t(t-t_w)+\gamma_{sat}t_w}\right]\frac{\tan\delta}{\tan\beta}+\frac{a/\sin\beta}{\gamma_t(t-t_w)+\gamma_{sat}t_w}$$

$$+\left[\frac{\gamma_t(t-t^*_w)+\gamma_b t^*_w}{\gamma_t(t-t_w)+\gamma_{sat}t_w}\right]\left[\frac{\tan\phi/(2\sin\beta\cos^2\beta)}{1-\tan\beta\tan\phi}\right]\frac{t}{h}$$

$$+\left[\frac{1}{\gamma_t(t-t_w)+\gamma_{sat}t_w}\right]\left[\frac{1/(\sin\beta\cos\beta)}{1-\tan\beta\tan\phi}\right]\frac{ct}{h} \qquad (1)$$

where:
- FS = factor of safety;
- δ = interface friction angle;
- a = apparent interface adhesion;
- ϕ = soil internal friction angle;
- c = apparent soil cohesion;

γ_t = moist soil unit weight;
γ_b = buoyant soil unit weight;
γ_{sat} = saturated soil unit weight;
t = depth of cover soil above critical interface;
t_w = water depth above critical interface;
t_w^* = water depth at slope toe;
β = slope inclination; and
h = vertical height of slope.

It should be noted that while the above equation is specifically for an interface above a geomembrane, or similar layers, it can also be applied to interfaces below the geomembrane by changing the coefficient of the first term, (i.e., the coefficient of $\tan\delta / \tan\beta$) to 1.0. The slope geometry, which is used to derive the above equation, is shown in Figure 1.

<u>Seismic Stability Analysis:</u>

A pseudo-static slope stability analysis is performed for the A/A final cover system. The pseudo-static factor of safety is estimated by performing an infinite slope stability analysis using Equation 2 [Matasović, 1991]:

$$FS = \frac{c/(\gamma z \cos^2\beta) + \tan\phi \; [1 - \gamma_w (z - d_w)/(\gamma z)] - k_s \tan\beta \tan\phi}{k_s + \tan\beta} \qquad (2)$$

where: FS = factor of safety;
k_s = peak average horizontal acceleration as a fraction of gravity;
γ = unit weight of slope material(s) in pcf;
γ_w = unit weight of water in pcf;
c = cohesion in psf;
β = slope angle in degrees;
ϕ = angle of internal friction on the assumed failure surface in degrees;
z = depth to the assumed failure surface in ft; and
d_w = depth to the water table (assumed parallel to the slope) in ft.

The peak horizontal acceleration at the top of the Piedmont Landfill is estimated using a chart (Figure 2) developed by Idriss [1990], as presented by Kavazanjian and Matasović [1994].

A calculated factor of safety greater than 1.0 suggests that no permanent seismic deformation is expected. A factor of safety less than 1.0, however, suggests permanent deformation can occur. The amount of seismic displacement can be computed based on k_s and the yield acceleration, K_y. The yield acceleration is the horizontal acceleration which results in a pseudo-static factor of safety of 1.0. The yield acceleration may be calculated using Equation 3 [Matasović, 1991]:

$$K_y = \frac{c/(\gamma \bullet z \bullet \cos^2\beta) + \tan\phi\,[1 - \gamma_w (z - d_w)/(\gamma \bullet z)] - \tan\beta}{1 + \tan\beta\tan\phi} \qquad (3)$$

The seismic displacement, corresponding to the computed K_y/k_s ratio, is estimated using the results presented by Hynes and Franklin [1984] and the "modified mean + one standard deviation curve" developed by GeoSyntec as presented in Figure 3. The "modified mean + one standard deviation curve" considers data associated with only large earthquakes, and therefore, is more conservative to use. This procedure is consistent with those given in the USEPA guidance document [USEPA; 1995].

TARGET FACTORS OF SAFETY

The target FS for the static stability analysis using peak and residual shear strength values are 1.5 and 1.2, respectively. For seismic analysis, the permanent deformation is considered acceptable if it is less than 6 to 12 in.

DESIGN PEAK GROUND ACCELERATION

The maximum horizontal acceleration (MHA) at the site was assumed to be the maximum expected horizontal acceleration depicted on a seismic hazard map, with a 90 percent or greater probability that the acceleration will not be exceeded in 250 years. Therefore, the MHA was selected to be 0.11g based on United States Geological Survey (USGS) map. Based on this value, the corresponding peak horizontal acceleration at the top of the Piedmont Landfill may be estimated using Figure 2 as 0.21 g.

FINAL COVER DATA

As stated earlier, the configurations of the currently permitted final cover systems will not change except that the geosynthetic components of the final cover systems (i.e., geomembrane and geocomposite drainage layer) will be replaced by an A/A Structured Geomembrane. The protective soil component of the proposed final cover system was assumed to have a unit weight of 120 pcf and shear parameters of $c = 0$ and $\phi = 30°$. Critical conditions for the evaluation of the stability of final cover system for the Piedmont Landfill considers a slope of 33.3 percent (3H:1V) and a length of 115 ft.

The water depth in the drainage layer above the geomembrane (t_w) was calculated using the HELP model [Schroeder, 1994] in the calculation package titled "*Hydraulic Performance Analysis of Final Cover System Containing Agru/America, Inc. Structured Geomembrane*". Based on the HELP analysis results, the average peak daily water depth was estimated to be 0.013 inches (0.0011 ft).

The final cover system static stability analyses were performed by solving the finite slope stability equation, (i.e., Equation 1) for various combinations of peak and residual internal/interface shear strength parameters (i.e., "δ" and "a" for above and below a geomembrane) based on the target factors of safety. The seismic stability analysis was performed assuming the final cover interfaces have the minimum peak interface/internal shear strength evaluated from the static stability analyses.

RESULTS AND CONCLUSIONS

Analyses were performed to estimate the minimum required peak and residual interface/internal shear strengths, expressed in terms of friction angle δ and adhesion a, that can achieve a static FS of 1.5 and 1.2, respectively. Equations used to calculate the FS above and below a geomembrane are coded in a spreadsheet presented herein as Tables 1 and 2, for peak and residual final cover shear strength parameters, respectively. The results of the static slope stability analyses are presented in Figures 4 and 5.

GeoSyntec have previously performed laboratory shear strength testing on a final cover system consisting of an A/A Structured Geomembrane (i.e., Super Gripnet® Liner). The Configuration for these tests includes a Super Gripnet® Liner sandwiched between a protective soil cover and a compacted clay liner. The results of these tests show the A/A final cover system has shear strength values that exceed the minimum requirements for peak and residual interface/internal shear strength parameters illustrated in Figure 4 and 5, respectively.

Results of the seismic stability analysis of the A/A final cover system using minimum peak interface/internal shear strength parameters are presented in Table 3. As illustrated in Table 3, calculated pseudo-static factors of safety for the final cover system were less than one, indicating permanent deformation can occur when subjected to the design earthquake event. However, the maximum calculated seismic deformation (illustrated in Figure 3) was 2.2 in., which is considered acceptable.

REFERENCES

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Hynes, M.E., and Franklin, A.G., "Rationalizing the Seismic Coefficient Method," *Miscellaneous Paper GL - 84 - 13,* U.S. Army Engineer Waterways Experiment Station, Vickburg, MS, 1984, pp. 34.

Idriss, I. M., "Response of Soft Soil Sites During Earthquakes," *Seed Memorial Symposium Proceedings,* Vol. 2, BiTech Publications, Berkeley, CA 1990.

Kavazanjian, E., Jr., and Matasović, N., "Seismic Analysis of Solid waste Landfills" Accepted for publication, Proceedings, GeoEnvironment 2000, Geotechnical Special Publication, ASCE, New York, NY, 1994.

Matasović, N., "Selection of Method for Seismic Slope Stability Analysis," *Proceedings of the 2nd International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics,* Vol. 2, St. Louis, 1991, pp. 1057-62.

Schroeder, P. R., N. M., Lloyd, C. M. and Zappi, P.A. (1994) "The Hydrologic Evaluation of Landfill Performance (HELP) Model: User's Guide for Version 3," EPA/600/R-94/168a, September 1994, U. S. Environmental Protection Agency Office of Research and Development, Washington, D.C.

USEPA, "*RCRA Subtitle D (258) Seismic Design Guidance for Municipal Solid Waste Landfill Facilities*", Office of Research and Development, EPA/600/R-95/051, April 1995.

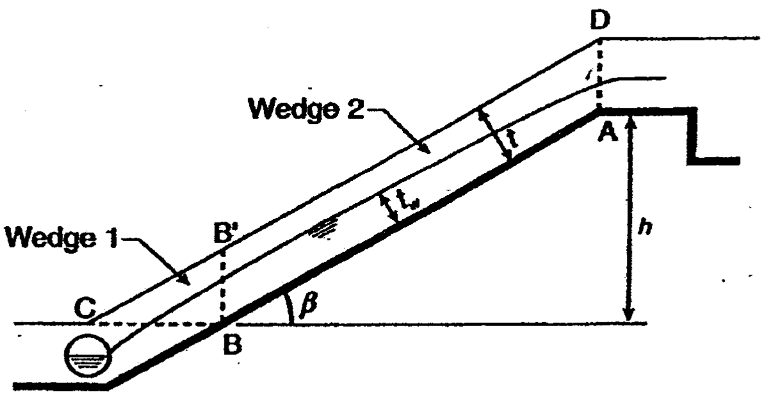
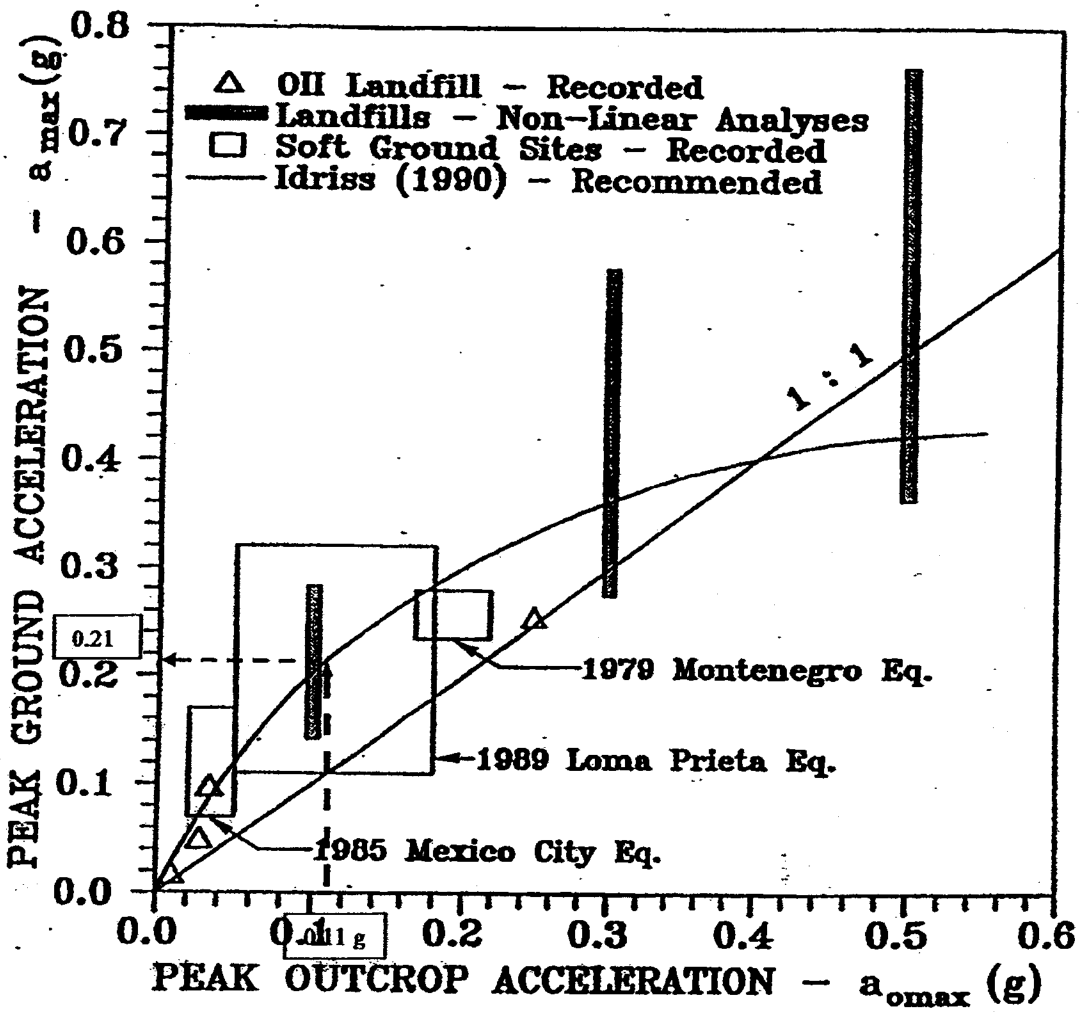


Figure 1. Slope Geometry Used to Derive the Finite Slope Stability Equation.



Observed Variations of Peak Horizontal Accelerations on Soft Soil and MSW Sites in Comparison to Rock Sites (Kavazanjian and Matasović, 1994).

Figure 2

Written by: Tamer Elkady Date: 04 /2 /18 (YY MM DD) Reviewed by: Majdi Othman Date: 04/ 2 /18 (YY MM DD)

Client: WMI Project: Piedmont LF Project/Proposal No.: GD3294 Task No: 01

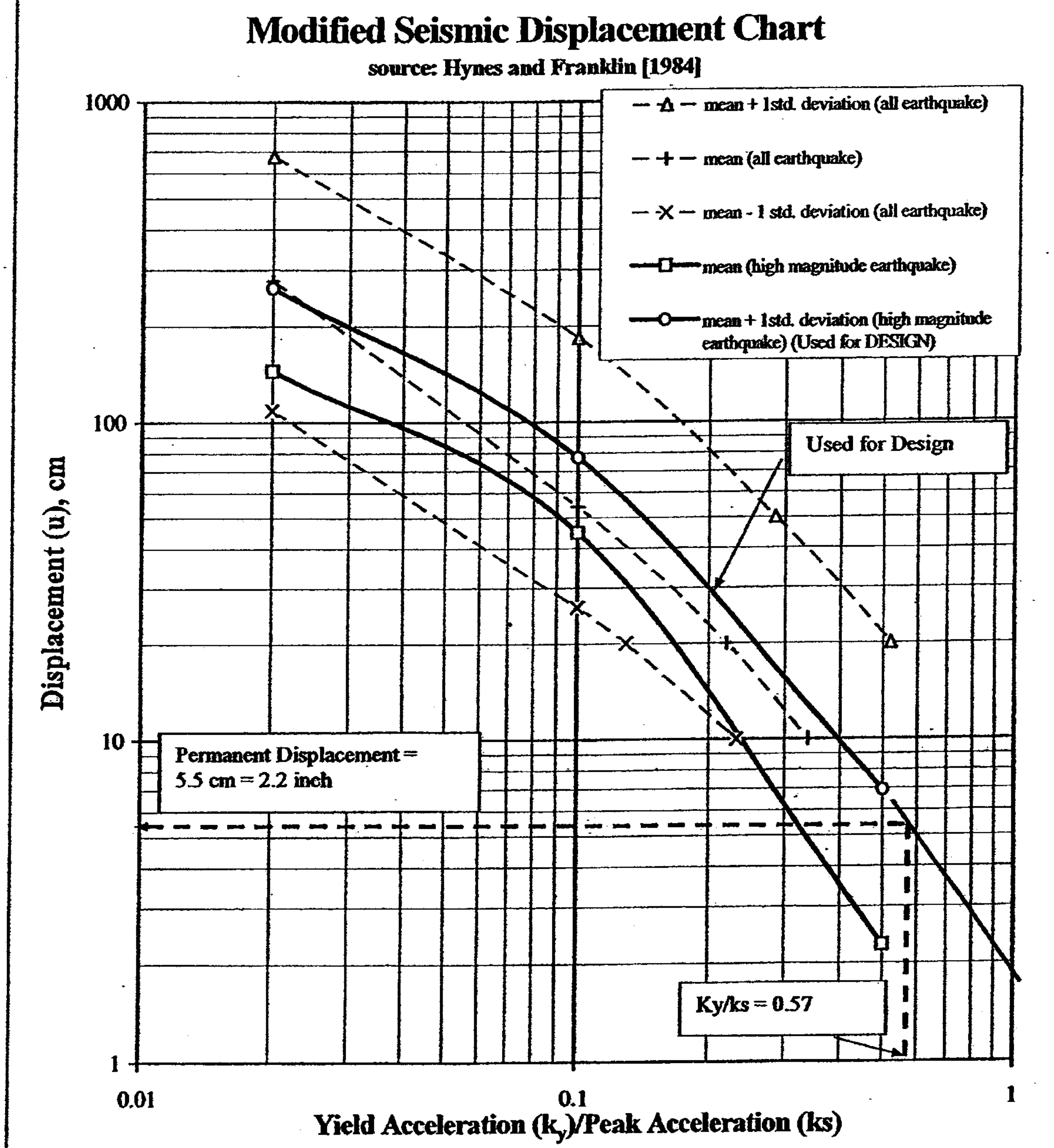


Figure 3

Written by: Tamer Elkady Date: 04 /2 /18 (YY MM DD) Reviewed by: Majdi Othman Date: 04/ 2 /18 (YY MM DD)

Client: WMI Project: Piedmont LF Project/Proposal No.: GD3294 Task No: 01

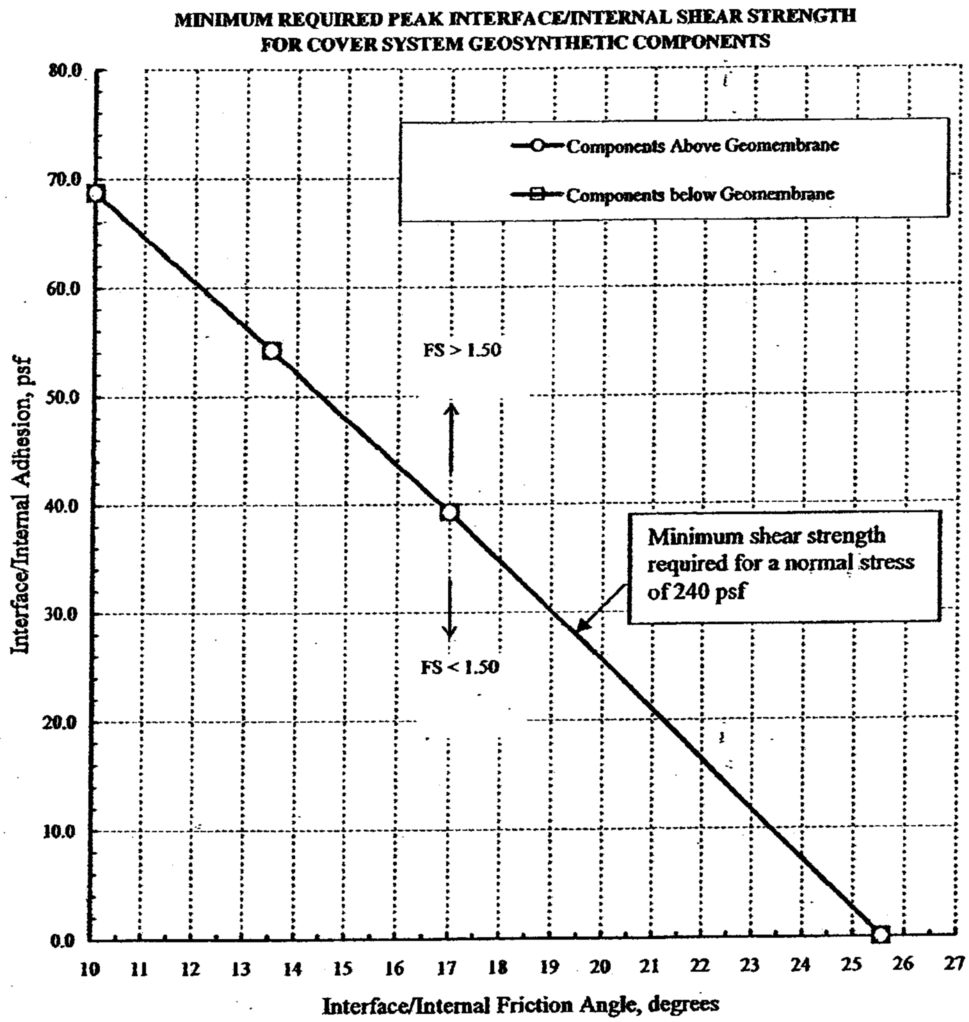


Figure 4. Peak Interface/Internal Shear Strength requirements for proposed final cover system

Written by: Tamer Elkady Date: 04 /2 /18 (YY MM DD) Reviewed by: Majdi Othman Date: 04/ 2 /18 (YY MM DD)

Client: WMI Project: Piedmont LF Project/Proposal No.: GD3294 Task No: 01

Table 1. Spreadsheet for the evaluation of peak interface/internal shear strength of proposed final cover system using Finite Slope Equation [Giroud et al., 1995].

FS Above GEOMEMBRANE		
Input Parameters:		
γ_t (Unit wt of soil):	120	pcf
γ_{sat} (Saturated unit wt of soil):	120	pcf
γ_w (Unit wt of water):	62.4	pcf
γ_b (Buoyant unit wt of soil):	57.6	pcf
t_w (water thickness):	0.001	ft
t* (water thickness at slope toe):	0.001	ft
δ (weakest interface friction angle):	25.6	deg
φ (friction angle of soil):	30	deg
a (interface adhesion)	0.0	psf
c (cohesion of soil above geomembrane)	0	psf
T (Tension in Geosynthetics)	0	psf
h (height of slope):	38.33	ft
t (thickness of soil layer)	2.0	ft
β (slope angle)	18.4	deg
FS	[redacted]	

FS Below GEOMEMBRANE		
Input Parameters:		
γ_t (Unit wt of soil):	120	pcf
γ_{sat} (Saturated unit wt of soil):	120	pcf
γ_w (Unit wt of water):	62.4	pcf
γ_b (Buoyant unit wt of soil):	57.6	pcf
t_w (water thickness):	0.001	ft
t* (water thickness at slope toe):	0.001	ft
δ (weakest interface friction angle):	25.6	deg
φ (friction angle of soil):	30	deg
a (interface adhesion)	0	psf
c (cohesion of soil above geomembrane)	0	psf
T (Tension in Geosynthetics)	0	psf
h (height of slope):	38.33	ft
t (thickness of soil layer)	2.0	ft
β (slope angle)	18.4	deg
FS	[redacted]	

Written by: Tamer Elkady Date: 04 /2 /18 (YY MM DD) Reviewed by: Majdi Othman Date: 04/ 2 /18 (YY MM DD)

Client: WMI Project: Piedmont LF Project/Proposal No.: GD3294 Task No: 01

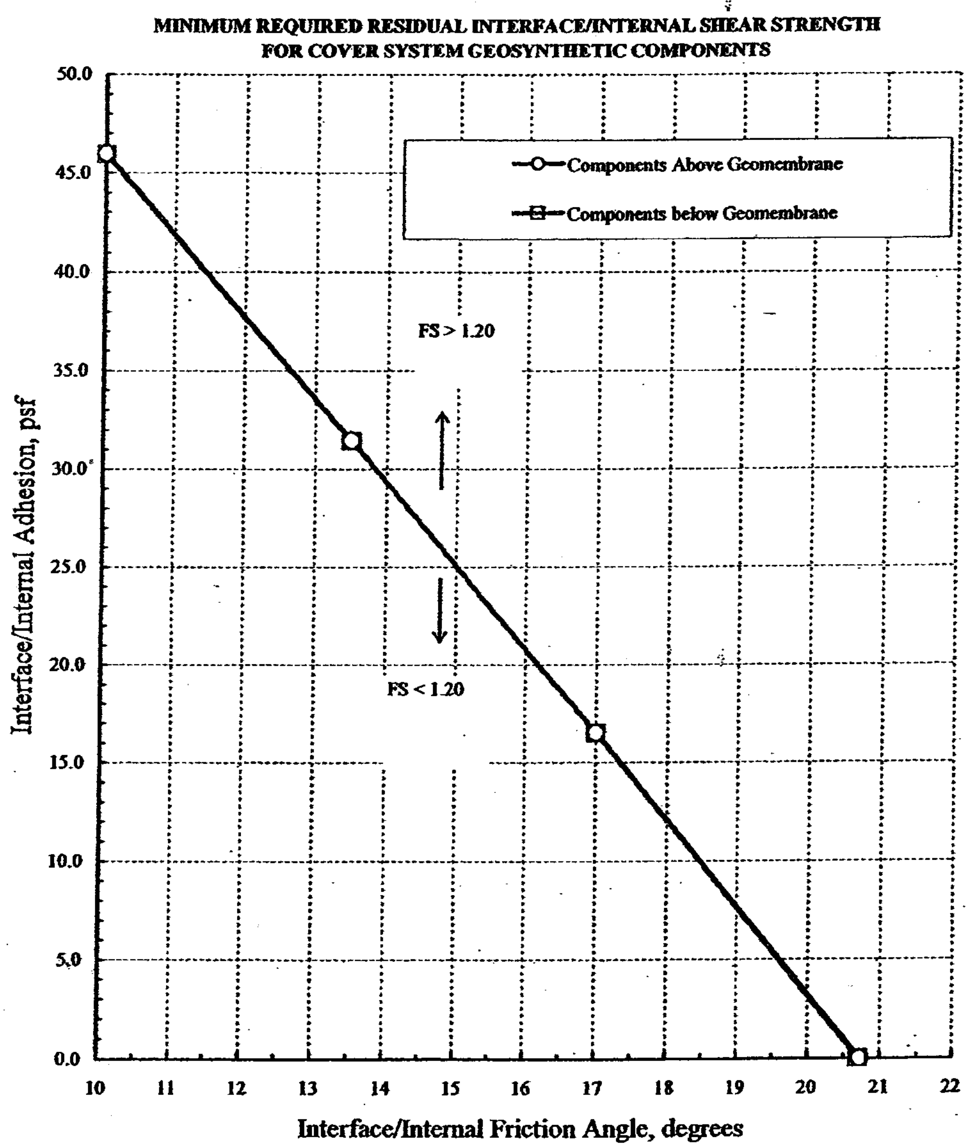


Figure 5. Residual Interface/Internal Shear Strength requirements for proposed final cover system

Written by:	Tamer Elkady	Date:	04 /2 /18 (YY MM DD)	Reviewed by:	Majdi Othman	Date:	04/ 2 /18 (YY MM DD)
Client:	WMI	Project:	Piedmont LF	Project/Proposal No.:	GD3294	Task No:	01

Table 2. Spreadsheet for the evaluation of residual interface/internal shear strength of proposed final cover system using Finite Slope Equation [Giroud et al., 1995].

FS Above GEOMEMBRANE		
Input Parameters:		
γ_t (Unit wt of soil):	120	pcf
γ_{sat} (Saturated unit wt of soil):	120	pcf
γ_w (Unit wt of water):	62.4	pcf
γ_b (Buoyant unit wt of soil):	57.6	pcf
t_w (water thickness):	0.001	ft
t* (water thickness at slope toe):	0.001	ft
δ (weakest interface friction angle):	20.7	deg
ϕ (friction angle of soil):	30	deg
a (interface adhesion)	0.0	psf
c (cohesion of soil above geomembrane)	0	psf
T (Tension in Geosynthetics)	0	psf
h (height of slope):	38.33	ft
t (thickness of soil layer)	2.0	ft
β (slope angle)	18.4	deg
FS	[redacted]	

FS Below GEOMEMBRANE		
Input Parameters:		
γ_t (Unit wt of soil):	120	pcf
γ_{sat} (Saturated unit wt of soil):	120	pcf
γ_w (Unit wt of water):	62.4	pcf
γ_b (Buoyant unit wt of soil):	57.6	pcf
t_w (water thickness):	0.001	ft
t* (water thickness at slope toe):	0.001	ft
δ (weakest interface friction angle):	20.7	deg
ϕ (friction angle of soil):	30	deg
a (interface adhesion)	0	psf
c (cohesion of soil above geomembrane)	0	psf
T (Tension in Geosynthetics)	0	psf
h (height of slope):	38.33	ft
t (thickness of soil layer)	2.0	ft
β (slope angle)	18.4	deg
FS	[redacted]	

Written by: Tamer Elkady Date: 04 /2 /18 (YY MM DD) Reviewed by: Majdi Othman Date: 04/ 2 /18 (YY MM DD)

Client: WMI Project: Piedmont LF Project/Proposal No.: GD3294 Task No: 01

Table 3. Seismic Analysis Using Peak Interface/Internal Shear Strength.

Calculation of Factor of Safety and Yield Acceleration
For Infinite Slope Conditions
Using Equation from Matasovic [1991]

$$FS=\frac{\left(\frac{c}{\gamma z\cos^2\beta}+\tan\phi\left(1-\frac{\gamma_w(z-d_w)}{\gamma z}\right)-K_s\tan\beta\tan\phi\right)}{Ks+\tan\beta}$$

$$k_y=\frac{\left(\frac{c}{\gamma z\cos^2\beta}+\tan\phi\left(1-\frac{\gamma_w(z-d_w)}{\gamma z}\right)-\tan\beta\right)}{1+\tan\beta\tan\phi}$$

Where:

- k_y = yield acceleration, g.;
- γ = unit weight of soil cover, pcf;
- γ_w = unit weight of water, pcf;
- c = cohesion along the assumed failure surface, psf;
- ϕ = friction angle along the assumed failure surface, degrees;
- β = slope angle, degrees;
- z = depth of the assumed failure surface, ft; and
- d_w = depth of water surface (assumed parallel to the slope), ft.
- k_s = peak average horizontal acceleration for potential slide mass, g. = a_{max}

Input parameters:	
γ, pcf	*120*
z, ft	*2*
β, degrees	*18.43*
γ_w, pcf	*62.4*
d_w, ft	*1.953*
ks, g	*0.21*

ϕ (degrees)	c (psf)	FS	k_y (g)	k_y/a_{max}	
25.6	0	0.809	0.121	0.57	<== minimum
17	39.3	0.851	0.137	0.65	
13.5	54.2	0.868	0.143	0.68	
10	68.7	0.884	0.150	0.72	

APPROVAL OF
18 FEBRUARY TECHNICAL DEMONSTRATION

NCDENR
North Carolina Department of Environment and Natural Resources

Dexter R. Matthews, Director | Division of Waste Management | Michael F. Easley, Governor
William G. Ross Jr., Secretary

March 26, 2004

Mike Loyd, P.G.
Market Area Manager
Piedmont Landfill and Recycling Center
251 New Hope Road
Wellford, SC 29385

RE: Modification to Solid Waste Permit No. 34-06
Revision to Geosynthetic Materials for Final Cover
Piedmont Landfill and Recycling Center

Dear Mr. Loyd:

The Solid Waste Section has completed its review of the Request for Approval of Alternate Geosythetic Materials for the Piedmont Landfill Final Cover System, prepared by GeoSyntec Consultants, received March 11, 2004. The section has determined that the Piedmont Landfill and Recycling Center has adequately demonstrated that the alternatives presented meet the requirements of Rule .1627(c)(2)(A) and (B), and hereby approves your request. The list of approved documents for Permit No. 34-06 has been revised to include the above-referenced document, and is attached.

If you have any questions or comments regarding this correspondence, please contact Sherri Coghill at (919)733-0692, ext. 259.

Sincerely,

James C Coffey

James C. Coffey, Chief
Solid Waste Section

cc: Majdi A. Othman, Ph.D., P.E., GeoSyntec Consultants
Brent Rockett, DWM
Wendy Simmons, DWM
Tim Jewett, DWM

24 MAY 2004 TECHNICAL DEMONSTRATION

GEOSYNTEC CONSULTANTS

1100 Lake Hearn Drive, Suite 200
Atlanta, Georgia 30342-1523• USA
~~Tel. (404) 705-9500 • Fax (404) 705-9400~~

24 May 2004

Ms. Sherri Coghill
Environmental Engineer
Division of Waste Management
North Carolina Department of Environment and Natural Resources
1646 Mail Service Center
Raleigh, NC 27699

Subject: Alternate Geosynthetic Materials
Piedmont Landfill Final Cover System
Forsyth County, North Carolina

Dear Ms. Coghill:

GeoSyntec Consultants (GeoSyntec) submitted a request to the North Carolina Department of Environment and Natural Resources (NCDENR) on 18 February 2004 to allow the use of alternate final cover system geosynthetic materials for the Piedmont Landfill located in Forsyth County, North Carolina. On 26 March 2004, NCDENR approved this request. The alternate materials consisted of Agru America, Inc. (A/A) high density polyethylene (HDPE) Structured Geomembranes (i.e., HDPE Super Gripnet® Liner and HDPE Micro Drain™ Liner)

The A/A Structured Geomembranes can also be manufactured from linear low density polyethylene (LLDPE) resin. LLDPE geomembranes are often used in final cover systems because of their flexibility and ability to accommodate differential settlements. Therefore, GeoSyntec is requesting NCDENR approve the use of 50-mil minimum A/A LLDPE Structured Geomembranes (i.e., LLDPE Super Gripnet® Liner and LLDPE Micro Drain Liner) for the use in the Piedmont Landfill final cover system.

The remainder of this letter includes technical information about the A/A LLDPE Structured Geomembranes, including results of laboratory shear strength testing performed on a final cover system containing an LLDPE Micro Drain Liner to demonstrate suitability of the proposed alternate geosynthetic materials for use in the final cover system of the Piedmont Landfill.

GD3294/ga040350.doc

A/A LLPDE STRUCTURED GEOMEMBRANE TECHNICAL INFORMATION

Technical information about the LLDPE Structured Geomembranes (i.e., LLDPE Micro Drain and LLDPE Super Gripnet® Liner) is provided in Attachment 1. As can be seen from the product data sheets included in Attachment 1, the physical properties of these structured geomembranes are equivalent to other commercially available LLDPE geomembranes.

GeoSyntec performed site-specific laboratory shear strength testing to evaluate interfacial shear strength characteristics of the Piedmont landfill final cover system encompassing an A/A LLDPE Geomembrane. The test configuration included an A/A LLDPE Micro Drain geomembrane sandwiched between materials similar to the currently permitted final cover system materials. Specifically, the test configuration consisted, form top to bottom, of: (i) protective soil; (ii) 8 oz/yd^2 geotextile; (iii) A/A LLDPE Micro Drain geomembrane; (iv) Bentomat ST GCL; and (v) compacted soil layer. The results of the testing are included in Attachment 2 to this letter.

In the 18 February 2004 letter from GeoSyntec to NCDENR, final cover shear strength requirements to achieve adequate factors of safety were described. Figures 4 and 5 of Attachment 5 of the 18 February 2004 letter illustrated these requirements. These figures are reproduced herein as Figures 1 and 2. The results of the site-specific shear strength test described in this letter (i.e., for a final cover system containing an LLDPE Micro Drain Liner) are also shown in Figures 1 and 2. From these figures, it is concluded that a final cover system containing an A/A LLDPE Structured Geomembrane should be able to meet the minimum shear strength requirements.

In the February 2004 request letter, the results of transmissivity tests were presented for the HDPE Structured Geomembrane. Based on analyses performed in that request, the measured transmissivity for the HDPE Structured Geomembrane were adequate. Although transmissivity testing was not specifically performed on the LLDPE Structured Geomembranes, the results are not expected to be significantly different than those measured for the HDPE Structured Geomembranes as the height and spacing of the drainage studs are identical.

Ms. Sherri Coghill
24 May 2004
Page 3

CLOSURE

NCDENR is requested to approve the A/A LLDPE Structured Geomembranes, for use in the Piedmont Landfill final cover system, as it meets the project specifications and performance requirements. Please contact either of the undersigned if you have any questions or comments related to this letter.

Sincerely,

T. Elkady

Tamer Elkady, Ph.D.
Senior Staff Engineer

Majdi Alone Othman

Majdi A. Othman, Ph.D., P.E.
Associate

Attachments

Copy to: Michael Loyd, Waste Management

GD3294/ga040350.doc

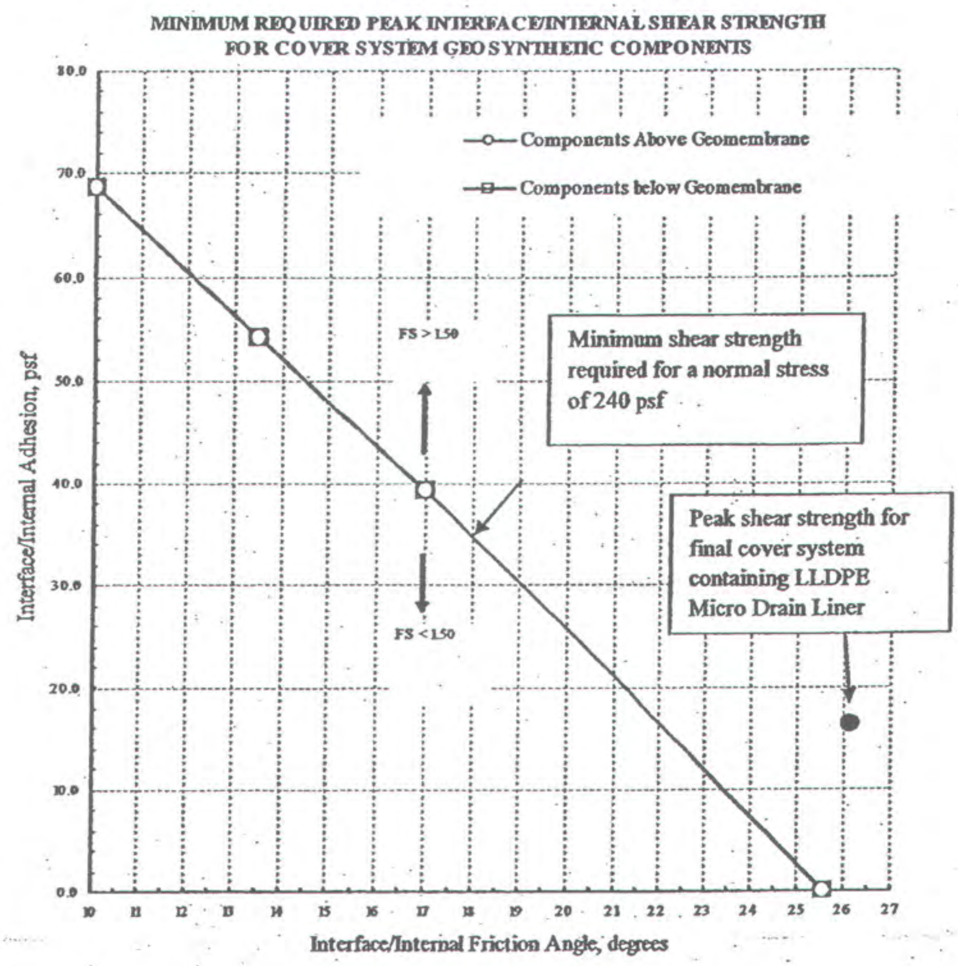


Figure 1. Peak Interface/Internal Shear Strength requirements for proposed final cover system

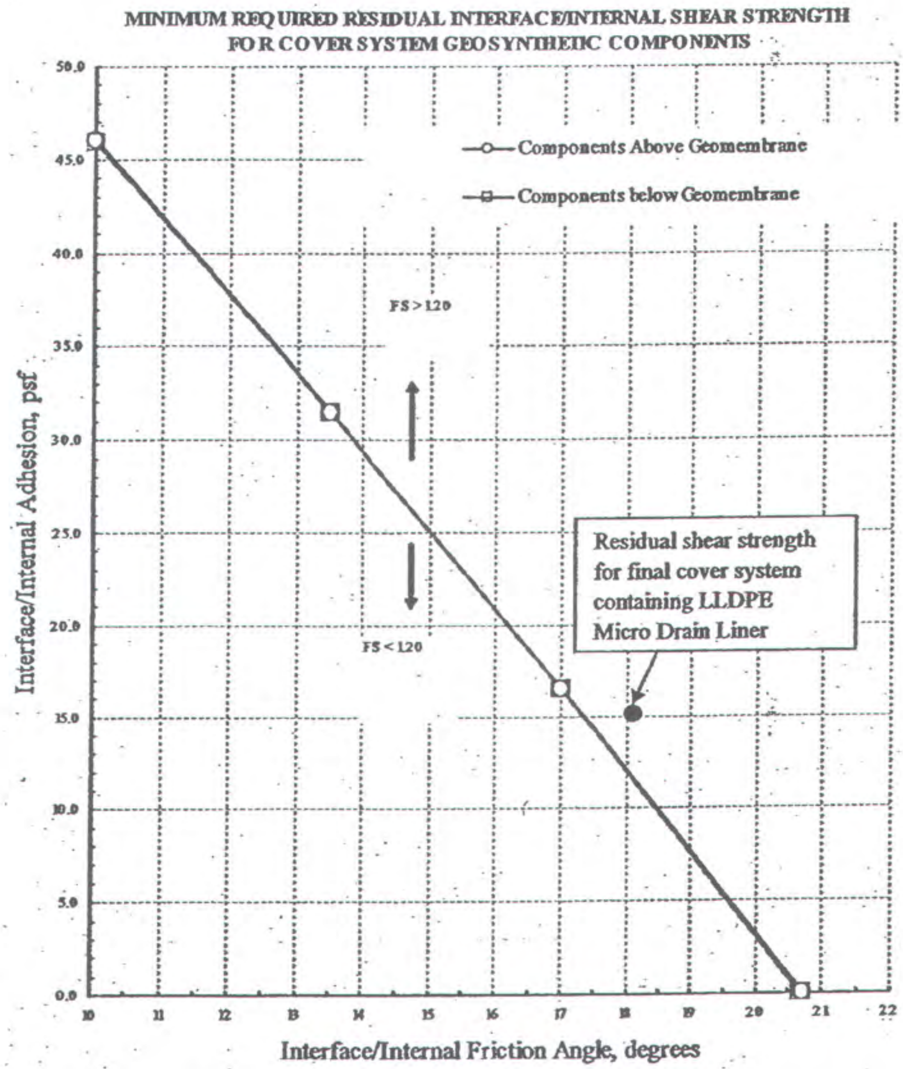


Figure 2. Residual Interface/Internal Shear Strength requirements for proposed final cover system

ATTACHMENT 1

AGRU/AMERICA, INC.
LLDPE STRUCTURED GEOMEMBRANE
TECHNICAL INFORMATION

LINEAR LOW DENSITY POLYETHYLENE SUPER GRIPNET® LINER

Product Data

Property	Test Method	Values			
Thickness (min. ave.), mil	ASTM D5994*	50	60	80	100
Thickness (lowest indiv.), mil	ASTM D5994*	50	54	72	90
Density, g/cc, minimum	ASTM D792, Method B	0.92	0.92	0.92	0.92
Tensile Properties (ave. both directions)	ASTM D6693, Type IV				
Strength @ Break (min. ave.), lb/in width	2 in/minute	105	126	168	210
Elongation @ Break (min. ave.), % (GL=2.0in)	5 specimens in each direction	300	300	300	300
Tear Resistance, lbs. (min. ave.)	ASTM D1004	30	40	53	67
Puncture Resistance, lbs. (min. ave.)	ASTM D4833	55	70	90	110
Carbon Black Content (range in %)	ASTM D4218	2 - 3	2 - 3	2 - 3	2 - 3
Carbon Black Dispersion (Category)	ASTM D5596	Only near spherical agglomerates for 10 views: 9 views in Cat. 1 or 2, and 1 view in Cat. 3			
Oxidative Induction Time, minutes	ASTM D3895, 200°C, 1 atm O_2	≥ 100	≥ 100	≥ 100	≥ 100
Melt Flow Index, g / 10 minutes	ASTM D1238, 190°C, 2.16kg	≤ 1.0	≤ 1.0	≤ 1.0	≤ 1.0
Oven Aging with HP OIT, (% retained after 90 days)	ASTM D5721 ASTM D5885, 150°C, 500psi O_2	60	60	60	60
UV Resistance	GRI GM11	20hr. Cycle @ 75°C / 4 hr. dark condensation @ 60°C			
with HP OIT, (% retained after 1600 hours)	ASTM D5885, 150°C, 500psi O_2	35	35	35	35
2% Secant Modulus, lb/in. (max.)	ASTM D5323	3000	3600	4800	6000
Axi-Symmetric Break Resistance Strain, % (min.)	ASTM D5617	30	30	30	30

These product specifications meet or exceed GRI's GM-13

* The thickness values may be changed due to project specifications (i.e., absolute minimum thickness)

Supply Information (Standard Roll Dimensions)

Thickness		Width		Length		Area		Weight	
mil	mm	ft	m	ft	m	ft^2	m^2	lbs	kg
50	1.25	23	7	197	60	4,531	420	2,060	934
60	1.5	23	7	197	60	4,531	420	2,060	934
80	2.0	23	7	165	50	3,795	350	1,973	895
100	2.5	23	7	165	50	3,795	350	2,418	1,096

Notes: 1.) All rolls are supplied with two slings. 2.) All rolls are wound on a 6 inch core. 3.) Special roll lengths are available on request.
4.) Standard rolls have a diameter of approximately 29 inches. 5.) A 40 foot standard container will hold 9 standard rolls. 6.) A 48 foot flatbed trailer will hold 12 standard rolls.

Executive Offices: 500 Garrison Road, Georgetown, SC 29440 843-546-0600 800-321-1379 Fax: 843-546-0516
Sales Office: 700 Rockmead, Suite 150, Kingwood, TX 77339 281-358-4741 800-373-2478 Fax: 281-358-5297
email: salesmkg@agruamerica.com

agru america

LINEAR LOW DENSITY POLYETHYLENE MICRO DRAIN™ LINER

Product Data

Property	Test Method	Values			
Thickness (min. ave.), mil	ASTM D5994*	50	57	76	95
Thickness (lowest indiv. for 8 of 10 spec.), mil	ASTM D5994*	50	54	72	90
Thickness (lowest indiv. for 1 of 10 spec.), mil	ASTM D5994*	50	51	68	85
Asperity Height (min. ave.), mil of textured side only	GRI GM-12	16	16	16	16
Density, g/cc, minimum	ASTM D792, Method B	0.92	0.92	0.92	0.92
Tensile Properties (ave. both directions)	ASTM D6693, Type IV				
Strength @ Break (min. ave.), lb/in width	2 in/minute	105	126	168	210
Elongation @ Break (min. ave.), % (GL=2.0in)	5 specimens in each direction	300	300	300	300
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Carbon Black Content (range in %)	ASTM D4218	2 - 3	2 - 3	2 - 3	2 - 3
Carbon Black Dispersion (Category)	ASTM D5596	Only near spherical agglomerates for 10 views: 9 views in Cat. 1 or 2, and 1 view in Cat. 3			
Oxidative Induction Time, minutes	ASTM D3895, 200°C, 1 atm O_2	≥ 100	≥ 100	≥ 100	≥ 100
Melt Flow Index, g / 10 minutes	ASTM D1238, 190°C, 2.16kg	≤ 1.0	≤ 1.0	≤ 1.0	≤ 1.0
Oven Aging with HP OIT, (% retained after 90 days)	ASTM D5721 ASTM D5885, 150°C, 500psi O_2	60	60	60	60
UV Resistance	GRI GM11	20hr. Cycle @ 75°C / 4 hr. dark condensation @ 60°C			
with HP OIT, (% retained after 1600 hours)	ASTM D5885, 150°C, 500psi O_2	35	35	35	35
2% Secant Modulus, lb/in. (max.)	ASTM D5323	3000	3600	4800	6000
Axi-Symmetric Break Resistance Strain, % (min.)	ASTM D5617	30	30	30	30

These product specifications meet or exceed GRI's GM-13

* The thickness values may be changed due to project specifications (i.e., absolute minimum thickness)

Supply Information (Standard Roll Dimensions)

Thickness		Width		Length		Area		Weight	
mil	mm	ft	m	ft	m	ft^2	m^2	lbs	kg
50	1.25	23	7	197	60	4,531	420	2,060	934
60	1.5	23	7	197	60	4,531	420	2,060	934
80	2.0	23	7	165	50	3,795	350	1,973	895
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Notes: 1.) All rolls are supplied with two slings. 2.) All rolls are wound on a 6 inch core. 3.) Special roll lengths are available on request.
4.) Standard rolls have a diameter of approximately 29 inches. 5.) A 40 foot standard container will hold 9 standard rolls. 6.) A 48 foot flatbed trailer will hold 12 standard rolls.

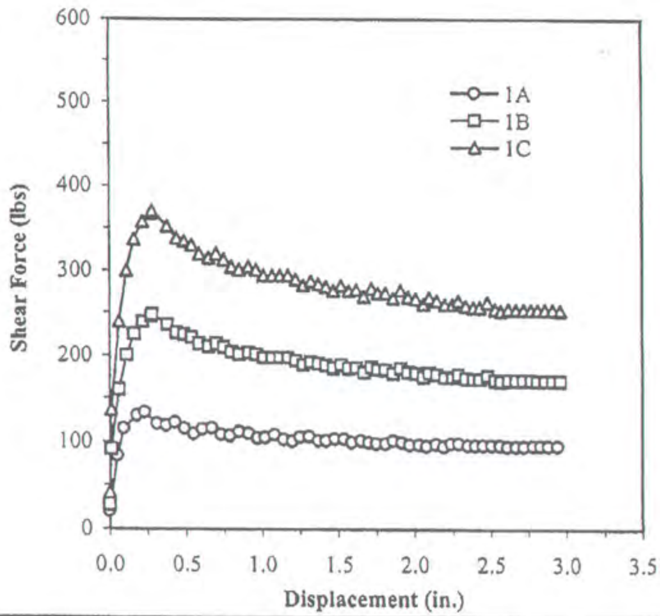
Executive Offices: 500 Garrison Road, Georgetown, SC 29440 843-546-0600 800-321-1379 Fax: 843-546-0516
Sales Office: 700 Rockmead, Suite 150, Kingwood, TX 77339 281-358-4741 800-373-2478 Fax: 281-358-5297
email: salesmkg@agruamerica.com

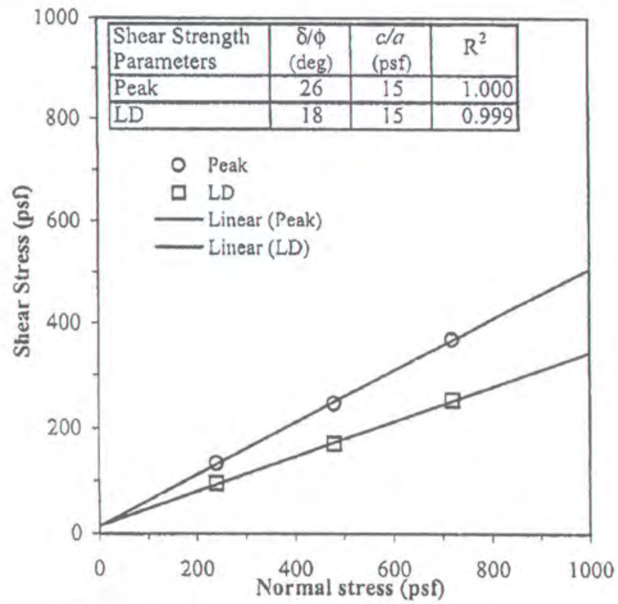
ATTACHMENT 2
RESULTS OF LABORATORY SHEAR STRENGTH TESTING ON FINAL COVER SYSTEM CONTAINING AGRU/AMERICA, INC. LLDPE STRUCTURED GEOMEMBRANE

GEOSYNTEC CONSULTANTS, INC. - WMI/PIEDMONT LANDFILL

INTERFACE DIRECT SHEAR TESTING (ASTM D 5321)

Test Series 1: site soil (SF-1) against wetted Nicolon S800 geotextile against wetted 50-mil (minimum) Agru MicroDrain LLDPE geomembrane (drainage cylinders up and Microspikes down) against woven geotextile side of prehydrated Bentomat ST GCL against wetted site soil (SF-1) under consolidated conditions





Shear Strength Parameters	δ/φ (deg)	c/a (psf)	R^2
Peak	26	15	1.000
LD	18	15	0.999

Test	Shear	Normal	Shear	Soaking		Consolidation		Lower Soil			Upper Soil			GCL		Shear Stress		Failure
No.	Box Size (in. x in.)	Stress (psf)	Rate (in./min)	Stress (psf)	Time (hour)	Stress (psf)	Time (hour)	γ_{di} (pcf)	ω_i (%)	ω_f (%)	γ_{di} (pcf)	ω_i (%)	ω_f (%)	ω_i (%)	ω_f (%)	τ_p (psf)	τ_{LD} (psf)	Mode
1A	12 x 12	240	0.040	-	-	240	0.5	101.0	16.2	18.5	100.3	17.0	23.1	16.4	89.5	133	95	(1)
1B	12 x 12	480	0.040	-	-	480	0.5	100.8	16.5	18.2	100.2	17.2	22.7	16.4	88.4	247	170	(1)
1C	12 x 12	720	0.040	-	-	720	0.5	100.9	16.3	17.7	100.5	16.8	22.5	16.4	87.9	370	253	(1)

Notes: (1) Sliding (i.e., shear failure) occurred between the woven geotextile side of the Bentomat ST GCL and the Microspike side of the Agru Microdrain geomembrane during each test.
(2) The reported total-stress parameters of friction angle and adhesion were determined from a best-fit line drawn through the test data. Caution should be exercised in using these strength parameters for applications involving normal stresses outside the range of the stresses covered by the test series. The large-displacement (LD) shear strength was calculated using the shear force measured at the end of the test.

SGI TESTING SERVICES, LLC

DATE OF TEST:	18 to 19 May 2004
FIGURE NO.	A-1
PROJECT NO.	SGI4035
DOCUMENT NO.	SGI04048
FILE NO.	

APPROVAL OF
24 MAY TECHNICAL DEMONSTRATION

NCDENR
North Carolina Department of Environment and Natural Resources

Dexter R. Matthews, Director

Division of Waste Management

Michael F. Easley, Governor
William G. Ross Jr., Secretary

May 26, 2004

Mike Loyd, P.G.
Market Area Manager
Piedmont Landfill and Recycling Center
251 New Hope Road
Wellford, SC 29385

RE: Modification to Solid Waste Permit No. 34-06
Revision to Geosynthetic Materials for Final Cover
Piedmont Landfill and Recycling Center

Dear Mr. Loyd:

The Solid Waste Section hereby approves the requested modification to the final cover system at Piedmont MSW Landfill submitted on May 25, 2004 by GeoSyntec Consultants. The modification allows use of 50-mil minimum LLDPE structured geomembrane manufactured by Agru America, Inc. The enclosed list of approved documents for Permit No. 34-06 includes the request.

If you have any questions or comments regarding this correspondence, please contact Sherri Coghill at (919) 733-0692, ext. 259.

Sincerely,

James C. Coffey

James C. Coffey, Chief
Solid Waste Section

cc: Majdi A. Othman, Ph.D., P.E., GeoSyntec Consultants
Brent Rockett, DWM
Wendy Simmons, DWM
Tim Jewett, DWM

1646 Mail Service Center, Raleigh, North Carolina 27699-1646
Phone 919-733-4996 \ FAX 919-715-3605 \ Internet http://wastenotnc.org

ATTACHMENT 3
Approved Documents
(Revised May 26, 2004)

PART I: GENERAL FACILITY CONDITIONS
PART II: MUNICIPAL SOLID WASTE LANDFILL CONDITIONS

1. Transition Application and Permit Renewal, Piedmont Landfill and Recycling Center, *Kernersville, North Carolina*, 2 Volumes, Prepared by Rust Environment and Infrastructure, Waste Management of Carolinas, Inc. December 1993, Revised April 1994, July 1994, October 1994, and February 1995.
2. Transition Application and Permit Renewal Site Development Plans for Waste Management of Carolinas, Inc., Piedmont Landfill and Recycling Center, Forsyth County North Carolina, Permit No. 34-06, 21 Drawings, Prepared by Rust Environment and Infrastructure, December 1993, Revised April 1994. (Drawings also in Vol 2 of Ref 1)
3. Report for the Design Hydrogeologic Investigation, Piedmont Landfill and Recycling Center, *Kernersville, North Carolina,* Rust Environment and Infrastructure, April 1994. Revised July 1994.
4. Water Quality Monitoring Plan for the Unconstructed Portion of the Piedmont Landfill and Recycling Center, *Kernersville, North Carolina,* Rust Environment and Infrastructure, May 1994. Revised July 1994.
5. Construction Quality Assurance Report, Phase 3, Cell 1, Subcell 1, Piedmont Landfill and Recycling Center, Kernersville, NC, prepared by Rust Environment and Infrastructure, January, 1995.
6. Construction Quality Assurance Report, Phase 3, Cell 1, Subcell 1, Piedmont Landfill and Recycling Center, Kernersville, NC, prepared by Rust Environment and Infrastructure, and letter approving operation dated May 19, 1995.
7. Construction Quality Assurance Report, Phase 3, Cell 2, Subcell 1, Piedmont Landfill and Recycling Center, Kernersville, NC, prepared by Rust Environment and Infrastructure, and letter of approval to operate dated January 18, 1996.
8. Interim and Final Construction Quality Assurance Reports, Phase 3, Cell 2, Subcell 2, Piedmont Landfill and Recycling Center, Kernersville, NC, prepared by Rust Environment and Infrastructure, and letter of approval to operate dated July 7, 1997.
9. Request to revise gas collection system design, and Section approval granted on May22, 1996.
10. Request to utilize alternative daily cover, dated July 19, 1996, and Section approval granted on July 31, 1996.
11. Request to modify anchor trench detail, dated November 6, 1996, and approval dated November 7, 1996.
12. Request to reduce permitted service area, dated November 6, 1996, and Section approval granted November 13, 1996.
13. Request to revise final cover on north side of Module 1, Phase II, dated December 6, 1996, and approval letter dated December 19, 1996.

14. Request to revise configuration of Cell 2, Subcell 2, dated 1/27/97, and Section approval granted 1/29/97.
15. Request of March 3, 1997, to utilize tire chips as protective layer, and letter of approval dated March 27, 1997.
16. Request of June 1997, prepared by EcoLogic Associates to revise final contours.
17. Request to revise final cover slopes from 4:1 to 3:1 and approval letter of October 31, 1997.
18. Request of December 7, 1997 to revise service area.
19. Request of May 20, 1998, to revise footprint and final cover design, prepared by EcoLogic Associates.
20. Request of June 24, 2002 to modify gas extraction system.
21. Revisions to Facility Plan text and site plan drawing, Operation Plan text, and Closure and Post-Closure Plan text, revised by Joyce Engineering, Inc., submitted May 2002, as revised through December 2002.
22. Request for Continued Operation dated November 18, 2003.
23. Request for Approval of Alternate Geosynthetic Materials for the Piedmont Landfill Final Cover System, prepared by Geosyntec Consultants, received March 11, 2004.
24. Request for Approval of Alternate Geosynthetic Materials for the Piedmont Landfill Final Cover System, prepared by Geosyntec Consultants, received May 25, 2004.

PART III: CONSTRUCTION AND DEMOLITION LANDFILL CONDITIONS
(NOT APPLICABLE)

PART IV: LAND CLEARING AND INERT DEBRIS LANDFILL CONDITIONS
(NOT APPLICABLE)

PART V: YARD WASTE CONDITIONS
(NOT APPLICABLE)

PART VI: MISCELLANEOUS TREATMENT AND PROCESSING FACILITIES CONDITIONS
(NOT APPLICABLE)

APPENDIX C

PHOTOGRAPHIC DOCUMENTATION

Photograph 1: Landfill gas uplifting geosynthetic cap 3 days prior to slope movement, looking south.

Photograph 2: Landfill gas uplifting geosynthetic cap 3 days prior to slope movement, looking east, cross section view of photograph 1.

NCP2005-3184\piedmont\photos-part-4.cdr

Photograph 3: Cracking within protective cover due to slope movement.
Date: 28 August 2004

Photograph 4: Cracking within protective cover due to slope movement.
Date: 28 August 2004

NCP2005-3184\piedmont\photos-part-4.cdr

Photograph 5: Gas header pipe shown deforming during slope movement.
Date: 28 August 2004

Photograph 6: Gas header pipe shown fully deformed during slope movement.
Date: 28 August 2004

Photograph 7: Gas well head displaced during slope movement.
Date: 28 August 2004

Photograph 8: Landfill gas uplifting geosynthetic cap during excavation of exploratory trench through the protective cover layer after the slope movement
Date: 31 August 2004.

NCP2005-3184\piedmont\photos-part-4.cdr

Photograph 9: Area of cover system movement located south of perimeter road and landfill gas flare. Photograph taken from western Phase 1 Closure slope looking east.

Photograph 10: Area of cover system movement showing exposed geosynthetic clay liner and displaced sideslope drainage swales.

Photograph 11: Area of cover system movement down slope of exposed geosynthetic clay liner area and upslope of sideslope haul road.

Photograph 12: Area of cover system movement down slope of exposed geosynthetic clay liner area. Photograph taken from sideslope haul road looking east.

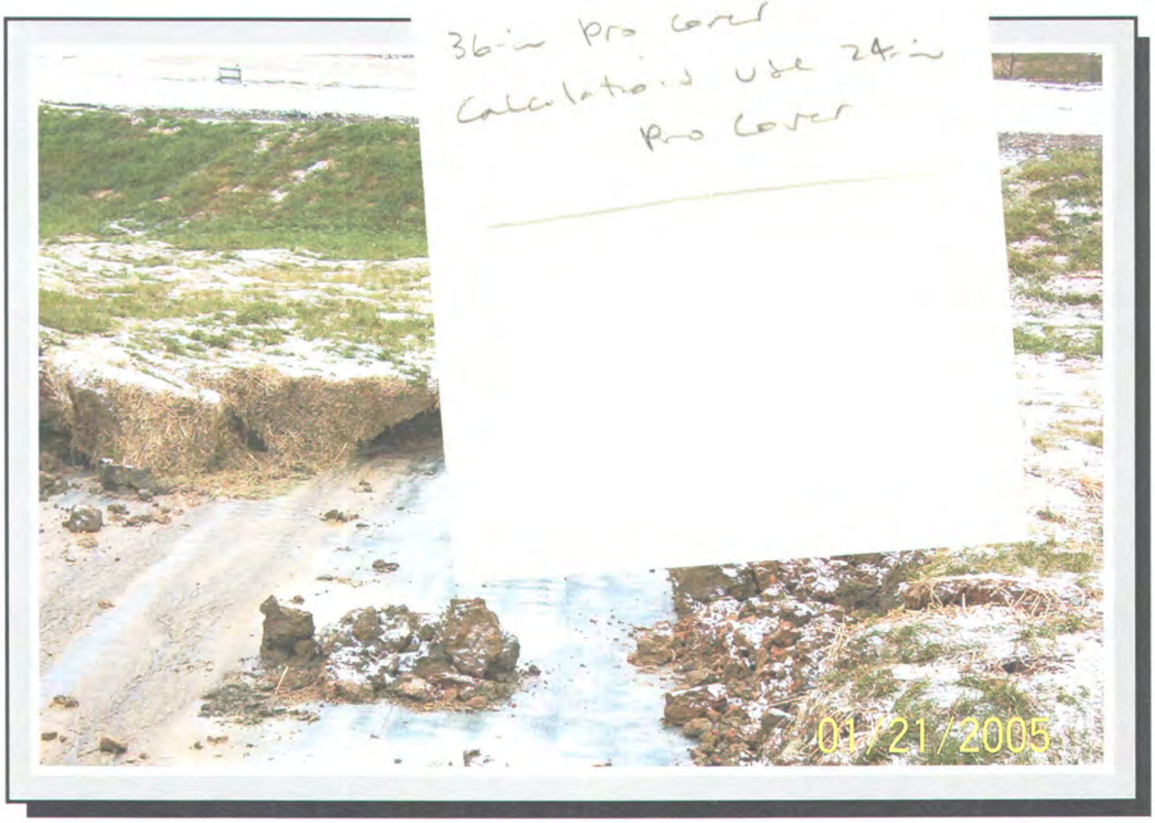
Photograph 13: Area of cover system movement down slope of exposed geosynthetic clay liner area. Photograph taken from sideslope haul road looking west.

Photograph 14: Exposed geosynthetic clay liner and protective cover layer looking west from top of landfill, Phase 1 Closure area located in background.

Photograph 15: Exposed geosynthetic clay liner and protective cover layer looking upslope from eastern most boundary of cover system movement to south west.

Photograph 16: Exposed geosynthetic clay liner and protective cover layer looking upslope from eastern edge of cover system movement to south west.

Photograph 17: Exposed eastern edge of geosynthetic clay liner, protective cover layer, and geomembrane cap material.



36-in Pro cover
Calculations use 24-in
Pro cover

Photograph 18: Exposed eastern edge of geosynthetic clay liner and protective cover layer looking down slope toward perimeter road and sideslope haul road.

NCP2005-3184\piedmont\photos-part-1.cdr

Photograph 19: Overturned well head located upslope of sideslope haul road near the base of the cover system movement looking west towards landfill gas flare station.

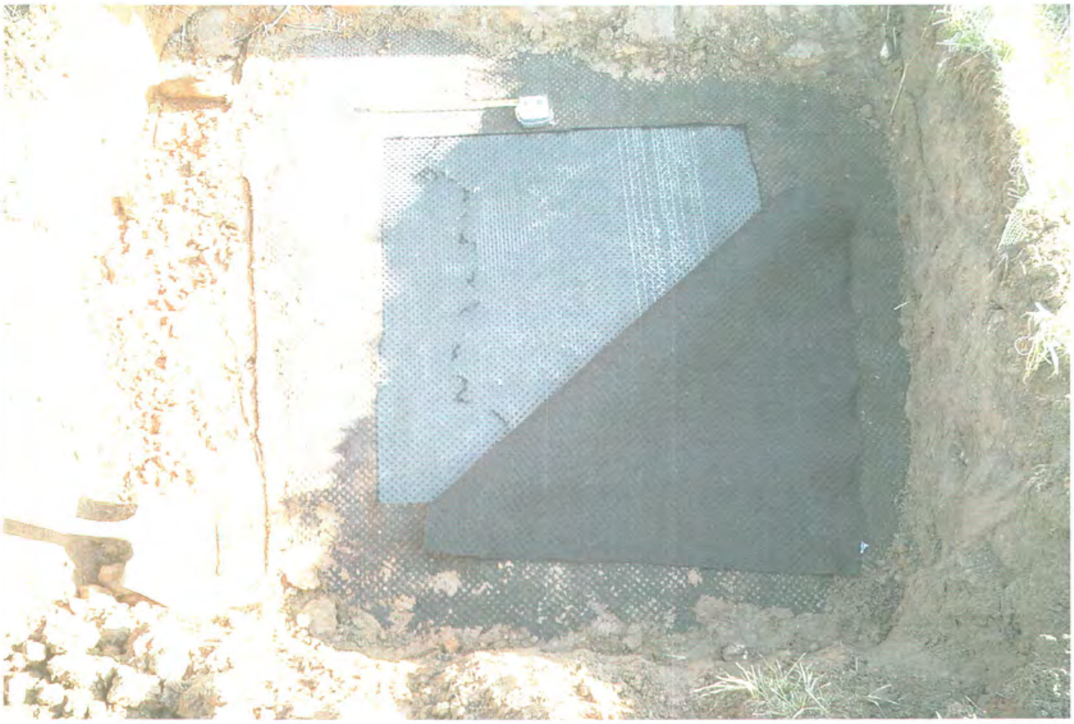
Photograph 20: Close-up view of upslope ruptured geomembrane edge (typical for geomembrane that is exposed).

Photograph 21: Test Pit 1.
Date: 2 February 2005

Photograph 22: Test Pit 1.
Date: 2 February 2005

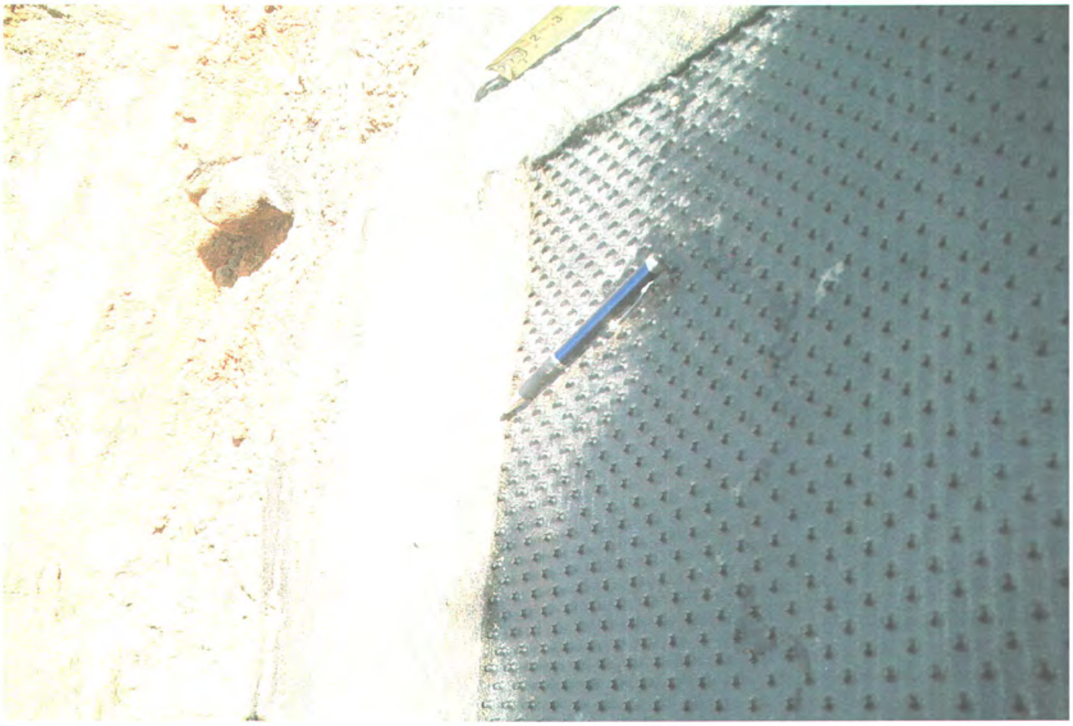
Photograph 23: Test Pit 1.
Date: 2 February 2005

Photograph 24: Test Pit 1 - top of non-woven geotextile filter overlying drainage features of geomembrane.
Date: 2 February 2005
NCP2005-3184\piedmont\photos-part-2.cdr

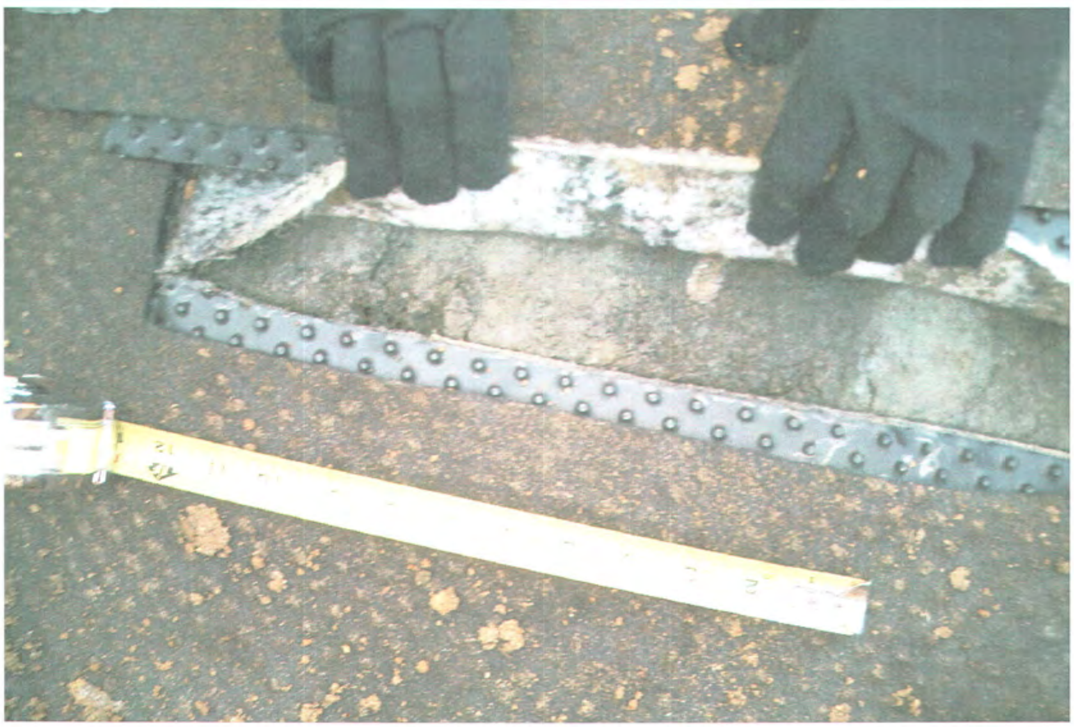


Photograph 25: Test Pit 1 - exposed geomembrane.
Date: 2 February 2005

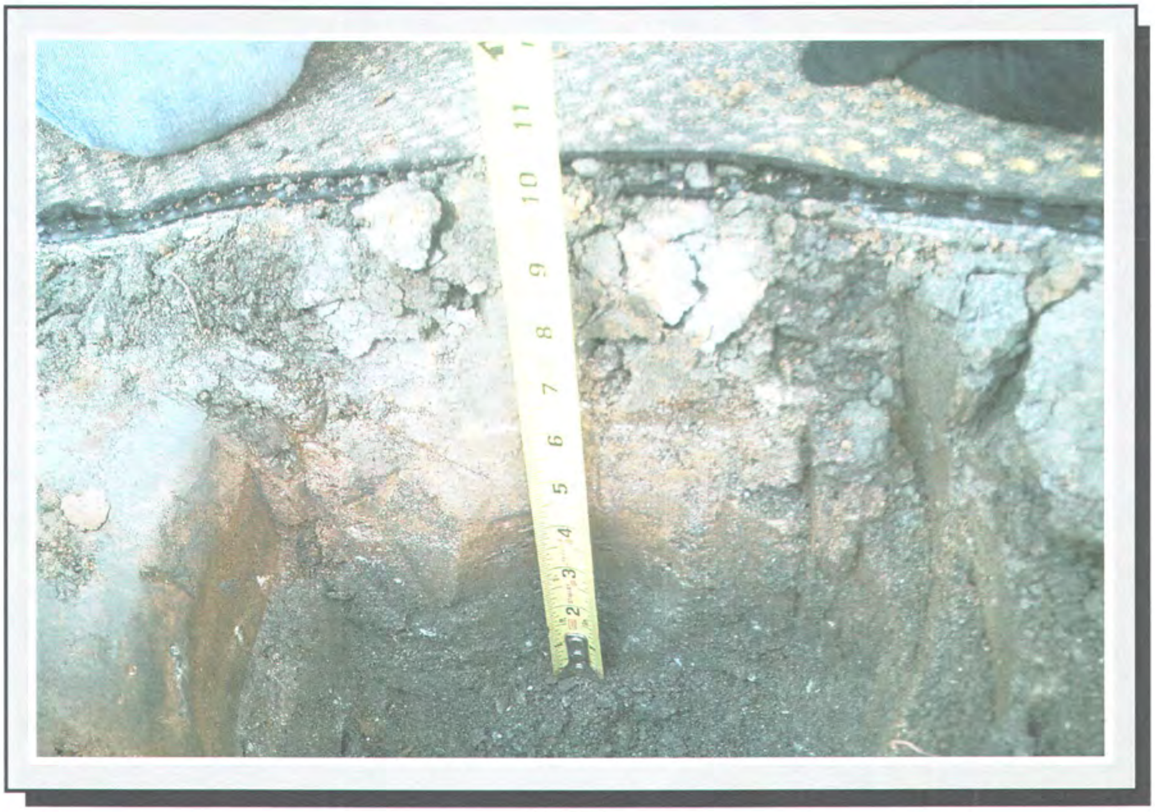
Photograph 26: Test Pit 1 - exposed geomembrane.
Date: 2 February 2005



Photograph 27: Test Pit 1 - geomembrane.
Date: 2 February 2005



Photograph 28: Test Pit 1 - cut through GCL.
Date: 2 February 2005



Photograph 29: Test Pit 1 - cover (foundation) soil layer.
Date: 2 February 2005

Photograph 30: Test Pit 1 - covering with plastic.
Date: 2 February 2005

NCP2005-3184\piedmont\photos-part-2.cdr

Photograph 31: Test Pit 2 - location.
Date: 2 February 2005

Photograph 32: Test Pit 2 - top of non-woven geotextile.
Date: 2 February 2005

Photograph 33: Test Pit 2 - top of non-woven geotextile.
Date: 2 February 2005

Photograph 34: Test Pit 2 - 3-foot thick protective soil cover.
Date: 2 February 2005

Photograph 35: Test Pit 2 - geotextile and geomembrane.
Date: 2 February 2005

Photograph 36: Test Pit 2 - lifting geomembrane up.
Date: 2 February 2005

Photograph 37: Test Pit 2 - top of GCL surface with bentonite film.
Date: 2 February 2005

Photograph 38: Test Pit 2 - cover (foundation) soil layer and bottom of GCL.
Date: 2 February 2005

Photograph 39: Test Pit 2 - cover (foundation) soil layer.
Date: 2 February 2005

Photograph 40: Test Pit 2 - cover (foundation) soil layer.
Date: 2 February 2005

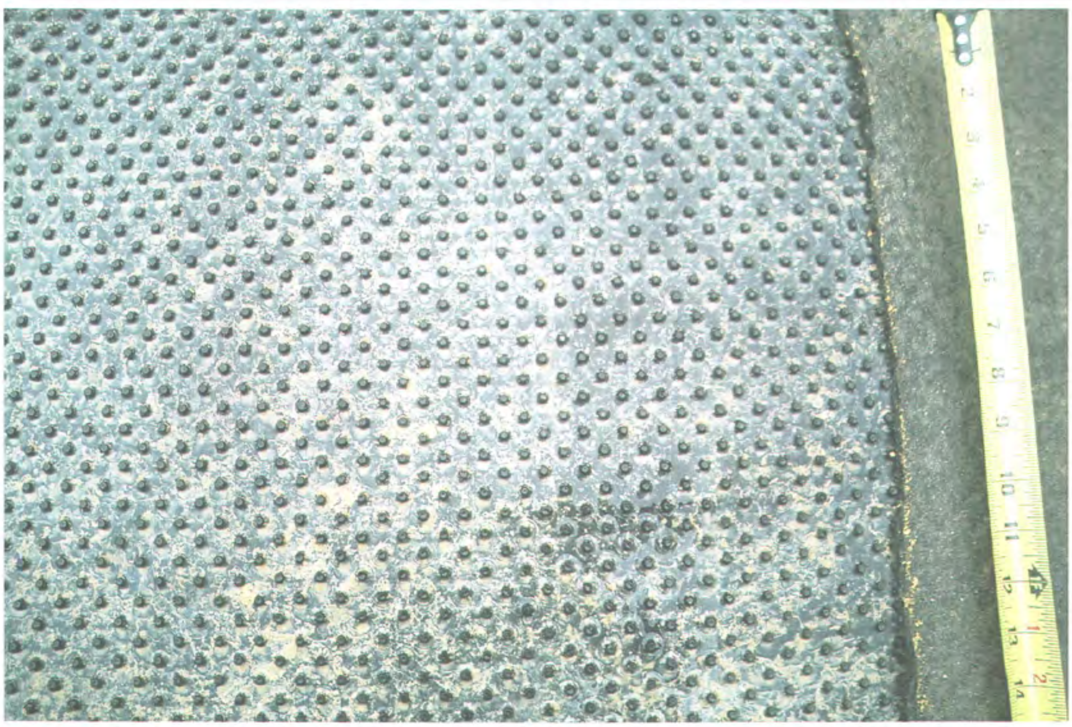
Photograph 41: Test Pit 2 - from the east looking west.
Date: 2 February 2005

Photograph 42: Test Pit 2 - covering with plastic.
Date: 2 February 2005

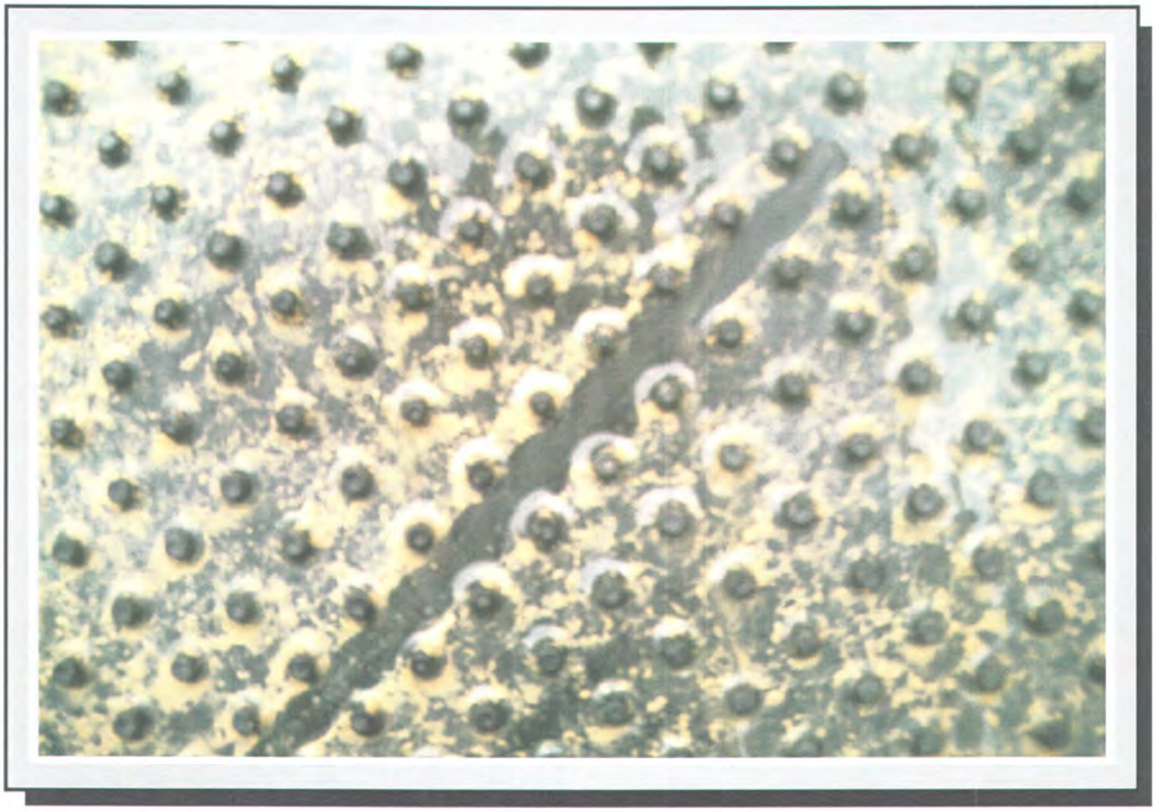
Photograph 43: Test Pit 3 - from the east looking west.
Date: 2 February 2005

Photograph 44: Test Pit 3.
Date: 2 February 2005

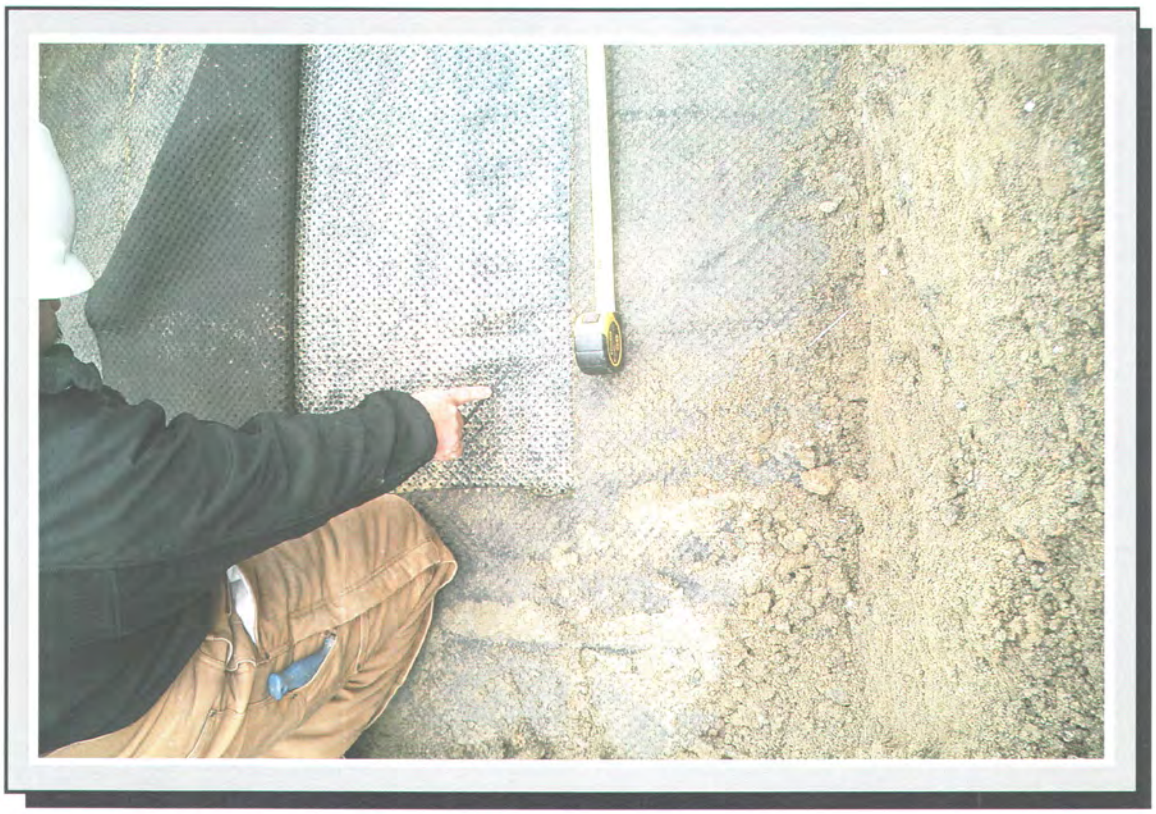
Photograph 45: Test Pit 3 - top of non-woven geotextile filter overlying drainage features of geomembrane.
Date: 2 February 2005



Photograph 46: Test Pit 3 - top of geomembrane.
Date: 2 February 2005



Photograph 47: Test Pit 3 - top of geomembrane - closeup.
Date: 2 February 2005



Photograph 48: Test Pit 3 - top of geomembrane.
Date: 2 February 2005

Photograph 49: Test Pit 3 - Lifting geomembrane up.
Date: 2 February 2005

Photograph 50: Test Pit 3 - bottom of geomembrane - bentonite film.
Date: 2 February 2005

Photograph 51: Eastern slope directly adjacent to scale house, uppermost slope containing green vegetation was installed during Phase 2 Final Closure, bottom most slope containing taller dormant vegetation installed during Southern and Eastern Sideslope Closure.

Photograph 52: Eastern slope directly adjacent to scale house, uppermost slope containing green vegetation was installed during Phase 2 Final Closure, area located upslope of Photograph 9.

NCP2005-3184\piedmont\photos-part-3.cdr

Photograph 53: Southern slope adjacent to Freeman Road, installed during Southern and Eastern Sideslope Closure.

Photograph 54: Drop inlet located on southern slope adjacent to Freeman Road, installed during Southern and Eastern Sideslope Closure.

Photograph 55: Mid-slope swale and drop inlet located on southern slope adjacent to Freeman Road, installed during Phase 2 Final Closure.

Photograph 56: Southern slope adjacent to Freeman Road, uppermost slope containing green vegetation was installed during Phase 2 Final Closure, bottom most slope containing taller dormant vegetation installed during Southern and Eastern Sideslope Closure, background tan vegetation install during Phase 1 Closure.

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Photograph 59: Northwestern slope installed during Phase 1 Closure.

Photograph 60: Northern sideslope western segment, facing due east, area installed during Phase 2 Final Closure.

Photograph 61: Northern sideslope directly down-slope of sideslope haul road, facing due east, area installed during Phase 1 Final Closure, and partially revegetated during Phase 2 Final Closure.

Photograph 62: Northeastern sideslope directly south of access road to borrow area installed during Phase 2 Final Closure.

APPENDIX D

LABORATORY TEST RESULTS

MOISTURE CONTENT MEASUREMENTS

SGI TESTING SERVICES, LLC

SUMMARY OF MOISTURE CONTENTS

WMI / PIEDMONT LANDFILL COVER EVALUATION

Test Number	1	2	3	4	5
Site Sample Name	TP-1 (Foundation Soil A)	TP-1 (Foundation Soil B)	TP-1 (Protective Cover Soil A)	TP-1 (Protective Cover Soil A)	TP-1 GCL
Lab Sample Number	S11191A	S11191B	S11194A	S11194B	S11182
Moisture Tin No.	A2	14	B	EC	Z3
WT. Of Wet Soil + Tin	49.1	56.9	51.7	70.7	19.4
WT. Of Dry Soil + Tin	42.7	48.8	45.0	61.4	11.4
WT. Of Tin	2.0	2.1	2.1	2.1	2.0
WT. Of Dry Soil	40.7	46.7	42.9	59.3	9.4
WT. Of Water	6.4	8.1	6.7	9.3	8.0
Moisture Content (%)	15.72	17.34	15.62	15.68	85.11

Test Number	6	7	8	9	10
Site Sample Name	TP-2 (Foundation Soil A)	TP-2 (Foundation Soil B)	TP-2 (Protective Cover Soil A)	TP-2 (Protective Cover Soil A)	TP-2 GCL
Lab Sample Number	S11192A	S11192B	S11195A	S11195B	S11183
Moisture Tin No.	A3	O	RP	Z3	A3
WT. Of Wet Soil + Tin	65.8	56.6	51.1	66.1	22.9
WT. Of Dry Soil + Tin	59.5	50.9	43.9	58.1	13.3
WT. Of Tin	2.2	2.1	2.1	2.1	2.2
WT. Of Dry Soil	57.3	48.8	41.8	56.0	11.1
WT. Of Water	6.3	5.7	7.2	8.0	9.6
Moisture Content (%)	10.99	11.68	17.22	14.29	86.49

Test Number	11	12	13	14	15	15 (Retest)
Site Sample Name	TP-3 (Foundation Soil A)	TP-3 (Foundation Soil B)	TP-3 (Protective Cover Soil A)	TP-3 (Protective Cover Soil A)	TP-3 GCL	TP-3 GCL
Lab Sample Number	S11193A	S11193B	S11196A	S11196B	S11184	S11184
Moisture Tin No.	G	BUB	BUD	GT	B	RP
WT. Of Wet Soil + Tin	63.5	80.4	70.6	59.5	22.0	26.3
WT. Of Dry Soil + Tin	57.8	72.6	62.4	51.3	13.6	16.1
WT. Of Tin	2.1	2.2	2.1	2.1	2.1	2.1
WT. Of Dry Soil	55.7	70.4	60.3	49.2	11.5	14.0
WT. Of Water	5.7	7.8	8.2	8.2	8.4	10.2
Moisture Content (%)	10.23	11.08	13.60	16.67	73.04	72.86

ASPERITY HEIGHT

SGI TESTING SERVICES, LLC

ASPERITY MEASUREMENT OF TEXTURED GEOMEMBRANES

GRI GM12 (1)

Project Name:	WMI / PIEDMONT LANDFILL COVER EVALUATION
Project Number:	SGI5011
Client Name:	GeoSyntec Consultants, Inc.
Client/Site ID:	TP-2 Geomembrane
Sample Number:	S11186
Material Type:	60-mil Agru MicroDrain LLDPE Geomembrane
Expected/Specified Value:	N/A

Specimen	Drain Core Side			Microspike Side		
Number	(in)	(mil)	(mm)	(in)	(mil)	(mm)
1				0.0150	15.0	0.38
2				0.0130	13.0	0.33
3				0.0140	14.0	0.36
4				0.0140	14.0	0.36
5				0.0150	15.0	0.38
6				0.0140	14.0	0.36
7				0.0135	13.5	0.34
8				0.0125	12.5	0.32
9				0.0140	14.0	0.36
10				0.0150	15.0	0.38
Avg.				0.0140	14.0	0.36
S.D.				0.0008	0.8	0.02

Observations: The Microspike side of the geomembrane had fairly uniform texture coverage.

Notes:

(1) Deviations:

Laboratory temperature at 21±3 °C, laboratory humidity at 50-70 %.

The test sample is conditioned in the above mentioned laboratory temperature and humidity conditions for a minimum of 3 hours prior to testing.

SGI TESTING SERVICES, LLC

ASPERITY MEASUREMENT OF TEXTURED GEOMEMBRANES

GRI GM12 (1)

Project Name:	WMI / PIEDMONT LANDFILL COVER EVALUATION
Project Number:	SGI5011
Client Name:	GeoSyntec Consultants, Inc.
Client/Site ID:	TP-3 Geomembrane
Sample Number:	S11187
Material Type:	60-mil Agru MicroDrain LLDPE Geomembrane
Expected/Specified Value:	N/A

Specimen	Drain Core Side			Microspike Side		
Number	(in)	(mil)	(mm)	(in)	(mil)	(mm)
1				0.0135	13.5	0.34
2				0.0145	14.5	0.37
3				0.0140	14.0	0.36
4				0.0135	13.5	0.34
5				0.0130	13.0	0.33
6				0.0135	13.5	0.34
7				0.0130	13.0	0.33
8				0.0135	13.5	0.34
9				0.0140	14.0	0.36
10				0.0130	13.0	0.33
Avg.				0.0136	13.6	0.34
S.D.				0.0005	0.5	0.01

Observations: The Microspike side of the geomembrane had fairly uniform texture coverage.

Notes:

(1) Deviations:

Laboratory temperature at 21±3 °C, laboratory humidity at 50-70 %.

The test sample is conditioned in the above mentioned laboratory temperature and humidity conditions for a minimum of 3 hours prior to testing.

SGI TESTING SERVICES, LLC

ASPERITY MEASUREMENT OF TEXTURED GEOMEMBRANES

GRI GM12 (1)

Project Name:	WMI / PIEDMONT LANDFILL COVER EVALUATION
Project Number:	SGI5011
Client Name:	GeoSyntec Consultants, Inc.
Client/Site ID:	Uninstalled Site Sample Geomembrane
Sample Number:	S11238
Material Type:	60-mil Agru MicroDrain LLDPE Geomembrane
Expected/Specified Value:	N/A

Specimen	Drain Core Side			Microspike Side		
Number	(in)	(mil)	(mm)	(in)	(mil)	(mm)
1				0.0185	18.5	0.47
2				0.0175	17.5	0.44
3				0.0180	18.0	0.46
4				0.0170	17.0	0.43
5				0.0175	17.5	0.44
6				0.0155	15.5	0.39
7				0.0175	17.5	0.44
8				0.0160	16.0	0.41
9				0.0155	15.5	0.39
10				0.0175	17.5	0.44
Avg.				0.0171	17.1	0.43
S.D.				0.0010	1.0	0.03

Observations: The Microspike side of the geomembrane had fairly uniform texture coverage.

Notes:

(1) Deviations:

Laboratory temperature at 21±3°C, laboratory humidity at 50-70 %.

The test sample is conditioned in the above mentioned laboratory temperature and humidity conditions for a minimum of 3 hours prior to testing.

SGI TESTING SERVICES, LLC

ASPERITY MEASUREMENT OF TEXTURED GEOMEMBRANES

GRI GM12 (1)

Project Name:	WMI / PIEDMONT LANDFILL COVER EVALUATION
Project Number:	SGI5011
Client Name:	GeoSyntec Consultants, Inc.
Client/Site ID:	TP-1 Geomembrane
Sample Number:	S11185
Material Type:	60-mil Agru MicroDrain LLDPE Geomembrane
Expected/Specified Value:	N/A

Specimen	Drain Core Side			Microspike Side		
Number	(in)	(mil)	(mm)	(in)	(mil)	(mm)
1				0.0165	16.5	0.42
2				0.0145	14.5	0.37
3				0.0140	14.0	0.36
4				0.0150	15.0	0.38
5				0.0130	13.0	0.33
6				0.0140	14.0	0.36
7				0.0150	15.0	0.38
8				0.0130	13.0	0.33
9				0.0130	13.0	0.33
10				0.0135	13.5	0.34
Avg.				0.0142	14.2	0.36
S.D.				0.0011	1.1	0.03

Observations: The Microspike side of the geomembrane had fairly uniform texture coverage.

Notes:

(1) Deviations:

Laboratory temperature at 21±3 °C, laboratory humidity at 50-70 %.

The test sample is conditioned in the above mentioned laboratory temperature and humidity conditions for a minimum of 3 hours prior to testing.

s11185.ASP.xls

SGI TESTING SERVICES, LLC

ASPERITY MEASUREMENT OF TEXTURED GEOMEMBRANES

GRI GM12 [1]

Project Name:	WMI / PIEDMONT LANDFILL COVER EVALUATION
Project Number:	SGI5011
Client Name:	GeoSyntec Consultants, Inc.
Client/Site ID:	Uninstalled Site Sample Geomembrane (After being sheared in Tests 5A & 5B)
Sample Number:	S11238 (Sheared)
Material Type:	60-mil Agru MicroDrain LLDPE Geomembrane
Expected/Specified Value:	N/A

Specimen Number	Drain Core Side			Microspike Side		
	(in)	(mil)	(mm)	(in)	(mil)	(mm)
1				0.0180	18.0	0.46
2				0.0170	17.0	0.43
3				0.0180	18.0	0.46
4				0.0175	17.5	0.44
5				0.0165	16.5	0.42
6				0.0185	18.5	0.47
7				0.0150	15.0	0.38
8				0.0155	15.5	0.39
9				0.0165	16.5	0.42
10				0.0160	16.0	0.41
Avg.				0.0169	16.9	0.43
S.D.				0.0012	1.2	0.03

Observations: The Microspike side of the geomembrane had fairly uniform texture coverage.

Notes:

(1) Deviations:

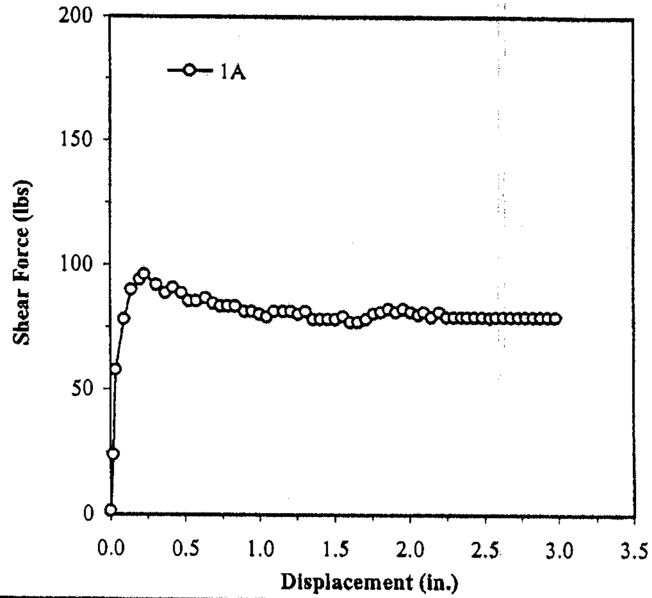
Laboratory temperature at 21±3 °C, laboratory humidity at 50-70 %.

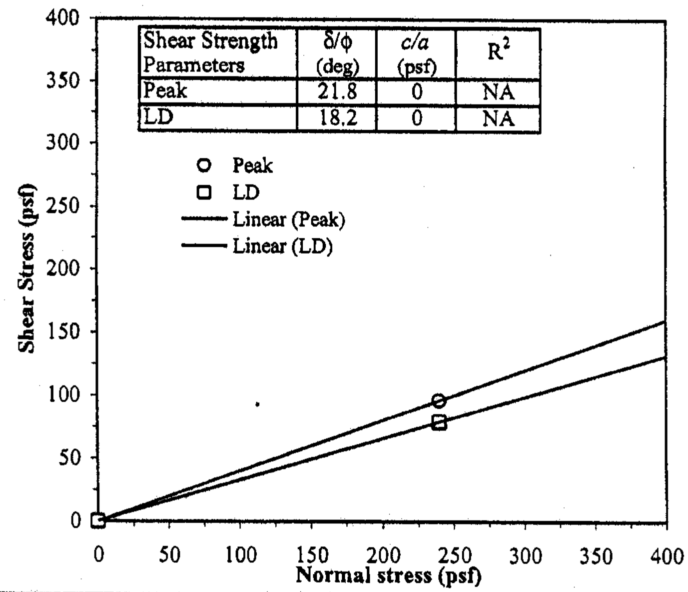
The test sample is conditioned in the above mentioned laboratory temperature and humidity conditions for a minimum of 3 hours prior to testing.

INTERFACE SHEAR STRENGTH

GEOSYNTEC CONSULTANTS, INC. - WMI/PIEDMONT LANDFILL COVER EVALUATION
INTERFACE DIRECT SHEAR TESTING (ASTM D 5321)

Test Series 1A: protective cover soil (TP-1A) against wetted nonwoven geotextile (TP-1A) against wetted 50-mil (minimum) Agru MicroDrain LLDPE geomembrane (TP-1A) with drainage cylinders up and Microspikes down) against woven geotextile side of prehydrated Bentomat ST GCL (typical sample) against wetted foundation soil (TP-1A) under consolidated conditions





Test No.	Shear Box Size (in. x in.)	Normal Stress (psf)	Shear Rate (in./min)	Soaking Stress (psf)	Soaking Time (hour)	Consolidation Stress (psf)	Consolidation Time (hour)	Lower Soil γ_{di} (pcf)	Lower Soil ω_i (%)	Lower Soil ω_f (%)	Upper Soil γ_{di} (pcf)	Upper Soil ω_i (%)	Upper Soil ω_f (%)	GCL ω_i (%)	GCL ω_f (%)	Shear Stress τ_p (psf)	Shear Stress τ_{LD} (psf)	Failure Mode
1A	12 x 12	240	0.040	-	-	240	0.5	100.5	16.8	17.9	101.7	15.4	16.9	19.8	80.6	96	79	(1)

Notes: (1) Sliding (i.e., shear failure) occurred between the woven geotextile side of the Bentomat ST GCL and the Microspike side of the Agru Microdrain geomembrane during the test.
(2) The reported total-stress parameters of friction angle and adhesion were determined from a best-fit line drawn through the test data. Caution should be exercised in using these strength parameters for applications involving normal stresses outside the range of the stresses covered by the test series. The large-displacement (LD) shear strength was calculated using the shear force measured at the end of the test.

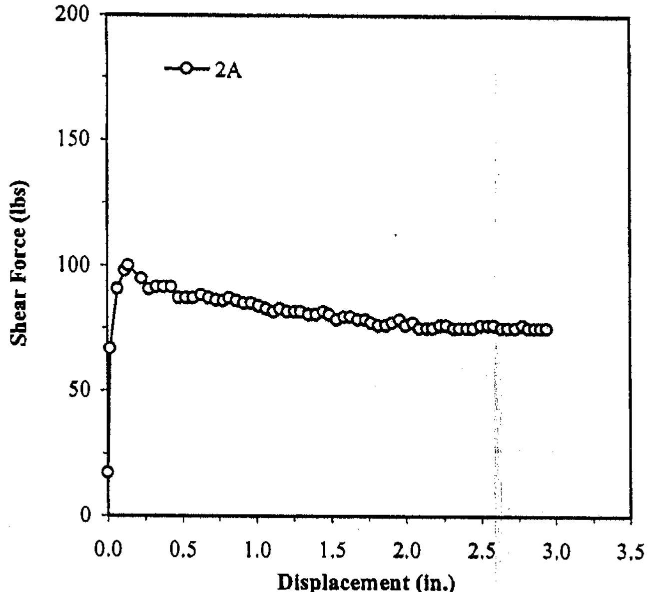
SGI TESTING SERVICES, LLC

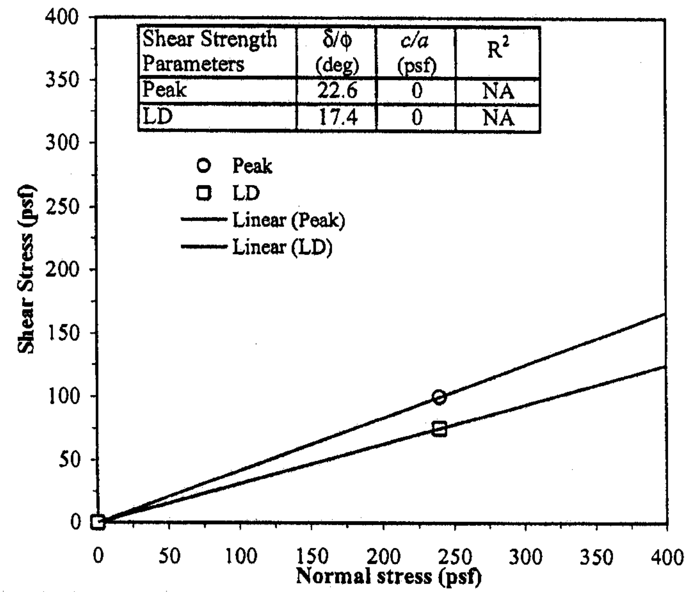
DATE OF TEST:	15 to 16 February 2005
FIGURE NO.	A-6
PROJECT NO.	SGI5011
DOCUMENT NO.	SGI05006
FILE NO.	

GEOSYNTEC CONSULTANTS, INC. - WMI/PIEDMONT LANDFILL COVER EVALUATION

INTERFACE DIRECT SHEAR TESTING (ASTM D 5321)

Test Series 2A: protective cover soil (TP-1A) against wetted nonwoven geotextile (TP-2A) against wetted 50-mil (minimum) Agru MicroDrain LLDPE geomembrane (TP-2A) with drainage cylinders up and Microspikes down) against woven geotextile side of prehydrated Bentomat ST GCL (typical sample) against wetted foundation soil (TP-1A) under consolidated conditions





Test No.	Shear Box Size (in. x in.)	Normal Stress (psf)	Shear Rate (in./min)	Soaking Stress (psf)	Soaking Time (hour)	Consolidation Stress (psf)	Consolidation Time (hour)	Lower Soil γ_{di} (pcf)	Lower Soil ω_i (%)	Lower Soil ω_f (%)	Upper Soil γ_{di} (pcf)	Upper Soil ω_i (%)	Upper Soil ω_f (%)	GCL ω_i (%)	GCL ω_f (%)	Shear Stress τ_p (psf)	Shear Stress τ_{LD} (psf)	Failure Mode
2A	12 x 12	240	0.040	-	-	240	0.5	100.4	16.9	18.1	101.9	15.2	16.9	19.8	80.8	100	75	(1)

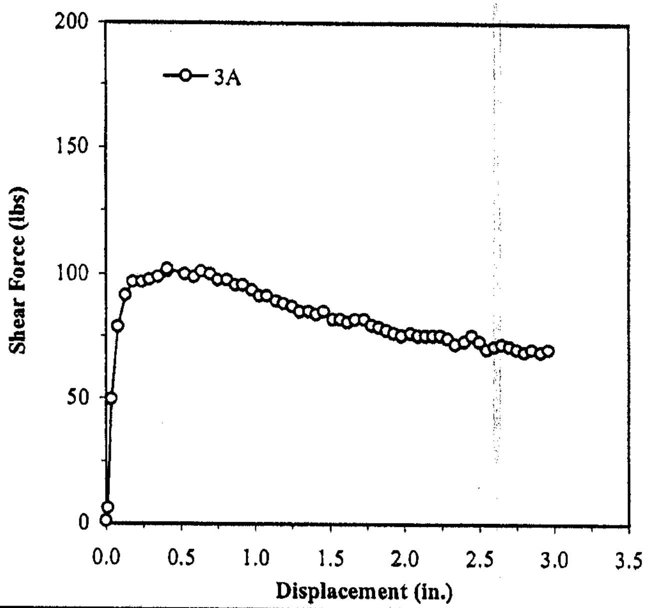
Notes: (1) Sliding (i.e., shear failure) occurred between the woven geotextile side of the Bentomat ST GCL and the Microspike side of the Agru Microdrain geomembrane during the test.
(2) The reported total-stress parameters of friction angle and adhesion were determined from a best-fit line drawn through the test data. Caution should be exercised in using these strength parameters for applications involving normal stresses outside the range of the stresses covered by the test series. The large-displacement (LD) shear strength was calculated using the shear force measured at the end of the test.

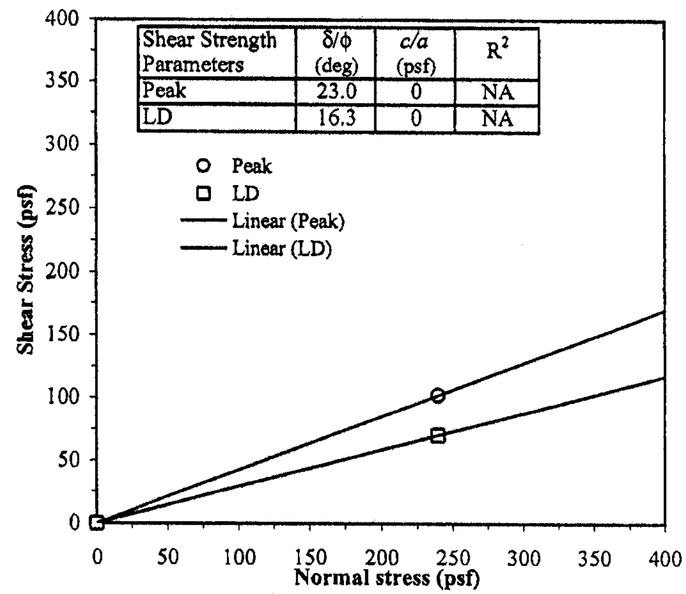
SGI TESTING SERVICES, LLC

DATE OF TEST:	15 to 17 February 2005
FIGURE NO.	A-7
PROJECT NO.	SGI5011
DOCUMENT NO.	SGI05006
FILE NO.	

GEOSYNTEC CONSULTANTS, INC. - WMI/PIEDMONT LANDFILL COVER EVALUATION
INTERFACE DIRECT SHEAR TESTING (ASTM D 5321)

Test Series 3A: protective cover soil (TP-1A) against wetted nonwoven geotextile (TP-3A) against wetted 50-mil (minimum) Agru MicroDrain LLDPE geomembrane (TP-3A) with drainage cylinders up and Microspikes down) against woven geotextile side of prehydrated Bentomat ST GCL (typical sample) against wetted foundation soil (TP-1A) under consolidated conditions





Shear Strength Parameters	δ/φ (deg)	c/a (psf)	R^2
Peak	23.0	0	NA
LD	16.3	0	NA

Test No.	Shear Box Size (in. x in.)	Normal Stress (psf)	Shear Rate (in./min)	Soaking Stress (psf)	Soaking Time (hour)	Consolidation Stress (psf)	Consolidation Time (hour)	Lower Soil γ_{di} (pcf)	Lower Soil ω_i (%)	Lower Soil ω_f (%)	Upper Soil γ_{di} (pcf)	Upper Soil ω_i (%)	Upper Soil ω_f (%)	GCL ω_i (%)	GCL ω_f (%)	Shear Stress τ_p (psf)	Shear Stress τ_{LD} (psf)	Failure Mode
3A	12 x 12	240	0.040	-	-	240	0.5	100.2	17.2	18.2	101.8	15.3	17.0	19.8	81.6	102	70	(1)

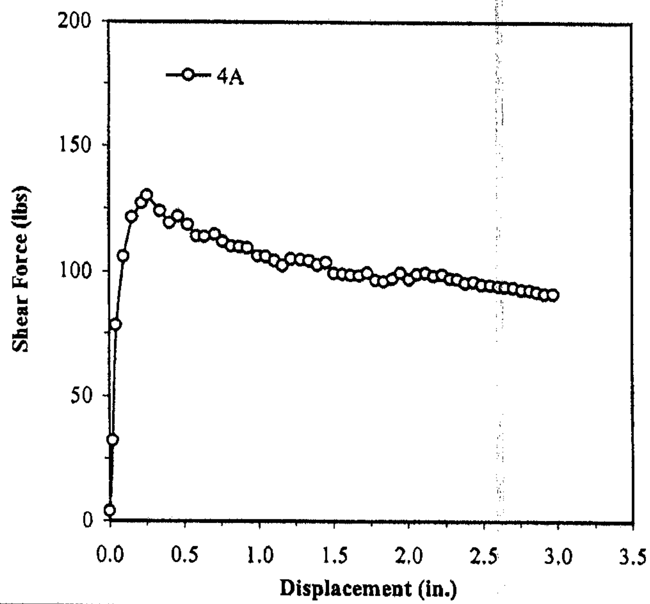
Notes: (1) Sliding (i.e., shear failure) occurred between the woven geotextile side of the Bentomat ST GCL and the Microspike side of the Agru Microdrain geomembrane during the test.
(2) The reported total-stress parameters of friction angle and adhesion were determined from a best-fit line drawn through the test data. Caution should be exercised in using these strength parameters for applications involving normal stresses outside the range of the stresses covered by the test series. The large-displacement (LD) shear strength was calculated using the shear force measured at the end of the test.

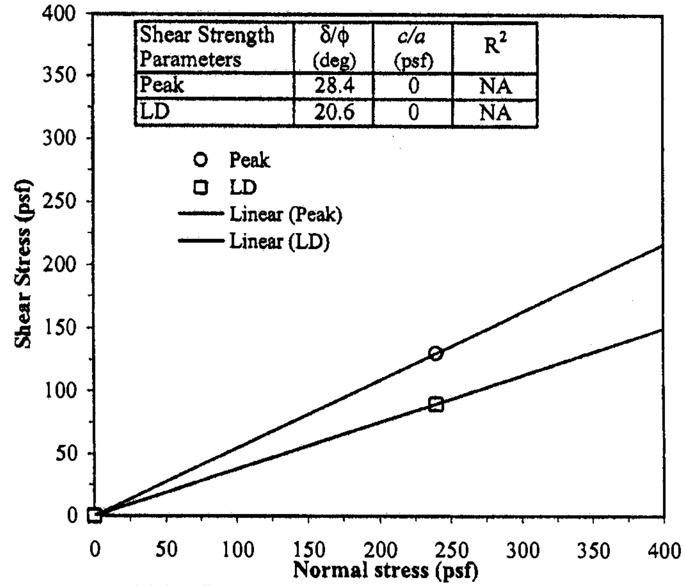
SGI TESTING SERVICES, LLC

DATE OF TEST:	15 to 17 February 2005
FIGURE NO.	A-8
PROJECT NO.	SGI5011
DOCUMENT NO.	SGI05006
FILE NO.	

GEOSYNTEC CONSULTANTS, INC. - WMI/PIEDMONT LANDFILL COVER EVALUATION
INTERFACE DIRECT SHEAR TESTING (ASTM D 5321)

Test Series 4A: protective cover soil (TP-1A) against wetted nonwoven geotextile (SGI supplied) against wetted 50-mil (minimum) Agru MicroDrain LLDPE geomembrane (Uninstalled Site Sample) with drainage cylinders up and Microspikes down) against woven geotextile side of prehydrated Bentomat ST GCL (typical sample) against wetted foundation soil (TP-1A) under consolidated conditions





Test	Shear	Normal	Shear	Soaking		Consolidation		Lower Soil			Upper Soil			GCL		Shear Stress		Failure
No.	Box Size (in. x in.)	Stress (psf)	Rate (in./min)	Stress (psf)	Time (hour)	Stress (psf)	Time (hour)	γ_{di} (pcf)	ω_i (%)	ω_f (%)	γ_{di} (pcf)	ω_i (%)	ω_f (%)	ω_i (%)	ω_f (%)	τ_p (psf)	τ_{LD} (psf)	Mode
4A	12 x 12	240	0.040	-	-	240	0.5	100.3	17.0	17.4	101.5	15.7	16.6	19.8	85.6	130	90	(1)

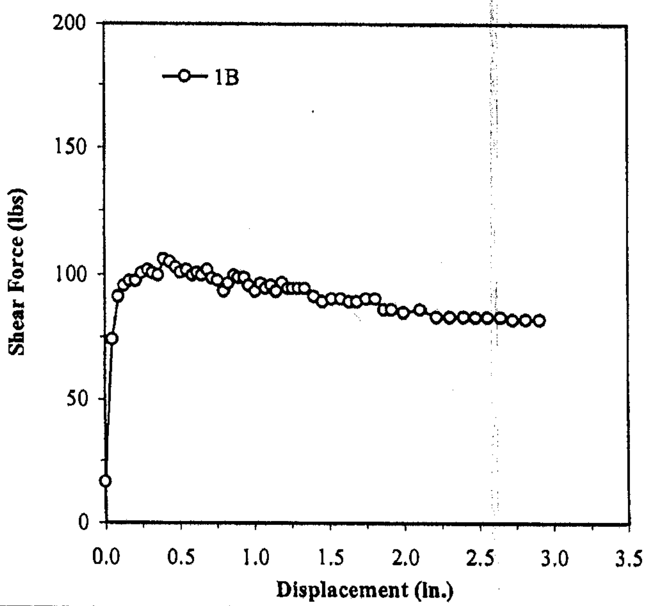
Notes: (1) Sliding (i.e., shear failure) occurred between the woven geotextile side of the Bentomat ST GCL and the Microspike side of the Agru Microdrain geomembrane during the test.
(2) The reported total-stress parameters of friction angle and adhesion were determined from a best-fit line drawn through the test data. Caution should be exercised in using these strength parameters for applications involving normal stresses outside the range of the stresses covered by the test series. The large-displacement (LD) shear strength was calculated using the shear force measured at the end of the test.

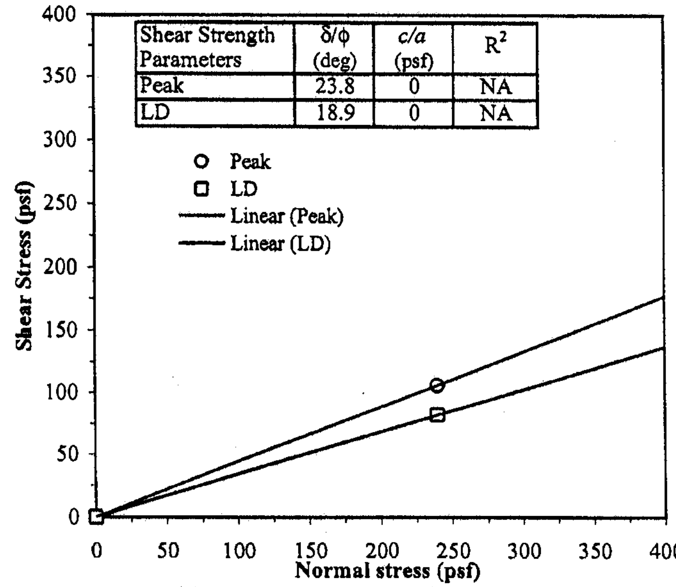
SGI TESTING SERVICES, LLC

DATE OF TEST:	22 to 23 February 2005
FIGURE NO.	A-9
PROJECT NO.	SGI5011
DOCUMENT NO.	SGI05006
FILE NO.	

GEOSYNTEC CONSULTANTS, INC. - WMI/PIEDMONT LANDFILL COVER EVALUATION
INTERFACE DIRECT SHEAR TESTING (ASTM D 5321)

Test Series 1B: protective cover soil (TP-1A) against wetted nonwoven geotextile (TP-1B) against wetted 50-mil (minimum) Agru MicroDrain LLDPE geomembrane (TP-1B) with drainage cylinders up and Microspikes down) against woven geotextile side of prehydrated Bentomat ST GCL (typical sample) against wetted foundation soil (TP-1A) under consolidated and slow shear conditions





Shear Strength Parameters	δ/φ (deg)	c/a (psf)	R^2
Peak	23.8	0	NA
LD	18.9	0	NA

Test No.	Shear Box Size (in. x in.)	Normal Stress (psf)	Shear Rate (in./min)	Soaking Stress (psf)	Soaking Time (hour)	Consolidation Stress (psf)	Consolidation Time (hour)	Lower Soil γ_{di} (pcf)	Lower Soil ω_i (%)	Lower Soil ω_f (%)	Upper Soil γ_{di} (pcf)	Upper Soil ω_i (%)	Upper Soil ω_f (%)	GCL ω_i (%)	GCL ω_f (%)	Shear Stress τ_p (psf)	Shear Stress τ_{LD} (psf)	Failure Mode
1B	12 x 12	240	0.004	-	-	240	24.0	100.4	16.9	18.0	101.6	15.6	17.0	19.8	81.8	106	82	(1)

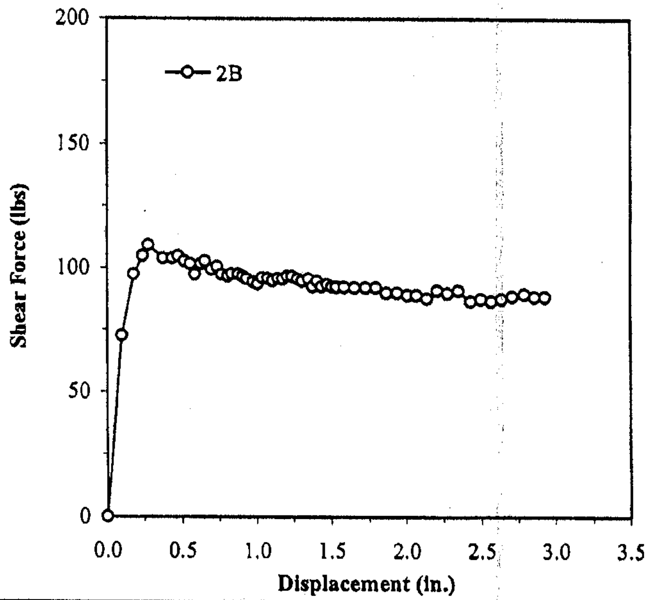
Notes: (1) Sliding (i.e., shear failure) occurred between the woven geotextile side of the Bentomat ST GCL and the Microspike side of the Agru Microdrain geomembrane during the test.
(2) The reported total-stress parameters of friction angle and adhesion were determined from a best-fit line drawn through the test data. Caution should be exercised in using these strength parameters for applications involving normal stresses outside the range of the stresses covered by the test series. The large-displacement (LD) shear strength was calculated using the shear force measured at the end of the test.

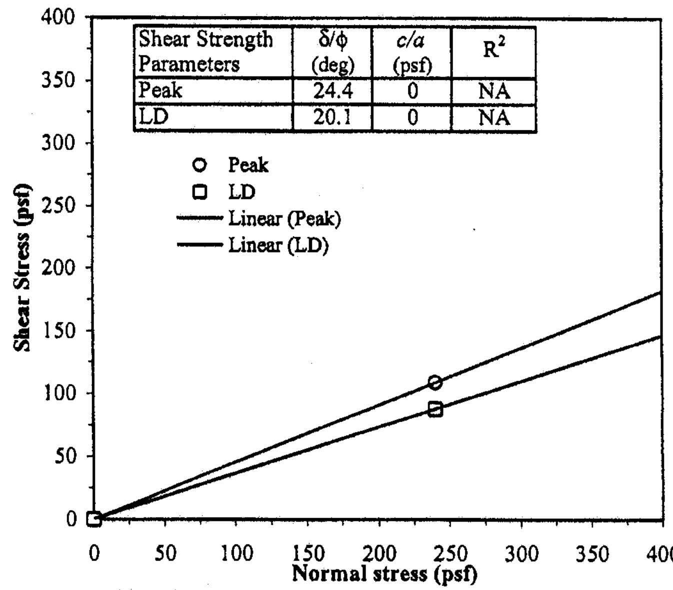
SGI TESTING SERVICES, LLC

DATE OF TEST:	20 to 22 February 2005
FIGURE NO.	A-10
PROJECT NO.	SGI5011
DOCUMENT NO.	SGI05006
FILE NO.	

GEOSYNTEC CONSULTANTS, INC. - WMI/PIEDMONT LANDFILL COVER EVALUATION
INTERFACE DIRECT SHEAR TESTING (ASTM D 5321)

Test Series 2B: protective cover soil (TP-1A) against wetted nonwoven geotextile (TP-2B) against wetted 50-mil (minimum) Agru MicroDrain LLDPE geomembrane (TP-2B) with drainage cylinders up and Microspikes down) against woven geotextile side of prehydrated Bentomat ST GCL (typical sample) against wetted foundation soil (TP-1A) under consolidated and slow shear conditions





Test No.	Shear Box Size (in. x in.)	Normal Stress (psf)	Shear Rate (in./min)	Soaking Stress (psf)	Soaking Time (hour)	Consolidation Stress (psf)	Consolidation Time (hour)	Lower Soil γ_{di} (pcf)	Lower Soil ω_i (%)	Lower Soil ω_f (%)	Upper Soil γ_{di} (pcf)	Upper Soil ω_i (%)	Upper Soil ω_f (%)	GCL ω_i (%)	GCL ω_f (%)	Shear Stress τ_p (psf)	Shear Stress τ_{LD} (psf)	Failure Mode
2B	12 x 12	240	0.004	-	-	240	24.0	100.5	16.8	17.1	101.5	15.7	16.6	19.8	87.4	109	88	(1)

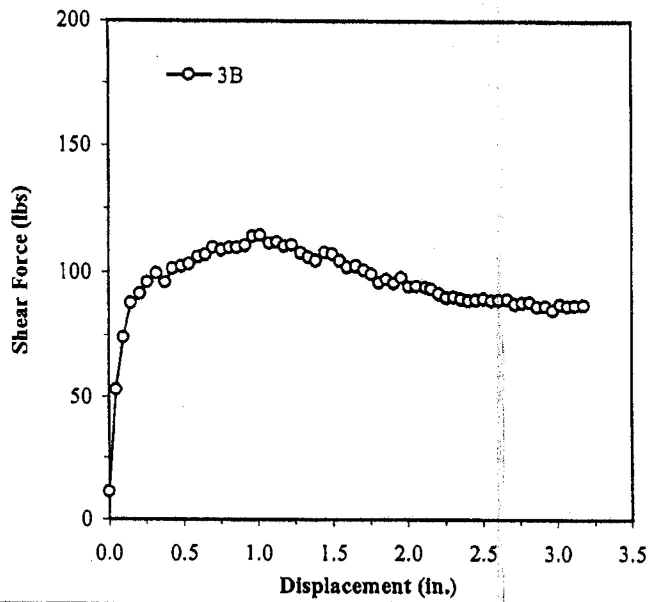
Notes: (1) Sliding (i.e., shear failure) occurred between the woven geotextile side of the Bentomat ST GCL and the Microspike side of the Agru Microdrain geomembrane during the test.
(2) The reported total-stress parameters of friction angle and adhesion were determined from a best-fit line drawn through the test data. Caution should be exercised in using these strength parameters for applications involving normal stresses outside the range of the stresses covered by the test series. The large-displacement (LD) shear strength was calculated using the shear force measured at the end of the test.

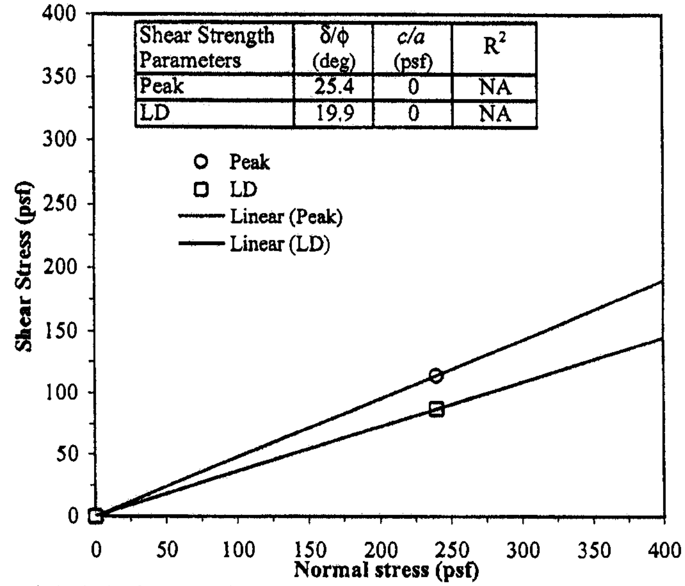
SGI TESTING SERVICES, LLC

DATE OF TEST:	22 to 24 February 2005
FIGURE NO.	A-11
PROJECT NO.	SGI5011
DOCUMENT NO.	SGI05006
FILE NO.	

GEOSYNTEC CONSULTANTS, INC. - WMI/PIEDMONT LANDFILL COVER EVALUATION
INTERFACE DIRECT SHEAR TESTING (ASTM D 5321)

Test Series 3B: protective cover soil (TP-1A) against wetted nonwoven geotextile (TP-3B) against wetted 50-mil (minimum) Agru MicroDrain LLDPE geomembrane (TP-3B) with drainage cylinders up and Microspikes down) against woven geotextile side of prehydrated Bentomat ST GCL (typical sample) against wetted foundation soil (TP-1A) under consolidated and slow shear conditions





Test No.	Shear Box Size (in. x in.)	Normal Stress (psf)	Shear Rate (in./min)	Soaking Stress (psf)	Soaking Time (hour)	Consolidation Stress (psf)	Consolidation Time (hour)	Lower Soil γ_{di} (pcf)	Lower Soil ω_i (%)	Lower Soil ω_f (%)	Upper Soil γ_{di} (pcf)	Upper Soil ω_i (%)	Upper Soil ω_f (%)	GCL ω_i (%)	GCL ω_f (%)	Shear Stress τ_p (psf)	Shear Stress τ_{LD} (psf)	Failure Mode
3B	12 x 12	240	0.004	-	-	240	24.0	100.8	16.5	17.5	101.3	15.9	16.5	19.8	88.3	114	87	(1)

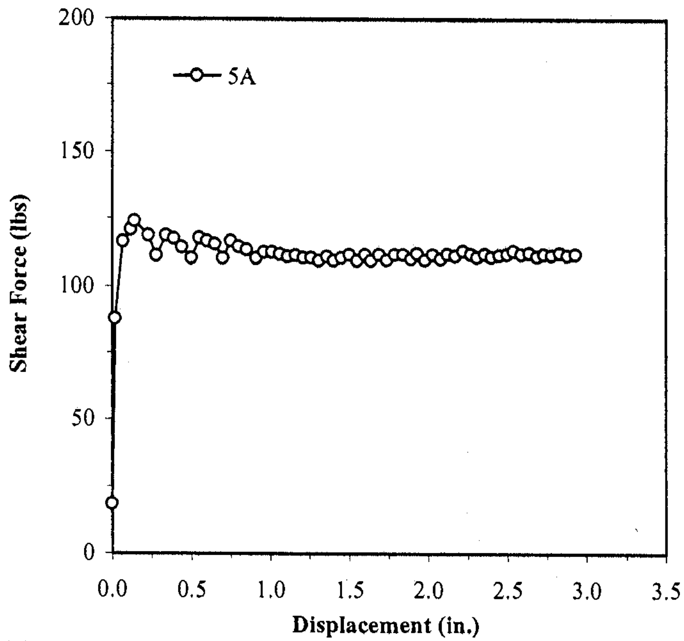
Notes: (1) Sliding (i.e., shear failure) occurred between the woven geotextile side of the Bentomat ST GCL and the Microspike side of the Agru Microdrain geomembrane during the test.
(2) The reported total-stress parameters of friction angle and adhesion were determined from a best-fit line drawn through the test data. Caution should be exercised in using these strength parameters for applications involving normal stresses outside the range of the stresses covered by the test series. The large-displacement (LD) shear strength was calculated using the shear force measured at the end of the test.

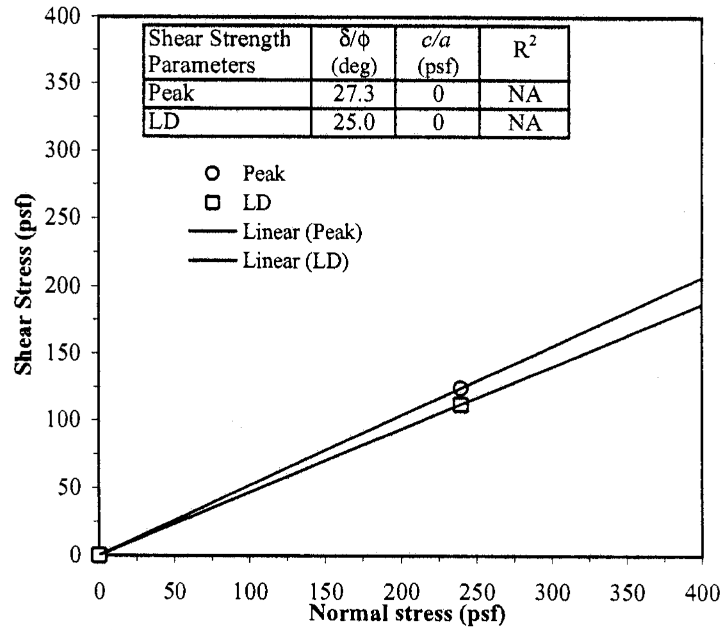
SGI TESTING SERVICES, LLC

DATE OF TEST:	1 to 3 March 2005
FIGURE NO.	A-12
PROJECT NO.	SGI5011
DOCUMENT NO.	SGI05006
FILE NO.	

GEOSYNTEC CONSULTANTS, INC. - WMI/PIEDMONT LANDFILL COVER EVALUATION
INTERFACE DIRECT SHEAR TESTING (ASTM D 5321)

Test Series 5A: woven geotextile side of prehydrated Bentomat ST GCL (typical sample) against wetted 50-mil (minimum) Agru MicroDrain LLDPE geomembrane (Uninstalled Site Sample) with drainage cylinders down and Microspikes up)





Shear Strength Parameters	δ/φ (deg)	c/a (psf)	R^2
Peak	27.3	0	NA
LD	25.0	0	NA

Test No.	Shear Box Size (in. x in.)	Normal Stress (psf)	Shear Rate (in./min)	Soaking Stress (psf)	Soaking Time (hour)	Consolidation Stress (psf)	Consolidation Time (hour)	Lower Soil γ_{di} (pcf)	Lower Soil ω_i (%)	Lower Soil ω_f (%)	Upper Soil γ_{di} (pcf)	Upper Soil ω_i (%)	Upper Soil ω_f (%)	GCL ω_i (%)	GCL ω_f (%)	Shear Stress τ_p (psf)	Shear Stress τ_{LD} (psf)	Failure Mode
5A	12 x 12	240	0.040	-	-	240	0.5	-	-	-	-	-	-	19.8	88.1	124	112	(1)

Notes: (1) Sliding (i.e., shear failure) occurred at the interface between the woven geotextile side of the Bentomat ST GCL and the Microspike side of the Agru Microdrain geomembrane during the test.

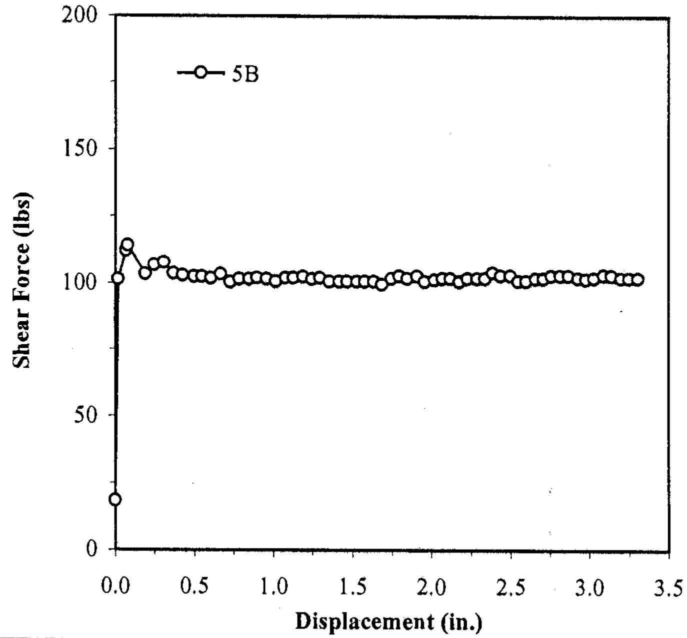
(2) The reported total-stress parameters of friction angle and adhesion were determined from a best-fit line drawn through the test data. Caution should be exercised in using these strength parameters for applications involving normal stresses outside the range of the stresses covered by the test series. The large-displacement (LD) shear strength was calculated using the shear force measured at the end of the test.

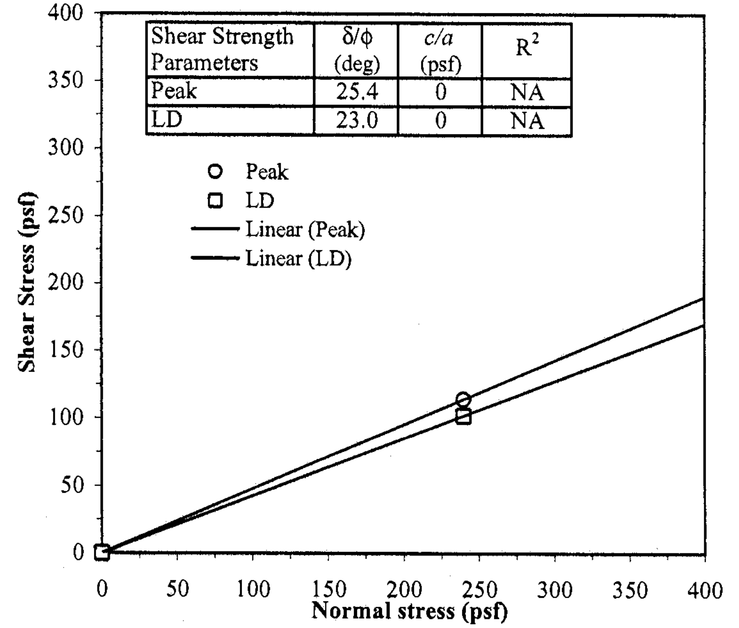
SGI TESTING SERVICES, LLC

DATE OF TEST:	7 to 8 March 2005
FIGURE NO.	A-13
PROJECT NO.	SGI5011
DOCUMENT NO.	SGI05006
FILE NO.	

GEOSYNTEC CONSULTANTS, INC. - WMI/PIEDMONT LANDFILL COVER EVALUATION
INTERFACE DIRECT SHEAR TESTING (ASTM D 5321)

Test Series 5B: unload, resetting, reload, and reshearing of samples from Test Series 5A (woven geotextile side of prehydrated Bentomat ST GCL (typical sample) against wetted 50-mil (minimum) Agru MicroDrain LLDPE geomembrane (Uninstalled Site Sample) with drainage cylinders down and Microspikes up))





Shear Strength Parameters	δ/φ (deg)	c/a (psf)	R^2
Peak	25.4	0	NA
LD	23.0	0	NA

Test	Shear	Normal	Shear	Soaking		Consolidation		Lower Soil			Upper Soil			GCL		Shear Stress		Failure
No.	Box Size (in. x in.)	Stress (psf)	Rate (in./min)	Stress (psf)	Time (hour)	Stress (psf)	Time (hour)	γ_{di} (pcf)	ω_i (%)	ω_f (%)	γ_{di} (pcf)	ω_i (%)	ω_f (%)	ω_i (%)	ω_f (%)	τ_p (psf)	τ_{LD} (psf)	Mode
5B	12 x 12	240	0.040	-	-	240	0.5	-	-	-	-	-	-	19.8	88.1	114	102	(1)

Notes: (1) Sliding (i.e., shear failure) occurred at the interface between the woven geotextile side of the Bentomat ST GCL and the Microspike side of the Agru Microdrain geomembrane during the test.

(2) The reported total-stress parameters of friction angle and adhesion were determined from a best-fit line drawn through the test data. Caution should be exercised in using these strength parameters for applications involving normal stresses outside the range of the stresses covered by the test series. The large-displacement (LD) shear strength was calculated using the shear force measured at the end of the test.

SGI TESTING SERVICES, LLC

DATE OF TEST:	7 to 8 March 2005
FIGURE NO.	A-14
PROJECT NO.	SGI5011
DOCUMENT NO.	SGI05006
FILE NO.	

APPENDIX E

PRECIPITATION DATA

Welcome back.

Home My Page | Health | Travel | Driving | Events | Recreation | Home & Garden World | News | Maps | Weather

Allergies | Skin Protection | Air Quality | Aches & Pains | Cold & Flu | Fitness

Enter city or US zip See weather related to ... GO

Local foreca
GA
(Change Lo

Home > Health > Air Quality > Air Quality Forecast > **Monthly Weather for Kernersville, NC**

Local Severe Weather Alerts

SPECIAL WEATHER STATEMENT UNTIL FRI JAN 21 2005 04:00 PM EST.. [More Details]

Yesterday **36-Hour** **Weekend** **10-Day** **Month**

Air Quality Weather Planner for Kernersville, NC

◂ Previous Month **January** Next Month▸

Sun	Mon	Tue	Wed	Thu	Fri	Sat
						1 OBSERVED Hi 71°F Lo 44°F Precip (in) 0.00in. + / - Avg Hi 24°F Lo 15°F
2 OBSERVED Hi 66°F Lo 45°F Precip (in) 0.00in. + / - Avg Hi 19°F Lo 16°F	3 OBSERVED Hi 69°F Lo 44°F Precip (in) 0.00in. + / - Avg Hi 22°F Lo 15°F	4 OBSERVED Hi 74°F Lo 54°F Precip (in) 0.00in. + / - Avg Hi 27°F Lo 25°F	5 OBSERVED Hi 69°F Lo 55°F Precip (in) 0.00in. + / - Avg Hi 22°F Lo 26°F	6 OBSERVED Hi 68°F Lo 53°F Precip (in) 0.05in. + / - Avg Hi 21°F Lo 25°F	7 OBSERVED Hi 58°F Lo 43°F Precip (in) 0.00in. + / - Avg Hi 11°F Lo 15°F	8 OBSERVED Hi 62°F Lo 45°F Precip (in) 0.01in. + / - Avg Hi 15°F Lo 17°F
9 OBSERVED Hi 53°F Lo 38°F Precip (in) 0.00in. + / - Avg Hi 6°F Lo 10°F	10 OBSERVED Hi 65°F Lo 38°F Precip (in) 0.00in. + / - Avg Hi 18°F Lo 10°F	11 OBSERVED Hi 61°F Lo 39°F Precip (in) 0.00in. + / - Avg Hi 14°F Lo 11°F	12 OBSERVED Hi 67°F Lo 46°F Precip (in) 0.00in. + / - Avg Hi 20°F Lo 18°F	13 OBSERVED Hi 71°F Lo 59°F Precip (in) 0.25in. + / - Avg Hi 24°F Lo 31°F	14 OBSERVED Hi 67°F Lo 35°F Precip (in) 0.88in. + / - Avg Hi 20°F Lo 7°F	15 OBSERVED Hi 41°F Lo 32°F Precip (in) 0.00in. + / - Avg Hi -6°F Lo 4°F

1.2" for the Month

Today in Kernersville

- Flu season update
- Ski resort conditions & forecas
- Allergy & air quality reports
- Airport conditions & delays
- Best places & times to exercis

16	17	18	19	20	Today	22
OBSERVED	OBSERVED	OBSERVED	OBSERVED	OBSERVED		
Hi 44°F Lo 30°F	Hi 32°F Lo 20°F	Hi 28°F Lo 14°F	Hi 32°F Lo 15°F	Hi 40°F Lo 25°F	Hi 37°F Lo 24°F	Hi 34°F Lo 23°F
Precip (in) 0.00in.	Precip (in) 0.00in.	Precip (in) 0.00in.	Precip (in) 0.00in.	~~Precip (in)~~ 0.01in.	Precip 20 %	Precip 60 %
+ / - Avg Hi -3°F Lo 2°F	+ / - Avg Hi -15°F Lo -8°F	+ / - Avg Hi -19°F Lo -14°F	+ / - Avg Hi -15°F Lo -13°F	+ / - Avg Hi -7°F Lo -3°F	Wind NE at 11 mph	Wind SSE at 7 mph

23	24	25	26	27	28	29
Hi 32°F Lo 17°F	Hi 45°F Lo 27°F	Hi 52°F Lo 32°F	Hi 52°F Lo 38°F	Hi 58°F Lo 33°F	Hi 52°F Lo 33°F	Hi 53°F Lo 37°F
Precip 30 %	Precip 10 %	Precip 0 %	Precip 20 %	Precip 20 %	Precip 0 %	Precip 60 %
Wind N at 15 mph	Wind WNW at 7 mph	Wind WNW at 4 mph	Wind SW at 8 mph	Wind NNW at 7 mph	Wind NE at 5 mph	Wind ESE at 3 mph

30	31
Hi 53°F Lo 31°F	AVERAGES Hi 48°F Lo 28°F
Precip 30 %	RECORDS Hi 74°F Lo 4°F
Wind N at 6 mph	

OBSERVED: As reported at Greensboro, NC

FORECAST: Jan 21 12:54 p.m. ET

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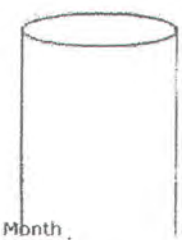
Month-to-Date Totals

Temperatures

78°F Record High
Highest so far 74°F
47°F Avg. High
28°F Avg. Low

Precipitation



Average 3.54 in.

Jay Beech

From: Nelson Breeden
Sent: Tuesday, January 25, 2005 3:45 PM
To: Jay Beech
Subject: piedmont december weather

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Home > Local Forecast > **Monthly Forecast for Kernersville, NC**

Yesterday 36-Hour Weekend 10-Day **Month**

Monthly Weather Planner
for Kernersville, NC

December Next Month

Sun	Mon	Tue	Wed	Thu	Fri	Sat
			1 OBSERVED Hi **61°F** Lo **37°F** Precip (in) **0.22in.**	2 OBSERVED Hi **58°F** Lo **30°F** Precip (in) **0.00in.**	3 OBSERVED Hi **56°F** Lo **33°F** Precip (in) **0.00in.**	4 OBSERVED Hi **55°F** Lo **27°F** Precip (in) **0.00in.**
5 OBSERVED Hi **61°F** Lo **30°F** Precip (in) **0.00in.**	6 OBSERVED Hi **55°F** Lo **46°F** Precip (in) **0.20in.**	7 OBSERVED Hi **68°F** Lo **51°F** Precip (in) **0.03in.**	8 OBSERVED Hi **66°F** Lo **43°F** Precip (in) **0.00in.**	9 OBSERVED Hi **51°F** Lo **43°F** Precip (in) **0.95in.**	10 OBSERVED Hi **59°F** Lo **45°F** Precip (in) **0.90in.**	11 OBSERVED Hi **52°F** Lo **37°F** Precip (in) **0.07in.**
12 OBSERVED Hi **52°F** Lo **33°F** Precip (in) **0.00in.**	13 OBSERVED Hi **52°F** Lo **34°F** Precip (in) **0.00in.**	14 OBSERVED Hi **40°F** Lo **27°F** Precip (in) **0.00in.**	15 OBSERVED Hi **39°F** Lo **22°F** Precip (in) **0.00in.**	16 OBSERVED Hi **49°F** Lo **20°F** Precip (in) **0.00in.**	17 OBSERVED Hi **54°F** Lo **29°F** Precip (in) **0.00in.**	18 OBSERVED Hi **53°F** Lo **27°F** Precip (in) **0.00in.**

19	20	21	22	23	24	25
OBSERVED Hi **43°F** Lo **16°F** Precip (in) **0.00in.**	OBSERVED Hi **27°F** Lo **9°F** Precip (in) **0.00in.**	OBSERVED Hi **53°F** Lo **23°F** Precip (in) **0.00in.**	OBSERVED Hi **63°F** Lo **30°F** Precip (in) **0.00in.**	OBSERVED Hi **64°F** Lo **32°F** Precip (in) **0.46in.**	OBSERVED Hi **35°F** Lo **28°F** Precip (in) **0.00in.**	OBSERVED Hi **33°F** Lo **25°F** Precip (in) **0.00in.**
26 ○	**27**	**28**	**29**	**30**	**31**	
OBSERVED Hi **40°F** Lo **26°F** Precip (in) **0.00in.**	OBSERVED Hi **39°F** Lo **23°F** Precip (in) **0.00in.**	OBSERVED Hi **41°F** Lo **21°F** Precip (in) **0.00in.**	OBSERVED Hi **59°F** Lo **29°F** Precip (in) **0.00in.**	OBSERVED Hi **63°F** Lo **43°F** Precip (in) **0.00in.**	OBSERVED Hi **63°F** Lo **38°F** Precip (in) **0.00in.**	

OBSERVED: As reported at Greensboro, NC

FORECAST: Jan 25 02:54 p.m. ET

APPENDIX F

STABILITY ANALYSES

FINAL COVER VENEER STABILTY ANALYSIS PRE-INSTALLATION CONDITION

GEOSYNTEC CONSULTANTS

COMPUTATION COVER SHEET

Client: WM **Project:** Piedmont Landfill **Project/Proposal #:** NCP2005 **Task #:** 3184

TITLE OF COMPUTATIONS FINAL COVER SYSTEM VENEER STABILITY ANALYSIS PRE-INSTALLATION CONDITION

COMPUTATIONS BY:
Signature: T. Elkady
DATE: 3-17-05
Printed Name and Title: Tamer Y. Elkady, Senior Staff Engineer

ASSUMPTIONS AND PROCEDURES CHECKED BY: (Peer Reviewer)
Signature: J.F. Beech
DATE: 17MAR2005
Printed Name and Title: Jay F. Beech, Principal

COMPUTATIONS CHECKED BY:
Signature: Cao Yiwen
DATE: 3.17.05
Printed Name and Title: Yiwen Cao, Engineer

COMPUTATIONS BACKCHECKED BY: (Originator)
Signature: T. Elkady
DATE: 3-17-05
Printed Name and Title: Tamer Y. Elkady, Senior Staff Engineer

APPROVED BY: (PM or Designate)
Signature: J.F. Beech
DATE: 17MAR2005
Printed Name and Title: Jay F. Beech, Principal

APPROVAL NOTES:

REVISIONS (Number and initial all revisions)

NO.	SHEET	DATE	BY	CHECKED BY	APPROVAL

Written by:	Tamer Elkady	Date:	05 /03 /03 (YY MM DD)	Reviewed by:	YC/J. Beech	Date:	05/ 3 17 (YY MM DD)
Client:	WMI	Project:	Piedmont LF	Project/Proposal No.:	NCP2005	Task No:	3184

FINAL COVER SYSTEM VENEER STABILITY ANALYSIS PRE-INSTALLATION CONDTION

PURPOSE

The purpose of this analysis is to replicate final cover stability analysis performed in support of the final cover equivalency technical demonstration (Technical Demonstration) submitted to North Carolina Department of Environmental and Natural Resources (NCDENR) on 18 February and 24 May 2004. As part of these technical demonstrations, analyses were performed to evaluate the static and seismic factor of safety (FS) against the potential for veneer-type slip surface to develop along or through components of the final cover system of the Piedmont Landfill.

METHOD OF ANALYSES

Static Stability:

Static slope stability of a landfill final cover system can be analyzed assuming infinite slope conditions or finite slope conditions. The infinite slope stability analysis method considers a slope of infinite length whereby the driving and resisting forces occur only along or parallel to an interface (i.e., slip plane). The finite slope stability analysis method considers a slope of finite length and additionally takes into account soil strength above a slip plane, primarily as a toe-buttressing effect. Since the final cover slopes at the Piedmont Landfill are relatively short, the finite slope stability analysis method is appropriate.

The finite slope stability factor of safety equation, as formulated by Giroud, et al. [1995], is:

$$FS = \left[\frac{\gamma_t(t-t_w)+\gamma_b t_w}{\gamma_t(t-t_w)+\gamma_{sat} t_w}\right]\frac{\tan\delta}{\tan\beta} + \frac{a/\sin\beta}{\gamma_t(t-t_w)+\gamma_{sat}t_w}$$

$$+\left[\frac{\gamma_t(t-t^*_w)+\gamma_b t^*_w}{\gamma_t(t-t_w)+\gamma_{sat}t_w}\right]\left[\frac{\tan\phi/(2\sin\beta\cos^2\beta)}{1-\tan\beta\tan\phi}\right]\frac{t}{h}$$

$$+\left[\frac{1}{\gamma_t(t-t_w)+\gamma_{sat}t_w}\right]\left[\frac{1/(\sin\beta\cos\beta)}{1-\tan\beta\tan\phi}\right]\frac{ct}{h} \qquad (1)$$

where: FS = factor of safety;
δ = interface friction angle;
a = apparent interface adhesion;
ϕ = soil internal friction angle;
c = apparent soil cohesion;
γ_t = moist soil unit weight;
γ_b = buoyant soil unit weight;
γ_{sat} = saturated soil unit weight;
t = depth of cover soil above critical interface;
t_w = water depth above critical interface;
t_w* = water depth at slope toe;
β = slope inclination; and
h = vertical height of slope.

It should be noted that while the above equation is specifically for an interface above a geomembrane, or similar layers, it can also be applied to interfaces below the geomembrane by changing the coefficient of the first term, (i.e., the coefficient of $\tan\delta / \tan\beta$) to 1.0. The slope geometry, which is used to derive the above equation, is shown in Figure 1.

INPUT PARAMTERS

Input parameters used for this analysis are similar to that used in analyses performed in support of Technical Demonstration submitted NCDENR on February 18 and May 24, 2004. These Technical Demonstrations are included as Appendix B of this report. For the sake of completeness, a brief description of input parameters is presented hereafter.

Critical conditions for the evaluation of the stability of final cover system for the Piedmont Landfill consider a slope of 33.3 percent (3H:1V) and a length of 115 ft. The 2-ft protective soil component of the final cover system was conservatively assumed to have a unit weight of 120 pcf and shear parameters of $c = 0$ psf and $\phi = 30^\circ$.

Information on the interface shear strength parameters (i.e., friction "δ" and adhesion "a") are obtained from laboratory interface shear strength test performed in support of Technical Demonstration submitted to NCDENR on May 24, 2004. A summary of interface shear strength parameters used in this analysis are presented in the following table:

Condition	Shear Strength Parameters	
	friction, δ (degrees)	Adhesion, a (psi)
Peak	26	15
Large displacement	18	15

The average water depth in the drainage layer above the geomembrane (t_w in Equation 1) on a peak day was estimated to be 0.013 inches (0.0001 ft).

RESULTS AND CONCLUSIONS

Analyses were performed to evaluate the static FS of the final cover veneer stability using peak and large displacement interface shear strength parameters. Equations used to calculate the FS below a geomembrane are coded in a spreadsheet presented herein as Tables 1 and 2. Based on the analysis results, the factors of safety were evaluated to be 1.73 and 1.24 for peak and large displacement shear strength parameters, respectively.

REFERENCES

Giroud, J.P., Bachus, R.C., and Bonaparte, R., "Influence of Water Flow on the Stability of Geosynthetic-Soil Layered Systems on Slopes," *Geosynthetics International*, Vol. 2, No. 6, 1995, pp. 1149-1180.

Written by: Tamer Elkady Date: 05 /03 /03 (YY MM DD) Reviewed by: YC/J. Beech Date: 05/ 3 17 (YY MM DD)

Client: WMI Project: Piedmont LF Project/Proposal No.: NCP2005 Task No: 3184

Table 1 Spreadsheet for the evaluation of static factor of safety for the Piedmont Landfill final cover system using peak interface shear strength

FS Below GEOMEMBRANE		
Input Parameters:		
γ_t (Unit wt of soil):	120	pcf
γ_{sat} (Saturated unit wt of soil):	120	pcf
γ_w (Unit wt of water):	62.4	pcf
γ_b (Buoyant unit wt of soil):	57.6	pcf
t_w (water thickness):	0.0001	ft
t* (water thickness at slope toe):	0.0001	ft
δ (weakest interface friction angle):	26	deg
ϕ (friction angle of soil):	30	deg
a (interface adhesion)	15	psf
c (cohesion of soil above geomembrane)	0	psf
T (Tension in Geosynthetics)	0	psf
h (height of slope):	38.33	ft
t (thickness of soil layer)	2.0	ft
β (slope angle)	18.4	deg
FS	1.73	

Written by:	Tamer Elkady	Date:	05 /03 /03 (YY MM DD)	Reviewed by:	YC/J. Beech	Date:	05/ 3 17 (YY MM DD)
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Table 2 Spreadsheet for the evaluation of static factor of safety for the Piedmont Landfill final cover system using residual interface shear strength

FS Below GEOMEMBRANE		
Input Parameters:		
γ_t (Unit wt of soil):	120	pcf
γ_{sat} (Saturated unit wt of soil):	120	pcf
γ_w (Unit wt of water):	62.4	pcf
γ_b (Buoyant unit wt of soil):	57.6	pcf
t_w (water thickness):	0.0001	ft
t* (water thickness at slope toe):	0.0001	ft
δ (weakest interface friction angle):	18.0	deg
ϕ (friction angle of soil):	30	deg
a (interface adhesion)	15	psf
c (cohesion of soil above geomembrane)	0	psf
T (Tension in Geosynthetics)	0	psf
h (height of slope):	38.33	ft
t (thickness of soil layer)	2.0	ft
β (slope angle)	18.4	deg
FS	**1.24**	

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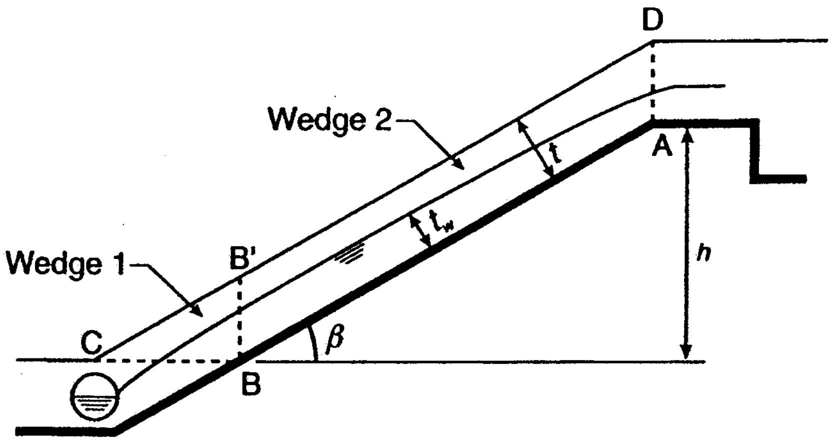

Figure 1 Slope Geometry Used to Derive Finite Slope Stability Equation

FINAL COVER VENEER STABILTY
ANALYSIS
POST-INSTALLATION CONDITION

GEOSYNTEC CONSULTANTS

COMPUTATION COVER SHEET

Client: WM **Project:** Piedmont Landfill **Project/Proposal #:** NCP2005 **Task #:** 3184

TITLE OF COMPUTATIONS FINAL COVER STYSTEM VEENER STABILITY ANALYSIS – POST-INSTALLATION CONDITIONS

COMPUTATIONS BY:

Signature T. Elkady

DATE 3/17/05

Printed Name and Title: Tamer Y. Elkady, Senior Staff Engineer

ASSUMPTIONS AND PROCEDURES CHECKED BY: (Peer Reviewer)

Signature J.F. Beech

DATE 17 MAR 2005

Printed Name and Title: Jay F. Beech, Principal

COMPUTATIONS CHECKED BY:

Signature Cao Yiwen

DATE 3.17.05

Printed Name and Title: Yiwen Cao, Engineer

COMPUTATIONS BACKCHECKED BY: (Originator)

Signature T. Elkady

DATE 3/17/05

Printed Name and Title: Tamer Y. Elkady, Senior Staff Engineer

APPROVED BY: (PM or Designate)

Signature J.F. Beech

DATE 17 MAR 05

Printed Name and Title: Jay F. Beech, Principal

APPROVAL NOTES:

REVISIONS (Number and initial all revisions)

NO.	SHEET	DATE	BY	CHECKED BY	APPROVAL

Written by:	Tamer Elkady	Date:	05 /03 /03 (YY MM DD)	Reviewed by:	YC/J. Beech	Date:	05/ 3 17 (YY MM DD)
Client:	WMI	Project:	Piedmont LF	Project/Proposal No.:	NCP2005	Task No:	3184

FINAL COVER SYSTEM VENEER STABILITY ANALYSIS POST-INSTALLATION CONDITION

PURPOSE

The purpose of the analyses presented in this calculation package is to evaluate the effects of installation on the static and seismic factor of safety (FS) against the potential for veneer-type slip surface to develop along and through the final cover system of the Piedmont landfill.

METHOD OF ANALYSES

Static Stability:

Static slope stability of a landfill final cover system can be analyzed assuming infinite slope conditions or finite slope conditions. The infinite slope stability analysis method considers a slope of infinite length whereby the driving and resisting forces occur only along or parallel to an interface (i.e., slip plane). The finite slope stability analysis method considers a slope of finite length and additionally takes into account soil strength above a slip plane, primarily as a toe-buttressing effect. Since the final cover slopes at the Piedmont Landfill are relatively short, the finite slope stability analysis method is appropriate.

The finite slope stability factor of safety equation, as formulated by Giroud, et al. [1995], is:

$$FS = \left[\frac{\gamma_t(t-t_w)+\gamma_b t_w}{\gamma_t(t-t_w)+\gamma_{sat} t_w}\right]\frac{\tan\delta}{\tan\beta} + \frac{a/\sin\beta}{\gamma_t(t-t_w)+\gamma_{sat} t_w}$$

$$+\left[\frac{\gamma_t(t-t^*_w)+\gamma_b t^*_w}{\gamma_t(t-t_w)+\gamma_{sat} t_w}\right]\left[\frac{\tan\phi/(2\sin\beta\cos^2\beta)}{1-\tan\beta\tan\phi}\right]\frac{t}{h}$$

$$+\left[\frac{1}{\gamma_t(t-t_w)+\gamma_{sat} t_w}\right]\left[\frac{1/(\sin\beta\cos\beta)}{1-\tan\beta\tan\phi}\right]\frac{ct}{h} \quad (1)$$

where: FS = factor of safety;

δ = interface friction angle;

a = apparent interface adhesion;
ϕ = soil internal friction angle;
c = apparent soil cohesion;
γ_t = moist soil unit weight;
γ_b = buoyant soil unit weight;
γ_{sat} = saturated soil unit weight;
t = depth of cover soil above critical interface;
t_w = water depth above critical interface;
t_w^* = water depth at slope toe;
β = slope inclination; and
h = vertical height of slope.

It should be noted that while the above equation is specifically for an interface above a geomembrane, or similar layers, it can also be applied to interfaces below the geomembrane by changing the coefficient of the first term, (i.e., the coefficient of $\tan\delta / \tan\beta$) to 1.0. The slope geometry, which is used to derive the above equation, is shown in Figure 1.

FINAL COVER DATA

The final cover system consists, from top to bottom, of the following:

- a 24-inch thick protective soil layer;
- a 8 oz/yd^2 geotextile filter;
- an Agru America, Inc. Drain® Liner (hereafter referred to as Drain® Liner) consisting of a 50-mil thick liner low density polyethylene (LLDPE) geomembrane with 0.18-inch thick drainage studs; and
- a geosynthetic clay liner (GCL).

Critical conditions for the evaluation of the stability of final cover system for the Piedmont Landfill consider (i) a slope of 33.3 percent (3H:1V) and a length of 340 ft (Case I); and (ii) a slope of 30.8 percent (3.25H:1V) and a length of 340 ft (Case II). The location of the maximum slope length (i.e., 340 ft) is shown on Figure 2.

The protective soil component of the final cover system was assumed to have a unit weight of 120 pcf and shear parameters of c = 72 psf and $\phi = 30^{\circ}$. The cohesion of 72 psf was assumed for the vegetative soil layer due to the root reinforcement effect, typically causing the apparent cohesion to increase in the range of 72 to 360 psf as reported by Abramson et al [1996].

The veneer stability analyses require information about post-installation interface shear strength between the components of the final cover system. To this end, GeoSyntec Consultants (GeoSyntec) performed a laboratory test using geomembrane samples obtained from test pits to evaluate the interface shear strength parameters (i.e., friction "δ" and adhesion "a"). Configuration for the test includes a Drain® Liner sandwiched between a GCL and protective soil layer obtained from site-specific borrow sources. From laboratory test results, the secant interface shear strength parameters for peak and large-displacement conditions were evaluated to be as follows:

Condition	Shear Strength Parameters(1)	
	Friction, δ (degrees)	Adhesion, a (psi)
Peak	24.5	0
Large-displacement	19.6	0

Note: (1) Average of values for samples 1B, 2B, and 3B in Appendix D

In addition, the veneer stability analyses require information on the depth of the water within the final cover system. To evaluate the depth of water (t_w) above the geomembrane component of the final cover, analysis was performed using HELP model [Schroeder, 1994]. Inputs to the model include climatic data, geometrical data and material properties of the components of the final cover system.

Climatic data required by HELP was modeled for Greensboro, North Carolina. Geometrical data includes the thicknesses of the components of the final cover system and the slope and length of the drainage layer. The analysis was performed considering a slope of 30.8 percent (3.25 horizontal to 1 vertical), and a maximum drainage length of 340 ft. The location of the maximum drainage path is shown on Figure 4.

The default material properties from the built-in database in the HELP model were used for components of the final cover system, with the exception of the hydraulic conductivity of the drainage layer. The hydraulic conductivity of the drainage layer is conservatively assumed to be 5 cm/sec based on laboratory hydraulic conductivity test results and an overall reduction factor of 2.0.

Results of the HELP model estimated the average water depth (t_w) on a peak day to be 0.063 inches (0.00525 ft). The output of the HELP analysis is presented in Attachment 1.

RESULTS AND CONCLUSIONS

Analyses were performed to evaluate the static FS of the final cover veneer stability using peak and large-displacement interface shear strength parameters. Equations used to calculate the FS below a geomembrane are coded in a spreadsheet presented herein as Tables 1 thru 4. For Case I, the factors of safety were evaluated to be 1.41 and 1.11 for peak and large-displacement shear strength parameters, respectively (Tables 1 and 2). For Case II, the factors of safety were evaluated to be 1.53 and 1.20, for peak and large-displacement interface shear strength parameters, respectively (Tables 3 and 4).

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Client:	WMI	Project:	Piedmont LF	Project/Proposal No.:	NCP2005	Task No:	3184

REFERENCES

Abramson L.W., Lee, T.S., Sharma, S. and Boyce, G.M. *Slope Stability and Stabilization Methods*. John Wiley & Sons, New York, 1996

Giroud, J.P., Bachus, R.C., and Bonaparte, R., "Influence of Water Flow on the Stability of Geosynthetic-Soil Layered Systems on Slopes," *Geosynthetics International*, Vol. 2, No. 6, 1995, pp. 1149-1180.

Schroeder, P. R., N. M., Lloyd, C. M. and Zappi, P.A. (1994) "The Hydrologic Evaluation of Landfill Performance (HELP) Model: User's Guide for Version 3," EPA/600/R-94/168a, September 1994, U. S. Environmental Protection Agency Office of Research and Development, Washington, D.C.

Written by: Tamer Elkady Date: 05 /03 /03 (YY MM DD) Reviewed by: YC/J. Beech Date: 05/ 3 17 (YY MM DD)

Client: WMI Project: Piedmont LF Project/Proposal No.: NCP2005 Task No: 3184

Table 1 Spreadsheet for the evaluation of static factor of safety for the Piedmont Landfill final cover system using peak interface shear strength

Case I: 3H:1V Final Cover Slope

FS Below GEOMEMBRANE		
Input Parameters:		
γ_t (Unit wt of soil):	120	pcf
γ_{sat} (Saturated unit wt of soil):	120	pcf
γ_w (Unit wt of water):	62.4	pcf
γ_b (Buoyant unit wt of soil):	57.6	pcf
t_w (water thickness):	0.005	ft
t* (water thickness at slope toe):	0.005	ft
δ (weakest interface friction angle):	24.5	deg
ϕ (friction angle of soil):	30	deg
a (interface adhesion)	0	psf
c (cohesion of soil above geomembrane)	72	psf
T (Tension in Geosynthetics)	0	psf
h (height of slope):	113.33	ft
t (thickness of soil layer)	2.0	ft
β (slope angle)	18.4	deg
FS	**1.41**	

Written by:	Tamer Elkady	Date:	05 (YY) /03 (MM) /03 (DD)	Reviewed by:	YC/J. Beech	Date:	05/ (YY) 3 (MM) 17 (DD)
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Table 2 Spreadsheet for the evaluation of static factor of safety for the Piedmont Landfill final cover system using large-displacement interface shear strength

Case I: 3H:1V Final Cover Slope

FS Below GEOMEMBRANE		
Input Parameters:		
γ_t (Unit wt of soil):	120	pcf
γ_{sat} (Saturated unit wt of soil):	120	pcf
γ_w (Unit wt of water):	62.4	pcf
γ_b (Buoyant unit wt of soil):	57.6	pcf
t_w (water thickness):	0.005	ft
t* (water thickness at slope toe):	0.005	ft
δ (weakest interface friction angle):	19.6	deg
ϕ (friction angle of soil):	30	deg
a (interface adhesion)	0	psf
c (cohesion of soil above geomembrane)	72	psf
T (Tension in Geosynthetics)	0	psf
h (height of slope):	113.3	ft
t (thickness of soil layer)	2.0	ft
β (slope angle)	18.4	deg
FS	**1.11**	

Written by:	Tamer Elkady	Date:	05 /03 /03 (YY MM DD)	Reviewed by:	YC/J. Beech	Date:	05/ 3 17 (YY MM DD)
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Table 3 Spreadsheet for the evaluation of static factor of safety for the Piedmont Landfill final cover system using peak interface shear strength

Case II: 3.25H:1V Final Cover Slope

FS Below GEOMEMBRANE		
Input Parameters:		
γ_t (Unit wt of soil):	120	pcf
γ_{sat} (Saturated unit wt of soil):	120	pcf
γ_w (Unit wt of water):	62.4	pcf
γ_b (Buoyant unit wt of soil):	57.6	pcf
t_w (water thickness):	0.005	ft
t* (water thickness at slope toe):	0.005	ft
δ (weakest interface friction angle):	24.5	deg
ϕ (friction angle of soil):	30	deg
a (interface adhesion)	0	psf
c (cohesion of soil above geomembrane)	72	psf
T (Tension in Geosynthetics)	0	psf
h (height of slope):	113.33	ft
t (thickness of soil layer)	2.0	ft
β (slope angle)	17.1	deg
FS	**1.53**	

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Table 4 Spreadsheet for the evaluation of static factor of safety for the Piedmont Landfill final cover system using large-displacement interface shear strength

Case II: 3.25H:1V Final Cover Slope

FS Below GEOMEMBRANE		
Input Parameters:		
γ_t (Unit wt of soil):	120	pcf
γ_{sat} (Saturated unit wt of soil):	120	pcf
γ_w (Unit wt of water):	62.4	pcf
γ_b (Buoyant unit wt of soil):	57.6	pcf
t_w (water thickness):	0.005	ft
t* (water thickness at slope toe):	0.005	ft
δ (weakest interface friction angle):	19.6	deg
ϕ (friction angle of soil):	30	deg
a (interface adhesion)	0	psf
c (cohesion of soil above geomembrane)	72	psf
T (Tension in Geosynthetics)	0	psf
h (height of slope):	113.3	ft
t (thickness of soil layer)	2.0	ft
β (slope angle)	17.1	deg
FS	**1.20**	

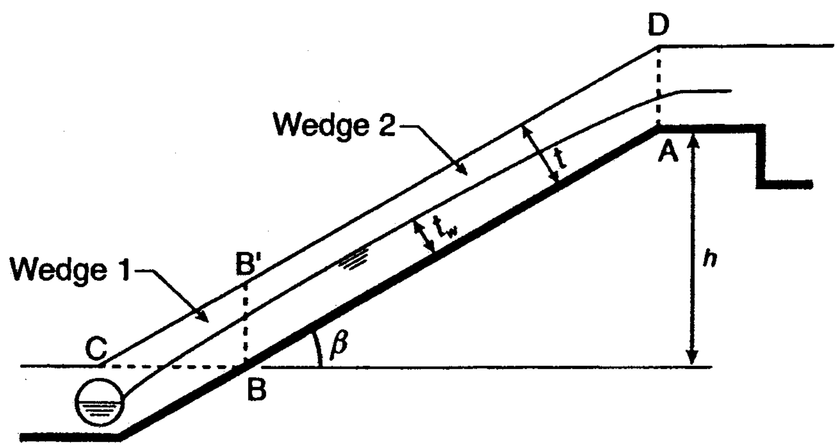

Figure 1 Slope Geometry Used to Derive Finite Slope Stability Equation

Written by: Tamer Elkady Date: 05/03/03 (YY/MM/DD) Reviewed by: YC/J. Beech Date: 05/3/17 (YY/MM/DD)

Client: WMI Project: Piedmont LF Project/Proposal No.: NCP2005 Task No: 3184

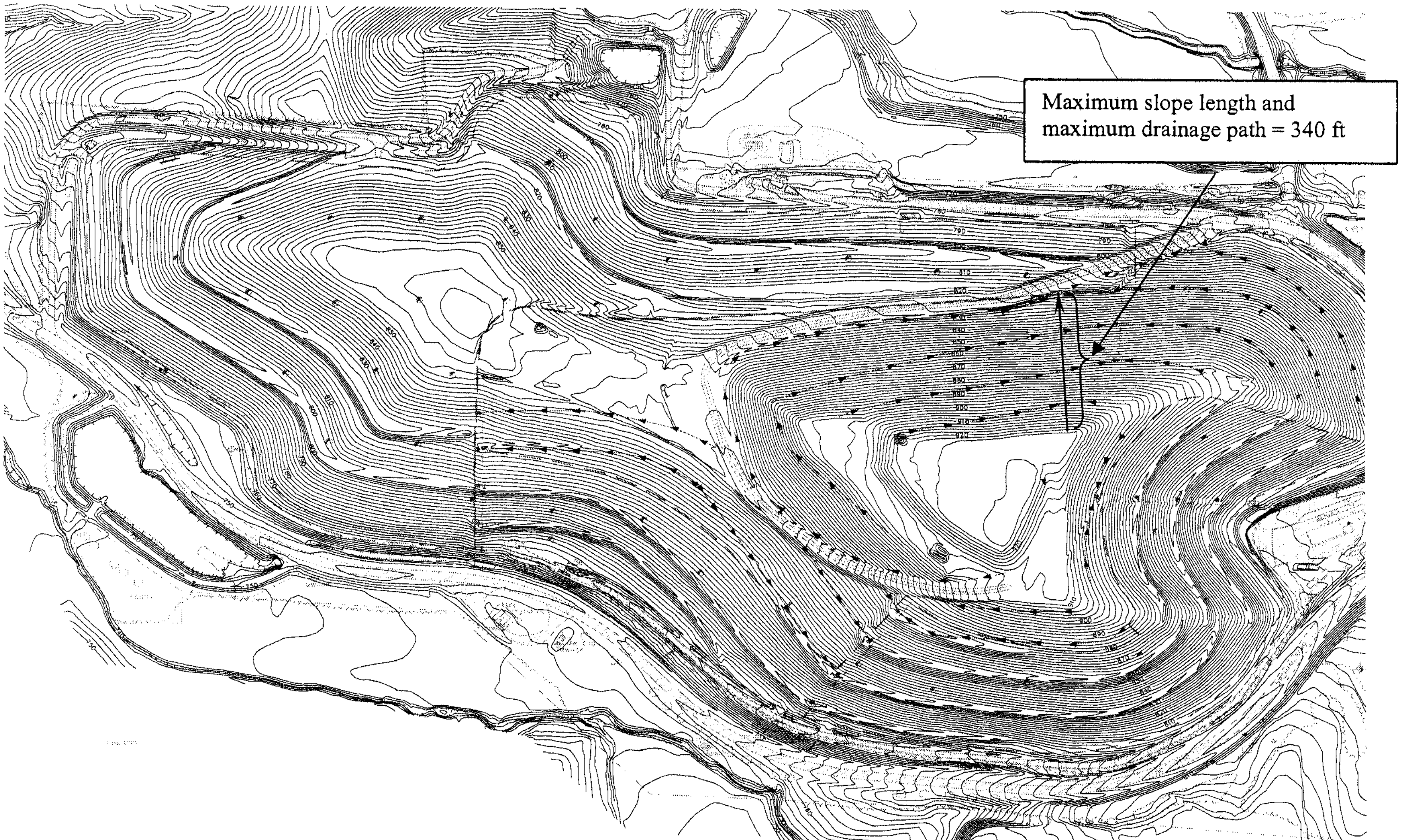

Figure 2. Maximum Slope Length and Maximum Drainage Path in the Area of Investigation

ATTACHMENT 1
HELP MODEL OUTPUT FILE

Written by: Tamer Elkady Date: 05 /03 /03 (YY MM DD) Reviewed by: YC/J. Beech Date: 05/ 3 17 (YY MM DD)

Client: WMI Project: Piedmont LF Project/Proposal No.: NCP2005 Task No: 3184

```
               TYPE 2 - LATERAL DRAINAGE LAYER
                MATERIAL TEXTURE NUMBER   0
THICKNESS                    =      0.18   INCHES
POROSITY                     =      0.8500 VOL/VOL
FIELD CAPACITY               =      0.0100 VOL/VOL
WILTING POINT                =      0.0050 VOL/VOL
INITIAL SOIL WATER CONTENT   =      0.0293 VOL/VOL
EFFECTIVE SAT. HYD. COND.    =   5.00000000000      CM/SEC
SLOPE                        =     30.80   PERCENT
DRAINAGE LENGTH              =    340.0    FEET


                          LAYER  3
                          --------

               TYPE 4 - FLEXIBLE MEMBRANE LINER
                MATERIAL TEXTURE NUMBER  35
THICKNESS                    =      0.06   INCHES
POROSITY                     =      0.0000 VOL/VOL
FIELD CAPACITY               =      0.0000 VOL/VOL
WILTING POINT                =      0.0000 VOL/VOL
INITIAL SOIL WATER CONTENT   =      0.0000 VOL/VOL
EFFECTIVE SAT. HYD. COND.    =  0.199999996000E-12 CM/SEC
FML PINHOLE DENSITY          =      0.00   HOLES/ACRE
FML INSTALLATION DEFECTS     =      1.00   HOLES/ACRE
FML PLACEMENT QUALITY        =  3 - GOOD


                          LAYER  4
                          --------

                 TYPE 3 - BARRIER SOIL LINER
                MATERIAL TEXTURE NUMBER  17
THICKNESS                    =      0.20   INCHES
POROSITY                     =      0.7500 VOL/VOL
FIELD CAPACITY               =      0.7470 VOL/VOL
WILTING POINT                =      0.4000 VOL/VOL
INITIAL SOIL WATER CONTENT   =      0.7500 VOL/VOL
EFFECTIVE SAT. HYD. COND.    =  0.300000003000E-08 CM/SEC


          GENERAL DESIGN AND EVAPORATIVE ZONE DATA
          ----------------------------------------

NOTE:  SCS RUNOFF CURVE NUMBER WAS COMPUTED FROM DEFAULT
         SOIL DATA BASE USING SOIL TEXTURE # 8 WITH A
         GOOD STAND OF GRASS, A SURFACE SLOPE OF 31.%
         AND A SLOPE LENGTH OF  340. FEET.

SCS RUNOFF CURVE NUMBER            =     74.30
FRACTION OF AREA ALLOWING RUNOFF   =    100.0    PERCENT
AREA PROJECTED ON HORIZONTAL PLANE =      1.000  ACRES
EVAPORATIVE ZONE DEPTH             =     24.0    INCHES
INITIAL WATER IN EVAPORATIVE ZONE  =      6.911  INCHES
UPPER LIMIT OF EVAPORATIVE STORAGE =     11.112  INCHES
LOWER LIMIT OF EVAPORATIVE STORAGE =      2.784  INCHES
```

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```
INITIAL SNOW WATER                    =    0.000  INCHES
INITIAL WATER IN LAYER MATERIALS      =    7.066  INCHES
TOTAL INITIAL WATER                   =    7.066  INCHES
TOTAL SUBSURFACE INFLOW               =    0.00   INCHES/YEAR


            EVAPOTRANSPIRATION AND WEATHER DATA
            -----------------------------------

  NOTE:  EVAPOTRANSPIRATION DATA WAS OBTAINED FROM
           GREENSBORO             NORTH CAROLINA

       STATION LATITUDE                         =  35.13 DEGREES
       MAXIMUM LEAF AREA INDEX                  =   3.50
       START OF GROWING SEASON (JULIAN DATE)    =     90
       END OF GROWING SEASON (JULIAN DATE)      =    305
       EVAPORATIVE ZONE DEPTH                   =  24.0  INCHES
       AVERAGE ANNUAL WIND SPEED                =   7.60 MPH
       AVERAGE 1ST QUARTER RELATIVE HUMIDITY    =  66.00 %
       AVERAGE 2ND QUARTER RELATIVE HUMIDITY    =  68.00 %
       AVERAGE 3RD QUARTER RELATIVE HUMIDITY    =  74.00 %
       AVERAGE 4TH QUARTER RELATIVE HUMIDITY    =  70.00 %


  NOTE:  PRECIPITATION DATA WAS SYNTHETICALLY GENERATED USING
           COEFFICIENTS FOR    GREENSBORO          NORTH CAROLINA

           NORMAL MEAN MONTHLY PRECIPITATION (INCHES)

JAN/JUL     FEB/AUG     MAR/SEP     APR/OCT     MAY/NOV     JUN/DEC
-------     -------     -------     -------     -------     -------
  3.51        3.37        3.88        3.16        3.37        3.93
  4.27        4.19        3.64        3.18        2.59        3.38


  NOTE:  TEMPERATURE DATA WAS SYNTHETICALLY GENERATED USING
           COEFFICIENTS FOR    GREENSBORO          NORTH CAROLINA

       NORMAL MEAN MONTHLY TEMPERATURE (DEGREES FAHRENHEIT)

JAN/JUL     FEB/AUG     MAR/SEP     APR/OCT     MAY/NOV     JUN/DEC
-------     -------     -------     -------     -------     -------
 37.50       39.90       48.00       58.30       66.50       73.50
 77.20       76.30       69.90       58.40       48.50       40.20


  NOTE:  SOLAR RADIATION DATA WAS SYNTHETICALLY GENERATED USING
           COEFFICIENTS FOR    GREENSBORO          NORTH CAROLINA
             AND STATION LATITUDE  =   35.13 DEGREES
```

Written by: Tamer Elkady Date: 05/03/03 (YY/MM/DD) Reviewed by: YC/J. Beech Date: 05/3/17 (YY/MM/DD)

Client: WMI Project: Piedmont LF Project/Proposal No.: NCP2005 Task No: 3184

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AVERAGE MONTHLY VALUES IN INCHES FOR YEARS 1 THROUGH 20

	JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
PRECIPITATION						
TOTALS	2.70	3.27	4.10	2.81	3.05	3.96
	4.76	5.13	3.82	2.52	2.60	3.80
STD. DEVIATIONS	1.65	1.38	1.92	1.36	1.29	2.53
	1.98	2.38	2.39	1.71	1.91	1.80
RUNOFF						
TOTALS	0.018	0.018	0.074	0.004	0.000	0.030
	0.028	0.045	0.059	0.038	0.037	0.031
STD. DEVIATIONS	0.067	0.042	0.219	0.011	0.001	0.116
	0.096	0.111	0.102	0.083	0.159	0.069
EVAPOTRANSPIRATION						
TOTALS	1.303	1.597	2.845	3.096	4.131	3.703
	4.606	4.140	2.797	1.269	1.132	1.041
STD. DEVIATIONS	0.216	0.331	0.296	0.671	0.835	1.699
	1.584	1.220	1.215	0.395	0.244	0.196
LATERAL DRAINAGE COLLECTED FROM LAYER 2						
TOTALS	1.7859	1.3721	1.9320	0.6217	0.2012	0.1089
	0.2144	0.2468	0.5319	0.8020	0.7332	1.9276
STD. DEVIATIONS	1.8739	0.9839	1.4982	0.6190	0.4939	0.3657
	0.6700	0.5375	0.8342	0.9593	1.1128	1.5029
PERCOLATION/LEAKAGE THROUGH LAYER 4						
TOTALS	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
STD. DEVIATIONS	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000

AVERAGES OF MONTHLY AVERAGED DAILY HEADS (INCHES)

DAILY AVERAGE HEAD ON TOP OF LAYER 3						
AVERAGES	0.0025	0.0021	0.0027	0.0009	0.0003	0.0002
	0.0003	0.0003	0.0008	0.0011	0.0010	0.0027
STD. DEVIATIONS	0.0026	0.0015	0.0021	0.0009	0.0007	0.0005
	0.0009	0.0007	0.0012	0.0013	0.0016	0.0021

****

Written by: Tamer Elkady Date: 05/03/03 (YY/MM/DD) Reviewed by: YC/J. Beech Date: 05/3/17 (YY/MM/DD)

Client: WMI Project: Piedmont LF Project/Proposal No.: NCP2005 Task No: 3184

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AVERAGE ANNUAL TOTALS & (STD. DEVIATIONS) FOR YEARS 1 THROUGH 20

	INCHES		CU. FEET	PERCENT
PRECIPITATION	42.52	(7.072)	154363.9	100.00
RUNOFF	0.383	(0.3024)	1388.60	0.900
EVAPOTRANSPIRATION	31.659	(3.7179)	114922.14	74.449
LATERAL DRAINAGE COLLECTED FROM LAYER 2	10.47762	(4.15683)	38033.773	24.63903
PERCOLATION/LEAKAGE THROUGH LAYER 4	0.00000	(0.00000)	0.005	0.00000
AVERAGE HEAD ON TOP OF LAYER 3	0.001	(0.000)		
CHANGE IN WATER STORAGE	0.005	(0.9704)	19.42	0.013

****

Written by: Tamer Elkady Date: 05 /03 /03 (YY MM DD) Reviewed by: YC/J. Beech Date: 05/ 3 17 (YY MM DD)

Client: WMI Project: Piedmont LF Project/Proposal No.: NCP2005 Task No: 3184

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******************************************************************************

              PEAK DAILY VALUES FOR YEARS    1 THROUGH   20
-------------------------------------------------------------------------
                                            (INCHES)      (CU. FT.)
                                           ----------    -------------
   PRECIPITATION                              3.76        13648.800

   RUNOFF                                     0.940        3413.4856

   DRAINAGE COLLECTED FROM LAYER  2           1.47240      5344.80859

   PERCOLATION/LEAKAGE THROUGH LAYER  4       0.000000        0.00004

   AVERAGE HEAD ON TOP OF LAYER  3            0.063

   MAXIMUM HEAD ON TOP OF LAYER  3            0.124

   LOCATION OF MAXIMUM HEAD IN LAYER  2
         (DISTANCE FROM DRAIN)                0.0 FEET

   SNOW WATER                                 2.87        10419.2432


   MAXIMUM VEG. SOIL WATER (VOL/VOL)                  0.3722

   MINIMUM VEG. SOIL WATER (VOL/VOL)                  0.1160

    ***  Maximum heads are computed using McEnroe's equations.  ***

         Reference:  Maximum Saturated Depth over Landfill Liner
                     by Bruce M. McEnroe, University of Kansas
                     ASCE Journal of Environmental Engineering
                     Vol. 119, No. 2, March 1993, pp. 262-270.

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Written by: Tamer Elkady Date: 05 /03 /03 (YY MM DD) Reviewed by: YC/J. Beech Date: 05/ 3 17 (YY MM DD)

Client: WMI Project: Piedmont LF Project/Proposal No.: NCP2005 Task No: 3184

****

FINAL WATER STORAGE AT END OF YEAR 20

LAYER	(INCHES)	(VOL/VOL)
1	7.0119	0.2922
2	0.0114	0.0635
3	0.0000	0.0000
4	0.1500	0.7500
SNOW WATER	0.000	

****
****

FINAL COVER VENEER STABILTY ANALYSIS POST-MOVEMENT CONDITION

GEOSYNTEC CONSULTANTS

COMPUTATION COVER SHEET

Client: WM **Project:** Piedmont Landfill **Project/Proposal #:** NCP2005 **Task #:** 3184

TITLE OF COMPUTATIONS FINAL COVER STYSTEM VEENER STABILITY ANALYSIS – POST-MOVEMENT CONDITIONS

COMPUTATIONS BY:

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ASSUMPTIONS AND PROCEDURES CHECKED BY: (Peer Reviewer)

Signature J. F. Beech DATE 17 MAR 2005

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COMPUTATIONS CHECKED BY:

Signature Cao Yiwen DATE 3.17.05

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COMPUTATIONS BACKCHECKED BY: (Originator)

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APPROVED BY: (PM or Designate)

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Printed Name and Title: Jay F. Beech, Principal

APPROVAL NOTES:

REVISIONS (Number and initial all revisions)

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Written by:	Tamer Elkady	Date:	05 /03 /03 (YY MM DD)	Reviewed by:	YC/J. Beech	Date:	05/ 3 17 (YY MM DD)
Client:	WMI	Project:	Piedmont LF	Project/Proposal No.:	NCP2005	Task No:	3184

FINAL COVER SYSTEM VENEER STABILITY ANALYSIS POST-MOVEMENT CONDITION

PURPOSE

The purpose of the analyses presented in this calculation package is to evaluate the static factor of safety (FS) against the potential for a veneer-type slip surface to develop along and through the final cover system of the Piedmont landfill for conditions subsequent to the movement that occurred in August 2004 (hereafter referenced as Post-Movement condition).

METHOD OF ANALYSES

Static Stability:

Static slope stability of a landfill final cover system can be analyzed assuming infinite slope conditions or finite slope conditions. The infinite slope stability analysis method considers a slope of infinite length whereby the driving and resisting forces occur only along or parallel to an interface (i.e., slip plane). The finite slope stability analysis method considers a slope of finite length and additionally takes into account soil strength above a slip plane, primarily as a toe-buttressing effect. Since the final cover slopes at the Piedmont Landfill are relatively short, the finite slope stability analysis method is appropriate.

The finite slope stability factor of safety equation, as formulated by Giroud, et al. [1995], is:

$$FS = \left[\frac{\gamma_t(t-t_w)+\gamma_b t_w}{\gamma_t(t-t_w)+\gamma_{sat} t_w}\right]\frac{\tan\delta}{\tan\beta} + \frac{a/\sin\beta}{\gamma_t(t-t_w)+\gamma_{sat}t_w}$$

$$+\left[\frac{\gamma_t(t-t^*_w)+\gamma_b t^*_w}{\gamma_t(t-t_w)+\gamma_{sat}t_w}\right]\left[\frac{\tan\phi/(2\sin\beta\cos^2\beta)}{1-\tan\beta\tan\phi}\right]\frac{t}{h}$$

$$+\left[\frac{1}{\gamma_t(t-t_w)+\gamma_{sat}t_w}\right]\left[\frac{1/(\sin\beta\cos\beta)}{1-\tan\beta\tan\phi}\right]\frac{ct}{h} \qquad (1)$$

where: FS = factor of safety;

δ = interface friction angle;
a = apparent interface adhesion;
ϕ = soil internal friction angle;
c = apparent soil cohesion;
γ_t = moist soil unit weight;
γ_b = buoyant soil unit weight;
γ_{sat} = saturated soil unit weight;
t = depth of cover soil above critical interface;
t_w = water depth above critical interface;
t_w* = water depth at slope toe;
β = slope inclination; and
h = vertical height of slope.

It should be noted that while the above equation is specifically for an interface above a geomembrane, or similar layers, it can also be applied to interfaces below the geomembrane by changing the coefficient of the first term, (i.e., the coefficient of $\tan\delta / \tan\beta$) to 1.0. The slope geometry, which is used to derive the above equation, is shown in Figure 1.

FINAL COVER DATA

The final cover system consists, from top to bottom, of the following:

- a 24-inch thick protective soil layer;
- a 8 oz/yd^2 geotextile filter;
- an Agru America, Inc. Drain® Liner (hereafter referred to as Drain® Liner) consisting of a 50-mil thick liner low density polyethylene (LLDPE) geomembrane with 0.18-inch thick drainage studs; and
- a geosynthetic clay liner (GCL).

Critical conditions for the evaluation of the stability of final cover system for the Piedmont Landfill consider (i) a slope of 33.3 percent (3H:1V) and a length of 340 ft (Case I); and (ii) a slope of 30.8 percent (3.25H:1V) and a length of 340 ft (Case II). The location of the maximum slope length (i.e., 340 ft) is shown on Figure 2.

The protective soil component of the final cover system was assumed to have a unit weight of 120 pcf and shear parameters of c = 72 psf and $\phi = 30^{\circ}$. The cohesion of 72 psf was assumed for the vegetative soil layer due to the root reinforcement effect, typically causing the apparent cohesion to increase in the range of 72 to 360 psf as reported by Abramson et al [1996].

The veneer stability analyses require information about post-movement interface shear strength between the components of the final cover system. To this end, GeoSyntec Consultants (GeoSyntec) performed a laboratory test using fresh samples of geomembrane to simulate the movement and uplift that was observed in August 2004. Based on laboratory test results 5A and 5B in Appendix B, the post-movement interface shear strength was evaluated to be approximately 90 percent of the peak interface shear strength. Therefore, the Post-Movement secant interface shear strength parameters were evaluated to be as follows:

Condition	**Shear Strength Parameters**	
	Friction, δ (degrees)	**Adhesion, a (psi)**
Peak	22.3$^{(1)}$	0

Note:
(1) $\tan^{-1}$ (0.9 * tan (24.5) = 22.3 degrees

Information on water depth in the drainage layer above the geomembrane (t_w in Equation 1) was obtained from analysis performed using HELP model [Schroeder, 1994] included as Attachment 1 in the calculation package titled "*Final Cover System Veneer Stability – Post-Installation Condition*". Based on the results of this analysis, the average water depth (t_w) on a peak day was estimated to be 0.063 inches (0.00525 ft).

RESULTS AND CONCLUSIONS

Analyses were performed to evaluate the static FS of the final cover veneer stability using Post-Movement interface shear strength parameters. Equations used to calculate the FS below a geomembrane are coded in a spreadsheet presented herein as Tables 1 and 2. Based on the Analyses, the factors of safety were evaluated to be 1.28 and 1.38 for Cases I and II, respectively (Tables 1 and 2).

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REFERENCES

Abramson L.W., Lee, T.S., Sharma, S. and Boyce, G.M. *Slope Stability and Stabilization Methods*. John Wiley & Sons, New York, 1996

Giroud, J.P., Bachus, R.C., and Bonaparte, R., "Influence of Water Flow on the Stability of Geosynthetic-Soil Layered Systems on Slopes," *Geosynthetics International*, Vol. 2, No. 6, 1995, pp. 1149-1180.

Schroeder, P. R., N. M., Lloyd, C. M. and Zappi, P.A. (1994) "The Hydrologic Evaluation of Landfill Performance (HELP) Model: User's Guide for Version 3," EPA/600/R-94/168a, September 1994, U. S. Environmental Protection Agency Office of Research and Development, Washington, D.C.

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Table 1 Spreadsheet for the evaluation of static factor of safety for the Piedmont Landfill final cover system using Post-Movement interface shear strength

Case I: 3H:1V Final Cover Slope

FS Below GEOMEMBRANE		
Input Parameters:		
γ_t (Unit wt of soil):	120	pcf
γ_{sat} (Saturated unit wt of soil):	120	pcf
γ_w (Unit wt of water):	62.4	pcf
γ_b (Buoyant unit wt of soil):	57.6	pcf
t_w (water thickness):	0.005	ft
t* (water thickness at slope toe):	0.005	ft
δ (weakest interface friction angle):	22.3	deg
ϕ (friction angle of soil):	30	deg
a (interface adhesion)	0	psf
c (cohesion of soil above geomembrane)	72	psf
T (Tension in Geosynthetics)	0	psf
h (height of slope):	113.33	ft
t (thickness of soil layer)	2.0	ft
β (slope angle)	18.4	deg
FS	**1.28**	

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Table 2 Spreadsheet for the evaluation of static factor of safety for the Piedmont Landfill final cover system using Post-Movement interface shear strength

Case II: 3.25H:1V Final Cover Slope

FS Below GEOMEMBRANE		
Input Parameters:		
γ_t (Unit wt of soil):	120	pcf
γ_{sat} (Saturated unit wt of soil):	120	pcf
γ_w (Unit wt of water):	62.4	pcf
γ_b (Buoyant unit wt of soil):	57.6	pcf
t_w (water thickness):	0.005	ft
t* (water thickness at slope toe):	0.005	ft
δ (weakest interface friction angle):	22.3	deg
ϕ (friction angle of soil):	30	deg
a (interface adhesion)	0	psf
c (cohesion of soil above geomembrane)	72	psf
T (Tension in Geosynthetics)	0	psf
h (height of slope):	113.33	ft
t (thickness of soil layer)	2.0	ft
β (slope angle)	17.1	deg
FS	1.38	

Written by: Tamer Elkady Date: 05 /03 /03 (YY MM DD) Reviewed by: YC/J. Beech Date: 05/ 3 17 (YY MM DD)

Client: WMI Project: Piedmont LF Project/Proposal No.: NCP2005 Task No: 3184

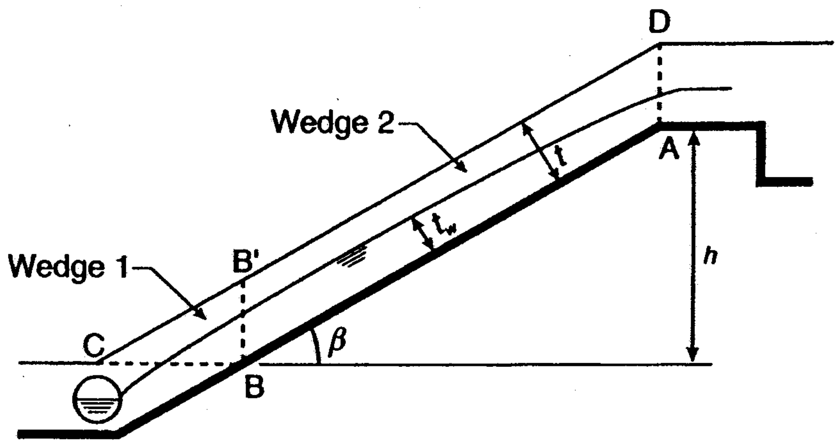

Figure 1 Slope Geometry Used to Derive Finite Slope Stability Equation

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Client: WMI Project: Piedmont LF Project/Proposal No.: NCP2005 Task No: 3184

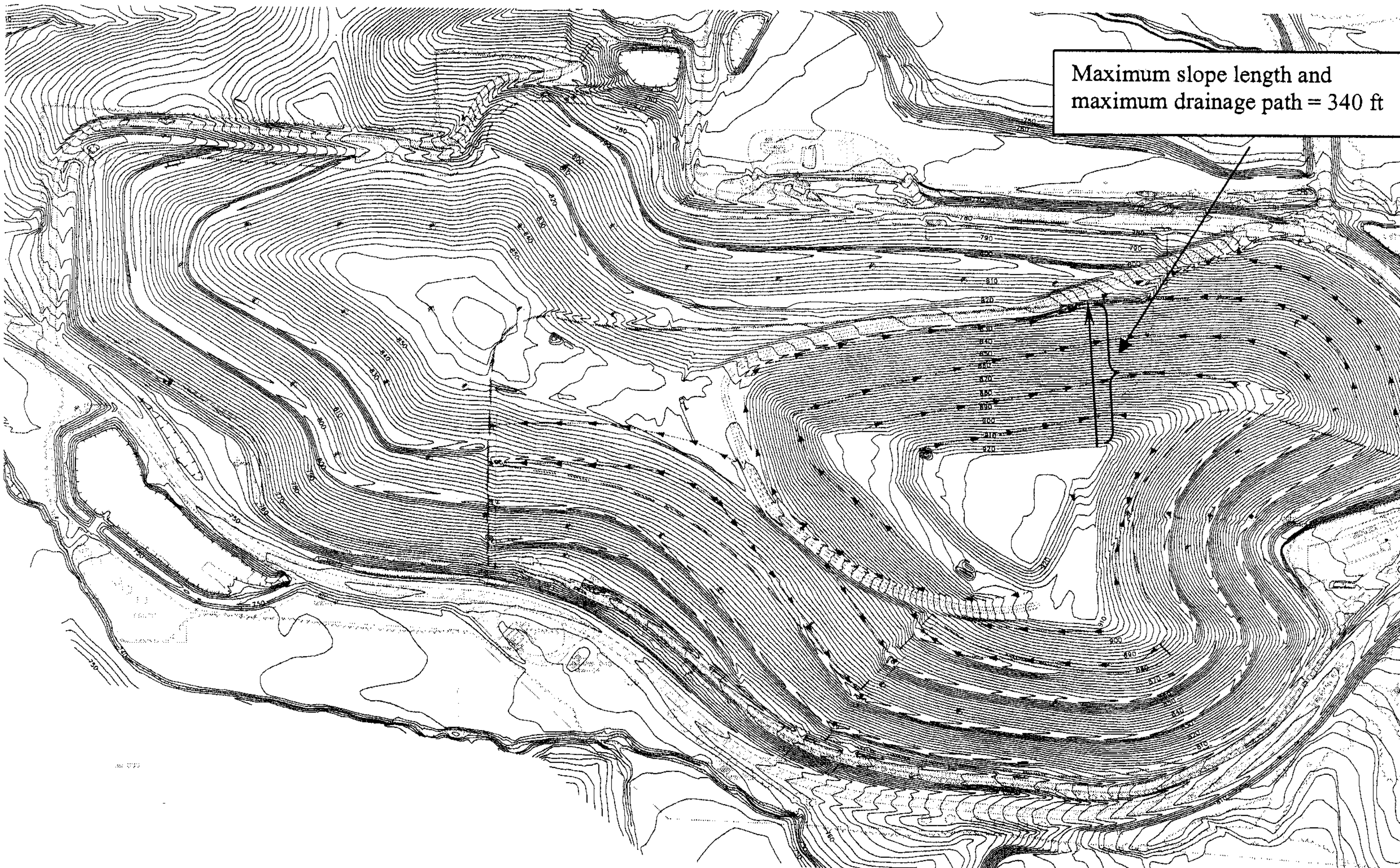

Figure 2. Maximum Slope Length and Maximum Drainage Path in the Area of Investigation

MAXIMUM ALLOWABLE LANDFILL GAS PRESSURE UNDERNEATH FINAL COVER SYSTEM

GEOSYNTEC CONSULTANTS

COMPUTATION COVER SHEET

Client: WM **Project:** Piedmont Landfill **Project/Proposal #:** NCP2005 **Task #:** 3184

TITLE OF COMPUTATIONS MAXIMUM ALLOWABLE GAS PRESSURE UNDERNEATH FINAL COVER SYSTEM

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APPROVAL NOTES:

REVISIONS (Number and initial all revisions)

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Client:	WMI	Project:	Piedmont LF	Project/Proposal No.:	NCP2005	Task No:	3184

MAXIMUM ALLOWABLE LANDFILL GAS PRESSURE UNDERNEATH FINAL COVER SYSTEM

PURPOSE

The purpose of the analysis presented in this calculation package is to evaluate the maximum allowable landfill gas pressure that can be applied underneath the geomembrane component of the final cover system before the final cover system becomes unstable.

METHOD OF ANALYSES

Static slope stability analysis of a landfill cover system considering landfill gas pressure underneath the geomemebrane can be estimated using an infinite slope stability analysis developed by Thiel [1998]. The infinite slope stability factor of safety equation, as formulated by Thiel [1998], is:

$$FS = \frac{a + (\gamma\, H \cos\beta - u_g)\tan\delta}{\gamma\, H \sin\beta} \qquad (1)$$

where:
- FS = factor of safety;
- a = effective geomembrane-soil interface adhesion;
- γ = total soil unit weight;
- H = thickness of the soil cover on top of the geomembrane;
- β = slope inclination; and
- u_g = maximum allowable gas pressure.

Rearranging Equation 1, the maximum allowable landfill gas pressure underneath the final cover system corresponding to minimum factor of safety of 1.0 can be calculated using the following equation:

$$u_{g\text{-allow}} = \gamma\, H \cos\beta - \frac{(H\,\gamma \sin\beta - a)}{\tan\delta} \qquad (2)$$

Because of the gas migration barrier provided by the geomembrane, it should be noted that the equations presented herein are mainly for an interface below the geomembrane where gas pressures can potentially occur.

Analyses were performed to evaluate the maximum allowable landfill gas pressure for the following two conditions:

(i) final cover post-installation conditions (Post-Installation conditions); and
(ii) after the movement that occurred in August 2004 (Post-Movement conditions)

INPUT PARAMTERS

Evaluation of the maximum allowable gas pressure underneath the final cover system (using Equation 2) requires information on the interface shear strength between the components of the final cover system. Information on the peak interface shear strength parameters during Post-Installation condition are obtained from the calculation package titled "*Final Cover Veneer Stability Analysis – Post-Installation Condition*". Similarly, information on interface shear strength parameters during Post-Movement condition are obtained from the calculation packages titled "*Final Cover Veneer Stability Analysis – Post-Movement Condition*". A summary of interface shear strength parameters used in the analysis are presented in the following table:

Condition	**Shear Strength Parameters**	
	Friction, δ (degrees)	**Adhesion, a (psi)**
Post-Installation	24.5	0
Post-Movement	22.3	0

The thickness of the protective soil cover layer is 2.0 ft. The total unit weight of the soil cover was assumed to be 120 pcf. Analyses were performed for the critical final cover slope configuration of 3 horizontal to 1 vertical (β = 18.4 degrees).

SUMMARY AND RESULTS

Analyses were performed to evaluate the maximum allowable gas pressure ($u_{g\text{-allow}}$) using Post-Installation and Post-Movement interface shear strength parameters. Results of these analyses are presented in the following table:

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Condition	Shear Strength Parameters		Maximum allowable gas pressure ($u_{g\text{-allow}}$)	
	Friction, δ (degrees)	Adhesion, a (psi)	psf	inches of water
Post-Installation	24.5	0	61.1	11.8
Post-Movement	22.3	0	42.6	8.2

REFERENCES

Thiel, R.S., "Design Methodology for a Gas Pressure Relief Layer Below a Geomembrane Landfill Cover to Improve Slope Stability," *Geosynthetics International*, Vol. 5, No. 6, 1998, pp. 589-617.