Carmen Johnson

23-02

23-02 2/7/12

BLE INC.

BUNNELL-LAMMONS ENGINEERING, INC.

GEOTECHNICAL, ENVIRONMENTAL AND CONSTRUCTION MATERIALS CONSULTANTS

RESPONSE TO NCSWS REVIEW COMMENTS SITE HYDROGEOLOGIC REPORT

JMN/CLEVELAND CONTAINER INDUSTRIAL LANDFILL CLEVELAND COUNTY, NORTH CAROLINA

Prepared for:

HODGES, HARBIN, NEWBERRY, & TRIBBLE, INC.
484 Mulberry Street, Suite 265
Macon, Georgia 31201

Prepared By:

BUNNELL-LAMMONS ENGINEERING, INC.
1200 Woodruff Road, Suite B-7
Greenville, South Carolina 29607

NORTH CAROLINA LICENSED SEAL 1043 GEOLOGIST MARK S. PREDDY
7/5/01

NORTH CAROLINA LICENSED SEAL 1264 GEOLOGIST THOMAS L LAMMONS
7/5/01

July 5, 2001

BLE Project Number J99-1307-04

Hodges, Harbin, Newberry & Tribble, Inc.

CONSULTING ENGINEERS

484 Mulberry Street, Suite 265 • Post Office Box 974
Macon, Georgia 31201

H. Lowry Tribble, Jr., PE
William F. Hodges, PE
Harold L. Newberry, PE
J. Steven Harbin, PE
Jeffrey M. Browne, PE

Telephone (478) 743-7175
Fax (478) 743-1703

July 17, 2001

Mr. Jim Coffey
NC Department of Environment & Natural Resources
1646 Mail Service Center
Raleigh, NC 27699-1646

Re: JMN/Cleveland Container Landfill
HHNT Project No. 6703-030-01

Dear Mr. Coffey:

We have enclosed a "Response to NCSWS Review Comments - Site Hydrogeological Report" on the subject project for your review. Three questions in the document will be addressed under separate cover within the next few weeks. These are:

NCSWS Item No. 2:
There appears to be no information concerning a "conceptual design plan", as this Rule (.0504(1)(d)) requires.

NCSWS Item No. 3:
This Rule (.0504(1)(g)(ii))requires that "type, quantity and source of waste" be listed. There is no mention as to what types of industrial waste will be received at the proposed landfill.

NCSWS Item No. 8:
On page 3-8, Section 3.6.4 of the report, it is not clear why future development of phase 2 will limit recharge "resulting in lower ground-water levels". Does the conceptual design have a liner system that would limit recharge?

We therefore request that the Section begin review of the hydrogeological elements of the report while we complete information required by items No. 2,3 and 8 above.

Should you have any questions, please call.

Sincerely,

HODGES, HARBIN, NEWBERRY & TRIBBLE, INC.

William F. Hodges, P.E.
Principal

WFH/jlm

cc: Don Edwards, w/encl.
John Murray, P.E., w/encl.
Brant Lane, w/o encl.
Mark Preddy, P.G., w/o encl.

BLE INC.
BUNNELL-LAMMONS ENGINEERING, INC.
GEOTECHNICAL, ENVIRONMENTAL AND CONSTRUCTION MATERIALS CONSULTANTS

JUL 2001
Received
Solid Waste
Section

July 5, 2001

Hodges, Harbin, Newberry, & Tribble, Inc.
484 Mulberry Street, Suite 265
Macon, Georgia 31201

Attention: Mr. William F. Hodges, P.E.

Subject: **Response to NCSWS Review Comments**
Site Hydrogeologic Report
JMN/Cleveland Container Industrial Landfill
Cleveland County, North Carolina
BLE Project Number J99-1307-04

Gentlemen:

The North Carolina Solid Waste Section (NCSWS) has completed their review of the *Site Hydrogeologic Report* dated October 2, 2000, for the JMN/Cleveland Container Industrial Landfill prepared by Bunnell-Lammons Engineering, Inc. (BLE). The NCSWS review comments were outlined in a letter to BLE dated April 9, 2001. This letter addresses the NCSWS review comments and provides supplemental information where requested.

We appreciate the opportunity to serve as your hydrogeological and geotechnical consultant on this project and look forward to continue working with you at the JMN/Cleveland Container Industrial Landfill. If you have any questions, please contact us at (864) 288-1265.

Sincerely,

BUNNELL-LAMMONS ENGINEERING, INC.

NORTH CAROLINA LICENSED SEAL 1043 GEOLOGIST MARK S. PREDDY

NORTH CAROLINA LICENSED SEAL 1264 GEOLOGIST THOMAS L LAMMONS

Mark S. Preddy, P.G.	Thomas L. Lammons, P.G.
Senior Hydrogeologist	Principal Hydrogeologist
Registered, North Carolina No. 1043	Registered, North Carolina No. 1264

d:\msp files\cleveland container lf nc\130704\text\1307-04 review comments.doc

1200 WOODRUFF ROAD, SUITE B-7
GREENVILLE, SOUTH CAROLINA 29607

PHONE (864) 288-1265
FAX (864) 288-4430

BLE INC.

LIST OF ATTACHMENTS

PROJECT INFORMATION

The subject industrial waste landfill is located in Cleveland County, North Carolina near the town of Shelby. The site consists of approximately 110 acres, which includes an existing landfill, soil borrow areas, and a proposed Phase 2 expansion area (approximately 40 acres). The existing landfill is unlined and has been receiving limited industrial and construction and demolition (C&D) waste since the 1970's. Since the facility continued to receive waste after January 1, 1998 and has plans for expansion (the Phase 2 area), the landfill must obtain a permit modification, or new permit, under applicable solid waste regulations 15A NCAC 13B.

The North Carolina Department of Environment and Natural Resources (NCDENR) has evaluated the compliance status of the existing facility with regards to solid waste management Rule 15A NCAC 13B .0503. Camp, Dresser, and McKee (CDM) prepared a *Landfill Design Plan*, dated December 1, 1997. The Division of Waste Management, Solid Waste Section (NCSWS) reviewed the Plan and determined that the information previously submitted did not meet the requirements of Rule .0503(2)(d)(ii). This rule pertains to the design of the landfill such that the ground-water standards under 15A NCAC 2L will not be exceeded in the uppermost aquifer at the compliance boundary. As stated in a letter dated January 25, 1999 from Mr. James C. Coffey of the NCSWS to Republic Services, Inc.:

> "Specifically, the submitted ground water monitoring information does not demonstrate current compliance with the ground water standards in the upper most aquifer at the compliance boundary; the modeling information submitted does not provide adequate hydrogeologic characterization of the site to demonstrate future compliance with ground water standards in the upper most aquifer at the compliance boundary; and the information provided concerning previously disposed waste does not provide accurate physical and chemical characteristics of the leachate."

BLE proposed a scope of work to address the requirements of Rule .0503(2)(d)(ii) (*Work Plan for Site Hydrogeologic Characterization, JMN/Cleveland Container Industrial Landfill*, dated March 3, 1999, BLE Job Number J99-1307-02). Additionally, the Work Plan addressed the geologic/hydrogeologic characteristic of the Phase 2 area under Rules .0503(2)(d) and .0504(1)(c). The Work Plan included opportunities for the NCSWS to be notified of project progress as tasks are completed.

BLE prepared a *Status Report of Site Hydrogeologic Characterization, JMN/Cleveland Container Industrial Landfill* dated October 4, 1999 (BLE Job Number J99-1307-04) that was submitted to the NCSWS. The Status Report included the results of the residential water well inventory and the fracture trace analysis. The Status Report also provided recommendations for drilling/piezometer installation on the existing landfill site and the Phase 2 area.

A *Site Hydrogeologic Report* (SHR) for the site dated October 2, 2000 was prepared by BLE (Project Number J99-1307-04) and submitted to the NCSWS. The SHR provided geological, hydrogeological, and geotechnical investigation data as required for site permitting under North Carolina's rules for solid waste management.

Upon completion of their review, the NCSWS requested additional information in a letter to BLE on April 9, 2000. This response document addresses the NCSWS review comments. Where necessary, the SHR has been modified to include the additional and/or revised information.

INFORMATION REQUESTED BY THE NCSWS

NCSWS Item No. 1:
BLE's SHR does not address Rule 15A NCAC 13B .0503(2)(d).

Supplemental Information:
Rule 15A NCAC 13B .0503(2)(d) addresses ground water standards regarding vertical separation, and landfill base-liner system design criteria.

Figure 8 (Attachment C) has been added to the SHR, which is an estimated long-term seasonal high water table map. The map was prepared using historical water levels in the monitoring wells and estimated values in the piezometers derived from historical precipitation data. The estimated values were derived by the methods described below.

Published precipitation data from the Cleveland County region show that in 1998, the combined winter-spring months had above average precipitation, and this period had the second highest precipitation totals between 1980 and 2000 (Attachment D). Ground-water levels are also assumed to be above average during this period.

Referring to Figure 8, only the monitoring wells were present on site in 1998, and as expected, the measured 1998 water levels were the highest values recorded since well installation (Table 4 in Attachment B). The July 27, 1998 values were on the average 2.11 feet higher than their highest measurements during 2000. Therefore, 2.11 feet was added to the maximum value of each of the new piezometers to establish estimated values for 1998. Therefore, the estimated long-term seasonal high water table elevation contour map (Figure 8) was prepared using the real data from the monitoring wells that were present in 1998, and the estimated values in the piezometers.

Conservatively, Figure 8 can be used for landfill subgrade design. A four-foot vertical buffer should be maintained between the estimated long-term seasonal high water levels and the bottom elevation of the solid waste. Using this potentiometric surface, Hodges, Harbin, Newberry, & Tribble, Inc. (HHNT), the design engineers, will prepare and submit a base liner system design under separate cover.

This information has been added to the SHR in Section 3.6.2 (Precipitation and Seasonal Ground-Water Level Trends), Table 4 (Ground-Water Elevation Measurements), Figure 8 (Estimated Long-Term Seasonal High Water Table Map), and Appendix J (Precipitation Data). Additionally, these data can be located in Attachments A-D of this document.

__NCSWS Item No. 2:__
There appears to be no information concerning a "conceptual design plan", as this Rule (.0504(1)(d)) requires.

Supplemental Information:
HHNT will prepare and submit the conceptual design plan under separate cover.

__NCSWS Item No. 3:__
This Rule (.0504(1)(g)(ii)) requires that "type, quantity and source of waste" be listed. There is no mention as to what types of industrial waste will be received at the proposed landfill.

Supplemental Information:
HHNT will discuss the waste stream for the proposed landfill in a facility plan to be submitted under separate cover.

__NCSWS Item No. 4:__
This Rule requires basic hydraulic characteristics (saturated hydraulic conductivity, volume percent water, and porosity) be provided for each major lithologic unit. The report appears to have some discussion of the major lithologic units on site. However, the information is not organized in a manner to provide the hydraulic characteristics representative of each hydrologic unit. Submit a table displaying each lithologic unit (residual soil, saprolite, partially weathered rock, and upper fractured rock) with their representative hydraulic characteristics and proper units.

Supplemental Information:
There are three primary lithologic units at the site: saprolite, partially weathered rock, and the upper fractured bedrock. The basic hydraulic properties of these units, based on laboratory and field testing, are summarized on Table 9 (Attachment B), which has been be added to the SHR.

__NCSWS Item No. 5:__
This information [Item 4 above] *is necessary in order to determine any possible preferential ground-water flow pathways and develop a basic understanding of the characteristics of the uppermost aquifer. Also an evaluation of the fracture trace data and any other natural or man-made influences that could possibly effect preferential ground-water flow is needed.*

Supplemental Information:
The fracture trace data was discussed in Section 3.5.2 of the SHR. Our analysis of the local fracture trends, bedrock joint orientations, and foliation orientations indicate that the prevailing fracture trend is northwest-southeast.

Influences on water levels were discussed in Section 3.6.4 of the SHR (which is now Section 3.6.5). The existing landfill is being developed in the upland area west of the proposed Phase 2 area. As cell construction proceeds to Phase 2, ground water infiltration and recharge of the water table will be somewhat limited due to the final compacted soil cap of relatively low permeability soil (i.e, 10^{-5} cm/sec), plus surface water will be directed off of the cell tops via drainage berms and piping in the proposed and existing landfill area. Consequently, the water table will be lowered near the proposed cells. Additionally, ground water in the seasonal discharge area between the existing landfill and Phase 2 may dry up after future cell development takes place in the upgradient recharge area.

With consideration to fracture trends, plus man-made and natural influences on ground-water flow, preferential ground-water flow in the uppermost aquifer occurs through the saprolite, partially weathered rock, and upper fractured bedrock towards the south and southeast. Hydraulically, these units exhibit similar hydraulic conductivity values (Table 9). Over most of the site, the water table is in the saprolitic soils. Ground-water discharges along the seasonal drainage feature between the existing landfill and the proposed Phase 2 area, and to Buffalo Creek. The ground-water flow direction across the site is generally parallel to the prevailing fracture trend of northwest-southeast.

This information has been added to Section 3.6.4.1 (Existing Flow Pathways) of the SHR.

NCSWS Item No. 6:
In a letter from Matt Gamble dated December 22, 1998, he made reference to a geologic "contact between the Cherryville Granite and a biotite gneiss". An evaluation is needed regarding the depth and location of this contact and if it could influence ground-water flow in the uppermost aquifer at the site.

Supplemental Information:
The subject site is located in an area that has been mapped by others and by BLE as gneiss and schist of the Inner Piedmont Belt, west of a contact with the Cherryville Granite. The Cherryville Granite is a weakly foliated medium-grained micaceous monzogranite, and occurs next to more abundant biotite gneiss and schist of the Inner Piedmont (Horton and Zullo, 1991). The gneiss and schist are characterized by strong foliation and abundant biotite, feldspar and quartz. Geologic contacts between different Paleozoic-age formations in the Inner Piedmont Belt have themselves been re-worked (i.e, recrystallized, folded) by metamorphic events during the Paleozoic Era and their locations are typically inferred.

According to Brown and others (1985), the geologic contact between the metamorphic rocks and the Cherryville Granite roughly parallels Buffalo Creek on the southeast side of the site. The closest boring to Buffalo Creek is PZ-4c (200 feet), which is in a lowland location and drilled to a 50-foot depth (lowest drilled elevation on site, 568 feet msl). PZ-4c did not encounter the Cherryville Granite. The contact is most likely moderately to steeply dipping, based on the relatively straight contact trace provided by Horton and Zullo (1991) and Goldsmith and others (1988). Hypothetically, if the contact between the two geologic formations aligned with the trend of Buffalo Creek and the bottom elevation of PZ-4c, the contact would roughly dip below the site at an angle greater than 11°. Therefore, the contact would have to be 100 to over 600 feet below ground surface over the majority of the site.

Based on the above information, the contact between the gneiss and schist of the Inner Piedmont Belt and the Cherryville Granite will not influence the uppermost aquifer ground-water flow in the residual soils and upper fractured bedrock in the vicinity of proposed cell construction on site.

This information has been added to Section 3.6.4.2 (Potential Flow Pathways) of the SHR.

<u>NCSWS Item No. 7:</u>
Other questions Matt [Gamble] *raised in his letter also need to be addressed: the relative importance of vertical flow, the relative importance of fracture flow, the depth to bedrock and how much of the aquifer system occurs in unconsolidated sediments (especially during dry seasonal low ground-water conditions), existing water quality at the site and whether the direction and rate of ground-water flow would indicate that future water quality assessment and possible corrective action could limit areas of future development at the site.*

Supplemental Information:
Vertical flow at the site was discussed in Section 3.6.6 of the SHR (which is now Section 3.6.7). Based on the site topography, the vertical gradients observed in the study area are typical for unconfined aquifers in the Piedmont. Ground-water recharge occurs in the upland areas. Ground-water discharge occurs to the drainage feature between the existing landfill and Phase 2, and to Buffalo Creek on the southern site boundary.

Recent drought conditions during 1999 and 2000 have rendered water levels at the site lower than average. However, ground water is still above the bedrock surface over the majority of the site, with the exception of the area near PZ-3 (central portion of Phase 2). Therefore, the majority of the uppermost aquifer system is in the soil mantle above the bedrock. Where sampled, bedrock beneath the site is severely to slightly weathered biotite-quartz-feldspar gneiss (average RQD of 48 percent), with very close to widely spaced, horizontal to shallow dipping foliation and fractures. In this highly weathered and fractured state, the upper bedrock zone is best characterized as an equivalent porous media where ground-water flow directions are not significantly influenced by fracture orientations.

Low concentrations (well below North Carolina 15A NCAC 2L standards) of volatile organic compounds (VOCs) have been detected in monitoring wells MW-5 (chlorobenzene and 1,2-dichlorobenzene) and MW-7 (cis 1,2-dichloroethene) during recent semi-annual sampling events. The concentrations of VOCs detected in MW-5 are not statistically significant increases (SSIs), but cis 1,2-dichloroethene in MW-7 has been calculated as a SSI. The source of cis 1,2-dichloroethene detected in well MW-7 is uncertain at this time. Landfill leachate is a possible source; however, leachate typically consists of an assemblage of several VOCs rather than an individual VOC. Unconfirmed information indicate the possible source of the cis 1,2-dichloroethene is inadvertent well contamination during installation (i.e., drilling immediately adjacent to or through trash during installation), or possibly landfill gas migration. MW-7 has been replaced, as indicated in Section 4.1.3 of the SHR.

If corrective action were to be required in the future, the most likely locations would be in the vicinity of the existing landfill near wells MW-5 and MW-7. However, the need for engineered systems or other intrusive corrective actions would be unlikely. Based on the low concentrations of VOCs detected, natural attenuation via advection, dispersion, adsorption, biodegradation, and biotransformation should render the contaminants non-detectable before reaching downgradient receptors. In the unlikely event that intrusive corrective measures would be needed, the affected areas would not be in the vicinity of Phase 2 and would not limit future landfill construction.

This information has been added to the SHR in Sections 3.6.4 (Ground-Water Flow Direction and Flow Pathways), 3.6.7 (Vertical Flow Gradients), and 3.6.8 (Ground-Water Quality at the Existing Landfill).

NCSWS Item No. 8:
On page 3-8, Section 3.6.4 of the report, it is not clear why future development of phase 2 will limit recharge "resulting in lower ground-water levels". Does the conceptual design have a liner system that would limit recharge?

Supplemental Information:
HHNT will provide a conceptual design for the landfill. However, even without a synthetic liner system, recharge to the aquifer will be somewhat restricted over the cell footprint by the final compacted soil cap and surface water conveyances.

NCSWS Item No. 9:
On page 3-12, Section 3.7.5, 2nd paragraph, it is stated that a "red-brown silt clay (CL" is found in the uppermost aquifer (3 – 5.5 feet below ground surface). However, there is no laboratory data to support this statement. Submit this supporting lab data.

Supplemental Information:
The borrow study conducted by Camp, Dresser & McKee (CDM) identified soils based on visual and laboratory testing. Although they identified "CL" type soils at several test pits, none of the samples they tested were of the CL classification. CDM's test pit logs and laboratory results are already attached to the SHR in Appendix K.

BLE conducted additional confirmation laboratory testing on two shallow soil samples from boring PZ-2ab. The Atterberg Limits test results identified the soils as CH and MH. The laboratory test results are in Attachment E, and will be added to the SHR (Table 3, Appendix E). Additionally, the text of the SHR has been changed to classify the shallow residual soils as CH (rather than CL), MH, and ML.

NCSWS Item No. 10:
Section 4.0 – Modification to the Groundwater Monitoring System for Existing Landfill:

The phase "relevant point of compliance" is from the .1600 rules for municipal solid waste landfill facilities and is not appropriate for industrial landfills. The appropriate phrase from the 2L rules is "compliance boundary". (Section 4.1.2)

It is my understanding that Mark Poindexter of the Solid Waste Section Compliance Branch has pre-approved the new upgradient wells (MW-1B and MW-1C). These wells are to be located between the waste boundary for phase 1 and the private well north of the landfill.

The desire to relocate wells MW-6 and MW-7 at a further distance from the waste boundary is understood. However, replacement of these wells needs to be on the landfill side of any natural or man-made drainage features. According to Figure 4, the proposed relocation of wells MW-6 and MW-7 does not appear to fall on the landfill side of the drainage feature.

Supplemental Information:
The SHR has been modified to replace "relevant point of compliance" with "compliance boundary".

The shallow ditch located near wells MW-6 and MW-7 is a man-made feature between the landfill and residual soils south of the landfill and is not structurally controlled by geology. Furthermore, the water table is about 20 feet below ground surface and the shallow ditch has minimal, if any influence over the ground-water flow to the southeast (Figure 7 of the SHR). These observations were described to Bobby Lutfy of the NCSWS on March 6, 2001 in a telephone conversation with Mark Preddy of BLE. Mr. Lutfy suggested moving both proposed wells (MW-6A and MW-7A) about 50 feet eastward. The corrected locations of MW-6A and MW-7A are indicated on the revised figures of the SHR.

This information has been added to the SHR in Section 4.1.3 (Monitoring Well Locations).

<u>NCSWS Item No. 11:</u>
Table 3 – On most of the soil samples, the Atterberg limits are not provided. This information is needed to properly identify the USCS soil classifications. How were these soils given USCS classifications without the Atterberg limits?

Supplemental Information:
Atterberg Limit tests were not required for the classification of the four samples in question. The USCS classifications were assigned based on field observation and grain size analysis. Field classification indicated the samples were non-plastic; therefore, Atterberg Limit testing was not performed. The samples have 61 to 85 percent retained in the No. 200 sieve (i.e., sandy samples). Under the UCSC, soils with greater then 50 percent sand and greater than 12 percent fines are classified as SM, SC or SC-SM. Since the majority of the fines are in the silt fraction, the soils were classified as SM.

<u>NCSWS Item No. 12:</u>
Even though Table 4 provides several months of water table measurements and Appendix J provides some precipitation data, there appears to be no evaluation of the seasonal trends in water table fluctuations.

Supplemental Information:
Based on the National Oceanic and Atmospheric Administration (NOAA) precipitation data from the 1980 to 2000, the Cleveland County region had below average precipitation during 1999 and 2000, and from this data we assume that ground-water levels were also below average at this time. During 1998, the winter, spring and summer months had above average precipitation and water levels were probably above average as well. Only the monitoring wells were present at the site during 1998 (i.e., no piezometers on site), and as expected, the 1998 data were the highest measured water levels in these wells (Table 4). For example, the water levels collected in the monitoring wells on July 27, 1998 were on the average 2.11 feet higher than their highest measurements during 2000.

Annual water level fluctuations at the site have ranged from 0.63 to 5.12 feet with an average of 2.5 feet.

This information has been added to the SHR in Section 3.6.2 (Precipitation and Seasonal Ground-Water Level Trends), Table 4 (Ground-Water Elevation Measurements), and Appendix J (Precipitation Data).

NCSWS Item No. 13:
Several of the piezometers have no time of boring (TOB) ground-water level measurements [Table 4]. *In the "Notes" at the bottom of the table, it is stated that the wells were "not stable at the time of measurement". Time of boring measurements are not expected to be stable. In fact, the difference in TOB and stabilized measurements provide useful information, such as time to obtain stabilized conditions, well recharge rates, and relative hydraulic conductivities.*

Supplemental Information:
Potable water was added during piezometer construction to the well annulus (between the PVC and the formation material) of PZ-6b and PZ-7b. Additionally, potable water was added during rock coring of PZ-1c and PZ-4c. The TOB water level for these four piezometers was at the ground surface. Table 4 has been revised (Attachment B).

NCSWS Item No. 14:
Why is the depth to water for PZ-3 reported at >43.98 feet for all measurements? Is the well dry? Submit an explanation of this.

Supplemental Information:
Field measurements of PZ-3 indicate that piezometer is dry to a depth of 43.98 feet. However, this number does not match Table 2 or the boring log in Appendix C of the SHR. Therefore, they have been corrected (Attachment B and F).

NCSWS Item No. 15:
Table 5 – The hydraulic conductivity is not broken down based on the major lithologic units identified at the site.

Supplemental Information:
Table 5 has been revised to break the hydraulic conductivity values into three units (saprolite, partially weathered rock, and fractured bedrock).

NCSWS Item No. 16:
Table 6 – The porosity and ground-water velocity data are not broken down based on the major lithologic units identified at the site.

Supplemental Information:
Table 6 has been revised to break the values into three units (saprolite, partially weathered rock, and fractured bedrock).

NCSWS Item No. 17:
Table 7 – The vertical hydraulic data is only provided for one date. Are there any changes in vertical gradients with seasonal fluctuations in potentiometric levels?

Supplemental Information:
The vector of vertical ground-water flow has not changed during the timeframe they have been recorded from the four well pairs. However, the gradient magnitude has varied in each of the well pairs as follows:

Piezometer Pair	Maximum Gradient (ft/ft)	Minimum Gradient (ft/ft)	Direction
PZ-1ab/PZ-1c	0.17	0.046	Downward
PZ-4ab/PZ-4c	0.014	0.0011	Slightly Downward
MW-4/PZ-6b	0.12	0.091	Upward
MW-7/PZ-7b	0.088	0.0097	Slightly Downward

This information has been added to the SHR in Section 3.6.7 (Vertical Flow Gradients).

NCSWS Item No. 18:
Figure 4 – Are the on-site drainage features perennial or intermittent streams? Are there any springs on site? Where are the limits of the 100-year floodplain? Are there any wetlands?

Supplemental Information:
As described in Section 3.4 of the SHR, the topographic drain between Phase 2 and the existing landfill is an intermittent stream and serves as a wet season conveyance for surface water and shallow ground-water discharges. Buffalo Creek (perennial stream) flows southwest from the site and ultimately converges with the Broad River four miles southwest of the site.

HHNT will provide information regarding the 100-year floodplain and wetlands under separate cover.

NCSWS Item No. 19:
Figure 7 – The ground-water contours appear fairly reasonable based on the existing boring density. The ground-water flow regime in the vicinity of the drainage feature between phases 1 and 2 needs to be better defined in order to determine to what extent contamination from phase 1 could impact the area in phase 2. This could possibly effect the buffers, footprint, and design for phase 2.

Supplemental Information:
The evaluation of the ground-water flow regime of the drainage feature between the existing landfill and the proposed Phase 2 area included:

1. Installing a piezometer-pair in an upgradient location (PZ-1ab/PZ-1c) to measure vertical hydraulic gradients;
2. Installing a piezometer-pair in a downgradient location (MW-4/PZ-6b) to measure vertical hydraulic gradients;
3. Hydraulic testing of piezometers across the site;
4. Performing a fracture trace analysis; and
5. Preparing water table contour maps.

The drainage feature is a seasonal (intermittent) stream. The year-round water table occurs within the saprolite; however, bedrock is relatively shallow in the upper end of the feature and deeper in the lower end. A shallow body of standing water was formerly located along the feature between MW-4 and MW-5, but has since been drained.

The north-south trend of the drainage feature does not parallel primary or secondary fracture trends, according to the fracture trace analysis performed by BLE. Therefore, the feature does not appear to be controlled by geologic structure.

The upper end of the drainage feature has a recharge gradient and the lower end of the drainage feature has a discharge gradient. Water table maps prepared for the area indicate convergent ground-water flow towards the drainage feature. Potential contamination from the existing landfill should flow south towards Buffalo Creek. Effective ground-water monitoring would include monitoring wells along the south side of the existing landfill and Phase 2.

This information has been added to the SHR as Section 3.6.9 (Ground-Water Flow Regime near the Centrally Located Drainage Feature).

NCSWS Item No. 20:
The Field Logs and Boring/Coring Records indicate that several of the piezometers were not properly constructed regarding the annular space above the bentonite seal. Based on Title 15 A Subchapter 2C Section .0100 Rule .0108(2)(c), "grout shall be placed in the annular space between the casing and the borehole from the land surface to the clay seal above the packing material". Many of the piezometers have either soil cuttings or bentonite in the annular space. Two of the piezometers are open boreholes in rock. For future reference, the Solid Waste Section may not accept data taken from incorrectly designed and/or constructed wells or piezometers. It is very important that the incorrectly installed piezometers be properly abandoned. The piezometers with soil cuttings in the annular space will need to be drilled out the full depth prior to being abandoned with an approved grout mixture. Also, it is the policy of the Solid Waste Section to have the sandpack no more than two feet above the top of the screen. The bentonite seal should be at least one-foot thick. The sandpack for PZ-1ab was 6.9 feet above the top of the screen.

Supplemental Information:
As stated in the SHR in Section 2.5 (page 2-2), the intent for these temporary piezometers is to be permanently abandoned in the future prior to landfill development:

> "The piezometers are intended only for investigation use, were not constructed as permanent monitoring wells, and will not be part of the permanent ground-water monitoring system. Prior to landfill construction activities, the piezometers will be abandoned in accordance with 15A NCAC 2C, Rule .0113(a)(2) by over-drilling and backfilling the resulting boreholes from the bottom to the ground surface with neat cement."

Proposed piezometer construction was submitted to the NCSWS in Appendix B of the *Work Plan for Site Hydrogeologic Characterization, JMN/Cleveland Container Industrial Landfill*, dated March 3, 1999, BLE Job Number J99-1307-02). In the Work Plan, piezometers completed in soil could have bentonite in their annular space up to the ground surface. Additionally, The "open bedrock" piezometer construction was described in the Work Plan. These types of piezometer construction are typical for projects of this type and have been acceptable according to other regulatory review by the NCSWS for other projects.

Each of the piezometers installed in soil were constructed with a bentonite seal at least two-feet thick.

Each of the piezometers installed in soil had the sand pack from 0.7 to 2.9 feet above the screened interval. The sand pack in PZ-1ab is not 6.9 feet above the top of the screen, but rather 2.9 feet as indicated on the field log in Appendix B and the boring log in Appendix C of the SHR.

NCSWS Item No. 21:
What is the reason for the difference in the auger refusal depths for piezometers PZ-1ab and PZ-1c, and PZ-4ab and PZ-4c? Based on the piezometer diagrams for piezometers PZ-1c and PZ-4c, how could a three-inch borehole be cored into the bedrock if a three-inch casing was grouted in-place to auger refusal?

Supplemental Information:
As stated in the SHR in Section 3.2, auger refusal depths can vary from one location to another, based on rock weathering irregularities and rock boulders in the soil mantle above the bedrock:

> "Fractures, joints, and the presence of less resistant rock types facilitate weathering. Consequently, the profile of the partially weathered rock and hard rock is quite irregular and erratic, even over short horizontal distances. Also, it is not unusual to find lenses and boulders of hard rock and zones of partially weathered rock within the soil mantle, well above the general bedrock level."

Therefore, the differences in auger refusal depths are most likely a rock lens at PZ-1ab and an irregular bedrock surface between PZ-4ab and PZ-4c.

The inside diameter of the PVC casing is slightly larger than three inches (3.068 inches) and the outside diameter of the core barrel's cutting head is slightly less than three inches (2.984 inches). Consequently by rounding these numbers, both are three inches.

REFERENCES

Brown, M.B., and others (compilers), 1985, *Geologic Map of North Carolina*: NCDNRCD, scale 1:500,000.
- General geologic setting, and fault locations.

Gair, J.E., 1989, *Mineral Resources of the Charlotte 1°x2° Quadrangle, North Carolina and South Carolina*: USGS Professional Paper 1462.
- Geologic Setting

Goldsmith, R.M., Milton, D.J., and Horton, J.W., 1988, *Geologic map of the Charlotte 1°x2° Quadrangle, North Carolina and South Carolina*: USGS Miscellaneous Investigation Map I-1251-E, 1:250,000.
- Geologic Setting

Horton, J.W. and Zullo, V.A., 1991, *The Geology of the Carolinas: Carolina Geological Society fifteenth anniversary volume*: The University of Tennessee Press, Knoxville, TN.
- General geologic setting.

Attachment A

Revised Text of the SHR

October 2, 2000
Revised July 5, 2001

Hodges, Harbin, Newberry, & Tribble, Inc.
484 Mulberry Street, Suite 265
Macon, Georgia 31201

Attention: Mr. William F. Hodges, P.E.

Subject: **Site Hydrogeologic Report**
JMN/Cleveland Container Industrial Landfill
Cleveland County, North Carolina
BLE Project Number J99-1307-04

Gentlemen:

Bunnell-Lammons Engineering, Inc. (BLE) has completed the Site Hydrogeologic Study for the JMN/Cleveland Container Industrial Landfill. This report addresses the relevant geologic and hydrogeologic site application requirements as outlined in the North Carolina Rules for Solid Waste Management, 15A NCAC 13B .0503(2)(d) and .0504(1)(c). The attached report describes the work performed and presents the results obtained.

We appreciate the opportunity to serve as your geological, hydrogeological, and geotechnical consultant on this project and look forward to continue working with you at the JMN/Cleveland Container Industrial Landfill. If you have any questions, please contact us at (864) 288-1265.

Sincerely,

BUNNELL-LAMMONS ENGINEERING, INC.

Mark S. Preddy, P.G.
Senior Hydrogeologist
Registered, North Carolina No. 1043

Thomas L. Lammons, P.G.
Principal Hydrogeologist
Registered, North Carolina No. 1264

Daniel B. Bunnell, P.E.
Principal Geotechnical Engineer
Registered, North Carolina No. 13814

d:\msp files\cleveland container lf nc\130704\text\1307-04 shrrev1.doc

TABLE OF CONTENTS

TABLE OF CONTENTS–Continued

LIST OF TABLES

Table

LIST OF FIGURES

Figure

EXECUTIVE SUMMARY

The North Carolina Department of Environment and Natural Resources (NCDENR) has evaluated the compliance status of the existing facility with regards to solid waste management Rule 15A NCAC 13B .0503 pertaining to site and design requirements for disposal sites. The NCDENR specified deficiencies in the existing landfill design plan pertaining to compliance with current ground-water standards, and hydrogeologic information of the site. This *Site Hydrogeologic Report* addresses the deficiencies specified by the NCDENR for the existing landfill and provides the required site suitability demonstrations for a proposed ±40-acre Phase 2 expansion. The suitability criteria and applicable geologic/hydrogeologic requirements for prospective industrial waste landfill sites are outlined in the North Carolina Rules for Solid Waste Management, Title 15A NCAC 13B .0503(2)(d) and .0504(1)(c). The evaluation methodology for this study was developed to satisfy these requirements.

The entire site covers approximately 110 acres located within rolling piedmont terrain of Cleveland County, North Carolina and consists of an existing landfill and the referenced 40-acre Phase 2 expansion area. The existing landfill occupies the western portion of the site. The eastern portion of the site consists of the Phase 2 area, where topography is characterized by a central high ridge, which drops off radially to the east, west, and south. A centrally located intermittent stream that flows south to Buffalo Creek separates the existing landfill area and the Phase 2 area.

The surface drainage pattern in Phase 2 is radial from the central ridge to the topographic ravine located between Phase 2 and the existing landfill, and to Buffalo Creek south of the site. The topographic ravine between Phase 2 and the existing site serves as a southward flowing wet season conveyance for surface water to Buffalo Creek. Buffalo Creek flows southwest from the site and ultimately converges with the Broad River four miles southwest of the site.

The site is located within the Inner Piedmont Belt of the Piedmont Physiographic Province. The crystalline rocks of the Inner Piedmont Belt occur in generally northeast-southwest trending geologic belts in the Carolinas, and consist of a stack of highly metamorphosed thrust sheets bound on the northwest by the Brevard Shear Zone and to the southeast by the Kings Mountain Shear Zone. The Inner Piedmont includes high-grade metamorphosed sedimentary and igneous rocks that have been exposed to multiple deformations. Rock types that resulted from the multiple metamorphisms include gneiss, schist and amphibolite with northeast/southwest trending foliation with varying degrees of dip. Quaternary-age sediments consisting of sand and gravel fill the stream valleys. Holocene and younger age faults were not found on site or within 200 feet of the site from the literature review or from the field reconnaissance.

Five soil borings and two rock corings were performed on the Phase 2 area and two soil borings were performed on the existing landfill site. At eight of these nine locations, ground water was encountered. The soil and rock borings ranged in depth from 17.5 to 52 feet below ground surface. Clayey/silty soils were encountered near the ground surface and grade with increasing depth into micaceous sandy silts, silty sands, and then partially weathered rock. Residual soil and partially weathered rock overly the basement bedrock. The overburden thickness varies from 29 to 52 feet, averaging 38 feet over most areas. The upper bedrock was cored at two locations. The rock cores generally exhibited moderate to severe fracturing with rock quality designation (RQD) values from 0 to 100 percent with an average of 48 percent.

Nine ground-water piezometers were installed in selected locations to measure ground-water elevations and characterize the site hydrogeology. Water level measurements were recorded from the piezometers and existing monitoring wells during January 2000 through May 2001. Additionally, water level information from the existing monitoring wells was obtained from July 1998 to July 1999. The highest water levels recorded at the site were in July 1998 after several months of above average precipitation. The water levels in the monitoring wells were on the average 2.11 feet higher in 1998 than in 2000. An estimated long-term seasonal high water table contour map was prepared using the 1998 water levels in the monitoring wells, and using estimated values in the piezometers; the estimated values were established by adding 2.11 feet to the highest recorded measurement of each piezometer.

Ground water is present above the bedrock surface over much of the site, with the exception of PZ-3, which is located in an upland location in the Phase 2 area. The saprolite and bedrock units are hydraulically connected, comprising a single unconfined aquifer where recharge rates, flow rates and storativity differ between the units. Generally, shallow ground water flows to the south from recharge areas in the north-central upland locations, and discharges to Buffalo Creek along the southern site boundary.

Based on slug tests, the hydraulic conductivity in the saprolite was measured at 3.0 x 10-4 cm/sec. Hydraulic conductivity in the residual soil zone ranges from 5.5×10^{-4} cm/sec to 3.8×10^{-5} cm/sec. Hydraulic conductivity in the bedrock piezometers ranges from 7.6×10^{-4} cm/sec to 7.0×10^{-4} cm/sec. The average ground-water seepage velocity across the site is about 0.34 ft/day.

The residual soils, partially weathered rock, and rock at the site provide a stable foundation for the landfill waste placement and the associated earthwork cut and fill slopes. Settlement of the subsurface profile due to waste placement will be minimal. Residual soils consisting of red-brown silty clay (CH) and sandy clayey silt (ML and MH) were found to depths of 3 to 5.5 feet below ground surface. These soils would readily achieve a remolded hydraulic conductivity (permeability, k) $\leq 1 \times 10^{-5}$ cm/sec acceptable for use as a soil base liner or low permeability soil cap. Soils capable of achieving a permeability $k \leq 1 \times 10^{-7}$ cm/sec were found in limited quantity. The remaining residual soils are acceptable for use as structural fill for embankments and final or daily cover.

Based on the results of field and laboratory testing, field observations, and data from published literature, the study area meets the North Carolina geological, hydrogeological, and geotechnical suitability criteria for siting of an industrial waste landfill.

-oOo-

1.0 PROJECT INFORMATION

The subject industrial waste landfill is located in Cleveland County, North Carolina near the town of Shelby (Figure 1). The site consists of approximately 110 acres, which includes an existing landfill, soil borrow areas, and a proposed Phase 2 expansion area (approximately 40 acres). The existing landfill is unlined and has been receiving limited industrial and construction and demolition (C&D) waste since the 1970's. Since the facility continued to receive waste after January 1, 1998 and has plans for expansion (the Phase 2 area), the landfill must obtain a permit modification, or new permit, under applicable solid waste regulations 15A NCAC 13B.

The North Carolina Department of Environment and Natural Resources (NCDENR) has evaluated the compliance status of the existing facility with regards to solid waste management Rule 15A NCAC 13B .0503. Camp, Dresser, and McKee (CDM) prepared a *Landfill Design Plan*, dated December 1, 1997. The Division of Waste Management, Solid Waste Section (Section) reviewed the Plan and determined that the information submitted does not meet the requirements of Rule .0503(2)(d)(ii). This rule pertains to the design of the landfill such that the ground-water standards under 15A NCAC 2L will not be exceeded in the uppermost aquifer at the compliance boundary. As stated in a letter dated January 25, 1999 from Mr. James C. Coffey of the Section to Republic Services, Inc.:

> "Specifically, the submitted ground water monitoring information does not demonstrate current compliance with the ground water standards in the upper most aquifer at the compliance boundary; the modeling information submitted does not provide adequate hydrogeologic characterization of the site to demonstrate future compliance with ground water standards in the upper most aquifer at the compliance boundary; and the information provided concerning previously disposed waste does not provide accurate physical and chemical characteristics of the leachate."

BLE prepared a proposed scope of work to address the requirements of Rule .0503(2)(d)(ii) (*Work Plan for Site Hydrogeologic Characterization, JMN/Cleveland Container Industrial Landfill*, dated March 3, 1999, BLE Job Number J99-1307-02), herein referred to as the "Work Plan". Additionally, the Work Plan provided a scope of work to address the geologic/hydrogeologic characteristic of the Phase 2 area under Rules .0503(2)(d) and .0504(1)(c). The Work Plan includes opportunities for the Section to be notified of project progress as tasks are completed.

BLE prepared a *Status Report of Site Hydrogeologic Characterization, JMN/Cleveland Container Industrial Landfill* dated October 4, 1999 (BLE Job Number J99-1307-04). The Status Report included the results of the residential water well inventory and the fracture trace analysis. The Status Report also provided recommendations for drilling/piezometer installation on the existing landfill site and the Phase 2 area. This report includes the results of the aforementioned tasks discussed in the Status Report.

-oOo-

2.0 FIELD INVESTIGATION

The North Carolina Division of Solid Waste Management (NCDSWM) requires that Site Hydrogeologic Studies include the performance of one boring per 10-acres of permitted site area. The acreage of the Phase 2 area is approximately 40 acres. Seven piezometers were installed on the Phase 2 area during this Site Hydrogeologic Study. Additionally, two piezometers were installed on the existing landfill site. These nine new piezometers supplement the previous nine monitoring wells at the site.

A discussion of the drilling and soil laboratory testing methodology used in the site evaluation is provided below. The field activities reported below were performed under the direction of a North Carolina licensed geologist. A North Carolina-licensed driller (Superior Drilling, Inc. of Raleigh, North Carolina; No. 1769) performed drilling and piezometer installation services. A North Carolina registered land surveyor (Tommy Fields of Troy, North Carolina; RLS-2906) surveyed the horizontal and vertical coordinates of the final boring and piezometer locations.

2.1 AREA AND FIELD RECONNAISSANCE

The study area was traversed by foot to map rock outcrops and surface drainage features. A reconnaissance of private and residential water-supply wells was conducted within a 2-mile radius surrounding the site. Well locations were identified by field observation, review of published topographic maps, and aerial photographs.

2.2 FRACTURE TRACE ANALYSIS

The fracture trace analysis consisted of evaluating exposed rock outcrops and topographic fracture traces and lineaments.

The orientations of bedrock fractures (open joints, open foliation, and open bedding planes) were measured using a Brunton-style compass. The orientation information was collected from exposed rock and saprolite outcrops at the site as well as along nearby roads within about two miles of the site. The field measurements were plotted on Schmidt lower hemisphere equal-area stereonets and Rose diagrams.

Topographic fracture traces and lineaments were evaluated using topographic maps. Regionally, pronounced depressions typically develop along zones of weakness in the bedrock where fractures induce preferential weathering. This preferential weathering along bedrock fractures is ultimately expressed topographically as linear valleys. The trend of fracture traces and lineaments greater than 1,000 feet in length within a 2-mile radius of the site were measured from USGS topographic maps and plotted on a Rose diagram.

2.3 SOIL TEST BORING AND ROCK CORING

Five soil test borings and two rock corings were performed on the Phase 2 area and two soil test borings were performed on the existing landfill site to study the subsurface geology. Soil samples were obtained from the borings at 2.5-foot intervals within the upper ten feet below the ground surface, and at five-foot intervals deeper than ten feet below the ground surface. Drilling techniques consisted of hollow-stem augering and rock coring. Refer to Appendix A for discussion of the various standard drilling techniques.

Copies of boring logs produced in the field are attached in Appendix B. Soil descriptions on the field logs were based on visual examination and grain-size estimations in accordance with the Unified Soil Classification System (USCS). Upon completion of laboratory grain-size and Atterberg Limit analyses, the preliminary field classifications were adjusted accordingly as reported on the final boring logs. Soil Test Boring/Rock Coring Records showing visual descriptions of the soil and rock strata encountered are included in Appendix C.

The soil test boring locations and depths were selected to comply with the applicable NCSWS rules.

2.4 LABORATORY TESTING

Laboratory tests were conducted to confirm the field classifications and quantify pertinent engineering soil properties. Soil samples were collected using split-spoon samplers, Shelby tubes (undisturbed), and from the auger cuttings (bulk samples). The laboratory tests were performed in general accordance with applicable ASTM specifications, where available. Brief descriptions of the test procedures are included in Appendix D and the laboratory results are included in Appendix E.

2.5 GROUND-WATER INVESTIGATION

Nine piezometers were installed to monitor water table elevations and further characterize the study area hydrogeology. At two locations on the Phase 2 area, piezometer pairs were installed to measure vertical hydraulic gradients. Additionally, two deeper piezometers were installed next to existing monitoring wells on the existing landfill property to measure vertical hydraulic gradients. Piezometer installation records are included with the boring logs in Appendix C, and field procedures are described in Appendix F. Survey information is presented on Table 1 and piezometer construction details are summarized on Table 2.

Ground-water elevations were measured in the piezometers at the time of boring and after 24 hours. Additionally, measurements were taken in the piezometers on site during the period from January to July 2000.

Field permeability (slug) tests were performed in four piezometers to measure the *in situ* hydraulic conductivity of different units of the water table aquifer. Slug test field procedures and data plots are presented in Appendix G.

The piezometers are intended only for investigation use, were not constructed as permanent monitoring wells, and will not be part of the permanent ground-water monitoring system. Prior to landfill construction activities, the piezometers will be abandoned in accordance with 15A NCAC 2C, Rule .0113(a)(2) by over-drilling and backfilling the resulting boreholes from the bottom to the ground surface with neat cement.

-oOo-

3.0 RESULTS OF INVESTIGATION

3.1 RESIDENTIAL WELL RECONNAISSANCE

The locations of private and public water supply wells within two miles of the site were identified in the field during our reconnaissance in April 1999 and September 2000. The regional Division of Environmental Management office in Mooresville, North Carolina was visited to obtain private well installation records. Additionally, the extent of the public water system in the vicinity of the site was determined and documented.

The reconnaissance identified 534 habitable residences within two miles of the site, which have, or most likely have, a private water supply well. Of these residences:

- 356 residences are on roads serviced by the public water system; and
- 178 residences are on roads not serviced by the public water system.

The Cleveland County Regional Water System supplies most of the residences near the site with potable water, although many of the residences have private wells. The source of the public water is from the First Broad River. No government-owned public water supply wells were identified within 2 miles of the site.

Figure 2 shows the locations of the 516 residences identified in the field. Areas serviced by the Cleveland County Regional Water System are also shown.

Well installation records at the Mooresville Regional Office were sparse, and only six private water well records were obtained for locations within two miles of the site. Appendix C includes copies of the well records and locations of the wells are indicated on the Figure 2. These wells were dug, range in depth from 45 to 60 feet, and constructed with 24-inch diameter concrete casing.

A detailed reconnaissance was performed in the vicinity of the landfill to locate residences still using private drinking water wells. The area of the detailed reconnaissance is indicated on Figure 2. This reconnaissance included obtaining a list of property owners (county tax maps), a list of residences connected to the county water department (Cleveland County Regional Water System), and a door-to-door inquiry. In summary, there are 47 residences and two churches in the limited study area. Of these, 32 residences are connected to the water system, and 15 residences and the two churches are not connected to the public water system, which instead use private well water. Additionally, most of the residences connected to the public water system also have private wells on their property. A summary of the property owners in the limited study area is included in Appendix H.

The reconnaissance also identified a Superfund site about 7000 feet upgradient of the site. The site is known as the Kosa Plant (former Hearst-Celanese Plant), which is located along highway NC-198, north of the town of Earl. The nature of contamination at the site is chlorinated solvents and glycols in an on-site landfill. The site has been in remediation since 1986 and recent monitoring data show that contaminants are not migrating off site.

3.2 REGIONAL GEOLOGY

The subject site is located within the Inner Piedmont Belt of the Piedmont Physiographic Province (Figure 3). The crystalline rocks of the Inner Piedmont Belt occur in generally northeast-southwest trending geologic belts in the Carolinas, and consist of a stack of highly metamorphosed thrust sheets bound on the northwest by the Brevard Shear Zone and to the southeast by the Kings Mountain Shear Zone.

The Inner Piedmont includes high-grade metamorphosed sedimentary and igneous rocks that have been exposed to multiple deformations (Horton and Zullo, 1991). Rock types that resulted from the multiple metamorphisms include gneiss, schist and amphibolite with northeast/southwest trending foliation with varying degrees of dip. Quaternary-age sediments consisting of sand and gravel fill the stream valleys.

Holocene and younger age faults were not indicated on site or within 200 feet of the site from the literature review or from the field reconnaissance.

The typical residual soil profile consists of clayey soils near the surface, where soil weathering is more advanced, underlain by micaceous sandy silts and silty sands. Residual soil zones develop by the *in situ* chemical weathering of bedrock, and are commonly referred to as "saprolite." Saprolite usually consists of micaceous sand with lessor amounts of clay, silt and large rock fragments. The thickness of the saprolite in the Piedmont ranges from a few feet to more than 100 feet. The boundary between soil and rock is not sharply defined.

A transitional zone of partially weathered rock is normally found overlying the parent bedrock. Partially weathered rock is defined, for engineering purposes, as residual material with standard penetration resistance in excess of 100 blows per foot (bpf). Fractures, joints, and the presence of less resistant rock types facilitate weathering. Consequently, the profile of the partially weathered rock and hard rock is quite irregular and erratic, even over short horizontal distances. Also, it is not unusual to find lenses and boulders of hard rock and zones of partially weathered rock within the soil mantle, well above the general bedrock level.

3.3 REGIONAL HYDROGEOLOGY

Ground water in the Piedmont usually occurs as unconfined, water table aquifers in three primary geologic zones: 1) residual soil; 2) partially weathered rock; and 3) fractured bedrock. These zones are typically interconnected through open fractures and pore spaces. The configuration of the water table aquifer generally resembles the local topography.

In the residual soil and partially weathered rock zone, ground water is stored within the pore spaces and is released to the underlying bedrock through gravity drainage. Ground water within the bedrock zones occurs primarily in fracture voids. Generally, fractures within the bedrock are very small but may extend to several hundred feet.

Infiltration of precipitation to recharge the water table aquifer is primarily affected by rainfall intensity and duration, pre-existing soil moisture conditions, temperature (evaporation), and plant uptake (transpiration). Seasonal high-water tables are typically observed during the late winter and early spring months of the year when maximum infiltration efficiency occurs due to lower temperatures and less plant uptake (i.e., many plants are dormant). Seasonal low-water tables are typically observed during the summer and fall months when minimum infiltration efficiency occurs due to higher temperatures and greater plant uptake of water.

3.4 STUDY AREA PHYSIOGRAPHY AND TOPOGRAPHY

The site is located in Cleveland County, North Carolina, as shown in Figure 1. The Phase 2 area is currently comprised of undeveloped densely wooded young timber (hardwood and pine) with paths throughout the tract. The existing landfill area is occupied by the landfill disposal area, soil borrow areas, a landfill office, and a scale house.

The Phase 2 topography is characterized by a central high ridge, which drops off radially to the east, west, and south. Phase 2 is bordered by a wet season stream and the existing landfill area to the west, and Buffalo Creek to the south. The highest elevations (approximately 688 ft above mean sea level [msl]) occur at the northeastern site boundary with the lower elevations (approximately 600 ft above msl) occurring along Buffalo Creek at the southern site boundary. The relief across Phase 2 is approximately 88 feet from north to south.

Bedrock and saprolitic outcrops in the study area consist of a few boulders at the higher elevations and a few locations along streambeds in lower elevations. Cobble size float rock at the ground surface is common across the site. Rock outcrops used for the fracture trace analysis were located on site and within two miles of the site.

The preliminary soil survey of Cleveland County, prepared by the US Department of Agriculture (USDA), indicates that soil types in the upland elevations in the expansion area include the Pacolet-Saw complex on 15 to 25 percent slopes and the Pacolet sandy clay loam on 8-15 percent slopes. Soil types in the lower elevations include the Taccoa sandy loam on 0-2 percent slopes, which is occasionally flooded (USDA, unpublished).

The surface drainage pattern in Phase 2 is radial from the central ridge to the topographic ravine located between Phase 2 and the existing landfill, and to Buffalo Creek south of the site. The topographic ravine between Phase 2 and the existing landfill serves as a southward flowing wet season conveyance for surface water and ground water (i.e., intermittent stream) to Buffalo Creek. Buffalo Creek (perennial stream) flows southwest from the site and ultimately converges with the Broad River four miles southwest of the site. A topographic map/site plan is provided as Figure 4.

No monitoring wells, piezometers, or water supply wells were on the Phase 2 area prior to this investigation based on our field reconnaissance. However, nine ground-water monitoring wells were present prior to our field work at the existing landfill.

3.5 STUDY AREA SUBSURFACE CONDITIONS

Five soil borings and two rock corings were performed on the Phase 2 area and two soil tests borings were performed on the existing landfill site during December 1999 and January 2000. The geologic conditions encountered while drilling were variable with boulders and seams of partially weathered rock occurring throughout the subsurface soil overburden profile. In general, three zones were encountered: 1) the residual soils from weathered gneiss and schist, 2) the partially weathered rock, and 3) the fractured gneiss and schist bedrock. Subsurface geology at the site is shown on three cross sections designated A-A', B-B', and C-C' (Figure 5). A description of the subsurface materials encountered is provided below.

3.5.1 Geologic Unit Description

3.5.1.1 Residual Soil (Saprolite)

The residual soils are the result of the in-place weathering of the gneiss and schist bedrock. The residual soil profile below the topsoil consists of two identifiable components based on the USCS.

An upper soil component consists of reddish-brown, micaceous silty clay (CH) and sandy clayey silt (ML, MH). This soil component was encountered in six of the seven soil borings, generally ranging from 3 to 5.5 feet below ground surface. N-values range from 6 to 17 with an average value of 9, indicating a stiff average consistency.

The upper soil component grades with depth into a coarser grained, less plastic, brown, gray, and white micaceous sandy silt and silty sand which extends to the depth of the partially weathered rock and/or auger refusal. This soil component was encountered in each of the seven soil borings, generally ranging in thickness from 11.5 to 39 feet. USCS classifications of these soils are ML and SM. N-values range from 5 to 100 with an average of 18, indicating a firm average consistency.

3.5.1.2 Partially Weathered Rock

The transition between soil and rock at the site is irregular and consists of partially weathered rock overlying the parent bedrock. This zone was encountered in each of the seven borings and ranges in thickness from 2 to 17 feet. Auger refusal depths represent competent bedrock or possibly boulders of hard rock within the residual soil unit. A map of the bedrock surface (represented by auger refusal) is shown as Figure 6.

3.5.1.3 Fractured Bedrock

At the following selected test boring locations, core samples were obtained of the upper bedrock:

BORING	ROCK CORE SECTIONS (FT)	RECOVERY (%)	RQD (%)	GENERAL DESCRIPTION
PZ-1c	29 – 30.5	100	100	sl. weathered q-f-b GNEISS
	30.5 – 35.5	92	100	fresh q-f-g ORTHOGNEISS
	35.5 – 40.5	100	80	sl. weathered to fresh q-f-b-g GNEISS
	40.5 – 45.5	100	68	sl. weathered to fresh q-f-b-g GNEISS
PZ-4c	30 – 35	0	0	partially weathered rock (no recovery)
	35 – 40	50	0	mod. sev. weathered f-b-q GNEISS
	40 – 45	64	0	mod. sev. weathered f-b-q GNEISS
	45 – 50	68	34	mod. sev. to sl. weathered f-b-q GNEISS

Notes: "q" = quartz; "f" = feldspar; "b" = biotite; "g" = garnet; "mod." = moderately;
"sev." = severely; "sl." = slightly; RQD = rock quality designation

The upper bedrock profile at the PZ-1c location is generally more competent and less weathered than at the PZ-4c location. The rock core at the PZ-1c location is slightly weathered to fresh biotite-quartz-feldspar-garnet gneiss, with moderately close to widely spaced fractures. The rock core at the PZ-4c location is severely to slightly weathered biotite-quartz-feldspar gneiss, with very close to moderately closely spaced fractures. At both locations, the metamorphic foliation is horizontal to shallow dipping and the bedrock fractures are shallow dipping.

The bedrock core from the two locations had generally "fair" recovery (range of 0 to 100 percent; average of 72 percent) and "poor" RQD (range of 0 to 100 percent; average of 48 percent).

3.5.2 Fracture Trace Analysis

A fracture trace analysis was performed for this phase of work. The data plots for the fracture trace analysis are in Appendix I and a summary of the fracture trace analysis is provided below.

The trend of 126 topographic fracture traces and lineaments within two miles of the site were measured and plotted on a Rose diagram utilizing a 10° interval. Two primary fracture trace trends were observed: N31°-60°W and N11°-20°E. Additionally, three secondary trends were observed: N31°-50°E, N0°-30°W, and N61°-90°W.

The orientations and trends of 16 open joint surfaces and 18 bedrock foliation planes were measured in the field from rock and saprolite outcrops, then plotted on Schmidt equal area projections and Rose diagrams. The plots consist of one Schmidt net for plotting poles to the joints and foliation, one Rose diagram utilizing a 10° interval for joint trends, and one Rose diagram utilizing a 10° interval for foliation trends. One primary joint orientation was observed: N71°-90°W, dipping 70°-90°S; and two secondary trends were observed: N71°-N80°E, near vertical, and N41°-50°W, dipping 80°-90°NE. The metamorphic foliation orientation is N21°-40°W, dipping 32°NE-25°SW.

Our analysis of the local fracture trends, bedrock joint orientations, and foliation orientations indicate that the prevailing fracture trend is northwest. Additionally, a west-northwest trend is present as indicated from local fracture traces and joint trends. Other less prominent trends include north-northeast (primary fracture trace trend), and north-northwest (secondary fracture trace trend), and east-northeast (secondary joint trend).

3.5.3 Laboratory Testing Results

A list of the soil laboratory tests performed in the Phase 2 area is provided in the table below. The laboratory test results are summarized in Table 3. Laboratory data sheets are in Appendix E.

SAMPLE ANALYSES	SPLIT SPOON SAMPLES TESTED	REMOLDED BAG SAMPLES TESTED	SHELBY TUBE SAMPLES TESTED
Grain-Size Analysis	4	1	1
Natural Moisture Content	4	1	1
Atterberg Limits	3	1	-
Total Porosity	-	-	1
In Situ Saturated permeability*	-	-	1
Standard Proctor	-	1	-
Remolded permeability	-	1	-

* Hydraulic Conductivity

3.5.3.1 Undisturbed Samples and Split-Spoon Samples

One undisturbed Shelby Tube sample and four split spoon samples were collected and tested in the laboratory to measure natural soil conditions in the study area. The hydraulic conductivity value of the sample in the Phase 2 area was 3.5 x 10^{-4} centimeters per second (cm/sec). Total porosity in the sample analyzed was 46.5 percent. Specific yield values were estimated from grain-size analyses (Fetter, 1988), and values ranged from 3.5 percent in the silty/clayey sand near the ground surface to 30 percent in the micaceous silty sand. Atterberg limit tests on shallow soil samples indicated Liquid Limit (LL) values of 51 and 59, and Plasticity Index (PI) values of 25 and 26.

3.5.3.2 Remolded Samples

One bulk soil sample (bag sample) was collected from boring B-3 (PZ-3) of the upper 5 feet below ground surface to evaluate potential landfill daily cover and clay liner materials. The sample was analyzed in the laboratory for plasticity characteristics, natural moisture, and grain size. The Atterberg limit test indicated a LL of 51 with a PI of 18. The amount of sand, silt, and clay in the sample tested was 40, 25, and 35 percent, respectively.

A standard Proctor compaction test was performed on the bulk sample, then it was tested for permeability (hydraulic conductivity) after remolding. The sample was remolded to 95 percent of the standard Proctor maximum dry density, and approximately 5 percent wetter than the Proctor optimum moisture. The results of the remolded permeability tests (hydraulic conductivity) yielded a value of 7.3 x 10^{-8} cm/sec.

3.6 STUDY AREA HYDROGEOLOGY

Nine ground-water piezometers were installed at the site during December 1999 to January 2000, at locations shown on Figure 4. Ground water is above the bedrock surface over the majority of the site, with the exception of a higher elevation area in the central portion of Phase 2. The majority of the uppermost aquifer system is in the soil mantle above the bedrock. The water-table aquifer consists of the residual soil, partially weathered rock, and fractured gneissic bedrock. These three units are hydraulically connected and thus comprise a single unconfined aquifer. Recharge rates, flow rates and storativity differ between the units based on the unique geologic conditions of each zone. The configuration of the water table surface is a subdued replica of the ground surface. Generally, shallow ground water flows to the south and southeast from recharge areas in the north-central upland locations, and discharges to the intermittent stream between the existing landfill and Phase 2, and to Buffalo Creek in the southern portion of the site. A description of the hydrogeologic conditions in the study area is provided below.

3.6.1 Piezometer Construction and Nomenclature

Piezometer identification numbers were designated with the letters "a," "b," or "c" depending on the location of the screened interval in the piezometer. Piezometers with a screened interval that brackets the water table and is above the depth of auger refusal were designated with the letter "a". Piezometers with a screened interval at the depth of auger refusal (top of bedrock surface) and below the water table were designated with the letter "b." In cases where the water table was near the depth of auger refusal, the piezometer identification number was designated using "ab". Bedrock piezometers were designated with the letter "c". Piezometers that are dry do not have a letter designation. A typical schematic diagram of piezometer construction and nomenclature is provided in Appendix C. A description of the piezometer construction procedures is provided in Appendix F.

3.6.1.1 Auger Refusal Piezometers

Four piezometers ("b" and "ab") in the Phase 2 area were installed with screened intervals at the depth of auger refusal in the residual soil and/or the partially weathered rock zones with the screened interval at or near the water table. These piezometers include PZ-1ab, PZ-2ab, PZ-4ab, and PZ-5ab. Additionally, two piezometers, PZ-6b and PZ-7b, were installed at the depth of auger refusal with the screened interval below the water table.

One piezometer, PZ-3, was installed with the screened interval at the depth of auger refusal, but did not intersect ground water.

3.6.1.2 Bedrock Piezometers

Two piezometers ("c") in the Phase 2 area were installed as open boreholes in the bedrock zone at locations to address vertical hydraulic gradients (PZ-1c and PZ-4c).

3.6.1.3 Piezometer Pairs

There are two well clusters in the Phase 2 area: PZ-1ab/PZ-1c and PZ-4ab/PZ-4c. These piezometer pairs are used to measure the vertical hydraulic gradients in at the upper and lower ends of Phase 2.

In order to evaluate vertical hydraulic gradients at the lower end of the existing landfill, deeper piezometers were installed next to existing monitoring wells. Piezometer PZ-6b was installed near monitoring well MW-4, and PZ-7b was installed near MW-7.

3.6.2 Precipitation and Seasonal Ground-Water Level Trends

Historical National Oceanic and Atmospheric Administration (NOAA) monthly precipitation data were obtained from Division 2, North Carolina for the period of January 1980 through December

2000. The data are summarized seasonally in Appendix J such that January-March represents *winter*, April-June represents *spring*, July-September represents *summer*, and October-December represents *fall.*

Historically in the Cleveland County area, the winter and summer months will experience the most amounts of precipitation, with less precipitation in the fall and spring. In the late summer and fall months, the effects of evapotranspiration offset the contribution of this precipitation to recharge of the uppermost aquifer. Because of these natural trends, the amount of ground-water recharge, and subsequent increase in the water table level is typically greatest during winter to early summer months.

In the Cleveland County region, precipitation was above average in 1998 during the winter, spring and summer months and water levels were probably above average as well. Furthermore, the combined winter and spring months had the second highest precipitation totals between 1980 and 2000.

Only the monitoring wells were present at the site during 1998, and as expected, they were the highest measured water levels in those wells (Table 4). The water levels collected in the monitoring wells on July 27, 1998 were on the average 2.11 feet higher than their highest measurements during 2000. Therefore, 2.11 feet was added to the maximum value of each of the new piezometers to establish estimated values for 1998. An estimated long-term seasonal high water table elevation contour map (Figure 8) was prepared using the real data from the monitoring wells that were present in 1998, and the estimated values in the piezometers.

Conservatively, Figure 8 can be used for landfill subgrade design. A four-foot vertical buffer should be maintained between the estimated long-term seasonal high water levels and the bottom elevation of the solid waste. HHNT will prepare and submit a base liner system design under separate cover.

3.6.3 Water Table Elevation

Ground-water level elevations were measured in the piezometers on site at the time of boring, after 24 hours, and between January 2000 and May 2001. Additionally, water level measurements from the existing monitoring wells were obtained from July 1998 to July 1999. Table 4 provides a summary of the water level measurements collected. A water-table surface contour map was prepared for the February 14, 2000 data (Figure 7); the depth to ground water varied across the site from about 3 feet below ground surface (PZ-6b) in low elevation areas to about 46 feet below ground surface (MW-1A) in high elevation areas.

Recent drought conditions during 1999 and 2000 have rendered water levels at the site lower than average. However, ground water is still above the bedrock surface over the majority of the site, with the exception of the area near PZ-3 (central portion of Phase 2). Therefore, the majority of the uppermost aquifer system is in the soil mantle above the bedrock.

3.6.4 Ground-Water Flow Direction and Flow Pathways

Generally, ground-water flows to the south and southeast beneath the site. However, beneath the existing landfill, flow is radial around the upland areas with a similar configuration as the topography and auger refusal. The higher elevations located in the central and northern portion of the site serve as recharge areas and influence the ground-water flow directions. Flow is convergent towards the central drainage feature, then flows southward to Buffalo Creek. Ground water flow is through the soil matrix, the weathered fracture openings in the saprolite, and the bedrock fractures.

3.6.4.1 Existing Flow Pathways

Ground-water flow in the uppermost aquifer occurs through the saprolite, partially weathered rock, and upper fractured bedrock towards the south and southeast. Hydraulically, these units exhibit similar hydraulic conductivity values (Table 9). Over most of the site, the water table is in the saprolitic soils. Ground-water discharges along the seasonal drainage feature between the existing landfill and Phase 2 and to Buffalo Creek.

Across the majority of the site, the water table is in the soil mantle above the bedrock; however, in the central portion of Phase 2 the water table is below the bedrock surface and ground-water flow is through fractures. The bedrock on site has been described as severely to slightly weathered biotite-quartz-feldspar gneiss (average RQD of 48 percent), with very close to widely spaced, horizontal to shallow dipping foliation and fractures. Therefore, the bedrock regime is characterized as an equivalent porous media. In general, ground-water flow is parallel to the prevailing fracture trend of northwest-southeast.

3.6.4.2 Potential Flow Pathways

A potential pathway for ground-water flow is the contact between major geologic formations. The subject site is located on an area that has been mapped by others and by BLE as gneiss and schist of the Inner Piedmont Belt, west of a contact with the Cherryville Granite. The Cherryville Granite is a weakly foliated medium-grained micaceous monzogranite, and occurs next to more abundant biotite gneiss and schist of the Inner Piedmont (Horton and Zullo, 1991). The gneiss and schist are characterized by strong foliation and abundant biotite, feldspar and quartz. Geologic contacts between different Paleozoic-age formations in the Inner Piedmont Belt have themselves been re-worked (i.e, recrystallized, folded) by metamorphic events during the Paleozoic Era and their locations are typically inferred.

According to Brown and others (1985), the geologic contact between the metamorphic rocks and the Cherryville Granite roughly parallels Buffalo Creek on the southeast side of the site. The closest boring to Buffalo Creek is PZ-4c (200 feet), which is in a lowland location and drilled to a 50-foot depth (lowest drilled elevation on site, 568 feet msl). PZ-4c did not encounter the Cherryville Granite. The contact is most likely moderately to steeply dipping, based on the relatively straight contact trace provided by Horton and Zullo (1991) and Goldsmith and others (1988). Hypothetically, if the contact between the two geologic formations aligned with the trend of Buffalo Creek and the bottom elevation of PZ-4c, the contact would roughly dip below the site at an angle greater than 11°. Therefore, the contact would have to be 100 to over 600 feet below ground surface over the majority of the site.

Based on the above information, the contact between the gneiss and schist of the Inner Piedmont Belt and the Cherryville Granite will not influence the uppermost aquifer ground-water flow in the residual soils and upper fractured bedrock in the vicinity of proposed cell construction on site.

3.6.5 Man-made Influences to Ground-Water Levels

The existing landfill is being developed in the upland area west of the proposed Phase 2 area. As cell construction proceeds to Phase 2, ground water infiltration and recharge of the water table will be somewhat limited due to the final compacted soil cap of relatively low permeability soil (i.e., 10^{-5} cm/sec), plus surface water will be directed off of the cell tops via drainage berms and piping in the existing and proposed landfill area. Consequently, the water table will be lowered near the proposed cells. Additionally, ground water in the seasonal discharge area between the existing landfill and Phase 2 may dry up after future cell development takes place in the upgradient recharge area.

3.6.6 Hydraulic Coefficients and Ground-Water Flow Velocity

3.6.6.1 Hydraulic Conductivity

Hydraulic conductivity is defined as the ability of the aquifer material to conduct water under a hydraulic gradient. Five slug tests were performed in the study area during January 2000 to measure the *in situ* hydraulic conductivity of the different zones of the water-table aquifer. The slug test results were evaluated using the Bouwer and Rice Method for partially-penetrating wells in an unconfined aquifer.

Three slug test were performed in piezometers installed in the residual soil zone (PZ-2ab, PZ-4ab, and PZ-7b); and two slug tests were performed in the open bedrock piezometers (PZ-1c and PZ-4c). The results of the tests are provided in Appendix G and summarized on Table 5. The hydraulic conductivity in the residual soil zone ranged from 5.5 x 10^{-4} cm/sec in piezometer PZ-4ab to 3.8 x 10^{-5} cm/sec in piezometer PZ-7b. Hydraulic conductivity values in the fractured bedrock zone ranged from 7.6 x 10^{-4} cm/sec in piezometer PZ-1c to 7.0 x 10^{-4} cm/sec in piezometer PZ-4c.

3.6.6.2 Hydraulic Gradient

The hydraulic gradient is determined by dividing the difference in ground-water elevations at two locations by the horizontal distance between those locations along the direction of ground-water flow. The steepest hydraulic gradient at the site is about 0.11, which is located in the northern area near MW-1 and PZ-1ab/PZ-1c. The shallowest gradient at the site is about 0.035, which is located in the southern area near MW-4 and MW-5.

3.6.6.3 Effective Porosity and Specific Yield

Effective porosity is the volume of void spaces through which water or other fluids can travel in a rock or sediment divided by the total volume of the rock or sediment. Effective porosity can be assumed to be approximately equal to specific yield for unconfined (water-table) aquifers. Specific yield is defined as the ratio of the volume of water that drains from a saturated rock owing to the attraction of gravity to the total volume of rock.

Specific yield measurements in the study area within the water bearing zone range from about 19 to 30 percent in the micaceous silty sands. The effective porosity can be expected to range from about 5 to 10 percent for fractured crystalline bedrock (Kruseman and deRidder, 1989).

3.6.6.4 Ground-Water Flow Velocity

The velocity of ground-water movement (V) is a function of existing hydraulic gradient (i), the hydraulic conductivity (K) and the effective porosity (n), in the equation $V = Ki/n$.

Based on these parameters and the data provided above, the horizontal movement of ground-water ranges from approximately 0.34 feet/day across the site. Table 6 summarizes the ground-water flow velocity calculations.

3.6.7 Vertical Flow Gradients

Vertical flow gradients were evaluated at the site by installing piezometer pairs. There are two vertical well pairs in the Phase 2 area: PZ-1ab/PZ-1c and PZ-4ab/PZ-4c. There are two vertical well pairs on the existing landfill site: MW-4/PZ-6b and MW-7/PZ-7b. The vector of vertical ground-water flow has not changed during the timeframe they have been recorded from the four well pairs. However, the gradient magnitude has varied in each of the well pairs as follows:

PIEZOMETER PAIR	SITE LOCATION DESCRIPTION	RECHARGE GRADIENT (FT/FT)	DISCHARGE GRADIENT (FT/FT)	NEARLY FLAT GRADIENT (FT/FT)
PZ-1ab/PZ-1c	Upper end of drainage feature between existing landfill and Phase 2	0.17 – 0.046		
PZ-4ab/PZ-4c	Southern portion of Phase 2 area near Buffalo Creek			0.014 – 0.0011 (recharge)
MW-4/PZ-6b	Lower end of drainage feature between existing landfill and Phase 2		0.12 – 0.091	
MW-7/PZ-7b	South of existing landfill			0.088 – 0.0097 (recharge)

Based on the site topography, the vertical gradients observed in the study area are typical for unconfined aquifers in the Piedmont. Ground-water recharge occurs in the upland areas. Ground-water discharge occurs to the drainage feature between the existing landfill and Phase 2, and to

Buffalo Creek on the southern site boundary. Table 7 summarizes the vertical gradient calculations.

3.6.8 Ground-Water Quality at the Existing Landfill

Low concentrations (well below North Carolina 15A NCAC 2L standards) of volatile organic compounds (VOCs) have been detected in monitoring wells MW-5 (chlorobenzene and 1,2-dichlorobenzene) and MW-7 (cis 1,2-dichloroethene) during recent semi-annual sampling events. The concentrations of VOCs detected in MW-5 are not statistically significant increases (SSIs), but cis 1,2-dichloroethene in MW-7 has been calculated as a SSI. The source of cis 1,2-dichloroethene detected in well MW-7 is uncertain at this time. Landfill leachate is a possible source; however, leachate typically consists of an assemblage of several VOCs rather than an individual VOC. Unconfirmed information indicate the possible source of the cis 1,2-dichloroethene is inadvertent well contamination during installation (i.e., drilling immediately adjacent to or through trash during installation), or possibly landfill gas migration. MW-7 has been replaced, as indicated in Section 4.1.3 of this report.

If corrective action were to be required in the future, the most likely locations would be in the vicinity of the existing landfill near wells MW-5 and MW-7. However, the need for engineered systems or other intrusive corrective actions would be unlikely. Based on the low concentrations of VOCs detected, natural attenuation via advection, dispersion, adsorption, biodegradation, and biotransformation should render the contaminants non-detectable before reaching downgradient receptors. In the unlikely event that intrusive corrective measures would be needed, the affected areas would not be in the vicinity of Phase 2 and would not limit future landfill construction.

3.6.9 Ground-Water Flow Regime near the Centrally Located Drainage Feature

The evaluation of the ground-water flow regime of the drainage feature between the existing landfill and the proposed Phase 2 area included:

1. Installing a piezometer-pair in an upgradient location (PZ-1ab/PZ-1c) to measure vertical hydraulic gradients;
2. Installing a piezometer-pair in a downgradient location (MW-4/PZ-6b) to measure vertical hydraulic gradients;
3. Hydraulic testing of piezometers across the site;
4. Performing a fracture trace analysis; and
5. Preparing water table contour maps.

The drainage feature is a seasonal (intermittent) stream. The year-round water table occurs within the saprolite; however, bedrock is relatively shallow in the upper end of the feature and deeper in the lower end. A shallow body of standing water was formerly located along the feature between MW-4 and MW-5, but has since been drained.

The north-south trend of the drainage feature does not parallel primary or secondary fracture trends, according to the fracture trace analysis performed by BLE. Therefore, the feature does not appear to be controlled by geologic structure.

The upper end of the drainage feature has a recharge gradient and the lower end of the drainage feature has a discharge gradient. Water table maps prepared for the area indicate convergent ground-water flow towards the drainage feature. Potential contamination from the existing landfill should flow south towards Buffalo Creek. Effective ground-water monitoring would include monitoring wells along the south side of the existing landfill and Phase 2.

3.7 GEOTECHNICAL CONSIDERATIONS

An evaluation of the potential impact from faults, seismic zones and unstable areas, as required by 15A NCAC13B.1622 is briefly presented below to provide a background for the geotechnical evaluation. Geotechnical related topics evaluated for the Phase 2 area include the stability of the planned cut and fill slopes, stability of the base liner system, and settlement of the subgrade and fill soils resulting from the planned waste placement. Construction considerations include surface water control, excavation, site subgrade preparation, and engineered fill placement. Each of these topics is reviewed in the following paragraphs.

3.7.1 Fault Areas

No Holocene faults are located within 200 feet of the subject site (Horton and Zullo, 1991).

3.7.2 Seismic Impact Zones

According to the definition of seismic impact zones in 15A NCAC 13B .1622 (5), this site is in a seismic impact zone. The maximum horizontal acceleration expressed as a percentage of the earth's gravity (g), in rock is about 0.14g with a 10 percent probability of being exceeded in 250 years (Algermissen, and others, 1990; partial reproduction attached as Figure 9). The landfill should be designed to resist the maximum horizontal acceleration in lithified earth material at the site. This magnitude of bedrock acceleration should not present any unusual design constraints and conventional design slopes will be appropriate.

3.7.3 Unstable Areas

An unstable area according to 15A NCAC 13B.1622 (6) is defined as a location that is susceptible to natural or human induced events or forces capable of impairing the integrity of some or all of the landfill structural components responsible for preventing releases from a landfill. Unstable areas could include poor foundation conditions, areas susceptible to mass movements, and karst terrains. Site and subsurface data obtained were evaluated to determine if unstable site areas exist. Settlement and slope stability were evaluated utilizing data obtained from soil test borings, the test pits, and from field observations. The results and conclusions of the evaluation are included below.

3.7.3.1 Subgrade Settlement

Site grading plans for construction of the landfill cells have not yet been prepared; however, we anticipate a combination of earthwork cut and fill will be made to establish the cell areas. Foundation support conditions for the landfill liner system will consist of either: 1) dense residual soils overlying partially weathered rock at shallow depths, 2) loose to dense residual soils with thicknesses of up to 40 feet over weathered rock, or 3) engineered fill with thicknesses of up to 15 feet to 20 feet overlying residual soils. Soil elastic modulus values for settlement analyses were based on previously developed correlations with standard penetration resistance values in similar soils. To simplify the analyses, we assumed the stress increase within the residual soil layer was equal to the full surcharge pressure of the refuse mound. The surcharge pressures were estimated based on an assumed unit weight of 60 pounds per cubic foot (pcf) of stored waste.

The rock and partially weathered rock underlying the site are relatively incompressible and will not realize appreciable settlements under the anticipated landfill loading. The residual soils are typically firm to very firm sandy clayey silts grading coarser with depth into dense silty sands with some gravel. Modest settlements will be realized from compression of the upper zones of residual soils and the anticipated fills. The subgrade settlement at a given location and differential settlements realized will be a function of the actual refuse and structural fill heights at a given point and the corresponding foundation materials. Maximum settlements on the order of 0.5 feet or less could be expected when placing the full height of refuse over the maximum height of structural fill and deepest thickness of residual soil. This situation could occur in the vicinity of borings PZ-2ab and PZ-3. Correspondingly, subsurface soil settlements due to even the full height of the landfill would result in insignificant soil settlements when bearing on dense residual soils overlying rock of a shallow depth.

Settlement near the edge of the landfill should be minimal. Residual soil settlement should occur rapidly as the cells are filled. Total and differential settlements are expected to be well within acceptable limits of the structural components at a municipal solid waste landfill and leachate collection system.

3.7.3.2 Slope Stability

The soil test borings and laboratory test results indicate that the on site residual soils may be used for construction of earthwork cut and engineered fill slopes. Slope angles of 2.5 horizontal to 1 vertical or flatter are acceptable in the construction of the landfill cells and cut and fill slopes are appropriate. The existing natural slope areas observed by a geologist showed no signs of slope instability.

3.7.3.3 Conclusion

Our settlement and slope stability evaluation did not indicate areas of potential mass movement exist. This site is not karst and is not subject to sinkhole activity or caves. Based on the above considerations, it is our opinion that this site is stable; no unstable areas were identified at the site.

3.7.4 Excavation

Excavation of the residual soils can be accomplished using conventional earth moving equipment. An estimated top of rock (auger refusal) contour map was developed as Figure 6 which is based on auger refusal depths in the soil borings drilled at this site. Materials sufficiently hard to cause refusal to the mechanical drill augers may result from continuous bedrock, boulders, lenses, ledges, or layers of relatively hard rock. Coring was performed at two locations (PZ-1c and PZ-4c) where refusal to augering occurred. Continuous rock was found with varying recovery and RQD as discussed above in Section 3.5.1.3. Due to its typically varying surface, the actual occurrence of hard rock during site grading may vary somewhat from that presented in Figure 6.

3.7.5 Permeability of Potential On-Site Soils for Clay Liner and Cover Construction

The permeability of selected potential on-site borrow soils were determined as indicated in Section 3.5.3 titled Laboratory Testing Results and compared with prior site laboratory test results presented by CDM. The samples were generally compacted to 95 percent of the standard Proctor maximum dry density at 3-5 percent over optimum moisture content. Hydraulic conductivities of 1.4×10^{-7} to 7.3×10^{-8} cm/sec were obtained for the selected samples (CH, MH, and ML).

Residual soils consisting of red-brown silty clay (CH) and sandy clayey silt (ML and MH) were found to depths of 3 to 5.5 feet below ground surface. These soils would readily achieve a remolded hydraulic conductivity (permeability, k) $\leq 1.0 \times 10^{-5}$ cm/sec acceptable for use as a soil base liner or low permeability soil cap. Soils capable of achieving a permeability $k \leq 1.0 \times 10^{-7}$ cm/sec were found in limited quantity. The remaining residual soils are acceptable for use as structural fill for embankments and final or daily cover.

-oOo-

4.0 MODIFICATION TO THE WATER QUALITY MONITORING SYSTEM FOR EXISTING LANDFILL

As an addition to this Site Hydrogeologic Report, modifications to the monitoring plan for the existing site are submitted herein. These revisions to the existing monitoring system are needed to better address requirements in Rule 15A NCAC 13B .0503. Currently, nine ground-water monitoring wells are present at the existing landfill as follows:

- Background: MW-1A;
- Compliance: MW-4, MW-5, MW-6, MW-7, and MW-8; and
- Unused wells: MW-1, MW-2, and MW-3.

Additionally, two surface water sampling locations are used:

- Upgradient: SW-3; and
- Downgradient: SW-4.

The five compliance wells are located about 0 to 100 feet from the waste boundary. The background well is located about 200 feet from the waste boundary. The monitoring wells located adjacent to the edge of the waste can capture only a narrow segment of the ground water flow regime. Therefore, the compliance well locations should be moved further downgradient (100 to 150 feet from the waste boundary) to better use the effects of dispersivity, which will result in a larger monitoring area for each well. Additionally, areas of convergent ground-water flow should be targeted.

4.1 GROUND-WATER MONITORING PLAN

4.1.1 Subsurface Considerations

Site specific factors were considered in redesigning this ground-water detection monitoring system, including the locations and construction details of each proposed monitoring well. In addition, environmental factors were considered, such as seasonal variations of the water table, the horizontal and vertical flow regimes, and lithology characteristics.

The residual soils and bedrock comprise the unsaturated and saturated zones of the uppermost water table aquifer.

4.1.2 Compliance Boundary

The compliance boundary is less than 250 feet from the boundary of the existing waste boundary. This compliance boundary is also more than 50 feet from the facility property boundary.

4.1.3 Monitoring Well Locations

Existing background monitoring well MW-1A is located on the west side of Roseborough Road, which is not located between the landfill and the closest upgradient residence. Therefore, a new upgradient location has been selected, which will consist of a well-pair: MW-1B and MW-1C. The proposed well pair location is approximately 95 feet from the waste boundary (Figure 4). MW-1B will be screened at the first occurrence of ground water in the residual soil. MW-1C will be screened at a similar depth as the nearby private drinking water wells, which is approximately 20-30 feet below the water table in the residual soil. Unused well MW-1 is located at the waste boundary and is therefore unusable.

Existing compliance monitoring wells MW-4 and MW-5 are in locations of convergent ground-water flow approximately 35 and 100 feet from the waste boundary, respectively. The southward flowing drainage feature in the center of the site restricts their distance from the waste boundary.

Existing compliance monitoring well MW-8 is in a side-gradient location on the west side of the landfill. This well is located about 30 feet from the waste boundary between the landfill and two residences with private drinking water wells, which are located on the west side of Roseborough Road. The property boundary and Roseborough Road restrict the distance of MW-8 from the waste boundary. Although MW-8 is located closer than 150 feet from the limits of waste and closer than 50 feet from the property boundary, the well should remain part of the monitoring system since it is located between the landfill and the residences using private drinking water wells.

Existing compliance monitoring wells MW-6 and MW-7 are located on the south side of the landfill approximately 50 and 0 feet from the waste boundary, respectively. The area of the landfill being monitored by these wells is not restricted by property boundaries or ground water divides. Therefore, two new replacement wells (MW-6A and MW-7A) should be installed further downgradient of the existing wells to better use the effects of dispersivity within the relevant zone of compliance. The new locations were selected based on the direction of ground-water flow shown on Figure 7. The shallow ditch located near wells MW-6 and MW-7 is a man-made feature between the landfill and residual soils south of the landfill and is not structurally controlled by geology. Furthermore, the water table is about 20 feet below ground surface and the shallow ditch has minimal, if any influence over the ground-water flow to the southeast (Figure 7). Monitoring wells MW-6 and MW-7 should be abandoned.

4.1.4 Monitoring Well Depths and Screened Intervals

The depth of the proposed monitoring wells (MW-1B, MW-6A and MW-7A) will be designed to monitor the uppermost aquifer present at the site. The wells will be constructed with 15-foot long screened intervals. The proposed well depths will be determined by either:

- the depth to ground water in the soil and partially weathered rock units, if a sufficient saturated thickness of the aquifer exists above the depth of auger refusal. The screened interval will be set to bracket the water table surface; or
- by the depth of water-bearing fractures in the bedrock unit. The screened interval will be set to intersect the water-bearing fractures.

The depth of MW-1C will be designed to be at a similar depth as private drinking water wells in the vicinity of the landfill. Area private wells are typically dug wells to a depth of about 20 to 30 feet below the water table. Therefore, MW-1C should be installed to approximately 25 feet below the water table (or to the depth of auger refusal if the 25-foot water column is not attainable)

The proposed depths for wells MW-1B, MW-1C, MW-6A and MW-7A are based on the subsurface geology and water table elevations encountered in the nearby borings (Table 8). The actual well depths may be adjusted during well installation based on field conditions (i.e., depth to water, depth to bedrock). The anticipated well depth for MW-1B is 50 feet, MW-1C is 68 feet, MW-6A is 33 feet, and for MW-7A is 40 feet below ground surface.

4.1.5 Proposed Monitoring Well Construction

The anticipated lithology at the new proposed well locations has been estimated based on the closest available boring/coring data and from cross-sections and plan view geologic maps. The proposed monitoring well construction details are presented on Table 8.

It is proposed that each of the new wells be constructed of 2-inch diameter PVC casing and 10 to 15-foot long screened interval, with a sand pack, bentonite seal and grout column in the annular space between the borehole and PVC casing. A lockable standup steel cover should be secured over each well along with a concrete pad at the cover's base.

4.2 SURFACE WATER MONITORING PLAN

There are two existing surface water sampling locations associated with landfill. These two locations are sufficient to monitor the upgradient (SW-3) and downgradient (SW-4) surface water for the site.

4.3 WATER QUALITY MONITORING SYSTEM

The revised water quality monitoring system for the existing landfill is shown on Figure 4. Once constructed, the monitoring system will include seven ground-water monitoring wells and two surface water sampling locations. The water quality monitoring system for the site will include:

- two upgradient monitoring well (MW-1B and MW-1C);
- five downgradient monitoring wells (MW-4, MW-5, MW-6A, MW-7A, and MW-8,);
- one upgradient surface water location (SW-3); and
- one downgradient surface water location (SW-4).

The remaining six unused monitoring wells (MW-1A, MW-1, MW-2, MW-3, MW-6, and MW-7) should be abandoned in accordance with 15A NCAC 2C, Rule .0113(a)(2) by drilling them out and filling the resulting boreholes with a grout mixture of cement, bentonite, and water.

-oOo-

5.0 CONCLUSIONS

The purpose of this evaluation was to provide supplemental hydrogeologic information for the existing landfill site, and to determine if the Phase 2 area meets North Carolina Department of Solid Waste suitability criteria for site permitting of an industrial waste landfill. The scope of the investigation and the criteria for site suitability are defined by 15A NCAC 13B, Rule .0503(2)(d) and .0504(1)(c).

Existing Landfill Area
Hydraulic gradients at the base of the existing landfill were measured to be nearly flat (MW-7 area) and discharging (MW-4 area).

Existing background monitoring well MW-1A is located on the west side of Roseborough Road, which is not located between the landfill and the closest upgradient residence. Therefore, a new upgradient location has been selected, which will consist of a well-pair: MW-1B and MW-1C. The proposed well pair location is approximately 95 feet from the waste boundary. MW-1B will be screened at the first occurrence of ground water in the residual soil. MW-1C will be screened at a similar depth as the nearby private drinking water wells, which is approximately 20-30 feet below the water table in the residual soil.

Existing monitoring wells MW-6 and MW-7 are currently located about 50 to 0 feet from the waste boundary, respectively. Since the general monitoring areas of these two wells are not restricted by property boundaries or hydraulic divides, they should be abandoned and replaced downgradient. The replacement monitoring wells (MW-6A and MW-7A) should be installed approximately 150 feet from the waste boundary. By adding these wells to the monitoring network, dispersion of contaminants downgradient from the landfill could be observed. This will result in a more effective monitoring system.

Phase 2 Area
No Holocene-age, or younger, faults or unstable areas were identified on site. The site is in a seismic impact zone and the design should include consideration of a maximum horizontal acceleration (g) in the bedrock of 0.14, however, conventional landfill design slopes and structural components should be appropriate.

An unconfined water table aquifer underlies the Phase 2 area. A detection monitoring system can be designed based on the characteristics of the site aquifer. Ground-water monitoring would be effective in areas of convergent ground-water flow such as near the linear drainage features.

Based on the results of field and laboratory testing, it is our professional opinion that the study area meets the minimum standards required for industrial waste landfill development.

-oOo-

6.0 ANNOTATED BIBLIOGRAPHY

Algermissen, S.T., Perkins, D.M., Thenhaus, P.C., Hanson, S.L., and Bender, B.L., 1990, *Probabilistic Earthquake Acceleration and Velocity Maps for the United States and Puerto Rico*: USGS Miscellaneous Field Studies Map MF-2120.
- Seismic impact zones.

Bouwer, H. and Rice, R.C., 1976, "A slug test method for determining hydraulic conductivity of unconfined aquifers with completely or partially penetrating wells," Water Resources Research, Vol. 12, No. 3, pp. 423-428.
- Slug testing data reduction procedures.

Brown, M.B., and others (compilers), 1985, *Geologic Map of North Carolina*: NCDNRCD, scale 1:500,000.
- General geologic setting, and fault locations.

Fetter, C.W., 1988, *Applied Hydrogeology*: Merrill Publishing Company, Columbus, Ohio.
- Basic hydrogeologic principles and estimates of effective porosity in soils.

Gair, J.E., 1989, *Mineral Resources of the Charlotte 1°x2° Quadrangle, North Carolina and South Carolina*: USGS Professional Paper 1462.
- Geologic Setting

Goldsmith, R.M., Milton, D.J., and Horton, J.W., 1988, *Geologic map of the Charlotte 1°x2° Quadrangle, North Carolina and South Carolina*: USGS Miscellaneous Investigation Map I-1251-E, 1:250,000.
- Geologic Setting

Horton, J.W. and Zullo, V.A., 1991, *The Geology of the Carolinas: Carolina Geological Society fifteenth anniversary volume*: The University of Tennessee Press, Knoxville, TN.
- General geologic setting.

Kruseman, G.P., and deRidder, N.A., 1989, *Analysis and Evaluation of Pumping Test Data*: *Publication 47*, International Institute for Land Reclamation and Improvement, Wageningen, The Netherlands.
- Estimates of effective porosity in fractured bedrock.

U.S. Department of Agriculture (USDA), unpublished, Soil Survey of Cleveland County, North Carolina. Cleveland County Soil Conservation Service.
- Soil survey of the area around the site.

-oOo-

Attachment B

Revised Tables of the SHR

TABLE 2 - REVISED JULY 2001

PIEZOMETER CONSTRUCTION DETAILS
JMN/Cleveland Container Industrial Landfill
Cleveland County, NC
BLE Project Number J99-1307-04

Piezometers & Monitoring Wells	Ground Elev.	TOC Elev.	Auger Refusal Depth	Auger Refusal Elev.	Bedrock Drilling Depth	Screened Interval Depth	Screened Interval Elvation
MW-1	773.99	777.14	Info Not Available	---	---	39.0 - 54.0	735.0 - 720.0
MW-1A	777.28	780.33	Not Reported	---	---	35.0 - 50.0	742.3 - 727.3
MW-2	606.15	607.76	Info Not Available	---	---	4.0 - 19.0	602.2 - 587.2
MW-3	605.96	607.75	Info Not Available	---	---	5.0 - 15.0	601.0 - 591.0
MW-4	630.13	632.83	Not Reported	---	---	4.5 - 14.5	625.6 - 615.6
MW-5	622.45	625.32	Not Reported	---	---	6.0 - 21.0	616.5 - 601.5
MW-6	636.27	639.29	Not Reported	---	---	12.0 - 27.0	624.3 - 609.3
MW-7	657.19	660.09	Not Reported	---	---	13.0 - 28.0	644.2 - 629.2
MW-8	736.21	739.39	Not Reported	---	---	35.0 - 50.0	701.2 - 686.2
PZ-1ab	677.48	680.14	17.5	660.0	---	12.2 - 17.2	665.3 - 660.3
PZ-1c	677.83	679.54	29.0	648.8	29.0 - 45.5	29.0 - 45.5	648.8 - 632.3
PZ-2ab	648.93	651.37	39.0	609.9	---	28.7 - 38.7	620.2 - 610.2
PZ-3	669.21	670.53	45.5	623.7	---	33.7 - 43.7	635.5 - 625.5
PZ-4ab	617.68	620.13	25.5	592.2	---	15.2 - 25.2	602.5 - 592.5
PZ-4c	618.15	619.48	30.0	588.2	30.0 - 50.0	30.0 - 50.0	588.2 - 568.2
PZ-5ab	646.50	648.44	38.0	608.5	---	27.7 - 37.7	618.8 - 608.8
PZ-6b	630.01	632.79	30.0	600.0	---	24.7 - 29.7	605.3 - 600.3
PZ-7b	656.96	659.86	52.0	605.0	---	41.7 - 51.7	615.3 - 605.3

NOTES:
1. Measurements are in feet; elevations are relative to mean sea level
2. TOC = Top of Casing
3. Surveying was performed by Wright & Fields (RLS) of Troy, NC
4. PZ-1c and PZ-4c are open bedrock piezometers with no well screen.

TABLE 3 - REVISED JULY 2001

SUMMARY OF LABORATORY RESULTS
JMN/Cleveland Container Industrial Landfill
Cleveland County, NC
BLE Project Number J99-1307-04

Boring	Split-Spoon Depth (ft)	Shelby Tube Depth (ft)	Bag Sample Depth (ft)	Unit	Nat. Moisture Content (%)	Standard Proctor Opt. Moisture Content (%)	Standard Proctor Max. Dry Density (pcf)	Hydraulic Conductivity (cm/sec) Remolded	Hydraulic Conductivity (cm/sec) In-Situ	Total Unit Weight (pcf)	Porosity (%) Effective	Porosity (%) Total	Atterberg Limits LL	Atterberg Limits PL	Atterberg Limits PI	Grain Size (% by wt) Gravel	Grain Size (% by wt) Sand	Grain Size (% by wt) Silt	Grain Size (% by wt) Clay	% Pass 200 Sieve	USCS
PZ-1c	23.5 - 25.0	-	-	saprolite	44.4%	-	-	-	-	-	19%	-	-	-	-	0.0%	61.8%	27.2%	11.0%	38.2%	SM
PZ-2ab	1.0 - 2.5	-	-	saprolite	-	-	-	-	-	-	-	-	51	26	25	-	-	-	-	-	CH
PZ-2ab	3.5 - 5.0	-	-	saprolite	-	-	-	-	-	-	-	-	59	33	26	-	-	-	-	-	MH
PZ-2ab	28.5 - 30.0	-	-	saprolite	35.1%	-	-	-	-	-	24%	-	-	-	-	0.0%	69.0%	24.0%	7.0%	31.0%	SM
PZ-3	-	-	1.0 - 5.0	saprolite	33.7%	21.0%	100.5	7.3E-08	-	-	3.5%	-	51	33	18	0.0%	40.4%	24.5%	35.1%	59.6%	MH
PZ-3	-	23.0 - 25.0	-	saprolite	10.1%	-	-	-	3.5E-04	100.6	25%	46.5%	-	-	-	2.0%	73.2%	18.2%	6.6%	24.8%	SM
PZ-4ab	18.5 - 20.0	-	-	PWR	13.8%	-	-	-	-	-	30%	-	-	-	-	0.8%	84.2%	11.9%	3.1%	15.0%	SM
PZ-5ab	1.0 - 2.5	-	-	saprolite	31.0%	-	-	-	-	-	-	-	59	46	13	-	-	-	-	60.3%	MH

NOTES:
Moisture Content % = (Weight of water/Weight of soil) * 100
Effective Porosity (specific yield) is based on grain size analyses and Figure 4.11 (Fetter, 1988)
pcf = pounds per cubic foot
USCS = Unified Soil Classification System

TABLE 4 - REVISED JULY 2001

GROUND-WATER ELEVATION MEASUREMENTS
JMN/Cleveland Container Industrial Landfill
Cleveland County, NC
BLE Project Number J99-1307-04

Wells	Ground Elev.	TOC Elev.	DGW TOB	Water Elev.	DGW 24-HR	Water Elev.	DGW 7/27/98	Water Elev.	DGW 1/25/99	Water Elev.	DGW 7/20/99	Water Elev.	DGW 1/11/00	Water Elev.	DGW 1/19/00	Water Elev.	DGW 2/14/00	Water Elev.	DGW 3/28/00	Water Elev.	DGW 5/2/00	Water Elev.	DGW 6/9/00	Water Elev.	DGW 7/14/00	Water Elev.	DGW 5/25/01	Water Elev.	Maximum Water Elev.	Minimum Water Elev.	Difference
MW-1	773.99	777.14	NA		NA		NM		NM		NM		NM		46.30	**727.69**	46.55	727.44	46.85	727.14	46.99	727.00	46.93	727.06	46.90	727.09	NP		727.69	727.00	0.69
MW-1A	777.28	780.33	NA		NA		39.98	**737.30**	42.90	734.38	45.10	732.18	NM		43.70	733.58	43.90	733.38	43.98	733.30	43.65	733.63	43.25	734.03	43.49	733.79	NP		737.30	732.18	5.12
MW-2	606.15	607.76	NA		NA		NM		NM		NM		NM		NM		NM		NM		NM		NM		NM		NP		-	-	-
MW-3	605.96	607.75	NA		NA		NM		NM		NM		NM		NM		NM		NM		NM		NM		NM		NP		-	-	-
MW-4	630.13	632.83	NA		NA		4.82	625.31	5.35	624.78	5.68	624.45	NM		5.21	624.92	5.01	625.12	4.95	625.18	4.92	625.21	5.11	625.02	5.05	625.08	4.75	**625.38**	625.38	624.45	0.93
MW-5	622.45	625.32	NA		NA		9.13	**613.32**	9.49	612.96	10.37	612.08	NM		10.20	612.25	9.76	612.69	9.39	613.06	9.30	613.15	10.13	612.32	10.55	611.90	10.60	611.85	613.32	611.85	1.47
MW-6	636.27	639.29	NA		NA		19.74	**616.53**	21.38	614.89	22.23	614.04	NM		23.57	612.70	21.75	614.52	21.06	615.21	20.58	615.69	21.69	614.58	23.04	613.23	NP		616.53	612.70	3.83
MW-7	657.19	660.09	NA		NA		16.56	**640.63**	18.53	638.66	19.76	637.43	NM		20.28	636.91	20.09	637.10	18.43	638.76	20.25	636.94	20.55	636.64	20.70	636.49	NP		640.63	636.49	4.14
MW-8	736.21	739.39	NA		NA		37.12	**699.09**	40.67	695.54	42.19	694.02	NM		43.47	692.74	43.74	692.47	44.02	692.19	44.03	692.18	43.80	692.41	43.73	692.48	45.78	690.43	699.09	690.43	8.66
PZ-1ab	677.48	680.14	Dry		15.92	661.56	NP		NP		NP		15.92	661.56	16.59	660.89	14.65	662.83	12.94	664.54	12.39	**665.09**	15.49	661.99	16.13	661.35	16.14	661.34	665.09	660.89	4.20
PZ-1c	677.83	679.54	0.0	677.8	17.91	659.92	NP		NP		NP		17.91	659.92	17.98	659.85	17.62	660.21	17.00	660.83	16.50	**661.33**	16.79	661.04	17.41	660.42	17.59	660.24	661.33	659.85	1.48
PZ-2ab	648.93	651.37	35.0	613.9	29.96	618.97	NP		NP		NP		29.96	618.97	29.99	618.94	29.97	618.96	29.77	619.16	29.29	619.64	29.28	**619.65**	29.79	619.14	30.51	618.42	619.65	618.42	1.23
PZ-3	669.21	670.53	Dry		Dry		NP		NP		NP		>43.98	<625.23	>43.98	<625.23	>43.98	<625.23	>43.98	<625.23	>43.98	<625.23	>43.98	<625.23	>43.98	<625.23	>43.98	<625.23		Dry	
PZ-4ab	617.68	620.13	18.0	599.7	16.49	601.19	NP		NP		NP		16.49	601.19	16.54	601.14	15.84	601.84	15.77	601.91	15.75	**601.93**	18.30	599.38	18.35	599.33	18.82	598.86	601.93	598.86	3.07
PZ-4c	618.15	619.48	0.0	618.2	17.16	600.99	NP		NP		NP		17.16	600.99	17.21	600.94	16.58	601.57	16.44	601.71	16.40	**601.75**	18.80	599.35	18.96	599.19	19.31	598.84	601.75	598.84	2.91
PZ-5ab	646.50	648.44	36.0	610.5	30.56	615.94	NP		NP		NP		30.56	615.94	30.56	615.94	30.52	615.98	30.34	616.16	30.10	616.40	30.02	**616.48**	30.31	616.19	30.92	615.58	616.48	615.58	0.90
PZ-6b	630.01	632.79	0.0	630.0	3.27	626.74	NP		NP		NP		NM		3.27	626.74	2.95	627.06	2.70	627.31	2.64	**627.37**	3.10	626.91	3.24	626.77	3.02	626.99	627.37	626.74	0.63
PZ-7b	656.96	659.86	0.0	657.0	20.94	636.02	NP		NP		NP		20.94	636.02	20.71	636.25	20.50	636.46	20.28	**636.68**	20.32	636.64	20.54	636.42	20.69	636.27	22.00	634.96	636.68	634.96	1.72

	DGW 7/27/98	DGW 1/25/99	DGW 7/20/99	DGW 1/11/00	DGW 1/19/00	DGW 2/14/00	DGW 3/28/00	DGW 5/2/00	DGW 6/9/00	DGW 7/14/00	DGW 5/25/01
Average Depth to Water	21.23	23.05	24.22	21.28	23.04	22.63	22.26	22.21	22.92	23.22	19.95
Maximum Depths to Water	39.98	42.90	45.10	30.56	46.30	46.55	46.85	46.99	46.93	46.90	45.78
Minimum Depths to Water	4.82	5.35	5.68	15.92	3.27	2.95	2.70	2.64	3.10	3.24	3.02

	Difference
Average	2.73
Maximum	8.66
Minimum	0.63

NOTES:

Measurements are in feet; elevations are relative to mean sea level
TOB = Time of Boring
DGW = Depth to Ground Water below ground surface
TOC = Top of Casing
NA = Not Available
NM = Not Measured
NP = Not Present at the time of measurement
Bold elevations represent highest observed elevations between 7/27/98 to 6/13/01.

TABLE 5 - REVISED JULY 2001

SUMMARY OF IN-SITU HYDRAULIC CONDUCTIVITY TESTING - SLUG TEST RESULTS
JMN/Cleveland Container Industrial Landfill
Cleveland County, NC
BLE Project Number J99-1307-04

Well	Method	Data Type	Aquifer Unit	K(ft/min)	K(cm/sec)	K(ft/day)
PZ-1c	Bouwer-Rice	Falling Head	Bedrock	1.5E-03	7.6E-04	2.1E+00
PZ-2ab	Bouwer-Rice	Rising Head	Saprolite	5.9E-04	3.0E-04	8.4E-01
PZ-4ab	Bouwer-Rice	Rising Head	PWR	1.1E-03	5.5E-04	1.5E+00
PZ-4c	Bouwer-Rice	Falling Head	Bedrock	1.4E-03	7.0E-04	2.0E+00
PZ-7b	Bouwer-Rice	Falling Head	PWR	7.4E-05	3.8E-05	1.1E-01
Saproilite Only	Hydraulic Conductivity			5.9E-04	3.0E-04	8.4E-01
PWR Only	Maximum Hydraulic Conductivity			1.1E-03	5.5E-04	1.5E+00
	Median Hydraulic Conductivity			5.7E-04	2.9E-04	8.3E-01
	Mean Hydraulic Conductivity			5.7E-04	2.9E-04	8.3E-01
	Geometric Mean Hydraulic Conductivity			2.8E-04	1.4E-04	4.1E-01
	Minimum Hydraulic Conductivity			7.4E-05	3.8E-05	1.1E-01
Rock Only	Maximum Hydraulic Conductivity			1.5E-03	7.6E-04	2.1E+00
	Median Hydraulic Conductivity			1.4E-03	7.3E-04	2.1E+00
	Mean Hydraulic Conductivity			1.4E-03	7.3E-04	2.1E+00
	Geometric Mean Hydraulic Conductivity			1.4E-03	7.3E-04	2.1E+00
	Minimum Hydraulic Conductivity			1.4E-03	7.0E-04	2.0E+00
All Units	Maximum Hydraulic Conductivity			1.5E-03	7.6E-04	2.1E+00
	Median Hydraulic Conductivity			1.1E-03	5.5E-04	1.5E+00
	Mean Hydraulic Conductivity			9.2E-04	4.7E-04	1.3E+00
	Geometric Mean Hydraulic Conductivity			6.3E-04	3.2E-04	9.0E-01
	Minimum Hydraulic Conductivity			7.4E-05	3.8E-05	1.1E-01

NOTES:

K = Hydraulic Conductivity

The data was reduced and the hydraulic conductivities calculated using SuperSlug Version 3.0.

Prepared By: MSP
Checked By: AWA

TABLE 6 - REVISED JULY 2001

INTERSTITIAL GROUND-WATER FLOW VELOCITY CALCULATIONS
JMN/Cleveland Container Industrial Landfill
Cleveland County, NC
BLE Project Number J99-1307-04

	Geometric Mean Values			
Geologic Unit	**Hydraulic Conductivity (K) (feet per day)**	**Hydraulic Gradient (i) (unitless)**	**Effective Porosity (n) (unitless)**	**Ground-Water Velocity (V) (feet per day)**
Saprolite	0.84	0.062	0.22	**0.24**
PWR	0.41	0.062	0.30	**0.08**
Fractured Bedrock	2.1	0.062	0.071	**1.8**
All Units	0.90	0.062	0.17	**0.34**

Notes:
1. Hydraulic conductivity values are from slug test data (Table 5).
2. The flow calculations for "All Units" combines the hydraulic properties of the different units and represents a range of flow velocities across the site.
3. Hydraulic gradient information is from the February 14, 2000 Water Table Contour Map (Figure 7
4. The *high velocity* hydraulic gradient is from the northern area near MW-1 and PZ-1ab/PZ-1c (maximum calculated hydraulic gradient of 0.11).
5. The *low velocity* hydraulic gradient in from the southern area near MW-4 and MW-5 (minimum calculated hydraulic gradient of 0.035).
6. Effective porosity is estimated from specific yield as described by Fetter (1988) and Kruseman and deRidder (1989).

Prepared by: MSP
Checked by: AWA

TABLE 7 - REVISED JULY 2001

VERTICAL HYDRAULIC GRADIENTS AND FLOW RATES
JMN/Cleveland Container Industrial Landfill
Cleveland County, NC
BLE Project Number J99-1307-04

Well Pairs	Ground Elev. (ft)	TOC Elev. (ft)	Ground Elevation Difference (ft)	Horizontal Distance Between Wells (ft)	Midpoint Screen Elev. (ft)	Vertical Separation Between Screen Midpoints (ft)	Water Level Information: Date	Water Level Information: Water Elev. (ft)	Water Level Information: Head Difference (ft)	Vertical Hydraulic Gradient (i)	Geometric Mean: Hydraulic Conductivity (ft/day)	Geometric Mean: Effective Porosity (n)	Vertical Flow Velocity (ft/day)	Flow Direction
PZ-1ab	677.48	680.14	0.35	17	661.56	20.98	2/14/00	662.83	2.62	0.12	1.3	0.12	1.3	Downward
PZ-1c	677.83	679.54			640.58			660.21						
PZ-4ab	617.68	620.13	0.47	13	597.16	19.01	2/14/00	601.84	0.27	0.014	1.3	0.12	0.15	Slightly Downward
PZ-4c	618.15	619.48			578.15			601.57						
MW-4	630.13	632.83	0.12	10	620.38	17.57	2/14/00	625.12	-1.94	-0.11	0.59	0.26	-0.25	Upward
PZ-6b	630.01	632.79			602.81			627.06						
MW-7	657.19	660.09	0.23	45	633.15	22.89	2/14/00	637.10	0.64	0.028	0.59	0.26	0.064	Slightly Downward
PZ-7b	656.96	659.86			610.26			636.46						

Notes:

1. PZ-1ab/PZ-1c is a well pair between the soil and bedrock units, respectively. They are located in an upland area along the central drainage feature.
2. PZ-4ab/PZ-4c is a well pair between the soil and bedrock units, respectively. They are located in a lowland area on the expansion site near Buffalo Creek.
3. MW-4/PZ-6b is a well pair between piezometers screened in the soil at the water table and top of bedrock, respectively. They are located in a lowland area along the central drainage feature.
4. MW-7/PZ-7b is a well pair between piezometers screened in the soil at the water table and top of bedrock, respectively. They are located at the base of the existing landfill area.
5. Negative values for head difference, vertical hydraulic gradients, and flow gradients represent an upward flow gradient.
6. If the water elevation was below the top of the screen elevation, the "Midpoint" elevation of the water column was used.

Prepared By: MSP
Checked By: AWA

TABLE 9

HYDRAULIC CHARACTERISTICS OF EACH MAJOR LITHOLOGIC UNIT
JMN/Cleveland Container Industrial Landfill
Cleveland County, NC
BLE Project Number J99-1307-04

Unit	Nat. Moisture Content (%)	Porosity (%)		Hydraulic Conductivity (cm/sec)	
		Effective	Total	Soil Sample	Slug Test
Saprolite	10.1% - 44.4%	3.5% - 25% [3]	46.0%	3.5E-04	3.0E-04
PWR	13.8%	30% [3]	too hard [2]; similar to Saprolite	too hard [2]	5.5E-04 to 3.8E-05
Upper Bedrock	too hard [2]		5% to 10% [4]	too hard [2]	7.6E-04 to 7.0E-04

NOTES:
1. PWR = Partially Weathered Rock
2. Undisturbed samples of the PWR and bedrock can not be obtained because the formation material is too hard.
3. Effective porosity in saprolite and PWR are estimated from specific yield as described by Fetter (1988).
4. Total porosity in bedrock is from Kruseman and deRidder (1989).

Attachment C

New and Revised Figures of the SHR (under seperate cover)

Attachment D

Additional Precipitation Data

APPENDIX I

MONTHLY PRECIPITATION DATA - 1980 TO 2000

North Carolina Division 5
JMN/Cleveland Container Industrial Landfill
Cleveland Co., North Carolina
BLE Job Number J99-1307-04

	YEAR																					
MONTH	**1980**	**1981**	**1982**	**1983**	**1984**	**1985**	**1986**	**1987**	**1988**	**1989**	**1990**	**1991**	**1992**	**1993**	**1994**	**1995**	**1996**	**1997**	**1998**	**1999**	**2000**	**21-Yr Monthly Avg.**
January	4.92	0.59	5.42	2.77	4.54	4.23	1.34	6.12	3.83	1.87	3.89	5.71	2.51	6.32	4.55	5.53	4.28	3.86	8.04	4.86	5.5	4.32
February	1.43	3.47	5.41	5.64	5.83	4.59	1.31	4.85	1.63	4.9	5.3	1.77	3.68	2.95	3.17	5.42	2.65	4.25	5.08	2.44	2.28	3.72
March	8.65	2.07	1.9	6.98	5.73	1.06	2.87	4.79	2.63	5.69	3.81	6.94	3.92	8.26	5.66	2.51	5.1	3.51	4.92	2.49	3.16	4.41
April	2.56	0.79	3.83	4.02	4.4	1.56	0.69	2.75	2.33	3.81	2.89	5.36	3.46	4.31	2.04	0.89	3.54	5.51	5.47	4.16	4.66	3.29
May	4.36	3.82	4.36	2.41	6.42	4.12	2.03	1.46	2.29	5.05	5.96	2.83	4.05	3.09	2.03	4.18	2.19	1.77	3.53	1.6	1.75	3.30
June	3.23	3.53	6.24	3.01	2.84	4.91	0.76	4.43	2.33	5.53	0.78	3.83	7.48	2.66	6.47	8.01	3.02	3.87	2.44	4.51	2.92	3.94
July	3.05	7.43	4.6	1.16	9.76	5.51	2.69	2.83	3.42	6.12	3.65	6.42	1.93	3.11	6	5.2	4.48	9.09	4.33	2.85	3.81	4.64
August	2.22	3.62	2.82	3.5	2.93	8.95	8.51	3.14	5.73	4.14	2.97	7	5.27	3.84	5.15	6.78	5.53	0.71	3.82	2.65	3.04	4.40
September	6.85	3.6	2.09	2.61	0.87	0.2	1.5	7	4.65	6.65	1.38	1.97	3.66	3.09	3.94	3.34	6.11	3.99	5.11	8.31	7.44	4.02
October	3.54	3.52	4.73	2.5	2.47	5.54	3.43	1.37	3.73	4.59	13.21	0.69	6.7	2.96	3.43	8.14	3.38	4.57	2.57	4.61	0	4.08
November	3.36	0.82	2.82	4.46	1.58	7.07	4.97	4.2	3.6	2.96	2.47	1.78	7.17	3.19	2.89	5	3.5	3.81	1.82	1.79	2.82	3.43
December	1.21	6.15	4.52	7.57	2.26	1.1	3.8	3.46	1.28	3.62	3.05	2.99	3.09	3.69	2.35	1.55	3.19	4.28	3.67	1.9	1.47	3.15
SEASON																						**21-Yr Seasonal Avg.**
Winter	15	6.13	12.73	15.39	16.1	9.88	5.52	15.76	8.09	12.46	13	14.42	10.11	17.53	13.38	13.46	12.03	11.62	18.04	9.79	10.94	12.45
Spring	10.15	8.14	14.43	9.44	13.66	10.59	3.48	8.64	6.95	14.39	9.63	12.02	14.99	10.06	10.54	13.08	8.75	11.15	11.44	10.27	9.33	10.53
Summer	12.12	14.65	9.51	7.27	13.56	14.66	12.7	12.97	13.8	16.91	8	15.39	10.86	10.04	15.09	15.32	16.12	13.79	13.26	13.81	14.29	13.05
Fall	8.11	10.49	12.07	14.53	6.31	13.71	12.2	9.03	8.61	11.17	18.73	5.46	16.96	9.84	8.67	14.69	10.07	12.66	8.06	8.3	4.29	10.66
																						Yearly Avg.
Yearly Totals	45.38	39.41	48.74	46.63	49.63	48.84	33.90	46.40	37.45	54.93	49.36	47.29	52.92	47.47	47.68	56.55	46.97	49.22	50.80	42.17	38.85	46.69
Rank from 1980 to 1999	16	18	9	14	5	8	21	15	20	2	6	12	3	11	10	1	13	7	4	17	19	

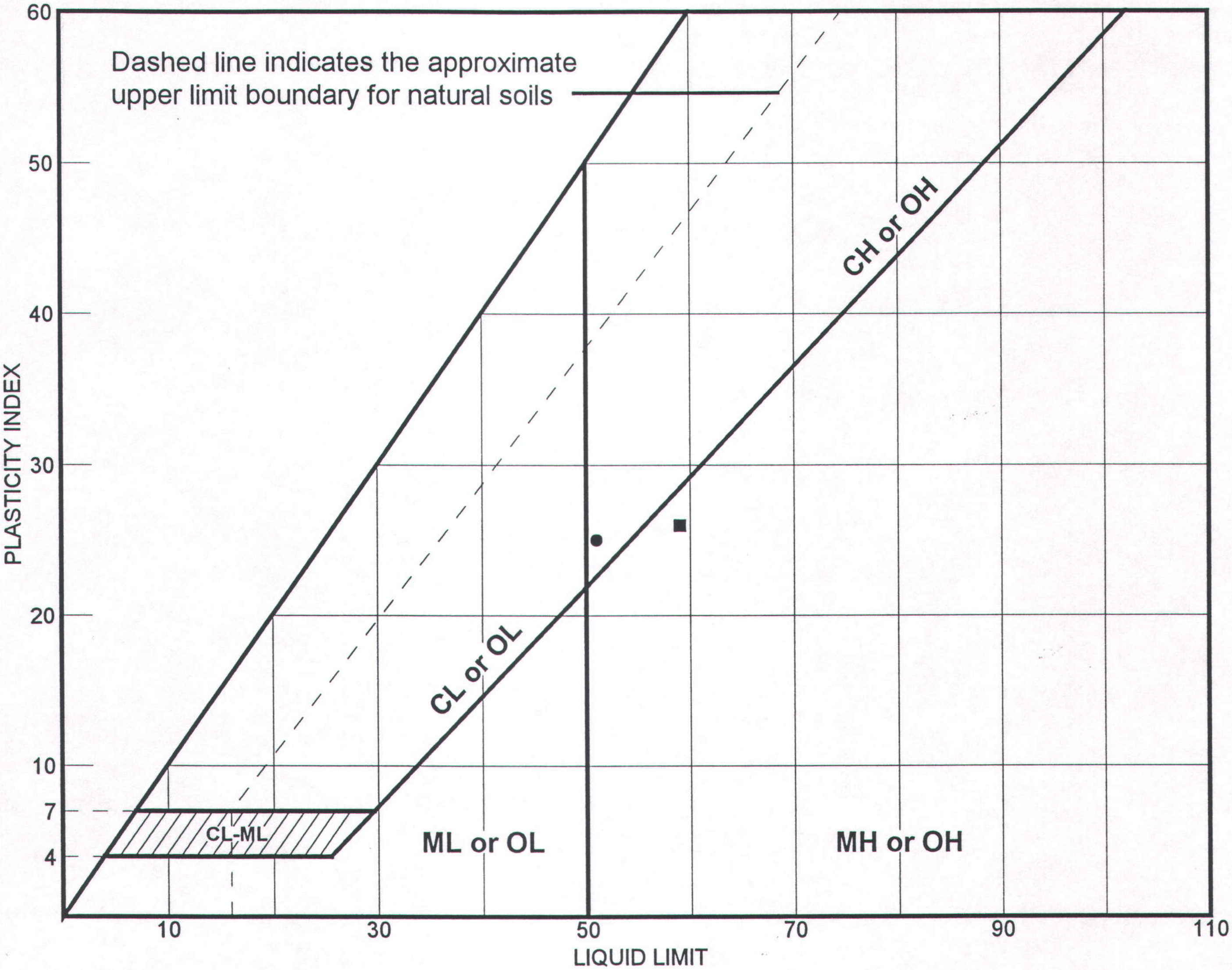
Year	**1980**	**1981**	**1982**	**1983**	**1984**	**1985**	**1986**	**1987**	**1988**	**1989**	**1990**	**1991**	**1992**	**1993**	**1994**	**1995**	**1996**	**1997**	**1998**	**1999**	**2000**
Winter (inches)	15	6.13	12.73	15.39	16.1	9.88	5.52	15.76	8.09	12.46	13	14.42	10.11	17.53	13.38	13.46	12.03	11.62	18.04	9.79	10.94
Rank from 1980 to 2000	6	20	11	5	3	17	21	4	19	12	10	7	16	2	9	8	13	14	1	18	15
Spring (inches)	10.15	8.14	14.43	9.44	13.66	10.59	3.48	8.64	6.95	14.39	9.63	12.02	14.99	10.06	10.54	13.08	8.75	11.15	11.44	10.27	9.33
Rank from 1980 to 2000	12	19	2	15	4	9	21	18	20	3	14	6	1	13	10	5	17	8	7	11	16
Winter plus Spring (inches)	25.15	14.27	27.16	24.83	29.76	20.47	9	24.4	15.04	26.85	22.63	26.44	25.1	27.59	23.92	26.54	20.78	22.77	29.48	20.06	20.27
Rank from 1980 to 2000	8	20	4	10	1	16	21	11	19	5	14	7	9	3	12	6	15	13	2	18	17

Data Source: NOAA, public information

Attachment E

Additional Soil Laboratory Results

LIQUID AND PLASTIC LIMITS TEST REPORT



SOIL DATA

SYMBOL	SOURCE	SAMPLE NO.	DEPTH (ft.)	NATURAL WATER CONTENT (%)	PLASTIC LIMIT (%)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	USCS
●	Borrow	B-2(1)	1.0-2.5		26	51	25	CH
■	Borrow	B-2(2)	3.5-5.0		33	59	26	MH

LIQUID AND PLASTIC LIMITS TEST REPORT

BUNNELL-LAMMONS ENGINEERING, INC.

Client: Hodges, Harbin, Newberry & Tribble Inc.

Project: Cleveland Container Landfill

Project No.: J99-1307-04

Plate

Attachment F

Revised Boring Logs

BLE INC.

BUNNELL-LAMMONS ENGINEERING, INC.

GEOTECHNICAL AND ENVIRONMENTAL CONSULTANTS

PIEZOMETER NO. PZ-2ab

PROJECT:	Cleveland Container Landfill	PROJECT NO.:	J99-1307-04
CLIENT:	HHNT	DATE START:	12-3-99 END: 12-3-99
LOCATION:	Shelby, North Carolina	ELEVATION:	648.93
DRILLER:	Superior Drilling, Inc., F. Cox	LOGGED BY:	MSP
DRILLING METHOD:	CME 550 ATV Hollow stem auger		

DEPTH TO - WATER> INITIAL: ▽ 35 AFTER 24 HOURS: ▼ 29.96 CAVING>

ELEVATION/ DEPTH (FT)	DESCRIPTION	STANDARD PENETRATION RESULTS BLOWS/FOOT
0	6-inches of TOPSOIL	
0.5–3	Firm, brown, moist, fine sandy, silty CLAY - (residuum)	6
3–5.5 (645 at 4)	Stiff, brown, moist, micaceous, clayey, fine sandy SILT	12
5.5–12 (640 at 9)	Firm, brown, moist, very micaceous, clayey, silty, fine SAND	13, 14
12–17 (635 at 14)	Firm, gray and white, moist, very micaceous, silty, fine to medium SAND	14
17–22 (630 at 19)	Firm, brown, moist, very micaceous, silty, fine to medium SAND	17
22–37 (625 at 24, 620 at 29, 615 at 34)	Firm and loose, gray, black and white, moist, very micaceous, silty, fine to medium SAND	19, 10, 15
37– (610 at 39)	PARTIALLY WEATHERED ROCK which sampled as black and white, moist,	50/4"

MONITOR WELL INSTALLATION DETAILS

- Bentonite seal, 0 to 27 feet
- Filter pack, sand 27 to 39 feet
- .010-inch slotted Schedule 40 PVC well screen, 28.7 to 38.7 feet
- Pipe cap

GEOT WELL 1307-0 1/01

BLE INC.

BUNNELL-LAMMONS ENGINEERING, INC.

GEOTECHNICAL AND ENVIRONMENTAL CONSULTANTS

PIEZOMETER NO. PZ-2ab

PROJECT: Cleveland Container Landfill — PROJECT NO.: J99-1307-04

CLIENT: HHNT — DATE START: 12-3-99 END: 12-3-99

LOCATION: Shelby, North Carolina — ELEVATION: 648.93

DRILLER: Superior Drilling, Inc., F. Cox — LOGGED BY: MSP

DRILLING METHOD: CME 550 ATV Hollow stem auger

DEPTH TO - WATER> INITIAL: ▽ 35 AFTER 24 HOURS: ▼ 29.96 CAVING>

ELEVATION/ DEPTH (FT)	DESCRIPTION	SOIL TYPE	SAMPLES	STANDARD PENETRATION RESULTS BLOWS/FOOT (2 5 10 20 30 40 50 70 90)	MONITOR WELL INSTALLATION DETAILS
	very micaceous, silty, fine to medium SAND				Total well depth, 39 feet
42	Auger refusal at 39 feet. No ground water encountered at time of drilling.				
605 / 44					
46					
48					
600 / 50					
52					
595 / 54					
56					
58					
590 / 60					
62					
585 / 64					
66					
68					
580 / 70					
72					
575 / 74					
76					
78					
570					

BLE INC.

BUNNELL-LAMMONS ENGINEERING, INC.

GEOTECHNICAL AND ENVIRONMENTAL CONSULTANTS

PIEZOMETER NO. PZ-3

PROJECT: Cleveland Container Landfill — PROJECT NO.: J99-1307-04

CLIENT: HHNT — DATE START: 12-6-99 END: 12-6-99

LOCATION: Shelby, North Carolina — ELEVATION: 669.21

DRILLER: Superior Drilling, Inc., F. Cox — LOGGED BY: MSP

DRILLING METHOD: CME 550 ATV Hollow stem auger

DEPTH TO - WATER> INITIAL: ▽ dry AFTER 24 HOURS: ▼ dry CAVING>

ELEVATION/ DEPTH (FT)	DESCRIPTION	SOIL TYPE	SAMPLES	STANDARD PENETRATION RESULTS BLOWS/FOOT	MONITOR WELL INSTALLATION DETAILS
0–0.5	6-inches of TOPSOIL				
0.5–3	Stiff, reddish-brown, moist, micaceous, fine sandy, clayey SILT - (residuum)		X	11	
3–12 (665 at 4, 660 at 9)	Firm, brown, slightly moist, very micaceous, clayey, silty, fine to medium SAND		X	15 / 11 / 9	
12–27 (655 at 14, 650 at 19, 645 at 24)	Loose to firm, gray and brown, moist, very micaceous, silty, fine to medium SAND		X	9 / 13	Bentonite seal, 0 to 31 feet
23–25	UD tube pushed from 23 to 25 feet		UD		
27–39 (640 at 29, 635 at 34, 630 at 39)	Loose to firm, gray, brown and white, moist, very micaceous, silty, fine to medium SAND		X	10 / 15 / 20	Filter pack, sand 31 to 43.5 feet; .010-inch slotted Schedule 40 PVC well screen, 33.7 to 43.7 feet

SPT scale: 2 5 10 20 30 40 50 70 90

BLE INC.

BUNNELL-LAMMONS ENGINEERING, INC.

GEOTECHNICAL AND ENVIRONMENTAL CONSULTANTS

PIEZOMETER NO. PZ-3

PROJECT:	Cleveland Container Landfill	PROJECT NO.:	J99-1307-04
CLIENT:	HHNT	DATE START:	12-6-99 END: 12-6-99
LOCATION:	Shelby, North Carolina	ELEVATION:	669.21
DRILLER:	Superior Drilling, Inc., F. Cox	LOGGED BY:	MSP

DRILLING METHOD: CME 550 ATV Hollow stem auger

DEPTH TO - WATER> INITIAL: ∇ dry AFTER 24 HOURS: ▼ dry CAVING>

ELEVATION/ DEPTH (FT)	DESCRIPTION	SOIL TYPE	SAMPLES	STANDARD PENETRATION RESULTS BLOWS/FOOT (2, 5, 10, 20, 30, 40, 50, 70, 90)	MONITOR WELL INSTALLATION DETAILS
42					
625 / 44	PARTIALLY WEATHERED ROCK which sampled as gray and white, moist, very micaceous, silty, fine to medium SAND		☒ 50/5"	50/5"	Pipe cap Soil 44 to 45.5 feet Total well depth, 44 feet
46					
48					
620 / 50	Auger refusal at 45.5 feet. No ground water encountered at time of drilling.				
52					
615 / 54					
56					
58					
610 / 60					
62					
605 / 64					
66					
68					
600 / 70					
72					
595 / 74					
76					
78					
590					

BLE INC.

BUNNELL-LAMMONS ENGINEERING, INC.
GEOTECHNICAL AND ENVIRONMENTAL CONSULTANTS

GROUND WATER MONITORING WELL NO. MW-1B

PROJECT: Cleveland Container Landfill — PROJECT NO.: J01-1307-09
CLIENT: HHNT — DATE START: 3-8-01 END: 3-8-01
LOCATION: Shelby, North Carolina — ELEVATION: 780.35
DRILLER: Superior Drilling, Inc., F. Cox — LOGGED BY: MSP
DRILLING METHOD: Hollow stem auger
DEPTH TO - WATER> INITIAL: 50.5 AFTER 24 HOURS: 50.0 CAVING>

ELEVATION/ DEPTH (FT)	DESCRIPTION	STANDARD PENETRATION RESULTS BLOWS/FOOT	MONITOR WELL INSTALLATION DETAILS
780 / 0	No topsoil Stiff to very stiff, reddish-brown and yellowish-brown, slightly moist, micaceous, slightly clayey, fine sandy SILT - (residuum)	11 (2 ft); 28 (4 ft); 27 (7 ft)	SURFACE COMPLETION stick-up Northing = 536,869.32' Easting = 1,252,622.31' Top of PVC elev. = 783.13' Ground elev. = 780.35'
770 / 10	PARTIALLY WEATHERED ROCK which sampled as yellowish-brown, slightly moist, micaceous, silty, fine SAND	50/5"	
14	Dense, brown, gray and white, slightly moist, micaceous, silty, fine to medium SAND	38	
18	Dense to firm, dark reddish-brown and yellowish-brown, slightly moist, very micaceous, silty, fine to medium SAND	35; 18	Neat cement, 0 to 36.6 feet
27	Firm, gray and light gray, slightly moist, micaceous, silty, fine to medium SAND	17	
32	Firm, dark brown, slightly moist, very micaceous, silty, fine to medium SAND	17	
37	Loose, light gray and yellowish-brown, slightly moist, micaceous, fine to medium SAND	7	Bentonite seal, 36.6 to 41.8 feet
42	Stiff, yellowish-brown, moist, clayey, fine sandy SILT	13	Filter pack, sand 41.8 to 59.3 feet
47	Firm to very firm, grayish-brown, moist, very micaceous, silty, fine to	15	

Elevation scale: 780, 775, 770, 765, 760, 755, 750, 745, 740, 735. Depth scale: 2 to 48 ft.

BLE INC.

BUNNELL-LAMMONS ENGINEERING, INC.
GEOTECHNICAL AND ENVIRONMENTAL CONSULTANTS

GROUND WATER MONITORING WELL NO. MW-1B

PROJECT: Cleveland Container Landfill — PROJECT NO.: J01-1307-09
CLIENT: HHNT — DATE START: 3-8-01 END: 3-8-01
LOCATION: Shelby, North Carolina — ELEVATION: 780.35
DRILLER: Superior Drilling, Inc., F. Cox — LOGGED BY: MSP
DRILLING METHOD: Hollow stem auger
DEPTH TO - WATER> INITIAL: ▽ 50.5 AFTER 24 HOURS: ▼ 50.0 CAVING>

ELEVATION/ DEPTH (FT)	DESCRIPTION	SOIL TYPE	SAMPLES	STANDARD PENETRATION RESULTS BLOWS/FOOT (2 5 10 20 30 40 50 70 90)	MONITOR WELL INSTALLATION DETAILS
730	medium SAND				.010-inch slotted Schedule 40 PVC well screen, 44 to 59 feet
54			X	23	
57.7	PARTIALLY WEATHERED ROCK which sampled as gray and yellowish-brown, moist, micaceous, silty, fine to medium SAND			50/2"	Pipe cap
59.3	Auger refusal at 59.3 feet. Ground water encountered at 50.5 feet at time of drilling and at 50.0 feet after 24 hours.				Total well depth, 59.3 feet Borehole diameter, 8.25-inches Well diameter, 2.0-inches

Elevation scale: 730, 725, 720, 715, 710, 705, 700, 695, 690, 685. Depth scale: 52, 54, 56, 58, 60, 62, 64, 66, 68, 70, 72, 74, 76, 78, 80, 82, 84, 86, 88, 90, 92, 94, 96, 98.

BLE INC.

BUNNELL-LAMMONS ENGINEERING, INC.
GEOTECHNICAL AND ENVIRONMENTAL CONSULTANTS

GROUND WATER MONITORING WELL NO. MW-1C

PROJECT:	Cleveland Container Landfill	PROJECT NO.:	J01-1307-09
CLIENT:	HHNT	DATE START:	5-25-01 END: 5-25-01
LOCATION:	Shelby, North Carolina	ELEVATION:	777.98
DRILLER:	A E Drilling, Inc., Kevin	LOGGED BY:	MSP
DRILLING METHOD:	Air hammer		

DEPTH TO - WATER> INITIAL: ▽ 63.0 AFTER 24 HOURS: ▼ 48.53 CAVING>

ELEVATION/ DEPTH (FT)	DESCRIPTION	SAMPLES	STANDARD PENETRATION RESULTS BLOWS/FOOT (2 5 10 20 30 40 50 70 90)	MONITOR WELL INSTALLATION DETAILS
0–10 (775 at 3; 770 at 8)	Temporary steel casing set to 7 feet No topsoil Stiff to very stiff, reddish-brown and yellowish-brown, slightly moist, micaceous, slightly clayey, fine sandy SILT - (residuum)			SURFACE COMPLETION stick-up Northing = 536,866.10 Easting = 1,252,606.53' Top of PVC elev. = 780.61' Ground elev. = 777.98'
9.5–13.5 (765 at 13)	PARTIALLY WEATHERED ROCK which sampled as yellowish-brown, slightly moist, micaceous, silty, fine SAND	R-1		
13.5–17.5	Dense, brown, gray and white, slightly moist, micaceous, silty, fine to medium SAND	R-2		
17.5–27 (760 at 18; 755 at 23)	Dense to firm, dark reddish-brown and yellowish-brown, slightly moist, very micaceous, silty, fine to medium SAND	R-3 R-4		
27–32 (750 at 28)	Firm, gray and light gray, slightly moist, micaceous, silty, fine to medium SAND	R-5		Neat cement, 0 to 58 feet
32–37 (745 at 33)	Firm, dark brown, slightly moist, very micaceous, silty, fine to medium SAND	R-6		
37–42 (740 at 38)	Loose, light gray and yellowish-brown, slightly moist, micaceous, fine to medium SAND	R-7		
42–47 (735 at 43)	Stiff, yellowish-brown, moist, clayey, fine sandy SILT	R-8		
47– (730 at 48 ▼)	Firm to very firm, grayish-brown, moist, very micaceous, silty, fine to			

BLE INC.

BUNNELL-LAMMONS ENGINEERING, INC.
GEOTECHNICAL AND ENVIRONMENTAL CONSULTANTS

GROUND WATER MONITORING WELL NO. MW-1C

PROJECT: Cleveland Container Landfill — PROJECT NO.: J01-1307-09
CLIENT: HHNT — DATE START: 5-25-01 END: 5-25-01
LOCATION: Shelby, North Carolina — ELEVATION: 777.98
DRILLER: A E Drilling, Inc., Kevin — LOGGED BY: MSP
DRILLING METHOD: Air hammer
DEPTH TO - WATER> INITIAL: ▽ 63.0 AFTER 24 HOURS: ▼ 48.53 CAVING>

ELEVATION/ DEPTH (FT)	DESCRIPTION	SOIL TYPE / SAMPLES	STANDARD PENETRATION RESULTS BLOWS/FOOT	MONITOR WELL INSTALLATION DETAILS
to 57	medium SAND	R-9, R-10		
57–60	PARTIALLY WEATHERED ROCK which sampled as gray and yellowish-brown, moist, micaceous, silty, fine to medium SAND	R-11		Bentonite seal, 58 to 63.5 feet
60–63.5	Gray, fine to medium grained biotite-feldspar-quartz GNEISS			
65–69	PARTIALLY WEATHERED ROCK which sampled as brown, moist, micaceous, silty, fine to medium SAND	R-12		Filter pack, sand 63.5 to 76 feet
69–70	Gray GNEISS			
70–74	PARTIALLY WEATHERED ROCK which sampled as brown, moist, micaceous, silty, fine to medium SAND	R-13		.010-inch slotted Schedule 40 PVC well screen, 65.7 to 75.7 feet
	Gray GNEISS	R-14		Pipe cap
	PARTIALLY WEATHERED ROCK which sampled as brown, moist, micaceous, silty, fine to medium SAND			Total well depth, 76 feet
	Gray GNEISS			Borehole diameter, 6-inches
	PARTIALLY WEATHERED ROCK which sampled as brown, moist, micaceous, silty, fine to medium SAND			Well diameter, 2.0-inches
	Gray GNEISS			
	Boring terminated at 76 feet. Ground water encountered at 63 feet at time of drilling. Soil descriptions from 0 to 57 feet are from MW-1B.			

Elevation scale: 725, 720, 715, 710, 705, 700, 695, 690, 685, 680. Depth scale: 52, 54, 56, 58, 60, 62, 64, 66, 68, 70, 72, 74, 76, 78, 80, 82, 84, 86, 88, 90, 92, 94, 96, 98.

Blows/foot scale: 2 5 10 20 30 40 50 70 90

GEOT WELL 1307-0… /5/01

BLE INC.

BUNNELL-LAMMONS ENGINEERING, INC.

GEOTECHNICAL AND ENVIRONMENTAL CONSULTANTS

GROUND WATER MONITORING WELL NO. MW-6A

PROJECT:	Cleveland Container Landfill	PROJECT NO.:	J01-1307-09
CLIENT:	HHNT	DATE START:	5-24-01 END: 5-24-01
LOCATION:	Shelby, North Carolina	ELEVATION:	634.58
DRILLER:	A E Drilling, Inc., Kevin	LOGGED BY:	MSP
DRILLING METHOD:	Air hammer		

DEPTH TO - WATER> INITIAL: ▽ 27.0 AFTER 24 HOURS: ▼ 25.15 CAVING>

ELEVATION/ DEPTH (FT)	DESCRIPTION	SAMPLES	STANDARD PENETRATION RESULTS BLOWS/FOOT	MONITOR WELL INSTALLATION DETAILS
0–7	Temporary steel casing set to 7 feet No topsoil			**SURFACE COMPLETION** stick-up Northing = 534,710.93' Easting = 1,253,114.90' Top of PVC elev. = 637.16' Ground elev. = 634.58'
2	Dense, dark gray and light gray, dry, micaceous, silty, fine to medium SAND - (residuum)	41		
630 / 4		7		
6	Loose, dark gray and brown, slightly moist, very micaceous, silty, fine to medium SAND	7		
8				Neat cement, 0 to 17 feet
625 / 10		6		
12	Very firm, dark gray and light gray, slightly moist, micaceous, silty, fine to medium SAND			
620 / 14		28		
18	PARTIALLY WEATHERED ROCK which sampled as dark gray and brown, slightly moist, micaceous, silty, fine to medium SAND	50/3"	50/3"	Bentonite seal, 17 to 20 feet
615 / 20	Gray, fine to medium grained biotite, feldspar, quartz GNEISS; dry	R-3		
22				Filter pack, sand 20 to 37.3 feet
610 / 24		R-4		
28	PARTIALLY WEATHERED ROCK which sampled as brown, moist, micaceous, silty, fine to medium SAND			
605 / 30	Gray GNEISS	R-5		.010-inch slotted Schedule 40 PVC well screen, 22 to 37 feet
32	PARTIALLY WEATHERED ROCK which sampled as brown, wet, micaceous, silty, fine to medium SAND			
600 / 34		R-6		
36	Gray, fine to medium grained biotite, feldspar, quartz GNEISS			Pipe cap
38	Boring terminated at 37.3 feet. Ground water encountered at 27.0 feet at time of drilling and at 25.25 feet after 24 hours.			Total well depth, 37.3 feet Borehole diameter, 6-inches Well diameter, 2.0-inches
595 / 40 – 585 / 50				

Standard penetration axis: 2 5 10 20 30 40 50 70 90