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DESIGN HYDROGEOLOGIC REPORT

and

WATER QUALITY MANAGEMENT PLAN

for

Phase 2, Cell I

Macon County MSWLF
Franklin, North Carolina
Permit # 57-03

June 19, 1997

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Design Hydrogeologic Report
for
Macon County MSWLF
Phase 2, Cell 1
Permit #57-03

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for
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This report has been prepared to comply with 15A NCAC 13B Rule .1623(b), Design Hydrogeologic Report.

RULE .1623

Area of Investigation: (b) (1)
The proposed Macon County MSWLF Phase 2, Cell 1 will occupy approximately 14.5 acres. Thirty-four test borings have been drilled into the water table and bedrock to determine the hydrologic and geologic conditions in and around the Phase 2 area.

Vertical Separation and Foundation Standards: (b) (1) (A)
Phase 2 of the Macon County MSWLF has been designed so that the base liner system is at least four feet above the projected high water table and bedrock (see Table 1).

Hydrogeologic Investigation: (b) (1) (B)
The hydrogeologic regime is discussed in full in the report.

Design Hydrogeologic Report: (b) (2) (A)

Boring Testing Program: (a) (4)
Thirty-four test borings were drilled within the area of investigation of the proposed Phase 2 (See Sheet 1). At each boring the following information was recorded: standard penetration-resistance, soil classification (based on the Unified Soil Classification System (USCS)), and soil or formation descriptions.

Boring logs with all of the information described above may be found in Appendix 1.

Standard Penetration - Resistance: (a) (4) (A)
The soil borings were advanced by mechanically twisting a continuous flight steel auger into the soil, or by rotary wash drilling. At regular intervals, soil samples were obtained with a standard 1.4-inch I.D., split-tube sampler. The sampler was first seated six inches to penetrate any loose cuttings, and then driven an additional 12 inches with blows of a 140-pound hammer falling 30 inches. The number of hammer blows required to drive the sampler the final 12 inches was recorded and is designated the "penetration resistance".

Samples of in-place soils were obtained during drilling. Several types of soil samples were obtained, including

- split-tube samples

- undisturbed samples, and
- bag (bulk) samples.

Particle Size Analyses: (a) (4) (B)
Particle size analyses were performed on selected, representative soil samples to determine the particle size distribution of the materials (see Appendix 2). After initial drying, the samples were washed over a U.S. standard No. 200 sieve to remove the fines. The samples were then dried and sieved through a standard set of nested sieves. This test was performed in a manner similar to that described by ASTM D 422.

USCS Soil Classification: (a) (4) (C)
Most borings encountered a surficial veneer of grass and topsoil up to 4 or 5 inches thick. The residual soils generally begin with a somewhat clayey zone to depths of 3 to 8 feet. These clayey soils are variably micaceous reddish-brown sandy silts and silty sands. The upper residual soils have Unified Soil Classifications of ML and SM.

The deeper residual soils (or saprolite) at the borings are variably micaceous silty sands with Unified Soil Classifications of SM. Some of these sands are interlayered with sandy silt (ML and MH).

Material dense enough to be termed "partially weathered rock" had a penetration resistance equivalent to or greater than 100 blows per foot.

Formation Descriptions: (a) (4) (D)
Refusal to the soil drilling equipment was encountered at 16 locations in the Phase 2 area at depths ranging between 20 feet (Boring PZ-5) and 94 feet (Boring PZ-7). Refusal material is defined as any material that cannot be penetrated by the soil drilling equipment. Samples of the underlying rock were obtained at 9 locations: PZ-2, PZ-5, PZ-7, PZ-11, PZ-14, B-21, B-26, MW-6A and MW-16A. A bedrock map (Sheet 2) was composed using all of the available auger refusal depths.

Core drilling procedures were required to penetrate refusal materials and determine their character and continuity. Refusal materials were cored according to ASTM D 2113 using a diamond-studded bit fastened to the end of a hollow double-tube core barrel. Core samples were identified and the percent core recovery and rock quality designation (RQD) was determined by a geologist. The percent core recovery (REC) is the ratio of the sample length to the depth cored, expressed as a percent. The RQD is obtained by summing only those pieces of recovered core which are four inches or longer and are at least moderately hard, and dividing by the total length cored. The percent recovery and the RQD are related to soundness and continuity of the refusal material.

The cored rock varies from being very weathered to non-weathered fresh garnetiferous biotite-gneiss. Recoveries and RQD's range from 15 to 96 percent and 0 to 89 percent, respectively. As suggested by the RQD values, the sampled rock at MW-16A had the highest number of joints and was the most fractured. Based upon the number of cores collected, it can be generally stated that weathering and fracturing is very severe in the vicinity of B-21, B-26, PZ-7, MW-16A and MW-6A. However, PZ-5 is located fairly close to B-26 and is relatively fresh, unweathered bedrock. Topographic position may play a role in the weathering zones: B-21, B-26, and PZ-11 are in topographically high locations, i.e. a ridge-top or side-slope. PZ-5, MW-16A and MW-6A are located in draws, in which weathering takes place at a much more rapid pace, thus leaving a very small weathered bedrock zone and a thick layer of soft saprolite. The topographically high borings may have a much broader weathering zone in the upper part of the bedrock because precipitation tends to runoff more readily from slopes and ridge-tops instead of infiltrating into the saprolite and then the fractured bedrock. Thus the bedrock weathering process is quite longer on ridge tops than in draws where runoff tends to accumulate.

Most of the fractures were low to medium-angled fresh joints with some steep, nearly vertical joints. There seemed to be no general trend across the site with regard to fracture density. Some cores that were taken out of topographically high locations were as fractured as those located in draws.

The rock core descriptions can be found with the appropriate boring log in Appendix 1.

Hydrologic Characteristics: (a) (4) (E)

Hydrologic characteristics of site soils were measured both in the laboratory and in the field (in-situ). In-place soils were characterized by conducting laboratory permeability tests on undisturbed samples. For that test, a portion of the undisturbed sample is placed in the permeability apparatus and saturated. Then water is pressed through the sample at a known head, and the rate of flow through the sample is measured. The test was performed in general accordance with ASTM D 5084. The hydraulic conductivity is calculated using Darcy's Law, $Q = kiA$, where "Q" is the measured flow through the sample, "i" is the hydraulic gradient, and "A" is the cross-sectional area of the soil sample. Laboratory permeability tests of undisturbed samples from borings PZ-3, PZ-5, PZ-6, PZ-7, PZ-9, PZ-10, and B-26 yielded results from 1.1×10^{-5} cm/sec to 5.0×10^{-6} cm/sec (see Table 2).

Inflow permeability tests were conducted at 14 observation wells. The results approximate the horizontal hydraulic conductivity of the formation materials exposed to the screened interval at each boring/well location. The field procedure used to measure hydraulic conductivity is as follows:

- measure the stabilized (static) water level in the well
- remove a slug of water from the borehole by bailing or pumping, and
- measure the groundwater level as it recovers to the static water level.

The data were reduced and hydraulic conductivity of the screened intervals calculated using techniques described by Hvorslev (Fetter, 1994). Results of the in-situ permeability tests ranged from 2.6×10^{-3} cm/sec to 8.8×10^{-5} cm/sec. Appendix 3 contains the raw data and results of the inflow permeability tests at all the locations, and Table 3 summarizes the permeability data.

Effective porosities were estimated for borings PZ-3, PZ-5, PZ-6, PZ-7, PZ-9, PZ-10, and B-26. The total porosities for the silty sands and sandy silts were determined in the lab and range between 42 and 58 percent. Effective porosities were estimated from information contained in groundwater textbooks and published handbooks (Fetter, 1988, and Rifai & Hopkins, 1996). Table 2 contains a summary of the porosity data and Appendix 5 shows the reference tables used to estimate the effective porosities.

Subsurface Conditions: (a) (5)
All of the subsurface conditions are discussed in the report.

Groundwater Flow Regime: (a) (6)
The groundwater flow regime within the proposed Phase 2 area is illustrated on the Potentiometric Surface Map (Sheet 3) and the subsurface profile maps (Sheets 5 through 9). (The plan view for the profile maps is shown on Sheet 4). The potentiometric map of Sheet 3 is based on water levels at the site measured on May 6, 1997. The map shows equipotential lines, or contour lines of equal groundwater elevations, and groundwater flow directions.

Local groundwater flow beneath the site is highly controlled by topography, which in turn is controlled by the bedrock. Groundwater flow appears to be radial in the Phase 2 area, and normal to the shoreline of Lake Emory. Groundwater discharges into Lake Emory and the small tributary which flows into the onsite lake. Horizontal flow gradients across the Phase 2 area range from 0.02 to 0.10 ft/ft. An average gradient is about 0.06 ft/ft.

The average groundwater flow velocity across the area was calculated to be approximately 2.3 ft/yr. The velocity was estimated using a formula derived from Darcy's Law:

$$V = \frac{ki}{ne}$$

where:
V = average linear groundwater seepage velocity
k = hydraulic conductivity (1.0×10^{-5} cm/sec)
i = hydraulic gradient (0.06 ft/ft)
ne = effective porosity (.27)

The hydraulic gradient was derived from the groundwater contours shown on Sheet 3, and the value of "ne" was obtained from literature sources.

Water Table Information: (a) (7)

Water Table Elevations: (a) (7) (A)

Water table elevations for all borings at time of boring, 24 hours, and stabilized are presented in Table 4.

Stabilized Water Table Elevations: (a) (7) (B)
Stabilized water table elevations were taken monthly since the installation of the piezometers up until the present. This information is also presented in Table 4.

Projected High Groundwater Levels: (a) (7) (C)
Groundwater fluctuations recorded in wells in the Phase 2 area appear to be mainly affected by seasonal variations in rainfall. Higher water levels are expected to occur in the winter and spring, and lower water levels in the summer and fall. Other causes of groundwater fluctuations are discussed in the following section.

Groundwater level measurements have been taken monthly in all of the piezometers up to the present. Unfortunately, those monthly measurements have amounted to only a half year of stabilized readings, which is not sufficient to determine a maximum annual fluctuation or maximum high water level reading. Therefore, groundwater level measurements from the nearest USGS recording well were taken into consideration. Water level measurements taken after 1995 were not available.

The long-term seasonal high water table was projected based primarily on the historical water level data from a nearby shallow saprolite USGS recording well. USGS well NC-40 is located off of U.S. Highway 276 at Camp Hope, 2 miles south of Cruso, Haywood County, N.C. The indicated location of the well is Latitude 35°23'15" and Longitude 82°48'44", and the elevation is 3,148.3 ft above MSL. The well is an 18.5 feet dug well in the muscovite-biotite gneiss saprolite; it is 12 inches in diameter, cased to 18.5 feet open end, and backfilled with gravel from 4 to 18.5 feet. The USGS has been monitoring water levels in this well since 1955. This well was chosen as a gauge of water level fluctuations because of its similar topographic subsurface setting to that of the Macon Co. MSWLF piezometers.

Based on the 10-year hydrograph (from 1986 to 1996) the greatest magnitude of fluctuation was 4.7 feet. The highest water level recorded for the 10-year period occurred at approximately 2.3 feet in February of 1990 and the low at 6.9 feet in October of 1986. The annual hydrograph for the year of 1995 showed a consistent difference between high and low water level recordings: the high was 2.81 feet in January and the low was 6.12 in August (with a difference of 3.31 feet). These findings are consistent with the above assumption that higher water levels are expected to occur in the winter and spring, and lower water levels in the summer and fall. The 10-year and 1995 hydrographs for NC-40 may be seen in Appendix 4.

Also, historical water levels measured on the monitoring wells surrounding Phase 1 cell were taken into account. MW-1A, MW-1B, MW-2, MW-3A, MW-3B, MW-4 and MW-5 were installed in 1991 and water levels were measured during the semi-annual water quality sampling events (see Table 5). The well that had the highest fluctuation was MW-4, which had a difference of 7.03 ft. between the highest and lowest water levels. The rest of the wells fluctuated between one and three feet from the highest to the lowest water levels. The highest historical water level for MW-4 was recorded on 5/7/97. Other wells that recorded highs on the same date were MW-1A, MW-1B and MW-3A. It is highly unlikely that the water level in MW-4 will rise another seven feet in the future, so it may be safe to assume that the reading on 5/7/97 could be near an all-time high. A safe assumption would be that the water level might rise another three feet (based on data from the other wells).

Applying the above information on the USGS well and the Phase 1 cell monitoring wells to the piezometers in the Phase 2 area, we could assume that the maximum annual fluctuation in the piezometers will be no more than five feet, and probably average around three feet. The projected high groundwater levels were estimated by adding the standard deviation to the high recorded water level for each boring, and then adding another two feet. This would add approximately three feet to each recorded high water level. Table 6 shows the calculation of the projected high water level for each boring.

Groundwater Fluctuations: (a) (7) (D)
Fluctuations in groundwater levels vary with subsurface conditions and topographic position. Groundwater levels also are subject to seasonal and longer term factors such as rainfall intensity and frequency, evapotranspiration due to plant growth and ground cover, and barometric pressure effects. Obviously tidal variations would not influence water level fluctuations because the site is too far inland. The Phase 2 cell is located sufficiently above the 100-year flood plain for Lake Emory. Also, there are no reservoirs, high volume production wells, or injection wells in the area of influence.

Man-made activities that might influence groundwater levels are those associated with site development for landfilling. Construction of the proposed Phase 2, once the liner and leachate collection system are in place, will essentially eliminate recharge from precipitation in the lined area. The net result of operation of the existing Phase 1 cell and the proposed Phase 2 is to reduce infiltration to recharge groundwater beneath the site. Thus, on a long-term basis, groundwater levels beneath the site are expected to decline, enhancing (increasing) separation between groundwater and the landfill bottom.

Horizontal & Vertical Flow Dimensions: (a) (8)
The generalized groundwater flow regime within the Phase 2 is illustrated on the Potentiometric Surface Map (Sheet 3) and the subsurface profile maps (Sheets 5 through 9). The plan view for the profile maps is shown on Sheet 4. The Potentiometric Surface Map of Sheet 3 is based on water levels at the site measured on May 6, 1997. The map shows equipotential lines, or contour lines of equal groundwater elevations, and groundwater flow directions.

Local groundwater flow beneath the site is highly controlled by topography, which in turn is controlled by the bedrock. Groundwater flow appears to be radial in the Phase 2 area, and normal to the shoreline of Lake Emory. Groundwater discharges into Lake Emory and the small tributary which flows into the onsite lake. Horizontal flow gradients across the Phase 2 area range from 0.02 to 0.10 ft/ft. An average gradient is about 0.06 ft/ft.

The average groundwater flow velocity across the area was calculated to be approximately 2.3 ft/yr. The velocity was estimated using a formula derived from Darcy's Law:

$$V = \frac{ki}{ne}$$

where:

V = average linear groundwater seepage velocity
k = hydraulic conductivity (1.0×10^{-5} cm/sec)
i = hydraulic gradient (0.06 ft/ft)
ne = effective porosity (.27)

The hydraulic gradient was derived from the groundwater contours shown on Sheet 3, and the value of "ne" was obtained from literature sources.

The calculated gradients and flow velocities for each cluster can be seen in Table 7.

Vertical gradients may be approximated by comparing water levels in nested well pairs. Five such pairs exist at the site: PZ-3/PZ-3A, PZ-8/PZ-8A, PZ-10/PZ-10A, MW-6/MW-6A and MW-16/MW-16A. The pairs consist of a shallow saprolite well in which the screened zone straddles the water table, and a deeper well installed in the saprolite just above auger refusal. From the water level data set May 6, 1997 the vertical gradients for the existing well pairs are as follows:

Well Cluster	Vertical Difference (ft)	Gradient(ft/ft)	Flow Direction
PZ-3 *PZ-3A	(-) 0.03	0.001	Upward (Discharging)
PZ-8 *PZ-8A	(-) 0.56	0.019	Upward (Discharging)
PZ-10 *PZ-10A	(+) 0.88	0.029	Downward (Recharging)
MW-6 *MW-6A	(+) 1.11	0.043	Downward (Recharging)
MW-16 *MW-16A	(-) 0.43	0.012	Upward (Discharging)

* Deeper well

It appears that discharging conditions exist in three of the clusters: PZ-3/PZ-3A, PZ-8/PZ-8A and MW-16/MW-16A. Discharging conditions probably exist because of the proximity of the three clusters to the large discharge area of Lake Emory. PZ-10/PZ-10A and MW-6/MW-6A are recharging. MW-6/MW-6A may be affected by mounding from a nearby sediment pond which makes it appear that recharging conditions exist, when in fact discharging conditions exist because of the proximity to Lake Emory.

Profiles A-A', B-B', D-D', and E-E' (Sheets 5, 6, 8 and 9) are drawn approximately parallel to groundwater contours, or perpendicular to flow. Because of the orientation of these profiles to flow direction, it was deemed inappropriate to show flow arrows.

Profile C-C' (Sheet 7) is drawn approximately perpendicular to groundwater contours, or parallel to groundwater flow. The extrapolated groundwater levels are annotated with arrows to indicate flow direction. Groundwater flows roughly from the southeast to the northwest. There appears to be a groundwater divide between MW-8 and MW-9. The Potentiometric Surface Map (Sheet 3) illustrates the divide in plan view. Thus, the groundwater flow direction is reversed (from the northwest to the southeast) at the divide.

The stratigraphic cross-section C-C' also illustrates the third (vertical) dimension of groundwater flow beneath Phase 2. Vertical gradients may be approximated by comparing water levels in clustered (nested) wells. Five such clusters exist at the site and were discussed above.

As mentioned in Section (a) (4) (E), inflow permeability tests were performed at 14 observation wells. The results approximate the horizontal hydraulic conductivity of the formation materials exposed to the screened interval at each boring/well location. Values of (horizontal) hydraulic conductivity (k) determined from the field tests range from from $2.6\text{x}10^{-3}$ to $8.8\text{x}10^{-5}$ cm/sec. For the purpose of estimating groundwater seepage velocity an average value for hydraulic conductivity of $1.0\text{x}10^{-5}$ cm/sec was selected.

Groundwater velocity beneath the area of Phase 2 was estimated using a formula derived from Darcy's Law:

$$V = \frac{ki}{ne}$$

where:

V = average linear groundwater seepage velocity
k = hydraulic conductivity ($1.0\text{x}10^{-5}$ cm/sec)
i = hydraulic gradient (0.06 ft/ft)
ne = effective porosity (.27)

The hydraulic gradient was derived from the groundwater contours shown on Sheet 3 and the value of "ne" relating specific sediment type to porosity was obtained from literature sources. The calculated, average seepage-flow velocity for groundwater based on the cited parameters is about 2.3 ft/year.

Potentiometric Surface Map: (a) (9)
Please see Sheet 3 for the Potentiometric Surface Map. This map was based on groundwater levels taken on May 6, 1997.

Topographic Map: (a) (10)
Please see Sheet 3 for a topographic map of the site that shows the soil boring locations.

Boring Log: (a) (11)
Please see Appendix 1 for all boring logs.

Other Geologic & Hydrologic Considerations: (a) (12)
The mountainous topographic setting of the Macon Co. MSWLF lends itself to a variety of geologic and hydrologic considerations: There are considerable slopes at the site since the landfill is situated on several ridges; one small drainage system flow from southern part of the landfill to the north and discharges into Lake Emory; and there are no springs. The only resemblance of gullies and trenches would be the steep natural draws; there are no solution/karst related features because the bedrock type is crystalline; rock corings exhibited no known dikes, sills, or faults at the site; no mines are present on the site; Lake Emory is the main discharge feature on the site; and recharge occurs on top of the ridges with groundwater discharging into Lake Emory.

Monitoring System Design Considerations: (b) (2) (B)
The number, spacing, and depths of wells in the monitoring system around the Phase 2 Expansion were determined based on the information gathered and discussed throughout this report and the more specific information that follows.

The uppermost aquifer beneath Phase 2 consists of a complex of soil, saprolite and fractured/weathered bedrock. The flow regime is understood and is discussed in section (a) (8) of this report. Groundwater in the soil and saprolite usually occurs under water table conditions. Water occupies the granular interstices between grains of the saprolite and is hydrostatic balance with the atmosphere at the water surface (water table). The saprolite thickness beneath Phase 2 varies from approximately 20 ft. thick to about 90 ft. thick. This information was based upon the depths to auger refusal. Saprolite is thickest in the northeastern section of the site (90 ft. at PZ-7).

In the crystalline rock beneath the Phase 2 area, groundwater occurs in fractures, joints, and other openings in the rock. These openings intersect the top of the bedrock and are recharged by groundwater in storage in the overlying saprolite. The fractured thickness of the bedrock is generally unknown but considered to exist in the partially weathered rock in the top portion of the bedrock. In other words, as the bedrock deepens, the fractures are less numerable and much smaller. Rock cores taken in and around the Phase 2 area show varying degrees of weathering, with some of the more fractured, less resistant found in the vicinities of B-26, B-21, PZ-7, MW-16A and MW-6A. More fresh, resistant bedrock was found in the areas of PZ-14, PZ-2 and PZ-5.

Groundwater flow appears to be radial in the Phase 2 area, and normal to the shoreline of Lake Emory. Groundwater discharges into Lake Emory and the small tributary which flows into the onsite lake. Horizontal flow gradients across the Phase 2 area range from 0.02 to 0.10 ft/ft. An average gradient is about 0.06 ft/ft. Horizontal gradients of groundwater flow and associated directions, as determined from water level data of May 6, 1997 for three 3-well groups, can be found in Table 7.

Groundwater velocity at the same three monitoring well/piezometer groups were estimated using a formula derived from Darcy's Law:

$$V = \frac{ki}{ne}$$

where:

V = average linear groundwater flow velocity
k = hydraulic conductivity (see Table 3)
i = hydraulic gradient (see Table 7)
ne = effective porosity (estimated to be about 0.27)

The calculated, approximate groundwater flow velocities can also be found in Table 7.

Fluctuations in groundwater levels vary with subsurface conditions and topographic position. Groundwater levels beneath Phase 2 are also subject to seasonal and longer-term factors such as rainfall intensity and frequency, plant growth and related ground cover, and barometric effects. Water level data collected monthly (Table 4) shows that the groundwater levels beneath Phase 2 fluctuate an average of approximately 2.6 ft/year with the largest fluctuation being 6.4 ft/year which occurred in PZ-9.

The subsurface materials in the near surface consist of unsaturated and saturated materials which are hydrologically connected. These materials represent weathering zones rather than distinct units. The uppermost zone consists of somewhat clayey soils which grade unconformably into a variably micaceous silty sand (called the saprolite), and then finally into partially weathered rock. Table 8 contains a listing of these three major weathering zones in the subsurface encountered by the borings, as defined by soil types. Associated values of porosity and hydraulic conductivity, established by in-situ or laboratory testing of site materials, or determined from the literature, also are listed in the table. The subsurface profiles (Sheets 5 through 9) are annotated with soil classification information and with field and laboratory determined hydraulic conductivities. Most of the subsurface penetrated by the borings is characterized by variably micaceous silty sands and some sandy silts. All tend to have hydraulic conductivities in the range of 10^{-5} to 10^{-4} cm/sec. Partially weathered rock at the site exhibits a range of conductivity similar to that of the micaceous silty sands/sandy silts tested, and fractured bedrock has an order of magnitude higher (10^{-3} cm/sec).

Point of Compliance Considerations: (b) (2) (C)
The 'relevant point of compliance', as can be seen by the proposed monitoring well configuration in Sheet 1, is located less than 250 feet from the Phase 2 waste boundary and greater than 50 feet from the property boundary. The factors outlined below were also taken into consideration when determining the 'relevant point of compliance'.

The hydrogeologic characteristics of the facility have been covered in depth in Section (a) (4) through (a) (12) of this report.

The leachate storage system is a surface impoundment. It is the same storage system that it is used for the existing Phase I cell, and is located adjacent to the water treatment facility. A monitoring well already exists (MW-2) downgradient of the surface impoundment to detect any leakage.

As mentioned in the discussion on groundwater movement in Section (a) (8), groundwater moves in a northerly and northwesterly direction beneath Phase 2 towards Lake Emory. By making the 'relevant point of compliance' the property boundary, or the eastern edge of the Lake (approximately at elevation 2000 ft.), there is sufficient area to monitor and remediate any contamination from Phase 2.

No groundwater is being taken from the Phase 2 area for public water use. There are two (2) existing water supply wells on the landfill property and two (2) water supply wells adjacent to the landfill property. Locations are shown on Sheet 3.

"WSW-1": This well is located on a church property which is adjacent to the landfill property. No information is available on the construction of the well . It is located approximately 575 feet from the Phase 2 waste boundary.

"WSW-2": This well is located on the landfill property and is approximately 250 feet deep. It is approximately 400 feet away from the waste boundary.

"WSW-3": This well is located on landfill property but there are no construction records. It is approximately 140 feet away from the waste boundary.

"WSW-4": This well is located on property adjacent to the landfill property and is a dry well. It is located approximately 600 feet from the waste boundary.

The existing groundwater quality beneath Phase 2 is good, based on water quality samples drawn from existing monitoring wells on the site.

There are no anticipated adverse effects on public health, safety, or welfare as a result of Phase 2.

Rock Corings: (b) (2) (D)
Rock corings were made at the Phase 2 area at the following locations: PZ-2, PZ-5, PZ-7, PZ-11, PZ-14, B-21, B-26, MW-6A and MW-16A (See Sheet 2). The cored rock varies

from being very weathered to non-weathered fresh garnetiferous biotite-gneiss. Recoveries and RQD's range from 15 to 96 percent and 0 to 89 percent, respectively. As suggested by the RQD values, the sampled rock at MW-16A had the highest number of joints and was the most fractured. Based upon the number of cores collected, it can be generally stated that weathering and fracturing is very severe in the vicinity of B-21, B-26, PZ-7, MW-16A and MW-6A. However, PZ-5 is located fairly close to B-26 and is relatively fresh, unweathered bedrock. Topographic position may play a role in the weathering zones: B-21, B-26, and PZ-11 are in topographically high locations, i.e. a ridge-top or side-slope. PZ-5, MW-16A and MW-6A are located in draws, in which weathering takes place at a much more rapid pace, thus leaving a very small weathered bedrock zone and a thick layer of soft saprolite. The topographically high borings may have a much broader weathering zone in the upper part of the bedrock because precipitation tends to runoff more readily from slopes and ridge-tops instead of infiltrating into the saprolite and then the fractured bedrock. Thus the bedrock weathering process is quite longer than in draws where runoff tends to accumulate.

Most of the fractures were low to medium-angled fresh joints with some steep, nearly vertical joints. There seemed to be no general trend across the site with regard to fracture density. Some cores that were taken out of topographically high locations were just as fractured as those located in draws.

The rock core descriptions can be found with the appropriate boring log in Appendix 1.

Projected High Groundwater Map: (b) (2) (E)
Please see Sheet 1 for the Projected High Groundwater Map.

Bedrock Contour Map: (b) (2) (F)
Please see Sheet 2 for the Bedrock Contour Map.

Vertical Groundwater Flow: (b) (2) (G)
The stratigraphic cross-section C-C' (Sheet 7) illustrates the third (vertical) dimension of groundwater flow beneath Phase 2.

Groundwater Flow Regime: (b) (2) (H)
Local groundwater flow beneath the site is highly controlled by topography, which in turn is controlled by the bedrock. Groundwater flow appears to be radial in the Phase 2 Expansion area, and normal to the shoreline of Lake Emory. Groundwater discharges into Lake Emory and the small tributary which flows into the onsite lake. Horizontal flow gradients across the Phase 2 area range from 0.02 to 0.10 ft/ft. An average gradient is about 0.06 ft/ft.

Inflow permeability tests were performed at 14 observation wells. The results approximate the horizontal hydraulic conductivity of the formation materials exposed to the screened interval at each boring/well location. Values of (horizontal) hydraulic conductivity (k) determined from the field tests range from from 2.6×10^{-3} to 8.8×10^{-5}

cm/sec. For the purpose of estimating groundwater seepage velocity an average value for hydraulic conductivity of 1.0×10^{-5} cm/sec was selected.

Groundwater velocity beneath the area of the Phase 2 Expansion was estimated using a formula derived from Darcy's Law:

$$V = \frac{ki}{ne}$$

where:

V = average linear groundwater seepage velocity
k = hydraulic conductivity (1.0×10^{-5} cm/sec)
i = hydraulic gradient (0.06 ft/ft)
ne = effective porosity (.27)

The hydraulic gradient was derived from the groundwater contours shown on Sheet 3 and the value of "ne" relating specific sediment type to porosity was obtained from literature sources. The calculated, average seepage-flow velocity for groundwater based on the cited parameters is about 2.3 ft/year.

Profiles A-A', B-B', D-D', and E-E' (Sheets 5, 6, 8 and 9) are drawn approximately parallel to groundwater contours, or perpendicular to flow. Because of the orientation of these profiles to flow direction, it was deemed inappropriate to show flow arrows.

Profile C-C' (Sheet 7) is drawn approximately perpendicular to groundwater contours, or parallel to groundwater flow. The extrapolated groundwater levels are annotated with arrows to indicate flow direction. Groundwater flows roughly from the southeast to the northwest. There appears to be a groundwater divide between MW-8 and MW-9. The Potentiometric Surface Map (Sheet 3) illustrates the divide in plan view. Thus, the groundwater flow direction is reversed (from the northwest to the southeast) at the divide. The stratigraphic cross-section C-C' also illustrates the third (vertical) dimension of groundwater flow beneath Phase 2.

Vertical gradients may be approximated by comparing water levels in nested well pairs. Five such pairs exist at the site: PZ-3/PZ-3A, PZ-8/PZ-8A, PZ-10/PZ-10A, MW-6/MW-6A and MW-16/MW-16A. The pairs consist of a shallow saprolite well in which the screened zone straddles the water table, and a deeper well installed in the saprolite just above auger refusal. From the water level data set May 6, 1997 the vertical gradients for the existing well pairs are as follows:

Well Cluster	Vertical Difference (ft)	Gradient(ft/ft)	Flow Direction
PZ-3 *PZ-3A	(-) 0.03	0.001	Upward (Discharging)
PZ-8 *PZ-8A	(-) 0.56	0.019	Upward (Discharging)

PZ-10 *PZ-10A	(+) 0.88	0.029	Downward (Recharging)
MW-6 *MW-6A	(+) 1.11	0.043	Downward (Recharging)
MW-16 *MW-16A	(-) 0.43	0.012	Upward (Discharging)

* Deeper well

It appears that discharging conditions exist in three of the clusters: PZ-3/PZ-3A, PZ-8/PZ-8A and MW-16/MW-16A. Discharging conditions probably exist because of the proximity of the three clusters to the large discharge area of Lake Emory. PZ-10/PZ-10A and MW-6/MW-6A are recharging. MW-6/MW-6A may be affected by mounding from a nearby sediment pond which makes it appear that recharging conditions exist, when in fact discharging conditions exist because of the proximity to Lake Emory.

Well Abandonment: (b) (2) (I)
All borings at the site that have not been converted to permanent monitoring wells will be properly abandoned in accordance with the procedures for permanent abandonment of wells, as delineated in 15A NCAC 2C Rule .0113(a)(2).

In addition to the borings four water supply wells that served residences on and adjacent to landfill property will be abandoned prior to development of Phase 2, Cell 1. These four wells were discussed in Section (b) (2) (C).

Water Quality Monitoring Plan: (b) (3)

Groundwater Monitoring Plan: (b) (3) (A)

Monitoring Wells Rationale: (b) (3) (A) (ii)
Eight new monitoring wells and three existing wells have been proposed to monitor subsurface conditions around Phase 2: MW-10, MW-14 and MW-15 (already in place); MW-17; MW-18; MW-19 and MW-19A (moved from MW-16 and MW-16A); MW-20; MW-21; MW-22 and MW-22A. Sheet 1 shows the proposed placement of the monitoring wells around the landfill.

The basic rationale for determining the depth and screened interval of the wells was based upon the intersection of the screened interval with the projected high water table and vertical flow conditions. All of the wells will be shallow wells, screened across the water table in the saprolite, except MW-19A and MW-22A which will have very short screens on top of the bedrock. The screen lengths in the shallow wells will be 15 ft, except for the existing wells which have screen lengths of 10 feet. This length will allow for the seasonal fluctuations in the water table. Water level readings have been taken monthly since the installation of the piezometers. An average water level fluctuation of 2.6 ft. has been observed with a maximum of approximately 6.4 ft. Water level high elevations have been observed to occur mainly in the spring months, i.e., April and May.

The locations for the proposed monitoring wells were chosen basically for the following reasons: location with respect to proposed waste fill boundary, ability to monitor directional extents of potential contaminant migration, and the hydrogeological aspects.

Monitoring well MW-17 will be the background well for Phase 2. It is located upgradient on the groundwater divide that exists over the area. The total depth of MW-17 is proposed to be approximately 60 feet with the screen depth from 45 feet to 60 feet.

Groundwater flow within the cell is mainly in the north to northwest directions. Thus it is considered more crucial to monitor the northern and western sides of the cell. However, two monitoring wells have been proposed to monitor the southeastern side of the cell: MW-18 and the existing MW-10. Monitoring well MW-18 will be approximately 50 feet deep with a screened zone of 35 to 50 feet. The boring log for MW-10 can be seen in Appendix 1. It is 65 feet deep and screened from 55 to 65 feet.

Monitoring wells MW-14 and MW-15 are already in place on the North-northeast side of the cell. These wells will track any migration in that direction, even though the general flow direction is more to the northwest. MW-14 is 39 feet deep and screened from 29 to 39 feet, and MW-15 is 17 feet deep and screened from 7 to 17 feet. The boring logs for these wells can be seen in Appendix 1.

As mentioned above, the predominant groundwater flow direction is in the northwest direction, or normal to the Lake Emory shoreline. It is therefore considered crucial to sufficiently monitor the northwest side of the cell, in between the waste and Lake Emory. Four locations for monitoring wells have been selected to monitor this side: MW-19/MW-19A, MW-20, MW-21 and MW-22/MW-22A. An upward flow gradient exists in this area therefore there will be clusters at two locations (MW-19/MW-19A and MW-22/MW-22A). The shallow well will be screened in the shallow saprolite and the deeper well will be screened in the deep saprolite.

Monitoring wells MW-19/MW-19A will be moved from the MW-16/MW-16A location because of the construction of the berm. This cluster is situated down-gradient of one of the two sumps that will be installed (see Sheet 1). The shallow well, MW-19, will be 20 feet deep with a screened zone from 10 to 20 feet. The deeper well, MW-19A, might be able to detect contamination before the more shallow well because of the upward flow direction. MW-19A will be 55 feet deep with a short screened zone from 52.5 to 55 feet. Monitoring well MW-20 is located downgradient of the second sump. It is proposed to be 20 feet deep with a screened zone of 5 to 20 feet. MW-20, along with MW-21, will detect any contamination that might leak from the sump area. MW-21 will also be 20 feet deep with a screened zone from 5 to 20 feet.

The second cluster, MW-22/MW-22A, is designed to detect contamination in the complex area (around PZ-14 and PZ-15) where the water table transitions from being in the bedrock to the saprolite. The exact location in the bedrock where the groundwater discharges into the saprolite in unknown, thus the design of the MW-22 cluster should detect any contamination flowing along the bedrock-saprolite interface at that transition zone. The shallow well, MW-22, will be 30 feet in depth with a screened zone of 15 to 30 feet. The deeper well, MW-22A, will be installed to auger refusal, around 40 feet in depth with a short screened zone from 37.5 to 40 feet.

Table 9 summarizes the depths and screened intervals for all or the proposed monitoring wells.

In addition to the above monitoring wells, the existing monitoring wells designed for the Phase I cell will continue to be sampled. These wells are MW-1A, MW-1B, MW-2, MW-3A, MW-3B, MW-4 and MW-5. They will be sampled during the same semi-annual sampling event as the new wells.

Monitoring Well Construction

The monitoring wells will be installed in the residuum or saprolite using the hollow stem auger drilling method. Immediately prior to boring activities, the drill rig, all downhole drilling rods, collars, split spoons, hollow stem augers, and other components will be decontaminated using the following procedures:

- thoroughly scrub with non-phosphate based detergent and potable grade water mixture
- rinse with potable grade water and steam clean
- rinse with nano-grade isopropyl alcohol and allow to air dry
- rinse twice with distilled water

Wells will be constructed using 2-inch diameter, flush threaded, schedule 40 PVC casing and screen. No PVC bonding compounds or glues will be used at any time during monitor well construction. In wells completed with the water table intersecting the screen, the screened section will be 15 ft. in length to allow for seasonal fluctuations. No risers will be put in the bottom of the holes.

Following placement of the screen and casing, the sand pack will be placed in the hole. The sand pack will consist of clean, well-sorted #1 sized sand grains. During the placement of the sand pack, care will be taken to avoid bridging and the depth of the sand pack will be monitored to prevent overpacking. The sand pack thickness will extend from the bottom of the well to a point two feet above the top of the well screen. Following the sand pack placement, a two foot thick bentonite seal will be emplaced immediately above the sand pack. The bentonite seal will be composed of bentonite having no additives such as synthetic or organic polymers. The bentonite seal will be emplaced by gravity means.

Neat cement grout will be placed above the bentonite seal using the tremie method. The grout mixture will consist of approximately seven gallons of clear potable water with about four pounds of bentonite per 94 pound bag of Portland cement. After placement of the grout, a lockable steel protective enclosure will be installed on the monitoring well. This enclosure will stand approximately two feet above ground surface and have a two foot square concrete pad base. The monitoring well identification number will be marked on the protective casing. Following completion, the enclosure will be locked with a pad lock. All pad locks for monitoring wells will be keyed alike. A metal pole approximately six feet in length with a reflective strip at the top will placed in the ground next to the well to provide easy location of the wells in the field.

An on-site geologist will supervise and record the work as described above. During all drilling and monitoring well activities, the on-site geologist will examine, log, and collect soil and rock samples and will complete all the logs, records, and completion reports.

All monitoring wells will be properly developed using the following procedure:

Prior to commencing development, the volume of water standing in a well will be calculated using the relationship

$$V = .041d^2h$$

where
V = volume of water (gallons)
d = diameter of well (inches)
h = length of water column (feet)

The wells will be developed by bailing or pumping until the water is free of fine-grained sediments or until turbidity values have stabilized. If clear sediment-free water cannot be obtained, at least 10 casing volumes will be removed. During the development of the well, indicating parameters for well stabilization will be taken. These include pH, specific conductance, temperature, and observations of color, clarity, and odor.

Water Level Monitoring

All new monitoring wells will be surveyed in to establish horizontal location and elevation of the measuring points. All elevations will be referenced to a benchmark previously established at the site. All wells will be located horizontally to the nearest 0.1 foot. Vertical elevations of measuring points will be made to the nearest 0.01 foot. Water levels will be collected during groundwater sampling events and measured with an electric water level indicator graduated in 0.01 foot increments. These data will be utilized to construct the potentiometric surface map, determine the horizontal hydraulic gradient, and groundwater flow direction.

Aquifer Testing

The purpose of the aquifer test is to determine the physical characteristics of the aquifer to allow evaluation of groundwater collection alternatives. Slug tests will be conducted on all monitoring wells to determine flow rates and hydraulic conductivities.

Groundwater Sampling and Analysis Plan

The sampling and analysis plan has been designed to comply with 15A NCAC 13B Rule .1632. The basic process for collection of groundwater quality data is to first purge the well prior to sampling, sample the well properly, transport the samples through the chain-of-custody to a Certified Laboratory, analyze the data properly, and interpret the analyses adequately.

Prior to sampling the well, the depth to water will be determined as discussed above. Purging the well will remove any stagnant water or stratified contaminants from the well bore and ensure that water being sampled is representative of the groundwater surrounding the well bore. Wells will be purged with a disposable Teflon bailer, thus eliminating the need for cleaning of bailers. All other equipment will be washed in Alconox and distilled water and then rinsed with distilled water. Groundwater sample collection will begin with the least contaminated wells and conclude with the most contaminated to prevent cross-contamination. The wells will be purged until a minimum of three to five times the volume of standing water in the well has been removed and the specific conductance, temperature, and pH of the groundwater have stabilized as indicated by at least three consecutive readings within 10% of each other. The well may be bailed dry. If the well is bailed dry it is considered a sufficient purge.

The samples will be collected using the disposable Teflon bailer. The bailer will be raised and lowered using new monofilament line. All lines will be discarded after each bailer use. Plastic sheeting will be placed on the ground surrounding the well. The first bailer of sample water will be used for the volatile organic analysis sample. Samples for VOC analyses will be placed into 40 ml vials by pouring the bailer contents down the side of the vial to minimize aeration and volatilization of the sample. The vials will be completed filled to create a meniscus, sealed using the Teflon septum cap, and then inverted and tapped lightly to ensure that no bubbles are present.

Sample labels will be properly marked using a water-proof pen. Samples will be placed on ice in a cooler provided by the laboratory. Field quality control checks will also include collection of blanks. These blanks will be used to evaluate the effects of general sample container collection techniques. Travel or trip blanks will be used to determine is contamination has occurred as a result of improper sample container cleaning. These trip blanks will be prepared prior to the sampling events by the laboratory. One trip blank for each volatile organic method will be provided per cooler used for storing volatile sample vials.

Sample custody procedures as outlined by the State protocol will be followed. Sample bottle chain-of-custody will begin at a certified laboratory, with empty sample containers properly decontaminated and preserved by the laboratory, sealed and ready for sampling. Immediately following the sampling procedures as outlined above, labels will be place on containers by the sampling personnel. The sample location, parameters to be analyzed, and any laboratory preservatives used will be noted on chain-of-custody. Field measurements of pH, temperature, and specific conductivity will also be noted. Samples will be hand delivered to ensure holding times are met.

Laboratory quality control and quality assurance procedures will be strictly met. For the Appendix I analyses, EPA Method 8260 organic method will be used with the following protocol procedures: one tune per 12 hours, one blank per 10 samples, one check standard per 10 samples, and one matrix spike duplicate per 10 samples. For the 15 metals of Appendix I analyses, the following quality control and quality assurance procedures will be enforced: one blank per 10 samples, one check standard per 10 samples, one duplicate sample per 10 samples, and one matrix spike duplicate per 20 samples.

The sampling schedule for existing wells is recommended by the Solid Waste Section to be on a semi-annual basis unless otherwise altered. Four baseline sampling episodes will carried out on the newly drilled monitoring wells. The initial sampling event will take place prior to issuing the Permit to Operate. The other three sampling events will be completed within six months of the issue date of the Permit to Operate. The four baseline sampling events will be spaced out over the six month period to provide as much information as possible on seasonal water quality variability.

Surface Water Monitoring Plan: (b) (3) (B)
Surface water will be sampled in four locations on the Macon Co. MSWLF property: SW-1, SW-2, SW-3 and SW-4. The SW-1 location will be taken out of Lake Emory near the discharge point for the water treatment plant. Surface water at SW-2 will be sampled from standing water on the wetlands area. The SW-3 location will be upstream on the small stream and SW-4 will be taken at the discharge point of the stream into the marshy area near Lake Emory.

Water Quality Monitoring Plan

This Water Quality Monitoring Plan for the Macon County Phase 2, Cell 1 will be effective in providing early detection of any release of hazardous constituents (from any point in the MSWLF unit) to the uppermost aquifer, so as to be protective of public health and the environment.

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6-19-97

TABLE 1

Projected High Groundwater Levels
Macon County MSWLF Phase 2, Cell 1

Well/Boring #	Projected High Water Level (ft above MSL)	Base Grade elev. (ft above MSL)	Vertical Separation (ft)
MW-7	2017.22	2060.0	42.8
MW-8	2074.54	2088.0	13.46
PZ-1	2022.88	2031.0	8.12
PZ-2	2036.98	2048.0	11.02
PZ-4	2020.97	2082.5	61.53
PZ-5	2037.62	2058.0	20.38
PZ-6	2041.61	2049.0	7.39
PZ-7	dry	2084.0	---
PZ-8	2029.85	2038.5	8.65
PZ-8A	2030.52	2038.5	7.98
PZ-9	2036.54	2078.5	41.96
PZ-11	2067.70	2083.0	15.30
PZ-12	2054.45	2060.5	6.05
PZ-13	2070.21	2098.0	27.79
PZ-16	2031.81	2045.0	13.19

TABLE 2
Summary of Lab Data

Boring/ Well Number	Interval Sampled (ft below ground surface)	Type of Material	Total Porosity* (%)	Effective Porosity** (%)	Lab Hydraulic Conductivity (cm/sec)
PZ-3	45 to 46	Silty Fine to Med. Sand	52	20 - 30	1.1×10^{-5}
PZ-5	20 to 21	Silty Fine to Coarse Sand	49	23 - 35	5.4×10^{-5}
PZ-6	45 to 46	Silty Fine to Med. Sand	48	20 - 30	1.8×10^{-5}
PZ-7	50 to 51	Fine to Med. Sandy Silt	53	13 - 25	1.4×10^{-5}
PZ-9	55 to 56	Silty Fine Sand	53	22 - 30	1.2×10^{-5}
PZ-10	25 to 26	Fine to Med. Sandy Silt	58	13 - 25	1.7×10^{-5}
B-26	15 to 20	Fine to Med. Sandy Silt	42	13 - 25	5.0×10^{-6}

* Taken from laboratory test results
** Lower value from Fetter, 1988; higher value from Table 3.1, Natural Attenuation Handbook, (Rifai & Hopkins, 1996)

TABLE 3

Summary of In-Situ Hydraulic Conductivity Tests
Macon CO. MSWLF - Phase 2

Boring/ Well Number	Depth of Screened Interval (ft below ground surface)	Type of Material Exposed to Screened Interval	Hydraulic Conductivity (k) (cm/sec)
MW-6	10 to 20	clayey sand	3.0×10^{-4}
MW-6A	43.5 to 46	partially weathered rock	5.1×10^{-4}
MW-7	15 to 25	silty sand	1.6×10^{-4}
MW-8	35 to 45	silty sand	5.3×10^{-5}
PZ-1	55 to 65	silty sand	2.2×10^{-5}
PZ-2	45 to 55	silty sand	1.4×10^{-5}
PZ-4	45 to 55	silty sand	4.2×10^{-5}
PZ-5	12.5 to 22.5	silty sand and sandy silt	8.8×10^{-5}
PZ-8	20 to 30	silty sand and sandy silt	2.5×10^{-4}
PZ-8A	50 to 60	silty sand	2.5×10^{-5}
PZ-11	45 to 55	partially weathered rock	5.7×10^{-5}
PZ-12	29 to 39	silty sand	1.9×10^{-4}
PZ-14	57 to 125*	fractured bedrock	6.2×10^{-3}
PZ-15	27 to 125*	fractured bedrock	2.6×10^{-3}

* open hole (no screen)

TABLE 4

Groundwater Levels

Macon Co. MSWLF

Depths and elevations to groundwater (ft.) from TOC

Well #	Time of Boring		24 Hours		7 Days		Date: 11/13/96		Date: 12/12/96		Date: 1/23/97	
	Depth	Elev.	Depth	Elev.	Depth	Elev.	Depth	Elev.	Depth	Elev.	Depth	Elev.
MW-6	14.50	2015.32	17.95	2011.87	18.50	2011.32	19.57	2010.25	19.37	2010.45	18.15	2011.67
MW-6A	15.00	2015.65	17.36	2013.29	17.01	2013.64						
MW-7	17.00	2017.40	20.63	2013.77	20.72	2013.68	21.31	2013.09	21.11	2013.29	20.99	2013.41
MW-8	40.00	2067.58	37.92	2069.66	36.56	2071.02	39.55	2068.03	40.02	2067.56	40.50	2067.08
MW-9	22.50	2070.78	25.05	2068.23	25.13	2068.15	26.50	2066.78	26.36	2066.92	***	***
MW-10	62.00	2053.08	53.08	2062.00	53.06	2062.02	54.28	2060.80	54.58	2060.50	***	***
MW-14	23.00	2026.54	34.48	2015.06	34.52	2015.02	34.70	2014.84	34.59	2014.95	34.75	2014.79
MW-15	11.00	2018.19	13.46	2015.73	13.50	2015.69	13.55	2015.64	13.60	2015.59	13.72	2015.47
MW-16	6.00	2014.67	19.06	2001.61	19.34	2001.33	19.09	2001.58	18.85	2001.82	18.89	2001.78
MW-16A	15.00	2004.98	17.24	2002.74								
PZ-1	50.00	2022.21	52.28	2019.93	52.26	2019.95	53.19	2019.02	53.21	2019.00	53.23	2018.98
PZ-2	50.00	2028.78	45.36	2033.42	46.67	2032.11	48.69	2030.09	49.16	2029.62	49.34	2029.44
PZ-3	63.50	2019.50	66.78	2016.22	68.30	2014.70	67.20	2015.80	67.99	2015.01	67.39	2015.61
PZ-3A	65.00	2018.12	67.89	2015.23	67.28	2015.84	***	***	67.33	2015.79	67.49	2015.63
PZ-4	45.00	2025.00	51.57	2018.43	51.53	2018.47	51.94	2018.06	51.87	2018.13	51.95	2018.05
PZ-5	15.50	2033.35	14.17	2034.68	14.27	2034.58	14.66	2034.19	14.40	2034.45	14.20	2034.65
PZ-6	38.00	2037.60	38.81	2036.79	39.10	2036.50	41.21	2034.39	41.74	2033.86	41.84	2033.76
PZ-7	50.00	2025.83	49.60	2026.23	49.75	2026.08	dry		dry		dry	
PZ-8	20.00	2035.54	30.07	2025.47	30.24	2025.30	31.15	2024.39	31.27	2024.27	31.27	2024.27
PZ-8A	23.00	2033.49	29.68	2026.81	31.34	2025.15	***	***	31.77	2024.72	31.68	2024.81
PZ-9	55.00	2032.97	55.20	2032.77	57.76	2030.21	58.54	2029.43	61.61	2026.36	59.04	2028.93
PZ-10	23.50	2034.08	29.13	2028.45	29.27	2028.31	29.82	2027.76	29.11	2028.47	27.19	2030.39
PZ-10A	24.00	2032.02	27.37	2028.65	27.79	2028.23	***	***	27.85	2028.17	26.79	2029.23
PZ-11	51.00	2059.44	46.76	2063.68	46.84	2063.60	47.90	2062.54	48.26	2062.18	***	***
PZ-12	35.00	2050.80	35.58	2050.22	35.56	2050.24						
PZ-13	50.00	2062.56	47.24	2065.32	47.09	2065.47						
PZ-14												
PZ-15												
PZ-16	40.00	2036.33										

*** Unable to measure – inclement weather; will measure in future

TABLE 4 (cont.)

Well #	Date: 3/11/97		Date: 4/8/97		Date: 5/2/97		Date: 5/6/97	
	Depth	Elev.	Depth	Elev.	Depth	Elev.	Depth	Elev.
MW-6	15.54	2014.28	14.48	2015.34			15.15	2014.67
MW-6A			16.50	2014.15	17.24	2013.41	17.09	2013.56
MW-7	20.19	2014.21	19.99	2014.41			19.74	2014.66
MW-8	39.57	2068.01	38.05	2069.53			36.64	2070.94
MW-9			21.71	2071.57			22.31	2070.97
MW-10	54.48	2060.60	53.77	2061.31			53.14	2061.94
MW-14	34.09	2015.45	35.39	2014.15				
MW-15		2029.19	14.64	2014.55				
MW-16	18.48	2002.19	18.47	2002.20			18.40	2002.27
MW-16A			17.46	2002.52	17.42	2002.56	17.28	2002.70
PZ-1	52.65	2019.56	52.20	2020.01			51.87	2020.34
PZ-2	48.25	2030.53	46.70	2032.08			45.76	2033.02
PZ-3	68.53	2014.47	68.50	2014.50			66.85	2016.15
PZ-3A			67.25	2015.87			66.94	2016.18
PZ-4	51.70	2018.30	51.57	2018.43			51.27	2018.73
PZ-5	13.75	2035.10	13.65	2035.20			13.61	2035.24
PZ-6	40.62	2034.98	38.15	2037.45			37.65	2037.95
PZ-7	dry		dry				dry	
PZ-8	29.85	2025.69	29.20	2026.34			28.67	2026.87
PZ-8A	30.50	2025.99	29.66	2026.83			29.06	2027.43
PZ-9	58.80	2029.17	58.46	2029.51			57.85	2030.12
PZ-10	25.56	2032.02	25.68	2031.90			26.13	2031.45
PZ-10A	25.77	2030.25	25.40	2030.62			25.45	2030.57
PZ-11	47.72	2062.72	46.71	2063.73			45.64	2064.80
PZ-12			34.70	2051.10	34.41	2051.39	34.35	2051.45
PZ-13			46.40	2066.16	45.64	2066.92	45.35	2067.21
PZ-14					79.34	2048.21	79.17	2048.38
PZ-15					85.07	2006.30	84.77	2006.60
PZ-16			48.33	2028.00	47.76	2028.57	47.52	2028.81

TABLE 5

Macon Co. MSWLF
Phase 1 Cell Monitoring Wells

	MW-1A		MW-1B		MW-2		MW-3A		MW-3B		MW-4		MW-5	
	Depth	Elev.	Depth	Elev.	Depth	Elev.	Depth	Elev.	Depth	Elev.	Depth	Elev.	Depth	Elev.
09/28/94	6.67	2005.43			12.84	2003.14	59.90	2012.95	dry		35.84	2062.42	51.90	2020.88
10/13/94	6.57	2005.53			12.90	2003.08	60.05	2012.80	dry		40.54	2057.72	52.19	2020.59
02/01/95	5.95	2006.15			12.74	2003.24	59.94	2012.91	dry		40.50	2057.76	52.15	2020.63
03/15/95	5.95	2006.15			12.29	2003.69	59.12	2013.73	dry		40.30	2057.96	51.65	2021.13
11/28/95	7.58	2004.52			13.15	2002.83	60.23	2012.62	dry		40.61	2057.65	52.66	2020.12
10/15/96	7.86	2004.24	7.77	2004.33	13.40	2002.58	60.03	2012.82	dry		35.06	2063.20	52.26	2020.52
05/07/97	4.86	2007.24	4.83	2007.27	12.50	2003.48	58.62	2014.23	dry		33.58	2064.68	51.36	2021.42

TABLE 6
Projected High Water Levels
Macon Co.- Phase 2, Cell 1

Well #	Mean	Std. Deviation (SD)	High (H)	Projected High (H + SD + 2ft.)	Bedrock Surface Elev.
MW-7	2013.73	0.56	2014.66	2017.22	----
MW-8	2068.98	1.52	2071.02	2074.54	----
PZ-1	2019.60	0.54	2020.34	2022.88	----
PZ-2	2031.29	1.56	2033.42	2036.98	2021.45
PZ-4	2018.33	0.24	2018.73	2020.97	
PZ-5	2034.76	0.38	2035.24	2037.62	2022.11
PZ-6	2035.71	1.66	2037.95	2041.61	----
PZ-7	dry	dry	dry	dry	1978.65
PZ-8	2025.33	0.98	2026.87	2029.85	----
PZ-8A	2025.96	1.09	2027.43	2030.52	----
PZ-9	2029.56	1.77	2032.77	2036.54	----
PZ-11	2063.32	0.90	2064.80	2067.70	2038.28
PZ-12			2051.45	2054.45	2043.76
PZ-13			2067.21	2070.21	----
PZ-14			2048.38	----	2067.40
PZ-15			2006.60	----	2057.26
PZ-16			2028.81	2031.81	2010.69
Boring #					
B-21					2057.3
B-22					2068.9
B-26					2065.8
B-27					2046.0
B-28					2036.9

TABLE 7

Calculated Groundwater Gradients and Flow Velocities
Macon Co. MSWLF

Well Cluster	Gradient (ft/ft)	Groundwater Flow Direction	Groundwater Flow Velocity (ft/yr)
PZ-11 MW-6 MW-16	0.10	Northwest	8.1
MW-9 PZ-13 PZ-12	0.020	North-Northwest	14.8
PZ-8 PZ-4 PZ-3	0.020	Northeast	19.5

TABLE 8

Summary of Hydrologic/Lithologic Data

Weathering Horizon	Unified Soil Classification	Total[1] Porosity (%)	Effective[2] Porosity (%)	Hydraulic[3] Conductivity (cm/sec)
Upper, Somewhat Clayey Soils	SC ML	20 to 50	24 to 25	N/A
Variably Micaceous Sandy Silt and Silty Sand	ML SM	53 to 58 48 to 53	25 to 28	$5.4x10^{-5}$ to $1.1x10^{-5}$
Partially Weathered Rock [4]	ML SM	48 to 53	25 to 30	$1.2x10^{-5}$
Fractured Bedrock (Gneiss)	-----	2 to 5*	1 to 5*	$6.2x10^{-5}$ to $2.6x10^{-5}$

(1) Derived from laboratory testing of undisturbed samples

(2) From Figure 4.11 of Fetter (1988) and Natural Attenuation Handbook (Rifai & Hopkins, 1996)

(3) From In-Situ Testing (see Table 3)

(4) Partially Weathered Rock has soil classification characteristics similar to Sandy Silt/Silty Sand Horizon

TABLE 9

Proposed Depths of Monitoring Wells
Macon Co. MSWLF
Phase 2, Cell I

Monitoring Well #	Type	Total Well Depth (ft)	Screen Depth (ft)
MW-10 (already in place)	shallow saprolite	65	55 - 65
MW-14 (already in place)	shallow saprolite	39	29 - 39
MW-15 (already in place)	shallow saprolite	17	7 - 17
MW-17	shallow saprolite	60	45 - 60
MW-18	shallow saprolite	50	35 - 50
MW-19	shallow saprolite	20	10 - 20
MW-19A	deep saprolite	55	52.5 - 55
MW-20	shallow saprolite	20	5 - 20
MW-21	shallow saprolite	20	5 - 20
MW-22	shallow saprolite	30	15 - 30
MW-22A	deep saprolite	40	37.5 - 40