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August 13, 2008

Project No. 063-6526.103

Department of Environment and Natural Resources
Division of Waste Management
Solid Waste Section
401 Oberlin Road, Suite 150
Raleigh, North Carolina 27605
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Attention: Ms. Christine Ritter
Hydrogeologist

Re: Response To Comment
Phase 2 Design Hydrogeologic Report and Water Quality Monitoring Plan
WCA of High Point C&D Landfill, Permit No. 41-16
Guilford County, North Carolina

Dear Ms. Ritter,

On behalf of WCA of High Point, LLC, a subsidiary of WCA Waste Corporation, Golder Associates NC, Inc. (Golder) is submitting this response to a comment presented in the May 14, 2008, letter from the Solid Waste Section regarding the *Phase 2 Design Hydrogeologic Report and Water Quality Monitoring Plan,* for the WCA of High Point Construction and Demolition Landfill. The comment from the May 14, 2008, letter requested further clarification of average seasonal high versus estimated long-term seasonal high water table elevations.

The April 18, 2007, submittal of the *Phase 2 Design Hydrogeologic Report and Water Quality Monitoring Plan* included long-term seasonal high groundwater elevation calculations that were used to design base grades for the proposed Phase 2 waste disposal unit. The average seasonal high groundwater elevations were not specifically addressed in the report. The text of section 6.1.2 of the *Phase 2 Design Hydrogeologic Report* has been revised to compare average and long-term seasonal high groundwater elevations. Also, an average seasonal high groundwater contour map has been added to the report as Drawing DH-6.

Three copies of revised cover pages, report text pages, table, and drawings are enclosed with this letter. Please replace the original pages with the revised pages. Also, a CD has also been included that contains PDF files of this letter report and attachments, as well as, a complete version of the revised *Phase 2 Design Hydrogeologic Report* text.

If you have any additional questions, please contact the undersigned at 336-852-4903. We appreciate your assistance with this project.

Sincerely,
GOLDER ASSOCIATES NC, INC.

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Project Hydrogeologist

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Enclosures

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DESIGN HYDROGEOLOGIC REPORT

WCA OF HIGH POINT CONSTRUCTION AND DEMOLITION LANDFILL
PHASE TWO
PERMIT NO. 41-16
GUILFORD COUNTY, NORTH CAROLINA

MARCH 2007
REVISED AUGUST 2008

PREPARED FOR:

WCA OF HIGH POINT
5830 RIVERDALE DRIVE
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1.0 INTRODUCTION

In accordance with Title 15A of the North Carolina Administrative Code (NCAC) Subchapter 13B.0538(b), WCA of High Point, LLC (WCA) is submitting this Design Hydrogeologic Report for the Phase 2 lateral expansion of a construction and demolition debris (C&D) landfill. The WCA of High Point C&D Landfill operates under Permit No. 41-16 issued by the North Carolina Department of Environment and Natural Resources (NC DENR). The purpose of this report is to characterize the hydrogeologic conditions in and around the Phase 2 waste footprint and compliance boundary. The report will also provide the relationship of the site groundwater flow regime to local and regional hydrogeologic features, with emphasis on the relationship of C&D landfill units to potential groundwater receptors and groundwater discharge features.

2.0 SITE DESCRIPTION

The WCA of High Point C&D Landfill is located along Riverdale Drive in Guilford County, North Carolina (Figure 1). The site consists of approximately 154 acres, of which, approximately 12 acres is currently permitted as the Phase 1 waste unit. The proposed Phase 2 waste unit will be approximately 9.4 acres. The site location, property boundaries, and surrounding topography are shown on Figure 1. The site is located in the central portion of the Piedmont physiographic province on a group of knolls and valleys.

There are three small unnamed streams that traverse the site that flow into an unnamed tributary of Richland Creek, which forms the southern property boundary as shown on Drawing DH-1. The first unnamed tributary is a southwest trending stream that forms the northwestern property boundary. The second tributary is a southwest trending drainage in the center of the property and lies just east of the proposed Phase 2 waste unit. The third tributary is southwest trending drainage in the eastern portion of the site. A large man-made pond is located near the eastern property boundary. A small storm water basin was constructed in the northwest corner of the facility and a larger storm water basin was constructed along the southwestern part of the site during the construction of Phase 1.

The western portion of the site mainly consisted of trees and heavy brush before Phase 1 of the C&D landfill was constructed. Now, most of that area consists of landfill operations. The eastern portion of the site has historically been used for agricultural purposes. A reclamation facility is now located at the northernmost part of the site. A large portion of the surrounding properties are agricultural or owned by the City of High Point for operations associated with Kersey Valley Landfill.

3.0 FIELD INVESTIGATIONS

The following sections summarize the previous and current field investigation activities. Pertinent data from previous investigations are also summarized in select tables as described below.

3.1 Previous Investigations

The field activities associated with the Site Plan Application for the C&D landfill were performed in February through December 2002. That investigation consisted of the installation of 23 piezometers, a field survey of the new piezometers, the drilling of three geotechnical borings, the excavation and investigation of 20 test pits, well development and aquifer testing of selected piezometers, measurement of groundwater elevations, general site reconnaissance to locate outcrops, and performance of a magnetometer survey. Three piezometers (P-15S, P-12, and P-18) were later converted into monitoring wells (MW-4S, MW-5, and MW-6) and incorporated into the facility's compliance network. Three additional monitoring wells (MW-1, MW-2, and MW-3) were installed in September and October 2003, to be incorporated into the facility's compliance network. Piezometer P-2 was decommissioned on June 4, 2003, in order to begin construction of the reclamation facility. Ten piezometers (P-8, P-9, P-10, P-13S, P-13D, P-14, P-17, P-19, P-20, and P-21) were properly decommissioned on September 24-27, 2003, in order to begin construction of the Phase 1 waste cell. All piezometers that were within the proposed waste footprint were decommissioned by overdrilling to remove well construction materials, and then grouted with a cement-bentonite grout. The monitoring well and piezometer construction data are summarized in Table 1 and the locations are provided on Drawing DH-1. The Site Plan Application was approved by the Solid Waste Section in May 2003.

3.2 Current Investigation

The field activities for this investigation were performed from June through October 2006. The initial fieldwork consisted of the installation of 14 piezometers, a field survey of the new piezometers, well development and aquifer testing of selected piezometers, and measurement of groundwater elevations.

3.2.1 Drilling Program

Eleven piezometers (P-23, P-24, P-25, P-26S, P-26D, P-27, P-28, P-29, P-30S, P-30D, and P-31) were installed in and around the proposed Phase 2 footprint from June 29, 2006, to July 13, 2006. South Atlantic Environmental Drilling and Construction Company (SAEDACCO) of Fort Mill, South Carolina, performed drilling and well installation activities. The drilling equipment consisted of a Gus Pech GP-1100B drilling rig equipped with 4.25-inch and 6.25-inch inner diameter (ID) hollow-stem augers, HQ core barrel, 5.75-inch outer diameter (OD) tricone bit, and 6-inch OD downhole hammer. Continuous split-spoon samples were obtained and logged by the geologist for all of the boreholes, except in locations where nested piezometer pairs were installed. A licensed Golder geologist was present to observe the drilling and rock coring, log the boreholes and supervise the piezometer construction. An experienced Golder representative developed piezometers selected for aquifer testing. The boring logs, well and piezometer construction records, and well development forms for the newly installed piezometers are included in Appendix DH-A. The piezometer construction data are summarized in Table 1 and the locations are provided on Drawing DH-1. Well and piezometer construction was performed in accordance with the standards described in the *RCRA Technical Enforcement Guidance Document* (1986*)* and the *Draft North Carolina Water Quality Monitoring Guidance Document for Solid Waste Facilities* (1995*)*.

Two of the piezometer locations were chosen for rock coring. Rock core was taken at P-26D and P-30D. Approximately 8 feet of rock core were obtained from P-26D and approximately 20 feet of rock core were obtained from P-30D. Coring was performed using an HQ rock coring device. The rock core data are summarized in Table 2.

Three additional shallow piezometers (HA-3, HA-4, and HA-5) were installed on July 11, 2006, along the drainage east of the proposed Phase 2 footprint, which was inaccessible by the drilling rig. These piezometers were installed using a hand auger by an experienced Golder geologist.

Piezometers HA-3, HA-4, and HA-5 were installed in saprolite and range in depth from 2.5 to 5 feet with screens lengths that range from 1 to 4 feet. Piezometers P-26S, P-30S, and P-31 were installed at the partially weathered rock (PWR) and range in depth from 28 to 45 feet with screen lengths that range from 10 to 15 feet. Piezometers P-23, P-24, P-25, P-26D, P-27, P-28, P-29, and P-30D were installed in bedrock and range in depth from 25 to 74 feet with screen lengths that range from 10 to 15 feet in length.

Unless specified below, piezometers were grouted to the surface and lockable expansion caps installed in each casing. Piezometers P-23, P-30S, P-30D, and P-31 were covered by lockable anodized aluminum protective casings installed into 3x3x0.5-foot concrete aprons to protect their integrity. Any other piezometers that may be converted into monitoring wells will have lockable anodized aluminum protective casings and concrete aprons installed prior to the first sampling event.

On September 18, 2006, piezometers P-23, P-26S, P-30S, and P-30D were developed with a submersible pump to remove accumulated sediments resulting from the drilling and construction process, and to hydraulically connect the piezometers with the surrounding soils. On September 19, 2006, piezometer P-30S was further developed with a submersible pump. Also, piezometers P-25 and P-27 were developed with disposable bailers and P-26S was further developed with a disposable bailer. All piezometers were purged dry during development. Because of the small yields from each of the piezometers, approximately 7.5 to 17 gallons of water were purged from each piezometer during development. Any other piezometers that will be converted into permanent monitoring wells will be developed using similar procedures prior to the first sampling event.

The newly installed piezometers, MW-5, and a springhead in the drainage east of the proposed Phase 2 waste unit were surveyed in August 2006 by Fleming Engineering, Inc. of Greensboro, North Carolina. The height for well MW-5 had been modified during previous site activities. Elevations and horizontal locations were referenced to the N.C. Grid Coordinate System. The piezometer survey map and a sealed letter prepared by Fleming Engineering, Inc. are provided in Appendix DH-B.

A historical summary of the static groundwater elevations for all wells and piezometers is included in Table 3. When possible, static water level measurements were collected at the time of drilling, approximately 24 hours after construction, and on or about seven days after construction. Water levels were periodically measured for all existing site monitoring wells and Phase 2 investigation piezometers during additional site visits through October 2006.

3.2.2 Soil Testing

Soil samples were collected from each piezometer location during drilling activities. Standard penetration tests were performed at 5-foot intervals for each boring using a 2-foot long split spoon sampler, except where nested piezometer pairs were installed. Soil samples from the split spoons were collected so that select samples could be reviewed at a later date or sent to a soil testing laboratory. Undisturbed soil samples were collected using Shelby Tubes at select boring locations to be analyzed at a soil testing laboratory. Bulk samples were collected from drilling cuttings in materials too dense to be collected using Shelby Tubes at select piezometer locations to be analyzed by a soil testing laboratory.

Soil samples were sent to the Golder Associates Inc. Soil Testing Laboratory in Atlanta, Georgia. Soil samples sent to the laboratory consisted of two Shelby Tubes (P-24 (2'-4') and P-30D (3'-5')) and two bulk samples (P-26S (10'-13') and P-30D (16'-19')). Samples were analyzed for gradation, Atterberg limits, USCS classification, permeability, total porosity, total unit weight, and moisture content. The results of the soil analysis are summarized in Table 4 and the laboratory report is provided as Appendix DH-C.

3.2.3 Hydrogeologic Testing

The groundwater elevations of newly installed monitoring wells were measured at the time of well construction and approximately 24 hours and 7 days after well construction. The groundwater elevations of existing piezometers and newly installed piezometers were measured periodically and data are summarized in Table 3. The measurements were collected using a decontaminated electric water level indicator. The surveyed point on the top of the well casing was used as reference, and the measurements were recorded to within 0.01 foot. The groundwater elevation data collected on July 25, 2006, were used to construct a groundwater surface contour map and to infer groundwater flow directions at the site (Drawing DH-2).

Following rigorous development, six piezometers (P-23, P-25, P-26S, P-27, P-30S, and P-30D) were chosen for aquifer testing (slug tests) during the field investigation. The purpose of the testing was to assess the values of horizontal hydraulic conductivity at various hydrogeologic units within the aquifer at the site. In situ rising- and falling-head slug tests were chosen for the assessment due to the relatively low well yields noted during well installation and development.

Slug tests were conducted on September 20, 28, and 29, 2006, and October 12, 2006, by experienced Golder representatives. Rising- and falling-head tests were performed on piezometers P-23, P-25, P-26S, P-27, and P-30D. Prior to slug testing, the piezometers were opened and groundwater levels were allowed to equilibrate. Water level measurements were then collected using an electric water level indicator referenced to a point on the top of casing. A 15 pounds per square inch (psi) pressure transducer was then lowered inside the well casing and placed 10 to 15 feet below the top of the water table. A polyvinyl chloride (PVC) slug measuring 5 feet in length was then used to displace water inside the piezometers.

The first portion of the test was a falling-head test that measured the rate water levels fell back to static levels after the injection of the PVC slug. The transducer recorded water level data from the transducer at logarithmic time intervals. Data from the transducer were verified with hand

held water level measurements. Falling-head tests were terminated after water levels had recovered to within at least 99% of their pre-test level. A rising-head test was performed on each well after the falling-head test was completed. The rising-head test was performed with the same methodology as the rising-head test with the exception that the PVC slug was removed simultaneously with the start of the test.

In situ rising- and falling-head tests provide a quantitative estimate of horizontal hydraulic conductivity and a qualitative estimate of aquifer anisotropy in water-bearing units. The slug test data were analyzed using the Bouwer and Rice (1976 and 1989) equation, which is applicable to fully or partially penetrating wells in unconfined or confined aquifers. Piezometer specific aquifer thicknesses of 12.6 to 17.6 feet were assumed for the partially weathered rock (PWR) hydrogeologic unit based upon information available from the boring logs. Aquifer thicknesses of 25-50 feet were assumed for the bedrock hydrogeologic unit. A Computer software AQTESOLV, produced by HydroSOLVE, Inc., was utilized to assist in the analysis and plotting of data. The individual data points and computer plots of time versus groundwater displacement are presented in Appendix DH-D. A summary of aquifer testing performed at the site is included in Table 5. A geometric mean for hydraulic conductivity values for each of the hydrogeologic units was calculated as shown on Table 5.

4.0 REGIONAL AND LOCAL GEOLOGY

This section describes the geology in the region of the landfill and site specific geological conditions that may affect the performance of engineered features upon the land surface in the proposed cell area, the groundwater flow regime, and the ability to effectively monitor water quality at the site.

4.1 Regional Geology

The WCA of High Point C&D Landfill is located within the Carolina Slate Belt, which is in the Central Piedmont physiographic province. A portion of 1985 State Geologic Map produced by the North Carolina Geological Survey showing the area around the WCA of High Point C&D Landfill is provided as Figure 2. Geologic unit designations and descriptions used in this report are defined on the *Geologic Map of Region G, North Carolina* (P. Albert Carpenter, III, 1982) and in *The Central Piedmont* (Butler and Secor, 1991).

Generally, the regional geology of the Carolina Slate Belt consists of Late Proterozoic to Cambrian volcanic and greenschist-facies metasedimentary rocks intruded by plutonic suites (Butler and Secor, 1991). The site geology is characterized by a regional-scale felsic intrusive complex, which consists mainly of white to gray, fine to coarse grained, massive to foliated metamorphosed granitic rocks. In several areas granodiorite, quartz diorite, and quartz monzonite are also present. Local shearing is common within Guilford County. The region is characterized by regional and small scale Mesozoic diabase dikes, and may locally be the dominant rock type. Several small gold and copper mines are mapped within the area (P. Albert Carpenter, III, 1982). No evidence of mining activity has been found at the site.

4.2 Local Geology

During the field activities associated with the Site Plan Application performed in 2002, several exposures of massive granite were found on hillsides and in stream exposures. In general, the granite is phaneritic and contains biotite, feldspar, and quartz. A strong preferential foliation was noted in some rock core while only a slight preferential orientation was noted in others. Other outcrops of granite were massive in nature, with very little to no observed structure or foliation. Quartz veins of varying sizes were noted in severely weathered outcrops near the pond. One severely weathered outcrop of metamorphic phyllite was also noted near the pond.

The region is characterized by regional and small scale Mesozoic diabase dikes, although no large scale dikes are mapped within the area in and around the site. Most of the dikes are near vertical in dip and trend north-south to northeast-southwest and are characterized by plagioclase, augite and olivine (Ragland, 1991). The diabase found in site exposures is aphanitic and black to green depending on the degree of weathering.

Three hydrogeologic units have been identified on site. They consist of the saprolite, PWR, and bedrock. For the purpose of this investigation, the saprolite hydrogeologic unit consists of in-place media below the water table with standard penetration test (SPT) results of less than 50 blows per 6 inches with a split spoon sampler. The PWR hydrogeologic unit is media below the water table with SPT results of greater than 50 blows per 6 inches with a split spoon sampler, but above auger refusal. For this investigation, auger refusal defines the top of bedrock for the site. Therefore, the bedrock hydrogeologic unit is media below the water table and below the depth that auger refusal was encountered. Bedrock elevation contours were constructed based on auger refusal data from on site borings on Drawing DH-3. The hydrogeologic units are described in the sections below.

4.2.1 Saprolite Hydrogeologic Unit

As stated above, the saprolite hydrogeologic unit consists of in-place media below the water table with SPT results of less than 50 blows per 6 inches with a split spoon sampler. Though saprolite appears to be continuous across the proposed Phase 2 area, it appears to have limited extent as a hydrogeologic unit. The only piezometers that encountered the water table in saprolite at the time of drilling and piezometer installation in and around the proposed Phase 2 footprint were the hand-auger piezometers HA-1, HA-3, HA-4, and HA-5. All four of these piezometers are located in a drainage feature east of the proposed phase. The other 17 piezometers installed in and around the proposed Phase 2 area encountered the saprolite unit above the water table. One piezometer (P-23) has a stabilized groundwater elevation that intersects the saprolite hydrogeologic unit. The PWR hydrogeologic unit was not identified during the drilling of P-23. Based on the groundwater elevation collected on July 25, 2006, piezometer P-23, which is screened in the bedrock hydrogeologic unit, had a groundwater elevation that extended approximately 1.6 feet into the saprolite. This is likely the result of a potentiometric head generated from a semi-confined condition within the bedrock hydrogeologic unit and may not be representative of the true groundwater surface; however, the groundwater elevation for this well was included in the groundwater contour map included as Drawing DH-2.

Across the proposed Phase 2 area, saprolite ranges in thickness from 4 to 10.5 feet. The Unified Soil Classification System (USCS) particle-size classification of this media, based on field observations and laboratory testing, consists primarily of silty sand. Clayey sand was determined to be present, based on laboratory testing, near the eastern property boundary. The soil seems to be more developed in the eastern portion of the site with greater variability, but in the area of proposed Phase 2 waste unit the soil seems to be fairly uniform, lacking much vertical and horizontal variability.

During the Site Application investigation, the soils at piezometer P-16 were saturated above the true water table at the time of drilling and continue to have anomalous water level measurements. The groundwater elevation for this well was not included in the groundwater contour map included as Drawing DH-2. Though the boring log for P-16 only indicates silty sand and PWR consisting of silty sand, clay-rich horizons are believed to cause the vertical retention of percolating water as further discussed below in section 5.2. Piezometer P-16 is the only existing well in the proposed Phase 2 area that shows anomalous water level readings. Therefore, the clay-rich horizon is not expected to be continuous and is not considered a separate hydrogeologic unit. Also, it appears that the vertical retention of percolating water has little affect, if any on the overall groundwater flow regime.

During the Site Plan investigation, four samples from borings were analyzed by a soil testing laboratory and are summarized in Table 4. These samples were collected from the Phase 1 area and the eastern portion of the site. During the current investigation, two samples were collected from in or around the Phase 2 area and were analyzed by a soil testing laboratory and are summarized in Table 4 and included as Appendix DH-C. Laboratory soils classifications of two undisturbed (Shelby tube) samples from the current investigation were defined as silty sand utilizing the USCS. Effective porosity calculations were performed utilizing the soil testing results and the textural classification triangle (Johnson, 1967). The effective porosity calculations of 23 and 24 percent are similar to those typically assigned to Piedmont saprolite of approximately 0.20 (Harned and Daniel, 1989). Laboratory derived permeabilities for the samples were 4.5E-06 centimeters per second (cm/sec) and 2.3E-04 cm/sec.

Aquifer tests for one piezometer (P-2) screened in saprolite hydrogeologic unit were performed during the Site Plan investigation. Though this piezometer is not in the proposed Phase 2 area, the hydrogeologic properties are assumed to be similar. Hydraulic conductivities are expected to be on the order of E-05 to E-04 cm/sec, based on slug test data from P-2 that is summarized in Table 5. The geometric mean of P-2's rising- and falling-head tests is 4.8E-05 cm/sec.

Groundwater within the saprolite hydrogeologic unit in and around the Phase 2 footprint is expected to be stored and transmitted in pore spaces, and the material can be reasonably described as hydraulically isotropic. The saprolite hydrogeologic unit is considered to represent a limited hydrogeologic unit for the area in and around the proposed disposal area.

4.2.2 Partially Weathered Rock Hydrogeologic Unit

As stated above, the PWR hydrogeologic unit is media below the water table with SPT results of greater than 50 blows per 6 inches with a split spoon sampler, but above auger refusal. The

PWR hydrogeologic unit is highly variable across the proposed Phase 2 area. The unit was not identified in borings P-23 or P-29 and ranges in thickness across the Phase 2 area from 3 to 33.5 feet in piezometers that did encounter the unit. Only three borings (P-26S, P-30S, and P-31) encountered PWR below the water table during drilling and piezometer construction. These piezometers are located near the drainage feature east of the proposed phase and have thicker than average PWR zones. Two additional piezometers (P-25 and P-28) have stabilized groundwater elevations that intersect the PWR hydrogeologic unit. Based on the groundwater elevations collected on July 25, 2006, piezometers P-25 and P-28, which are screened in the bedrock hydrogeologic unit, had groundwater elevations that extended approximately 1.4 and 0.5 feet into the PWR, respectively. This could be the result of a potentiometric head generated from a semi-confined condition within the bedrock hydrogeologic unit and not representative of the true groundwater surface; however, the groundwater elevations for these wells were included in the groundwater contour map included as Drawing DH-2.

During the Site Plan investigation, four samples from borings were analyzed by a soil testing laboratory and are summarized in Table 4. Three of these samples were collected from outside of the Phase 2 area; the sample collected from the P-14 boring was collected from within the proposed waste unit. During the current investigation, two samples were collected from in or around the Phase 2 area and were analyzed by a soil testing laboratory and are summarized in Table 4 and included as Appendix DH-C. The two samples collected during the current investigation were collected as bulk samples because the material was too dense to collect undisturbed samples. The USCS particle-size classification of PWR, based on field observations and laboratory testing, consists mainly of silty sand with some locations classified as silty clay or clayey sand. Effective porosity calculations were performed utilizing the soil testing results and the textural classification triangle (Johnson, 1967). The effective porosity calculations for samples collected in and around the proposed Phase 2 footprint for the PWR hydrogeologic unit range from 13 and 28 percent. Laboratory derived permeabilities for those samples were 5.8E-05 cm/sec and 7.9E-04 cm/sec.

An aquifer test for one piezometer (P-7) screened in the PWR hydrogeologic unit was performed during the Site Plan investigation. Though this piezometer is not in the proposed Phase 2 area, the hydrogeologic properties are assumed to be similar based on proximity and similar soil types. During the current investigation, aquifer tests for two piezometers in and around the proposed Phase 2 footprint were performed. Hydraulic conductivities for the PWR hydrogeologic unit are expected to be on the order of E-05 cm/sec with a geometric mean of 2.1E-05 cm/sec (Table 5).

Based on field observations, laboratory soils testing, and aquifer tests, the composition of the PWR hydrogeologic unit is similar to the predominantly silty sands that overlay it in the saprolite hydrogeologic unit. Therefore, the PWR unit is difficult to distinguish from the saprolite unit based solely on its soil classifications and hydraulic properties. However, it is visually distinct from the saprolite unit due to the prominent granitic fabric and fresh feldspar, quartz, and biotite grains observed in drill cuttings and soil samples. Also, a significant increase in blow counts is associated with the transition into the PWR unit as seen in the boring logs in Appendix DH-A.

Groundwater within the PWR hydrogeologic unit in and around the Phase 2 footprint is expected to be stored and transmitted in pore spaces, and the material can be reasonably described as

hydraulically isotropic. The saprolite may have areas that exhibit relict foliation and these structures can result in locally anisotropic groundwater flow directions. Though the PWR hydrogeologic unit is limited in vertical and lateral extent across the proposed Phase 2 area, the unit is considered to represent an important hydrogeologic unit for the area in and around the proposed disposal area, especially at the compliance boundary.

4.2.3 Bedrock Hydrogeologic Unit

As stated in Section 4.2, the bedrock hydrogeologic unit is media below the water table and below the depth that auger refusal was encountered. The uppermost aquifer for much of the proposed Phase 2 area is in the bedrock hydrogeologic unit. Of the 21 piezometers installed in and around the proposed Phase 2 waste unit, 7 piezometers were screened in a hydrogeologic unit other than bedrock. Bedrock is shallow in the proposed Phase 2 footprint with top of bedrock depths ranging from 5 to 20 feet below ground surface. In the low lying drainage areas east and south of the proposed Phase 2 area, the depth to bedrock increases to 28 to 43 feet below ground surface as groundwater transitions to the saprolite and PWR hydrogeologic units, based on boring logs for P-26S/D and P-30S/D piezometer pairs.

During the field work associated with the Site Plan investigation, geologic mapping of the site did not reveal significant fracture patterns, fabrics, or joint patterns in rock outcrops that would cause preferential groundwater flow. Some horizontal fracture patterns were noted in outcrops, which is to be expected in massive plutonic complexes.

During the Site Plan investigation, bedrock was cored at five boring locations. One of these rock coring locations (P-10) is located in the proposed Phase 2 footprint. During the current investigation, rock coring was performed at two locations in the proposed Phase 2 area at piezometers P-26D and P-30D. Based on the data from the three rock core locations in and around the proposed Phase 2 footprint, the upper portion of the bedrock hydrogeologic unit is highly variable across the Phase 2 area. At piezometers P-10 and P-26D, the PWR unit transitions quickly to bedrock within 10 to 18 feet below where PWR was first encountered. At P-30D, the PWR unit is 33.5 feet thick based on auger refusal, but bedrock continues to be highly weathered through the 27 feet of bedrock that the boring was advanced through. This is supported by the rock quality designation (RQD) acquired from each core run performed in the borings. Core runs at piezometers P-10 and P-26D had RQDs that range from 89 to 100 percent. The RQDs at P-30D range from 31 to 82 percent. A summary of all rock core data collected at the site is provided on Table 2.

The composition of the bedrock is very similar at the three rock core locations in and around the proposed Phase 2 footprint. The prominent rock type is granite composed of quartz, feldspar, and biotite. All three locations show a preferential orientation of biotite, which typically dips approximately 55 degrees. Photographs of the rock core are provided as Figure 3. Fractures commonly occur along this preferential orientation, especially in the more competent rock. In the highly weathered bedrock, fractures may sometimes cross the preferential orientation of biotite at a nearly perpendicular angle. As observed in rock cores, weathered bedrock is typically highly fractured, while competent bedrock has very few fractures, which is typical of bedrock in the Piedmont terrane.

An effective porosity of 9 percent is assumed for the bedrock hydrogeologic unit based on Heath, 1982. Aquifer tests of four piezometers screened in the bedrock hydrogeologic unit were performed during the Site Plan investigation. Only one of the piezometers is located in the proposed Phase 2 footprint, but the hydrogeologic properties of the other piezometers are assumed to be similar to those in the Phase 2 area based on proximity. During the current investigation, aquifer tests of four piezometers in and around the proposed Phase 2 footprint were performed. Hydraulic conductivities for the bedrock hydrogeologic unit are expected to be on the order of E-06 cm/sec and E-05 cm/sec with a geometric mean of 9.5E-06 cm/sec (Table 5). Hydraulic conductivities of the fractured bedrock unit are expected to fall within this range, and vary locally with fracture intensity. Hydraulic conductivities for the bedrock aquifer are expected to be low due to the generally highly competent nature of the granite that underlies the site, as described below.

Based on the data provided above, groundwater flow through the bedrock hydrogeologic unit is expected to be limited along the topographic highs were bedrock becomes competent relatively quickly below auger refusal, as seen at P-10 and P-26D. Groundwater flow is expected to increase along the slopes of drainage features where bedrock is highly weathered and the groundwater surface transitions to the overlying hydrogeologic units.

4.2.4 Diabase Dikes

In May 2002 during the Site Application field work, a magnetometer survey was performed at the site as part of the Site Plan investigation. This geophysical survey included the proposed Phase 2 area. A large area with magnetic anomalies was identified trending northeast through the center of the site. Though other magnetic anomalies were identified, this was the only one that transected the proposed Phase 2 waste unit, as seen on Drawing DH-3. These anomalies are interpreted as diabase dike swarms, as opposed to large-scale diabase dikes. This interpretation is based on field observation of diabase dike outcrops in the tributary of Richland Creek along the southern property boundary, which occurred in swarms. Also, granite outcrops were identified in areas with magnetic anomalies, showing the inconsistent nature of the features. The diabase dike swarm interpretation is supported by the shape of the anomalies. The magnetic anomalies shown on Drawing DH-3 have somewhat random shapes and do not appear linear.

Diabase was encountered in seven of the piezometer borings (P-10, P-11, P-23, P-26D, P-29, P-30S, and P-30D) during drilling in and around the proposed Phase 2 footprint. Only two of those borings (P-11 and P-29) are located in the area of the magnetic anomalies shown on Drawing DH-3. In each boring that encountered diabase, the utra-mafic rock appeared in relatively thin zones that range from 1 to 5 feet thick. Borings P-11, P-26D, and P-30D encountered multiple zones of diabase supporting the interpretation that the diabase dikes on site appear in swarms. Based on the rock cores, the diabase appears to weather at a similar rate as the country rock it intersects.

It is interpreted that diabase dikes occur across the Phase 2 area as small, discontinuous, swarms within the granite. Significant fractures associated with the diabase dikes is not expected, based on the small size of the dikes, and if present they are expected to have little impact on

groundwater flow. This is supported by the groundwater surface contour map (Drawing DH-2) and the cross-sections (Drawing DH-5), which do not show any mounding of groundwater that could result from impenetrable zones of diabase or preferential flow that could result from fracture zones in the granite formed during the intrusion of the dikes.

5.0 SITE HYDROGEOLOGY

The following section discusses the hydrogeologic aspects of the site, with emphasis on the proposed Phase 2 area. A significant amount of hydrogeologic investigation was performed in and around the proposed phase 2 footprint. During the original Site Plan investigation, 4 piezometers (P-9, P-10, P-14, and P-16) were installed in the proposed Phase 2 footprint. Two additional piezometers (P-8 and P-11) were installed in close proximity to the proposed footprint. Rock core was collected in 7 piezometer borings, including 1 in the proposed Phase 2 footprint. Also during that investigation, a geophysical survey for diabase dikes and test pits were dug across the site, which included the Phase 2 area. During the Phase 2 hydrogeologic investigation, 14 additional piezometers were installed in and around the proposed Phase 2 footprint and rock core was collected from 2 of these borings.

5.1 Hydrology and Discharge Features

As discussed in section 2.0, topography at the site consists of knolls and valleys that range from 700 to 820 feet above mean sea level as seen on Figure 1. Surface water and groundwater at the site flows south to southwest towards a tributary of Richland Creek. Richland Creek is approximately 1300 feet south of the site and flows to the Deep River, which is approximately 0.5 miles from the site. This tributary is approximately 1300 feet south of the property line and the Deep River is approximately 0.5 mile east of the site. Groundwater beneath the site flows in three distinguishable and vertically interconnected hydrogeologic units; saprolite, PWR, and bedrock.

5.2 Unsaturated Zone of Water Accumulation

During the 2002 field investigation, unusually high groundwater elevations were measured in several piezometers (P-4A, P-12, P-13S, P-16, and P-17). In order to explain these anomalous measurements, additional piezometers were installed and test pits were excavated adjacent to piezometers with anomalous groundwater measurements.

During the installation of the test pits, water seeped in from depths of approximately 3 to 5 feet below ground surface. The origin of the water often coincided with the occurrence of orange/gray mottled clayey sand or sandy clay horizons. In most of the test pits that encountered water, the water entered the excavations at depths above the anticipated stabilized water table. The water levels in many of the test pits and piezometers dropped significantly over time. The anomalous water levels are believed to be the result of the accumulation of water percolating through the unsaturated zone due to the low permeability of clay-rich saprolitic horizons.

Piezometer P-16, one of the wells with anomalous groundwater measurements, is located in the Phase 2 area. During the current investigation, groundwater measurements of P-16 are still

elevated compared to surrounding piezometers. During the drilling investigation in 2006, the shallow clayey horizons were not encountered and no anomalous water levels were recorded in the newly installed piezometers. Therefore, the clayey horizons do not appear to be continuous across the Phase 2 area. It is believed that much of the accumulated water was drained during the excavation of these soils during construction of the Phase 1 waste unit.

5.3 Groundwater Flow Regime

As previously discussed, shallow groundwater flow (upper 50 feet) occurs in 3 vertically interconnected hydrogeologic units: saprolite, PWR, and bedrock. The groundwater surface based on water level measurements taken on July 25, 2006, is shown on Drawing DH-2 and the historical groundwater surface elevations are summarized in Table 3. Groundwater flow arrows on Drawing DH-2 and site-specific data were used to determine groundwater flow velocities. Several hydrogeologic cross-sections are provided to illustrate the vertical component of groundwater flow on Drawings DH-4 and DH-5. Groundwater in all three hydrogeologic units beneath the proposed Phase 2 waste unit flows towards and discharges to streams along the western and southern property boundaries and discharge to the Deep River as seen on Figure 1.

Given the competence of the bedrock unit along the knolls, it is expected that a significant amount of *interflow* occurs in the unsaturated zone as discussed in Fetter, 1988. Based on site-specific hydrogeologic characteristics, groundwater is expected to move laterally more than vertically, thus limiting the amount of aquifer recharge occurring in the bedrock unit in and around the proposed Phase 2 footprint.

5.3.1 Saprolite Hydrogeologic Unit

As discussed in section 4.2.1, the saprolite hydrogeologic unit is limited in lateral extent in the proposed Phase 2 area. The only area identified where groundwater was located in saprolite is in the drainage feature east of the proposed waste unit. The saprolite hydrogeologic unit is expected to behave as a relatively isotropic, porous medium, where groundwater flow directions are expected to mimic topography. Locally, variation in soil type, clayey horizons, and hydrogeologic structures may cause an irregular groundwater surface and/or flow paths.

5.3.2 Partially Weathered Rock Hydrogeologic Unit

As discussed in section 4.2.2, the PWR hydrogeologic unit is limited in the proposed Phase 2 area. The unit was not identified in two borings (P-23 and P-29) in the Phase 2 investigation area, and only three borings (P-26S, P-30S, and P-31) encountered PWR below the water table during drilling and piezometer construction. These piezometers are located near the drainage feature east of the proposed phase and have thicker than average PWR zones. Two additional piezometers (P-25 and P-28) have stabilized groundwater elevations that intersect the PWR hydrogeologic unit, but this could be the result of a potentiometric head generated from a semi-confined condition within the bedrock hydrogeologic unit and may not be representative of the true groundwater surface as described above.

The upper part of the PWR hydrogeologic unit is expected to behave as a relatively isotropic, porous medium, where groundwater flow directions are expected to mimic topography. With increasing depth, groundwater flow within the PWR is expected to occur in fractures. Based on rock cores within the PWR unit and upper bedrock unit, the lower PWR is expected to be highly fractured with random orientations. The rock cores and split spoon samples do not indicate a prominent fabric or structure are not likely to control groundwater flow direction. Therefore, groundwater within the lower portion PWR unit is expected to mimic topography.

5.3.3 Bedrock Hydrogeologic Unit

As discussed in section 4.2.3, bedrock is the prominent hydrogeologic unit underneath the proposed Phase 2 footprint. Groundwater flow in the bedrock hydrogeologic unit occurs in fractures. Based on rock cores, the bedrock is expected to be highly fractured with random orientations on the side-slopes leading to drainage features, where the top of groundwater is transitioning from the bedrock to the overlying units. Groundwater in these highly weathered, highly fractures areas is expected to mimic topography.

On the knolls, competent bedrock occurs at relatively shallow depths with limited fractures. Fractures are typically aligned with preferential biotite orientations, which seem to vary laterally. As noted in section 4.2.2, the groundwater elevations measured from piezometers screened in the bedrock hydrogeologic unit may be slightly elevated, representing potentiometric heads rather than the groundwater surface. These effects are expected to be minimal with bedrock fractures interconnecting with the overlying units causing groundwater flow to mimic topography.

5.3.4 Horizontal Gradient Calculations

Horizontal hydraulic gradients for the proposed Phase 2 area were calculated using gradient segments from the July 25, 2006, groundwater surface contours shown on Drawing DH-2. Groundwater flow within the proposed Phase 2 area occurs predominantly in the bedrock hydrogeologic unit; groundwater flow in the monitoring zone downgradient of Phases 1 and 2 occurs predominantly above bedrock).

Hydraulic conductivity estimates were obtained from single well aquifer tests (slug tests), which are summarized on Table 5. Hydraulic conductivity values generated by slug test recovery data on individual wells are considered to be higher than actual conductivities for site-scale movement of groundwater. Curves generated during the analysis of slug test recovery data are typically impacted by near-field, or skin effects, and are reflective, in part, of the properties of the well screen filter pack judged to have greater hydraulic conductivity than the surrounding in-situ soils. The hydraulic conductivities for saprolite range from 1.9E-05 to 1.2E-04 cm/sec, with a geometric mean of 4.8E-05. The hydraulic conductivity for PWR ranges from 1.1E-05 to 4.7E-05 cm/sec, with a geometric mean of 2.1E-05. The hydraulic conductivity for bedrock ranges from 1.2E-06 to 4.8E-05 cm/sec, with a geometric mean of 9.5E-06. The geometric mean of the three hydrogeologic units combined is 1.4E-05 cm/sec.

The average horizontal gradients for gradient calculation segments along with the geometric mean for available hydraulic conductivity values were used to estimate linear groundwater flow

velocities. Average linear groundwater flow velocities were computed using the following modified Darcy equation:

$$V = Ki/n_e$$

where V = average linear velocity (feet per day), K = hydraulic conductivity (ft/day), i = horizontal hydraulic gradient, and n_e = effective porosity. Effective porosity values were calculated for each of the three hydrogeologic units, as described in sections 4.2.1 through 4.2.3 and shown on Table 4.

Horizontal gradients for the proposed Phase 2 area are presented in Table 6. The calculated horizontal gradients from the groundwater flow segments on Drawing DH-2 range from approximately 0.05 to 0.06 feet per feet. Groundwater velocities were calculated using a hydraulic conductivity of 1.4E-05, which is the geometric mean of the hydraulic conductivities for each of the hydrogeologic units. The mean hydraulic conductivity represents a conservative estimate of groundwater flow, given the amount of groundwater flow within bedrock in the Phase 2 area. An effective porosity of 9 percent was used in the calculations, which is the estimated effective porosity for bedrock. The bedrock effective porosity was used in order to obtain conservative velocity estimations. Calculated linear groundwater velocities are approximately 7.9 to 8.8 feet per year.

Though the average linear groundwater flow velocity was calculated assuming that the majority of groundwater occurs in the bedrock hydrogeologic unit, the values calculated utilizing the above equation are not considered valid for groundwater flow in fractured media (Parker et al., 1994). Groundwater flow velocities within the fractured bedrock are likely to be highly variable and may or may not be similar to the calculated values.

5.3.5 Vertical Gradient Calculations

Three nested well or piezometer pairs (MW-4S/D, P-26S/D, and P-30S/D) were utilized for purpose of measuring a vertical hydraulic gradient. Piezometer pair MW-4S/D is located downgradient of the Phase 1 waste unit near the confluence of the two main drainage features at the site. Piezometer pair P-26S/D is located upgradient of proposed Phase 2 less than 20 feet from the proposed waste boundary. Piezometer pair P-30S/D is located downgradient of Phase 2 footprint on a steep slope above a south trending drainage feature that bisects the southern portion of the facility. Calculations were performed using water levels measured on three different dates. Vertical gradients were calculated using the vertical distance between the screen midpoints of the pairs. If the water table intersected the well or piezometer screen, the screen midpoint for the shallow well was based on the portion of the screen that was below the water table for each given date. Vertical gradients calculated for MW-4S/D indicate a strong upward gradient and therefore suggest that groundwater discharge is taking place in this area, as would be anticipated given the size and flow of the adjacent drainages. The vertical gradients calculated for P-26S/D indicate a strong downward gradient and therefore suggest that groundwater recharge is occurring in this area as expected on the topographic high. The vertical gradients for P-30S/D indicate a slight downward gradient as would be expected as the aquifer transitions to an upward gradient at the drainage at the bottom of the slope. A summary of

vertical gradient data is presented in Table 7. Hydrogeologic cross-sections are provided to illustrate the vertical component of groundwater flow as shown on Drawings DH-4 and DH-5.

6.0 VERTICAL SEPARATION, RECEPTOR AND MONITORING CRITERIA

The following paragraphs discuss the vertical separation, receptor and monitoring criteria for the proposed Phase 2 C&D waste unit.

6.1 Relationship of Hydrogeology and the Proposed Phase 2 Waste Disposal Unit

A sufficient number of piezometers have been installed in and around the proposed Phase 2 footprint and soil samples and rock cores have been collected and evaluated to adequately characterize the hydrogeology of the area. The hydrogeologic characteristics of the site are typical of the Piedmont and no conditions were encountered that will require unusual monitoring requirements.

6.1.1 Vertical Separation from Bedrock

The top of bedrock elevations are based on the criteria described in section 4.2.3. Bedrock surface contours were generated from these elevations and are provided as Drawing DH-3. Proposed base grade contours of the Phase 2 waste unit are also provided on Drawing DH-3 in order to evaluate vertical separation from bedrock requirements. The proposed base grades were designed to have a minimum separation of 4 feet above the top of bedrock. Bedrock is generally above groundwater within the proposed Phase 2 footprint, except for areas described in section 4.2.2. The vertical separation of the proposed base grades for Phase 2 from bedrock is also shown on the hydrogeologic cross-sections provided as Drawings DH-4 and DH-5.

6.1.2 Vertical Separation from Seasonal High Groundwater Elevations

At the time design base grades for the proposed Phase 2 area were finalized, a limited amount of historical groundwater elevation data were available for the newly installed piezometers. Because monthly groundwater elevation data are not available for any of the facility monitoring wells and piezometers, the maximum groundwater elevation recorded for each well or piezometer was assumed to be equal to the average seasonal high groundwater elevation. The maximum groundwater elevation for each well and piezometer is shown on Table 9.

Long-term seasonal high groundwater elevations for the piezometers were estimated using the historical groundwater elevations of existing site monitoring wells and piezometers with at least 3 years of groundwater elevation data. The potentiometric surface of on-site wells and piezometers has historically shown limited variability as shown on Figure 4. Precipitation data for the area was reviewed for trends and compared to the groundwater elevation data as described below.

The long-term seasonal high groundwater calculations are based on historical groundwater elevations available from the site wells and piezometers with at least 3 years of groundwater

elevation data (Table 3). The historical groundwater elevation data were examined and evaluated with respect to the average precipitation for Guilford County, North Carolina (Piedmont Triad International Airport), for the same time period (Table 8). A graphic representation of the precipitation data since 2002 is shown on Figure 5. A comparison of the precipitation data on Figure 5 with the groundwater elevation data on Figure 4 shows that periods of greater precipitation typically result in minimal fluctuations of groundwater elevations. The average precipitation for 2002-2005 is slightly below the long-term average for Guilford County, based on data from 1931-2005, as shown on Table 8. Every month since June 2006, when the new piezometers were installed, has been above each month's historical average except for October, which was approximately equal to the average.

Each of the newly installed piezometers in and around the proposed Phase 2 footprint was compared to the wells installed in 2002, by grouping the wells and piezometers based on similar topographic settings and proximity to drainage features. The maximum groundwater elevation for each well or piezometer with at least 3 years of groundwater elevation data was subtracted from the July 25, 2006, groundwater elevation for each respective well or piezometer. The difference was rounded up to the nearest whole number as the long-term seasonal high adjustment. The long-term seasonal high adjustment values ranged from 1 to 5 feet. That number was added to the each respective well or piezometer's July 25, 2006, groundwater elevation. The long-term seasonal high adjustment values were then added to the newly installed piezometers with similar topographic and hydrogeologic settings. The long-term seasonal high groundwater elevation estimations are shown on Table 9.

The average and long-term seasonal high groundwater elevations were then contoured and included as Drawings DH-6 and DH-7. Water levels from piezometer P-16 were not included in the average and long-term seasonal high groundwater surface maps because of anomalous groundwater elevations as described in section 5.2 above. Ground surface elevations were used as average and long-term seasonal high groundwater elevations for the hand auger piezometers (HA-1, HA-3, HA-4, and HA-5). Proposed cell base grades are at least 4 feet above the estimated long-term seasonal high groundwater elevations, as seen on Drawing DH-6 and in the hydrogeologic cross-sections on Drawing DH-5.

6.2 Potential Groundwater Receptors

One unoccupied house exists in the northeastern portion of the site property. The house has an associated water well that is not currently in use (identified as WW-1 on Drawing DH-1). Well WW-1 will be decommissioned prior to or during construction of disposal phases within a 500-foot radius of the well. The site is supplied water from the City of High Point. One residence located approximately 1,200 feet downgradient of the landfill is believed to be supplied by well water. Other nearby residences are located upgradient of the waste disposal area as described below.

Several residences are located on the north side of Riverdale Drive. Public water supply is available to these homes, but its use cannot be assumed. Some of these residences are known to be supplied by water from the City of High Point. The remaining residences must be considered

potential receptors. These houses are located upgradient of the proposed Phase 2 footprint and are well outside of the 500-foot buffer zone.

A subdivision consisting of approximately 14 residential homes is located northwest of the proposed site. Each residence is supplied water from the City of High Point. Based on the groundwater flow paths on Drawing DH-2, it is not anticipated that groundwater would flow towards the subdivision. An agreement between the owner of the subdivision and WCA has been made that the residences within 500 feet of the landfill's waste footprint will be demolished as required.

During the original Site Plan investigation, continuous water levels were taken from piezometer P-1. Based on the data collected, it was determined that the aquifers within the site are not affected by any significant local pumping from private potable wells or industrial pumping of groundwater from a nearby quarry. No appreciable decreases or increases in water levels were noted for this well other than precipitation-related changes.

6.3 Ability to Effectively Monitor Groundwater

No significant sources of potential contamination were identified within 2000 feet of the property boundary during the Site Plan investigation or during the current investigation. Groundwater downgradient of the existing and proposed disposal areas will be monitored by the well network described in the *Water Quality Monitoring Plan* included as Appendix DH-E.

No known physical or hydrogeologic conditions have been identified that effect the monitorability of the proposed Phase 2 waste unit. The discharge feature along the southern facility boundary serves as an effective groundwater flow boundary, as does the drainage feature east of the Phase 2 footprint. Surface water monitoring points are included in the current and proposed monitoring program, as well as additional groundwater monitoring points to detect a potential release from the waste unit.

7.0 CONCLUSIONS

The purpose of the Hydrogeologic Report is to present the assessment of geologic and hydrogeologic characteristics of the proposed Phase 2 footprint. Furthermore, the report must present data in support or non-support of the proposed Phase 2 expansion's use for C&D waste management activities. This report presents the current understanding of the groundwater flow regime and the relationship of the solid waste management units to groundwater receptors and groundwater discharge features.

The geologic and hydrogeologic data indicate that the site is located within typical Piedmont terrane. It appears that the amount of interflow inferred for this area relatively high, and will serve to reduce the infiltration of groundwater into bedrock. Depths to groundwater and bedrock are well defined in and around the site. Few site-specific preferential fractures were identified in the rock core or outcrops during this study. Diabase, where present, occurs in swarms, which will severely limit its affect on groundwater flow. The hydrogeologic conditions are well-

defined and the report demonstrates that the waste unit can be effectively monitored for potential releases.

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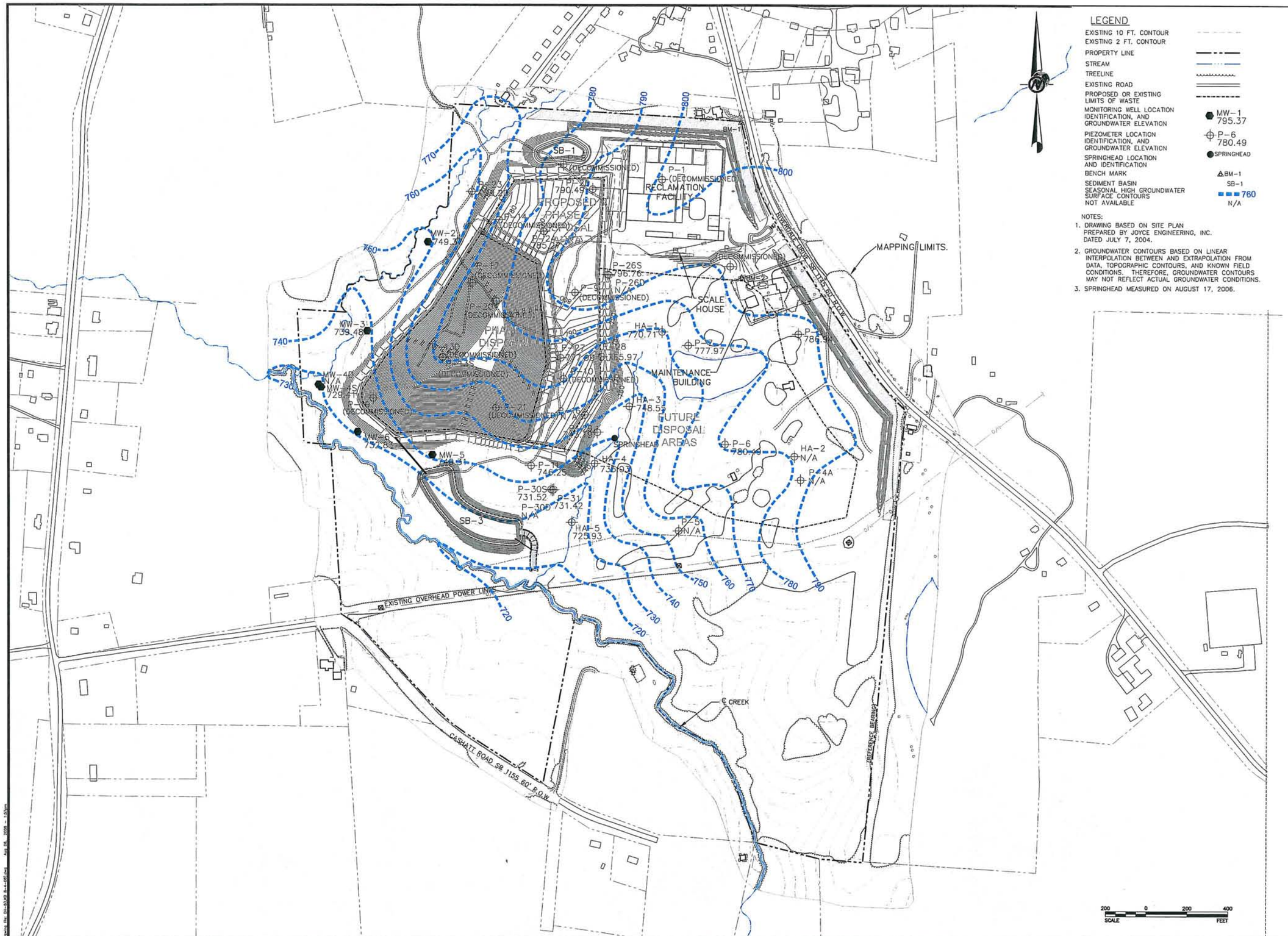
TABLE 9
Estimated Average Seasonal High and Long-Term Seasonal High Groundwater Elevation Calculations
WCA of High Point, LLC

WELL/PIEZOMETER IDENTIFICATION	MW-1	MW-2	MW-3	MW-4S	MW-5	MW-6	P-3	P-6	P-7	P-11
MAXIMUM GW ELEVATION/AVERAGE SEASONAL HIGH GW ELEVATION	795.37	749.37	739.48	729.41	740.31	732.83	786.94	780.49	777.97	746.25
7/25/2006 GW ELEVATION	793.43	749.37	737.41	728.59	735.59	727.84	785.26	778.64	776.00	746.18
MAXIMUM - 7/25/06	1.94	0.00	2.07	0.82	4.72	4.99	1.68	1.85	1.97	0.07
ADJUSTMENT FOR LONG-TERM SEASONAL HIGH	2.00	1.00	3.00	1.00	5.00	5.00	2.00	2.00	2.00	1.00
LONG-TERM SEASONAL HIGH GW ELEVATION	**795.43**	**750.37**	**740.41**	**729.59**	**740.59**	**732.84**	**787.26**	**780.64**	**778.00**	**747.18**

WELL/PIEZOMETER IDENTIFICATION	P-23	P-24	P-25	P-26S	P-27	P-28	P-29	P-30S	P-31
MAXIMUM GW ELEVATION/AVERAGE SEASONAL HIGH GW ELEVATION	759.20	785.27	790.49	796.76	777.99	765.97	743.19	731.52	731.42
7/25/2006 GW ELEVATION	759.20	784.90	790.43	795.18	777.99	765.60	742.94	731.52	731.42
MAXIMUM - 7/25/06	0.00	0.37	0.06	1.58	0.00	0.37	0.25	0.00	0.00
ADJUSTMENT FOR LONG-TERM SEASONAL HIGH	1.00	2.00	2.00	2.00	2.00	2.00	2.00	2.00	2.00
LONG-TERM SEASONAL HIGH GW ELEVATION	**760.20**	**786.90**	**792.43**	**797.18**	**779.99**	**767.60**	**744.94**	**733.52**	**733.42**

NOTES:
1. GW = Groundwater
2. Elevations are in feet above mean sea level.
3. Average seasonal high groundwater elevations are assumed to be equal to the maximum groundwater elevation recorded for each well.
4. The adjustments for long-term seasonal high groundwater elevations were determined by evaluating groundwater fluctuations in wells and piezometers that had a minimum of 3 years of groundwater elevation data. Wells and piezometers with less than 3 years of groundwater elevation data were compared topographically and spatially with those with 3 or more years of groundwater elevation data and adjusted by a similar value.
5. Seasonal high groundwater elevation estimates for all hand auger piezometers (HA-1 through HA-5) are equal to ground surface.



Golder Associates
GREENSBORO, NORTH CAROLINA

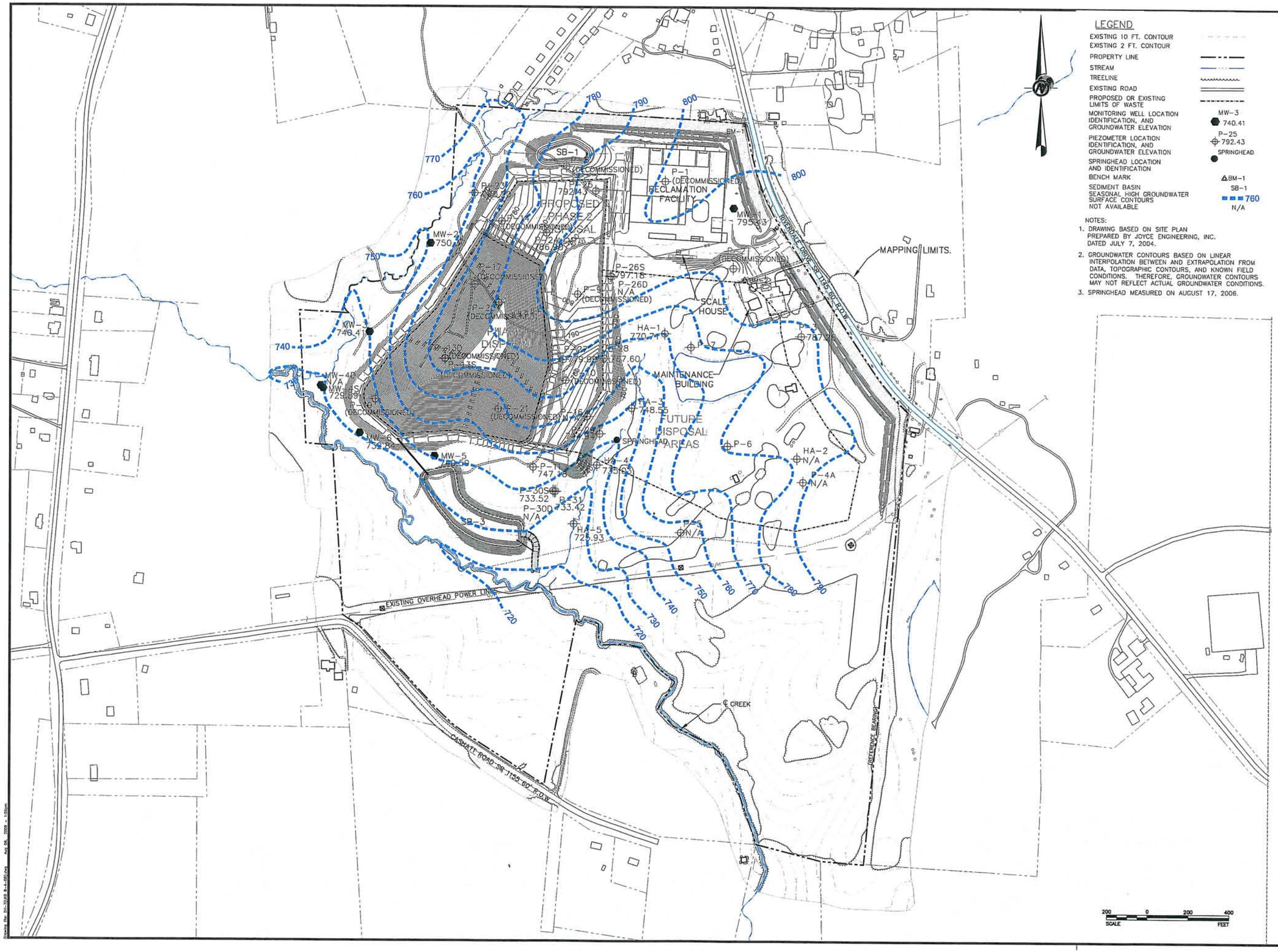
REV	DATE	DES	REVISION DESCRIPTION	CADD	CHK	RVW

PROJECT
WCA OF HIGH POINT
CONSTRUCTION AND
DEMOLITION DEBRIS
LANDFILL AND RECLAMATION
FACILITY
GUILFORD COUNTY, NC

TITLE
AVERAGE SEASONAL HIGH GROUNDWATER SURFACE CONTOUR MAP WITH PROPOSED BASE GRADES

PROJECT No.	063-6526	
FILE No.	0636526A06	
REV.	SCALE	AS SHOWN
DESIGN	DYR	9/25/06
CADD	CLF	10/02/06
CHECK		
REVIEW		

DWG. DH-6



LEGEND

EXISTING 10 FT. CONTOUR	
EXISTING 2 FT. CONTOUR	
PROPERTY LINE	
STREAM	
TREELINE	
EXISTING ROAD	
PROPOSED OR EXISTING LIMITS OF WASTE	
MONITORING WELL LOCATION IDENTIFICATION, AND GROUNDWATER ELEVATION	MW-3 740.41
PIEZOMETER LOCATION IDENTIFICATION, AND GROUNDWATER ELEVATION	P-25 792.43
SPRINGHEAD LOCATION AND IDENTIFICATION	SPRINGHEAD
BENCH MARK	BM-1
SEDIMENT BASIN	SB-1
SEASONAL HIGH GROUNDWATER SURFACE CONTOURS	760
NOT AVAILABLE	N/A

NOTES:

1. DRAWING BASED ON SITE PLAN PREPARED BY JOYCE ENGINEERING, INC. DATED JULY 7, 2004.
2. GROUNDWATER CONTOURS BASED ON LINEAR INTERPOLATION BETWEEN AND EXTRAPOLATION FROM DATA, TOPOGRAPHIC CONTOURS, AND KNOWN FIELD CONDITIONS. THEREFORE, GROUNDWATER CONTOURS MAY NOT REFLECT ACTUAL GROUNDWATER CONDITIONS.
3. SPRINGHEAD MEASURED ON AUGUST 17, 2006.

Golder Associates
GREENSBORO, NORTH CAROLINA

REV	DATE	DES	REVISION DESCRIPTION	CADD	CHK	RVW

PROJECT
WCA OF HIGH POINT
CONSTRUCTION AND
DEMOLITION DEBRIS
LANDFILL AND RECLAMATION
FACILITY
GUILFORD COUNTY, NC

TITLE
LONG-TERM SEASONAL HIGH GROUNDWATER SURFACE CONTOUR MAP WITH PROPOSED BASE GRADES

PROJECT No.	063-6526	
FILE No.	0636526A06	
REV.	SCALE	AS SHOWN
DESIGN	DYR	9/25/06
CADD	CLF	10/02/06
CHECK		
REVIEW		

DWG. DH-7