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ENGINEERING PLAN

WHITE OAK MSW LANDFILL
PHASES 1 – 3

HAYWOOD COUNTY, NORTH CAROLINA

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ENGINEERING PLAN

White Oak MSW Landfill
Phases 3 & 4

This Engineering Plan incorporates the plans and specifications relative to detailed design and performance of the Phase 3 and proposed Phase 4 Expansion at the White Oak MSW Landfill. This certifies that the plan was developed in accordance with Section .1620 of the Solid Waste management Rules (15 NCAC 13B). Sections .1624 (Construction Requirements for MSWLF Facilities) and .1627 (Closure and Post-Closure Requirements for MSWLF Facilities) were also closely referenced during the development of this plan. This Engineering Plan discusses the engineering and design of MSW Phase 3 of the Phases 3 & 4 expansion. Phases 1-3 have an estimated life expectancy of approximately five (5) years.

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ENGINEERING PLAN

White Oak MSW Landfill
Phases 3 & 4

1.0 Introduction

The Engineering Plan incorporates by reference, the original Permit to Construct submittal Plans and Specifications relative to detailed design and performance of the Phases 3 & 4 Expansion at the White Oak MSW Landfill. This plan was developed in accordance with Section .1620 of the Solid Waste management Rules (15 NCAC 13B). Sections .1624 (Construction Requirements for MSWLF Facilities) and .1627 (Closure and Post-Closure Requirements for MSWLF Facilities) were also closely referenced during the development of this plan. This Engineering Plan discusses the engineering and design of Phase 3. Proposed Phase 4 is discussed in this Report due to its proposed design and location influencing the design of Phase 3. Phases 1-3 have an estimated life expectancy of approximately five (5) years.

2.0 Engineering Report

2.1 Composite Base Liner and Leachate Collection System

The Phase 3 Expansion has the following: clay layer, Geosynthetic Clay Liner (GCL), HDPE liner and Leachate Collection System (LCS) design characteristics (from bottom to top):

- A minimum eight (8)-foot vertical buffer between the average seasonal high water table and the top of the composite liner,
- A minimum 6.5-foot vertical buffer between bedrock and the top of the composite liner.
- A compacted select backfill layer
- An eighteen (18)-inch thick compacted clay liner with a maximum permeability of 1.0×10^{-5} cm/sec overlain with a geosynthetic clay liner (GCL) material with a demonstrated hydraulic conductivity of not more than 5×10^{-9} cm/sec under the anticipated confining pressure,
- A primary sixty (60)-mil, high density polyethylene (HDPE) geomembrane liner, smooth on the floor and textured on the slopes,
- A sixteen (16)-once geotextile cushion on the floor and side slopes, and
- A two (2)-foot thick leachate collection layer constructed of washed stone, and also including leachate collection and transfer piping constructed of HDPE.

2.2 Cap System

The Phases 1-3 cell areas will have the following cap system design characteristics (from bottom to top):

- A one (1)-foot thick layer of compacted intermediate cover placed directly on top of the waste,
- An eighteen (18)-inch thick compacted clay layer, with maximum permeability of 1.0×10^{-5} cm/sec
- A forty (40)-mil textured HDPE geomembrane,
- A double-sided, eight (8)-ounce geocomposite, and
- A twenty-four -inch (24") thick vegetative\erosion layer

2.3 Leachate Collection

The Phase 3 leachate collection lines consist of eight (8)-inch perforated HDPE main collector lines which run from south to north near the center of the phase, and a series of six (6)-inch, perforated, HDPE lateral collectors that connect to the main lines. The main collection lines were constructed with a minimum slope of 2.5%. The lateral collectors are spaced approximately 110 feet apart and maintain a minimum grade of 2.8%. See Sheet C4 of the Permit Amendment Drawings for the layout of the leachate collection piping for Phase 3.

Phase 3 was constructed with two (2) leachate collection sumps and pumping stations. Both sumps are located at the lowest point in the phase along the northern margin of Phase 3. Two (2) pumping stations were designed in order to provide redundancy and to increase the safety factor against a pump failure and leachate spill. The proposed sump area consists of an eight (8)-foot by sixty (60)-foot recessed area and is double-lined with HDPE geomembrane. The pumping stations utilize a side-slope riser with a submersible pump.

The sump area contains a section of eighteen (18)-inch perforated HDPE pipe that houses the submersible leachate pump. The submersible pump was lowered into the leachate collection sump through the side-slope riser by means of a steel cable and a rolling carriage. The side-slope riser rests on the 16-ounce geotextile cushion along the slope and lies between the sump and the top of the perimeter berm.

2.4 Leachate Generation

The amount of leachate that Phase 3 and proposed future Phase 4 will generate was estimated by using the HELP Model analysis included in Section 10 of the original Permit to Construct Report for Phases 3 & 4 and by examining landfill records for the past six years. See Section 11 of the original Permit to Construct Report for Phases 3 & 4 for landfill leachate records for the past six years. The HELP model analyzed ten stages of

landfill development during the lifetime of Phases 3 & 4. Each stage of development was comprised of cross-sections of landfill construction, which included clay barrier soils, flexible membrane liners, gravel drainage layers and layers of compacted MSW wastes and cover soils. The stages of development varied from the first, which analyzed the landfill with only a small portion active with relatively thin layers of compacted waste, to the tenth, which analyzed the landfill after the final cap was installed. Stages 1 through 5 occurred during the development of MSW Phase 3 and stages 6 through 10 occurred during the development of MSW Phase 4. The HELP model estimated the average monthly leachate production will reach 596,400 gallons/month during the initial stages of Phase 4 development. Landfill records over a six year study period showed an average monthly leachate generation of approximately 220,000 gallons per month, and 323,500 gallons/month for F.Y. 2005, the wettest year during the past six years. The highest total leachate production for a calendar month was 486,000 gallons in September 2004, although during a 30-day period in September and October of that year, 600,000 gallons of leachate were removed. This anomaly was due to two separate hurricanes, Ivan and Francis, that impacted the area in an approximate two-week period in 2004. The monthly average calculated by the HELP model for 15.5 acres of lined landfill was the same as the greatest actual observed leachate production for a 21.5 acre area of lined landfill. During the six-year time period, there were only 4 months where leachate generation exceeded 400,000 gallons (for 21.5 acres). The HELP Model analysis is a conservative estimate of leachate production and will continue to be considered when designing improvements to the landfill leachate system; however, leachate production records will be utilized as well. The leachate collection system must be prepared to handle extreme events as well as function efficiently at average production levels.

2.5 Average Leachate Production Flows and Pumping

Utilizing the existing leachate production information included in Section 11 of the original Permit to Construct Report for Phases 3 & 4, and pro-rating those quantities over the 15.5-acre footprint of Phase 3 and proposed future Phase 4, it is estimated that Phases 3 and 4 will produce leachate at an average monthly rate of 250,000 gallons. Applying a 1.5 safety factor to this quantity results in an average rate of 375,000 gallons per month. To remove the leachate from the phases, it was assumed that on average each pump would operate for 8 hours per day, which requires that the pumps remove leachate at an average flow of 13 gpm. Applying a peaking factor of 2 to the flow calculations, it is necessary for each pump to remove leachate at the rate of 26 gpm. Therefore, each pump was designed to pump 26 gpm at a total dynamic head (TDH) of 25.5 feet. An additional safety factor was incorporated into the pumping system; therefore, each pump was designed to pump 35 gpm at 25.5 feet total dynamic head (TDH). The pumps utilize a 3" x 6" dual-contained force main located along the northern margin of Phase 3, which runs for approximately 150 l.f. to a point where the force main discharges into the leachate collection system gravity sewer manhole #2. The gravity sewer flows approximately 375 l.f. where it will discharge to the leachate collection system gravity sewer at manhole #1, just upstream of the leachate storage lagoon. See Sheet C18 of the Permit Amendment

drawings for the leachate force main and gravity sewer piping. The force main will accommodate a minimum flow of 35 gpm with a minimum velocity of 2 feet per second. The leachate storage lagoon located north of Phase 1 has a total storage volume of 1,017,000 gallons.

2.6 Contingency Plan For Peak Flow Conditions And Leachate System Disruption

In the event of an extreme rainfall event, the HELP model projected a peak leachate production of 30,950 cubic feet/day, or approximately 231,000 gal/day. This peak event would be realized at Stage II of landfill development within the first two years of operation of Phase 3. Over time, the effects of an intense storm event would be greatly dampened due to the increase in the length of time that it would take stormwater to percolate through the layers of waste and daily cover. The available storage volume below the perimeter liner edge elevation of 2515 is approximately 2,200,000 gallons. During the peak event as described by the HELP model, the landfill can contain leachate for 10 days of peak discharge intensity. It is unlikely that the HELP model 1-day peak intensity rate would continue for 10 days. Therefore it is highly unlikely for the landfill to produce more leachate than the pumps can transport to the leachate lagoon. In the event of extreme conditions, the discharge from the County's stormwater removal pump may be diverted to manhole #2 of the leachate gravity system, which will allow leachate to be pumped to the storage lagoon at approximately 300 gpm.

Under average operating conditions, in the event that there was a temporary failure with any of the leachate removal and storage equipment, the geometry of the landfill would allow for the landfill to contain the leachate for a period of several months. However, the system should never be out of operation for more than a few hours, and then only under extreme circumstances. Due to the conservative design of the leachate removal and storage equipment and the geometry of the landfill, the possibility of leachate buildup overflowing the perimeter berm is very low.

In conjunction with the construction of Phase 3, the existing leachate lagoon was raised and a new liner installed on top of the existing liner. The lagoon was raised by approximately 3.25' in order to provide an additional 375,000 gallons of storage volume. During the construction of the additional storage volume, a new 60-mil textured HDPE liner was installed over the existing liner. A double-sided geocomposite was installed between the existing liner and the proposed liner. The final volume of the lagoon is approximately 1,017,000 gallons.

Section .1624 (2)(A)(ii) of the Solid Waste Management Rules states that "the geometry of the landfill shall be designed to control and contain the volume of the leachate generated by the 24-hour, 25-year storm." The input parameters of the HELP model included maintaining a minimum head of one (1) foot on the geomembrane liner, and the additional calculations were performed to assure that rule .1624 (2)(A)(ii) was met.

2.7 Existing Dual-Contained Gravity Sewer Connection Beneath Phase 3

A portion of Phase 3 was constructed over the existing dual-contained gravity sewer that services the sump areas of existing Phase 2 and existing Phase 1, Cell 4. The existing dual-contained gravity sewer was replaced with new dual-contained gravity sewer to ensure that there was a minimum 4' separation between the bottom of the Phase 3 clay liner and the top of the new sewer. The dual-contained gravity sewer daylights into the HDPE manhole located outside of the Phase 3 liner limits along the access road to Phase 3. The new gravity sewer connects to the existing leachate gravity system near the north margin of the Phase 1 cell, which flows to the existing leachate lagoon.

3.0 DESIGN SUMMARY

3.1 Analytical Methods Used for Design Evaluation

3.1.1 Liner Components.

Review of EPA literature, NCDENR regulations, and Geomembrane liner manufacturer data.

3.1.2 Leachate collection system

The leachate collection system was designed to handle leachate production flows as determined by evaluation of documented leachate generation rates. Pipe spacing was based upon HELP Model and Manning Equation analysis.

3.1.3 Leachate Production

Volumes – Documented leachate generation rates, and HELP Model
Storage Facilities - HELP Model during Phases 3 & 4

3.1.4 Gas Collection System

EPA design manual and assumption that gas well radius of influence = 100-125 feet.

3.2 Definition of Critical Conditions and Assumptions Made

Critical components

Phase 3 connects to the existing Phases 1 and 2 liner edge and waste will piggyback over the existing Phases 1 and 2 MSW waste areas.

Base liner system separation of four (4) feet above average seasonal high water table

- Maximize use of existing property
- Existing soils required a GCL composite liner with 1.0×10^{-5} permeability compacted clay soils overlain with a GCL material with a demonstrated hydraulic conductivity of not more than 5×10^{-9} cm/sec under the anticipated confining pressure.
- Constructed with washed stone leachate collection drainage layer to minimize potential of clogging
- Base liner system and proposed final cap system in accordance with current regulations

3.3 Technical References Used

- EPA 530-R-93-017 "Solid waste disposal facility criteria, technical manual"
- NC Regulations (15A NCAC 13B)
- Federal Regulations
- HELP Model
- Geomembrane liner manufacturer literature
- "Subtitle D technical training manual"
- EPA/625/R-94/008 Seminar publication "Design, operation, and closure of municipal soil waste landfill"
- EPA/600/R-94/168a "Hydraulic evaluation of landfill performance (HELP) Model user's guide for version3"
- Bagchi, Amalendu. (1994). Design, Construction, and Monitoring of Landfills, John Wiley and Sons, Inc., New York, NY.

3.4 Location Restrictions

The Phase 3 expansion complies with all location restrictions given in Section .1622 of the Solid Waste Management Rules (15A NCAC 13B).

3.4.1 Airports

There are no Public Use Airports within the two-mile perimeter of the White Oak Landfill. The closest airport is located just over nine miles north of the Facility.

3.4.2 Water Supply

There are two public water supply wells located within a two-mile perimeter of the White Oak Landfill, at the Fines Creek Methodist Church and the Panther Creek Baptist Church. No water supply wells are located within 500' of Phase 3. Private water supply wells located near the White Oak Landfill are illustrated on Sheet C1 of the Permit Amendment drawings for Phases 1-3.

3.4.3 Zoning

The area of the Phase 3 landfill cell area at the White Oak Landfill is not located within a zoned area.

3.4.4 Contamination Sources

The White Oak Landfill is the only known potential source of contamination in the area, with the exception of agricultural activities in the area.

3.4.5 Historical Sites

Six historical sites on the White Oak Landfill property were identified and examined during the preparation of the "MSWLF Facility Site Study, White Oak Landfill", prepared by Municipal Engineering and dated November 4, 1998. Four of the sites are prehistoric and two are historic. Three of the sites were to be impacted during the construction of the originally proposed Phases 3 & 4 expansion. None of the sites are recommended as eligible for inclusion on the National Register of Historic Places. The Site Study report concludes that no additional work is necessary at the sites. For an extensive discussion of Cultural Resources in the vicinity of the White Oak Landfill, see Section 4.7 of the "MSWLF Facility Site Study, White Oak Landfill".

3.4.6 Floodplains

Portions of the White Oak property along the Pigeon River are located in the 100-year floodplain; however, the extent of the floodplain is located over 1,000 feet horizontally and 150 feet vertically downgradient of the Phase 3 cell area. No area of Phase 3 is located within a 100-year floodplain.

3.4.7 Wetlands and Streams

A wetlands area and a stream were present in the area of the Phase 3 cell area. A jurisdictional delineation was performed by ClearWater Environmental and a representative of the U.S. Army Corps of Engineers (USCOE) conducted a field verification of the delineation. Approximately 0.45 acres of wetlands and 185 linear feet of stream were impacted by the construction of Phase 3. An application for the wetlands and stream impact was submitted to the USCOE and the NCDENR-Division of Water Quality for 404/401 permitting. A copy of the 404/401 permits was submitted to the NCDENR-Solid Waste Section prior to the issuance of the Permit To Construct for Phase 3. Beyond the areas of the stream/wetlands impact, the Phase 3cell area will not have detrimental effects on fish and wildlife. All of the following have been conformed to:

- Endangered Species Act of 1973
- Marine Protection, Research, and Sanctuaries Act of 1972
- Section 404 of the Clean Water Act

3.4.8 Seismic Design

See the Design Hydrogeologic Report prepared by Bunnell-Lammons, Inc.

3.4.9 Fault Areas

See the Design Hydrogeologic Report prepared by Bunnell-Lammons, Inc.

3.4.10 Unstable Areas

See the Design Hydrogeologic Report prepared by Bunnell-Lammons, Inc.

4.0 MATERIALS AND CONSTRUCTION PRACTICES

All materials and methods used for constructing the Phase 3 cell area met the requirements set forth in Section .1624 of the Solid Waste Management Rules (15A NCAC 13B).

4.1 Subgrade

The landfill subgrade was constructed adequately free of organic materials and consisted of on-site soils or select fill material previously approved by the Division. The subgrade was graded according to original plans approved by the Division.

Before construction of the liner system, the Project Engineer visually inspected the subgrade. The Project Engineer evaluated the integrity of the surface and documented proper preparation and elevations according to plans approved by the Division. The subgrade was proof-rolled using procedures specified by the technical specifications. The subgrade was tested for moisture and density requirements at the minimum frequency given in the Division approved technical specifications and the Site Specific Construction Quality Assurance Plan.

4.2 Clay Liner and Geosynthetic Clay Liner (GCL)

The materials used to construct the compacted clay liner consisted of native on-site materials. Any material that was used to construct the clay liner portion of the base liner system met all of the requirements of the project technical specifications.

The GCL was installed beneath and in uniform contact with the geomembrane liner. The GCL overlaid a compacted clay liner, in accordance with the specifications. Any material that was used to construct the GCL portion of the base liner system met all of the requirements of the project technical specifications and Section .1624 of the Solid Waste Rules.

Construction methods used for placement of the compacted clay liner or GCL were in accordance with the project technical specifications.

4.3 Geomembrane Liner

The geomembrane material had a demonstrated water vapor transmission rate less than or equal to 0.03 gm/M^2-day. The geomembrane also had physical and chemical resistance to environmental exposure, waste placement, and leachate generation. The primary liner was a high density polyethylene with a minimum thickness of 60 mils.

The installation of the geomembrane conformed to manufacturer's recommendations and the project technical specifications. The liner edge was installed and secured into an anchor trench at the Phase limits. Refer to Appendix B for anchor trench calculations.

4.4 Leachate Collection Pipes

Leachate collection piping has a minimum nominal diameter of six (6) inches for lateral collectors and eight (8) inches for main collectors, and was made of high density polyethylene material. The pipe provides adequate structural strength to support static and dynamic loads produced by materials and equipment used during construction and operation of the landfill. The pipe also provides adequate structural strength to support static loads produced by the waste fill and components of the final cap. Refer to Appendix A for pipe stability calculations.

The leachate collection pipes for Phase 3 were installed according to Sheet C4 of the Permit Amendment drawings for Phases 1-3. All piping was constructed with cleanouts where physically possible for periodic cleaning and maintenance. The bedding material for the collection lines was #5 washed aggregate with no more than five (5) percent by weight passing the #200 sieve. This aggregate is chemically compatible with leachate generated in the phase and met all the requirements of the project technical specifications.

4.5 Drainage Layers

The aggregate used as the drainage layer in Phase 3 will not be adversely affected by leachate produced in the phase and will promote lateral drainage of leachate. The drainage layer material met all the requirements of the project technical specifications.

The drainage layer was placed by methods given in the project technical specifications and the Site Specific Construction Quality Assurance Plan. The drainage layer material was stable on the 3:1 side slopes of the phase. Please refer to the Design Hydrogeologic Report prepared by Bunnell-Lammons, Inc., included in the original Permit to Construct Report submitted for Phases 3 & 4.

4.6 Filter Layers

Filter layers in the leachate collection system will prevent migration of fine soil particles from entering the aggregate drainage layer, while allowing water and gases to enter the drainage medium without clogging. Geosynthetic filters demonstrated adequate permeability and soil particle retention, while having chemical and physical resistance from waste placement, leachate, and any overlying material. Geosynthetic materials met all of the requirements of the project technical specifications.

All filter layers were installed according to the project technical specifications, construction drawings, and the Site Specific Construction Quality Assurance Plan. Geosynthetic filters were not wrapped directly around leachate collection piping.

4.7 Erosion Control

The erosion control structures were designed and will be maintained to manage the run-off generated by the 24-hour, 25-year storm event and conformed to the requirements of the Sedimentation Pollution Control Law (15A NCAC 4). A copy of the approved erosion control plan was forwarded to the Solid Waste Section prior to the issuance of the Permit to Construct for Phase 3. Erosion control measures for the construction of Phase 3 are shown on Sheet C5 of the Permit Amendment drawings. Final erosion control measures are shown on Sheet C11 of the Permit Amendment drawings.

5.0 DESIGN HYDROGEOLOGIC REPORT

The Design Hydrogeologic Report for Phases 3 & 4 was prepared by BLE, Inc. A copy of this report was included with the design package submitted to the Solid Waste Section under separate cover.

6.0 ENGINEERING DRAWINGS

6.1 Existing Conditions

Sheet C1 of the original Permit to Construct drawings for Phases 3 & 4 shows existing site topography, borings, piezometers, existing and proposed ground water monitoring wells, existing and proposed gas probes, existing roads, existing buildings, existing MSW Phases 1 and 2, existing landfill facilities, and the location of Phase 3 and proposed future Phase 4.

6.2 Grading Plans

The grading plans identified the proposed limits of excavation, clay liner elevations, and drainage layer elevations. Sheet 3A of the Permit Amendment drawings for Phases 1-3 showed the subgrade elevation if two (2)-feet of compacted clay was utilized. The subgrade elevations associated with the GCL liner are six (6)-inches above those shown on the aforementioned Sheet C3A. Sheet C3B of the Permit Amendment drawings or Phases 1-3 showed the top of clay liner or top of GCL elevations. Sheet C4 of the Permit Amendment drawings for Phases 1-3 shows the top elevation of the drainage layer.

6.3 Base Liner System

The grades for the top of the Phase 3 clay liner or GCL are shown on Sheet C3B of the Permit Amendment drawings. As shown on the drawings, the minimum floor slope within Phase 3 is 2.75 percent. Sheet C21 of the Permit Amendment drawings shows the post-settlement contours for the top of clay. The minimum post-settlement slope of the base liner system is greater than two (2) percent. The interior 3:1 side slopes and exterior fill slopes are also shown on Sheet C3A of the Permit Amendment drawings. The anchor configuration and details of the base liner system are shown on the detail Sheets, D1-D12 of the Permit Amendment drawings.

6.4 Leachate Collection System

The leachate collection system for Phase 3 is shown on Sheet C4 of the Permit Amendment drawings. The drainage layer washed stone for the leachate collection system is placed directly on the geotextile cushion and primary geomembrane. The cleanouts and sump locations are also identified on the drawings. The top of the stone drainage layer is shown on Sheet C4 of the Permit Amendment drawings. Details for the

leachate collection system, including sump details, cleanout details, and piping details are shown on the detail Sheets, D1-D12 of the Permit Amendment drawings. The proposed leachate system force main and gravity sewer beyond the limits of the landfill cells are shown on Sheet C18 of the Permit Amendment drawings.

6.5 Stormwater Segregation System

Phase 3 will be covered by a synthetic stormwater control liner that lies on top of the drainage layer. (Refer to Sheet C5 of the Permit Amendment drawings). This stormwater control liner allows stormwater runoff to collect in the down-gradient end of the phase. Stormwater runoff is then pumped over the perimeter embankment though a six–inch (6") line and discharged into a gravity drainage pipe. The drainage pipe discharges at a point near the unnamed stream just north of Phase 3. An energy dissipater was installed at the discharge point. Prior to entering an inactive area of landfill, the stormwater control liner will be partially removed. The overlying synthetic cover will be pulled back in stages, therefore keeping even active areas exposed to a minimum amount of rainwater. As the active working face increases in elevation, berms will be used to direct stormwater runoff to periphery ditches. See Sheets D1-D2 of the Permit Amendment drawings for a detail of the temporary stormwater control berm.

6.6 Cap System

Sheet C6 of the Permit Amendment drawings shows proposed temporary and permanent cap grades of Phases 1-3 Intermediate cover will be placed on the temporary cap slopes. As adjacent phases are developed, the temporary slopes will increase to the ultimate build-out of the phases. The final cap will include a one-foot (1') thick compacted intermediate cover, an eighteen-inch (18") thick clay layer, a forty-mil (40) textured HDPE geomembrane, a double-sided eight-ounce (8) geocomposite, and an eighteen-inch (18") thick vegetative\erosion layer. Details of the cap system are shown on Sheets D1-D12 of the Permit Amendment drawings.

Although not required by Solid Waste Rules or air permitting rules, Haywood County has prepared a preliminary landfill gas collection and combustion system at the White Oak Landfill. A landfill gas collection system for Phases 3 & 4 was included in the original Permit to Construct Phases 3 & 4 submittal. The proposed landfill gas collection system for Phase 3 has been shown on Sheet C12 of the Permit Amendment drawings. The proposed gas collection system for Phase 3 will connect to the proposed gas collection system for Phases 1 and 2. The proposed expansion of the system may consist of vertical gas extraction wells, horizontal gas collection wells, intermediate gas collection piping connecting the extraction wells to 12" and 16" transmission headers located at the eastern margin of Phase 1. The transmission lines will lead to a future blower and flare located just to the east of the existing Phase 1 waste area. The methane extraction wells will be constructed prior to construction of the final cap. A typical gas collection well has a radius of influence of approximately 100 to 125 feet. This resulted in approximately

eight (8) wells proposed for the Phase 3 area. The typical well penetrates the waste mass and allows a conduit for vertical migration of methane gas. The wells will be installed to a depth that is 10' above the top of stone leachate collection system layer. An HDPE boot will be placed at the area where the extraction well penetrates the cap geomembrane to retard surface water and oxygen infiltration into the waste layers. A bentonite/soil mixture will also be placed and compacted around the vertical collection wells where the wells penetrate the clay liner.

6.7 Erosion Control

An erosion control plan was prepared, submitted and approved by the Division of Land Quality for the originally proposed Phase 3 & 4 expansion. In addition to the existing sediment basin located near the leachate lagoon north of the Phase 1 waste area, three additional sediment basins were constructed to the north and west of Phase 3 These basins were utilized to control sedimentation during construction and are used to control stormwater runoff during the operation of Phase 3. Stormwater runoff is directed to these basins prior to release to nearby streams. An additional sediment basin was constructed to the south of the soil stockpile area located to the south of Phase 3 and the originally proposed location of Phase 4. In all cases, ditches, piping, and sediment basins were sized to accommodate the greatest flow that will be encountered throughout the lifetime of the landfill phase. Often, the greatest flow will be realized as the landfill reaches its capacity when the slopes are steepest, many years into the future. A copy of the erosion control permits associated with the development of Phases 3 & 4 was forwarded to the Solid Waste Section. The White Oak Landfill is being operated under a general stormwater permit that was issued in November of 2007.

The erosion and sedimentation control law is performance based and although existing erosion control permits and measures are in-place, it is still the responsibility of the landfill operator to ensure that the erosion control systems are functioning properly. Existing and proposed erosion/sedimentation control structures include sediment basins, storm drains, temporary slope drains, check dams, and diversion ditches. Sediment basins will be checked after periods of significant runoff. Sediment will be removed from the basin to its original dimension when sediment accumulates to one half of the design depth. The sedimentation basins, embankments, ditches, inlets and outlets will also be inspected for erosion damage. All necessary repairs will be made immediately. Any trash or debris within the riser pipes will be removed. Storm drain outlets and diversion ditches will be inspected for damage after each runoff event. Rip rap will be placed in ditches and at pipe outlets to prevent erosion and wash outs. Provisions for a vegetative ground cover sufficient to control erosion must be accomplished within fifteen days upon completion of any phase of grading. Embankment slopes shall be periodically inspected for erosion. The embankment slopes shall be mowed at a frequency sufficient to maintain a good stand of vegetation. The slopes shall be mowed once in any one (1) year period. The embankment slopes shall be refertilized in the second year unless vegetation growth is fully adequate. Any damaged areas will be reseeded, fertilized, and

mulched immediately. Seeding, fertilizing and mulching shall be in accordance with the North Carolina Erosion and Sedimentation Control Guidelines.

The erosion control plan for the construction of Phase 3 was included on Sheet C5 of the Permit Amendment drawings. The final erosion control plan is shown on Sheet C11 of the Permit Amendment drawings.

6.8 Vertical Separation Requirements

Cross sections of Phase 3 are shown on Sheets C22 through C24 of the Permit Amendment drawings. These drawings illustrate the base liner thickness, the depth of waste, the cap system thickness, and the existing ground elevation, the potentiometric surface, and bedrock elevations. Cross sections of the subsurface conditions, created from boring data at the site, are shown in the Phases 3 & 4 Design Hydrogeologic Report prepared by BLE, Inc. The report also contains an average seasonal high water contour map, a bedrock contour map, and a potentiometric surface map. The minimum separation between the **bottom** of the proposed clay liner and the water table in Phase 3 and the originally proposed Phase 4 is 6 feet, located near the center of the Phases 3 & 4 area and at the sump area at the northern margin of Phase 3. This vertical separation increases up to 16 feet near the high point at the western edge of the new phase. Therefore, although the long-term settlement across the footprint of the landfill could be as much as 18 inches in places, the original separation between the base grades and the average high water table will allow for this magnitude of potential settlement. The estimated subgrade settlement is shown in the BLE Design Hydrogeologic Report. The projected settlement is shown on Sheet C21 of the Permit Amendment drawings.

Differential settlement was evaluated to ensure that if one area of the landfill subsides greater than adjacent areas, the resultant slope will remain sufficient for leachate migration to the sump areas. The worst-case scenario would be if the southern margin of Phase 3 subsided the greatest amount estimated by the BLE, Inc. Design Hydrogeologic Report – 15", while the northern margin showed no subsidence. In this event, the resultant grade of the landfill floor would still be greater than 2%, which would allow for leachate to migrate to the sumps.

APPENDIX A

Pipe Stability Calculations

Appendix A

Pipe Stability Calculations

Max. landfill elevation: 2676
Clay grade elevation: 2548
Maximum overburden depth: 128'

Overburden Description (landfill fully developed)

Unit Weights:

Drainage layer stone: 130 lbs/ft^2
Compacted waste: 55 lbs/ft^2
Compacted soil material: 110 lbs/ft^2

Overburden Pressure

Drainage layer: 2 ft @ 130 lbs/ft^2 = 250 lbs/ft^2
Compacted waste: 128 ft @ 55 lbs/ft^2 = 7,040 lbs/ft^2
Compacted soil: 4 ft @ 110 lbs/ft^2 = 440 lbs/ft^2
7,740 lbs/ft^2

or $\frac{7{,}740 \text{ lbs/ft}^2}{144 \text{ in}^2/1 \text{ ft}^2} = 53.8$ psi

Check for Wall Crushing

$S_a = \frac{(SDR - 1)}{2} P_t = 430.4$ psi

$P_t = 53.8$ psi
SDR = 17

Assume compressive yield strength of HDPE pipe is approximately 1,500 psi, therefore, safety factor against wall crushing: $\frac{1{,}500 \text{ psi}}{430.4 \text{ psi}} = 3.4$

Check for Wall Buckling

$P_{cd} = 0.8 \sqrt{(E' \times P_c)}$
$P_t = 53.8$ psi
$P_c = \frac{2.32 (E')}{(SDR)^3} = 14.17$

- P_{cd} must be greater than P_t to resist against wall buckling
- Assume E' = 30,000 psi (crushed stone compacted)
- (Ref. pg. 37 of DriscoPipe design manual) Time dependent modulus of eleasticity @ 150 psi tensile stress = 30,000 psi

$P_{cd} = 0.8\sqrt{(3{,}000 \times 14.17)}$
$P_{cd} = 165$ psi

•150 psi assumption for tensile stress was based on discussion with DricoPipe engineer Bill Shultz

165 psi > 53.8 psi, therefore, there is resistance to wall buckling
Safety Factor: 165/53.8 = 3.1

Ring Deflection

Allowable ring deflection: $\Delta Y/D = (0.25)E(SDR)$
$\Delta Y = (6.625)(0.25)(0.01)(17)$
$\Delta Y = 0.28$"

$\Delta Y/D = 0.28"/6.625" = 4.23\%$

ΔY = vertical deflection (in.)
D = Pipe O.D. (in.)
E = tangential strain in surface of pipe ring due to reflection (=0.01) pg. 39 DriscoPipe manual)
SDR = Standard dimension ratio, D/t
t = pipe wall thickness
For 6" SDR 17, D = 6.625", t= 0.39"

Vertical soil strain: $E = P_t/E'$
$E = 53.8/3{,}000$
$E = 0.0179$

E' = Soil modulus, psi (3,000 psi)
E = Vertical soil strain
P_t = total vertical soil pressure = 53.8 psi

Allowable ring deflection > vertical soil strain
$\Delta Y/D = 0.0423/0.0179$

Safety Factor: 0.0423/0.0179 = 2.36 Pipe meets requirements

APPENDIX B

HDPE Liner Anchor Trench Calculations

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PROJECT: Haywood County - White Oak LF
PROJECT NO.: Phases 3 & 4
DESCRIPTION: Anchor Trench
CALCULATED BY: WHS CHECKED BY:
DATE: 9/2/08 SHEET NO. 1 OF 6

MATERIAL PROPERTIES:

Washed Stone: $\gamma_s \approx 130$ pcf
$\phi_s = 40°$

Embankment Fill: $\gamma_F \approx 125$ pcf
$\phi_F = 32°$

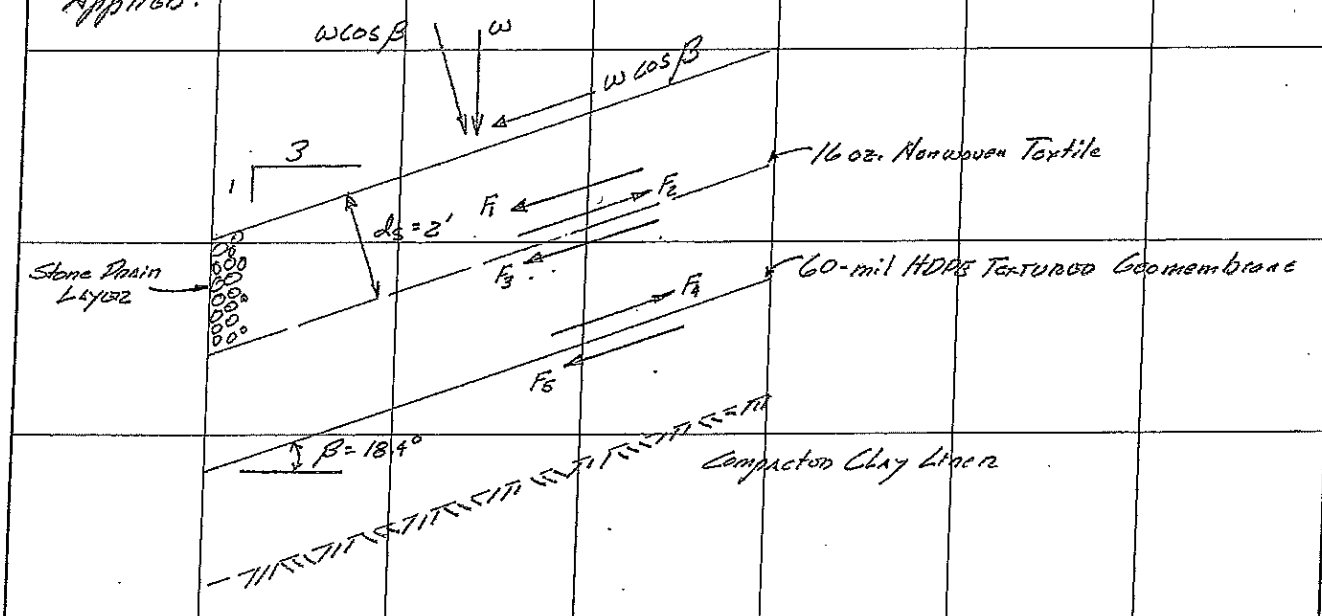
Compacted Clay: $\gamma_c \approx 120$ pcf
$\phi_c \approx 25°$

Minimum interface friction angle between Compacted Clay Liner (L) and the 60 mil HDPE Textured Geomembrane (G): $\delta_{LG} = 17°$

Minimum interface friction angle between Nonwoven Geotextile Fabric (T) and the 60-mil HDPE Textured Geomembrane (G): $\delta_{TG} = 20°$

Minimum interface friction angle between Nonwoven Geotextile Fabric (T) and the washed stone drainage layer (S): $\delta_{TS} = 24°$

Resolution of Shear Stress in Section @ Construction End with No Waste Applied:



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DESCRIPTION: Anchor Trench
CALCULATED BY: WHS CHECKED BY:
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$\omega = (d_s / \cos\beta)(\gamma_s)$ $\gamma_s \approx 130$ pcf, $d_s = 2'$, $\beta = 18.4°$

$= (2' / \cos 18.4°)(130 \text{ pcf})$

$\therefore \omega = 274$ psf

$N = \omega \cos\beta$ $\omega = 274$ psf, $\beta = 18.4°$

$= (274 \text{ psf})(\cos 18.4°)$

$\therefore N = 260$ psf

$F_1 = (N)(\tan \delta_{TS})$ $N = 260$ psf, $\delta_{TS} = 24°$

$= (260 \text{ psf})(\tan 24°)$

$\therefore F_1 = 116$ psf

$F_2 = (N)(\tan \delta_{TG})$ $N = 260$ psf, $\delta_{TG} = 20°$

$= (260 \text{ psf})(\tan 20°)$

$\therefore F_1 = 95$ psf

$F_1 - F_2 =$ Tensile stress carried by 16oz nonwoven geotextile

$= 116 \text{ psf} - 95 \text{ psf}$

$= 21$ psf

Maximum inside slope length @ Tie to existing liner $\approx 70'$
Total Tensile Stress in Fabric, T, is

$T_F = (70')(21 \text{ psf})$

$= 1470$ lbs/LF of fabric

From published data, 16 oz nonwoven geotextile fabric has a tensile yield strength @ 195 to 220 #/inch (2340 lbs/ft to 2640 lbs/ft.)

$F_1 S_{(min)} = 2340 \text{ lbs/ft} / 1470 \text{ lbs/ft} = 1.59$ O.K.

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PROJECT NO.: Phase 3 & 4
DESCRIPTION: Anchor Trench
CALCULATED BY: WHS CHECKED BY:
DATE: 9/2/08 SHEET NO. 3 OF 6

By inspection: $F_3 = F_2$ (in opposing direction)

$\therefore F_3 = 95$ psf

$F_4 = (N)(\tan \delta_{LG})$ $N = 260$ psf; $\delta_{LG} = 17°$
$(260 \text{ psf})(\tan 17°)$

$\therefore F_4 = 79$ psf

Tensile Strength carried by HDPE Textured Geomembrane $= F_3 - F_4$
$= 95 \text{ psf} - 79 \text{ psf}$
$= 16$ psf

Max inside slope length @ Tie-in to Existing Liner ≈ 70'

$T_G = (70')(16 \text{ psf})$
$= 1120$ lbs/LF of Geomembrane

From published data, 60-mil HDPE Textured Geomembrane liners have an approximate yield strength in wide width tension of approximately 130 lbs/inch (1560 lbs/LF)

F.S. 1560 lbs/LF / 1120 lbs/ft = 1.39 O.K.

By inspection: $F_5 = F_4$ in opposing direction

$\therefore F_5 = 79$ psf

Internal shear in Compacted Clay:

$\tau = N(\tan \phi_c)$ $N = 260$ psf; $\phi_c = 25°$
$= (260 \text{ psf})(\tan 25°)$

$= 121$ psf

F.S. (min) = 121 psf / 79 psf = 1.5 O.K.

By inspection, stresses w/ waste deposition should result in decreased stresses in Liner components due to an increase in normal forces and friction developed at the edge of waste at the transition from base grades up the interior slope.

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DESCRIPTION: Anchor Trench
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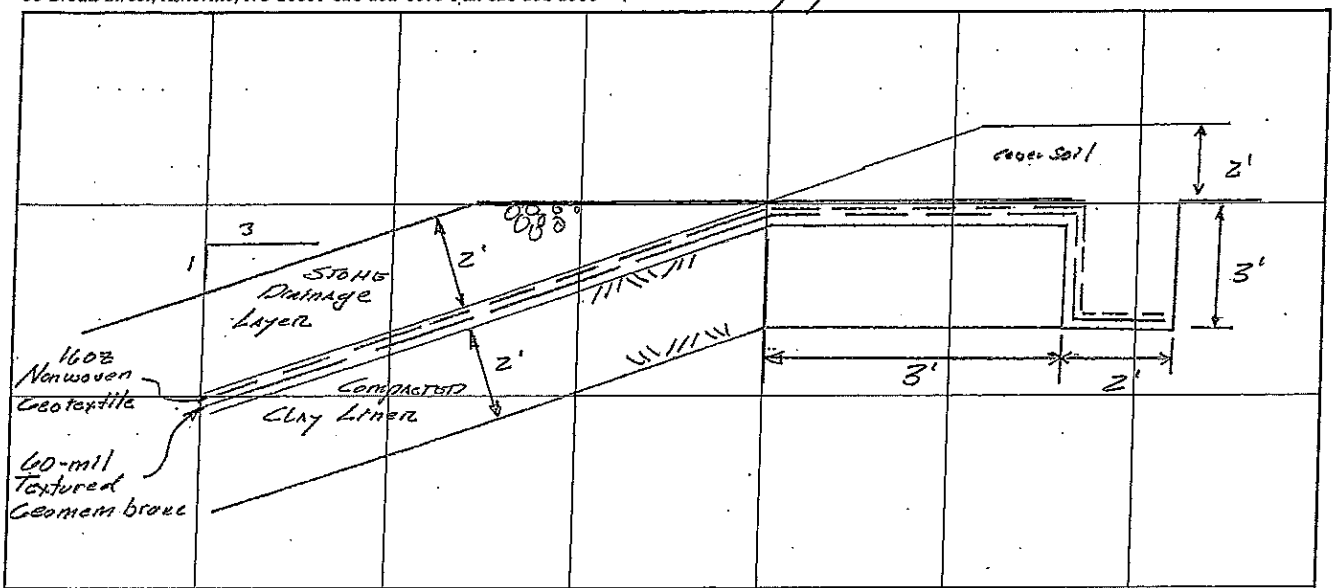

Cover soil Density: $\gamma_F \approx 125$ pcf
Anchor Trench Soil Density: $\gamma_F \approx 125$ pcf
Anchor Trench width: $L_{AT} = 2'$
Anchor Trench depth: $d_{AT} = 3'$
Depth of Cover Soil: $d_{CS} \approx 2'$
Runout Trench Length: $L_{RO} = 3'$
Anchor Trench Backfill Material friction Angle: $\delta = 25°$
Minimum Interface Adhesion: $a = 0$

Anchor Trench Runout Resistance (Liner to clay)

$$P_{R1L} = (S_{n1} \tan \delta_{LG} + a) L_{RO}$$
where $S_{n1} = (\gamma_F)(d_{CS})$
$= (125 pcf)(2')$
$S_{n1} = 250$ psf

$$= ((250 psf)(\tan 17°) + 0)(3')$$
$P_{R1L} = 229$ lbs/ft.

Anchor Trench Bottom Resistance: (Liner)

$$P_{R2L} = (S_{n2} \tan \delta_{LG} + a) L_{AT}$$
where $S_{n2} = (\gamma_F d_{CS}) + (\gamma_F d_{AT})$
$= (125 pcf)(2') + (125 pcf)(3')$
$= 250 psf + 375 psf$
$S_{n2} = 625$ psf

$$= ((625 psf)(\tan 17°) + 0)(2)$$
$P_{R2L} = 382$ lbs/ft

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PROJECT NO.: Phase 3 & 4
DESCRIPTION: Anchor Trench
CALCULATED BY: WHS CHECKED BY:
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Anchor Trench Sidewall Resistance (Liner)

$K_o = (1 - \sin f)$
$= 1 - \sin 25°$
$= 0.58$

Avg. normal stress through anchor trench

$S_{AVGV} = (S_{n1} + S_{n2})/2$
$= (250\ psf + 625\ psf)/2$
$S_{AVGV} = \underline{438\ psf}$

$S_{AVGH} = (K_o)(S_{AVGV})$
$= (0.58)(438\ psf)$
$S_{AVGH} = \underline{254\ psf}$

$S_{HAVG} = S_{n3} = 254\ psf$

$P_{R3L} = (S_{n3} \tan \delta_{LR} + a)(d_{AT})$
$= ((254\ psf)(\tan 17°) + 0)(3')$
$\underline{P_{R3L} = 233\ lbs/LF}$

Geotextile to Liner:
minimum Geotextile to Liner Interface Friction Angle: $\delta_{TL} = 20°$
minimum Geotextile interface adhesion: $a = 0$

Anchor Trench Rollout

$P_{R1T} = (S_{n1} \tan \delta_{TL} + a)(L_{RO})$
where $S_{n1} = (\gamma_P)(d_{AS})$
$= (125\ pcf)(2')$
$= 250\ psf$

$P_{R1T} = ((250\ psf)(\tan 20°) + 0)(3')$
$\underline{P_{R1T} = 273\ lbs/LF}$

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PROJECT: Haywood County - White Oak LF
PROJECT NO.: Phase 3&4
DESCRIPTION: Anchor Trench
CALCULATED BY: WAS CHECKED BY:
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Anchor Trench Bottom Resistance (Geotextile)

$P_{R2T} = (S_{n2} \tan \delta_{TG} + a)(L_{AT})$

where $S_{n2} = (\gamma_P)(d_{CS}) + (\gamma_T)(d_{AT})$

$= (125 pcf)(2') + (125 pcf)(3')$

$= 250 psf + 375 psf$

$= 625 psf$

$= ((625 psf)(\tan 20°) + 0)(2')$

$P_{R2T} = 455 lbs/ft$

Anchor Trench Sidewall (Geotextile)

$K_o = (1 - \sin f)$

$= 1 - \sin 25°$

$K_o = 0.58$

Avg normal Stress Through Anchor Trench

$S_{AVGU} = (S_{n1} + S_{n2})/2$

$= (250 psf + 625 psf)/2$

$= 438 psf$

$S_{AVGH} = K_o * S_{AVGU}$

$= (0.58)(438 psf)$

$= 254 psf$

$S_{AVGH} = S_{n3}$

$P_{R3T} = (S_{n3} \tan \delta_{TG} + a)(d_{AT})$

$= ((254 psf)(\tan 20°) + a)(3')$

$P_{R3T} = 277 lbs/ft$

TOTAL Anchor Trench Rollout:

$T_{AR} = P_{R1L} + P_{R2L} + P_{R3L} + P_{R1T} + P_{R2T} + P_{R3T}$

$= 229 lbs/ft + 382 lbs/ft + 233 lbs/ft + 273 lbs/ft + 455 lbs/ft + 277 lbs/ft$

$= 1849 lbs/ft$

F.S. = 1849 lbs/ft / 1490 lbs/ft = 1.26 OK