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CONSTRUCTION PLAN APPLICATION
FGD RESIDUE LANDFILL

DUKE ENERGY CORP.
BELEWS CREEK STEAM STATION
STOKES COUNTY, NC

Sells Project Number: 046119

APPROVED
DIVISION OF WASTE MANAGEMENT
SOLID WASTE SECTION
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Prepared By:

CHAS. H. SELLS, INC.
Consulting Engineers, Surveyors & Photogrammetrists
128 Overhill Drive, Suite 105, Mooresville, NC 28117
Phone: (704) 662-0100
Fax: (704) 662-0101

September 30, 2005

NORTH CAROLINA PROFESSIONAL SEAL 17582 ENGINEER DAN E. BREWER
9/30/05

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CONSTRUCTION PLAN APPLICATION

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CONSTRUCTION PLAN APPLICATION REPORT

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FGD RESIDUE LANDFIL, PHASE 1
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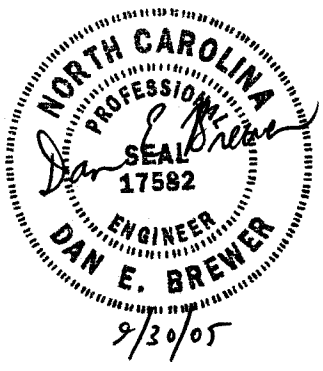

1.0 INTRODUCTION

1.1 Project Description

The Clean Smokestacks Act passed in 2002 requires significant reductions in emissions from coal-fired power plants operating in North Carolina. NC utilities must reduce the actual emissions of nitrous oxides (NO_x) by 77% by the year 2009 and must reduce actual sulfur dioxide (SO_2) emissions by 49% by the year 2009 and by 73% by 2013. As part of Duke Power's effort to meet the SO_2 reduction requirements, a Flue Gas Desulfurization (FGD) system will be installed at the Belews Creek Steam Station in Stokes County, NC. Duke's intent is that a large portion of the residue from this process will be utilized in beneficial products and the remainder will be placed in the proposed FGD gypsum landfill.

The FGD gypsum landfill will be located on Duke Power property, south of the Belews Creek plant. The Phase 1 landfill footprint contains approximately 24 acres. The FGD residue will be conveyed to the landfill site by conveyor and truck, where the material will be spread and compacted. The landfill will begin receiving FGD residue in early 2007. The volumetric capacity of the landfill is 1.6 million cubic yards.

The FGD gypsum landfill at Belews Creek will receive residue solely generated from the FGD process. The residue placed in the landfill will be:

1. FGD residue material that is not needed to meet the production needs of any beneficial use options.

2. FGD residue material that does not meet the specifications required for beneficial use.

3. FGD residue material that is removed from settling or clarifier stages of the associated waste water treatment facility.

1.2 Site Conditions

The Belews Creek Steam Station is located in the Piedmont physiographic region. The footprint for the landfill contains an area of approximately 24 acres.

The FGD gypsum landfill will be located on Duke Power property. The landfill is located to the south of a railroad line, running east to west. Located to the west of the landfill is Craig Road. Located to the south of the landfill footprint is a surface water drainage feature that drains to Belews Lake. There is a topographic divide and access road running north-south along the east side of the landfill footprint. Surface drainage to the east of the topographic divide also drains to Belews Lake.

2.0 LANDFILL DESIGN

2.1 Site Development

The FGD gypsum landfill consists of one phase containing four cells. The landfill has an exterior perimeter berm ranging from 4 to 40 feet in height. The perimeter berm will prevent surface water run-on from entering the landfill during development. All fill placement for the berms will be compacted to a minimum of 95 percent of the standard Proctor maximum dry density. The berms will be in excavated slopes or constructed with on-site borrow material.

A 200-foot buffer will be maintained around the entire perimeter. A 500-foot buffer will be maintained from existing residences and water supply wells.

Each cell will be constructed with a minimum 1.2 percent base grade. The subgrade, as shown on the Top of Liner Grading Plan, Sheet CP-2, will be a minimum of four feet above the estimated seasonal high groundwater level and four feet above bedrock. Based on the Site Plan Application Report, excavation will be allowed within the entire landfill footprint. Sufficient quantities of borrow material for construction is available from excavation within the landfill. Excavated material will be stockpiled for future use as final cover.

The landfill base will be lined with a 60 mil HDPE geomembrane. A leachate collection system is designed with a geocomposite and series of HDPE piping to drain the leachate within each active cell to the downgradient stormwater storage basin. The stormwater storage basin is designed to detain a 25-year storm runoff from two active cells. Stormwater and leachate will equalize in the basin and be pumped to the on-site wastewater treatment plant.

A 12-inch thick layer of operational cover is designed over the geomembrane on the cell floor. Cell divider berms will be constructed between each cell as shown on the Operational Cover Grading Plan, Sheet CP-4.

Stormwater within inactive cells will be routed to the on-site erosion control structures. Erosion control structures are sized in accordance with the North Carolina Erosion and Sedimentation Pollution Control Act. Surface water control structures located downgradient of the landfill are designed to handle a 25 year, 24-hour storm event.

Existing access roads will be utilized for access to the landfill. Temporary access roads will be constructed throughout the operation of the landfill. The roads are a minimum of 15 feet wide with side ditches to prevent erosion of the roadway surface.

Final cover elevations, as shown on the Final Closure Plan and Details, Sheet CP-8, will be sloped no steeper than 4H:1V (horizontal to vertical) with erosion control benches. The final cover (from top to bottom) will include a 24-inch thick vegetative soil layer, a geocomposite drainage layer, a 40 mil textured LLDPE geomembrane, and an 18-inch compacted soil layer on top of the waste.

2.2 Landfill Operation

The landfill will be operated from the upgradient end to the downgradient end of each cell so that stormwater that falls into an active area will be routed to the stormwater basin.

Waste acceptance and operational procedures are presented in the FGD Gypsum Landfill Operations Plan, Section IV.

Closure of cells will commence at the 5-year permit period. Partial closure of cells may be implemented during landfill operations. Site phasing diagrams are presented in the FGD Gypsum Landfill Operations Plan, Section IV.

2.3 Closure Design

The primary purpose of a landfill cap is to minimize infiltration of water into the waste.

The typical cap cross section is presented on the Final Closure Plan and Details, Sheet CP-8. As depicted on the cross section, the final closure includes an 18-inch thick soil cover with 6-inch thick vegetative cover soil layer, a geocomposite drainage layer, a 40 mil textured LLDPE geomembrane, and an 18-inch thick compacted soil layer on top of the waste. Specifics regarding the closure and post-closure activities are included in the Closure/Post-Closure Plan, Section V.

2.4 Groundwater Considerations

The estimated seasonal high groundwater levels and depth to bedrock at the project site were used to select the maximum excavation of each cell. As indicated on the Phase 1 Landfill Profiles, Sheets CP-13 and CP-14, a minimum four feet separation has been maintained from the estimated seasonal high groundwater level or depth to bedrock to the top of liner system (below the geomembrane).

2.5 Engineering Considerations of Landfill Components

2.5.1 Landfill Subgrade

A perimeter berm and excavated subgrade will be compacted and maintained to provide support for the overlying geomembrane.

Data from the subsurface exploration was used in the calculations to determine anticipated settlement and available bearing capacity of the soils beneath Phase 1. Bearing capacity was determined from empirical methods based upon standard penetration test blow counts (SPT's). Based on the bearing capacity calculation (see Appendix A), the allowable bearing capacity of the subsurface soils is greater than the maximum overburden stress due to waste placement in Phase 1. During construction proofrolling will be performed prior to fill placement. Any areas exhibiting excessive rutting or pumping will be undercut and replaced with suitable fill material.

A settlement analysis was performed to determine subgrade settlement due to fill and waste placement. The analysis based conditions encountered in soil test borings BC-2 and BC-10. No soft zones were encountered in soil test borings.

The FHWA settlement analysis procedure was applied to each soil layer encountered in the soil test borings listed above. The maximum differential settlement within a cell is estimated to be approximately 2 inches which would allow positive drainage within the landfill floor. The settlement calculations are presented in Appendix A.

2.5.2 Geomembrane

A 60-mil thick, high density polyethylene (HDPE), smooth geomembrane will be placed as the synthetic liner for Phase 1 floor and textured 60-mil HDPE geomembrane will be placed on the side slopes.

The required geomembrane mechanical property analysis is included in Table 1 of the Technical Specifications presented in Section II. The following paragraphs address construction and post-construction strength requirements for the geomembrane. The design of Phase 1 and construction quality assurance procedures are intended to reduce or eliminate puncture, impact or tearing stresses on the geomembrane.

Tensile stress analyses indicate that the maximum geomembrane stresses most often occur due to down drag forces on slopes from equipment during waste placement. The analyses indicate that operational cover is not stable on a 3H:1V side slope. Also, a textured geomembrane with geocomposite is required to provide suitable friction angles to prevent overstressing the geomembrane and providing a suitable factor of safety against sliding.

The analyses indicate that an anchor trench depth of 18 inches is sufficient to provide anchorage of the geomembrane. The anchor trench will provide sufficient anchorage to prevent pull-out during anticipated loading conditions, but total anchorage capacity will be less than the HDPE geomembrane yield stress. This reduces the potential for large scale yielding or tearing of the geomembrane while allowing a pullout type failure from the anchor trench to occur under extreme loading conditions rather than total failure of the geomembrane.

2.5.3 Leachate Collection and Removal System (LCRS)

Above the geomembrane will be a leachate collection and removal system (LCRS) that will direct the leachate toward a downgradient collection sump. The collection system along the bottom will consist of a geonet/geotextile with perforated collector/header pipe that drain to the sump area. The slope on the geomembrane will be a minimum of 1.2 percent.

The geonet is a polyethelene synthetic mesh with a transmissivity equal to or greater than 1 X 10^{-4} m^2/sec. The geonet will transmit leachate to the leachate collection pipe.

The leachate collection/header pipe in each cell meets the performance criteria of providing a

maximum 1 foot of head of leachate on the liner system under normal operating conditions. The pipe spacing is based on results from the Mound model that incorporates the leachate infiltration rates as well as the maximum allowable head. A maximum pipe spacing of 75 feet meets the performance criteria. The Mound model does not consider the effects of the underlying geocomposite which adds to the reduction of head build up on the geomembrane.

The leachate collection pipe diameter has been conservatively designed to drain the leachate generated from a 25-year storm event. The pipe flow area is conservatively assumed to be within the perforation zone only and does not consider potential additional flow of the remaining pipe area or graded filter area surrounding the pipe.

The landfill perimeter berm has been designed to contain run-off from the 25 year-24 hour storm event. As shown in the calculations, Appendix A, each cell divider berm has also been designed to contain the 25 year-24 hour storm event.

The sump is located at the downgradient end of each cell. This sump will be a 1 inch thick pre-fabricated HDPE sump that is to be extrusion welded to the 60 mil HDPE geomembrane. The leachate will gravity flow from the sump location through a solid HDPE pipe to the stormwater basin.

2.5.4 Geotextile

An 8 ounce per square yard geotextile will be used to separate and provide a filter for the operational cover and the leachate collection and removal system for the cell bottom. A geocomposite (geotextile/geonet/geotextile) is required for the side slopes to provide suitable friction angles against a textured geomembrane. The geotextile will be continuous filament, UV stabilized, polyester or polypropylene, non-woven, needle-punched geotextile.

The properties of the geotextile incorporated into construction of Phase 1 have been examined for filtration, puncture resistance and wide width tensile strength. The geotextile must provide puncture resistance to protect the LCR system and geomembrane. The specified properties of the geotextile are presented in the Technical Specifications, Section II.

2.5.5 Operational Cover

Operational cover will be placed on the cell bottom over the geomembrane and LCR system to provide a buffer between the operations equipment and the liner system. Soil will be placed at a minimum 12-inch thickness for the operational cover. The requirements for the operational cover, placement and compaction are addressed in the Technical Specifications, Section II.

2.5.6 Temporary Side Slope Liner and Ballast System

Due to the steepness of the excavated side slopes and potential long-term exposure of the side slope liner system, a temporary side slope liner with ballast system is required. The temporary side slope liner and ballast system will protect the geocomposite and geomembrane against potential damage and ultraviolet degradation. The temporary side slope liner will be a

minimum 20-mil thick LLDPE geomembrane. The temporary ballast system will consist of 40 pound sand bags spaced at 10 feet on center. The ballast system is designed to offset minor wind uplift pressures.

2.5.7 Surface Water Collection and Removal (SWCR)

A surface water collection system has also been designed for each cell in Phase 1. A temporary series of pipes and cell divider berms are provided within each cell to allow rainwater that has not contacted waste to be drained out of the cell and into the erosion and sediment control structures outside of the landfill footprint.

The general operation required to activate a cell for waste placement will be the removal of the temporary SWCR pipe and opening the sump valve for draining the cell into the stormwater basin.

Surface water within an active cell that infiltrates through the operational cover and into the LCR system will be treated as leachate. By effectively managing surface water collection and removal, the volume of leachate requiring treatment can be reduced.

2.5.8 Geomembrane Cap

The function of the geomembrane cap is to prevent water that infiltrates through the final cover soil from entering the underlying waste cell.

The design of the cap provides for a minimum number of penetrations through the geomembrane cap. The only required penetrations are for clean out pipes and the gas venting system.

The material selected for the cap will be a 40-mil, textured, linear low density polyethylene (LLDPE). LLDPE geomembrane provides higher elongation properties and does not have the environmental stress cracking potential as found with exposed HDPE geomembrane caps.

2.5.9 Gas Venting System

Based upon our past experience with gypsum landfills, H_2S gas generation is prevalent under high organic contents in the gypsum or natural soils mixed with the gypsum. The gypsum from the FGD process will not have any organic content. Also, on-site soils will be limited to the bottom of the landfill for operational cover and the top of the landfill for the final cover. Soils will not be mixed with the gypsum. Therefore, the gypsum will not have sufficient organic material for an anaerobic bacteria to produce a surmountable quantity of H_2S gas.

A study provided by Duke Power Company entitled "Chemical/Physical Stability of Flue Gas Cleaning, Section 9, Hydrogen Sulfide Production by Bacterial Reduction of FGD Sludges" (EPRI FP-671-V2, Project 786-2, Final Report, dated January 1979) concluded that small amounts of H_2S gas was produced in experimental gypsum/soil mixtures, and the gas production diminished after five days.

Due to the possibility of minor H_2S gas production at the gypsum/soil interface, a passive gas system has been designed for the final cover system of the FGD gypsum landfills. During landfilling activities H_2S gas measurements will be taken at the leachate collection cleanout pipes located along the landfill perimeter and the sump discharge pipe inverts in the stormwater basin. Following landfill closure activities, H_2S gas measurements may also be taken from the gas vents. In the event that H_2S gas is detected, the proposed passive gas system can be converted into an active gas system with gas control devices if required.

The gas venting system is shown on the Final Closure Gas Venting System Plan, Sheet CP-9.

The gas venting wells are to be constructed prior to final capping of Phase 1. For Phase 1, Cells 1 through 4 approximately 26 gas vents are designed over the 24 acres. The typical gas vent cross section penetrates the final cover and allows a vertical migration of gases. The gas vent is connected to the upper geomembrane cap by means of a geomembrane boot to prevent surface water infiltration into the landfilled waste.

2.5.10 Final Cover Soil

The final cover soil will be a total of 24-inches thick including at least 6-inches of topsoil. The final cover soil function is to provide support for the vegetative cover and to protect the geomembrane cap.

The final cover stability was evaluated on the geomembrane on the 4H:1V slopes. Using a textured geomembrane, the resulting factor of safety against sliding exceeds the minimum acceptable factor of safety of 1.5.

The HELP Model was also performed for final closure conditions. Due to the potential head encountered on the geomembrane cap due to saturated cover soils, a geocomposite with perforated drains (edge drains) will be placed on the geomembrane. The edge drains will be placed along bench locations within the cover to relieve the cover soils of excessive pore water pressure and aid in overall soil stability. The edge drains will be routed to the final closure stormwater management system.

2.5.11 Vegetative Cover

A grass cover is proposed for Phase 1 to provide the required vegetative cover while minimizing cap maintenance. The surface will be prepared by fertilizing and placing seed in accordance with the North Carolina Erosion and Sediment Control Standards and the North Carolina Sedimentation Pollution Control Act of 1973, as amended 1987. The design objectives in evaluating the vegetative cover include:

- a drought resistant vegetative cap with a two-foot storage limitation
- low maintenance cover
- cover selection to minimize volunteer woody vegetation
- erodability of erosion control
- tolerant of final 4H:1V side slopes.

3.0 LEACHATE MANAGEMENT

Leachate is described as the water that infiltrates through the landfill and potentially leaches contaminants from the waste. Leachate is produced as precipitation falls on the surface of the active landfill cell and infiltrates through the waste.

The leachate collection system intercepts leachate as it infiltrates the landfill and routes it to the stormwater basin for pumping to the wastewater treatment system. The stormwater basin design is presented on the Stormwater Basin Plan & Details, Sheet CP-6. The pump station design for the stormwater basin is included on the Pump Station Details, Sheet CP-7.

3.1 Leachate Generation

Leachate generation rates are based on the HELP Model using Greensboro, North Carolina precipitation data. Based on the HELP Model, average daily rates for an open cell at initial operating conditions are on the order of 6,005 gallons per day with a maximum head of less than 1 foot above the composite liner. Based on a 5.7 acre cell, this amount is estimated to be approximately 1,054 gallons per acre per day. Intermediate cases with a 66-foot waste height and 89-foot waste height over the entire Phase 1 (24 acres) indicate average leachate generation rates on the order of 820 gallons per acre per day and 701 gallons per acre per day, respectively. Closure conditions reveal leachate generation estimates of less than 852 gallon per acre per day. Leachate generation after closure conditions is expected to diminish over time due to the prevention of water infiltration.

3.2 Leachate Storage

The stormwater basin is designed to contain stormwater runoff from two active landfill cells. The stormwater basin pump to the wastewater treatment system is sized for 500 gallons per minute. A 25-year storm event will drain the stormwater basin within 7 days.

In the event of storm surges that result in extreme leachate generation conditions or down time in the wastewater treatment system, a valve is provided at the downgradient discharge pipe from each cell sump.

4.0 ENVIRONMENTAL CONTROL AND MANAGEMENT

4.1 Gas Migration

Methane or H_2S gas is not anticipated to be generated in the landfill. During landfill operations, gas testing from the leachate header cleanout piping and sump discharge pipe inverts in the stormwater basin will be performed in order to identify if gas generation is a problem and if further testing or monitoring is deemed necessary. Furthermore, a passive gas collection system is included for closure to reduce the likelihood of gas migration.

4.2 Nuisance and Vectors

Potential nuisances to the areas surrounding the landfill from the landfill operations include dust and sedimentation. Odors, vectors, scavengers, potential fires or blowing litter is not anticipated to be a problem at this type of landfill. Dust will be minimized by use of road surfaces with aggregate and regular spraying with water. If the gypsum exhibits dusting problems, misting operations will be performed to reduce the dust potential. Sedimentation will be controlled with appropriate erosion and sedimentation control devices.

4.3 Wetlands

No wetlands have been identified inside the landfill footprint. Wetlands have been identified outside the landfill footprint. These wetlands have been surveyed and are pending USACOE approval.

5.0 WASTE STREAM

5.1 Types of Waste

The type of waste specified for disposal in the landfill area is FGD residue, made up primarily of gypsum (93% to 95%). Typical parameters for the residue materials to be produced at Belews Creek Steam Station are:

Gypsum	93% to 95%
Sulfite	0.35%
CO3	1.3%
CaF2	0.2%
Inerts	2.5% to 3.5%
Fly Ash Content	0.5% to 0.8%
pH	6.0 to 8.3
Unit Weight	76 lb/ft^3 to 97 lb/ft^3
Specific Gravity	2.35
Moisture	10% to 12%

5.2 Disposal Rates

The FGD residue generation rate projected for this facility will be on the order of 350,000 dry tons (438,000 wet weight) per year. In place density is estimated to be 89 lbs/ft^3.

The wastewater treatment system clarifier sludge that may be disposed in the landfill has a disposal rate of 50,000 tons per year. In place density is estimated to be 44 lbs/ft^3.

Future disposal rates may vary due to beneficial reuse or recycling efforts.

5.3 Service Area

The FGD gypsum landfill at Belews Creek Steam Station will receive FGD residue generated at only the Belews Creek facility.

5.4 Equipment Requirements

The equipment used to operate the landfill include tandem and tri-axle dump trucks, D65 Komatsu and/or CAT D7 low ground pressure dozers, watering trucks and maintenance trucks.

6.0 LANDFILL CAPACITY AND BORROW SOIL

6.1 Design Parameters

Landfill capacity calculations were performed for Phase 1 from proposed grading and closure plans using anticipated annual disposal rate data presented previously. The calculations account for operational cover and the final cover. The calculations presented in Appendix A assume the maximum waste generation rate of 350,000 dry tons per year of gypsum. An additional 50,000 tons of clarifier sludge per year may also be accepted at the landfill.

6.2 Operating Capacity

The gross capacity for the proposed landfill will be approximately 1.6 million cubic yards. Based on the worst case gypsum waste generation rates (assuming no reuse of the gypsum) at a density of 89 pounds per cubic foot and the potential clarifier sludge waste generation rates at a density of 44 pounds per cubic foot, the operating capacity for Phase 1 will be on the order of 3 years. If half of the gypsum is reused from recycling efforts, the operating capacity of Phase 1 is approximately 5 years. Capacity calculations are presented in Appendix A.

6.3 Borrow Soil Quantities

Based on the borrow soil quantity calculations presented in Appendix A, about 280,000 cubic yards of soil will be required for construction and closure of Phase 1. Borrow soil from excavation within the landfill footprint will provide approximately 600,000 cubic yards of borrow soil. Excess soil will be stockpiled north and east of the Phase 1 footprint as shown on the Erosion and Sediment Control Plan Stockpile Areas, Sheet CP-11.

TECHNICAL SPECIFICATIONS

DUKE ENERGY CORP.
BELEWS CREEK STEAM STATION
FGD RESIDUE LANDFIL, PHASE 1
STOKES COUNTY, NC

Prepared by:

CHAS. H. SELLS, INC.
Consulting Engineers, Surveyors & Photogrammetrists
128 Overhill Drive, Suite 105, Mooresville, NC 28117
Phone: (704) 662-0100
Fax: (704) 662-0101

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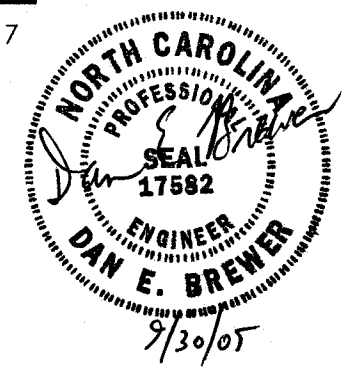

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1.0 INTRODUCTION

1.1 Scope of Work

The Work consists of furnishing all labor, materials, supervision, equipment and services necessary to complete the construction and installation of the Belews Creek FGD Gypsum Landfill, Phase 1 and all associated facilities and structures presented within these technical specifications and contract documents.

1.2 Definitions

The following list of definitions is provided for reference.

- Authorized Representative: shall mean the following named individuals, any of whom has the authority to approve construction quality assurance testing on behalf of the Owner: Dan E. Brewer, P.E. – Chas. H. Sells, Inc.

- Atterberg Limits: the liquid limit and plastic limit, (ASTM D4318-84).

- Classification System: the soil classification system shall be in accordance with the standard test method for classification of soils for engineering purposes (ASTM D2487-83).

- Compaction: the process of increasing the density or unit weight of soil by rolling, tamping, vibrating, or other mechanical means. Compaction tests are determined by the Sand Cone Method (ASTM D1556), Drive Cylinder Method (ASTM D2937), or Nuclear Density Gauge (ASTM D2922).

- Density: mass density of a soil is its weight per unit volume; usually reported in pounds per cubic foot. Maximum dry density as determined by Standard Proctor Test (ASTM D698) or Modified Proctor Test (ASTM D1557).

- Dewatering: process of lowering the groundwater level to permit construction activities to proceed "in the dry".

- Gas Well: shall mean a vertically installed slotted, perforated, or porous pipe with a solid riser pipe surrounded by a gravel-packed zone over the perforated pipe section to allow removal of landfill gas and any intercepted leachate.

- Geomembrane: an impermeable membrane liner or barrier used in civil engineering projects for waste and wastewater containment.

- Geonet: a geosynthetic net consisting of two or more traverse strands of synthetic fiber used in civil engineering to convey fluids and facilitate drainage.

- Geotextile: a relatively porous construction or reinforcement fabric used in civil engineering for geotechnical projects. The fabric structure may be knit, woven or nonwoven. Filter geotextile is a material which provides separation of materials with different pore size openings to prevent clogging. Drainage geotextiles are materials with adequate transmissivity to provide planar flow of fluid. Reinforcing geotextile is a material with sufficient strength to support some or all of the load applied to a composite system.

- Extrusion Weld: a bond between two high density polyethylene (HDPE) materials that is achieved by extruding a bead of HDPE material over the overlap area or between the overlapped sheets followed by pressure to create a water tight, homogeneous bond.

- In-Situ: "As is", or as it exists in-place natural.

- Double or Single Fusion Weld: a bond between two high density polyethylene (HDPE) materials that is achieved by heating both surfaces to the melting point followed by pressure in order to fuse the two HDPE surfaces together in a homogeneous bond. The welding method must use a power-driven system for heating and pressurizing with temperature control.

- Moisture Content: ratio of quantity of water in the soil (by weight) to the weight of the soil solids (dry soil), expressed in percentage; also referred to as water content (ASTM D2216).

- Optimum Moisture Content (OMC): moisture content corresponding to maximum dry density as determined in the Standard Proctor Test (ASTM D698-78) or Modified Proctor Test (ASTM D-1557).

- Permeability: ability of pore fluid to travel through a soil mass via interconnected voids. "High" permeability indicates relatively rapid flow of pore fluid and vice versa. Coefficients of permeability are generally reported in centimeters per second (cm/sec) and are determined by a flexible wall permeameter test (ASTM D5084-90).

- Plasticity: ability of a soil mass to be remolded without raveling or breaking apart. The plasticity index, numerically equal to the difference between the liquid and plastic limit, is a comparative number which describes the range of moisture contents over which a soil behavior is plastic.

1.3 Sequence of Construction

1.3.1 Phase 1 shall be constructed to the lines and elevations shown on the Contract Drawings and in accordance with these Technical Specifications. Any deviations from the Drawings or Technical Specifications require the prior written approval of the ENGINEER and must be documented by "as-built" revisions to the Drawings and/or Technical Specifications.

1.3.2 Phase 1 shall be constructed and prepared for receipt of waste in the following sequence:

SECTION	TITLE
2.0	Earthwork
3.0	Geomembrane Liner
4.0	Leachate Collection and Removal System (LCRS)
5.0	Operational Cover and Interior Berms
6.0	Stormwater Collection and Removal System (SWCR)
7.0	Roadways
8.0	Final Cap (not in contract)
9.0	Stormwater Basin
10.0	Lift Station, Valve Vault, and HDPE Forcemain

1.3.3. At or near completion of operation (filling of waste) of Phase 1, a final cap shall be installed in accordance with the permitted Closure Plan. This cap shall be completed by others under a separate contract.

1.3.4 During all phases of Phase 1 construction, construction will be tested, monitored, and evaluated prior to approval.

2.0 EARTHWORK

2.1 Site Preparation

2.1.1 Scope of Work

The EARTHWORK CONTRACTOR shall furnish all labor, materials, supervision, and equipment to complete the site preparation for the Belews Creek FGD Gypsum Landfill, Phase 1.

Site preparation includes the stripping of areas designated for earth fill and undercutting of unsuitable materials within earth fill foundation areas. Site preparation also includes the installation of stormwater management structures and erosion/sediment control as designated on the Contract Drawings.

2.1.2 Specification

2.1.2.1 Stripping

In areas to be stripped, all materials shall be removed to a depth necessary to eliminate topsoil and other soils containing more than 5 percent by weight fibrous organic matter, rubbish, vegetable matter, roots, small stones and all other perishable or objectionable matter. The minimum stripping depth shall be 6 inches or as determined by the ENGINEER. Stripped material shall be stockpiled on-site in areas for spoil materials as designated by the ENGINEER.

2.1.2.2 Undercutting

Materials determined unsuitable by the ENGINEER shall be excavated from a depth of 6 inches below the existing ground surface within the foundation area, the area 10 feet beyond the foundation area and any other area designated by the ENGINEER. (Depths of removal of 6 inches or less in areas designated above shall be considered stripping.) The lateral extent and required depth of undercutting will be determined by the ENGINEER. If necessary, the area of undercutting shall be dewatered to a depth of at least 2 ft. below the bottom of the excavation and shall be maintained in the dewatered condition until compacted earth fill is placed to at least 3 ft. above the original water level or to original ground level, whichever is higher.

The excavated material shall be stockpiled or disposed of as directed by the ENGINEER.

2.1.2.3 Stormwater Management Structures and Erosion/Sediment Control

Stormwater management structures shall include but not be limited to diversion ditches, drainage culverts, and drop inlets. Diversion ditches (permanent, temporary or along access roads) shall be constructed to divert run-on or route run-off to the erosion/sediment control structures as specified on the Contract Drawings.

Drop inlets shall be pre-cast concrete. Pre-cast concrete shall have a minimum 28 day compressive strength of 3000 psi.

Grates for drop inlets shall have NCDOT H-20 loading capacity. Proposed grates shall be submitted to the ENGINEER for review and approval prior to installation.

Culverts shall be reinforced concrete pipe with tongue and groove joints and shall conform to ASTM C76. Pipe shall be Class III. Pipe manufacturer certification shall be submitted to the ENGINEER.

Drop inlets shall be constructed to the lines and grades shown on the Contract Drawings. Grates shall be set in a full bed of mortar.

Culverts shall be extended through the drop inlet walls beyond the outside surface and sealed with masonry to provide a watertight connection.

Culverts shall be laid to the lines and grades shown on the Contract Drawings. The subgrade for the culverts shall conform with Section 2.2 of these Specifications. Unsuitable subgrade material shall be replaced with sand. Culverts shall be laid so that the grooved end is upgradient and the tongue end enters the grooved end to its full depth. Culvert joints shall be provided with a prefabricated seal.

Backfill of the drop inlets and culverts shall be placed and compacted in 6 inch thick lifts around the drop inlets and up to 2 feet above the culverts. Compaction shall be performed by hand tampers or small hand operated compactors such as a Rammax. Compaction shall be at a minimum 93 percent of the Standard Proctor maximum dry density. Additional compaction of lifts 2 feet or greater above the culverts shall conform to Section 2.2 of these Specifications.

Erosion/sediment control shall include but not be limited to installation of sediment basins, outlet protection aprons, silt fencing and grassing as specified on the Contract Drawings.

2.1.3 Measurement and Payment

2.1.3.1 Measurement of stripping will be the surface area of the original ground surface that is stripped. Computation of the quantities of stripping will be based on surveyed surface areas (provided by the EARTHWORK CONTRACTOR) stripped and will be mutually agreed to in writing by the ENGINEER and EARTHWORK CONTRACTOR for each particular area.

Payment will be made at the contract price per acre for stripping. This price will include excavation, hauling, stockpiling and/or disposing of the stripped material.

2.1.3.2 The quantity of undercutting will be based upon the volume between the stripped surface and the undercut surface as determined by surveys after completion of the undercutting and before placement of the earth fill. In the case of small volume areas of undercutting, it will be acceptable to use an average depth times area volume computation, if both the EARTHWORK CONTRACTOR and OWNER agree prior to measurement. Both the EARTHWORK CONTRACTOR and OWNER should be represented when average depth measurements are made.

Payment will be made at the contract price per cubic yard for undercutting. This price will include the excavation, hauling and stockpiling of the undercut material, replacement with compacted structural fill and any dewatering associated with the undercutting.

2.1.3.3 The quantity of diversion ditches will be based on the measured length of the ditch. Payment will be made at the contract price per foot of diversion ditch. This price will include excavation and establishment of an erosion-resistant grass cover for grass-lined ditches and placement of riprap and geotextile for riprap-lined ditches.

2.1.3.4 Payment for the drainage culverts will be made at the contract unit price per linear foot. This price will include trench excavation, placement of the culvert and backfill around the culvert.

2.1.3.5 Payment for sediment basins and outlet protection aprons will be made at the contract unit price. This price shall include complete construction of these structures as shown on the Contract Drawings.

2.1.3.6 Silt fencing will be paid for at the contract price per linear foot. This price shall include furnishing, constructing and maintaining the fences as may be required to obtain the specified results.

2.1.3.7 Payment for grassing will be made at the contract price per acre. This price shall include all labor, equipment and materials required to establish a permanent grass cover on all graded surfaces and maintain this grass cover until acceptance by the ENGINEER. This price shall also include any maintenance and protection required to control erosion on embankment fill surfaces prior to beginning permanent grassing operations.

2.2 Structural Fill

2.2.1 Scope of Work

The EARTHWORK CONTRACTOR shall furnish all labor, materials, supervision, and equipment, to complete earthwork for the Belews Creek FGD Gypsum Landfill, Phase 1 including, but not limited to excavation, hauling, scraping, stockpiling, and removing accumulated rainwater during construction. Phase 1 shall be excavated to the lines and grades shown on the Contract Drawings.

2.2.2 Materials

Structural fill material shall consist of the on site or borrow area natural soils free of all topsoil, roots, stumps, brush, vegetation, and other deleterious material. Structural fill shall have a maximum size aggregate of 4 inches with no more than 15% retained on the No. 4 sieve.

Excavated material that does not meet the above criteria for structural fill shall be termed non-select material and shall be stockpiled at the ENGINEER's direction on the property.

The ENGINEER will assist the EARTHWORK CONTRACTOR in the determination of structural fill, and non-select material. The EARTHWORK CONTRACTOR will be responsible for excavating, transporting, stockpiling, placing and compacting the structural fill.

2.2.3 Construction

During construction of a cell, a temporary sump may be constructed for accumulation of impounded waters resulting from rain or seepage. A pump with adequate capacity shall be provided and operated to maintain a low water level in the sump.

Structural fill, as defined above, shall be stockpiled in a designated area free of contaminating soil, clearing debris, or other objectionable materials. Stockpiles that will remain inactive for a period greater than 12 months shall be stabilized with seed. All reconditioning of structural fill to obtain the required compaction will be the responsibility of the EARTHWORK CONTRACTOR.

Non-select material, as defined above, shall be segregated from structural fill and disposed of as designated by the ENGINEER.

Excavation of a cell is complete when the ENGINEER determines the lines and elevations shown on the Contract Drawings have been achieved in the field. The EARTHWORK CONTRACTOR will be responsible for all surveying necessary for construction including as-built drawings.

The EARTHWORK CONTRACTOR shall protect all existing facilities, including, but not limited to existing electrical lines and poles, and monitor wells during excavations and stockpiling. Damaged facilities shall be promptly replaced at the EARTHWORK CONTRACTOR'S expense.

Structural fill shall be compacted in not greater than 8 inch thick lifts to a minimum of 95 percent of the standard Proctor maximum dry density, ASTM D-698, or as specified on the Contract Drawings.

2.2.4 Measurement and Payment

A grid pattern as approved by the ENGINEER of ground surface elevations in the cell area shall be surveyed and reference points installed by the EARTHWORK CONTRACTOR prior to excavation and prior to placement of the soil liner. The ENGINEER shall check the as-built finished grades and determine the excavated volume of soil based on survey data provided by the EARTHWORK CONTRACTOR.

Requested payment quantities will be submitted by the EARTHWORK CONTRACTOR with final approval by the ENGINEER.

If a dispute exists relative to payment quantities, the EARTHWORK CONTRACTOR at his expense will uncover any buried or covered material for re-evaluation.

Payment for excess excavation (stockpiled material not used for structural fill) will be based on the volume excavated times the unit price for excess excavation.

Payment for structural fill will be based on the volume filled times the unit price for structural fill.

2.3 Compacted Soil Layer

2.3.1 Scope of Work

Phase 1 shall include a smooth soil layer for placement of the geomembrane. The EARTHWORK CONTRACTOR shall furnish all labor, material, supervision, and equipment to complete the Compacted Soil Layer for the cells, including hauling, sieving, raking, discing, compacting, drying, wetting, removal of rainwater and removal of all previously placed material rendered unsuitable due to weather conditions or construction operations, final grading and sealing and all necessary and incidental items as detailed or required to complete the compacted soil layer, all in accordance with the Contract Documents.

2.3.2 Materials

Soil layer fill materials are potentially available in on-site and off-site stockpile areas that have been located by the OWNER and ENGINEER. Soil layer fill materials shall be classified according to the Unified Soil Classification System as SC, CL, CH, ML or MH (ASTM D 2487-83). Liquid limit, plasticity index, and percent passing the No. 200 sieve will be considered for proper classification.

Soil layer fill materials shall be reasonably free of gypsum, ferrous, and/or calcareous concretions and nodules or other deleterious substances. Soil layer fill material shall have a maximum size aggregate of 1 inch. The soil layer material shall be raked or sieved by the EARTHWORK CONTRACTOR, if necessary, to remove all aggregate greater than 1 inch in diameter. No more than 5 percent of the soil layer fill material should be retained on the No. 4 sieve.

Continuous and repeated visual inspection of the materials will be performed by the EARTHWORK CONTRACTOR to ensure proper soils are being used. In addition, the ENGINEER will make frequent inspections of the soil layer placement operations and materials, and will consult with the site personnel on suitable fill and locations of such. All soil layer fill proposed shall be inspected by the ENGINEER prior to actual use.

2.3.3 Sampling and Testing

2.3.3.1 Bulk samples will be taken by the ENGINEER at intervals of 1 sample per 5,000 cubic yards of material to be placed from the excavation areas or stockpile area(s).

The samples will be transported to a soils laboratory for compaction testing. The samples will be compacted to at least 95 percent of standard Proctor at moisture contents within 2 percent dry of optimum and 5 percent wet of optimum.

2.3.3.2 Sampling and testing of the compacted soil layer shall be as follows:

Test	ASTM Method	Frequency
Natural Moisture Content	D-2216	At every density test location and at least once per every 5,000 cubic yards placed.
Field Density	D-2937	Once per every other 100 foot by 100 foot grid per lift.

2.3.4 Construction

2.3.4.1 Prior to geomembrane placement, the prepared subgrade shall be proofrolled with a loaded dump truck (minimum 20 tons) or similar rubber tired pneumatic vehicle, a minimum of two passes in each direction. Proofrolling shall be conducted at the discretion of the ENGINEER or his representative. Any soft, saturated or yielding areas exhibited by pumping and/or rutting will require removal and replacement with the appropriate compacted soil as specified in the Site Preparation and Earthwork sections of these specifications.

2.3.4.2 After the lift to be compacted is conditioned, representative samples will be taken by the ENGINEER and tested for moisture content prior to any compactive efforts. If the moisture content is within the range specified in Section 2.3.4.7 below, compaction may begin. If the moisture content is outside of this range, the soil layer fill will be wetted or dried and reworked accordingly. The soil layer fill should be sprinkled or sprayed with water, utilizing equipment creating a uniform application and dozed, wind-rowed, and/or disc-plowed to uniformly increase the moisture content of the soil layer if the material moisture content is too low. The soil layer fill shall be dozed, wind-rowed, and/or disc-plowed to help air dry the soil if the moisture content is too high.

2.3.4.3 Each lift shall be thoroughly compacted and satisfy the moisture and density criteria. Field testing shall be completed before a subsequent lift is placed.

2.3.4.4 Compaction of lifts shall be as follows:

Compaction of lifts shall be performed with an appropriately heavy, properly ballasted, deep penetrating foot compactor equivalent to a CAT 815 and shall be subject to approval of the ENGINEER.

The daily work area shall extend a distance so as to maintain moist soil conditions (facilitate bonding) and continuous operations. Desiccation, crusting, or cracking of the lift surface shall be avoided as much as possible. Each lift shall be protected, at all times after placement, from desiccation and crusting.

If desiccation, crusting, or cracking of the lift surface occurs before placement of the next lift, this area shall be scarified to a sufficient depth to mix with moist materials, or sprinkled with water and then scarified at the direction of the ENGINEER.

The transition between the bottom and side slopes shall be accomplished by compacting parallel (bottom to top) to the slope.

Dozer equipment shall not be used for primary compaction efforts.

2.3.4.5 During compaction of the soil layer material, the soil moisture content and dry density shall be maintained within the limits specified below.

To determine the moisture content and dry density requirements of the compacted soil are being satisfied, field and laboratory tests shall be performed at the frequencies specified.

Compacted moisture content shall be between 2 and 5 percent wet of optimum moisture content or as determined by the results of the soil liner test pad.

The soil layer shall be compacted to a minimum dry density of 95 percent of the maximum dry density determined from the standard Proctor test (ASTM D-698). Where densities less than the specified density are measured, the soil layer shall be recompacted and/or removed and reworked to meet density criteria.

2.3.4.6 Soil layer fill shall not be placed or compacted during sustained periods with air temperature below 32°F. Soil layer fill may be placed and compacted during periods of early morning and early evening freezing temperatures with warming trends above 45°F during the day. No fill shall be placed on frozen subgrade. If the soil layer or structural fill freezes or ices the fill section shall be rescarified and recompacted, at the discretion of the ENGINEER.

2.3.4.7 During construction, finished lifts or sections of compacted soil layer shall be sprinkled with water as needed to avoid excessive cracking or at least twice daily.

2.3.4.8 The surface of the Compacted Soil Layer shall be smooth drum rolled and maintained free of rocks, organics, voids and sharp edges.

2.3.5 Measurement and Payment

Preparation of the subgrade for geomembrane placement shall be included in the measurement and payment for the structural fill and excavation.

3.0 GEOMEMBRANE LINER

3.1 Scope of Work

The GEOSYNTHETIC CONTRACTOR shall furnish all labor, materials, supervision and equipment to complete the Geomembrane Liner for the Belews Creek FGD Gypsum Landfill, Phase 1 including, but not limited to, anchor trench excavation and backfill, liner layout, seaming, patching, and all necessary and incidental items required to complete the work, in accordance with the Contract Drawings and these Specifications.

3.2 Materials

The geomembrane liner shall be made of high density polyethylene (HDPE) that is 60 mil nominal thickness. The geomembrane used shall meet, at a minimum, the standards included in Table 1.

The chemical resistance of the geomembrane liner shall be in keeping with typical properties of high quality polyethylene products currently available through commercial sources.

Geomembrane liner shall be shipped rolled with a protective wrap around each roll, labeled with roll number and manufacturer's batch number. Manufacturers quality control documentation shall be included with each roll.

The geomembrane shall be free of holes, blisters, undispersed raw materials, or any sign of contamination by foreign matter. Any such defect shall be repaired in accordance with the geomembrane manufacturer's recommendations. The ENGINEER may reject all or portions of units (or rolls) of the geomembrane if in his opinion significant quantities of production flaws are observed.

The GEOSYNTHETIC CONTRACTOR shall submit proposed geomembrane panel layouts to the ENGINEER at least 14 days prior to mobilization of crews (2 copies). Once the panel layout is approved, the GEOSYNTHETIC CONTRACTOR may not change the layout without permission of the ENGINEER.

3.3 Construction

The geomembrane liner shall be constructed as soon as practical after completion and approval of the Compacted Soil Layer. The top of the Compacted Soil Layer will be surveyed to ensure specified grades toward the collection sump area have been achieved. The geomembrane is to cover the bottom of the cell and the side slopes in accordance with the Contract Drawings.

3.3.1 Areas to receive liner installation should be relatively smooth and even, free of ruts, voids, etc, to the extent required by the ENGINEER. This shall be accomplished by final dressing of the compacted liner with smooth drum rollers. No vehicles are permitted on final dressed surfaces unless authorized by the ENGINEER.

3.3.2 An anchor trench (as illustrated on the Contract Drawings) will be required to secure the geomembrane. No loose soil will be allowed to underlie the geomembrane in the anchor trenches. The time schedule for excavation and backfilling of the anchor trenches is to be approved by the ENGINEER so that desiccation of trench soils does not occur prior to backfilling (See Section 3.3.10).

3.3.3 Installation of the geomembrane shall be as follows:

Unroll only those sections that are to be seamed together in one day. Panels should be positioned with the overlap recommended by the manufacturer, but not less than 4 in. For stormwater drainage purposes, the upstream panel should be overlapped on top of the downstream panel. The side slope geomembrane will be placed in an anchor trench that is then backfilled with soil and compacted as shown on the Contract Drawings.

After panels are initially in place, remove wrinkles as directed by the ENGINEER. Unroll several panels and allow the liner to "relax" before beginning field seaming. The purpose of this is to make the edges that are to be bonded as smooth and free of wrinkles as possible.

Once panels are in place and smooth, commence field seaming operations.

The GEOSYNTHETIC CONTRACTOR'S Field Superintendent will complete the Daily Report form (included at the end of this section) at the end of each day and submit the form daily to the engineer.

3.3.4 Field seaming shall be in accordance with EPA Technical Guidance document: "The Fabrication of Polyethylene FML Field Seams" EPA/530/SW-89/069 or as follows. Where conflicts exist between the two guidance specifications, the most stringent specification prevails unless otherwise directed by the ENGINEER.

All foreign matter (dirt, water, oil, etc.) shall be removed from the edges to be bonded. For extrusion welds, the bonding surfaces must be thoroughly cleaned by mechanical abrasion or alternate methods approved by the ENGINEER to remove surface cure and prepare the surfaces for bonding. All abrasive buffing shall be

performed using No. 80 grit or finer sandpaper. The grinding shall be performed so that any and all grind marks are perpendicular to the edge of sheet. No grinding greater than 1/4 inch outside the welds is permitted or the ENGINEER can require patching. No solvents shall be used to clean the geomembrane liner.

As much as practical, field seaming shall start from the top of the slope down. This will minimize large wrinkles from becoming trapped that requires cutting and patching. Tack welds (if used) shall use heat only; no double sided tape, glue or other method will be permitted. The geomembrane should be seamed completely to the ends of all panels to minimize the potential of tear propagation along the seam.

The completed liner shall not exhibit any "trampolining" during late morning to early evening hours. All areas exhibiting trampolining must be repaired as directed by the ENGINEER. Additional slack (ie: 1-3%) shall be allowed on the side slopes to reduce the potential for trampolining.

Seaming of the cell bottom membrane to the sidewall membrane (toe seam) shall be conducted when conditions minimize thermal expansion effects. All toe seams must be a minimum of 10 feet horizontally from the toe of the slope. Horizontal seams will not be allowed on the side slopes unless approved by the ENGINEER.

At the end of each day or installation segment, all unseamed edges shall be anchored by sand bags or other approved device. Sand bags securing the geomembrane on the side slopes should be connected by a rope fastened at the top of the slope by a temporary anchor. If high winds are expected, boards along the edge of unseamed panels, with weighted sand bags on top, may be used to anchor the geomembrane on the bottom of the cell. Sand bags fastened by rope should be used to secure unseamed edges on the side slopes. Staples, U-shaped rods or other penetrating anchors shall not be used to secure the geomembrane on the side slopes. Any damage to the liner or soil liner including damage due to construction activities or wind, rain, hail, or other weather shall be the sole responsibility of the GEOSYNTHETIC CONTRACTOR.

3.3.5 Field seaming may be fusion or extrusion welding or a combination of these methods. Solvent welding is not acceptable. The ENGINEER reserves the right to reject any proposed seaming method it believes unacceptable. Double hot wedge fusion welding shall be the predominant seaming method. Additional concepts and requirements of proper field seaming include the following:

Extrusion welding applies a molten bead of material to preheated sheets of geomembrane. The sheets are then joined by pressure.

The fusion welding process heats the area to be joined to the melting point and then applies pressure to join the melted surfaces.

The sheets to be joined shall be overlapped at least 4 inches after the necessary cleaning, aligning and cutting.

The seams should be oriented parallel to the line of maximum slope, i.e., oriented up and down, not across, the slope. In corners and odd shaped geometric locations, the number of field seams should be minimized.

No horizontal seams should be within 10 feet from the toe of the slope. No horizontal seams will be allowed on the side slopes unless approved by the ENGINEER.

No seaming should be attempted above 40°C (104°F) ambient air temperature. Below 5°C (41°F) ambient air temperature, preheating of the geomembrane will be required, unless it is demonstrated that this is not necessary (i.e., acceptable test (start-up) seams that duplicate, as closely as possible, actual field conditions can be achieved). Preheating may be achieved by natural and/or artificial means (shelters and heating devices). Ambient temperature is measured 18 inches above the liner surface. The GEOSYNTHETIC CONTRACTOR shall supply instrumentation for measurement of ambient temperature.

A moveable protective layer of plastic or approved material may be placed directly below each overlap of geomembrane that is to be seamed. This is to prevent any moisture build-up between the sheets to be welded. The protective layer must be removed after welding.

Seaming will extend to the outside edge of panels to be placed in anchor trenches.

If required, a firm substrata should be provided by using a flat board, or similar hard surface directly under the seam overlap to achieve proper support across the anchor trench.

Grinding prior to welding shall be done perpendicular to the sheet edge. Overgrind greater than 1/4 inch beyond the welded seam or improperly ground areas shall be replaced at the GEOSYNTHETIC CONTRACTOR'S expense.

Seams at the panel corners of 3 or 4 sheets shall be completed with a circular patch approximately 12 inches in diameter, extrusion welded to the parent sheets, or with a "T" weld at suitable locations.

3.3.6 Quality Control

All geomembrane sheet, seams and patches will be tested and evaluated prior to acceptance. In general, testing of the sheet will be conducted by the manufacturer. Testing of the seams will be conducted by the GEOSYNTHETIC CONTRACTOR under observation by the ENGINEER. The ENGINEER or a designated, independent geosynthetics laboratory may perform additional testing, as required by these detailed Specifications or as required in the judgement of the ENGINEER to verify that the HDPE sheet and seams meet the specifications. Testing requirements are detailed in the following subsections:

3.3.6.1 Pre-shipping Sheet Tests - The GEOSYNTHETIC CONTRACTOR or supplier (manufacturer) will be required to submit his Quality Control program to the ENGINEER prior to initiating field work. At a minimum, the Manufacturer will perform the tests at the frequencies given in Table 2 on the HDPE sheet prior to shipping HDPE material to the site.

3.3.6.2 Test Seams (Destructive Tests) - The Installer shall maintain and use equipment and personnel at the site to perform testing of test seams. Test seams will be made at least twice each day prior to commencing field seaming. These seams will be made on fragment pieces of geomembrane liner to verify that seaming conditions are adequate. Such test seams will be made at the beginning of each seaming period, when changes in storing equipment occur, at the ENGINEER'S discretion, and at least once every four hours during continuous operation of each welding machine. The ENGINEER shall complete the Test Seam Form immediately after each test. Requirements for test seams are as follows:

The test seam sample will be at least 1.8m (6 ft) long by 0.3m (1 ft) wide with the seam centered lengthwise. Six adjoining specimens 25mm (1 in) wide each will be cut from the test seam sample. At the ENGINEER'S option the shear tests may be eliminated for test seams. These specimens will be tested in the field with a tensiometer and/or manual seam tester for both shear (3 specimens) and peel (3 specimens). For dual wedge both inside and outside welds should be tested in peel. Test seams will be tested by the GEOSYNTHETIC CONTRACTOR under observation of the ENGINEER. The specimens should not fail in the weld. The GEOSYNTHETIC CONTRACTOR shall supply all necessary knowledgeable

personnel and all necessary testing equipment. Quantitative strength measurements from a calibrated field tensiometer, supplied by the Installer, need only be obtained for one specimen in shear and peel for the test seams. A passing test seam will be achieved when the criteria described below are satisfied. If a test seam fails, the entire operation will be repeated. If the additional test seam fails, the seaming apparatus or welding technician will not be accepted and will not be used for seaming until the deficiencies are corrected and two consecutive successful full test seams are achieved. Test seam failure is defined as failure of any one of the specimens tested in shear or peel.

The ENGINEER will approve all test seam procedures and results. The remainder of the successful test seam sample and the tested specimens will be assigned a number and marked accordingly by the ENGINEER, who will also log the date, hour, ambient temperature, number of seaming unit, name of seamer, and pass or fail description. The sample itself should be retained in the ENGINEER'S archives. The ENGINEER will transmit these specimens to the OWNER following acceptance of the geomembrane materials and installation by the ENGINEER.

3.3.6.3 Production Seams - 100% of the production seams will be tested by the GEOSYNTHETIC CONTRACTOR continuously using non-destructive techniques and at specified intervals using destructive tests.

All areas failing nondestructive test procedures described below shall be clearly marked both on the liner itself and on the Seam Inspection Quality Control Form included at the end of this section.

3.3.6.4 Nondestructive Testing

Single Weld Seams - the GEOSYNTHETIC CONTRACTOR shall maintain and use equipment and personnel at the site to perform continuous vacuum box testing under the observation of the ENGINEER on all single weld production seams except those corner seams where vacuum box testing is impossible. The system shall be capable of applying a vacuum of at least 3 psi. The vacuum shall be held for a minimum of 15 seconds for each section of seam. Where vacuum box testing is not possible, spark testing or an approved alternative will be used.

Double Weld Seams - the GEOSYNTHETIC CONTRACTOR shall maintain and use equipment and personnel to perform air pressure testing under the observation of the ENGINEER of all double weld seams greater than 20 ft. (double weld seams less than 20 ft. may be vacuum box tested as specified above). The system

shall be capable of applying a pressure of at least 30 psi for not less than 5 minutes. After the 5 minute testing period, the end of the seam shall be cut open and pressure loss monitored to verify the entire length of the seam channel is open. If no pressure loss is realized, the location of the blocked channel must be found and the remainder of the seam tested separately. The air pressure test results shall be documented on the Seam Inspection Quality Control Form enclosed in the "Supplement to the Technical Specification". Pressure loss tests shall be conducted in accordance with the procedures outlined in "Pressurized Air Channel Test for Dual Seamed Geomembranes," Geosynthetic Research Institute Test Method GM-6. As outlined by the test method, following a 2 minute pressurized stabilization period, pressure losses over a measurement period of 5 minutes shall not exceed 3 psi.

Double weld seams will also be visually inspected on 100% of the seam. If necessary the outside flap can be pulled back to aid in the visual inspection.

3.3.6.5 Destructive Testing

Laboratory destructive testing (LDT) is defined as 18" X 36" samples cut on 500' centers for both extrusion and double welded seams. Field destructive testing (FDT) is defined as 3" X 6" samples cut at the end of each seamed area exceeding 200 feet. Both are described below:

Laboratory destructive testing will be performed on an average of every 500 linear feet of production seam. The locations will be selected by the ENGINEER. Samples will be 18" X 36" in order to provide one sample to the archive, one sample to the ENGINEER for laboratory testing, and one sample to be retained by the GEOSYNTHETIC CONTRACTOR for possible field and/or additional laboratory testing at the option of the ENGINEER OR GEOSYNTHETIC CONTRACTOR.

Before the sample is sent to the laboratory, two specimens, one from each end of the specimen will be tested in peel in a calibrated field tensiometer, supplied by the GEOSYNTHETIC CONTRACTOR. Both specimens must meet the qualitative and quantitative criteria listed below. Testing requirements are as follows: Each sample shall be large enough to test five specimens in peel and five specimens in shear. The average values of each set of five specimens must meet the specification, and four of the five specimen tests must meet the specifications for the seam to be considered a passing seam. If the average of the five specimens is adequate, but one of the specimens is failing, values for the failing specimen must be at least 80 percent of the values required for the seam for the sample to

pass. All samples must fail in film tear bond (FTB). Tests shall be conducted using a calibrated tensiometer and must meet the criteria outlined below. Samples which do not pass the shear and peel tests will be re-sampled from locations at least 10 feet on each side of the original location. These two re-test samples must pass both shear and peel testing. If these two samples do not pass, then additional samples will continue to be obtained until the questionable seam area is defined. If the length of the questionable seam area is defined to be excessive by the ENGINEER, a cap pad may be required over the entire seam with nondestructive testing prior to acceptance of the seam.

Field Destructive Testing (FDT) shall be 3" X 6" samples cut out at the beginning and end of each seam exceeding 200' in length. Three 1" X 6" specimens will be tested in peel from each sample using the GEOSYNTHETIC CONTRACTOR's Field Tensiometer or qualitative peel tester at the option of the GEOSYNTHETIC CONTRACTOR. No qualitative peel strength values need to be determined with the Field Destructive test, but each specimen must meet all qualitative criteria listed below.

The ENGINEER will approve all seam field and laboratory test procedures and results. All laboratory destructive test specimens will be marked with the seam number and letters then bound together for a particular seam and stored in the OWNERs archives. The specimens for the FDT need not be retained.

Each sample area will be clearly marked both on the liner itself (LDT or FDT) using the procedures outlined in the marking section and on the "as- built" record drawing prepared by the GEOSYNTHETIC CONTRACTOR. Destructive Test Forms will be retested and completed by the ENGINEER.
All areas cut out for testing should be immediately patched by the GEOSYNTHETIC CONTRACTOR and the patches should be tested and approved by the ENGINEER. Patches shall extend a minimum of 6 inches beyond the cut area.

A passing double welded seam will be achieved in peel (ASTM D413 modified) when (1) failure is by Film Tear Bond (FTB), National Sanitation Foundation (NSF), Standard 54, definition 2.16, and (2) yield strength for the seam is not less than 62 percent of the minimum tensile strength at yield as specified in Table 1, and (3) no greater than 10 percent of the seam width peels (separates) at any point and (4) the failed sheet exhibits elongation, prior to failure. Both sides of the double welded seam must be tested and must meet all of the criteria listed above. A passing double welded seam will be achieved in shear (ASTM D3083 modified) when (1) failure is by FTB, and (2) yield strength for the seam is not

less than 95 percent of the minimum tensile strength at yield specified in Table 1, and (3) yield strain for the seam is at least 10 percent and (4) break strain for the seam is at least 50 percent.

A passing extrusion welded seam will be achieved in peel (ASTM D413 modified) when (1) failure is by FTB, and (2) yield strength for the seam is not less than 62 percent of the minimum tensile strength at yield specified in Table 1, and (3) no greater than 1/8-inch separation occurs from the edge of the sheet at any point and (4) the failed sheet exhibits ductility prior to failure. A passing extrusion welded seam will be achieved in shear (ASTM D3083 modified) when (1) failure is by FTB, and (2) yield strength for the seam is not less than 95 percent of the minimum tensile strength at yield specified in Table 1, and (3) yield strain for the seam is at least 10 percent, and (4) break strain for the seam is at least 50 percent.

3.3.7 Repair of Damaged and Sampled Areas

Damaged and sample coupon areas of geomembrane shall be repaired by the GEOSYNTHETIC CONTRACTOR by construction of an extrusion welded patch. Patches shall extend a minimum of 6 inches beyond the damaged or cut area. No repairs shall be made to seams by application of an extrusion bead to a seam edge previously welded by fusion or extrusion methods. Repaired areas will be tested for seam integrity as outlined in Section 3.3.6.4. Damaged materials are the property of the GEOSYNTHETIC CONTRACTOR and will be removed from the site at the GEOSYNTHETIC CONTRACTOR's expense. The GEOSYNTHETIC CONTRACTOR will retain all ownership and responsibility for the geomembrane until acceptance by the ENGINEER. The geomembrane shall be accepted by the ENGINEER after the installation and repair are complete, and after the ENGINEER has received all necessary documentation for the installation in accordance with these specifications.

3.3.8 Seams at Panel Corners

Seams at the panel corners of 3 sheets shall be completed with a circular patch approximately 12 inches in diameter, extrusion welded to the parent sheet, or with a "T" weld at suitable locations. If the GEOSYNTHETIC CONTRACTOR wishes to use a different method, samples must be submitted to the ENGINEER and tested accordingly.

3.3.9 Potentially Damaging Activities

No support equipment unless approved by the ENGINEER shall be allowed on the geomembrane. Personnel working on the geomembrane shall not smoke, wear damaging shoes, or engage in any activity that could damage the geomembrane.

3.3.10 Anchor Trench Backfilling

The anchor trench will be backfilled and compacted by the GEOSYNTHETIC CONTRACTOR to a dry density not less than 90 percent of the maximum dry density determined by the standard Proctor (ASTM D-698). Care should be taken when backfilling the trench to prevent any damage to the geomembrane. Anchor trench spoil shall be used as backfill material, wherever acceptable by the ENGINEER. The time schedule for excavation and backfilling of the anchor trench is to be approved by the ENGINEER so that desiccation of trench soils does not occur prior to backfilling.

The geomembrane shall be anchored with continuous sand bags immediately after installation of the geomembrane. The anchor trench shall be backfilled a minimum of 12 inches as specified above. The GEOSYNTHETIC CONTRACTOR shall be responsible for keeping the anchor trenches from accumulating ponding surface water. This may be accomplished by supplying temporary drainage outlets along the outside edge of the anchor trench.

3.3.11 Protection of Leading Edges

Between construction of partial sections of the geomembrane liner, leading edges of the geomembrane may be exposed or buried for extended periods of time prior to their joining to adjacent, subsequent membrane sections. The combined action of abrasive soil and equipment impact stresses may "etch" unprotected membrane surfaces sufficiently to affect seam strengths. Therefore, it is necessary to protect leading edges in high activity areas with sacrificial layers of geotextile and HDPE sheet until they are ready for final seaming. As a minimum, each leading edge to be seamed that must be buried or which must be exposed for periods of one month or longer shall be continuously covered by a layer of HDPE sheet. The geotextile shall be nonwoven and have a minimum weight of 8 oz per square yard. The sacrificial HDPE sheet shall have a minimum thickness equal to that of the geomembrane liner to be protected. Both protective layers shall have a minimum width of 2 feet. The protective cover sheets shall be either covered with soil or weighted with sand bags to prevent displacement by wind. The edge of the sheet to be protected shall be approximately centered beneath the overlying protective layers prior to burial or weighing with sandbags. Leading edges located in areas

expected to receive direct traffic from construction equipment shall be buried under a minimum thickness of one foot of buffer soil.

3.3.12 Warranty

The GEOSYNTHETIC CONTRACTOR shall provide a one year non pro-rata warranty on materials and installation workmanship including all joints, penetrations, seams and connections, whether prefabricated or constructed in the field for the following geosynthetic materials: geomembrane liner, geonet, geotextiles, temporary cover liner, sump, manhole, leachate collection pipe and leachate removal pipe. The warranty shall cover the damage due to the following occurrences:

- Stress cracking, weathering, and ultra violet degradation.
- Excessive wrinkling or trampolining due to the effects of thermal expansion/contraction.
- Ruts in the soil liner from erosion or construction vehicles.
- Any activity from the GEOSYNTHETIC CONTRACTOR such as backfilling and soil placement on top of the liner.
- Over heating, over grinding, or improper or inadequate quality control by the GEOSYNTHETIC CONTRACTOR of any joint, seam, penetration or connection, whether prefabricated or field installed.
- Any other cause or Contractor-related activity not herein mentioned excluding non-compatibility between the geosynthetic materials and the leachate; vandalism; or natural and unnatural occurrences including but not limited to tornadoes, hurricanes, flooding, etc.

If it is determined by the procedures described herein that the damage is the fault of the GEOSYNTHETIC CONTRACTOR, the GEOSYNTHETIC CONTRACTOR will, at no cost to the OWNER, expose the damaged area as needed to replace and/or repair the damaged material at the discretion of the ENGINEER, perform the necessary quality control testing and rebackfill or restore the area as required.

If it is determined that the damage is the fault of the OWNER, the GEOSYNTHETIC CONTRACTOR will be reimbursed at a mutually agreed

price for the services rendered. Within the warranty period at the request of the OWNER, regardless of which party is ultimately held responsible, the GEOSYNTHETIC CONTRACTOR will send a representative to the site within five days of the request to assist in the assessment and evaluation of the problem. The GEOSYNTHETIC CONTRACTOR will then agree to send a crew to repair and/or replace as agreed within ten days from the date of evaluation.

3.3.13 Marking on Liner by the Geosynthetic Contractor

The GEOSYNTHETIC CONTRACTOR will mark directly on the liner as described herein for the purpose of readily identifying panels, seams, repairs and destructive test locations.

Panel Identification

Each panel indicated on the pre-construction panel layout drawings will be numbered sequentially using the format P1, P2, etc. Panels in the field must be numbered in the order in which the panels are actually laid regardless of preconstruction numbering. The panels will be permanently marked in white (red for white surfaced geomembrane) with letters approximately 12" high (and 1/3 the way down the slope for liners on slope). In addition, each panel will be marked with the Manufacturer's roll number.

Seam Identification

Each seam will be assigned a sequential number. Documentation will be completed for each seam showing panels that are joined by the seam.

Quality Control Marking

Following the completion of each seam, patch or repair the welding technician will write, at the end of the seam or in the middle of the patch or repair, the following: the initials of the technician, date welded, time welded, and welder unit number. The markings will be done clearly with a white or red permanent marking pen or pencil.

Similarly, after each quality control test, the CQAO or a representative will record the following immediately adjacent to the area tested: initials of QC TECHNICIAN performing the test, date of the test, type of test (i.e. VB, SP, AP for vacuum box, spark test and air pressure test respectively) and the words "pass" or "fail". For the air pressure test, the QC TECHNICIAN must also define the

limits or zone of the test as well as the amount of pressure loss observed. Again, a permanent white or red marking pen is required. If the test fails and the necessary repair is made, the technician will cross out the previous markings and mark appropriately for the new test results.

Destructive test samples will be clearly circled with white or red permanent pen or pencil and the words "FDT" or "LDT" as defined in the specifications and marked in white or red. The CQAO will mark the words "pass" or "fail" as appropriate. Similarly, any other area needing repair will be clearly marked in white or red permanent pen or pencil to identify where the repair is required to be made.

The CQAO will also mark areas in need of repair using white (red for white surfaced geomembrane) marking pens.

Graffiti, offensive marking or any marking not necessary to the project will not be permitted.

3.3.14 Documentation

Forms by the GEOSYNTHETIC CONTRACTOR:

- Subgrade Certification (signed daily by the GEOSYNTHETIC CONTRACTOR'S Superintendent and submitted to the CQAO)
- Seam Test Weld Form (completed after each seam test by the Welding Technician for approval by the CQAO.)
- Pre-construction shop drawings and post construction as built record drawings.

Forms by the CQAO:

- CQAO Daily Report
- Field Inventory Control
- Seam Inspection Report
- Geomembrane Deployment
- Laboratory Destructive Testing
- Geomembrane Repair Record

3.3.15 Daily Meetings

At the beginning of each work day the EARTHWORK CONTRACTOR'S Superintendent, the GEOSYNTHETIC CONTRACTOR'S Superintendent, and the CQAO will meet to discuss the upcoming work plan for all parties to ensure cooperation, communication and understanding.

3.3.16 GEOSYNTHETIC CONTRACTOR Equipment and Personnel

The GEOSYNTHETIC CONTRACTOR shall provide an individual whose title is "Quality Control Foreman" (QCF) who shall be experienced in all phases of quality control testing and procedure. The QCF will be dedicated to performing or directing the GEOSYNTHETIC CONTRACTOR'S quality control activities, (i.e. air pressure, vacuum box and spark non-destructive testing and field destructive testing). The QCF will not be involved in liner seaming or installation. The QCF and the Superintendent may be the same person if approved by the ENGINEER.

During the installation of the 60 mil geomembrane liner for the landfill, the GEOSYNTHETIC CONTRACTOR shall supply and maintain at least three extrusion welders and three double hot wedge fusion welders, at least one of which must be available at the working space at all times. At least one extra generator shall be supplied and maintained by the GEOSYNTHETIC CONTRACTOR to be used as a spare.

During the installation of the 60 mil geomembrane liner for the landfill, the GEOSYNTHETIC CONTRACTOR shall provide a minimum crew size of 9, for which at least 6 must be qualified installers (a Superintendent and/or QCF and a minimum of 4 skilled technicians).

3.4 Measurement and Payment

3.4.1 Payment for work shall be made at the contract unit price per square foot of in-place geomembrane liner excluding the anchor trench, scrap and overlap. Measurements will be based upon the survey conducted for installation of the compacted soil liner. Pay limits shall be measured to the lower outside corner of exterior anchor trenches.

3.4.2 All testing, patches, repairs, and material beyond the required pay limits will be considered incidental to the construction and will be excluded from payment. All materials and work required of the GEOSYNTHETIC CONTRACTOR for

excavation, backfill, and compaction of the anchor trench are also considered incidental to the installation and will also not be included as pay items.

3.4.3 Requested payment quantities for in-place geomembrane will be submitted by the GEOSYNTHETIC CONTRACTOR with final approval by the ENGINEER.

TABLE 1

REQUIRED PHYSICAL PROPERTIES OF MEMBRANE LINER (HDPE) SHEET

Property	Test Method	Required Values 60 Mil HDPE
Thickness	ASTM D-751	57/60 Mil*
Sheet Density	ASTM D-1505	0.940 g/cm^3 min.
% Elongation at Yield	ASTM D-638	12
% Elongation at Break	ASTM D-638	560
Tensile Stress at Yield	Test Specimen Type IV	130 lb/in. min.
Tensile Strength at Break	ASTM D-638	228 lb/in. min.
Carbon Black Content	ASTM D-1603	2% min-3% max
Carbon Black Dispersion	ASTM D-3015	A1, A2, or B1
Environmental Stress-Crack Resistance	ASTM D-1693 (As Modified in NSF54 Appendix A)	1500 hours min.
Low Temperature Impact	ASTM D-746	-60°F or lower
Dimensional Stability	ASTM D-1204 (As Modified in NSF54 Appendix A)	± 2.0% max.
Puncture Resistance	FTMS 101B Method 2065	70 lb. min.
Tear Resistance	ASTM D 1004	42 lb. min.
Water Vapor Transmission Rate	ASTM E96 DIE C	0.03 gm/m^2-day
Seam Strengths		
1. Shear Strength	ASTM D-3083 as modified in NSF 54 Appendix A	FTB, 95% of Parent material (min.)
2. Shear Strain @ Yield	ASTM D-3083 as modified in NSF 54 Appendix A	10% (min.)
3. Peel Strength	ASTM D-413 as modified in NSF 54 Appendix A	FTB, 62% of Parent Material (min.)
Non-Destructive Testing		
1. Single Weld	Continuous Vacuum Box;	Maintain Vacuum of
2. Double Weld	Air Testing	Maintain 30 psi for not greater than 3 psi for last 3 minutes

* Thickness - first value represents lowest individual value
- second value represents average across roll

TABLE 2

REQUIRED PRE-SHIPPING SHEET TESTING
OF
MEMBRANE LINER (HDPE)

Property	Test Method	Frequency
Thickness	ASTM D-1593 Par. 8.1.3	Each Roll
Sheet Density	ASTM D-792 Method A	Each Roll
Tensile Properties	ASTM D638, Type IV (As modified in NSF54)	Each Roll
Tear Resistance	ASTM D1004 Die C	Every Fifth Roll
Puncture Resistance	FTMS 101B Method 2065	Every Fifth Roll
Carbon Black Content	ASTM D1603	Every Fifth Roll
Carbon Black Dispersion	ASTM D-3015 (As modified in NSF54)	Every Fifth Roll
Dimensional Stability	ASTM D1204 (1 Hour @ 100°C)	Every Tenth Roll
Low Temperature Impact	ASTM D746	Every Tenth Roll
Melt Flow Index	ASTM D1238 Condition 190/2.16 (At two normal loads)	Every 180,000 lb. (81.6 mg) of Resin*
Environmental Stress Crack	ASTM D-1693 Appendix A (As modified in NSF54)	Every 180,000 lb. (81.6 mg) of Resin*

* Or at least once per railcar for railcars containing less than 180,000 lb (81.6 mg).

4.0 LEACHATE COLLECTION AND REMOVAL SYSTEM (LCRS)

4.1 Scope of Work

The GEOSYNTHETIC CONTRACTOR shall furnish all labor, materials, supervision, and equipment necessary to complete the LCR System including geonet and geotextile, geocomposite, temporary cover liner with ballast system, pipe and sump, aggregate with geotextile, and all necessary and incidental items required to complete the work, all in accordance with the Contract Drawings and these Specifications.

4.2 Materials

4.2.1 Pipe and Sump

PE piping and sump shall have nominal diameters as noted on the Contract Drawings.

PE piping inside the landfill cell shall have a Standard Dimension Ratio (SDR) of 17 unless otherwise specified on the Contract Drawings. The sump shall be to the dimensions as specified on the Contract Drawings. The PE pipe sections and sump shall be perforated as shown on the Contract Drawings.

The PE pipe and sump shall be manufactured from first quality virgin high density polyethylene resin conforming to ASTM D1248 (Type III, Class C Category 5, Grade P34), ASTM D3350 (Cell Classification PE 345434C), and having a Plastic Pipe Institute (PPI) Rating of PE 3408. The HDPE pipe shall have the following nominal properties.

Property	ASTM Test Method	Required Value
Density	D1505	0.941 to 0.955 gm/cm^3
Melt Flow Index (Cond. °F)	D1238-E	<0.15 gms/10 min
Environmental Stress	D1693-C	5000 hours
Tensile Strength @ Yield	D638	3500 psi
Modulus of Elasticity	D790	110,000 psi
Hydrostatic Design Basis	D2837	1500 psi @ 23°C
Carbon Black Content	D1603	2% min

The chemical resistance of the PE pipe, manholes, sumps and all fittings shall be in keeping with typical properties of high quality polyethylene products currently available through commercial sources and equal to or greater than that of the 60 mil

HDPE geomembrane specified. All mechanical fasteners or fittings shall be stainless steel.

All interior pipe welds shall be machine butt fusion welded and sufficiently strong to develop the full strength of the joined sections, e.g. failure should not occur in a weld.

All exterior pipe welds (leachate discharge pipe) between HDPE manholes shall be machine double-butt fusion welded and demonstrated to be watertight and sufficiently strong to develop the full strength of the joined sections. The pipe shall be air or hydrostatically tested with a minimum of 10 psi. A loss of pressure shall not exceed 1 psi over a 30 minute period. If a pressure drop or loss of head is encountered, the defective joint shall be located and satisfactorily repaired followed by additional testing until satisfactory results are achieved.

HDPE pipe connections to the manholes shall be extrusion welded to provide a water-tight seal. The HDPE manholes shall be water-tight the entire height of the manhole.

4.2.2 Aggregate

Gravel surrounding the perforated PE piping shall be of subangular, subrounded, rounded or well rounded particle shape, and shall meet NC DOT No. 57 aggregate specifications. Gravel surrounding the HDPE sump shall meet NC DOT No. 5 aggregate specifications.

The aggregate for the leachate collection pipe shall be wrapped with a nonwoven geotextile as specified in Section 4.2.3.

4.2.3 Nonwoven Geotextile

Nonwoven geotextile for the cell floor, side slopes, and leachate pipe shall be 8 oz/sy geotextile. Nonwoven geotextile used beneath the cell divider berms shall be 12 oz/sy geotextile. The geotextile used shall be ultraviolet stabilized: 1) a nonwoven, needlepunched, continuous filament polyester material, 2) a nonwoven, needlepunched continuous filament polypropylene material, or 3) a nonwoven, needlepunched staple fiber polypropylene material.

The nonwoven material shall meet or exceed the following criteria using Minimum Average Values:

Property	ASTM Test Method	8 oz Geotextile Required Value	12 oz Geotextile Required Value
Fabric Weight	D3776	7.5 oz/yd^2	11.5 oz/yd^2
Grab Strength	D4632	200 lbs.	300 lbs.
Grab/Tensile	D4632	50%	60% Elongation
Wide Width Tensile Strength (M.D.)	D4595	50 lbs/inch	50 lbs/inch
Hydraulic Conductivity	D4491	0.2 cm/sec	0.2 cm/sec
Puncture Strength	D4833	95 lbs.	135 lbs.
Tear Strength	D4533	60 lbs.	110 lbs.
Apparent Opening Size (AOS)	D4751	100 Sieve	70 Sieve
Mullen Burst Strength	D3786	275 psi	400 lbs.
UV Resistance	D4355	70% (@ 500 hours)	70% (@ 500 hours)

The CONTRACTOR shall submit quality control certificates for each batch of resin. The certificates shall contain lot, batch and roll numbers; sampling and test methods; and results of quality control tests. The quality control testing and frequencies shall meet the following:

Property	ASTM Test Method	Frequency
Fabric Weight	D3776	Every 100,000 ft^2
Grab Strength	D4632	Every 100,000 ft^2
Grab/Tensile Elongation	D4632	Every 100,000 ft^2
Wide Width Tensile	D4595	Every 100,000 ft^2
Hydraulic Conductivity	D4491	One Per Lot
Puncture Strength	D4833	Every 100,000 ft^2
Tear Strength	D4533	Every 100,000 ft^2
Apparent Opening Size	D4751	One Per Lot
Mullen Burst Strength	D3786	Every 100,000 ft^2

Samples that do not meet the specified properties shall be rejected, and additional testing of rolls may be required by the manufacturer to identify any additional non-complying rolls.

Geotextile rolls shall be supplied in a protective wrap and tagged with the manufacturer, product name, lot or batch number, and roll number.

The ENGINEER, at his discretion, may randomly sample the geotextile for conformance testing.

4.2.4 Geonet

The synthetic mesh structure shall consist of solid rib extruded high density polyethylene. A sample of the geonet shall be submitted to the ENGINEER for approval prior to acceptance.

The geonet shall have an 8 oz. nonwoven geotextile above the geonet. The 8 oz. nonwoven geotextile shall conform to Section 4.2.3. No bonding is required between the nonwoven geotextile and the geonet.

The geonet shall meet or exceed the following minimum average roll values:

Property	ASTM Test Method	Required Value
Density	D1505 or D792	0.94 g/cm^3
Wide Width Tensile Strength (M.D.)	D4595	45 lb/in.
Carbon Black Content	D1603	2% min to 3% max

The geonet and filter geotextile for the floor shall have a minimum transmissivity of $1x10^{-4}$ m^2/sec at a hydraulic gradient of 0.25 and a normal load of 10,000 psf at a time of 100 hours when tested with compacted soil against the filter geotextile and 60 mil HDPE against the geonet, tested in accordance with ASTM D4716. Transmissivity curves with the specified conditions must be submitted to the ENGINEER before the geonet and geotextile materials will be accepted.

The CONTRACTOR shall submit quality control certificates for each batch of resin. The certificates shall contain lot, batch and roll numbers; sampling and test methods; and results of quality control tests. The quality control testing and frequencies shall meet the following:

Property	ASTM Test Method	Frequency
Density	D792 or D1505	Every 50,000 ft^2
Carbon Black Content	D1603	Every 50,000 ft^2
Transmissivity	D4716	One Test Per Landfill Phase

Samples that do not meet the specified properties shall be rejected, and additional testing of rolls may be required by the manufacturer to identify any additional non-complying rolls.

Geonet rolls shall be tagged with the manufacturer, product name, lot or batch number, and roll number.

The ENGINEER, at his discretion, may randomly sample the geonet for conformance testing.

4.2.5 Geocomposite

The geocomposite for the sideslopes shall consist of a geotextile/geonet/geotextile laminate. The geotextile and geonet shall meet the minimum criteria specified above.

4.2.6 Temporary Cover Liner and Ballast System

The temporary cover liner shall be a non-spec UV stabilized minimum 20-mil LLDPE material. The ballast system for the temporary cover liner shall be UV stabilized woven polyester or polypropylene sandbags with a volume capacity and strength to contain 40 pounds of sand. The sandbags shall be continuously tied with ½-inch diameter UV stabilized polyester or polypropylene rope.

4.2.7 Grout for Sump

The grout for the sump shall consist of a mixture of Portland cement, fine aggregate, and water so proportioned and mixed as to provide a readily pumpable slurry. Admixtures and/ or pozzolan may be used if the grout becomes stiff. The hardened grout shall exhibit a compressive strength of 3000 psi at 28 days when specimens are made and tested according to the provisions of ASTM C-31 and C-39.

4.3 Construction

4.3.1 Leachate Pipe

The perforated polyethylene header pipe shall be as shown on the Contract Drawings connected to the sump.

The pipe section connections shall be made by the heat fusion welding technique.

The pipe will be laid such that the perforations are on the underside of the pipe.

The leachate header pipe shall be covered with NC DOT No. 57 Stone and NC DOT No. 78M Stone, and wrapped with geotextile as shown in the Contract Drawings.

4.3.2 Aggregate

The aggregate for the leachate collection and header piping shall be placed in a phased manner during operational cover placement. A minimum 2 foot thick operational cover is required prior to equipment access for aggregate placement. Alternative methods for aggregate placement may be submitted for review and approval by the ENGINEER.

The pipes, gravel and sand over the leachate header pipe shall be completely covered with a temporary geotextile to avoid clogging during placement of the operational cover. The sections of the geotextile shall be installed with a minimum overlap of 6 inches and continuously heat tacked. The overlapped edges of the geotextile shall be temporarily secured during periods of windy weather.

4.3.3 Nonwoven Geotextile

Nonwoven geotextiles in the landfill floor shall be continuously heat tacked. Geotextiles shall be overlapped a minimum of 6 inches prior to heat tacking.

Any holes, burnouts, or tears in the geotextile shall be repaired with a patch made from the same geotextile. The patch shall be heat-tacked in place with a minimum of 12 inches overlap in all directions.

4.3.4 Geonet

Adjacent edges of geonet on the landfill bottom shall be overlapped at least 4 inches. These overlaps shall be secured by tying at 5 foot intervals.

Geonet roll ends (butt seams) shall be overlapped one foot, and ties shall be applied at 12 inch intervals.

Any holes or tears in the geonet shall be repaired by placing a geonet patch extending 2 ft. beyond the edges of the hole or tear. The patch shall be secured by tying fasteners through the geonet. The patch shall secure every 6 inches with approved tying devices. If the hole or tear width across the roll is more than 50 percent of the width of the roll, the damaged area shall be cut out and the two portions of the geonet shall be joined as noted above.

4.3.5 Geocomposite

Adjacent edges of geonet on the landfill slope shall be secured by tying at 5 foot intervals. Geotextile ends shall be overlapped a minimum of 4 inches and continuously heat-tacked.

Any holes, burnouts, or tears in the geotextile shall be repaired with a patch made from the same geotextile. The patch shall be heat-tacked in place with a minimum of 12 inches overlap in all directions.

No horizontal seams will be permitted on the side slopes.

4.3.6 Temporary Cover Liner and Ballast System

Installation shall be performed in accordance with the geomembrane installation procedures in Section 3.0. The leading edge of the temporary cover liner shall be continuously ballasted with sandbags. The sand bags shall be tied and spaced at 10-foot on center and anchored at the top of slope as shown on the Contract Drawings.

4.3.7 Grout for Sump

The grout shall be placed in 12-inch maximum lifts with vibration equipment to eliminate voids and inclusions. The grout must cure a minimum of 24 hours prior to applying the geomembrane.

4.4 Measurement and Payment

4.4.1 Payment for polyethylene piping shall be based on the unit price bid per linear foot installed. Pay quantity will not include wastage or extension of material beyond required limits.

4.4.2 Payment for prefabricated HDPE sump and clean out pipe saddles shall be based on a unit price basis. Pay quantity will not include a wastage or extension or material beyond the required limits. Payment will include furnishing and installing the sump, cleanout saddles, and all necessary concrete or grout.

4.4.3 Payment for the filter stone (NC DOT No. 57) shall be based on the unit price bid per ton. This price will include furnishing, hauling and placing the stone and wrapping with geotextile.

4.4.4 Payment for geotextile, geonet, geocomposite, and temporary cover liner with ballast system shall be based on the unit price bid per square foot installed. Pay quantity will include installed material and material extending to the lower outside corner of the exterior anchor trench. Quantities shall be field measured by the ENGINEER. Pay quantity will not include overlaps, wastage, or extension of material beyond required limits. Pay quantity will include additional quantities necessary to provide a complete item of work.

4.4.5 Installation quantities shall not include quantities placed beyond the lines and grades shown on the Drawings.

4.4.6 Requested payment quantities will be submitted by the CONTRACTOR with final approval by the ENGINEER.

4.4.7 If a dispute exists relative to payment quantities, the CONTRACTOR at his expense will uncover any buried or covered material for re-evaluation.

4.4.8 Any damage to the underlying geomembrane shall be repaired and retested per Section 3.0 of these specifications at no additional costs to the OWNER.

5.0 OPERATIONAL COVER AND INTERIOR BERMS

5.1 Scope of Work

The EARTHWORK CONTRACTOR shall furnish all labor, materials, supervision and equipment necessary to complete the construction of the 12-inch thick operational cover and interior berms for the Belews Creek FGD Gypsum Landfill, Phase 1 including but not limited to excavation, hauling, scraping, stockpiling, placement, maintenance, and removing accumulated rain water during construction. The operational cover and interior berms shall be constructed to the lines and grades shown on the Contract Drawings.

5.2 Materials

5.2.1 The operational cover shall consist of on-site silty sands and sandy silts from the designated excavated area and/or stockpiles with a minimum d_{10} = 0.004 millimeters. Sieve analyses of the proposed operational cover must be submitted to the ENGINEER for approval at least 4 weeks prior to scheduled installation.

5.2.2 Bulk samples of the operational cover will be taken at intervals of 1 sample per 3,000 cubic yards of material to be placed. Gradation analysis (ASTM D-422 or C-136) will be performed on the sample.

5.2.3 Structural fill as specified in Section 2.2 shall be used for the construction of the interior berms.

5.2.4 The stormwater drainage channel and leachate header pipe shall be covered with a temporary cover liner. The temporary cover liner shall be a non-spec, minimum 20 mil, UV-stabilized polyethylene geomembrane. The proposed material specifications must be submitted to the ENGINEER for review and approval.

5.3 Construction

5.3.1 The operational cover and interior berms shall be constructed to the lines and grades shown on the Contract Drawings.

5.3.2 An approved method of placing the operational cover on the geotextile and geomembrane to minimize wrinkles in the underlying liner is to enclose a small area using a trackhoe. With the trackhoe operating from a staging area, the operational cover shall be placed in piles to enclose a small area away from the working face. The operational cover may then be pushed outward using a small dozer into the enclosed area. Other methods that are demonstrated to be acceptable by the ENGINEER may be used.

5.3.3 Operational cover shall be placed in lifts not less than 12 inches. Compaction of the operational cover shall be performed by a minimum of 3 passes of tracked equipment. The final lift shall be rolled with a smooth drum roller (non-vibratory) to seal the surface, and enhance stormwater runoff.

5.3.4 Construction of the operational cover and interior berms is complete when the ENGINEER determines the lines and elevations shown on the drawings have been achieved in the field by reviewing an as-built survey drawing submitted by the EARTHWORK CONTRACTOR.

5.3.5 The temporary cover liner shall be placed over the stormwater channels and leachate header pipe following thickness verification. The EARTHWORK CONTRACTOR shall be responsible for installing a temporary ballast system on the temporary cover liner to reduce wind damage.

5.3.6 The EARTHWORK CONTRACTOR shall be responsible for maintaining the operational cover until the final completion date.

5.4 Measurement and Payment

5.4.1 Payment will be based on in-place quantities. Measurement of the operational cover and interior berms will be based on the as-built final survey. The thickness of the operational cover shall be determined by non-destructive survey methods by comparison of the top of compacted soil liner to the final grade topographic survey.

5.4.2 Quantities for the operational cover shall not include overbuild or replacement volumes of defective areas resulting from erosion. Overbuilt areas are defined as areas having operational cover thicknesses greater than 1.1 feet thick.

5.4.3 If a dispute exists relative to payment quantities, the EARTHWORK CONTRACTOR at his expense will uncover any buried or covered material for re-evaluation.

5.4.4 The entire operational cover must meet the specified thickness requirements prior to payment.

5.4.5 Payment for the operational cover will be made at the contract unit price per cubic yard. This price shall included furnishing, hauling, placing and maintaining the operational cover.

5.4.6 Payment for the interior berms will be made at the contract price per cubic yard. This price shall include furnishing, hauling, placing, compacting and maintaining the interior berms.

5.4.7 Payment for the temporary cover liner will be made at the contract price per square foot of installed material. This price shall include furnishing, installing, anchoring and providing a temporary ballast system.

6.0 STORMWATER COLLECTION AND REMOVAL (SWCR) SYSTEM

6.1 Scope of Work

The CONTRACTOR shall furnish all labor, materials, supervision and equipment necessary to complete the construction of the SWCR system for the Belews Creek FGD Gypsum Landfill, Phase 1 including but not limited to supply and installation. The SWCR system shall be constructed to the lines and grades shown on the Contract Drawings.

6.2 Materials

Pipe for the SWCR system at each interior berm and perimeter berm shall be corrugated HDPE pipe with the diameters specified on the Contract Drawings. The HDPE pipe shall meet the requirements of Type S pipe under AASHTO M252, M294 and MP7. The HDPE pipe shall be equivalent to ADS N-12 WT.

HDPE pipe shall be manufactured with first quality virgin polyethylene as referenced in Section 4.0.

6.3 Construction

The SWCR pipe shall be placed to the lines and grades shown on the Contract Drawings.

The HDPE geomembrane shall be extrusion welded to the SWCR pipe (top of pipe corrugation) exiting the landfill at the perimeter berms to provide a water-tight seal. The HDPE geomembrane boot to SWCR pipe shall be spark tested to verify a water-tight seal.

6.4 Measurement and Payment

6.4.1 Payment for the SWCR pipe shall be based on a unit price bid per linear foot installed. Pay quantity will not include wastage or extension of materials beyond required limits shown on the Contract Drawings.

6.4.2 Requested quantities shall be submitted by the CONTRACTOR with final approval by the ENGINEER.

7.0 ROADWAYS

7.1 Scope of Work

The EARTHWORK CONTRACTOR shall furnish all labor, material, supervision, and equipment necessary to complete the Roadways along the landfill perimter, including, stripping, grading, ditching, compacting soil, excavation, rolling and sealing, aggregate base course placement and all necessary and incidental items, all in accordance with the Contract Drawings and these Specifications.

7.2 Materials

7.2.1 Aggregate Base Course

All material shall conform with Section 1010 of the NCDOT "Standard Specifications for Roads and Structures", July 1995, for Type A Aggregate Base Course.

7.3 Construction

7.3.1 Aggregate Base Course for Access Road

The subgrade shall be prepared in accordance with the requirements shown on the Contract Drawings and with Section 500 of the NCDOT "Standard Specifications for Roads and Structures", July 1995.

Base course material may be delivered to the site with the fines already mixed in. Additional fines shall be added as necessary to obtain the desired density and stability. Material shall be placed on the prepared subgrade by use of spreader boxes or other approved methods to the required thickness in one uniform operation. Care shall be taken to prevent segregation of the fines from the coarse aggregate during the spreading and shaping of the material.

Self-propelled vibratory steel rollers weighing not less than 10 tons each shall be used to compact the base course material where practical. In places inaccessible to a 10-ton roller, the required compaction shall be secured with approved mechanical tampers, or small rollers. Rolling shall start at the edge and proceed to the center. Compaction shall be continued until all aggregates are firmly keyed or set and a density of 100 percent of ASTM D1557 is achieved. At completion of initial compaction, if any voids remain, fine aggregate shall be placed on the base in an amount only sufficient to fill the voids. The base course shall be broomed, wetted

and rolled until all coarse aggregate is firmly set, bonded, and base thoroughly compacted for the full width and depth.

The finished surface of the base course shall be level or gradually sloping as required to obtain proper alignment of surface course with edges and gradual slopes at elevation changes.

Base course shall have a minimum thickness as shown on the Contract Drawings. Depth measurements will be made at intervals designated by the ENGINEER. Measurement will be made by test holes dug through the base course not by probe-rod inspection. Where the base course thickness is more than 1/2 inch less than that shown or specified, it shall be corrected as directed by the ENGINEER.

7.4 Measurement and Payment

7.4.1 Requested quantities will be submitted by the CONTRACTOR with final approval by the Engineer.

7.4.2 Installation quantities will not include quantities placed beyond the lines and grades shown on the Contract Drawings.

7.4.3 Payment for the ABC stone shall be based on the unit price bid per ton installed. This price shall include furnishing, placement and compaction.

7.4.4 Payment for the culverts shall be based upon the unit price bid per linear foot installed. This price shall include layout, excavation, subgrade preparation, placement, and backfill placement and compaction.

7.4.5 If a dispute exists relative to payment quantities, the EARTHWORK CONTRACTOR, at his expense, will uncover any buried or covered material for re-evaluation.

8.0 FINAL CAP (Not In Contract)

8.1 Scope of Work

The EARTHWORK CONTRACTOR shall furnish all labor, materials, supervision, and equipment to complete site preparation, construction of the compacted soil cover, final cover and stormwater control features.

The GEOSYNTHETIC CONTRACTOR shall furnish all labor, materials, supervision, and equipment to complete installation, and equipment to complete installation of the geosynthetic clay liner and geomembrane cap including, but not limited to, layout, seaming, patching, and testing in accordance with the Contract Drawings and these Specifications.

8.2 Materials

8.2.1 Compacted Soil Layer

Cap soil liner material requirements will be in accordance with Section 2.3.2 of these specifications with the exception of the required thickness of 18 inches shall meet Section 2.3.3 of these specifications with the exception that the bulk samples of soil liner material will be obtained for every 5,000 cubic yards of material to be placed. The material will be tested for natural moisture content, grain size, Atterberg limits, and standard proctor.

8.2.2 The geomembrane liner shall be a minimum 40 mil textured geomembrane. The final selection will be based on the final closure plan and will account for regulation changes and/or updated technology.

8.2.3 The geocomposite shall be The geocomposite for the sideslopes shall consist of a geotextile/geonet/geotextile laminate. The geotextile and geonet shall meet the minimum criteria specified above.

8.2.4 The piping for the perimeter drain shall be as specified on the Contract Drawings. Samples of the piping material shall be submitted to the ENGINEER for approval.

8.2.5 Topsoil for the final cap shall be classified according to the Unified Soil Classification System (USCS) as ML, OL, SM or SC. Topsoil shall be loamy in texture and capable of supporting native grasses.

8.3 Construction

8.3.1 The waste surface shall be graded in accordance with the Operations Plan.

8.3.2 An 18-inch thick cap soil liner shall be installed prior to the geomembrane installation.

Compaction of lifts shall be as follows:

Compaction of lifts shall be performed with an appropriately heavy, properly ballasted, penetrating-foot compactor subject to approval of the Engineer.

The daily work area shall extend a distance so as to maintain moist soil conditions (facilitate bonding) and continuous operations. Desiccation and crusting of the lift surface shall be avoided as much as possible.

If desiccation and crusting of the lift surface occurs before placement of next lift, this area shall be scarified with a disc-plow, to a sufficient depth to mix with moist materials, or sprinkled with water and scarified.

During compaction, the placement soil moisture content and dry density shall be maintained within the limits specified below.

To determine the moisture content and dry density requirements of the compacted soil are being satisfied, field density and moisture tests shall be made at a frequency of one test per 100 foot by 100 foot grid for every other lift at locations designated by the ENGINEER.

The compacted soil layer shall be compacted to a minimum dry density of 92 percent (or as determined by the results of barrier soil test pad) of the maximum dry density determined from the standard Proctor Test (ASTM D-698). Densities less than the specified compaction of the maximum dry density as determined from the standard Proctor test shall be recompacted and/or removed and reworked to meet density objectives.

No compacted soil layer shall be placed or compacted during sustained period of temperatures below 30°F. Compacted soil layer soil may be placed and compacted during periods of early morning and early evening freezing temperatures with warming trends during the day. The placement of barrier soil during times of marginal weather shall be at the discretion of the ENGINEER.

At the end of each construction day's activities, completed lifts or sections of compacted soil layer shall be sealed by rolling with a rubber tired or smooth drum rollers and sprinkled with water as needed.

The thickness of the compacted soil layer shall be a minimum of 18 inches thick above the waste in the cell.

The as-built thickness of the compacted soil layer shall be determined by survey methods (non-destructive). An individual lift may be sampled upon completion (but prior to subsequent lift placement) with an approved sampler or other investigative tool, but the resulting penetration shall be properly backfilled with a soil-bentonite mixture.

8.3.3 The geomembrane cap shall be installed following completion of the cap foundation layer.

Areas to receive geomembrane installation should be relatively smooth and even, free of ruts, voids, etc. This shall be accomplished by final dressing of the compacted soil liner with rubber-tired or smooth drum rollers. No rubber-tired equipment is permitted on soil surfaces after final dressing unless authorized by the Engineer. The entire cell is to be capped with the geomembrane.

Installation of the geomembrane shall be as follows:

Unroll only those sections which are to be anchored or seamed together in one day. Panels should be positioned with the overlap recommended by the manufacturer, but not less than 2 in.

After panels are initially in place, remove as many wrinkles as possible. Unroll several panels and allow the liner to "relax" before beginning field seaming. The purpose of this is to make the edges which are to be bonded as smooth and free of wrinkles as possible.

Once panels are in place and smooth, commence field seaming operations. The geomembrane placement, seaming, and testing shall be in accordance with Section 3.3 of these specifications.

8.3.4 The geocomposite drainage layer and toe drains should be constructed concurrently with the geomembrane placement at specified locations. The geocomposite shall conform to Section 4.2 of these Specifications.

8.3.5 The gas venting system should be constructed at specified locations concurrently with the geomembrane placement.

8.3.7 The 24 inches soil cover shall be placed above the geomembrane as specified below.

The initial lift placed directly on top of the cap geomembrane shall be compacted to the specified density, within the specified moisture content, with a standard track width CAT D-6 dozer or equivalent. The use of penetrating foot compactors immediately above the cap geomembrane, or other sensitive element is not permissible until the overlying fill has achieved a thickness equal to or greater than 1-1/2 times the length of the penetrating foot.

Topsoil shall be placed above the soil cover for the cap, to a minimum thickness of 6 inches. Topsoil should be left loose, not compacted, to allow seeding and fertilizer to produce adequate vegetative cover (shallow rooted) to retard cap erosion. The types and application amounts of fertilizer and native grass seed, as well as the watering frequency, will be as indicated in the Contract Drawings.

The down slope ditches should be constructed, concurrently with final cover placement, at specified locations.

9.0 STORMWATER BASIN

9.1 Scope of Work

The CONTRACTOR shall furnish all labor, materials, supervision, and equipment necessary to complete the stormwater basin. The scope of work shall include but not be limited to earthwork for basin, geosynthetic clay liner (GCL), geomembrane, geotextile, concrete, fabric form, invert piping valves, riser structure, discharge pipe, riprap forebay, and incidental items required to complete the work in accordance with the Contract Drawings and these Specifications.

9.2 Materials

9.2.1 Earthwork shall be in accordance with Section 2.2.

9.2.2 The geosynthetic clay liner (GCL) on the basin bottom shall be equivalent to Claymax 600 CL. The GCL for the basin side slopes shall be stitched reinforced GCL equivalent to Bentomat DN.

The bentonite shall be high quality sodium bentonite comprised of 90 percent montmorillonite. The bentonite shall be continuously adhered to the underlying and overlying geotextile with a water soluble adhesive.

GCL panels shall be a minimum of 12 feet wide. Proposed GCL panel layout shall be submitted to the engineer at least 2 weeks prior to installation.

The GCL shall be shipped rolled with a protective wrap around each roll, labeled with a roll number. Manufacturer's quality control documentation shall be included with each roll.

9.2.3 The stormwater basin geomembrane shall be 40 mil textured HDPE in accordance with Section 3.0.

9.2.4 The geotextile above the geomembrane on the basin floor shall be 12 oz/sy geotextile in accordance with Section 4.0.

9.2.5 The geomembrane on the basin floor shall have an overlying 6-inch thick concrete slab. The slab shall be fibermesh reinforced with construction joints as shown on the Contract Drawings. The concrete shall be a minimum 3,000 psi 28-day compressive strength. The FGD project standard concrete plant mix may be submitted to the ENGINEER for review and approval for an alternative concrete mix design.

9.2.6 The geomembrane on the side slopes shall be protected by a 3-inch thick unimat fabric form equivalent to Armorform 3-inch USM as manufactured by Nicolon Corporation. The fabric form shall consist of a double layer of woven polyester or polypropylene fabric joined together by spacer cords (3 inch on center) to serve as a form for casting a pressure grouted concrete lining.

Grout stops shall be installed parallel to the mill width on approximately 18 feet intervals or as recommended by the manufacturer to regulate the flow of grout.

An expansion joint based on the manufacturer's recommendations shall be incorporated in to the fabric form at approximately 80 feet on center to relieve stresses associated with expansion and contraction.

Grout shall consist of a mixture of Portland cement, fine aggregate, and water so proportioned and mixed as to provide a readily pumpable slurry. Admixtures and/ or pozzolan may be used if the grout becomes stiff. The hardened grout shall exhibit a compressive strength of 2500 psi at 28 days when specimens are made and tested according to the provisions of ASTM C-31 and C-39. The grout mixed in the field shall be tested in a laboratory for compressive strength using a unimat sleeve or sock.

9.2.7 The invert piping valves shall be 18-inch to 24-inch (as specified on the Contract Drawings) gate valve. The gate valves shall be butterfly valves with mechanical joint ends conforming to all requirements of AWWA C-504 Class 150B. Each butterfly valve shall be furnished with a manual operator equipped with a 2-inch square operating nut. The operator shall open the valve when the operating nut is turned to the right or clockwise.

The manufacturer shall submit certification that the valves comply with all applicable provisions of AWWA C-504. Butterfly valves shall be manufactured by American Darling, Clow, Kennedy, Mueller, Pratt, or approved equal.

9.2.8 The HDPE riser structure shall be SDR 21 HDPE pipe and meet the requirements of Section 4.0. The riser structure shall meet the dimensions as specified on the Contract Drawings. Shop drawings for the riser shall be submitted to the ENGINEER for review and approval.

9.2.9 The HDPE discharge piping to the pump station shall meet the requirements of Section 4.0.

9.2.10 The riprap for the forebay shall be NCDOT Class 1 riprap with a d_{50} equal to 6 inches.

9.3 Construction

9.3.1 Earthwork for the stormwater basin shall be performed in accordance to Section 2.3 and to the lines and grades shown on the Contract Drawings.

9.3.2 The GCL shall be placed on a graded surface smooth and even, free of ruts, and voids.

An anchor trench shall be required to secure the GCL. No loose soil will be allowed to underlie the GCL in the anchor trench. The schedule for excavation of the anchor trench shall be approved by the ENGINEER so that desiccation of trench soils or surface water accumulation does not occur prior to backfilling. The anchor trench shall be temporarily sandbagged until all layers of geosynthetics are installed. The anchor trench shall be backfilled as soon as possible. A temporary cover over the open trenches may be required to keep the GCL dry prior to backfilling.

Daily installation of GCL shall be limited to the sections of GCL that can be overlain by geomembrane in one day. Panels shall be positioned with the overlap recommended by the manufacturer, but not less than 6 inches for panel sides and 24 inches for panel ends.

Bentomat GCL panels shall be placed so that the white nonwoven geotextile side is placed down onto the soil. Granular bentonite shall be placed along the seams for the entire seam width at an application rate of 0.25 pounds per linear foot.

Claymax GCL panels shall be placed so that the stenciled "Claymax" faces up. Granular bentonite along the seams will be required only in seam areas that do not exhibit continuous seam contact (i.e., minor seam wrinkles).

After panels are initially in place, remove wrinkles as directed by the ENGINEER. Unroll several panels and allow the GCL to relax before beginning adjacent panels. The panels shall be placed on the side slopes first and extend a minimum of 3 feet beyond the toe of slope. No horizontal seams of GCL will be permitted on the side slopes. The panels in the basin floor shall be placed from the upgradient end of the basin and progress downgradient in order to keep stormwater runoff from accumulating under the GCL.

Once the daily panels are placed and smooth, commence placement of the geomembrane in accordance with Section 3.0. If exposed GCL cannot be covered by geomembrane before the end of the working day, the panels or sections of panels shall be temporarily covered with plastic and ballasted until geomembrane placement can commence.

If GCL panels become saturated from rainfall and exhibit bentonite swelling prior to geomembrane placement, the saturated sections of GCL shall be removed and replaced.

9.3.3 The geomembrane shall be installed as specified in Section 3.0.

9.3.4 The geotextile shall be installed as specified in Section 4.0.

9.3.5 The concrete slab shall be poured prior to installation of the unimat fabric form. The slab shall be formed so that the edge of the slab is a minimum of 3 feet from the toe of slope.

9.3.6 The unimat fabric form shall be installed by a crew experienced in the installation of this type of product. The GCL panel ends of geotextile shall be sewn across its entire length. No butt joints between panels will be permitted (other than expansion joints). Prior to grout injection, unimat fabric form panels shall be positioned at their design location. Panel ends shall extend to the edge of the concrete slab (3 feet from the toe of slope). Following panel placement, grout will be injected between the top and bottom layers of fabric, inflating the revetment to the recommended thickness. The unimat fabric form shall be anchored beyond the geomembrane anchor trench as shown on the Contract Drawings.

9.3.7 The HDPE piping shall be installed as specified in Section 4.0.

9.3.8 Riprap placement for the forebay shall be from the concrete ramp into the stormwater basin. Low ground pressure equipment such as rubber tire loader or bobcats shall be used to place the riprap into position. Heavy dump trucks or scrapers are not permitted on top of the stormwater basin floor.

9.4 Measurement and Payment

9.4.1 Measurement and payment for the stormwater basin earthwork shall be as specified in Section 2.0.

9.4.2 Measurement and payment for the GCL shall be made at the contract unit price per square foot of in-place GCL. This price shall exclude scrap and overlay. Pay limits shall be measured to the lower inside corner of the anchor trench.

9.4.3 Measurement and payment of the geomembrane shall be in accordance with Section 3.0.

9.4.4 Measurement and payment of the geotextile shall be in accordance with Section 4.0.

9.4.5 Measurement and payment of the concrete slab shall be based upon the unit price per square foot of in-place slab. This price shall include forming, pouring, curing, and surfacing.

9.4.6 Measurement and payment of the unimat fabric form shall be based upon the unit price per square foot installed. Pay quantities shall include anchor trench quantities.

9.4.7 Measurement and payment for the HDPE piping shall be in accordance with Section 4.0.

9.4.8 Measurement and payment for the valves shall be based upon the unit price per valve. This price includes furnishing and installing the valves.

9.4.9 Measurement and payment for the HDPE riser structure shall be based on a lump sum price. This price shall include the material, installation, and concrete anchor.

9.4.10 Measurement and payment for the riprap forebay shall be based upon the unit price basis per ton of riprap installed. This price shall include furnishing and placement of the riprap.

10.0 LIFT STATION, VALVE VAULT AND HDPE FORCEMAIN

10.1 Scope of Work

The CONTRACTOR shall provide supervision and equipment necessary to install the lift station, valve vault, and 6-inch diameter HDPE forcemain including grading, excavating, dewatering, granular material placement, backfilling, compacting soil, lift station and valve vault placement, pump installation, control panel installation, installation of electrical wiring and any other necessary and incidental items required to provide an operating lift station, all in accordance with the Contract Drawings.

10.2 Materials

10.2.1 The lift station shall consist of a single containment, pre-cast, reinforced concrete lift station with aluminum access cover. The CONTRACTOR shall submit shop drawings for pump station, pump mounting details, and the aluminum access cover to the ENGINEER for review and approval prior to construction.

10.2.2 The valve vault shall be constructed with reinforced concrete and shall include an Aluminum access hatch as shown on the Contract Drawings. The CONTRACTOR shall submit shop drawings for the valve vault to the ENGINEER for review and approval prior to construction.

10.2.3 All HDPE pipe and HDPE fittings shall be manufactured from first quality virgin polyethylene with the following nominal properties:

Property	ASTM Test Method	Required Value
SDR		17
Density	D1505	0.95 gm/cm^3
Melt Flow	D1238-E	1.5 gms/10 min
Environmental Stress	D1693-C	5000 hrs.
Tensile Strength @ Yield	D638	3500 psi
Modulus of Elasticity	D790	110,000 psi
Hydrostatic Design Basis	D2837	1500 psi @23°C

10.2.3.1 Pipe and Fittings

The pipe shall be produced to the nominal dimensions and tolerances specified by ASTM F-714. The pipe shall contain no recycled compound except that generated in the manufacturer's

plant from resin of the same specification from the same raw material. The pipe shall be free of visible cracks, holes, voids, or other deleterious defects and shall be homogeneous in color, density, melt index, and other physical properties throughout.

Standard HDPE fittings shall be standard commercial products manufactured by injection molding or by extrusion and machining, or shall be fabricated from HDPE pipe conforming to this specification. All fittings shall be fully pressure rated by the manufacturer to provide a working pressure equal to the pipe for 50 years service at 73.4°F with a minimum factor of safety of 2. The fittings shall be free of visible cracks, holes, voids, or other deleterious defects and shall be homogeneous in color, density, melt index, and other physical properties throughout.

Ductile iron pipe shall meet the requirements of AWWA Standard C-151 and shall have a cement-mortar lining of standard thickness in accordance with AWWA C-104. Ductile iron pipe shall be Pressure Class 350. Mechanical joint bolts and gaskets for mechanical joint pipe and fittings shall be furnished by the pipe manufacturer and shall conform to ANSI A21.11 (AWWA C-111). Flange joints shall be in accordance with ANSI A21.10. Flanges, bolts and nuts, and gaskets shall conform to the dimensional requirements of ANSI B16.2 for Class 250. Bolts shall have standard hexagonal heads and shall be provided with standard hexagonal cold pressed nuts.

Flange connections on pipe and fittings shall be made using stub end and backing ring arrangements to provide 150 psi flanged connections. Backing rings and bolts shall be constructed of 300 series stainless steel. Each flanged connection shall utilize a full-face EPDM gasket approximately 1/8-inch thick.

10.2.4 Valves

The CONTRACTOR shall provide and install two 6-inch diameter ductile iron ball check valves and two 6-inch diameter ductile iron plug valves within the valve vault as shown on the contract drawings. Shop drawings for the proposed valves shall be submitted to the ENGINEER for review and approval. 300 series stainless steel backing rings and bolts shall be installed on the HDPE flange adaptors at each valve connection. Each flanged connection shall have a full face EPDM gasket.

10.2.5 Pumps and Controls

The CONTRACTOR shall provide the pumps and mounting hardware, pump discharge fittings, control panel, level sensors, and upper guide bar brackets. Pumps shall be Flygt 5100 Series Abrasion Resistant Submersible Pumps rated at 500 gpm at 30 feet TDH, or approved equivalent. The CONTRACTOR shall be responsible for installation of all pump components.

10.3 Construction

10.3.1 Concrete Lift Station and Valve Vault Installation

The concrete lift station and valve vault shall be installed to the elevations shown on the Contract Drawings. Gravel bedding placed beneath the manhole and valve vault shall be NCDOT #57 aggregate. The EARTHWORK CONTRACTOR shall take measures to dewater the excavations during installation of the manhole and valve vault. Structural fill as defined in Section 2.2 of these specifications shall be compacted in not greater than 8-inch thick lifts to a minimum of 95% of the standard Proctor maximum dry density around the manhole and valve vault.

Electrical cable and level sensors shall be routed through 4" diameter, Schedule 80 PVC from the manhole to the pump control panel. Non-metallic, gas-tight cable fittings shall be provided at the entrance of each cable into the control panel.

Underground electrical service will be supplied to a power pole near the lift station by the OWNER. The CONTRACTOR shall provide underground service from the pole to the control panel. Three (3) phase, 60 Hertz, 460 Volt service is required. A duplex ground fault interrupting receptacle providing 120 volt, 60 hertz, single phase service shall also be provided in a waterproof enclosure beneath the pump control panel. Receptacle circuit shall be protected by a 15 ampere thermal magnetic circuit breaker. At a minimum, electrical wiring shall be installed through Schedule 80 PVC conduit.

Underground telephone service will be provided to the control panel by the OWNER. The CONTRACTOR shall be responsible for extending phone service to the control panel and auto dialer as required.

The CONTRACTOR shall submit shop drawings for a control panel mounting rack to the ENGINEER for review and approval prior to construction. The CONTRACTOR shall provide and install the control panel mounting rack. The

CONTRACTOR shall install the control panel as recommended by the panel manufacturer.

10.3.2 Forcemain Piping Installation

10.3.2.1 Trenching and Backfilling

The CONTRACTOR shall install the 6-inch diameter, SDR 17, HDPE forcemain along the lines and grades shown in the Contract Drawings. The forcemain shall be installed to minimize abrupt changes in direction if possible. Slight changes in direction can be accommodated by field sweeping the excavated trench.

The forcemain pipe shall be installed in a trench type excavation a minimum of two feet below finished grade measured from the top of the pipe. The trench bottom should be relatively smooth and free of rock.

If rock or large stones are encountered they should be removed and the bottom of the trench re-built to the proper depth with compacted structural fill free of loose rock greater than 1/2-inch in diameter. If unstable soils are encountered during trenching, they should be undercut and replaced with compacted structural fill free of loose rock greater than 1/2-inch in diameter.

Soils excavated from the trench should be suitable if free from rock and if well broken up during excavation. If the excavated material is unsuitable, structural fill as defined in Section 2.2 of these specifications shall be used to backfill the forcemain pipe. All backfill material shall be compacted to 93 percent of Standard Proctor maximum dry density ASTM D-698. Care should be taken during backfilling to maintain the pipe alignment in the center of the trench.

Prior to completing backfilling of the forcemain pipe, detectable marking tape such as Terra Tape by Reef Industries, Inc. (or approved equivalent) shall be placed over all HDPE piping within 50 feet of the lift station manhole and valve vault. Standard marking tape shall be placed over the remainder of the forcemain line.

10.3.2.2 Joining

Sections of HDPE pipe should be joined into continuous lengths on the job site, above ground where applicable. The HDPE pipe shall be joined by the butt fusion welding method and shall be performed in strict accordance with the pipe manufacturer's recommendations.

Butt fusion joining shall be 100% efficient offering a joint weld strength equal to or greater than the tensile strength of the pipe. Unions, grooved-couplers, transition fittings and some mechanical couplings may be used without butt fusion if approved by the ENGINEER.

10.3.2.3 Pressure Testing

All discharge piping shall be pressure tested in the field in the presence of the ENGINEER. Any pressure testing conducted without being observed by the ENGINEER will not be accepted. Pressure testing shall be performed prior to backfilling the pipe. The EARTHWORK CONTRACTOR is responsible for providing all equipment necessary to perform pressure testing.

Sections of the system may be tested separately, but any defect that may develop in a section previously tested and accepted shall be promptly corrected and retested. Pressure tests shall be made between valves, to demonstrate ability of the valves to sustain pressure. Test plugs or bulkheads required at the ends of the test section shall be furnished, installed and removed at the end of the test by the CONTRACTOR. The CONTRACTOR is solely responsible for any damage that may result from the failure of test plugs or bulkhead fittings.

Before joints are covered, each completed section of the forcemain shall be plugged at both ends and slowly filled with water. As the main is being filled in preparation for the pressure test, all air shall be expelled from the pipe through blow-offs, air relief valves, or temporary taps. Temporary pressure gauges shall be installed at the downgradient ends of the section to be tested. The forcemain shall be subjected to a total gauge pressure of 50 psi, for a period of one hour. Pressure shall be applied to the piping by means of a pump. During the duration of the test, the lines shall be thoroughly

inspected for leakage at all joints and fittings. Any cracked or defective pipes, joints, fittings, or valves discovered in consequence of the pressure test shall be replaced by the CONTRACTOR at his own expense. All defects shall be remedied to the satisfaction of the ENGINEER. Tests will be continued until all visible leaks (if any) have been eliminated from the section of the system being tested. Alternate pressure testing methods may be submitted to the ENGINEER for review and approval.

10.3.2.4 Thrust Blocking

Gradual field sweeps (any alignments less than a 33 foot radius described in Section 10.3.2.1) should not require thrust blocks. Thrust Blocks shall be installed along the forcemain at all abrupt changes in direction (tees, elbows, etc.) both vertical and horizontal, changes in size (reducers, etc.), dead ends (plugs, caps, hydrants, etc.), valves, and slopes or as directed by the ENGINEER. Thrust blocking shall be constructed against the undisturbed trench wall with 3000 psi concrete. Reinforcement rods shall be installed as required. Rods shall be ASTM A-242 or A-36 steel, No. 4 rods. The ENGINEER shall approve all locations and reinforcement details for thrust blocks.

10.3.3 Lift Station Piping

10.3.3.1 The CONTRACTOR shall install all piping inside the lift station and valve vault as shown on the Contract Drawings. All penetrations through the manhole and valve vault shall be sealed as shown on the Contract Drawings.

10.3.3.2 All lift station HDPE piping shall be joined by the butt fusion welding method as described in Section 10.3.2.2 of these specifications. The 6-inch diameter discharge pipes shall be connected to the pump discharge fittings by means of a flanged connector utilizing a 300 series stainless steel backing ring arrangement with stainless steel bolts to provide a minimum 150 psi connection. Each flange connection shall have a full face EPDM gasket.

10.3.4 Pump and Control Panel Installation

10.3.4.1 The CONTRACTOR shall install the pumps, pump discharge fittings, upper guide bar brackets, guide bars and level control switches as shown on the Contract Drawings. Anchor bolt patterns for the pump mounting brackets and details for mounting the upper guide bar brackets will be provided by the ENGINEER prior to installation.

The CONTRACTOR shall install the control panel and make all necessary connections as recommended by the pump supplier and control panel manufacturer.

DO NOT ATTEMPT TO ENERGIZE THE CONTROL PANEL OR PUMPS. ENERGIZING THE SYSTEM MAY CAUSE DAMAGE TO THE PUMPS OR CONTROL PANEL. THE EARTHWORK CONTRACTOR IS RESPONSIBLE FOR REPLACING ALL PUMP EQUIPMENT DAMAGED DURING INSTALLATION.

Once all connections are made, piping installation is complete, pumps are installed, and system is ready for operation, a representative from the pump manufacturer will visit the site to perform a system check and startup. The CONTRACTOR shall notify the ENGINEER approximately two days before the system is complete so that the manufacturer's representative can be scheduled. The CONTRACTOR shall provide assistance to the pump manufacturer's representative as necessary during start-up of the system.

10.4 Measurement and Payment

10.4.1 Requested quantities for all work will be submitted by the CONTRACTOR with final approval by the ENGINEER. No payment shall be made until the components to be paid for are actually installed.

10.4.2 Payment for the lift station and valve vault shall be based on the contract lump sum price for the materials and the CONTRACTOR'S work to complete the installation including invert connections and hydrostatic testing.

10.4.3 Payment for HDPE piping shall be based on the unit price bid per linear foot installed, including trenching, pipe installation, pressure testing, trench bedding

and backfilling, and all connections within the valve vault and manhole. Pay quantities will not include wastage or extension of material beyond required limits.

10.4.4 Payment for the valves will be based on the contract unit price for each valve installed by the CONTRACTOR.

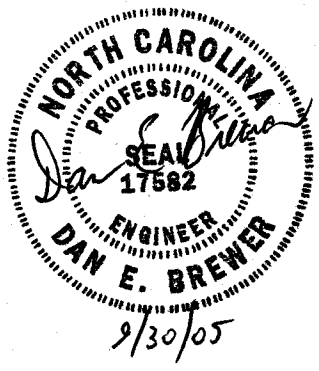
10.4.5 Payment for installation of the pumps, controls, wiring, and materials required for installation of the pump system including concrete, piping. control panel mounting rack, and necessary electrical work shall be based on the contract lump sum price for the work by the CONTRACTOR.

CONSTRUCTION QUALITY ASSURANCE PROGRAM

DUKE ENERGY CORP.
BELEWS CREEK STEAM STATION
FGD RESIDUE LANDFILL, PHASE 1
STOKES COUNTY, NC

Prepared By:

CHAS. H. SELLS, INC.
Consulting Engineers, Surveyors & Photogrammetrists
128 Overhill Drive, Suite 105, Mooresville, NC 28117
Phone: (704) 662-0100
Fax: (704) 662-0101



September 30, 2005

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CONSTRUCTION QUALITY ASSURANCE (CQA) PROGRAM

1.0 PROJECT TEAM AND RESPONSIBILITIES

Construction Quality Assurance (CQA) for the construction of the Belews Creek FGD Gypsum Landfill, Phase 1 will be provided by an engineering and testing firm (ENGINEER) independent of the CONTRACTOR specializing in the inspection and testing of soils and geosynthetics.

The project team providing the CQA services shall consist of the following:

A. Project Engineer - Responsible for defining quality assurance requirements compatible with the project objectives, reviewing and approving shop drawings and manufacturer/installer literature, outlining procedures for the analysis of test data and preparing quality assurance memorandums and quality control reports. The Project Engineer is responsible for design changes (as approved by the State Solid Waste Section), clarifications, and specification addenda. The Project Engineer also has the ultimate responsibility for approving or disapproving any element of the project. The responsibility to stop work is held by the Owner. Construction quality assurance documents will be prepared, signed and sealed by the Project Engineer. The Project Engineer will be a registered Professional Engineer in the State of North Carolina.

B. Construction Quality Assurance Officer - The Construction Quality Assurance Officer (CQAO) will be experienced in quality assurance testing and inspection. The Construction Quality Assurance Officer will report to the Project Engineer. The CQAO serves as the on-site representative of the Owner and is responsible for the field implementation of the approved quality assurance program as follows:

- Monitor the quality assurance activities of the field testing and assure conformance with test procedures and the technical specifications.
- Inform the Project Engineer of non-conformance to the approved CQA program.
- Logging in of geosynthetic samples and establishing laboratory conformance testing lots.
- Insure that sampling is conducted in a manner consistent with ASTM methods.
- Insure that sample handling procedures are in accordance with the appropriate guidelines for the testing to be conducted.
- Approve all field and laboratory test data before the data are reported or

entered into the data base for analysis.

- Assign and direct field technicians.
- Maintain an awareness of the entire field testing operation to detect conditions that may jeopardize the quality of testing.

C. Engineering Technicians - Responsible for field observations, testing and inspection. Technicians will be assigned to the project as deemed necessary by the Construction Quality Assurance Officer. Technicians will be responsible to the Construction Quality Assurance Officer.

2.0 STRUCTURAL FILL

2.1 Material

2.1.1 Structural Fill is defined as compacted fill required for area fill, berms, surface water control systems, roadways or other systems not intended to function as a leachate migration barrier.

2.2.2 Structural Fill may consist of on-site or off-site soils that are free of organic material, refuse or debris and can be constructed and compacted as required by the Technical Specifications.

2.2 Construction

2.2.1 Structural Fill will be compacted in lifts not exceeding 10 inches (loose measure). The ENGINEER may reduce maximum allowable lift thickness depending on soil type used, construction equipment and methods employed.

2.2.2 For construction of berms or roadways, in-place density of Structural Fill will be determined by the ENGINEER or his designated representative, at a frequency of at least one test per 300 linear feet of berm or roadway.

2.2.3 For area fills, density tests will be taken at a frequency no less than one test per 20,000 square feet of fill per lift.

2.2.4 All required field density and moisture content tests will be completed before the overlying lift of soil is placed. The surface preparation (e.g. wetting, drying, scarification, etc.) will be completed before the ENGINEER will allow placement of subsequent lifts.

2.2.5 Bulk (100-lb) samples of the Structural Fill will be retained for each 10,000 cubic yards of soil placed. The ENGINEER may reduce the number of bulk samples needed depending on the variability of the soil being used. The following testing will be performed:

- Method for Laboratory Determination of Water (Moisture) Content of Soil, Rock and Soil-Aggregate Mixtures, ASTM D-2216.
- Method for Particle-Size Analysis of Soils, ASTM D-422.
- Test Method for Liquid Limit, Plastic Limit, and Plasticity Index of Soils, ASTM D-4318.
- Moisture-Density Relationship (Standard Proctor), ASTM D-698 or

(Modified Proctor) ASTM D-1557.

3.0 GEOMEMBRANE

3.1 Material

The Geomembrane will be 60 mil High Density Polyethylene (HDPE) as specified in the Technical Specifications.

Seams for the geomembrane shall be double hot wedge fusion or extrusion welds as specified in the Technical Specifications.

3.2 Geosynthetic Manufacturer and Contractor

3.2.1 The geomembrane installer is the party responsible for installation of the HDPE geomembrane. "Installer" is used interchangeably with "Geosynthetics Contractor". The manufacturer is the party that supplies the geosynthetic products. In some cases the installer and manufacturer may be the same party. The GEOSYNTHETIC CONTRACTOR will submit the following as obtained from the Geosynthetic Manufacturer to the ENGINEER:

- Production Certification
- Testing Program of Compound Ingredients
- Material Certification
- Test Data for Material and Resin

All of the above submittals will be reviewed and retained by the ENGINEER.

3.2.2 The GEOSYNTHETIC CONTRACTOR will submit the following to the ENGINEER prior to installation:

- Qualifications and resumes of the GEOSYNTHETIC CONTRACTOR Superintendent and Foreman

- Four sets of proposed geosynthetic field panel layout drawings

3.3 Geomembrane Installation

3.3.1 A liner pre-construction meeting will be held prior to installation. The GEOSYNTHETIC CONTRACTOR, ENGINEER or his representative, and a representative of the OWNER will be in attendance. The technical

specifications and quality assurance and quality control procedures will be discussed and agreed upon by all parties.

3.3.2 The ENGINEER or his representative will sample rolls from each shipment of geomembrane delivered to the site. The minimum number of rolls to be sampled from each shipment will be determined by computing the cube root of the total of number of rolls delivered in the shipment and rounding this value upward to the nearest integer. For instance, if 40 rolls of geomembrane are delivered in a shipment, at least 4 rolls will be sampled.

3.3.3 The random samples must be representative of the materials supplied and exclude the outer wrap of geomembrane if evidence of scuffing or other damage is observed. Samples should be full roll width and at least 2 feet long.

3.3.4 The ENGINEER or his representative will measure the geomembrane thickness of each of the random samples at the project site prior to geomembrane installation. Material that does not fall within acceptable thickness criteria will be rejected.

3.3.5 At the discretion of the ENGINEER, samples will be selected at random and sent to the laboratory for additional conformance testing. The laboratory testing program will be directed by the ENGINEER and include but not be limited to the following properties:

- Thickness (ASTM D751)
- Density (ASTM D792)
- Carbon Black Content (ASTM D1603)
- Tensile Properties (ASTM D638)
- Tear Resistance (ASTM D1004)

3.3.6 During extrusion welding, the ENGINEER or his representative will mark all areas where grinding is considered to be excessive. The location and repair method for the excessive grinding will be recorded in the daily field reports. The method of repair will be determined in the field by the ENGINEER.

3.3.7 Overheating of the geomembrane will be monitored by the ENGINEER or his representative. Coupons will be cut from the end of the extrusion seams and the bottom side of the seam will be observed for visible warping or deformation. The location and repair method of overheated areas will be recorded in the daily field reports. The method of repair will be determined in the field by the

ENGINEER.

3.3.8 During seaming, the ENGINEER or his representative will observe the seams for the proper preparation, grinding technique and overheating.

3.3.9 The ENGINEER or his representative will observe the geomembrane during the coolest part of the day to check for slack. Any areas where "trampolining" occurs will be marked by the ENGINEER for repair by the GEOSYNTHETIC CONTRACTOR.

3.3.10 All repair locations shall be patched and tested in accordance with the Technical Specifications prior to acceptance by the ENGINEER. All patches shall extend a minimum of 6 inches beyond the repair location.

3.4 Test Seams

3.4.1 The GEOSYNTHETIC CONTRACTOR will perform a test seam at least twice daily. A test seam is also required if there is a change of machine or welding technician or a loss of power for the machine. The CQAO may request additional test seams if weather conditions significantly change. The test seam should be approximately 6 feet long. The ENGINEER will sample the test seam from the center 3 feet of the test sample.

3.4.2 The welder, date, time and equipment, as well as liner temperature, welding temperature, and seaming parameters will be recorded for each test seam.

3.4.3 A minimum of six specimens from each sample will be tested; three in peel and three in shear. Film Tear Bond (FTB) type failures will be the criterion for qualification of the test seam. Testing will be performed in the field by the GEOSYNTHETIC CONTRACTOR under full-time observation by the ENGINEER or his representative.

3.4.4 Untested portions of the test seam will be retained by the ENGINEER for the project records and future testing as required.

3.4.5 All test seams must pass the field testing before production seaming is performed by the GEOSYNTHETIC CONTRACTOR.

3.5 Field Destructive Samples

3.5.1 The GEOSYNTHETIC CONTRACTOR will obtain samples of field seams, suitable for testing, from the beginning and end of each field run exceeding 200 feet in length. In addition, samples will be removed at a frequency of one sample per 500 linear feet of weld for laboratory destructive testing. The date, time and equipment, seam number, and seaming parameters will be marked on each sample and recorded by the ENGINEER.

3.5.2 Samples retained will be tested in the field by the Geomembrane Installer. A minimum of three specimens from each sample will be tested in peel (ASTM D413). FTB type failures will be the criterion for qualification of the production seam. Testing should be performed from 15 minutes to one hour after completion of the seam by the GEOSYNTHETIC CONTRACTOR.

3.5.3 The ENGINEER or the OWNER may require additional random samples to be taken for testing in areas which visually appear defective and not in accordance with the Technical Specifications.

3.6 Non-Destructive Testing

3.6.1 The GEOSYNTHETIC CONTRACTOR is responsible for the completion of non-destructive testing of the entire length of all field seams, verifying that the seam is watertight. The testing will be vacuum, air pressure and spark testing as specified in the Technical Specifications.

3.6.2 Non-destructive testing will be observed by the ENGINEER or his representative on a full-time basis.

3.7 Laboratory Destructive Testing

3.7.1 Destructive seam samples will be laboratory tested by the ENGINEER. Testing frequency is at least one sample per 500 linear feet of field seam.

Test samples will be at least 18 inches long. A minimum of five peel specimens will be tested for each sample in accordance with ASTM D413. At least five specimens from each sample will be tested for bonded shear strength in accordance with ASTM D3083.

All laboratory specimens will be conditioned for a minimum of one hour prior to testing at a relative humidity of 65±5% and a temperature of 21±2°C (70±4°F).

Peel tests will be performed on both sides of a double-wedge fusion seam.

3.7.2 The load and type of failure will be recorded for each specimen. FTB is the qualified criterion.

The ENGINEER will describe the type of failure for each specimen and record the presence of any disbonding, delamination, foreign material in the bond area, etc.

As a general guideline, the bonded shear strength should equal or exceed the tensile strength (in pounds per unit width) of the parent material. If a shear specimen exhibits a failure within the grinding or preparation area adjacent to the seam that falls below 80 percent of the parent material strength, the ENGINEER may require additional seam repair.

As a general guideline, the peel adhesion should exceed 62 percent of the geomembrane yield strength of the parent material. The test report should note if a peel specimen fails in the grinding preparation area. Both sides of the double-wedge fusion seam must pass the testing requirements to constitute a passing test.

4.0 LEACHATE COLLECTION AND REMOVAL SYSTEM (LCRS)

4.1 Geocomposite

4.1.1 On the landfill floor, the leachate collection system will consist of a geonet overlain by a nonwoven needle-punched geotextile. On the landfill side slopes, the leachate collection system will comprise of a geocomposite (geotextile/geonet/geotextile). A one foot thick operational cover soil will overlie the LCRS on the landfill floor. Within the operational cover will be a series of leachate collection piping wrapped with gravel and geotextile.

4.1.2 Upon delivery to the site, the geotextile will be inspected by the ENGINEER. All applicable product information will be recorded. The following is a summary of the documentation that will be submitted by the GEOSYNTHETIC CONTRACTOR prior to material approval:

- mill certificates/delivery tickets
- grab strength (ASTM D4632)
- grab/tensile elongation (ASTM D4632/D4595)
- wide width tensile strength, M.D., C.D. (ASTM D4595)
- puncture strength (ASTM D4833)
- fabric weight (ASTM D3776)
- apparent opening size, AOS (ASTM D4751)

4.1.3 The geonet will be conformance tested by the GEOSYNTHETIC MANUFACTURER to determine its ability to transmit water in-place (ASTM D4716) under the conditions outlined below:

4.1.3.a Normal Compressive stress equal to the maximum vertical stress for the completed landfill.
4.1.3.b Hydraulic gradients ranging from 0.25 to 1.0.
4.1.3.c Seating Period of at least 48 hours
4.1.3.d Compacted operational cover soil placed over the Geonet/Geotextile and a 60 mil HDPE geomembrane beneath the Geonet.

4.1.4 The transmissivity test must confirm that the Geonet exhibits a hydraulic transmissivity of at least $1x10^{-4}$ m^2/sec.

4.1.5 Upon delivery to the site, the geonet will be inspected by the ENGINEER. All applicable product information will be recorded. The following is a summary of

the documentation that will be submitted by the Contractor prior to material approval:

- mill certificates/delivery tickets
- hydraulic transmissivity test data (ASTM D4716)
- density (ASTM D1505)
- wide width tensile strength and elongation (M.D. and C.D.) (ASTM D4595)

4.1.6 Any test not meeting the technical specifications criteria will result in possible retesting, at the direction of the ENGINEER, or rejection of the geotextile.

4.1.7 Roll tags or packing lists will be retained for all rolls of geonet, geotextile, and geocomposite delivered to the site.

4.1.8 Storage and handling conditions for the geonet, geotextile, and geocomposite will be observed by the ENGINEER or his representative. The material should be kept as dry and as free of dirt and debris as possible.

4.1.9 Geonet, geotextile, and geocomposite should be kept in protective wrapping until installation. Any rolls delivered to the site without UV and moisture protective wrapping may be rejected by the ENGINEER or his representative.

4.1.10 Placement of the geonet, geotextile, and geocomposite will be observed and recorded to assure conformation with the Technical Specifications and GEOSYNTHETIC MANUFACTURER'S recommendations. Any damaged material (including underlying geomembrane) will be marked for repair and/or replacement.

4.2 Leachate Collection Pipe and Sump

4.2.1 Manufacturer's technical data shall be provided to the ENGINEER by the supplier.

4.2.2 A minimum 1-foot long specimen of each solid and perforated pipe of each pipe size may be retained by the ENGINEER.

4.2.3 The butt fusion weld of the drainage pipe will be observed by the ENGINEER or his representative for any holes or burrs in the weld that may potentially damage the geomembrane.

4.2.4 The prefabricated sumps will be observed by the ENGINEER or his representative prior to placement to assure that they are smooth and free of burrs or sharp edges which may potentially damage the geomembrane.

4.2.5 The ENGINEER or his representative will verify the proper geomembrane protection has been provided before the sumps are placed.

4.2.6 Pressure testing of the HDPE discharge lines will be observed by the ENGINEER or his representative. The pressure and time at the beginning and end of the test will be recorded for each section of pipe tested.

4.2.7 The base of the trench of the leachate discharge piping (outside of landfill cell) will be observed by the ENGINEER or his representative prior to placement of the piping. Any sharp stones, soft material or other potentially damaging material should be removed. One bulk sample of the trench backfill material will be collected during placement or compaction. A standard Proctor maximum dry density (ASTM D-698) test will be performed on the pipe backfill material.

4.3 Operational Cover

Operational cover will consist of on-site sandy/silty soils. One sample per 3,000 cubic yards of the proposed operational cover soils shall be obtained by the ENGINEER for classification testing. Placement of the operational cover will be placed in minimum 12 inch thick lifts in a method to be approved by the ENGINEER. The ENGINEER will monitor operational cover placement on a full-time basis. Wrinkles in the geomembrane resulting from operational cover placement will be "walked out" prior to additional operational cover placement. Any excessive wrinkles will be observed by the ENGINEER for possible repair by the GEOSYNTHETIC CONTRACTOR. The final surface of the operational cover will be rolled with a non-vibratory smooth drum roller.

5.0 FINAL COVER SYSTEM

5.1 Compacted Soil Layer

5.1.1 The compacted soil layer will consist of on-site or off-site soil meeting the criteria for the compacted soil layer.

5.1.2 The compacted soil layer will be at least 18 inches thick and shall be compacted in nine inch lifts in accordance with the Technical Specifications.

5.1.3 The thickness of the compacted lift will be measured at random locations after spreading, leveling and compacting is completed prior to placement to the Geomembrane Cap.

5.1.4 Density testing will be performed on the compacted soil layer at a frequency of at least one test per 100 foot by 100 foot grid for every other lift. The location of each test will be randomly determined by the ENGINEER or his representative. Densities less than the specified compaction as outlined in the Technical Specifications shall be recompacted and/or removed and reworked to meet density objectives.

5.2 Geomembrane Cap

The geomembrane cap shall consist of a 40 mil textured LLDPE. The CQA procedures established for the landfill bottom geomembrane (Section 3.0) will be modified and submitted to the Division of Waste Management for approval prior to closure.

5.3 Final Cover Soil

The final cover and topsoil will be at least two feet thick and compacted in accordance with the Closure/Post-Closure Plan

The thickness of the cap will be randomly tested by the ENGINEER to verify adequate coverage.

5.4 Drainage System

The drainage system will consist of a geocomposite, underdrains and surface water ditches. The ENGINEER shall review shop drawings and manufacturer's literature for the geosynthetics. The materials must be approved by the ENGINEER prior to installation. The ENGINEER

shall monitor installation of the drainage system to verify conformance with the Closure Plan.

5.5 Gas Venting System

The gas venting system trenching and piping shall be installed as soon as possible following installation of the compacted soil layer. The gas vents should be installed prior to the geomembrane cap installation.

6.0 AS-BUILT CONDITIONS

The finished lines and grades of the surface of the excavated and/or constructed cell and operational cover will be presented to the ENGINEER by the CONTRACTOR. Final topographic surveys for the top of liner and operational cover should be performed in a minimum 100 foot by 100 foot grid.

An As-built drawing showing the location and identifying number of the geomembrane panels, seams and destructive samples will be maintained and updated by the ENGINEER. This does not replace the required Panel Layout As-Built to be submitted by the GEOSYNTHETIC CONTRACTOR.

As-built location and detail for construction of the Leachate Collection System will be submitted to the ENGINEER by the CONTRACTOR.

The contractor shall be responsible for submitting to the ENGINEER the following:

- Survey drawings verifying that finished lines and grades of the excavated and/or constructed cell, operational cover and final cover (by others following closure activities) have been obtained.
- All material certifications and warranty information for the installed material and equipment.
- As-Built utility line locations.

7.0 CQA SUBMITTAL

The Construction Quality Assurance document will contain the results of all tests performed on the following items for each completed phase:

1. General Structural Fill for cell floor, berms, roadways and surface water control structures
2. Geomembrane Liner
3. Leachate Collection and Removal System
4. Operational Cover and Intermediate Berms
5. Final Cover System (ie: compacted soil layer, geomembrane cap, and final soil cover after closure)*

Portions of the above items may be submitted to the Division as individual certification reports during construction. Following construction, the individual certification reports shall be compiled into one CQA document for an Application for Permit to Operate.

"As-Built" drawings will be included with the CQA document. A comprehensive narrative of the construction process and CQA activities including daily reports from the CQA Officer and documentation of progress meetings. Color photographs of key elements for landfill construction shall also be included in the CQA document.

The CQA document will be submitted within 60 days following completion of construction of each phase.

* Testing will be performed during closure of the phase. Test results will be provided as part of the closure certification.

REFERENCED LIST OF STANDARDS

ASTM C-31	Standard Practice for Making and Curing Concrete Test Specimens in the Field
ASTM C-33	Standard Specification for Concrete Aggregates
ASTM C-39	Standard Test Method for Compressive Strength of Cylindrical Concrete Specimens
ASTM C136	Standard Method for Sieve Analysis of Fine and Coarse Aggregates
ASTM D422	Standard Method for Particle - Size Analysis of Soils
ASTM D448	Standard Classification for Sizes of Aggregate for Road and Bridge Construction
ASTM D638	Standard Test Method for Tensile Properties of Plastics
ASTM D698	Standard Test Method Moisture-Density Relations of Soils and Soil-Aggregate Mixtures Using 4.4 lb. (2.49 kg) Rammer and 12-inch (305-mm) Drop
ASTM D751	Standard Methods of Testing Coated Fabrics
ASTM D792	Standard Test Methods for Specific Gravity and Density of Plastics by Displacement
ASTM D1004	Test Method for Initial Tear Resistance of Plastic Film and Sheeting
ASTM D1238	Test Method for Flow Rates of Thermoplastic by Extrusion Plastometer
ASTM D1556	Standard Test Method for Density of Soil in-Place by the Sand-Cone Method
ASTM D1557	Standard Test Methods for Moisture-Density Relations of Soils and Soil - Aggregate Mixtures Using 10-lb. (4.54-kg) Hammer and 18-in. (457-mm) Drop
ASTM D1603	Test Method for Carbon Black in Olefin Plastics

ASTM D2172	Standard Test Methods for Extraction of Bitumen from Bituminous Paving Materials
ASTM D2216	Standard Method for Laboratory Determination of Water (Moisture) Content of Soil, Rock, Soil - Aggregate Mixtures
ASTM D2922	Standard Test Methods for Density of Soil and Soil-Aggregate in Place by Nuclear Methods (Shallow Depth)
ASTM D2937	Standard Test Methods for Density of Soil in Place by the Drive-Cylinder Method
ASTM D3776	Test Method for Weight (mass) per Unit Area of Woven Fabric
ASTM D3786	Standard Test Method for Hydraulic Bursting Strength of Knitted Goods and Nonwoven Fabrics - Diaphragm Bursting Strength Test Method
ASTM D4318	Standard Test Method for Liquid Limit, Plastic Limit, and Plasticity Index of Soils
ASTM D4595	Standard Test Method for Tensile Properties of Geotextiles by the Wide-Width Strip Method
ASTM D4632	Standard Test Method for Breaking Load and Elongation of Geotextiles (Grab Method)
ASTM D4716	Standard Test Method for Constant Head Hydraulic Transmissivity (In-Plane Flow) of Geotextiles and Geotextile related Products
ASTM D4751	Standard Test Method for Determining Apparent Opening Size of a Geotextile
ASTM D4833	ndex Puncture Resistance of Geotextiles, Geomembranes, and Related Products
ASTM D5084	Standard Test Method for Measurement of Hydraulic Conductivity of Saturated Porous Materials Using a Flexible Wall Permeameter.
EPA/530/SW-89/069	The Fabrication of Polyethylene FML Field Seams

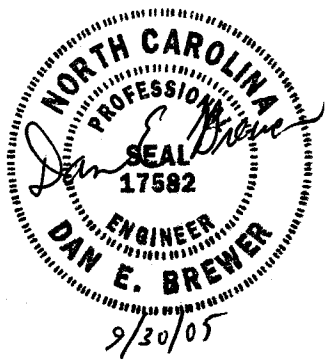
4

OPERATIONS PLAN

DUKE ENERGY CORP.
BELEWS CREEK STEAM STATION
FGD RESIDUE LANDFILL, PHASE 1
STOKES COUNTY, NC

Prepared By:

CHAS. H. SELLS, INC.
Consulting Engineers, Surveyors & Photogrammetrists
128 Overhill Drive, Suite 105, Mooresville, NC 28117
Phone: (704) 662-0100
Fax: (704) 662-0101



September 30, 2005

OPERATIONS PLAN
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OPERATIONS PLAN

1.0 LANDFILL OPERATIONS AND MAINTENANCE

The Belews Creek Steam Station Flue Gas Desulfurization (FGD) system will create a residue generally comprising of gypsum. Residue will be landfilled in the proposed FGD Gypsum Landfill located just south of the Belews Creek Steam Station Plant. The landfill footprint encompasses approximately 24 acres. Phase 1 will be developed in four cells with a capacity of three to five years (depending upon the amount of gypsum reused). The gypsum landfill will be owned and operated by Duke Energy Corp.

This Operations Plan has been prepared in accordance with Section 0.0505 of the North Carolina Solid Waste Management Rules.

The following drawings provided at the end of this section and in Appendix B shall be included as part of the Operations Plan. The drawings illustrate existing conditions, progression of construction and operation, leachate and stormwater controls, and site erosion control.

- Existing site conditions: Refer to Existing Conditions Plan, Sheet CP-1.
- Progression of construction and operation: Refer to the Site Phasing Diagrams, Sheets OP-1 through OP-4 at the end of this Operations Plan section.
- Leachate and stormwater controls: Refer to the Leachate Collection System, Sheet CP-3, Operational Cover Grading Plan, Sheet CP-4, and Stormwater Basin Plan and Details, Sheet CP-6.
- Erosion and Sediment Control Plan, Sheets CP-10 through CP-12.

For a description of the landfill components, please refer to Section I, Construction Plan Application Plan Report.

1.1 Landfilling Sequence

The development of the landfill will be in phased cells. Landfilling operations will generally proceed from the upgradient end of the cell towards the downgradient end of the cell. Each cell will have a cell divider berm for stormwater segregation. An active cell will be filled to approximately 10 feet on the initial lift over the entire cell floor prior to beginning the second lift of waste. This method is intended to reduce leachate generation by diverting stormwater within the downgradient cells. Site Phasing Diagrams, Sheets OP-1 through OP-4 are presented at the end of this Operations Plan section.

1.2 Stormwater Collection and Removal System

Outside the landfill limits, the perimeter berms with adjacent permanent drainage ditches will divert surface water run-on from entering the landfill limits.

Cell divider berms and a temporary series of pipes are provided within each cell to allow removal of surface water that has not contacted waste. These temporary pipes, located at the low point of the cell, pass through the cell divider berms to allow open channel flow drainage to the downgradient surface water collection and removal pipe where surface water is discharged into a sediment basin.

Prior to landfilling within a cell the surface water drainage pipes within the cell shall be removed and replaced with backfill. The surface water drainage pipes exiting the landfill (Cell 1 and Cell 4) shall be abandoned and capped with a geomembrane patch prior to landfilling within that particular cell. The surface water within the active cell will infiltrate through the operational cover and flow into the leachate header pipe and sump located at the low point of the cell. The surface water within an active cell will discharge from the sump to the stormwater basin and will be treated as leachate.
Landfilling will progress from upgradient to downgradient; therefore, surface water will drain downgradient of the active face and flow into the leachate collection system. Landfilled areas upgradient of the active face will be graded so that surface water drains around the active face.

1.3 Leachate Collection and Removal System

The leachate collection and removal system on the landfill side slopes consists of a geocomposite (geotextile/geonet/geotextile). The leachate collection and removal system on the landfill floor consists of a geonet overlain by a geotextile with a series of collection pipes. The pipe is connected to an HDPE sump that provides gravity drainage of the leachate to a stormwater basin. A lift station is designed in the stormwater basin for discharge to the on-site wastewater treatment system.

The general operation required to activate a cell for waste placement will include the activation of the leachate collection and removal system within the active cell. This task is accomplished by opening the valve of the leachate discharge line from the sump. The temporary cover liner on the side slopes shall be removed within the cell to be activated. The exposed edge of the temporary cover liner shall have continuous sandbags along the edge to prevent wind from blowing beneath the temporary liner.

Prior to landfilling over the sump area, the riser pipe at the sump shall be cut and covered with geotextile and operational cover as shown on the Cell Details, Sheet CP-5.

Clean-out pipes have been provided at each end of the leachate header pipe in the event that the leachate collection and removal pipes become clogged. If a clogging problem exists within a pipe, the pipe can be cleaned out by the use of a clean-out snake or high pressure water flushing. Water flushing is recommended annually for each landfilled cell.

1.4 Placement of Waste and Cover Material

The gypsum residue will be stockpiled adjacent to the landfill by a conveyor system. The waste will then be loaded onto dump trucks and hauled to the landfill active face. As waste is dumped from trucks, the waste shall be spread with a dozer in lifts no greater than 12 inches.

Landfilling shall proceed as shown on the Phasing Diagrams, Sheets OP-1 through OP-4.

Waste shall be compacted in 8-inch thick lifts with a dozer and placed on the smallest active face as feasible. The waste shall be compacted to a minimum of 90 percent of standard Proctor maximum dry density. A 10-foot lift of FGD waste shall be placed within the cell floor prior to increasing the height of the gypsum.

Loads of waste that exhibit higher moisture content than anticipated shall be placed no closer than 50 feet from the active face in thin lifts and dozed into place. No waste shall be placed in areas of accumulated water.

Loads of residue from the wastewater treatment clarifier shall be mixed with the gypsum and spread in a thin, maximum 6-inch thick layer away from the side slopes.

The waste shall be compacted as densely as practical using dozers. Waste placement will be in lifts not less than 10 feet in vertical thickness for the initial fill placement adjacent to the lined side slopes. A minimum of two feet of waste must be placed on the cell floor before truck access is allowed. Based on the moisture content of the waste, additional compaction by a dozer may be required to achieve higher compaction rates. Initially the waste shall be placed from upgradient to downgradient, but as higher waste elevations are achieved, the waste may be placed from downgradient to upgradient on the active face slope as long as landfill surfaces are graded to allow proper drainage. A minimum five percent slope shall be graded on the landfill surface to promote surface water runoff. Gypsum waste shall not be disposed of in water, and surface water shall not be impounded over or in the gypsum waste.

No waste shall be placed within 10 feet of the top of slope. In order to reduce stresses on the geomembrane, no waste shall be dozed up the side slopes.

As waste is being placed directly onto the operational cover, care shall be taken so that the dozer does not make sharp turns on the operational cover to eliminate the possibility of tearing or puncturing the liner system.

Due to the recycling potential for the gypsum, daily or intermediate soil cover is not recommended over the gypsum. Soil cover may contaminate the gypsum and cause its reuse potential to be eliminated. Also, the surface of the gypsum creates a crust that aids in stormwater runoff and prevents blowing of dust.

An intermediate cover may be placed on areas needed for equipment access. As final closure grades are reached, the 18-inch thick compacted soil layer shall be placed over the gypsum. The final cover system shall be placed in accordance with the Closure/Post-Closure Plan, Section V.

1.5 Waste Acceptance and Disposal Requirements

The permit requirements for the FGD gypsum landfill will allow the facility to accept the following waste types:

1. FGD residue material that is not needed to meet the production needs of any beneficial use options.
2. FGD residue material that does not meet the specifications required for beneficial use.
3. FGD residue material that is removed from settling or clarifier stages of the associated waste water treatment facility.

The Operations Manager shall notify the Division within 24 hours of attempt to dispose of any other waste products. No hazardous, liquid, or infectious waste shall be accepted or disposed of in the FGD residue landfill.

1.6 Hours of Operation

The FGD landfill will be open seven days a week, 24 hours a day as required.

1.7 Training of Facility Personnel

Due to the diversity of job tasks required at landfills and the critical nature of the landfill components, personnel are properly trained to handle the operation and maintenance of the facility. Some of the critical tasks include:

- Equipment operations;
- Inspection and maintenance of storm water and erosion control devices;
- Accurate records of waste loading (quantitative and qualitative);
- Identification of hazardous and liquid wastes;
- Control of accidental fires; and
- Control of dust.

The proposed staff for this facility are properly trained for operation and maintenance of this type of landfill.

1.8 Operations Record Keeping

The Belews Creek Steam Station maintains an on-site operating record. The record keeping shall include:

a. Solid waste facility permit
b. Approved groundwater monitoring plan
c. Groundwater monitoring results
d. Erosion & sediment control plan permit
e. NPDES stormwater discharge permit
f. FGD Gypsum Landfill Operations Plan
g. Closure and post-closure monitoring plan and inspection results
h. Regulatory agency inspection reports
i. Employee training program and records
j. Site drawings and technical specifications

The above records will be kept in the operating record for the active life of the landfill and the post-closure care period. All information contained in the operating record must be furnished upon request to the Division or be made available for inspection by the Division.

2.0 SITE OPERATIONS AND MAINTENANCE

The Duke Energy Belews Creek Steam Station FGD Residue Landfill is owned and operated by Duke Energy Corporation. Operation and maintenance of the landfill will be the responsibility of the Operations Manager, Mr. Randolph W. Price.

2.1 Access and Security Requirements

The site lies entirely within Duke Energy property. Security is currently in place that includes fencing, wooded buffers and security check stations.

Access roads to the site shall be of all weather construction and maintained in good condition.

Directional signs shall be placed along the access road to the landfill. A sign shall also be posted at the landfill that includes the permit number, hours of operation, and a statement reading, "ONLY FGD RESIDUE WASTE IS ACCEPTED AT THIS LANDFILL, NO HAZARDOUS OR LIQUID WASTE PERMITTED."

2.2 Erosion/Sediment Control Maintenance

The site is designed with erosion and sedimentation control in accordance with the requirements of the Sedimentation Pollution Control Law (15A NCAC4). Erosion/sedimentation control structures include sediment basins, outlet protection aprons

and diversion ditches. Sediment basins shall be checked after periods of significant runoff and as specified in the Erosion and Sediment Control Plan drawings, Sheets CP-10, CP-11, and CP-12. Sediment shall be removed to its original dimensions when the sediment accumulates to one half of the design depth. Excavated sediment shall be transported to the soil stockpile area. The sediment basins, embankments, and outlets shall also be inspected for erosion damage after each significant rainfall event. All necessary repairs shall be made immediately.

Diversion ditches shall be inspected for damage after each significant rainfall event, as specified on the Erosion and Sediment Control Plan. Riprap channels and outlet protection aprons shall be inspected for wash outs. Riprap shall be added to these areas as needed.

Exterior embankment slopes shall be periodically inspected for erosion. These areas shall be mowed at least twice a year. The embankment slopes shall be refertilized in the second year unless vegetation growth is fully adequate. Reseed, fertilize, and mulch damaged areas immediately. Seeding, fertilizing and mulching shall be in accordance with the seeding specifications in the Erosion and Sediment Control Plan.

2.3 Stormwater Structures

All culverts and inlets shall be inspected quarterly for signs of damage, settlement, clogging, siltation build-up or washouts. Repairs to the stormwater structures shall be made as soon as possible.

2.4 Dust, Litter, Odors and Vectors

Dust generated due to landfill activities will be controlled through the application of water by truck or other approved dust control products, if necessary. The gypsum surface typically generates a crust which reduces the dust potential for the gypsum. Additionally, final cover will be vegetated as soon as is practical in order to minimize the blowing of dust on-site. Odors and vectors are typically not a problem at FGD gypsum landfills.

2.5 Fire Control

The Belews Creek Steam Station has employees trained in fire control at the plant. In the event of fire to equipment at the landfill site, Duke's fire control personnel will be immediately dispatched.

2.6 Groundwater Monitoring Wells

Groundwater monitoring wells will be located around the landfill perimeter. Care must be taken around the wells to prevent any damage to the wells. The proposed groundwater monitoring plan including well locations, screened intervals, depths and construction details

is included in the Groundwater Monitoring Plan in the Site Suitability report.

2.7 Gas Monitoring

Due to the possibility of minor H_2S gas production at the gypsum/soil interface, a passive gas system has been designed for the final cover system of the FGD gypsum landfill. During landfilling activities H_2S gas measurements will be taken at the leachate collection cleanout pipes and sump discharge pipe inverts in the stormwater basin. Following landfill closure activities, H_2S gas measurements may also be taken from the gas vents. In the event that H_2S gas is detected, the proposed passive gas system can be converted into an active gas system with gas control devices if required.

Under Seperate Cover

PHASING DIAGRAMS

CLOSURE/POST-CLOSURE PLAN

DUKE ENERGY CORP.
BELEWS CREEK STEAM STATION
FGD RESIDUE LANDFILL, PHASE 1
STOKES COUNTY, NC

Prepared By:

CHAS. H. SELLS, INC.
Consulting Engineers, Surveyors & Photogrammetrists
128 Overhill Drive, Suite 105, Mooresville, NC 28117
Phone: (704) 662-0100
Fax: (704) 662-0101

NORTH CAROLINA PROFESSIONAL SEAL 17582 ENGINEER DAN E. BREWER
9/30/05

September 30, 2005

CLOSURE/POST-CLOSURE PLAN

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1.0 INTRODUCTION

The purpose of the Closure/Post-Closure Plan is to outline the sequence for closing the landfill phase and the post-closure maintenance activities for each phase. Closure is designed to minimize the need for long term maintenance and to control the post-closure release of contaminants. Closure activities may be revised as appropriate for materials, specifications, technology advances or changes in regulations at that time.

Phasing of landfill development is designed so that final cover can be established as soon as possible. The final cover will be constructed in stages as cells of the landfill reach final grade, at the end of the cell's life. Intermediate grading, that indicate the contours prior to development of adjacent cells, is shown on the Phasing Diagrams in the Operations and Maintenance Plan. The final landfill contours will have erosion control benches and side slopes at a maximum 4H:1V. The central portion of the landfill will have a minimum 4% slope. Complete closure of Phase 1 will encompass a gross waste volume (including cover soils) of approximately 1.6 million cubic yards. Final closure of the FGD residue landfill will commence when Duke Energy declares that no more waste will be accepted or as directed by NCDENR, Division of Waste Management, Solid Waste Section.

The owner or operator shall notify the Division that a notice of intent to close the FGD gypsum landfill has been placed in the operating record. Closure activities for the FGD gypsum landfill shall begin no later than 30 days after final receipt of waste unless otherwise approved by NCDENR.

The final cover system for a completed phase will be completed within 180 days following the beginning of closure activities unless otherwise approved by NCDENR. The final cover system for the closed phase will be certified by a professional engineer as being completed in accordance with the Closure Plan. Duke Energy shall record a notation on the deed to the landfill property stating that the property has been used as a landfill and its use is restricted under the Closure Plan approved by the NCDENR Division of Waste Management. NCDENR will be notified by Duke Energy of the closure completion, certification, deed notation and placement in the operating record.

Following closure operations, the landfill will be developed as a grassed area. If the landfill must be closed prior to reaching the final contours, the surface of the landfill will be sloped to a minimum grade of 4 percent and a maximum grade of 25 percent. A final cover will be established over the landfill in accordance with the Technical Specifications for Closure, Section II.

2.0 FINAL COVER SYSTEM

2.1 Compacted Soil Layer

The compacted soil layer will consist of an 18-inch thick compacted soil layer directly above the gypsum waste. The compacted soil layer provides a smooth surface for the geomembrane and reduces the infiltration of water into the waste.

2.2 Geomembrane

The geomembrane is a 40-mil thick, textured linear low density polyethylene (LLDPE) material. The geomembrane provides a barrier layer that reduces leachate generation.

2.3 Geocomposite

The geocomposite is a non-woven geotextile/ HDPE geonet/ non-woven geotextile composite. The geocomposite provides a drainage layer for water that percolates through the vegetative soil layer. The geocomposite also reduces the build-up of water directly on top of the geomembrane. A perimeter toe drain is provided at the toe of the landfill slope and at each bench to drain out water flowing through the geocomposite.

2.4 Vegetative Soil Layer

The vegetative layer will consist of 18 inches of general fill with a 6-inch thick topsoil layer suitable for establishing a grass cover. The soil will be spread in two lifts and lightly compacted with a dozer. This layer will protect the geomembrane from damage due to ultraviolet degradation, desiccation (drying), freeze-thaw, erosion and vectors. A mixture of fertilizer and seed will be applied to the surface which will rapidly establish a continuous dense cover of grass.

2.5 Gas Venting System

A gas venting system will be incorporated into the final cap configuration. A passive gas venting system will be required to remove gas that collects beneath the geomembrane cap. The gas venting system will consist of screened pipes encased in a gravel trench. The vents will be connected to the upper geomembrane cap by means of a geomembrane boot to retard surface water infiltration into the landfilled waste. The locations and quantity of vents are shown on the Gas Venting System Plan.

3.0 POST-CLOSURE MAINTENANCE

3.1 Post-Closure Period

Post closure maintenance of the facility after closure will be conducted for 30 years and consist of the following elements:

- Inspection and maintenance of final cover,
- Control of access, and
- Operation, inspection and maintenance of surface water and erosion/sedimentation control systems.

The post-closure care period may be decreased by the NCDENR Division of Waste Management if the owner or operator demonstrates that the decreased period is sufficient to protect human health and the environment. However, the post-closure care period may also be increased by the NCDENR Division of Waste Management if the Division determines that an increased period is required to protect human health and the environment.

Following completion of the post-closure care period of the FGD residue landfill, the owner or operator shall notify the NCDENR Division of Waste Management that a certification, by a registered professional engineer, verifying that post-closure care has been completed in accordance with the post-closure plan and has been placed in the operating record.

3.2 Erosion and Sediment Control

Surface water running off the landfill during and after a rainfall event will be collected and routed off the cover by erosion control benches. Surface water that flows towards the landfill from off-site uphill areas (run-on) will be intercepted and channeled away from the landfill and final cover surface by diversion ditches and perimeter berms.

Erosion will be controlled by vegetation, erosion control benches and run-on diversion. Vegetation will aid in reducing soil erosion. Benches break the velocity of sheet flow, control development of erosion features before they damage the final cover, and divert runoff into manageable flow volumes. Run-on diversion helps eliminate the source of erosion. Sediment laden runoff will be collected in the sediment basins.

All diversion ditches and erosion control benches shall be inspected quarterly for signs of wash-outs or siltation. Riprap channels and outlet protection aprons shall be inspected for wash-outs. Damaged areas shall be repaired as soon as possible.

The sediment basins shall be inspected after significant rainfall events. Sediment shall be removed when the sediment accumulates to one half the design depth.

Culverts and inlets shall be inspected quarterly for signs of damage, siltation buildup, settlement or clogging. Repairs shall be made as soon as possible.

3.3 Final Cover

The final cover shall be inspected quarterly for signs of settlement, erosion, vector damage and bare spots. The grass cover shall be mowed at least twice a year. Bare spots shall be re-vegetated with grass seed. Any deep-rooted or woody vegetation that may have established itself on the cover soil shall be removed so that the deep root growth will not penetrate the geomembrane. Areas exhibiting settlement shall be filled to provide positive drainage. Eroded areas shall be filled and reseeded as needed.

3.4 Gas Venting System

The gas venting system shall be inspected quarterly for signs of damage. Geomembrane connections to each vent shall also be inspected. Any damaged vents shall be repaired.

3.5 Groundwater Monitoring Wells

All groundwater monitoring wells shall be inspected during groundwater sampling events. Any damaged wells shall be repaired immediately. Access to the wells shall be maintained to provide all-weather access.

4.0 POST-CLOSURE MONITORING

Following closure of the landfill, a monitoring program will be implemented to monitor the quality of groundwater and surface water in the areas surrounding the landfill. The monitoring program will be in accordance with the State approved monitoring plan and will continue for a period of 30 years after final closure. The length of the post-closure care period may be decreased or increased by the NCDENR Division of Waste Management if the reduced period is sufficient or the lengthened period is necessary to protect human health and the environment. Following completion of the post-closure period, the owner or operator shall notify the Division that a certification signed by a registered professional engineer, verifying that the post-closure care has been completed in accordance with the post-closure plan, has been placed in the operating record.

4.1 Groundwater

The Groundwater Monitoring Plan will be continued semi-annually (or as required) after final closure. The results of the analytical testing will be submitted to NCDENR as directed in the Groundwater Monitoring Plan.

4.2 Surface Water

Surface water monitoring of the downgradient tributaries to Belews Lake will be continued after closure as required by the NCDENR Division of Waste Management. Surface water discharged from the sediment basins will be sampled and analyzed if required by the NPDES stormwater permit.

Appendices

A

CALCULATIONS

Landfill Capacity/Borrow Soil Quantities

Stability

- Stability of Side Slope Liner System
- Stability of Final Liner System
- Design of Ballast System for Side Slope Liner System
  Evaluation of Stresses Due to Liner Self Weight

Foundation Evaluation

- Subgrade Bearing Capacity
- Subgrade Settlement Evaluation

Geosynthetics

- Geotextile Design
- Geonet Design
- Anchor Trench Design

Leachate Generation

- Help Model, Discussion and Results

Leachate Collection and Removal Pipe

- LCR Pipe Spacing
- LCR Pipe Flow Calculations
- LCR Pipe Stresses

Stormwater Management

- Stormwater Basin Detention Calculations
- Cell Riser Calculations
- Stormwater Culvert Calculations
- Outlet Protection Apron Calculations
- Permanent Diversion Channel Calculations

Landfill Capacity/Borrow Soil Quantities

CHAS. H. SELLS, INC.
Consulting Engineers, Surveyors & Photogrammetrists

Comp. By DEB Date 9/29/05
Chkd. By WMM Date 9/29/05

Project Belews Creek FGD Landfill
Subject Landfill Capacity

Sheet No. 1 of 1
Job No. 046119

LANDFILL CAPACITY

Given: Gypsum Disposal Rate = 380,000 CY per Year* (Ref: Duke Power)

* Assumes no reuse of gypsum and 100% disposal in landfill (worst case)

Clarifier Sludge Rate = 50,000 Tons/Yr

γ_{sludge} = 44 lbs/CF (Ref: S+ME Report)

$$50{,}000 \frac{\text{Tons}}{\text{Yr}} \left(\frac{2000 \text{ lbs}}{1 \text{ Ton}}\right)\left(\frac{\text{ft}^3}{44 \text{ lbs}}\right)\left(\frac{1 \text{ CY}}{27 \text{ ft}^3}\right)(1 \text{ Yrs}) = 84{,}175 \text{ CY}$$

Gross Landfill Volume (from DTM ACAD LDD Model) = 1,614,000 CY

Gross Volume from Final Cover to Top of Operational Cover

Subtract Final Cover

$$24 \text{ ac}\,(43{,}560 \text{ ft}^2/\text{ac})(3.5 \text{ ft})(1 \text{ CY}/27 \text{ CF}) = 135{,}500 \text{ CY}$$

Net Landfill Volume = 1,614,000 CY Gross

− 135,500 CY Final Cover

1,478,500 CY Net

$$\text{Capacity} = \frac{1{,}478{,}500 \text{ CY}}{380{,}000 \text{ CY/Yr} + 84{,}175 \text{ CY/Yr}} = \boxed{3.2 \text{ Years}}$$

If 50% Gypsum reused, Capacity = 5.4 Years

CHAS. H. SELLS, INC.
Consulting Engineers, Surveyors & Photogrammetrists

Comp. By DEB Date 9/19/05
Chkd. By WMM Date 9/29/05

Project Belews Creek FGD Landfill
Subject Landfill Borrow Soil

Sheet No. 1 of 1
Job No. 046112

BORROW SOIL QUANTITIES

Determine if Suitable Quantities of borrow Soil Available to Operate and Close Landfill

Given: Total Cut from DTM = 597,600 CY
Total Fill from DTM = 115,000 CY

Total Excess Material = 482,600 CY

Subtract Operational and Final Cover (from Landfill Capacity Calcs)

29,000 CY Bottom Cover
3,200 CY Berms
135,500 CY Final Cover

167,700 CY Needed

Total Excess borrow

482,600 CY
− 167,700 CY

314,900 CY OK 2

Stability

- Stability of Side Slope Liner System
- Stability of Final Liner System
- Design of Ballast System for Side Slope Liner System
- Evaluation of Stresses Due to Liner Self Weight

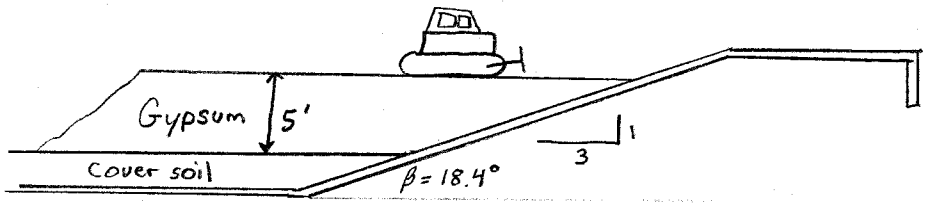
Evaluate Stability of Side Slope Liner System

Design Slope 3H : 1V

Liner Design 60 mil textured geomembrane
Geo composite

Friction Angles Geocomposite / Textured Geomembrane $\delta_2 = 26°$
Gypsum / Geocomposite $\delta_1 = 20°$
Textured Geomembrane / Subgrade $\delta_3 = 22°$
$\emptyset$ Gypsum = 32° (per S*ME triaxial test data)

FS_{MIN} Against Sliding = 1.5



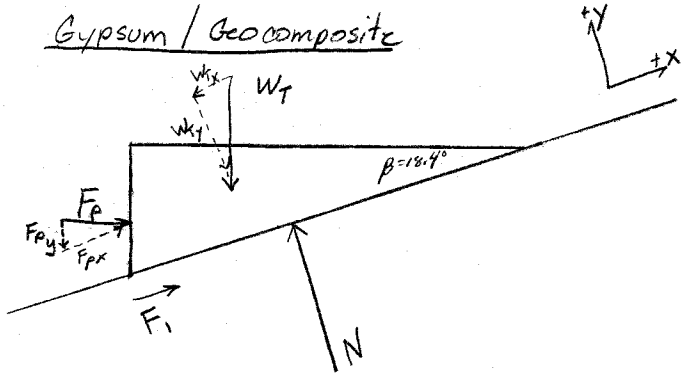
Assume CAT LGP Dozer : 18,200 lbs, 2 tracks, 2ft wide

$$W_D = \frac{18{,}200 \text{ lbs}}{2 \text{ tracks } (2\text{ft})} = \underline{4{,}550 \text{ lbs/ft}}$$

$$W_{Gypsum} = \tfrac{1}{2}(15\text{ ft})(5\text{ ft})(89 \text{ PCF}) = \underline{3338 \text{ lb/ft}}$$

Assume 5ft wedge of Gypsum on slope

Gypsum / Geocomposite



$W_T = W_D + W_{Gypsum}$
$= 4,550 \text{ lbs/ft} + 3338 \text{ lbs/ft}$
$= 7,888 \text{ lbs/ft}$

$W_{Tx} = W_T \sin\beta$
$= 7,888 \text{ lb/ft} \sin 18.4°$
$= \underline{2490 \text{ lbs/ft}}$

$W_{Ty} = W_T \cos\beta$
$= 7,888 \text{ lbs/ft} \cos 18.4°$
$= 7,485 \text{ lbs/ft}$

$\Sigma f_y = 0$ $\quad N = W_{Ty}$ (Assume F_{py} very small)
$= \underline{\underline{7,485 \text{ lbs/ft}}}$

Passive Force = F_p

$F_p = \frac{1}{2} \gamma H^2 \tan^2(45 + \phi/2)$
$= \frac{1}{2}(89 \text{ PCF})(5\text{ft})^2 \tan^2(45 + 32/2)$
$= 3620 \text{ lbs/ft}$

$F_{px} = F_p \cos\beta = 3620 \text{ lbs/ft} \cos 18.4°$
$= \underline{3436 \text{ lbs/ft}}$

Friction force = F_1

$F_1 = \mu N$
$= \tan\delta_1 N$
$= N \tan 20°$
$= 7,485 \text{ lbs/ft} \tan 20$
$= \underline{2724 \text{ lbs/ft}}$

$F_R = F_{px} - W_{Tx} = 3,436 \text{ lbs/ft} - 2,490 \text{ lbs/f}$
$= \underline{946 \text{ lbs/ft}}$

$F_R << F_1$ No tension, sliding will not occur ok
$946 \text{ lbs/ft} < 2724 \text{ lbs/ft}$ set $F_1 = F_R = \underline{\underline{946 \text{ lbs/ft}}}$

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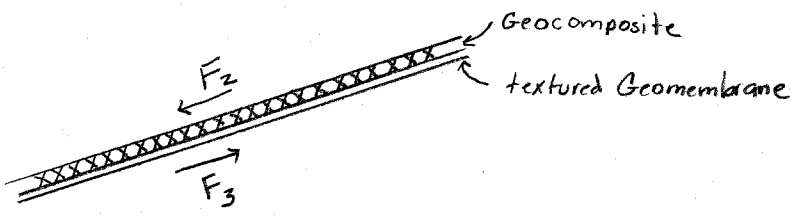
Comp. By PMB Date 9-30-05 | Project Belows Creek FGD Landfill | Sheet No. 3 of 3
Chkd. By WMM Date 9/30/05 | Subject Linear Stability | Job No. 046119

Geocomposite / Textured Geomembrane

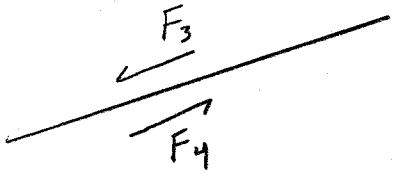


$F_2 = F_1 = 946\ ^{lbs}/_{ft}$

$F_{3\,Avail} = N \tan \delta_2$
$= 7{,}485\ ^{lbs}/_{ft} (\tan 26°)$
$= 3650\ ^{lbs}/_{ft}$

$FS = \frac{3650}{946} = 3.9$ ok

Textured Geomembrane / soil



$F_3 = F_2 = 946\ ^{lbs}/_{ft}$

$F_{4\,Avail} = N \tan \delta_3$
$= 7{,}485\ ^{lbs}/_{ft} (\tan 22°)$
$= 3{,}024\ ^{lbs}/_{ft}$

$FS = \frac{3024}{946} = 3.2$ ok

∴ Specify Min. 5ft Gypsum is required on Cell Floor prior to any equiptment on slope. Stresses on Geomembrane will dissipate within Gypsum passive pressure (F_p).

FrictionFlex® Application Data

GSE's FrictionFlex® process provided the industry's first textured liner. It is the only geomembrane texturing process ever to be granted a U.S. Patent. In fact, it has been awarded two (U.S. Patent numbers 4,885,201 and 5,075,135). The FrictionFlex process begins with 24-foot wide **GSE HyperFlex®** or **UltraFlex®** sheet manufactured to stringent industry standards. After the smooth sheet passes all QA testing, texturing is added to one or both sides as required. The patented manufacturing process enables **GSE** to produce a textured liner exhibiting outstanding mechanical and chemical properties demanded of **GSE's** premium grades of smooth geomembrane liners – **HyperFlex** and **UltraFlex**.

An engineer can utilize **GSE** geomembranes textured by the **FrictionFlex** process to improve the factor of safety and steepness of slopes. This increases facility design capacity, service life, and ultimately total revenue potential. GSE's textured geomembranes can be used to improve the factors in a number of applications.

An added feature of the **GSE** FF process is that an edge, approximately six inches wide, remains smooth. This smooth edge means that GSE's seaming procedures are the same for FrictionFlex textured geomembranes and smooth geomembranes and requires no change in field quality control.

The following summarizes reference data regarding FrictionFlex liner performance in contact with soils and synthetics:

* High coefficient of friction with soils
* High coefficient of friction with synthetics
* Premium grade mechanical and chemical properties
* Excellent strength elongation

Material	GSE FrictionFlex Textured Liner Materials Adhesion (lb/ft^2)	Friction Angle (deg.)	Smooth HDPE Friction Angle (deg.)
Sandy Glacial Till	27	36	20
Sandy Clay	65	35	18
Smooth Clay	39	32	16 *
Ottawa Sand	21	30	19
Non-woven Polyester Geotextile	116	28 Use 26°	11
Non-woven Polypropylene Geotextile	133	33	12

NOTE: The above values are appoximate. Testing performed according to ASTM D 5321. GSE recommends that specific data be developed for all application designs. Shearbox testing of the specific geosynthetic and natural components of the composite is necessary to establish an appropriate design basis. GSE will be pleased to provide any necessary material samples for such purposes.

This information is provided for reference purposes only and is not intended as a warranty or guarantee. GSE assumes no liability in connection with the use of this information. Check with GSE for current, standard minimum quality assurance procedures.

GSE Lining Technology, Inc.
19103 Gundle Road
Houston, Texas 77073
USA
800-435-2008
713-443-8564
FAX: 713-875-6010

GSE Lining Technology, GmbH
Buxtehuder Strasse 112
D-21073 Hamburg
Germany
49-40-767-420
FAX: 49-40-767-42-33

GSE Lining Technology Ltd.
198 Brooklands Road
Weybridge, Surrey KT 13 ORJ
United Kingdom
44-1-932-828-580
FAX: 44-1-932-821-551

GSE Lining Technology Pte Ltd.
182 Tagore Lane
Singapore 787581
65-459-2466
FAX: 65-459-4366

GSE Lining Technology Pty Ltd.
24 Regent Cresent
Moorebank, New South Wales
Australia 2170
61-2-821-2977
FAX: 61-2-821-3611

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Reference (1)

c = the cohesion of soil,
δ = the friction angle of soil to fabric, and
ϕ = the friction angle of soil.

Results from such a test setup by Martin, et al. [13], are presented in Table 2.6 for four geotextile types against three different cohesionless soils. Soil-to-fabric friction angles are given, as well as the fabric efficiency versus the soil friction angle by itself as per Equation 2.8. Here it is seen that most geotextiles can mobilize a high percentage of the soil's friction and can be used to advantage in situations requiring this feature. A review and compilation of a number of direct shear tests on various fabrics against different granular soils is given by Richards and Scott [14]. Another review by Williams and Houlihan [15] covers a wider range of soils, including some sands, silts, and mixed soils.

2.2.3.11 Pullout (Anchorage) Tests

Geotextiles are often called upon to provide anchorage for many applications within the reinforcement function. Such anchorage usually has the fabric sandwiched between soil on each side of it. The resistance can be modeled in the laboratory using a pullout test, shown schematically in Figure 2.8b. The pullout resistance is obviously dependent on the normal force applied to the soil surrounding it, which mobilizes shear forces on both sides of the fabric.

Test results by Collios et al. [16] show a relationship of pullout test results to shear test results with some notable exceptions. If the soil particles are smaller than the fabric openings, efficiencies are higher; if not, they can be lower. In all cases, however, pullout test resistances are less than shear test resistances. This is due to the fact that the fabric is taut and exhibits large deformations. This in turn causes the soil particles to reorient themselves into a reduced-shear-strength situation at the soil-fabric interfaces, resulting in lower pullout resistance. The stress state mobilized in this test is a very complex one requiring additional research.

TABLE 2.6 SOIL-TO-FABRIC FRICTION ANGLES AND EFFICIENCIES (IN PARENTHESES) IN COHESIONLESS SOIL

Geotextile type	Manufacturer's designation	Concrete sand ϕ = 30 deg.	Rounded sand ϕ = 28 deg.	Silty sand ϕ = 26 deg.
Woven, monofilament	Polyfilter X	26 deg. (84%)	—	—
Woven, slit film	500X	24 deg. (77%)	24 deg. (84%)	23 deg. (87%)
Nonwoven, melt-bonded	3401	26 deg. (84%)	—	—
Nonwoven, needle-punched	CZ600	30 deg. (100%)	26 deg. (92%)	25 deg. (96%)

Source: After Martin et al. [13]

BLACK

REFERENCE 1

y cs. This may occur

erally has a high efficiency
is generally higher for wo-
nded nonwovens. This is
of soil-to-fabric interaction
factors or trends observed
tiles include:

xtiles decreases the shear
ri et al., 1986).
ctions produce different in-
typically being lower (Ei-

ect on the interface friction
geotextiles (Eigenbrod and

t due to the interlocking of
nwoven geotextiles (Eigen-

nediate and high confining
e adhesion decrease
Williams and Houli-

ent on the soil and type of
sts using the actual materials
preliminary designs, Table
ts reported in the literature
986; Eigenbrod and Locker,
., 1990; Koutsourais et al.,
Table 3.13 are due to varia-
geotextiles. The range also

tain openings or pores, the
argely dependent on whether
ugh to push the failure plane
into the adjacent soils, the

TABLE 3.13 Typical Range of Reported Soil Geotextile Friction Angles

Geotextile	Sand Friction Angle (deg) (Efficiency)	Clay Friction Angle (deg) (Efficiency)
Woven	23–42 (0.68–1.0)	16–26 (0.61–0.93)
Nonwoven, Needle-punched	25–44 (0.67–1.0)	15–28 (0.62–0.99)
Nonwoven, resin or heat bonded	22–40 (0.56–0.91)	17–33 (0.60–0.85)

interface friction strength is generally similar to the soil strength. Factors that affect the soil strength include items such as the soil type, density, moisture content, and confining stress. For clays, the loading and shearing conditions, such as consolidated drained (CD), consolidated undrained (CU), or unconsolidated undrained (UU), also have significant influence.

If the failure plane is not pushed into the adjacent soils, low interface friction values may result. For example, the interface strength between smooth HDPE geomembranes and clay can be less than 10°. This low interface friction strength can lead to significant stability problems. Also, if the interface between the clay and geomembrane is wetted (i.e., due to condensation of water under the geomembrane, clay swelling, or excess moisture during construction), the interface strength can be further reduced (VonPein and Prasad, 1990; Mitchell et al., 1990). It is therefore critical that interface friction tests accurately model potential field conditions.

Table 3.14 summarizes soil geomembrane interface strengths based on the results reported by several researchers (Martin et al., 1984; Williams and Houlihan,

TABLE 3.14 Typical Range of Reported and Recommended Soil Geomembrane Friction Angles

Geomembrane	Reported Sand Friction Angles (deg) (Efficiency)	Recommended Sand Friction Angles, δ (deg)	Reported Clay Friction Angles (deg) (Efficiency)	Recommended Clay Friction Angles, δ (deg)
PVC	21–33 (0.62–0.93)	20–30	6–39 (0.53–1.0)	6–15
HDPE	17–28 (0.45–0.81)	17–25	5–29 (0.47–0.88)	5–10
Textured HDPE	30–45 (0.86–1.0)	30–40	7–35 (0.70–1.0)	9–15
VLDPE[a]	21–28 (0.62–0.67)	—	—	—

[a] Since VLDPE is a relatively new product, limited results were reported in the literature. It is anticipated that the range of efficiencies for VLDPE to sand interfaces is broader than shown. Blank (—) means insufficient data at this time.

TABLE 3.15 Typical Range of Reported Geosynthetic to Geosynthetic Friction Angles (Degrees)

	PVC	HDPE Smooth	HDPE Textured	Geonet
Woven Geotextile	10–28	7–11	9–17	9–18
Nonwoven, needle-punched Geotextile	16–26	8–12	15–33	10–27
Nonwoven, resin/heat-bonded Geotextile	18–21	9–11	15–16	17–21
Geonet	11–24	5–19	7–25	—

The testing conditions may also have a significant effect on results. Mitchell et al. (1990) noted that polishing of geomembrane surfaces by geotextiles reduced interface friction. Also, the orientation of geonet strands can affect the interface strength between geonets and geomembranes (Geotek, 1987; Mitchell et al., 1990). Site-specific tests should therefore be performed using the actual materials and anticipated shear conditions.

3.6.4 Geosynthetic Clay Liner Shear Strength

Limited information is currently available on the internal shear strength of GCLs, due primarily to their relatively short history. The tests that have been performed are also difficult to compare, due to the numerous variations in test conditions. Many of these variations, such as strain rate, normal load, sample size, and consolidation conditions, are similar to the variations experienced when comparing shear strength testing of other geosynthetics. An additional variation of GCLs, however, is the hydrating conditions, including the hydrating liquid. Hydration can occur under free swell, constrained swell, or partially constrained swell, or the sample may be tested unhydrated. Even if hydrated under free-swell conditions, it may be difficult to assess whether full hydration has occurred since the bentonite may be restricted from free swell by the bonded geotextiles. Also, due to the large water absorption of bentonite, most shear strength test results will incorporate some immeasurable pore pressure effects unless the test is performed at extremely low displacement rates.

Table 3.16 presents the results of direct shear testing performed under various hydration conditions. The tests were performed at a strain rate of 9 mm/min and at normal stresses up to 60 kPa. Although these test results provide some information on the internal shear strength of GCLs, it is highly recommended that project specific testing be performed.

Evaluate Stability of Final Cover Liner System

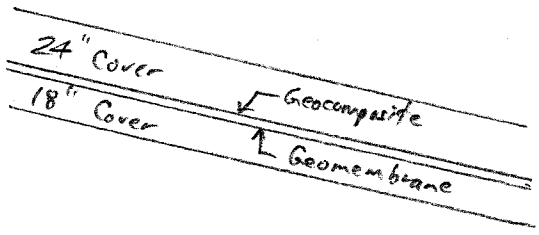
Design Slope 4 H : 1 V

Liner Design 40 mil textured geomembrane
Geocomposite

Friction Angles Soil / Geocomposite $\delta_1 = 20°$
Geocomposite / Textured Geomembrane $\delta_2 = 26°$
Textured Geomembrane / Cover Soil $\delta_3 = 22°$
$\phi_{soil} \approx 32°$, $\gamma_{sat} = 90$ PCF

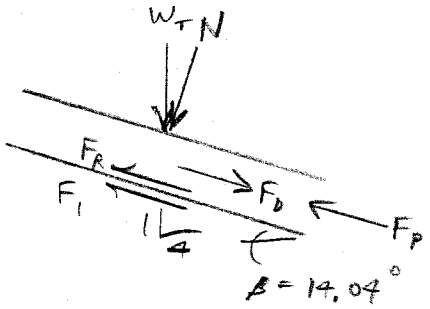
FS_{min} sliding = 1.5

Assume CAT D5 LGP Dozer



W_D = 4550 lb/ft (per Side Slope Liner Calcs)

Final Cover / Geocomposite



$W_T = W_D + W_S$ (Length between bench)

$= 4550 + 90 \text{ lbs/ft}^3 (100 \text{ ft})(2 \text{ ft})$

$= \underline{22{,}550 \text{ lb/ft}}$

$N = W_T \cos\beta$

$= 22{,}550 \cos 14°$

$= \underline{21{,}880 \text{ lb/ft}}$

$F_D = W_T \sin\beta$

$= 22,550 \sin 14°$

$= \underline{5455 \text{ lb/ft}}$

$F_P = \frac{1}{2}\gamma H^2 \tan^2(45 + \phi/2)$

$= \frac{1}{2}(90 \text{ pcf})(2 \text{ ft})^2 \tan^2(45 + 32/2)$

$= \underline{586 \text{ lb/ft}}$

$F_R = F_D - F_P = 5455 - 586 = \underline{4869 \text{ lb/ft}}$

$F_{1\,AVAIL} = N \tan\delta_1$

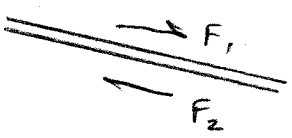
$= 21,880 \tan 20°$

$= \underline{7964 \text{ lb/ft}}$

$FS = \frac{F_1}{F_R} = \frac{7964}{4869} = \underline{1.6}$ OK

Geocomposite / Geomembrane

$F_1 = F_R = 4869 \text{ lb/ft}$



$F_{2\,AVAIL.} = N \tan\delta_2$

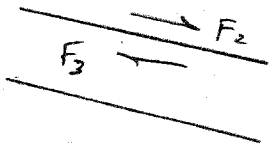
$= 21,880 \tan 26°$

$= \underline{10,672 \text{ lb/ft}}$

$FS = \frac{F_2}{F_1} = \frac{10,672}{4869} = \underline{2.2}$ OK

Geomembrane / Soil

$F_2 = F_1 = 4869$ lb/ft



$F_{3\ AVAIL.} = N \tan \delta_3$

$= 21,880 \tan 22°$

$= \underline{8840}$ lb/ft

$FS = \frac{F_3}{F_2} = \frac{8840}{4869} = \underline{1.8}$ OK

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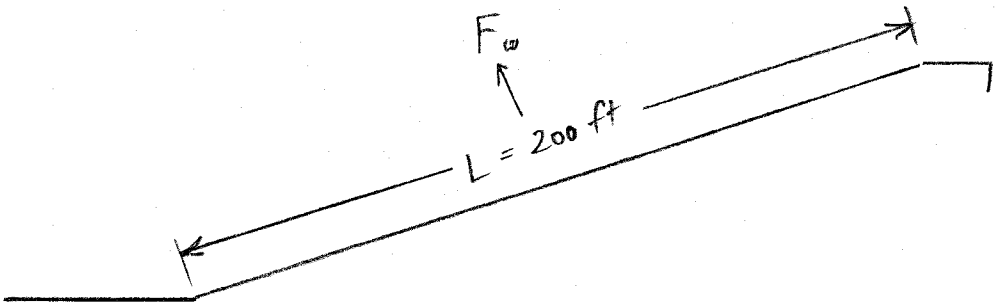
Comp. By DEB Date 9/26/05 Project Belews Creek FGD Landfill Sheet No. ..... of .....
Chkd. By WMM Date 9/30/05 Subject Ballast System Job No. .....

Design Ballast System for Side Slope Liner



Wind uplift, $S_e = 0.00124\ V^2$ where V = design velocity, ft/sec
S = uplift pressure, PSF

For design velocity of 40 mph Ref: Richardson, Scheer GFR, Sept. 2003 "Design of Geomembrane Protective Rainsheets"

$$40\ \frac{mi}{hr}\left(\frac{1\ hr}{3600\ sec}\right)\left(\frac{5280\ ft}{1\ mi}\right) = 58.67\ ft/sec$$

$$S_e = 0.00124\,(58.67)^2 = 4.27\ PSF$$

Assume 10 ft Horz. Spacing

For 200 ft slope, uplift = $4.27\ lb/ft^2\,(200\ ft)(10\ ft) = 8540\ lb$

Try ½ Tires, $\frac{10\ lb}{2\ SF\ Contact} = 5\ lb/ft^2$ NG

Try 40 lb sandbags = $40\ lb/ft^2$

$$\frac{8540\ lb}{40\ lb/bag} = 214\ Sandbags$$

$$\frac{2000\ SF}{214\ Sandbags} = 9.3\ ft \quad \text{Say } \underline{\underline{1\ bag/10\ ft}} \text{ both ways}$$

∴ Leading Edge of Temp. Liner Must Have Continuous Sandbags

By Gregory N. Richardson, Ph.D., P.E.
and Pieter K. Scheet, P.E.

Design of geomembrane protective rainsheets

While driving through the San Joaquin Valley on a recent vacation, it was apparent that the farmers of that region were particularly adept in the use of very large protective tarps over mountains of god knows what. While we can debate the "geo" aspect of such designs, the use of geomembrane rainsheets to protect granular leachate collection systems is essential in many contemporary landfill designs. It is equally apparent that many engineers are not as familiar with this application as the California farmers. This article will attempt to review alternatives to rainsheets, the design of rainsheets, and operational considerations regarding rainsheets.

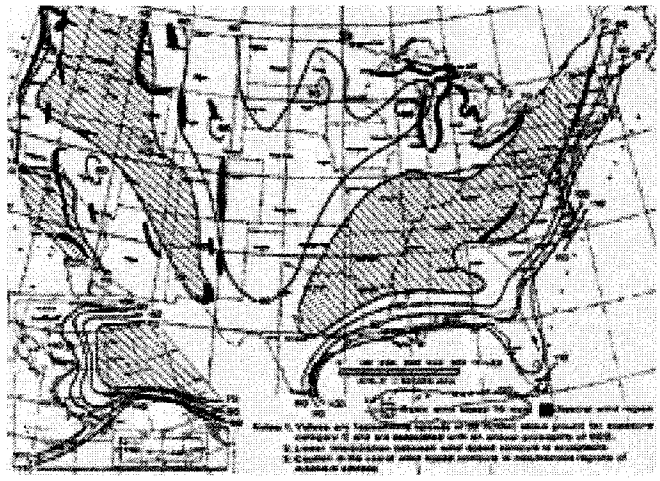
The dilemma and alternatives

A newly completed landfill leaves acres of sloped granular leachate collection system or soil operational (protective) cover soil exposed to the elements. With ensuing rains, the resulting surface erosion can reduce the thickness of the leachate collection system or operational cover below regulatory minimums or allow unwanted sedimentation to clog the leachate collection system. Alternative solutions to covering these surfaces with rainsheets have included the following:

- Hydroseeding and application of erosion control matting such as straw or jute base products. This is particularly beneficial if a fine grained operational cover soil layer is used. Obviously, this is less suitable for application to sand or gravel surface layers.
- Application of certain sludge such as primary or secondary sludge common to paper mills. The authors are not aware of the use of waste water treatment sludge in this application.
- When daily tonnage allows, placement of a 5 ft. lift of MSW over the entire surface as rapidly as possible. This is particularly attractive for bioreactor landfills that require additional leachate for recirculation.

Failed efforts include geotextile and asphalt impregnated geotextile covers. Given the swing in East Coast precipitation the past two years from record drought to record wet, the designer cannot anticipate the actual potential for short term erosion.

Figure 1. Maximum service wind velocities.



Rainsheet geomembranes

Rainsheets are just large tarps, so it is understandable that the majority of applications have used the same materials large tarps are fabricated from. The most commonly used "tarp" materials include 10 to 15 mil scrim-reinforced polyethylene geomembranes such as Griffolyn by Reef Industries Inc. of Houston and Dura-Skrim by Raven Industries of Sioux Falls, S.D.

Geomembrane manufacturers also provide 20-mil polyethylene membranes that are economical enough to serve as rainsheets. Dollarwise, the goal is to install a rainsheet for less than about $9,000 per acre ($6,000 rainsheet cost plus the cost of anchorage/ballasting).

Depending on the type of membrane, rainsheet panels have been seamed by sewing, thermal welding, and taping. Sewing is performed using a common prayer seam in the same manner used to seam the upper geotextile in a drainage composite. The stitch must be adjusted to develop approximately 60% of the tensile strength of the rainsheet. Sewing is particularly effective on the scrim-reinforced materials. While "Duck" tape may have saved NASA, it is not suitable for joining rainsheet panels. However, a truck tarp repair tape manufactured by The Tapecoat Company of Evanston, Ill. (Type G-25) has worked well.

Rainsheet design

A previous three-part series in Designer's Forum (see references) reviewed design considerations for exposed geomembrane covers. In this application, the geomembrane has a longer service life and must have a tensile strength adequate to resist large uplift forces. The exposed geomembrane covers can have dimensions exceeding 50 ft. between anchorage points. Rainsheets are similar in function, but rely on tightly spaced anchorage points to reduce the tension in the membrane cover.

Like the anchorage for exposed geomembrane covers, the anchorage for rainsheets must resist the uplift caused by winds blowing over the landfill. Guidelines for evaluating the wind uplift force have been provided by both Wayne and Koerner 1988 and Giroud et al. 1995. Both of these papers rely on the earlier work performed by Dedrick 1975.

The wind uplift pressure (at sea level), S_e, for a given wind velocity is expressed as follows:

S_e (Pa) = 0.6465 V^2 (m/s)

S_e (psf) = 0.00124 V^2 (ft./s)

This force decreases with increasing site elevation (i.e., the force is greater near the beach than in the mountains). Also, the force is greater on areas that are more exposed to the wind (i.e., the force is typically greater on the upper portions of landfill side slopes versus the more protected landfill bottom areas). A general guideline for maximum service wind velocities is provided in **Figure 1**. Designs based on lesser wind velocities are commonly justified due to the short service life of the rainsheet. A design speed of less than 40 to 50 mph is not recommended. This produces a calculated uplift pressure of between 4.3 and 6.7 psf (0.32 Pa).

Anchorage for the rainsheet is commonly provided using sandbags and/or tires. **Photo 1** shows both tires and sandbags incorporated in an anchorage system. Note that ropes are required on side slopes to maintain proper spacing of the anchors through the short service life of the rainsheet. For 60 lb. (27.2 kg) sandbags, the 6.7 psf uplift pressure requires bags spaced on 3 ft. centers! This is approximately 4,800 sandbags per acre, which is obviously cost prohibitive at a cost of about $2 to $3 per bag installed. Given that the average passenger car tire weights only 20 lb. (9.07 kg), the contact pressure from a tire lying flat on a rainsheet is less than 5 psf. Thus, the mathematics would require tires placed tread to tread. One significant disadvantage of using whole tires is that they cannot legally be placed within the waste as the landfill advances over the rainsheet. The use of split tires would allow their burial but increase the cost of using tires. Both tires and sandbags require cabling and cable anchorage to maintain their position on landfill side slopes.

Field performance has shown that spacing sandbags (or half sandbags and half tires) on a spacing of 7 to 10 ft. on center (Note that this is a greater spacing than the wind uplift calculations would indicate.) will perform with little to no damage in winds up to about 40 mph—within the typical maximum annual wind speed of most inland locations. Some damage, however, would be expected beyond 40 mph. This would likely be satisfactory for a 1 to 2 year service life, particularly if portions of the rainsheet are being removed from service with advancing operations (i.e., the exposed area is constantly being reduced).

Photo 1. **Typical anchorage system.**

An alternative to tires and sandbags is the use of proprietary earth anchors driven into the underlying sands or operational cover. Cost and the obvious concern regarding accidentally driving anchors through the liner have limited the use of such devices. Additional alternatives considered include the use of water filled geomembrane tubes. However, the target cost of $9,000 per acre limits creative alternatives.

Operational considerations

The rainsheet provides a potential hydraulic barrier and slip surface if allowed to remain over the landfill systems. However, experience has shown that trafficking the rainsheet with a trash compactor can shred the membrane and eliminate the need for its removal. Whole tires that were used as anchors must be collected as the waste advances and not be buried. One of the most important operational responsibilities is prompt repair of minor tears to the cover to prevent wind entry. Repairs can be performed quickly and economically using the truck tarp repair tape mentioned above. This reliance on operational repair of the rainsheets cannot be overstated. This is the primary reason that most liner installers resist giving warrantees for rainsheets, since operations are out of their control.

Summary

The use of geosynthetic rainsheets offers one alternative for the protection of the landfill leachate collection system prior to waste placement. Such systems are predictable in their performance and not dependent on rapid vegetation growth. It is, however, not practical to design such rain covers for maximum winds, so the potential for damage to the system is real. Generally, we find that rainsheet systems are disliked when present and sorely missed when omitted. ■

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Germain, Ann, Houlihan, Michael and Richardson, Gregory N. 2001. "Exposed Geomembrane Covers: Part 2—Geomembrane Selection." *Geotechnical Fabrics Report*, vol. 19, no. 1.

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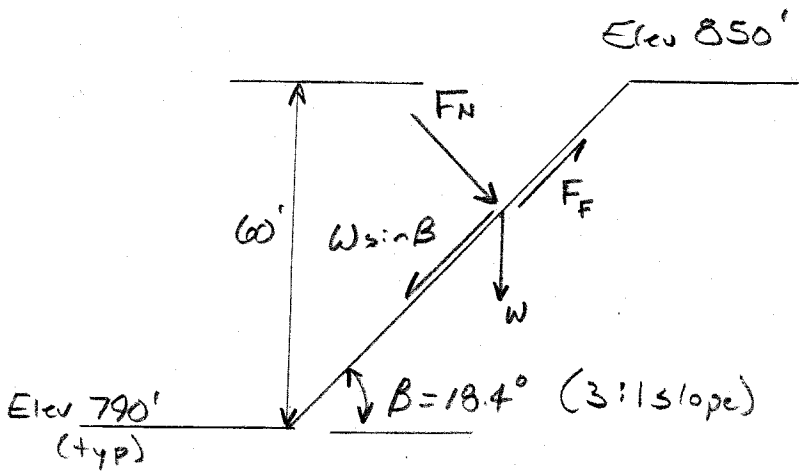
Comp. By WMM Date 10/3/05 Project Belews Creek FGD Landfill Sheet No. 1 of 2
Chkd. By DEB Date 10/7/05 Subject Tensile Stress - Liner Self Weight Job No. 046119

Objective: Evaluate stresses in geomembrane (along side slope) due to weight of liner + geocomposite.



References:

1) Geosynthetic Design Guidance for Hazardous Waste Landfills and Surface Impoundments, Richardson, Ex 3.14

2) Texdrain Geocomposite Products – Property of Texdrain 250 Geonet

Tension in Geomembrane = T

$$T = W \sin B - F_F$$

Where:
W = weight of liner system, lb/ft
B = angle of slope, degrees
F_F = friction force

W = wt of liner system x length along slope

$$= \left[\left(.941 \times 62.4 \frac{lb}{ft^3} \times \frac{0.060''}{12''/ft}\right) + \left(.20\ lb/ft^2\right)\right]\left[\frac{60'}{\sin 18.4°}\right]$$

(wt geomembrane) (wt. geonet) (length slope)

$$W = \left[\left(0.294 \frac{lb}{ft^3}\right) + \left(0.20\ lb/ft^2\right)\right][190.1\ ft]$$

$$W = 93.8\ lb/ft$$

$$F_N = W \cos\beta$$

$$= 93.8\ lb/ft \cos 18.4°$$

$$F_N = 89.0\ lb/ft$$

$$F_F = F_N \tan\delta_1$$

$$= 89.0 \frac{lb}{ft} \tan 22°$$

$$F_F = 36\ lb/ft$$

δ_1 = friction angle between soil and geomembrane = 22°

Calculate Tension, T in geomembrane:

If the resisting friction force, F_F, is greater than the weight forces, $W \sin\beta$, acting downward along the slope, then no tension forces are present in the geomembrane.

$$F_F = 36 \frac{lb}{ft} \qquad W \sin\beta = 93.8 \frac{lb}{ft} \sin 18.4° = 29.6 \frac{lb}{ft}$$

$$F_F > W \sin\beta$$

∴ No tension in geomembrane liner.

PRODUCT SELECTIONS

TexDrain is available in a wide range of product combinations allowing economical performance driven product selection.

TEXDRAIN 200 GEONET - PROPERTIES

Property	Test	Units	Typical Value
Thickness	ASTM D 5199	mil	200
Mass Per Unit Area	ASTM D 5261	lb/ft²	.162
Density, Min.	ASTM D 1505	g/cc	.94
Carbon Black Content, Min.	ASTM D 1603	%	2
Tensile Strength	ASTM D 5035 mod	lbs/inch	45
Transmissivity* (MD)	ASTM D 4716	gal/min. ft	4.8
Transmissivity*, (MD)	ASTM D 4716	m²/s	1 x 10-3

* tested between two metal plates at 15,000 lb/ft² with a hydraulic gradient of 1.0

TEXDRAIN 250 GEONET - PROPERTIES

Property	Test	Units	Typical Value
Thickness	ASTM D 5199	mil	250
Mass Per Unit Area	ASTM D 5261	lb/ft²	.20
Density, min.	ASTM D 1505	g/cc	.94
Carbon Black Content, Min.	ASTM D 1603	%	2
Tensile Strength	ASTM D 5035 mod	lbs/inch	60
Transmissivity* (MD)	ASTM D 4716	gal/min. ft	7.2
Transmissivity* (MD)	ASTM D 4716	m²/sec	1.5 x 10-3

* tested between two metal plates at 15,000 lb/ft² with a hydraulic gradient of 1.0

TexDrain Geonet can be combined with a broad range of nonwoven geotextiles, thermally bonded to either one or two sides to achieve a wide range of product performance. The chart below illustrates typical transmissivity data for several common products. For more complete data, refer to the latest TexDrain Certified Properties Sheet.

Product	Transmissivity (gal/min. ft)	Transmissivity (m²/sec)
TexDrain 200 SS 6	0.97	2×10^{-4}
TexDrain 200 SS 8	0.48	1×10^{-4}
TexDrain 200 DS 6	0.48	1×10^{-4}
TexDrain 200 DS 8	0.19	4×10^{-5}
TexDrain 250 SS 6	2.17	4.5×10^{-4}
TexDrain 250 SS 8	1.2	2.5×10^{-4}
TexDrain 250 DS 6	1.44	3×10^{-4}
TexDrain 250 DS 8	0.84	1.75×10^{-4}

The above values, unless otherwise specified, are the minimum acceptable average test results for any roll based on the specified test methods and do not refer to an individual test specimen. The data provided is for informational purposes only and is not intended as a warranty or guarantee. Values are subject to change without notice. Rev. 4/04

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1500 West Shure Drive
Arlington Heights, Illinois 60004 U.S.A. www.cetco.com
PH 847.392.5800 800.527.9948 FX 847.577.5566 LTinfo@cetco.com

Foundation Evaluation

- Subgrade Bearing Capacity
- Subgrade Settlement Evaluation

Foundation Evaluation

- Discussion of Foundation Evaluation and Conclusions
- List of References
- Bearing Capacity
- Settlement Analysis BC-10
- Settlement Analysis BC-2
- Copies of References
- Logs for Borings Inside the Footprint

Discussion of Foundation Evaluation and Conclusions

A geologic and hydrogeologic investigation was performed at the Belews Creek FGD Residue landfill site. The results of this investigation are included in Reference 1. This investigation included field tests and laboratory tests to determine properties of interest of the soils and other materials underlying the site soils.

As described in Reference 1, the site is typical of the Piedmont physiographic province. The site is underlain by gneiss, a crystalline bedrock, overlain by a variable thickness of partially weathered rock, or saprolite. This layer typically grades into a more weathered material termed residuum.

The excavation for the landfill will place the final grade at the elevations shown on the cross sections found in Reference 1 and on drawing C-2.

The following pages present the result of the foundation evaluation. This evaluation is performed to determine the structural integrity of the subgrade to support the loads and stresses imposed by the weight of the material placed in the landfill.

As shown on drawing C-2, the landfill footprint will be primarily located in excavated material. The exception to this is an area in the northwest corner of the landfill. The width of this fill area is about 220' at its maximum width (measured from the western edge of the footprint) approximately 120' north of BC-2 to approximately 40 wide at roughly 220' feet south of BC-2.

Material Beneath the Landfill Subgrade

Table 1 presents information on the SPT blow counts and the thickness of material above auger refusal for the 10 borings within the footprint. This table and a review of the boring logs show that the excavated area is in sound material that will not be subject to excessive settlement or experience other undesirable responses due to the loads imposed by the FGD waste.

Evaluation of the Bearing Capacity of the Landfill Subgrade

An evaluation of the bearing capacity of the landfill subgrade material was performed and is presented on the attached pages. This evaluation considered the ultimate bearing capacity of the landfill subgrade, determined from SPT blow count, and the applied loads from the waste placement. The minimum ratio of applied load to the bearing capacity was 3.0, with the average ratio of approximately 5. The bearing capacity of the landfill subgrade is adequate for the applied load.

Settlement Analysis

The settlement analysis of the landfill subgrade was performed and is presented on the attached pages. Based on a review of the boring logs and on the loading applied from the waste, the maximum settlement will occur at (or near) BC-10. The settlement at this location is calculated to be 0.11'.

Since the area near BC-2 will be constructed of fill material, the settlement in this area was also evaluated. The calculated settlement at BC-2 is 0.18'.

Settlement Analysis (continued)
The following calculations evaluate the impact of these settlements at these locations and the associated strain on the geomembrane liner.

BC-10 to BC-2:
Distance between borings (approx) = 310'

Settlement at BC-10 =	0.11'
- Settlement at BC-2 =	0.18'
Differential	0.07'

Calculate slope from BC-10 to BC-2:

0.07 feet/ 310 feet = 0.0002 ft/ft = 0.02%

This will result in negligible strain.

BC-10 to BC-16:
Assume no settlement at BC-16 to check slope (and strain) between BC-10 and BC-16.

Distance between borings (approx) = 305'

Settlement at BC-10 =	0.11'
- Settlement at BC-16 =	0.0'
Differential	0.11'

Calculate slope from BC-10 to BC-16:

0.11 feet/ 305 feet = 0.0004 ft/ft = 0.04%

This will result in negligible strain.

Check Slope From BC-2 to Anchor Trench
Assume no settlement at anchor trench.

Distance from BC-2 to anchor trench (approx) = 30'

Settlement at BC-2 = 0.18'

Calculate slope from BC-2 to anchor trench:

0.18'/30 ft = 0.006 ft/ft = 0.6%

This will result in negligible strain.

Settlement Analysis (continued)
Therefore the settlements of the subgrade are acceptable.

Conclusion
A review was performed of the geotechnical information for the landfill soils and subgrade. Evaluations were performed of the bearing capacity of the landfill subgrade and the anticipated settlements of the landfill subgrade. Based on these evaluations, the foundation of the landfill was found to be acceptable.

Table 1 - Foundation Evaluation - Information From Boring Logs

	Boring	Existing Ground Elev. (ft)	Elevation Subgrade (ft)	Depth to Auger Refusal (AR) (bgs) (ft)	Elevation of AR (ft)	Thickness of Material Below Grade & Above AR	Thickness of Material (ft) Below Grade & Above N ≥50	Minimum SPT Blow Count Below Grade	Comments (on material below grade and above auger refusal)
BC	2	764.74	773	43.5	721.2	51.8	22	17	mainly SM and ML (12' of fill to grade)
BC	3	842.34	834	38.0	804.3	27.7	9	25	mainly MH/SM
BC	9	773.54	767	13.8	759.7	7.3	0	50/5"	mainly SP/SM
BC	10	781.84	772	38.5	743.3	28.7	8	18	mainly sand, layer where N=18 is silt
BC	11	785.15	765	67.9	717.3	47.8	0	21	N = 50+ below layer where N=31
BC	12	802.91	768	62.8	740.1	27.9	3	31	sand and silt
BC	13	812.06	792	25.0	787.1	4.9	0	50/2"	4' until coring began
BC	16	833.77	772	68.3	765.5	6.5	0	N > 50	N = 50+ below grade
BC	18	780.97	766.6	38.3	742.7	23.9	0	50/3"	N = 50+ below grade
BC	19	813.09	787	57.2	755.9	30.7	7	20	Thin layer (approx. 10') of silty clayey sand saprolite. Below this layer, N>50.

Belews Creek FGD Residue Landfill
Foundation Evaluation

Job No.	**46119**		
Comp. By	**WMM**	**Date**	**9/20/2005**
Chkd. By	DEB	**Date**	9/28/05
Sheet	5	**of**	13

Objective: Evaluate subgrade settlement due to placement of waste.

References: 1) *Geologic and Hydrologic Siting Report*
FGD Scrubber Residue Disposal Site
Belews Creek Steam Station
3195 Pine Hall Road
Belews Creek, North Carolina
S&ME Project 1054-04-955
February 2005

2) *FGD Scrubber Sludge Testing (Revision 1),*
Power Coal Fired Steam Stations in North Carolina,
Duke S&ME Project No. 1264-03-57, February 2004.

3) FHWA Settlement Analysis Method by U.S. Dept. of Transportation, publication No. FHWA-HI-88-005.

4) *Department of The Navy, Soil Mechanics Design Manual 7.1, NAVFAC DM-7.1* May 1982.

5) *Physical and Geotechnical Properties of Soil, Jospeh E. Bowles, 2nd Edition,*

6) Principles of Foundation Engineering, 2nd Edition, Das.

Belews Creek FGD Residue Landfill
Foundation Evaluation
Bearing Capacity

Job No.	**046119**		
Comp. By	**WMM**	**Date**	**9/20/2005**
Chkd. By	DEA	**Date**	9/26/05
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Landfill Loads

	Boring	Existing Ground Elev. (ft)	Proposed Subgrade Elev. (ft) (approx.)	Proposed Cut (ft)	Proposed Final Elev. (ft) (approx.)	Height of Material Over Subgrade ΔH (ft)	Load From Landfill (ksf)
BC -	2	764.74	773	(Fill 8.3')	779	6	1.7
BC -	3	842.34	834	8.34	846	12	1.2
BC -	9	773.54	767	6.54	814	47	4.3
BC -	10	781.84	772	9.84	840	68	6.2
BC -	11	785.15	765	20.15	816	51	4.6
BC -	12	802.91	768	34.91	826	58	5.3
BC -	13	812.06	792	20.06	853	61	5.5
BC -	16	833.77	772	61.77	855	83	7.5
BC -	18	780.97	766.6	14.37	839	72.4	6.6
BC -	19	813.09	787	26.09	832	45	4.1

Unit weight of :

site soil (assumed)	125	lb/ft^3
gypsum	89	lb/ft^3

Thickness of cover soil

over liner	1	ft
over cover	2	ft

Belews Creek FGD Residue Landfill
Foundation Evaluation
Bearing Capacity

Job No.	046119		
Comp. By	WMM	**Date**	9/20/2005
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Bowles (Reference 5) gives the following equation for the ultimate bearing pressure on cohesionless soils:

q_{ult} = 30 N (kPa) (Eqn 15-16)

where N = SPT for layer

using the following conversion: kPa = 20.8 lb/ft²
then kPa = 0.0208 ksf

	Boring	Proposed Final Elev. (ft) (approx.)	Height of Material Over Subgrade ΔH (ft)	Load From Landfill (ksf)	SPT Blow Count Below Grade	Ultimate Bearing Capacity (ksf)	Ratio: Applied Load to Ultimate Bearing Capacity
BC -	2	779	6	1.7	17	10.6	6.3
BC -	3	846	12	1.2	25	15.6	13.3
BC -	9	814	47	4.3	50	31.2	7.3
BC -	10	840	68	6.2	70	43.6	7.1
BC -	11	816	51	4.6	85	53.1	11.4
BC -	12	826	58	5.3	31	19.3	3.7
BC -	13	853	61	5.5	50	31.2	5.6
BC -	16	855	83	7.5	50	31.2	4.2
BC -	18	839	72.4	6.6	50	31.2	4.8
BC -	19	832	45	4.1	70	43.5	10.6

Belews Creek FGD Residue Landfill
Settlement Analysis - BC-10

Job No.	**046119**		
Comp. By	**WMM**	**Date**	**9/20/2005**
Chkd. By	DEB	**Date**	9/30/05
Sheet	8	**of**	13

Calculate Settlement at BC-10

Step 1 Stresses are calculated at the point of interest.

Step 2 Settlement is then calculated according the procedure in described in the FHWA Settlement Analysis Method.

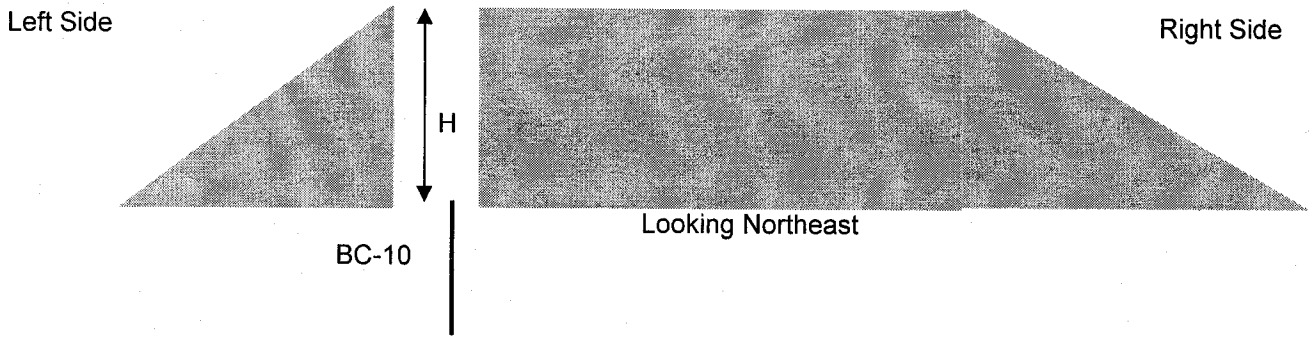
Residual Soil assume unit wt =	125 pcf	Gypsum unit wt =	89 pcf
Height of fill over subgrade,	68 ft	Unit Weight Cover Soil = (Assume 3' thick)	125 pcf
Depth to Water Table	28.9 feet		

The increase in stress due to the landfill loads is:

$\Delta P(psf)$ = unit weight of materail * H

$\Delta P(psf)$ = 6427 psf

Depth of Subgrade Below Existing Ground	10 feet



Belews Creek FGD Residue Landfill
Settlement Analysis - BC-10

Job No.	**046119**		
Comp. By	**WMM**	**Date**	**9/20/2005**
Chkd. By	DEJ	**Date**	9/30/05
Sheet	9	**of**	13

Calculate stresses at BC-10

P'o = effective stress at center of layer

Layer #	Depth Below Existing Ground to Bottom of Layer (ft)	Depth below Subgrade to Bottom of Layer (ft)	thickness of layer (ft)	z, depth to middle of layer (ft)	P'o at top of layer (psf)	P'o at bottom of layer (psf)	Po' @midlayer (psf)	ΔP(psf)	Po' + ΔP / Po' (psf)	log Po' + ΔP / Po'
1	15	5	5	2.5	0	625	313	6427.0	21.6	1.33
2	20	10	5	7.5	625	1250	938	6427.0	7.9	0.90
3	25	15	5	12.5	1250	1875	1563	6427.0	5.1	0.71
4	28.9	18.9	3.9	17.0	1875	2363	2119	6427.0	4.0	0.61
5	33.9	23.9	5	21.4	2362.5	2676	2519	6427.0	3.6	0.55
6	38.5	28.5	4.6	26.2	2675.5	2963	2819	6427.0	3.3	0.52

CN, Correction Factor for SPT Count Das Eqn 2.6

Cn = (1/Po') ^ 0.5
Po' is in tons/sf

Correct SPT Results for Effective Stresses:

Layer Number	Depth Below Existing Ground to Bottom of Layer (ft)	Depth below Subgrade to Bottom of Layer (ft)	Description of Soil used to Determine C'	SPT Blow Count, N, for Layer	CN, Correction Factor for SPT Count	Corrected SPT, N'	Bearing Capacity Index, C'
1	15	5	silty sand	18	2.5	46	120
2	20	10	sandy clay	50	1.5	73	153
3	25	15	fine to coarse sand	50	1.1	57	180
4	28.9	18.9	silty sand	50	1.0	49	148
5	33.9	23.9	silty sand	50	0.9	45	118
6	38.5	28.5	silty sand	50	0.8	42	114

Belews Creek FGD Residue Landfill
Settlement Analysis - BC-10

Job No.	**046119**		
Comp. By	**WMM**	**Date**	**9/20/2005**
Chkd. By	**DEB**	**Date**	**9/30/05**
Sheet	**10**	**of**	**13**

Calculate Settlement:

Settlement will be calculated for each layer by the following equation:

$$\Delta H = H * 1/C' * \text{Log} \frac{Po' + \Delta P}{Po'} \qquad \text{(Reference 3)}$$

Layer Number	H, layer thickness (ft)	SPT Blow Count, N, for Layer	C_N, Correction Factor for SPT Count	Corrected SPT, N'	Bearing Capacity Index, C'	$\log \frac{Po' + \Delta P}{Po'}$	ΔH (ft)
1	5	18	2.5	46	120	1.33	0.06
2	5	50	1.5	73	153	0.90	0.03
3	5	50	1.1	57	180	0.71	0.02
4	3.9	50	1.0	49	148	0.61	0.02
5	5	50	0.9	45	118	0.55	0.02
6	4.6	50	0.8	42	114	0.52	0.02

Settlement **0.16** **ft**

Multiply the calculated settlement by 2/3:

Corrected Settlement **0.11** **ft**

Belews Creek FGD Residue Landfill
Settlement Analysis BC-2

Job No.	**046119**		
Comp. By	**WMM**	**Date**	**9/20/2005**
Chkd. By	DEB	**Date**	9/28/05
Sheet	11	**of**	13

Calculate Settlement at BC-2

Step 1 Stresses are calculated at the location of interest.

Step 2 Settlement is then calculated according the procedure in described in the FHWA Settlement Analysis Method.

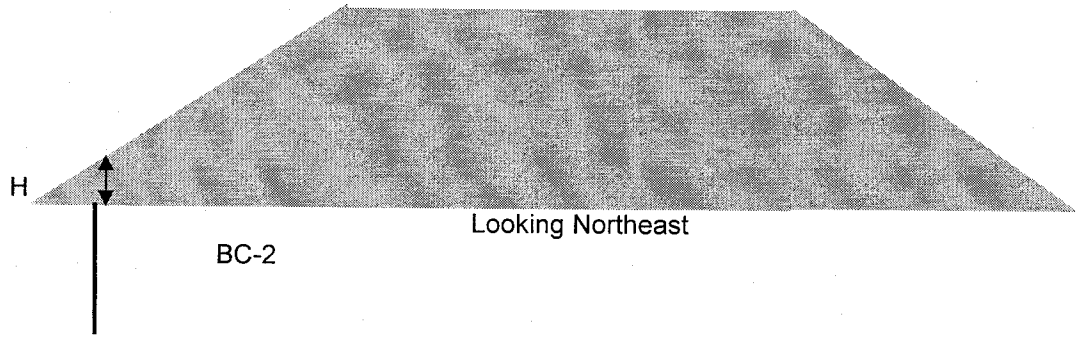
Residual Soil assume unit wt =	125 pcf	Gypsum unit wt =	89 pcf	
Height of fill over subgrade, H =	6 ft	Depth of Subgrade Below Existing Ground	8.3 feet	FILL
Depth to Water Table	17 feet			
Unit weight Gypsum, γ	87 lb/ft^3			

The increase in stress due to the landfill loads is:

$\Delta P(psf)$ = unit weight of material * H

$\Delta P(psf)$ = 2717.0 psf

Note: the ΔP at this location includes the fill materail used to bring this area up to grade.



Assume that placed fill will have SPT blow count equal to uppermost existing ground layer.

Belews Creek FGD Residue Landfill
Settlement Analysis BC-2

Job No.	046119		
Comp. By	WMM	**Date**	9/20/2005
Chkd. By	DEB	**Date**	9/28/05
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Calculate stresses at BC-2

P'o = effective stress at center of layer

Layer #	Depth Below Existing Ground to Bottom of Layer (ft)	Depth below Subgrade to Bottom of Layer (ft)	thickness of layer (ft)	z, depth to middle of layer (ft)	P'o at top of layer (psf)	P'o at bottom of layer (psf)	Po' @midlayer (psf)	ΔP (psf)	Po' + ΔP / Po' (psf)	log Po' + ΔP / Po'
1	(Fill)	8.3	8.3	4.2	0	1038	519	2717.0	6.24	0.80
2	4.5	12.8	4.5	10.6	1037.5	1600	1319	2717.0	3.06	0.49
3	10	18.3	5.5	15.6	1600	2206	1903	2717.0	2.43	0.39
4	19.5	27.8	9.5	23.1	2206.38	2801	2504	2717.0	2.09	0.32
5	43.5	51.8	24	39.8	2801.08	4303	3552	2717.0	1.76	0.25

Correct SPT Results for Effective Stresses:

CN, Correction Factor for SPT Count Das Eqn 2.6

Cn = (1/Po') ^ 0.5
Po' is in tons/sf

Layer Number	Depth Below Existing Ground to Bottom of Layer (ft)	Depth below Subgrade to Bottom of Layer (ft)	Description of Soil used to Determine C'	Po' @midlayer (psf) (see Note)	SPT Blow Count, N, for Layer	CN, Correction Factor for SPT Count	Corrected SPT, N'	Bearing Capacity Index, C'
1	(Fill)	8.3	silty sand	See Note	17	1.2	21	55
2	4.5	12.8	silty sand	281.25	17	1.2	21	55
3	10	18.3	silty sand	865.69	29	1.0	30	70
4	19.5	27.8	silty sand	1466.23	48	0.9	43	117
5	43.5	51.8	silty sand	2514.78	50	0.8	38	105

Note: Effective stress used to correct the blow count the stress at time of investigation. The correction factor for Layer 2 is then used for Layer 1, since these blow counts are assumed to be equal to those in Layer 2.

Belews Creek FGD Residue Landfill
Settlement Analysis BC-2

Job No.	**046119**		
Comp. By	**WMM**	**Date**	**9/20/2005**
Chkd. By	DEB	**Date**	9/28/05
Sheet	13	**of**	13

Calculate Settlement:

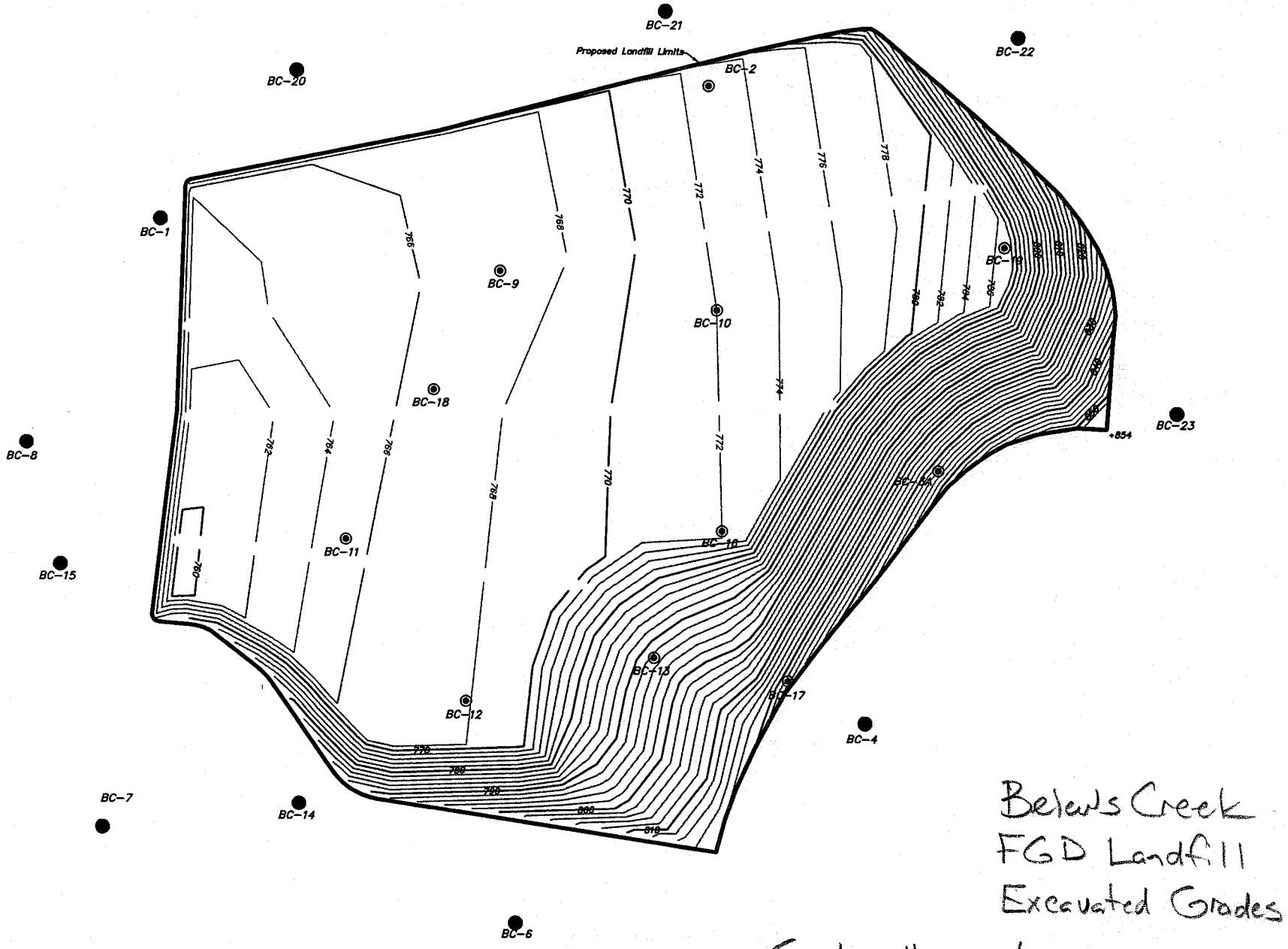
Settlement will be calculated for each layer by the following equation:

$$\Delta H = H * 1/C' * \text{Log} \frac{Po' + \Delta P}{Po'}$$ (Reference 3)

Layer Number	H, layer thickness (ft)	SPT Blow Count, N, for Layer	C_N, Correction Factor for SPT Count	Corrected SPT, N'	Bearing Capacity Index, C'	log (Po' + ΔP)/Po'	ΔH (ft)
1	8.3	18	2.0	35	55	0.80	0.12
2	4.5	50	1.2	62	55	0.49	0.04
3	5.5	50	1.0	51	70	0.39	0.03
4	9.5	50	0.9	45	117	0.32	0.03
5	24	50	0.8	38	105	0.25	0.06
						Settlement	**0.27** ft

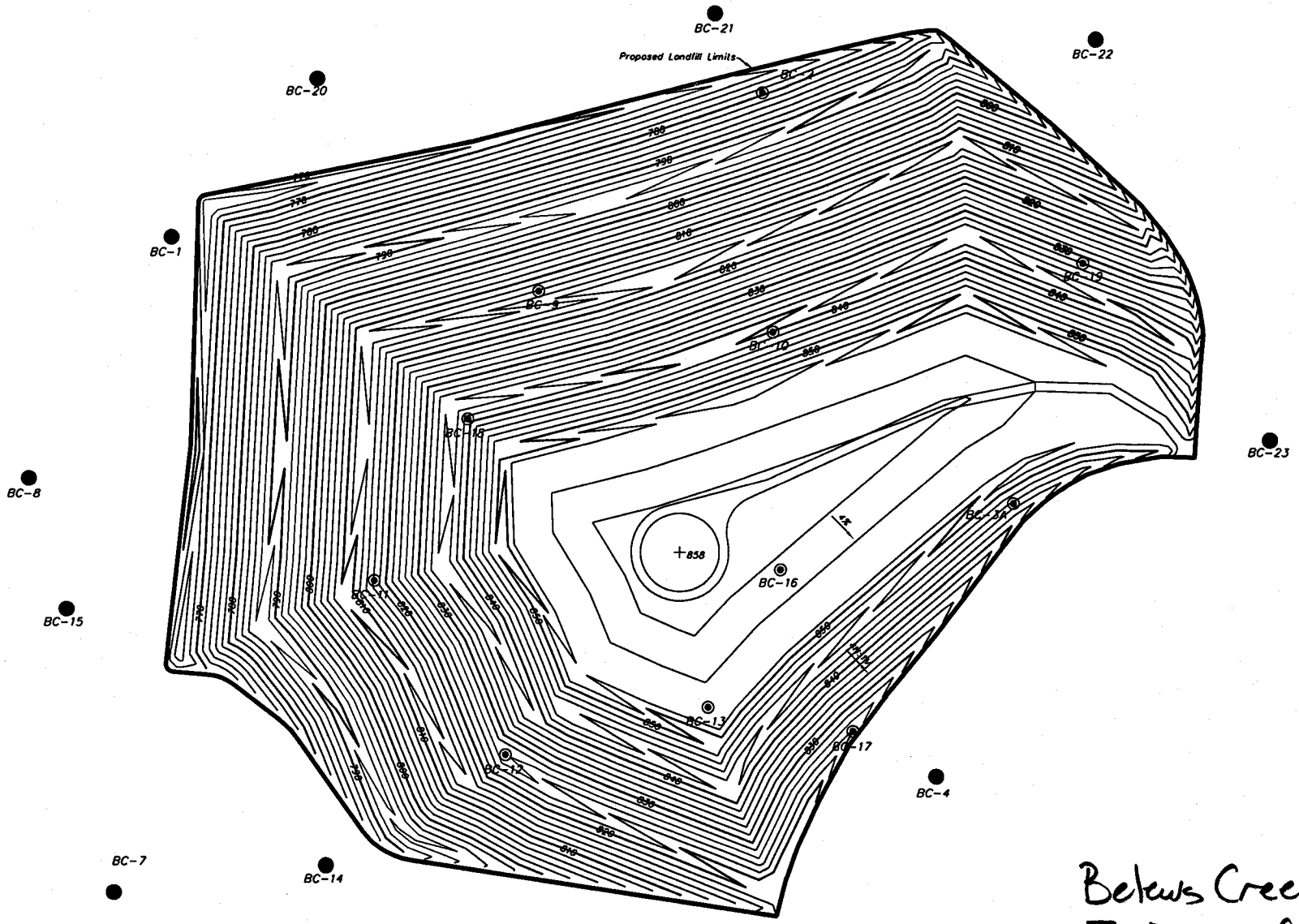
Multiply the calculated settlement by 2/3:

Corrected Settlement **0.18** ft



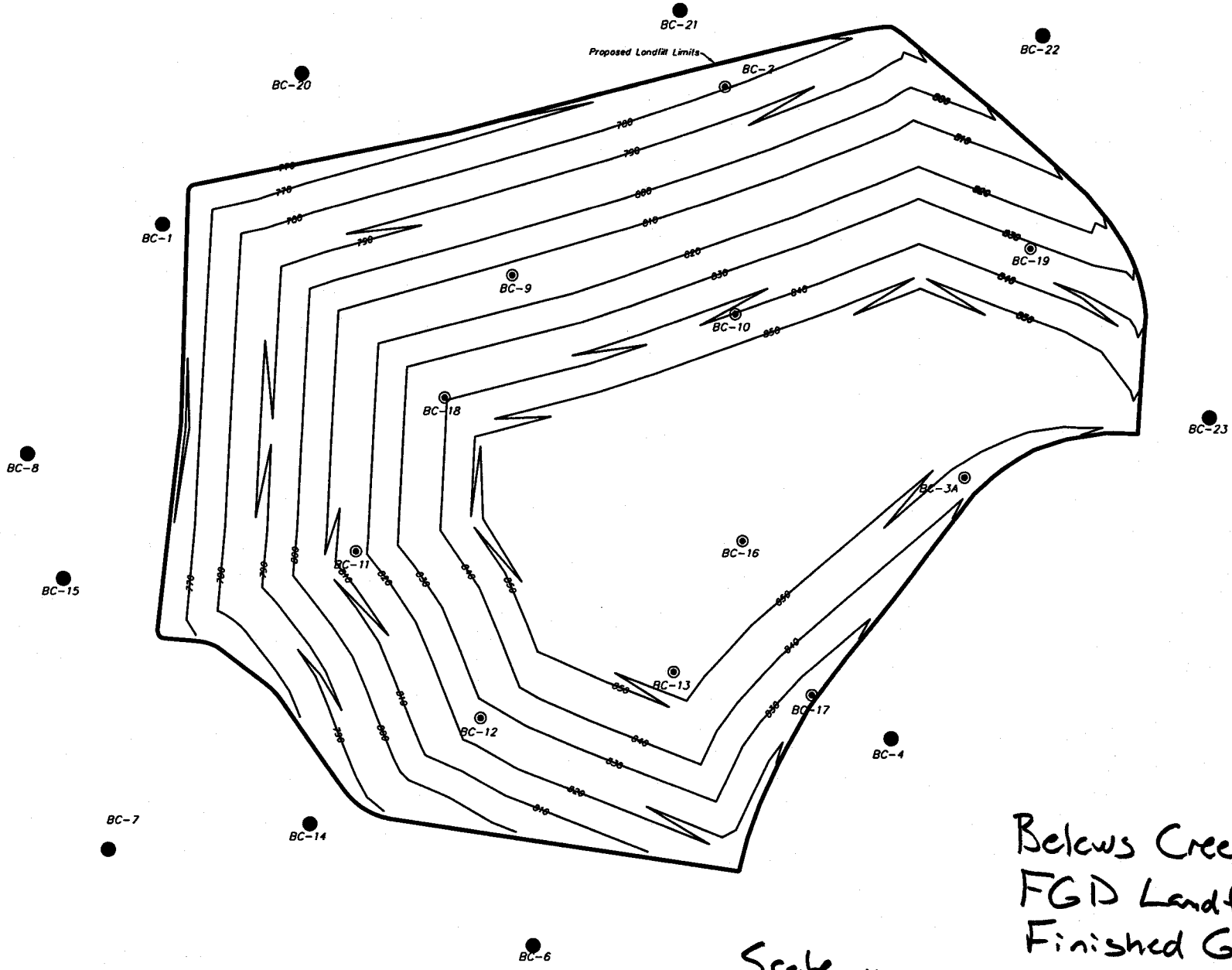
Belews Creek
FGD Landfill
Excavated Grades

Scale 1" = 200'



Bekews Creek
FGD Landfill
Finished Grades

Scale
1" = 200'



Belews Creek
FGD Landfill
Finished Grades

Scale 1" = 200'

For mat foundations which have a very large B,

$$q_a = \frac{N}{F_2} K_d \tag{15-15}$$

where terms are as defined in Eq. (15-14). For settlements other than 25 mm, assume a linear variation in q_a for both Eqs. (15-14) and (15-15).

Parry (1977) suggested that the ultimate bearing pressure on cohesionless soils using the SPT number N could be computed as

$$q_{ult} = 30N \qquad \text{(kPa)} \tag{15-16}$$

The cone penetration test (CPT) can be used in cohesionless soils according to Meyerhof (1956) to obtain

$$\begin{aligned} q_a &= \frac{q_c}{30} && B \le F_4 \\ q_a &= \frac{q_c}{50}\left(\frac{B + F_3}{B}\right)^2 && B > F_4 \end{aligned} \tag{15-17}$$

where q_c = cone resistance, kPa or k/ft^2, and other terms as defined in Eq. (15-14).

In cohesive soils, the undrained shear strength s_u can be estimated according to Begemann (1974) as

$$s_u = \frac{q_c - p_o}{N_c'} \tag{15-18}$$

where p_o = effective overburden pressure at CPT depth
N_c' = constant ranging from 5 to 70 depending on deposit and OCR (Values ranging from 9 to 15 are most common.)

Since $s_u = c$, we can use Eqs. (15-10) to obtain q_{ult}. This rough approximation would not justify the use of "refined" bearing capacity equations such as Eq. (15-11). The cone resistance q_c should be an average obtained in a zone similar to N of about 0 to 0.75B below the estimated footing depth.

Example 15-6 The average SPT blow count N in the effective zone beneath a footing is 15. Estimate the allowable bearing capacity q_a, assuming that the footing depth D is 3.3 m and the soil in the zone of influence is a medium dense, coarse sand.

SOLUTION Since q_a depends on B we will make a graph as in Fig. E15-6 of q_a versus B. We should also be aware that while the value of q_a should limit sand settlement to 25 mm, if there is a soft clay layer in close proximity which may consolidate, the total settlement will be more than 25 mm unless q_a is further reduced.

Reference 5

Reference 6

Several authors, such as Bazaraa (1967); Peck and Bazaraa (1969); Peck, Hanson, and Thornburn (1974); Tang (1962); Seed (1976, 1979); and Tokimatsu and Yoshimi (1983), have proposed empirical relationships for C_N. However, the simplest relationship is that proposed by Liao and Whitman (1986), which has given as good a result as any other. This relationship can be given as

$$C_N = \sqrt{\frac{1}{\sigma'_v}} \tag{2.5}$$

where σ'_v is in (U.S.) tons/ft^2. In SI units

$$C_N = 9.78\sqrt{\frac{1}{\sigma'_v\ (\text{kN/m}^2)}} \tag{2.6}$$

In Eqs. (2.5) and (2.6), the standard value of σ'_v is 95.6 kN/m^2 (1 ton/ft^2). Figures 2.8 and 2.9 show the plot of C_N against σ'_v.

An approximate relation between the standard penetration number and the relative density of sand is given in Table 2.4.

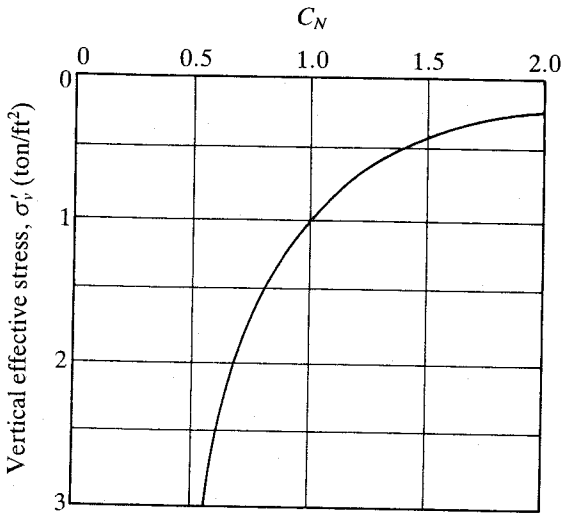


Figure 2.8 Variation of C_N with σ'_v [Eq. (2.5)]

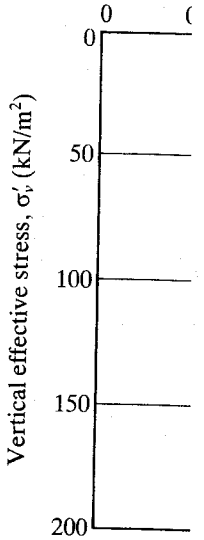


Figure 2.9 Variatic

Table 2.4 Relation
Density, and Angle o

Standard penetration number, N	Appr re den
0–5	
5–10	
10–30	3
30–50	6

Note: See also Figur

When the sta
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following should l

1. The equat
2. Because th
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3. In soil de
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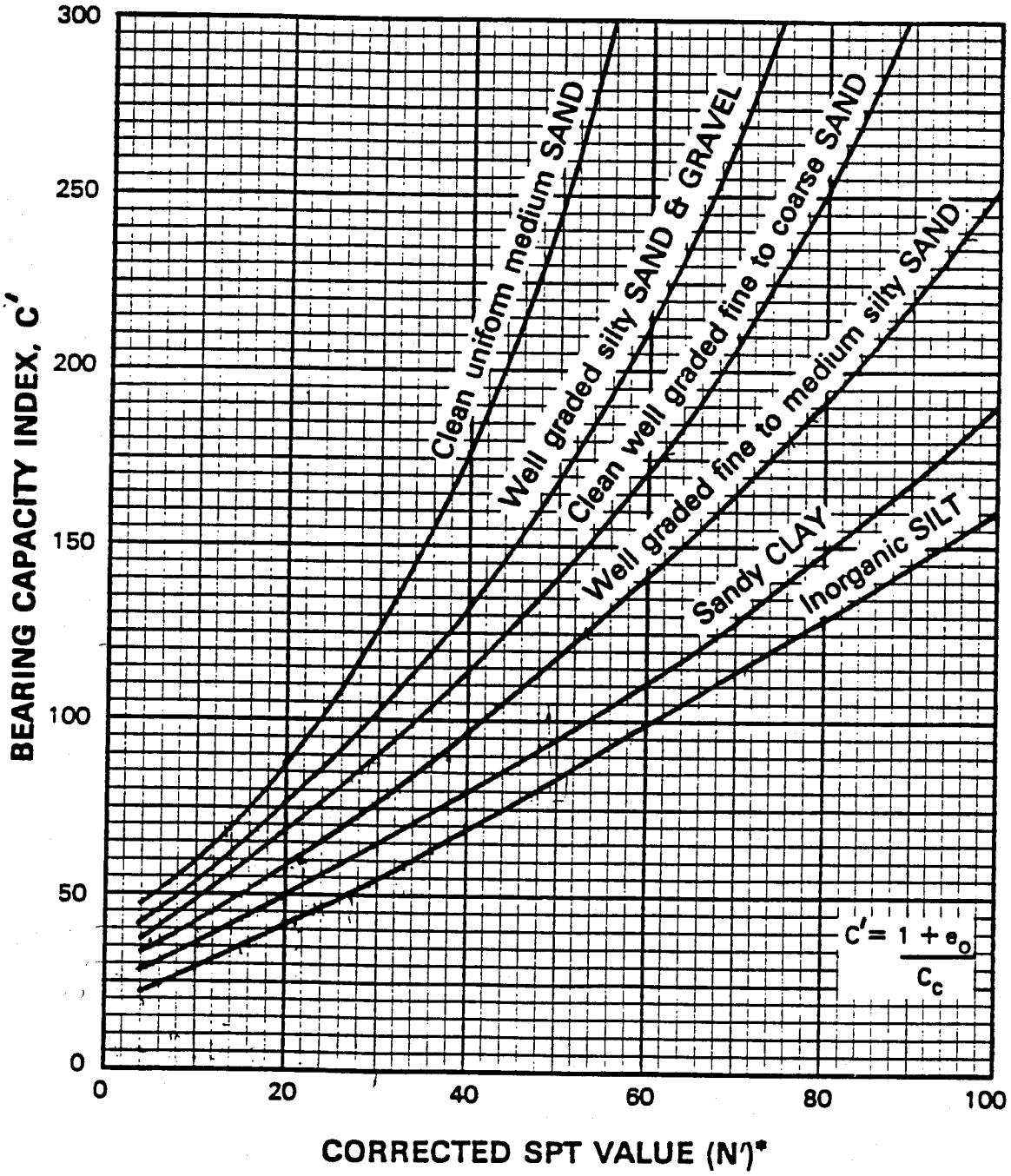
From FHWA-HI-88-005



*N'—*SPT (N) Value Corrected for Overburden Pressure.*

Reference: Hough, "Compressibility as a Basis for Soil Bearing Value" ASCE 1959

BORING LOG BC-2

PROJECT:	**Duke Power-Belews Creek**	DRILLING CONTRACTOR:	**S&ME, Inc.**
LOCATION:	**Belews Creek, NC**	DRILLING METHOD:	**3¼" H.S.A.**
PROJECT NO.:	**1264-02-576**	DRILL RIG:	**Mobile B-57**
LOGGED BY:	**S. Watts**	TOP OF CASING ELEVATION (ft):	**766.99**
DATE DRILLED:	**11/20/2002**	WATER LEVEL TOB (ft bgs):	**19.75**
BORING DEPTH (ft):	**43.5**	WATER LEVEL 24 HRS (ft bgs):	**16.97**

Depth (feet)	Sample Number	Sample Interval (feet)	Blow Counts per 6 inches	Recovery (inches)	Rock RQD (%)	Description & Remarks	Well Construction Details	Elevation (feet)
	S-1	0.0-10.0	–	18		Crush and run GRAVEL.	**Protective Casing** Dia: 4.0 in. Type: Stick-Up Interval: 2.25 ft.	765
	SS-1	0.5-2.0	3-2-1					
	SS-2	3.5-4.5	4-6-11	12		Moderate yellow brown to light brown very fine sandy SILT (ML), A-horizon material, slightly moist.		
5						Light brown to orangish brown slightly micaceous very fine sandy SILT (MH/ML), slightly moist.	**Riser Casing** Dia: 2.0 in. Type: Sch 40 PVC Interval: 0.0 - 27.28 ft.	760
	SS-3	8.5-10.0	10-15-14	18		Grayish orange to pale yellow orange fine silty SAND (SM) w/relict layering structures, loose and crumbly, slightly moist.		
10	S-2	10.0-20.0	–				**Grout** Type: Portland Cement Interval: 0.0 - 18.0 ft.	755
	SS-4	13.5-15.0	17-27-21	18		• No Cut at this location • Fill of ~8.3'	**Seal** Type: Bentonite Chips Interval: 18.0 - 23.0 ft.	
15							**Filterpack** Type: Filter Sand #2 Interval: 23.0 - 42.28 ft.	750
	SS-5	18.519.5	18-27-24	12				
20	S-3	20.0-30.0	–				**Screen** Dia: 2.0 in. Type: Sch 40 PVC Interval: 27.28 - 42.28	745
	SS-6	23.5-23.7	50/2–	2		Grayish orange to pale yellow orange silty SAND (SM) (saprolite), crumbly. Very hard layer identified at 36.5 ft.		
25								740
	SS-7	28.5-28.7	50/2--	2				
30	S-4	30.0-40.0	–					735
	SS-8	33.5-33.7	50/2--	2				

NOTES

SS = Split Spoon Sample
ST = Shelby Tube Sample
S = Bulk Sample
R = Core Run
RS = Rock Sample
REC = Rock Core Recovery
RQD = Rock Quality Designation

1. ▽ = Water Level At Termination of Boring
2. ▼ = Water Level After 24 Hours

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BORING LOG BC-2

PROJECT:	**Duke Power-Belews Creek**	DRILLING CONTRACTOR:	**S&ME, Inc.**
LOCATION:	**Belews Creek, NC**	DRILLING METHOD:	**3¼" H.S.A.**
PROJECT NO.:	**1264-02-576**	DRILL RIG:	**Mobile B-57**
LOGGED BY:	**S. Watts**	TOP OF CASING ELEVATION (ft):	**766.99**
DATE DRILLED:	**11/20/2002**	WATER LEVEL TOB (ft bgs):	**19.75**
BORING DEPTH (ft):	**43.5**	WATER LEVEL 24 HRS (ft bgs):	**16.97**

Sample Information						Symbol	Description & Remarks	Well Construction Details	Well Construction Graphic	Elevation (feet)
Depth (feet)	Sample Number	Sample Interval (feet)	Blow Counts per 6 inches	Recovery (inches)	Rock RQD (%)					
							Grayish orange to pale yellow orange silty SAND (SM) (saprolite), crumbly. Very hard layer identified at 36.5 ft. *(continued)*	**Protective Casing** Dia: 4.0 in. Type: Stick-Up Interval: 2.25 ft. **Riser Casing** Dia: 2.0 in. Type: Sch 40 PVC Interval: 0.0 - 27.28 ft.		730
	SS-9	38.5-38.6	50/.5--	0.5						
40										725
							Auger Refusal - Boring terminated at 43.5 ft.	**Grout** Type: Portland Cement Interval: 0.0 - 18.0 ft. **Seal** Type: Bentonite Chips Interval: 18.0 - 23.0 ft. **Filterpack** Type: Filter Sand #2 Interval: 23.0 - 42.28 ft. **Screen** Dia: 2.0 in. Type: Sch 40 PVC Interval: 27.28 - 42.28		

NOTES

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BORING LOG BC-3

PROJECT:	**Duke Power-Belews Creek**	DRILLING CONTRACTOR:	**S&ME, Inc.**
LOCATION:	**Belews Creek, NC**	DRILLING METHOD:	**4¼" H.S.A.**
PROJECT NO.:	**1264-02-576**	DRILL RIG:	**Mobile B-57**
LOGGED BY:	**S. Watts**	TOP OF CASING ELEVATION (ft):	**844.84**
DATE DRILLED:	**11/12/2002**	WATER LEVEL TOB (ft bgs):	
BORING DEPTH (ft):	**48**	WATER LEVEL 24 HRS (ft bgs):	

Depth (feet)	Sample Number	Sample Interval (feet)	Blow Counts per 6 inches	Recovery (inches)	Rock RQD (%)	Description & Remarks	Well Construction Details
	S-1	0.0-10.0	--	12		Reddish brown clayey SILT (ML/MH), slightly plastic, moist.	Protective Casing Dia: 4.0 in. Type: Stick-Up Interval: 2.50 ft.
	SS-1	0.5-3.5	7-6-6			Light red to pink micaceous very fine sandy SILT (MH/SM) w/relict structures visible (saprolite).	
	SS-2	3.5-5.0	9-14-15	12			
5							Riser Casing Dia: 2.0 in. Type: Sch 40 PVC Interval: 0.0 - 30.07 ft.
	SS-3	8.5-10.0	11-12-13	10			
10	S-2	10.0-20.0	Grade			Light red to pink micaceous very fine sandy SILT (MH/SM) w/layering visible (saprolite).	Grout Type: Portland Cement Interval: 0.0 - 27.0 ft.
	SS-4	13.5-15.0	14-16-20	8			Seal Type: Bentonite Chips Interval: 27.0 - 29.0 ft.
15						Light red to pink very fine silty SAND (MH/SM) w/Mg staining along relict fractures and quartz and biotite crystals (saprolite), crumbly, dry.	Filterpack Type: Filter Sand #2 Interval: 29.0 - 48.0 ft.
	SS-5	18.5-19.4	36-28-50/4	12			
20	S-3	20.0-30.0	--			Moderate red to moderate reddish brown micaceous silty SAND (MH/SM) w/dark brown and pale yellow orange streaks (saprolite), crumbly, dry.	Screen Dia: 2.0 in. Type: Sch 40 PVC Interval: 30.07 - 45.07 ft.
	SS-6	23.5-25.0	50/3--	3			
25							
	SS-7	28.5-30.0	16-15-14	18		White silty SAND (MH/SM), mineral rich layer (feldspar/albite).	
30	SS-8	30.0-40.0	50/3--	3		Moderate red to light red micaceous silty fine SAND (MH/SM) w/layering and rock fragments (saprolite), crumbly, dry.	
	SS-9	33.5-35.0	50/0--	0			

Well Construction Graphic

Elevation (feet): 840, 835, 830, 825, 820, 815

NOTES

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BORING LOG BC-3

PROJECT: **Duke Power-Belews Creek**
LOCATION: **Belews Creek, NC**
PROJECT NO.: **1264-02-576**
LOGGED BY: **S. Watts**
DATE DRILLED: **11/12/2002**
BORING DEPTH (ft): **48**

DRILLING CONTRACTOR: **S&ME, Inc.**
DRILLING METHOD: **4¼" H.S.A.**
DRILL RIG: **Mobile B-57**
TOP OF CASING ELEVATION (ft): **844.84**
WATER LEVEL TOB (ft bgs):
WATER LEVEL 24 HRS (ft bgs):

Sample Information						Symbol	Description & Remarks	Well Construction Details	Well Construction Graphic	Elevation (feet)
Depth (feet)	Sample Number	Sample Interval (feet)	Blow Counts per 6 inches	Recovery (inches)	Rock RQD (%)					
							Moderate red to light red micaceous silty fine SAND (MH/SM/GM) w/granitic rock fragments and layering. Auger refusal at 38.0 ft. *(continued)*	**Protective Casing** Dia: 4.0 in. Type: Stick-Up Interval: 2.50 ft.		
	R-1	38.0-43.0	–	40	0		Highly weathered and highly fractured interlayered SCHIST and Quartz GRANITE. REC = 40% RQD = 0%	**Riser Casing** Dia: 2.0 in. Type: Sch 40 PVC Interval: 0.0 - 30.07 ft.		805
40										
	R-2	43.0-48.0	–	45	0		Highly weathered and highly fractured interlayered SCHIST and Quartz GRANITE. REC = 45% RQD = 0%	**Grout** Type: Portland Cement Interval: 0.0 - 27.0 ft.		800
45								**Seal** Type: Bentonite Chips Interval: 27.0 - 29.0 ft.		
							Boring terminated at 48.0 ft.	**Filterpack** Type: Filter Sand #2 Interval: 29.0 - 48.0 ft.		
								Screen Dia: 2.0 in. Type: Sch 40 PVC Interval: 30.07 - 45.07 ft.		

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BORING LOG BC-9

PROJECT: **Duke Power-Belews Creek**
LOCATION: **Belews Creek, NC**
PROJECT NO.: **1264-02-576**
LOGGED BY: **S. Watts**
DATE DRILLED: **11/21/2002**
BORING DEPTH (ft): **28.8**

DRILLING CONTRACTOR: **S&ME, Inc.**
DRILLING METHOD: **4¼" H.S.A.**
DRILL RIG: **Mobile B-57**
TOP OF CASING ELEVATION (ft): **775.54**
WATER LEVEL TOB (ft bgs):
WATER LEVEL 24 HRS (ft bgs): **21.5**

Depth (feet)	Sample Number	Sample Interval (feet)	Blow Counts per 6 inches	Recovery (inches)	Rock RQD (%)	Symbol	Description & Remarks	Well Construction Details	Well Construction Graphic	Elevation (feet)
	S-1	0.0-10.0	–	2			Crush and run GRAVEL (geotextile fabric).	Protective Casing Dia: 4.0 in. Type: Flush Interval:		
–	SS-1	0.0-0.4	50/5–				Moderate yellow brown to dark yellowish orange silty gravelly SAND (SP/SM) w/Mg staining along high angle fractures and weathered granite and quartz fragments, relict structures visible, crumbly. Auger refusal at 13.8 ft.			
	SS-2	3.5-5.0	50/5–	2			Grade	Riser Casing Dia: 2.0 in. Type: Sch 40 PVC Interval: 0.0 - 13.8 ft.		770
5										
	SS-3	8.5-10.0	50/5–	2				Grout Type: Portland Cement Interval: 0.0 - 6.5 ft.		765
10										
	SS-4	13.5-13.8	50/.5–	0				Seal Type: Bentonite Chips Interval: 6.5 - 12.5 ft.		
	R-1	13.8-18.8	–	34	0		Medium gray, tan, white, black micaceous weathered Quartz GNEISS, highly fractured, sandy silt fillings located within fractures. REC = 57% RQD = 0%			760
15								Filterpack Type: Filter Sand #2 Interval: 12.5 - 28.8 ft.		
	R-2	18.8-23.8	–	53	0		Gray green, medium brown, white, tan micaceous weathered Quartz GNEISS, highly fractured, saprolite located at 19.5 ft. REC = 88% RQD = 0%	Screen Dia: 2.0 in. Type: Sch 40 PVC Interval: 13.8 - 28.8 ft.		755
20										
	R-3	23.8-28.8	–	49	32		Light to medium gray, dark red, black, white, tan micaceous weathered Quartz GNEISS, moderately fractured, highly fractured from 26.8 - 28.8 ft. REC = 82% RQD = 22% Fracture at 26.50' : 80			750
25										
							Boring terminated at 28.8 ft.			

NOTES
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BORING LOG BC-10

PROJECT:	**Duke Power-Belews Creek**	DRILLING CONTRACTOR:	**S&ME, Inc.**
LOCATION:	**Belews Creek, NC**	DRILLING METHOD:	**4¼" H.S.A.**
PROJECT NO.:	**1264-02-576**	DRILL RIG:	**Mobile B-57**
LOGGED BY:	**J. Zschiedrich**	TOP OF CASING ELEVATION (ft):	**784.07**
DATE DRILLED:	**11/20/2002**	WATER LEVEL TOB (ft bgs):	**33.77**
BORING DEPTH (ft):	**47.5**	WATER LEVEL 24 HRS (ft bgs):	**28.21**

Depth (feet)	Sample Number	Sample Interval (feet)	Blow Counts per 6 inches	Recovery (inches)	Rock RQD (%)	Description & Remarks	Well Construction Details	Elevation (feet)
-	S-1	1.0-10.0	–	12		Crush and run gravel	Protective Casing Dia: 4.0 in. Type: Stick-Up Interval: 2.23 ft.	
-	SS-1	1.0-2.5	5-7-10			Tan to light brown sandy SILT w/some Mg staining with thin visible layers, slightly moist.		780
-	SS-2	3.5-5.0	11-16-24	10				
5							Riser Casing Dia: 2.0 in. Type: Sch 40 PVC Interval: 0.0 - 31.12 ft.	
-	SS-3	6.0-7.5	15-20-23	11		Tan to light brown sandy SILT w/some quartzite and some Mg staining with thin visible layers, slightly moist.		775
-	SS-4	8.5-10.0	16-23-36	9		Dark brown to orange silty fine SAND w/some quartz fragments and Mg/Fe staining and layers, moist.		
10	S-2	10.0-20.0	–			Grade	Grout Type: Portland Cement Interval: 0.0 - 26.5 ft.	
-	ST-1	12.0-14.0	–					770
-	SS-5	13.5-15.0	6-7-11	13		Dark brown to orange micaceous SILT (mica semischist), moderately soft.	Seal Type: Bentonite Chips Interval: 26.5 - 30.5 ft.	
15							Filterpack Type: Filter Sand #2 Interval: 30.5 - 47.5 ft.	765
-	SS-6	18.5-20.0	44-41-37	8		Light brown fine SAND w/quartz and mica fragments, dry.		
20	S-3	20.0-30.0	–				Screen Dia: 2.0 in. Type: Sch 40 PVC Interval: 31.12 - 46.12 ft.	760
-	SS-7	23.5-25.0	40-34-16	14		Partially weathered ROCK (Quartzite) w/some silty Sand, dry.		
25						Red and brown micaceous silty SAND, slightly moist.		
-	SS-8	28.5-28.9	50/4.4–	2		Reddish brown and gray Partially Weathered ROCK (Mica Schist), moist. Auger refusal at 38.5 ft.		755
30	S-4	30.0-38.5	–					750
-	SS-9	33.5-33.9	50/.35–	0				

NOTES

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BORING LOG BC-10

PROJECT:	**Duke Power-Belews Creek**	DRILLING CONTRACTOR:	**S&ME, Inc.**
LOCATION:	**Belews Creek, NC**	DRILLING METHOD:	**4¼" H.S.A.**
PROJECT NO.:	**1264-02-576**	DRILL RIG:	**Mobile B-57**
LOGGED BY:	**J. Zschiedrich**	TOP OF CASING ELEVATION (ft):	**784.07**
DATE DRILLED:	**11/20/2002**	WATER LEVEL TOB (ft bgs):	**33.77**
BORING DEPTH (ft):	**47.5**	WATER LEVEL 24 HRS (ft bgs):	**28.21**

Depth (feet)	Sample Number	Sample Interval (feet)	Blow Counts per 6 inches	Recovery (inches)	Rock RQD (%)	Symbol	Description & Remarks	Well Construction Details	Well Construction Graphic	Elevation (feet)
							Reddish brown and gray Partially Weathered ROCK (Mica Schist), moist. Auger refusal at 38.5 ft. *(continued)*	**Protective Casing** Dia: 4.0 in. Type: Stick-Up Interval: 2.23 ft.		745
	R-1	38.5-47.5	--	60	31		Light to dark gray, light brown weathered Mica SCHIST, moderately fractured. REC = 86% RQD = 31% Fractures at 38.83' : 28 39.15' : 28 39.4' : 20 39.44' : 25 39.65' : 20, 40 40.1' : 76 40.4' : 40, 25 41.0' : 70 41.1' : 20 42.0' : 25 42.4' : 24 43.0' : 20 43.5' : 55 43.9' : 20 44.8' : 72, 20 ,65 46.45' : 30 47.0' : 45 Boring terminated at 47.5 ft.	**Riser Casing** Dia: 2.0 in. Type: Sch 40 PVC Interval: 0.0 - 31.12 ft. **Grout** Type: Portland Cement Interval: 0.0 - 26.5 ft. **Seal** Type: Bentonite Chips Interval: 26.5 - 30.5 ft. **Filterpack** Type: Filter Sand #2 Interval: 30.5 - 47.5 ft. **Screen** Dia: 2.0 in. Type: Sch 40 PVC Interval: 31.12 - 46.12 ft.		740
40										
45										

NOTES

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BORING LOG BC-11

PROJECT:	**Duke Power-Belews Creek**	DRILLING CONTRACTOR:	**S&ME, Inc.**
LOCATION:	**Belews Creek, NC**	DRILLING METHOD:	**4¼" H.S.A.**
PROJECT NO.:	**1264-02-576**	DRILL RIG:	**Mobile B-57**
LOGGED BY:	**J. Zschiedrich**	TOP OF CASING ELEVATION (ft):	**786.85**
DATE DRILLED:	**11/21/2002**	WATER LEVEL TOB (ft bgs):	**53.3**
BORING DEPTH (ft):	**67.9**	WATER LEVEL 24 HRS (ft bgs):	**41.02**

Depth (feet)	Sample Number	Sample Interval (feet)	Blow Counts per 6 inches	Recovery (inches)	Rock RQD (%)	Description & Remarks	Well Construction Details	Elevation (feet)
	S-1	0.0-10.0	--	15		Moderate reddish brown clayey SILT w/organic material and few small Gravel (disturbed or fill material) (CL), slightly plastic, slightly moist, large root encountered at 4.5 ft.	**Protective Casing** Dia: 4.0 in. Type: Stick-Up Interval: 1.70 ft.	785
	SS-1	0.0-1.5	6-7-9					
	ST-1	1.0-2.9	--					
	SS-2	3.5-5.0	8-12-15	12				
5							**Riser Casing** Dia: 2.0 in. Type: Sch 40 PVC Interval: 0.0 - 51.45 ft.	780
	SS-3	8.5-10.0	8-10-13	16				
10	S-2	10.0-20.0	--				**Grout** Type: Portland Cement Interval: 0.0 - 41.0 ft.	775
	SS-4	13.5-15.0	23-13-11	14		Dark yellow orange slightly micaceous very fine sandy SILT (SM), dry.	**Seal** Type: Bentonite Chips Interval: 41.0 - 47.0 ft.	
15						Partially weathered ROCK (Mica Schist).		770
						Light brown to reddish orange very fine silty SAND (saprolite) (SM/SP), crumbly, dry.	**Filterpack** Type: Filter Sand #2 Interval: 47.0 - 67.9 ft.	
	SS-5	18.5-20.0	15-50/3- Grade	8		Moderate pink orange micaceous silty SAND (saprolite) (SM) w/white streaks, relict structures visible, high angle paleo fractures w/Mg staining, dry.	**Screen** Dia: 2.0 in. Type: Sch 40 PVC Interval: 51.45 - 66.45 ft.	
20	S-3	20.0-30.0	--					765
	ST-2	23.0-25.0	--					
	SS-6	23.5-25.0	9-10-11	17		Dark yellow orange silty SAND (SM), relict structures visible, layering and high angle fractures w/Mg staining (saprolite).		
25								760
	SS-7	28.5-28.9	50/4--	4				
30	S-4	30.0-40.0	--					755
	SS-8	33.5-35.0	19-23-25	18		Tan-yellow SILT (saprolite), relict structure layering, dry.		

NOTES

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S = Bulk Sample
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BORING LOG BC-11

PROJECT:	**Duke Power-Belews Creek**	DRILLING CONTRACTOR:	**S&ME, Inc.**
LOCATION:	**Belews Creek, NC**	DRILLING METHOD:	**4¼" H.S.A.**
PROJECT NO.:	**1264-02-576**	DRILL RIG:	**Mobile B-57**
LOGGED BY:	**J. Zschiedrich**	TOP OF CASING ELEVATION (ft):	**786.85**
DATE DRILLED:	**11/21/2002**	WATER LEVEL TOB (ft bgs):	**53.3**
BORING DEPTH (ft):	**67.9**	WATER LEVEL 24 HRS (ft bgs):	**41.02**

Depth (feet)	Sample Number	Sample Interval (feet)	Blow Counts per 6 inches	Recovery (inches)	Rock RQD (%)	Description & Remarks
						Tan-yellow SILT (saprolite), relict structure layering, dry. *(continued)*
	SS-9	38.5-38.8	50/3"	3		Partially weathered ROCK (Mica Schist), black relict layering, dry.
40	S-5	40.0-50.0	–			
	SS-10	43.5-43.9	50/4"	4		Partially weathered ROCK (Mica Schist) w/white streaks, black relict layering, dry.
45						
	SS-11	48.5-49.0	50/5"	5		
50	S-6	50.0-60.0	–			
	SS-12	53.5-53.9	50/4"	4		Partially weathered ROCK (Mica Schist), black relict layering, dry.
55						
	SS-13	58.5-59.0	50/5"	5		Partially weathered ROCK (Mica Schist) w/angular quartzite fragments (1 inch diameter), black relict layering, wet.
60	S-7	60.0-67.9	–			
	SS-14	63.5-63.8	50/3"	3		
65						
	SS-15	67.9	50/0"	0		Auger Refusal - Boring terminated at 67.9 ft.

Well Construction Details

Protective Casing
Dia: 4.0 in.
Type: Stick-Up
Interval: 1.70 ft.

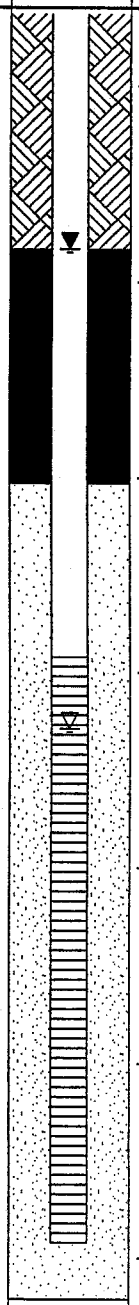
Riser Casing
Dia: 2.0 in.
Type: Sch 40 PVC
Interval: 0.0 - 51.45 ft.

Grout
Type: Portland Cement
Interval: 0.0 - 41.0 ft.

Seal
Type: Bentonite Chips
Interval: 41.0 - 47.0 ft.

Filterpack
Type: Filter Sand #2
Interval: 47.0 - 67.9 ft.

Screen
Dia: 2.0 in.
Type: Sch 40 PVC
Interval: 51.45 - 66.45 ft.



Elevation (feet): 750, 745, 740, 735, 730, 725, 720

NOTES

SS = Split Spoon Sample
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BORING LOG BC-12

PROJECT: **Duke Power-Belews Creek**
LOCATION: **Belews Creek, NC**
PROJECT NO.: **1264-02-576**
LOGGED BY: **J. Drummond**
DATE DRILLED: **11/13/2002**
BORING DEPTH (ft): **76.8**

DRILLING CONTRACTOR: **S&ME, Inc.**
DRILLING METHOD: **4¼" H.S.A.**
DRILL RIG: **CME-45**
TOP OF CASING ELEVATION (ft): **804.62**
WATER LEVEL TOB (ft bgs):
WATER LEVEL 24 HRS (ft bgs): **56.25**

Depth (feet)	Sample Number	Sample Interval (feet)	Blow Counts per 6 inches	Recovery (inches)	Rock RQD (%)	Symbol	Description & Remarks	Well Construction Details	Well Construction Graphic	Elevation (feet)
	S-1	0.0-5.0	–				Orange brown micaceous SILT, trace fine Sand, slightly moist.	Protective Casing Dia: 4.0 in. Type: Stick-Up Interval: 1.71 ft.		
	SS-1	1.0-2.5	3-2-3	6						
	ST-1	3.0-5.0	–							800
	SS-2	3.5-5.0	4-7-10	18			Orange brown to light brown very micaceous SILT and fine to coarse SAND, slightly moist.			
5	S-2	5.0-15.0	–					Riser Casing Dia: 2.0 in. Type: Sch 40 PVC Interval: 0.0 - 61.09 ft.		
	SS-3	6.0-7.5	4-5-7	18			Orange and white micaceous SILT and fine SAND (saprolite), dry, crumbly.			
	SS-4	8.5-10.0	4-5-11	18						795
10								Grout Type: Portland Cement Interval: 0.0 - 56.5 ft.		
	SS-5	13.5-15.0	12-15-16	18				Seal Type: Bentonite Chips Interval: 56.5 - 60.0 ft.		790
15	S-3	15.0-25.0	–					Filterpack Type: Filter Sand #2 Interval: 60.0 - 76.8 ft.		
	SS-6	18.5-20.0	8-9-11	18						785
20							Light brown micaceous SILT and fine to coarse SAND w/quartzite rock fragments (saprolite), dry, crumbly.	Screen Dia: 2.0 in. Type: Sch 40 PVC Interval: 61.09 - 76.09 ft.		
	SS-7	23.5-25.0	25-18-15	18						780
25	S-4	25.0-35.0	–							
	ST-2	27.0-28.7	–				Light brown, white, and light yellowish brown micaceous SILT and fine to coarse SAND (saprolite), dry, crumbly.			
	SS-8	28.5-30.0	10-11-12	18						775
30							Orange brown and light yellowish brown micaceous SILT and fine to coarse SAND (saprolite), dry, crumbly.			
	SS-9	33.5-35.0	12-15-16	18						770
							Grade			

NOTES
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BORING LOG BC-12

PROJECT: **Duke Power-Belews Creek**
LOCATION: **Belews Creek, NC**
PROJECT NO.: **1264-02-576**
LOGGED BY: **J. Drummond**
DATE DRILLED: **11/13/2002**
BORING DEPTH (ft): **76.8**

DRILLING CONTRACTOR: **S&ME, Inc.**
DRILLING METHOD: **4¼" H.S.A.**
DRILL RIG: **CME-45**
TOP OF CASING ELEVATION (ft): **804.62**
WATER LEVEL TOB (ft bgs):
WATER LEVEL 24 HRS (ft bgs): **56.25**

Depth (feet)	Sample Number	Sample Interval (feet)	Blow Counts per 6 inches	Recovery (inches)	Rock RQD (%)	Symbol	Description & Remarks	Well Construction Details	Well Construction Graphic	Elevation (feet)
	S-5	35.0-45.0	--				Light brown and orange brown micaceous HARD WEATHERED ROCK samples as SILT and fine to coarse SAND w/quartzite rock fragments (gneissic saprolite), dry.	Protective Casing Dia: 4.0 in. Type: Stick-Up Interval: 1.71 ft. Riser Casing Dia: 2.0 in. Type: Sch 40 PVC Interval: 0.0 - 61.09 ft. Grout Type: Portland Cement Interval: 0.0 - 56.5 ft. Seal Type: Bentonite Chips Interval: 56.5 - 60.0 ft. Filterpack Type: Filter Sand #2 Interval: 60.0 - 76.8 ft. Screen Dia: 2.0 in. Type: Sch 40 PVC Interval: 61.09 - 76.09 ft.		
	SS-10	38.5-40.0	50/6--	18						765
40										
	SS-11	43.5-44.0	38-50/.45-	6						760
45	S-6	45.0-55.0	--				Light brown, light yellowish brown, and white micaceous HARD WEATHERED ROCK samples as SILT and fine to coarse SAND w/quartzite rock fragments (gneissic saprolite), dry. Fractures at 50.7' : 40 51.6' : 40			
	SS-12	48.5-50.0	50/.45--	0						755
50										
	SS-13	53.5-55.0	50/.35--	0						750
55							Light brown and green gray micaceous HARD WEATHERED ROCK samples as SILT and fine to coarse SAND (gneissic saprolite), dry. Auger refusal at 62.8 ft. Fracture at 55.6' : 60			
	SS-14	58.5-60.0	50/.45--	0						745
60										
	R-1	62.8-71.8	--	102	55		Medium gray, dark green gray, and white layered medium to coarse grained quartz-feldspar GNEISS, very weathered, moderately fractured, fine sand and clay fracture filling. REC = 94% RQD = 55% Fractures at 63.5' : 71 71.3' : 80			740
65										
										735

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BORING LOG BC-12

PROJECT: **Duke Power-Belews Creek**
LOCATION: **Belews Creek, NC**
PROJECT NO.: **1264-02-576**
LOGGED BY: **J. Drummond**
DATE DRILLED: **11/13/2002**
BORING DEPTH (ft): **76.8**

DRILLING CONTRACTOR: **S&ME, Inc.**
DRILLING METHOD: **4¼" H.S.A.**
DRILL RIG: **CME-45**
TOP OF CASING ELEVATION (ft): **804.62**
WATER LEVEL TOB (ft bgs):
WATER LEVEL 24 HRS (ft bgs): **56.25**

Sample Information						Symbol	Description & Remarks	Well Construction Details	Well Construction Graphic	Elevation (feet)
Depth (feet)	Sample Number	Sample Interval (feet)	Blow Counts per 6 inches	Recovery (inches)	Rock RQD (%)					
	R-2	71.8-76.8	--	60	52		Medium gray, black, and white layered medium to coarse grained quartz-feldspar GNEISS, very weathered, moderately fractured, fine sand seam. REC = 100% RQD = 52%	Protective Casing Dia: 4.0 in. Type: Stick-Up Interval: 1.71 ft. Riser Casing Dia: 2.0 in. Type: Sch 40 PVC Interval: 0.0 - 61.09 ft. Grout Type: Portland Cement Interval: 0.0 - 56.5 ft. Seal Type: Bentonite Chips Interval: 56.5 - 60.0 ft. Filterpack Type: Filter Sand #2 Interval: 60.0 - 76.8 ft. Screen Dia: 2.0 in. Type: Sch 40 PVC Interval: 61.09 - 76.09 ft.		730
75							Boring terminated at 76.8 ft. in weathered GNEISS.			

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BORING LOG BC-13

PROJECT: **Duke Power-Belews Creek**
LOCATION: **Belews Creek, NC**
PROJECT NO.: **1264-02-576**
LOGGED BY: **S. Watts**
DATE DRILLED: **11/5/2002**
BORING DEPTH (ft): **45**

DRILLING CONTRACTOR: **S&ME, Inc.**
DRILLING METHOD: **4¼" H.S.A.**
DRILL RIG: **Mobile B-57**
TOP OF CASING ELEVATION (ft): **813.56**
WATER LEVEL TOB (ft bgs):
WATER LEVEL 24 HRS (ft bgs):

Depth (feet)	Sample Number	Sample Interval (feet)	Blow Counts per 6 inches	Recovery (inches)	Rock RQD (%)	Description & Remarks	Well Construction Details
	S-1	0.0-10.0	–				
	SS-1	0.5-2.0	3-4-5	15		Moderate reddish brown clayey SILT, slightly plastic, slightly moist (ML/MH).	Protective Casing Dia: 4.0 in. Type: Stick-Up Interval: 1.50 ft.
	SS-2	3.5-5.0	5-6-8	12			
5	ST-1	5.0-8.0	–				Riser Casing Dia: 2.0 in. Type: Sch 40 PVC Interval: 0.0 - 23.3 ft.
	SS-3	8.5-10.0	5-4-3	12		Moderate reddish brown micaceous SILT w/layering (MH), crumbly, dry (saprolite).	
10	S-2	10.0-20.0	–				Grout Type: Portland Cement Interval: 0.0 - 18.0 ft.
	SS-4	13.5-15.0	3-3-3	10		Dark yellowish orange to light brown fine sandy SILT w/relict structures (MH/SM), crumbly, dry (saprolite). Auger refusal at 25.0 ft.	Seal Type: Bentonite Chips Interval: 18.0 - 22.0 ft.
15	ST-2	15.0-18.0	–				Filterpack Type: Filter Sand #2 Interval: 25.0 - 45.0 ft.
	SS-5	18.5-19.4	14-50/4.5-	10			
20	S-3	20.0-25.0	–			Grade 4'	Screen Dia: 2.0 in. Type: Sch 40 PVC Interval: 23.3 - 43.3 ft.
	SS-6	23.5-25.0	50/2–	2			
25	R-1	25.0-30.0	–	25	0	Highly weathered GNEISS and SCHIST layered w/sandy silt and saprolite. -A layer of lt. gray calc-silicate GNEISS encountered at 25.0'. -A layer of lt. brown to gray mica schist encountered at 25.7'. REC = 42% RQD = 0% Fractures : Low angle partings at 10 26.65' : 69 29.60' : 60	
30	R-2	30.0-35.0	–	24	0	Brown foliated mica SCHIST, low angle fractures along bedding and high angle fractures w/Mg staining. -Moderate brown sandy SILT, stiff (saprolite) at 30.0'. -Lt. gray calc-silicate layer encountered	

Well Construction Graphic

Elevation (feet): 810, 805, 800, 795, 790, 785, 780

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BORING LOG BC-13

PROJECT: **Duke Power-Belews Creek**
LOCATION: **Belews Creek, NC**
PROJECT NO.: **1264-02-576**
LOGGED BY: **S. Watts**
DATE DRILLED: **11/5/2002**
BORING DEPTH (ft): **45**

DRILLING CONTRACTOR: **S&ME, Inc.**
DRILLING METHOD: **4¼" H.S.A.**
DRILL RIG: **Mobile B-57**
TOP OF CASING ELEVATION (ft): **813.56**
WATER LEVEL TOB (ft bgs):
WATER LEVEL 24 HRS (ft bgs):

Depth (feet)	Sample Number	Sample Interval (feet)	Blow Counts per 6 inches	Recovery (inches)	Rock RQD (%)	Symbol	Description & Remarks	Well Construction Details	Well Construction Graphic	Elevation (feet)
	R-3	35.0-40.0	–	24	0		at 30.5'. REC = 40% RQD = 0% Fractures at 30.40' : 40 Weathered SCHIST and biotite GNEISS. REC = 40% RQD = 0% Fractures at 36.1' : 72 39.5' : 65	**Protective Casing** Dia: 4.0 in. Type: Stick-Up Interval: 1.50 ft. **Riser Casing** Dia: 2.0 in. Type: Sch 40 PVC Interval: 0.0 - 23.3 ft. **Grout** Type: Portland Cement Interval: 0.0 - 18.0 ft. **Seal** Type: Bentonite Chips Interval: 18.0 - 22.0 ft. **Filterpack** Type: Filter Sand #2 Interval: 25.0 - 45.0 ft. **Screen** Dia: 2.0 in. Type: Sch 40 PVC Interval: 23.3 - 43.3 ft.		775
40	R-4	40.0-45.0	–	41.5	16		Weathered augen GNEISS and SCHIST. REC = 69% RQD = 16%			770
45							Boring terminated at 45.0 ft.			

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BORING LOG BC-16

PROJECT:	**Duke Power-Belews Creek**	DRILLING CONTRACTOR:	**S&ME, Inc.**
LOCATION:	**Belews Creek, NC**	DRILLING METHOD:	**4¼" H.S.A.**
PROJECT NO.:	**1054-04-955**	DRILL RIG:	**CME-550X**
LOGGED BY:	**L. Ennis/A. Smith**	TOP OF CASING ELEVATION (ft):	**835.65**
DATE DRILLED:	**9/9/2004**	WATER LEVEL TOB (ft bgs):	
BORING DEPTH (ft):	**106.4**	WATER LEVEL 24 HRS (ft bgs):	**73.8**

Depth (feet)	Sample Number	Sample Interval (feet)	Blow Counts per 6 inches	Recovery (inches)	Rock RQD (%)	Description & Remarks	Well Construction Details	Elevation (feet)
						TOPSOIL Pink/tan brown silty SAND, dry	Protective Casing Dia: Type: N/A Interval:	
	S-1	1-5	–					
	SS-1	3.5-5	12-16-10					830
5							Riser Casing Dia: 1.0 in. Type: Schedule 40 PVC Interval: 0 - 96.0	
	SS-2	8.5-10	10-16-29			Orange and pink fine sandy SILT, with clay layers, dry to moist with some rock fragments		825
10							Grout Type: Portland Cement Interval: 0.0 - 90.5	
	SS-3	13.5-15	13-12-18			Purple and tan clayey SAND with gravel, moist	Seal Type: Bentonite Interval: 90.5 - 93.0	820
15							Filterpack Type: Sand Interval: 93.0 - 106.0	
	SS-4	18.5-20	8-13-17	1.5		Red pink and yellow brown sandy SILT, micaceous, dry		815
20							Screen Dia: 1" Type: Schedule 40 PVC Interval: 96.0 - 106	
	SS-5	23.5-24.8	22-40-50/3"	1.1		Pink brown micaceous very fine sandy Silt SAPROLITE, layering visible, dry		810
25								
	SS-6	28.5-30	45-22-23	1.5				805
30								
	SS-7	33.5-35	21-35-50	1.2		Light tan/pink Silt SAPROLITE with rock fragments, dry		

NOTES

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BORING LOG BC-16

PROJECT: **Duke Power-Belews Creek**
LOCATION: **Belews Creek, NC**
PROJECT NO.: **1054-04-955**
LOGGED BY: **L. Ennis/A. Smith**
DATE DRILLED: **9/9/2004**
BORING DEPTH (ft): **106.4**

DRILLING CONTRACTOR: **S&ME, Inc.**
DRILLING METHOD: **4¼" H.S.A.**
DRILL RIG: **CME-550X**
TOP OF CASING ELEVATION (ft): **835.65**
WATER LEVEL TOB (ft bgs):
WATER LEVEL 24 HRS (ft bgs): **73.8**

Depth (feet)	Sample Number	Sample Interval (feet)	Blow Counts per 6 inches	Recovery (inches)	Rock RQD (%)	Symbol	Description & Remarks	Well Construction Details	Well Construction Graphic	Elevation (feet)
							Light tan/pink Silt SAPROLITE with rock fragments, dry *(continued)*	Protective Casing Dia: Type: N/A Interval:		
	SS-8	38.5-40	13-12-15	1.5			Yellow orange tan silty SAND, with some clay	Riser Casing Dia: 1.0 in. Type: Schedule 40 PVC Interval: 0 - 96.0		795
40										
	SS-9	43.5-45	23-12-25	1.5			Gray weathered QUARTZITE, PWR	Grout Type: Portland Cement Interval: 0.0 - 90.5		790
45							Yellow orange tan very fine sandy SILT, dry			
	SS-10	48.5-49	50--	0.5			Tan silty medium sand PWR, with quartz rock fragments, dry	Seal Type: Bentonite Interval: 90.5 - 93.0		785
50								Filterpack Type: Sand Interval: 93.0 - 106.0		
	SS-11	53.5-54.2	21-50/2"-	0.5			Tan silty sand PWR, visible layering, with weathered garnets, micaceous, damp	Screen Dia: 1" Type: Schedule 40 PVC Interval: 96.0 - 106		780
55										
	SS-12	58.5-60	7-15-39	1.5			White very fine sandy SILT, dry			775
60			Grade				Yellowish light brown silty sand PWR, visible layering, micaceous, dry			
	SS-13	63.5-64	50--	0.5						770
65										
	R-1	68.3-71.4	--	52	13		Medium gray Quartz Biotite GNEISS			

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BORING LOG BC-16

PROJECT: **Duke Power-Belews Creek**
LOCATION: **Belews Creek, NC**
PROJECT NO.: **1054-04-955**
LOGGED BY: **L. Ennis/A. Smith**
DATE DRILLED: **9/9/2004**
BORING DEPTH (ft): **106.4**

DRILLING CONTRACTOR: **S&ME, Inc.**
DRILLING METHOD: **4¼" H.S.A.**
DRILL RIG: **CME-550X**
TOP OF CASING ELEVATION (ft): **835.65**
WATER LEVEL TOB (ft bgs):
WATER LEVEL 24 HRS (ft bgs): **73.8**

Depth (feet)	Sample Number	Sample Interval (feet)	Blow Counts per 6 inches	Recovery (inches)	Rock RQD (%)	Symbol	Description & Remarks
	R-2	71.4-76.4	--	60	41		Medium gray Quartz Biotite GNEISS *(continued)*
							Medium to dark gray Quartz Biotite GNEISS
75	R-3	76.4-81.4	--	60	23		
80	R-4	81.4-86.4	--	60	52		
85	R-5	86.4-91.4	--	60	27		Medium gray highly fractured Biotite GNEISS
90	R-6	91.4-96.4	--	60	0		
95	R-7	96.4-101.4	--	60	70		Gray GNEISS and Mica SCHIST alternating layers
100	R-8	101.4-106.4	--	57	21		

Well Construction Details

Protective Casing
Dia:
Type: N/A
Interval:

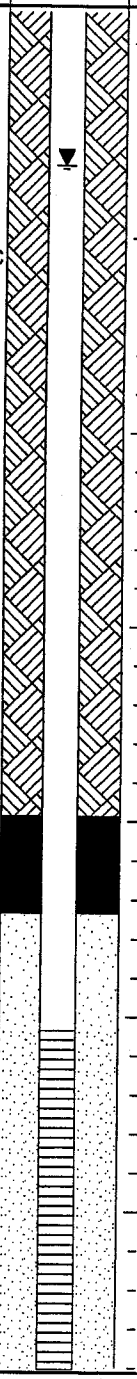
Riser Casing
Dia: 1.0 in.
Type: Schedule 40 PVC
Interval: 0 - 96.0

Grout
Type: Portland Cement
Interval: 0.0 - 90.5

Seal
Type: Bentonite
Interval: 90.5 - 93.0

Filterpack
Type: Sand
Interval: 93.0 - 106.0

Screen
Dia: 1"
Type: Schedule 40 PVC
Interval: 96.0 - 106



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BORING LOG BC-16

PROJECT:	**Duke Power-Belews Creek**	DRILLING CONTRACTOR:	**S&ME, Inc.**
LOCATION:	**Belews Creek, NC**	DRILLING METHOD:	**4¼" H.S.A.**
PROJECT NO.:	**1054-04-955**	DRILL RIG:	**CME-550X**
LOGGED BY:	**L. Ennis/A. Smith**	TOP OF CASING ELEVATION (ft):	**835.65**
DATE DRILLED:	**9/9/2004**	WATER LEVEL TOB (ft bgs):	
BORING DEPTH (ft):	**106.4**	WATER LEVEL 24 HRS (ft bgs):	**73.8**

Sample Information						Symbol	Description & Remarks	Well Construction Details	Well Construction Graphic	Elevation (feet)
Depth (feet)	Sample Number	Sample Interval (feet)	Blow Counts per 6 inches	Recovery (inches)	Rock RQD (%)					
105							Gray GNEISS and Mica SCHIST alternating layers *(continued)*	Protective Casing Dia: Type: N/A Interval:		730
							Boring Terminated at 106.4 ft	Riser Casing Dia: 1.0 in. Type: Schedule 40 PVC Interval: 0 - 96.0		
								Grout Type: Portland Cement Interval: 0.0 - 90.5		
								Seal Type: Bentonite Interval: 90.5 - 93.0		
								Filterpack Type: Sand Interval: 93.0 - 106.0		
								Screen Dia: 1" Type: Schedule 40 PVC Interval: 96.0 - 106		

NOTES

SS = Split Spoon Sample
ST = Shelby Tube Sample
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1. ▽ = Water Level At Termination of Boring
2. ▼ = Water Level After 24 Hours

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BORING LOG BC-18

PROJECT: **Duke Power-Belews Creek**
LOCATION: **Belews Creek, NC**
PROJECT NO.: **1054-04-955**
LOGGED BY: **A. Smith**
DATE DRILLED: **8/30/2004**
BORING DEPTH (ft): **38.1**

DRILLING CONTRACTOR: **S&ME, Inc.**
DRILLING METHOD: **4¼" H.S.A.**
DRILL RIG: **Mobile B-57**
TOP OF CASING ELEVATION (ft): **783.74**
WATER LEVEL TOB (ft bgs):
WATER LEVEL 24 HRS (ft bgs): **22.4**

Depth (feet)	Sample Number	Sample Interval (feet)	Blow Counts per 6 inches	Recovery (inches)	Rock RQD (%)	Description & Remarks	Elevation (feet)
						FILL	
						Brown fine sandy SILT, moist	
	SS-1	3.5-5	10-12-15				
5							780
	SS-2	8.5-10	20-30-50/4				
10						Brown clayey Sand PWR, moist, rock fragments at 13.5	775
	SS-3	13.5-13.9	50/4--				
15						Grade	770
	SS-4	18.5-18.8	50/3--				
20						Tan brown/brown fine sandy Silt PWR, moist, with rock fragments	765
	SS-5	23.5-24.3	39-50/2-				
25							760
	SS-6	28.5-28.3	50/3--				
30							755
	SS-7	33.5-33.6	50/0.1--				750

Well Construction Details

- **Protective Casing**
  Dia:
  Type: N/A
  Interval:
- **Riser Casing**
  Dia: 2.0 in.
  Type: Schedule 40 PVC
  Interval: 0.0 - 27.8
- **Grout**
  Type: Portland Cement
  Interval: 0.0 - 22.0
- **Seal**
  Type: Bentonite
  Interval: 22.0 - 25.5
- **Filterpack**
  Type: Sand
  Interval: 25.5 - 37.8
- **Screen**
  Dia: 2"
  Type: Schedule 40 PVC
  Interval: 27.8 - 37.8

NOTES

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BORING LOG BC-18

PROJECT: **Duke Power-Belews Creek**
LOCATION: **Belews Creek, NC**
PROJECT NO.: **1054-04-955**
LOGGED BY: **A. Smith**
DATE DRILLED: **8/30/2004**
BORING DEPTH (ft): **38.1**

DRILLING CONTRACTOR: **S&ME, Inc.**
DRILLING METHOD: **4¼" H.S.A.**
DRILL RIG: **Mobile B-57**
TOP OF CASING ELEVATION (ft): **783.74**
WATER LEVEL TOB (ft bgs):
WATER LEVEL 24 HRS (ft bgs): **22.4**

Sample Information						Symbol	Description & Remarks	Well Construction Details	Well Construction Graphic	Elevation (feet)
Depth (feet)	Sample Number	Sample Interval (feet)	Blow Counts per 6 inches	Recovery (inches)	Rock RQD (%)					
							Tan brown/brown fine sandy Silt PWR, moist, with rock fragments *(continued)*	Protective Casing Dia: Type: N/A Interval:		745
							Boring Terminated at 38.3	Riser Casing Dia: 2.0 in. Type: Schedule 40 PVC Interval: 0.0 - 27.8 Grout Type: Portland Cement Interval: 0.0 - 22.0 Seal Type: Bentonite Interval: 22.0 - 25.5 Filterpack Type: Sand Interval: 25.5 - 37.8 Screen Dia: 2" Type: Schedule 40 PVC Interval: 27.8 - 37.8		

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BORING LOG BC-19

PROJECT:	**Duke Power-Belews Creek**	DRILLING CONTRACTOR:	**S&ME, Inc.**
LOCATION:	**Belews Creek, NC**	DRILLING METHOD:	**4¼" H.S.A.**
PROJECT NO.:	**1054-04-955**	DRILL RIG:	**CME-550X**
LOGGED BY:	**L. Ennis**	TOP OF CASING ELEVATION (ft):	**814.81**
DATE DRILLED:	**9/21/2004**	WATER LEVEL TOB (ft bgs):	
BORING DEPTH (ft):	**56**	WATER LEVEL 24 HRS (ft bgs):	**52**

Depth (feet)	Sample Number	Sample Interval (feet)	Blow Counts per 6 inches	Recovery (inches)	Rock RQD (%)	Description & Remarks	Well Construction Details	Elevation (feet)
	S-1	2-7	–			TOPSOIL Orange brown clayey sandy SILT with orange and black oxidation, damp	Protective Casing Dia: Type: N/A Interval:	
	SS-1	3.5-5	12-8-10	1.3				810
5							Riser Casing Dia: 1.0 in. Type: Schedule 40 PVC Interval: 0.0 - 46.0	
	SS-2	8.5-10	4-4-5	1.3		Yellow orange sandy SILT, damp		805
10							Grout Type: Portland Cement Interval: 0.0 - 41.0	
	SS-3	13.5-15	4-4-4	1.3			Seal Type: Bentonite Interval: 41.0 - 42.6	800
15							Filterpack Type: Sand Interval: 42.6 - 56.0	
	SS-4	18.5-20	5-5-8	1.5		Tan sandy SILT		795
20							Screen Dia: 1" Type: Schedule 40 PVC Interval: 46.0 - 56.0	
	SS-5	23.5-25	5-8-12	1.3		Yellow orange brown silty clayey Sand SAPROLITE, micaceous, with some weathered k-feldspar lenses		790
25		Grade						
	ST-1	27-29	–					785
	SS-6	33.5-34.5	5-11-15	0.9				
30								
	SS-7	38.5-39.8	10-50-	1				

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BORING LOG BC-19

PROJECT: **Duke Power-Belews Creek**
LOCATION: **Belews Creek, NC**
PROJECT NO.: **1054-04-955**
LOGGED BY: **L. Ennis**
DATE DRILLED: **9/21/2004**
BORING DEPTH (ft): **56**

DRILLING CONTRACTOR: **S&ME, Inc.**
DRILLING METHOD: **4¼" H.S.A.**
DRILL RIG: **CME-550X**
TOP OF CASING ELEVATION (ft): **814.81**
WATER LEVEL TOB (ft bgs):
WATER LEVEL 24 HRS (ft bgs): **52**

Depth (feet)	Sample Number	Sample Interval (feet)	Blow Counts per 6 inches	Recovery (inches)	Rock RQD (%)	Symbol	Description & Remarks	Well Construction Details	Well Construction Graphic	Elevation (feet)
							Yellow orange brown silty clayey Sand SAPROLITE, micaceous, with some weathered k-feldspar lenses *(continued)*	Protective Casing Dia: Type: N/A Interval:		
	SS-8	43.5-44	13-31-50/4"	1.25						775
40							Light brown and gray silty sand PWR, highly weathered micaceous rock, layering visible, dry	Riser Casing Dia: 1.0 in. Type: Schedule 40 PVC Interval: 0.0 - 46.0		
	SS-9	48.5-48.9	50/6"–	4.5						770
45								Grout Type: Portland Cement Interval: 0.0 - 41.0		
	SS-10	53.5-53.7	50/5"–	1				Seal Type: Bentonite Interval: 41.0 - 42.6		765
50								Filterpack Type: Sand Interval: 42.6 - 56.0		
										760
55								Screen Dia: 1" Type: Schedule 40 PVC Interval: 46.0 - 56.0		
							Auger Refusal at 57.2			

NOTES

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Geosynthetics

- Geotextile Design
- Geonet Design
- Anchor Trench Design

Geotextile Design

- Soil Retention Properties of Geotextile
- Hydraulic Requirements
- Anti-Clogging Requirements
- Survivability Requirements
- Durability Requirements
- Puncture Resistance Requirements

Belews Creek FGD Residue Landfill
Geotextile Design

Job No.	**46119**		
Comp. By	**WMM**	**Date**	**9/14/2005**
Chkd. By	DEB	**Date**	9/26/05
Sheet	1	**of**	10

Objective: **Design the drainage system for the landfill.**

The specific applications where geotextile functions are to be designed are:

Application 1 **Section 1-5, Drawing C-5 CELL DETAILS, BOTTOM LINER SYSTEM**

For this application the geotextile must

- retain the site soils, gypsum, and clarifier sludge that will be placed against the LCRS Collector Pipe system.
- provide an adequate hydraulic path for liquid to enter into the system.

Application 2 **Section 2-5, Drawing C-5 CELL DETAILS, LCRS COLLECTOR SYSTEM**

For this application the geotextile must

- retain the site soils, gypsum, and clarifier sludge that will be placed against the LCRS Collector Pipe system.
- provide an adequate hydraulic path for liquid to enter into the system.

Application 3 **Section 3-5, Drawing C-5 CELL DETAILS, LCRS HEADER PIPE**

For this application the geotextile must

- retain the site soils used in the Cell Divider Berm from intruding into the sand/stone filter system adjacent to the lined channel.

References:

1) *Design of Lateral Drainage Systems for Landfills (Draft), Richardson, Giroud, Zhao, 2000.*

2) *Designing With Geosynthetics*, 3rd Edition, Koerner, 1994.

3) *NAVFAC DM-7.1 Soil Mechanics, 1982*

4) *Geologic and Hydrologic Siting Report, FGD Residue Disposal Site, Belews Creek Steam Station 3195 Pine Hall Road, Belews Creek, North Carolina S&ME Project 1054-04-955.*

Belews Creek FGD Residue Landfill
Geotextile Design

Job No.	**46119**		
Comp. By	**WMM**	**Date**	**9/14/2005**
Chkd. By	DGB	**Date**	9/21/05
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1) Determine soil retention properties of geotextile

Calculations:
- Geotextile should be designed for silty sandy soil to be used as a operational cover.

- Determine particle size distribution for 12" thick cover soil layer.
(from Reference 4)

				Soil Particle Sizes in mm						
Boring #	**Sample #**	**Depth (ft)**	**PI**	**d100**	**d85**	**d60**	**d50**	**d30**	**d15**	**d10**
BC-12	SS-6	18.5-20.0	9	1.13	0.14	0.0545	0.0345	0.0145	0.095	0.0034
BC-16	SS-3	13.5-15	12	10.2	0.822	0.12	0.09	0.0345	0.012	0.0068
BC-3	-	0.0-10.0	17	1.4	0.16	0.082	0.0525	0.0145	0.005	0.0023
BC-10	-	0.0-10.0	3	10.9	0.315	0.135	0.113	0.05	0.011	0.0066
BC-11	-	0-10	21	9.4	0.255	0.09	0.05	0.00145	0.00117	0.00117
BC-13	BAG-1	0.0-10.0	10	8.0	0.2	0.0613	0.0428	0.008	0.00122	0.00122
BC-16	BAG-1	0-5	8	3.0	0.217	0.111	0.073	0.0255	0.003	0.00145
BC-19	BAG-1	2-7	28	9.0	0.084	0.00822	0.0036	0.00116	0.00116	0.00116
BC-10	UD-1	12.0-14.0	14	2.0	0.093	0.0422	0.032	0.014	0.009	0.00413
BC-12	UD-1	27.0-28.0	6	2.45	0.122	0.07	0.053	0.0322	0.013	0.0111

indicates no value measured at this %, used smallest measured particle size.

Particle Size Distribution for Gypsum and Clarifier Sludge

			Soil Particle Sizes in mm						
		PI	**d100**	**d85**	**d60**	**d50**	**d30**	**d15**	**d10**
(Reference 6)	Clarifier Sludge	NP	2.00	0.0529	0.022	-	0.1707	0.0146	0.0146
(Reference 5)	Gypsum	1	0.841	0.06475	0.04744	-	0.03125	0.02645	0.02469

Belews Creek FGD Residue Landfill
Geotextile Design

Job No.	46119		
Comp. By	WMM	**Date**	9/14/2005
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Method 1
Use d85 and Cu to Check Ability of Geotextile to Retain Site Soil (Reference 1)

Check $O_{95}/d85 < B$

For soil with < 50% passing the #200 sieve:

B = 1	Cu <2 or > 8
B = 0.5*Cu	2 < Cu < 4
B = 8/Cu	4 Cu < 8

For soil with > 50% passing the #200 sieve:
B = 1.8

Where:
B = a function of the coefficient of uniformity, Cu
$Cu = \frac{d30^2}{d60 * d10}$, for soils where > 50% is coarser than #200

Steps:
1) Check for minimum d85* B
2) Check $O_{95}/d85 < B$ for AOS of #100 sieve

AOS (O_{95}) #100 Sieve = 0.149 mm

Boring #	Sample #	Depth (ft)	% Fines (< #200)	Cu	d85 (mm)	B	O_{95}= d85 * B	Check $O_{95}/d85<B$
BC-12	SS-6	18.5-20.0	66.3%	16.0	0.14	1.8	0.25	1.06
BC-16	SS-3	13.5-15	45.6%	17.6	0.82	1	0.82	0.18
BC-3	-	0.0-10.0	57.6%	35.7	0.16	1.8	0.29	0.93
BC-10	-	0.0-10.0	39.5%	20.5	0.32	1	0.32	0.47
BC-11	-	0-10	56.5%	76.9	0.26	1.8	0.46	0.58
BC-13	BAG-1	0.0-10.0	65.5%	50.2	0.20	1.8	0.36	0.75
BC-16	BAG-1	0-5	49.7%	76.6	0.22	1	0.22	0.69
BC-19	BAG-1	2-7	83.8%	7.1	0.08	1.8	0.15	1.77
BC-10	UD-1	12.0-14.0	79.9%	10.2	0.09	1.8	0.17	1.60
BC-12	UD-1	27.0-28.0	62.3%	6.3	0.12	1.8	0.22	1.22

All soil samples meet criterion of O95/d85 < B
Therefore, a non-woven geotextile with AOS of #100 sieve will retain site soils.

Check Minimum Hydraulic Requirements

from Reference 1, Table 2.2 (after AASHTO M 288-96)

Filter Criteria	% Soil Passing #200 (0.075mm) Sieve		
	< 15	15 to 50	> 50
Minimum Permittivity	0.5 sec^{-1}	0.2 sec^{-1}	0.1 sec^{-1}
Maximum AOS	0.43 mm	0.25 mm	0.22 mm

The site soils generally have particle sizes such that > 50 % of the soils are finer than a No. 200 sieve (0.075mm).
Likewise, the gypsum and clarifier sludge have particle sizes such that > 50 % of the soils are finer than a No. 200 sieve (0.075mm).

Therefore, From Reference 1, Table 2.2, the geotextile should have the following:

Minimum Permittivity 0.1 sec^{-1}
Maximum AOS 0.22 mm

Since the openings in a #100 sieve are 0.149 mm, non-woven geotextile with AOS of #100 sieve will meet the Maximum AOS criterion.

Belews Creek FGD Residue Landfill
Geotextile Design

Job No.	46119		
Comp. By	WMM	**Date**	9/14/2005
Chkd. By	DEB	**Date**	9/26/05
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Check Clogging Potential

Reference 1, page 14 states that generally particles do not move within soils having a PI (plasticity index) of greater than 15 %, so there is no clogging potential.

Reference 1 recommends that soils be characterized as stable or unstable.
Stable soils perform an internal filtration process that limits migration of fines within the soil.
Typically, these soil types include well-graded soils. Unstable soils are those that cannot perform self-filtration.

For a soil to be well-graded and stable, it must meet the criteria:

$$Cu = \frac{d60}{d10} > 4 \quad \text{and} \quad 1 < Cc = \frac{d^2 30}{d10 * d60} < 3 \qquad \text{(Eqn. 2.1)}$$

Boring #	Sample #	Depth (ft)	PI	d60	d30	d10	Cu	Cc	Meets Criteria
BC-12	SS-6	18.5-20.0	9	0.0545	0.0145	0.0034	16.0	1.1	Yes
BC-16	SS-3	13.5-15	12	0.12	0.0345	0.0068	17.6	1.5	Yes
BC-3	-	0.0-10.0	17	0.082	0.0145	0.0023	35.7	1.1	Yes
BC-10	-	0.0-10.0	3	0.135	0.05	0.0066	20.5	2.8	Yes
BC-11	-	0-10	21	0.09	0.00145	0.00117	76.9	0.0	**No**
BC-13	BAG-1	0.0-10.0	10	0.0613	0.008	0.00122	50.2	0.9	**No**
BC-16	BAG-1	0-5	8	0.111	0.0255	0.00145	76.6	4.0	Yes
BC-19	BAG-1	2-7	28	0.00822	0.00116	0.00116	7.1	0.1	**No**
BC-10	UD-1	12.0-14.0	14	0.0422	0.014	0.00413	10.2	1.1	Yes
BC-12	UD-1	27.0-28.0	6	0.07	0.0322	0.0111	6.3	1.3	Yes

Check particulate clogging:
Reference 1 recommends that particulate clogging be checked by:

Check O_{95}>d15 for Cu > 3 AOS (O_{95}) #100 Sieve = 0.149 mm

Boring #	d15	Cu	Meets O_{95}>d15 for Cu > 3?
BC-12	0.095	16.0	Yes
BC-16	0.012	17.6	Yes
BC-3	0.005	35.7	Yes
BC-10	0.011	20.5	Yes
BC-11	0.00117	76.9	Yes
BC-13	0.00122	50.2	Yes
BC-16	0.003	76.6	Yes
BC-19	0.00116	7.1	Yes
BC-10	0.009	10.2	Yes
BC-12	0.013	6.3	Yes

Geotextile with AOS (O_{95}) #100 Sieve is not susceptible to particulate clogging.

Check potential for biological clogging:

From Reference 1, the minimum permeability of the geotextile should satisfy the following criteria:

$K_{geotextile} \geq 100 * K_{soil}$ for "active" liquid

$K_{geotextile} \geq 10 * K_{soil}$ for "inactive" liquid

The reference states that the potential for biological clogging is related to the biological activity of the pore water liquid.

Boring	Sample Depth (ft)	K (cm/sec)
BC-3	0 - 10	2.40E-05
BC-7	0 - 10	3.42E-06
BC-10	0 - 10	9.06E-05
BC-11	0 - 10	1.67E-07
BC-12	5 - 10	5.61E-06
BC-13	0 - 10	4.81E-06
BC-13	10 - 20	8.65E-06
BC-15	0 - 5	7.61E-06
BC-16	1 - 5	2.30E-05
BC-17	0 - 6	1.82E-05
BC-19	0 - 10	1.79E-05
BC-20	0 - 5	2.03E-06
BC-21	0 - 5	3.95E-06
BC-22	0 - 5	2.87E-04
BC-23	1 - 6	4.39E-05

Average K = 3.61E-05 $100 * K_{soil\ (avg)}$ = 3.61E-03 cm/sec

Minimum K = 1.67E-07 $100 * K_{soil\ (min)}$ = 1.67E-05 cm/sec

For a typical non-woven geotextile with an AOS of #100,

$K_{geotextile}$ = 0.2 cm/sec

Geotextile with AOS (O_{95}) #100 Sieve is not susceptible to biological clogging.

Determine Survivability Requirements

From Reference 1, and From Table 2.1 Nonwoven Geotextile Survivability Requirements

Property	Test Method	Class 2
Grab Strength	ASTM D 4632	158 lb
Sewn Seam Strength	ASTM D 4632	142 lb
Tear Strength	ASTM D4533	57 lb
Puncture Strength	ASTM D4833	57 lb
Burst Strength	ASTM D 3786	189 psi

Reference 1 states that the default class for subsurface drainage is Class 2.

Durability Requirements

Minimum UV exposure ==> no special requirements
Chemical compatibility ==> use polyethylene or polypropylene material.

Determine Puncture Resistance Requirements of Non-Woven Geotextile to Stone

Koerner (Reference 2) provides the following equation for determining the vertical force that can be resisted by a geotextile.

$F_{reqd} = p' \times d_a^2 \times S_1 \times S_2 \times S_3$ Eqn 2.30

F_{reqd} = the required vertical force to be resisted
p' = the pressure exerted on the geotextile
d_a = the average diameter of the puncturing aggregate
S_1 = protrusion factor = h_h/d_a
h_h = protrusion height ≤ d_a
S_2 = scale factor to adjust ASTM D4833 test value using 5/16" diameter puncture probe to actual punching object = 0.31/da
S_3 = scale factor to adjust flat puncture probe of ASTM D4833 to actual shape of puncturing object = 1 - A_p/A_c
Ap/Ac values are:
0.8 for Ottawa sand
0.7 for run-of-bank gravel
0.4 for crushed rock
0.6 for shot rock

p'	= 67.5 psi	(Cat 623B Scraper)
d_a	= 1.0 "	for #57 Stone (conservative)
S_1	= 1.0	assume protrusion height =1", so 1"/1"
S_2	= 0.31	= 0.31/1"
S_3	= 0.6	=1 - 0.4

F_{reqd} = 67.5 psi x 1.0 ^2 (in²) x 1.0 x 0.31 x 0.6

F_{reqd} = 12.6 lbs

From Reference 1, and From Table 2.1 Nonwoven Geotextile Survivability Requirements the Puncture Strength for a Class 2 application is 57 lb.

$FS = \frac{57 \text{ lb}}{12.6 \text{ lbs}} = 5$ OK

SUMMARY OF REQUIRED PROPERTIES

Select Geotextile with the Following Properties:

	AOS US Sieve #100
Minimum Permeability	0.2 cm/sec
Grab Strength ≥	158 lbs
Puncture Strength ≥	57 lbs
Burst Strength ≥	189 psi
Trapezoid Tear Strength ≥	57 lbs
Min. Mass per Unit Area =	8 oz.
Permittivity ≥	0.1 sec^{-1}

Geotextile Design

References

DRAFT

DESIGN OF LATERAL DRAINAGE SYSTEMS FOR LANDFILLS

Gregory N. Richardson, Ph.D., P.E.
G.N. Richardson & Associates
Raleigh, North Carolina 27603

Jean-Pierre Giroud, E.C.P., Ph.D.
GeoSyntec Consultants
Boca Raton, Florida 33487

Aigen Zhao, Ph.D., P.E.
Tenax Corporation
Baltimore, Maryland 21205

Reference 1

2000

friction and cushion protection for the underlying geomembrane. Fortunately, minimum strength and filter properties can be quickly evaluated using AASHTO M-288 requirements for similar transportation related drainage applications. In addition, there are many soil retention criteria, such as Luettich et al. (1992), Holtz et al. (1997), available for further examinations.

2.1.1 Geotextile Minimum Strength for "Survivability"

The term "geotextile survivability" refers to the fabric's ability to withstand installation stresses associated with a particular application, and to perform as intended in the project design. In 1996, AASHTO published a set of survivability criteria for geotextiles, designated as AASHTO M 288-96. The guidelines prescribe four tests to assess the strength of a geotextile and its ability to survive installation stress. These guidelines address both woven and nonwoven geotextiles. In drainage composites, nonwovens are used exclusively so that only a portion of M288 is presented in Table 2-1. Class1 geotextiles meet the most stringent survivability criteria and Class 3 geotextiles meet the minimum survivability criteria.

The class of geotextile required on a geocomposite is dependent on the specific application. AASHTO M 288-96 provides guidelines for five general applications: subsurface drainage, separation, stabilization, permanent erosion control, and paving. The default geotextile class for subsurface drainage applications is Class 2. However, for each application, the engineer may specify geotextile class based on past field experience or laboratory test data. The "survivability" strength for the geotextile in a drainage composite commonly does not control the selection of the geotextile but "bonding" requirements to physically connect the geotextile to the drainage core will control.

Table 2.1 Nonwoven Geotextile Survivability Requirements
(After AASHTO M 288-96)

Property	Test Method	Class 1	Class 2	Class 3
Grab Strength	ASTM D 4632	900 N (203 lb)	700 N (158 lb)	500 N (113 lb)
Sewn Seam Strength	ASTM D 4632	810 N (182 lb)	630 N (142 lb)	450 N (101 lb)
Tear Strength	ASTM D 4533	350 N (79 lb)	250 N (57 lb)	180 N (41 lb)
Puncture Strength	ASTM D 4833	350 N (79 lb)	250 N (57 lb)	180 N (41 lb)
Burst Strength	ASTM D 3786	1700 kPa (247 psi)	1300 kPa (189 psi)	950 kPa (138 psi)

2.1.2 Geotextile Filter Criteria

Geotextile filtration properties must be selected based on the up-gradient soil gradation and plasticity, and site specific hydraulic conditions. The following filter criteria assume that the soil being retained is "set" in intimate contact with the geotextile, i.e., no voids exist between the fabric

and the soil. For non-vertical drains this is accomplished by gravity, for vertical drains this must be accomplished by compaction of the soils adjacent to the geocomposite.

Most geotextile filter designs are based on the retained soil Atterberg limits and grain size distribution. Atterberg limits (ASTM D-4318) include the liquid limit (LL), plastic limit (PL), and plasticity index (PI) of the soil. Such information is helpful in assessing the flow capacity and clogging potential of a geotextile. The PI value provides a measure of soil cohesiveness, a high PI value indicates high clay content of a highly cohesive soil. For filter design criteria presented in this manual, it is important to determine if the PI is greater or less than 15 percent. Generally, particles do not move within soils having a PI greater than 15 percent so there is no clogging potential.

For the purposes of filtration design, soils can be characterized as stable or unstable. Stable soils perform an internal filtration process that limits migration of fines within the soil. Typically, these soil types include well-graded soils. Unstable soils are those which cannot perform self-filtration (i.e. they have the potential to pipe internally). They may include gap-graded, and broad-graded soils, and other highly erodable soils. In gap-graded soils, there exists a coarse and fine fraction, but very little medium fraction. If there is an insufficient quantity of soil particles in the medium fraction, fine soil particles pipe through the coarse fraction. In broad-graded soils, the gradation is distributed over a very wide range of particle sizes such that fine soil tends to pipe through coarser particles.

For a soil to be classified as both well-graded and stable , it must meet the following criteria:

$$C_U = \frac{D_{60}}{D_{10}} > 4 \quad \text{and} \quad 1 < C_c = \frac{D_{30}^2}{D_{10} \, x \, D_{60}} < 3 \qquad \text{Eq. 2.1}$$

where: D_{10} = the diameter at which 10 percent of the soil is finer.
D_{30} = the diameter at which 30 percent of the soil is finer.
D_{60} = the diameter at which 60 percent of the soil is finer.
C_u = the coefficient of uniformity, it quantifies distribution of particle sizes
C_c = the coefficient of curvature, it identifies internal soil stability

Sample gradation curves of well-graded and uniformly graded soil are shown in Figure 2.5 and gap-graded and broad-graded soils are shown in Figure 2.6. The value of D_{60}/D_{10} distinguishes unstable broad-graded soil from a well-graded soil. For broad graded-soils, this value exceeds 20. Another characteristic of unstable broad-graded soils is that the gradation curve is concave upward.

The geotextile hydraulic requirements are based on its through-the-plane flow capacity, permittivity, and its apparent opening size, AOS. The AOS is defined as the diameter of glass beads that 5% or less will pass through the geotextile during the ASTM D4751 dry sieve test. This is also commonly referred to as the O_{95} of the geotextile. Minimum hydraulic requirements shown in Table 2-2 are from AASHTO M 288-96 and should be considered in the selection of a geotextile.

Table 2-2 Geotextile criteria for subsurface drainage
(after AASHTO M 288-96)

Filter Criteria	Percent Soil Passing No. 200 (0.075mm) Sieve		
	< 15	15 to 50	>50
Minimum Permittivity, ASTM D-4491	0.5 sec^{-1}	0.2 sec^{-1}	0.1 sec^{-1}
Maximum AOS, ASTM D-4751	0.43 mm	0.25 mm	0.22 mm

An additional check on the ability of the geotextile to retain the up gradient soil can also be verified using the retention criteria, such as proposed by Holtz et al. (1997). Use of this criterion requires performing a grain size analysis on the retained soil to determine the D_{85}, and C_u of the soil, where D_{85} is the particle size at which 85% of the retained soil particles are finer than, and C_u is the coefficient of uniformity.

$$O_{95}/D_{85} < B \qquad \text{Eq. 2.2}$$

For sands, gravely sands, silty sands, and clayey sands (with less than 50% passing the 0.075 mm sieve), B is a function of coefficient of uniformity

B=1	$C_u < 2$ or > 8
$B=0.5*C_u$	$2 < C_u < 4$
$B=8/C_u$	$4 < C_u < 8$

For silts and clays (with more than 50% passing the 0.075 mm sieve) B =1.8. ← Fine soil

Thus, the critical physical properties of a retained soil that impact the selection of a geotextile include the "stability" of its particle size distribution, the percent fines, and the D_{85}. These factors can be economically evaluated in a simple grain size analysis (ASTM D-421 & 422).

An evaluation of clogging potential (including particulate clogging and biological clogging potential) for the geotextile should also be conducted. To minimize particulate clogging:

$$O_{95} > 3\ D_{15} \qquad \text{for } C_u > 3$$

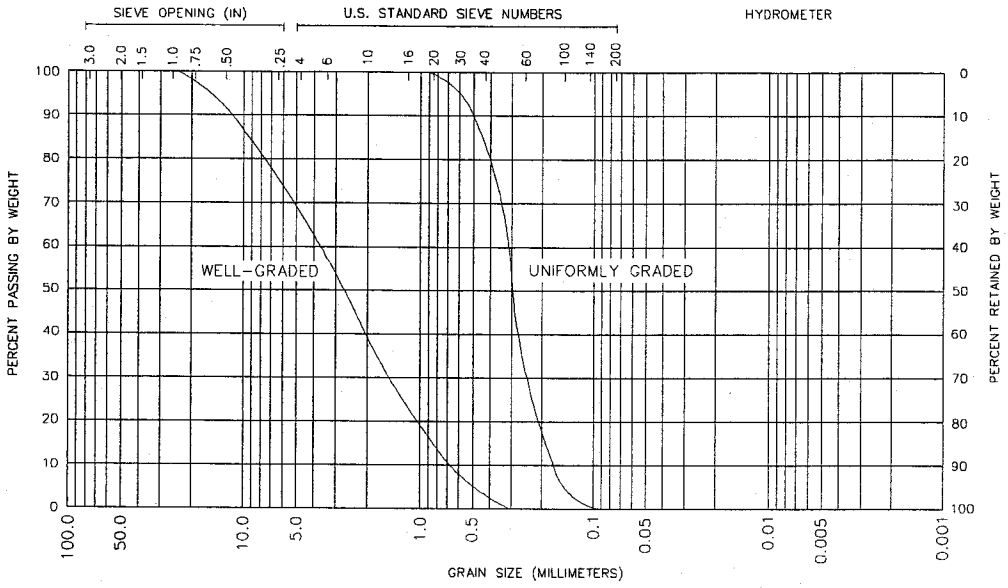


Figure 2.5 Well-graded and uniformly-graded soils.

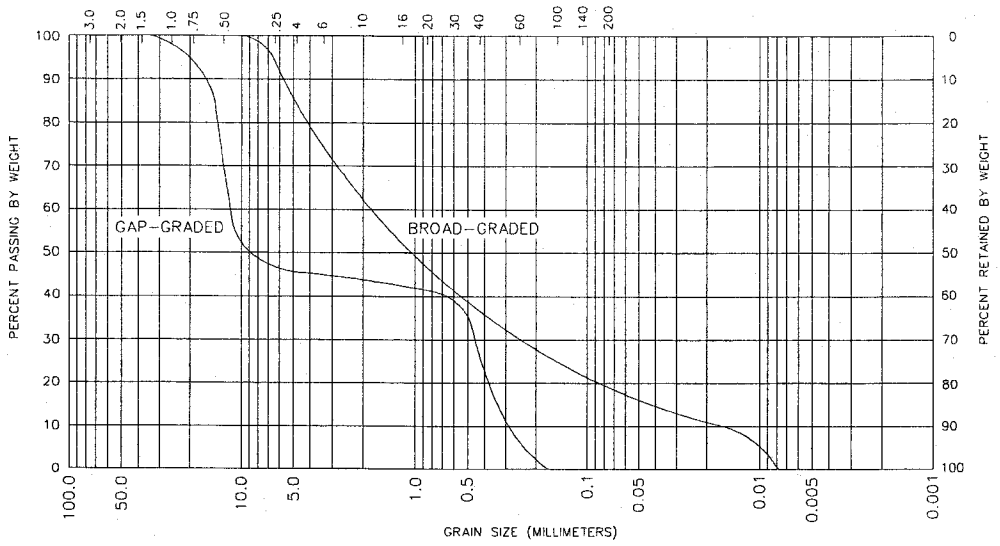


Figure 2.6 Gap-graded and broad-graded soils

In addition to the above empirical equation, there are two laboratory test methods available specifically for this evaluation:

- ASTM D5101, Gradient Ratio test (Standard test method for measuring the soil-geotextile system clogging potential at low normal loads)

- ASTM D5567, Hydraulic Conductivity Ratio test (Standard test method for measuring the soil-geotextile system clogging potential at higher normal loads)

The potential for biological clogging is influenced by the relative biological activity of the pore water liquid. Typical ground waters have a low biological, e.g. inactive, potential while leachates collected in most landfills is 'active' and has a high clogging potential. The actual biological clogging potential can be evaluated by using the actual leachate in the two tests described above. Lacking this data, the minimum permeability of the geotextile in a geocomposite lateral drainage layer should satisfy the following criteria (based on data from Koerner et al. 1993):

$$K_{geotextile} > 100\ K_{soil} \qquad \text{'Active' Liquid}$$

$$K_{geotextile} > 10\ K_{soil} \qquad \text{'Inactive' Liquid}$$

The permeability of the geotextile must be significantly larger than the permeability of the soil to address biological clogging concerns. This does not mean, however, that the soil itself will not clog. The above relationship must not be used for sock covers over perforated pipes. Test data indicated reductions of permeability in sock drain applications that reached 5 order-of-magnitude reduction.

2.2 Interface Friction

Geocomposite drains have proven to be very stable in slope applications due to the high interface friction of a nonwoven geotextile with both textured geomembranes and soils. Similar to the soil shear behavior, the

Solution: (a) Using a maximum strain of 33%, the value of $f(\epsilon) = 0.52$. Thus the required grab tensile strength is as follows:

$$\begin{aligned} T_{reqd} &= p'(d_v)^2(0.52) \\ &= p'(0.33\ d_a)^2(0.52) \\ &= 0.057\ p'd_a^2 \\ &= 0.057(100)(2.0)^2 \\ &= 22.6 \text{ lb.} \end{aligned}$$

(b) The global factor of safety on a 125-lb. ultimate grab tensile geotextile with partial factors of safety of 2.5 is as follows:

$$\begin{aligned} FS &= \frac{T_{allow}}{T_{reqd}} \\ &= \frac{125/2.5}{22.6} \\ &= 2.2, \text{ which is acceptable.} \end{aligned}$$

2.5.4 Puncture Resistance

Although not only related to the separation function, the geotextile during its placement must survive the installation process. Indeed, fabric survivability is critical in all types of applications; without it, the best of designs are futile (recall Section 2.2.5.1). In this regard, sharp stones, tree stumps, roots, miscellaneous debris, and other things on the ground beneath the geotextile could puncture through the geotextile after stone base and traffic loads are imposed above it. The design method suggested for this situation is shown schematically in Figure 2.29. For these conditions, the vertical force exerted on the geotextile (which is gradually tightening around the protruding object) is as follows:

$$F_{reqd} = p'd_a^2 S_1 S_2 S_3 \tag{2.30}$$

where F_{reqd} = the required vertical force to be resisted,
p' = the pressure exerted on the geotextile (approximately 100% of tire inflation pressure at the ground surface for small stone thicknesses),
d_a = the average diameter of the puncturing aggregate or sharp object,
S_1 = protrusion factor = h_h/d_a,
h_h = protrusion height $\leq d_a$,
S_2 = scale factor to adjust ASTM D4833 test value using 5/16-in.-diameter puncture probe to actual puncturing object = $0.31/d_a$,
S_3 = shape factor to adjust flat puncture probe of ASTM D4833 to actual shape of puncturing object = $1 - A_p/A_c$ (values of A_p/A_c to be used

Reference 2

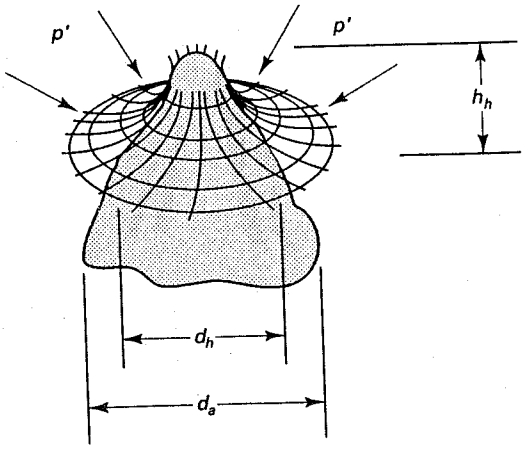


Figure 2.29 Visualization of a stone puncturing a geotextile as pressure is applied from above.

range from 0.8 for Ottawa sand, 0.7 for run-of-bank gravel, 0.4 for crushed rock, and 0.3 for shot rock),

A_p = projected area of particle, and

A_c = area of smallest circumscribed circle.

Example:

What is the factor of safety against puncture of a geotextile from a 2.0-in. stone by a loaded truck with tire inflation pressure of 80 lb./in.^{2} traveling on the surface of the stone base? The geotextile has an ultimate puncture strength of 45 lb. according to ASTM D4833.

Solution: Using the full stress on the geotextile of 80 lb./in.^{2} and factors of 0.33, 0.155, and 0.6 for S_1, S_2, and S_3, respectively,

$$F_{reqd} = p' d_a^2 S_1 S_2 S_3$$
$$= 80 \times (2.0)^2(0.33)(0.155)(0.6)$$
$$= 9.82 \text{ lb.}$$

Assuming that the cumulative partial factor of safety is 2.0, the global factor of safety is as follows:

$$FS = \frac{F_{allow}}{F_{reqd}}$$
$$= \frac{45/2.0}{9.82}$$
$$= 2.3, \text{ which is acceptable.}$$

Particle Size Analysis for "Cut" Material

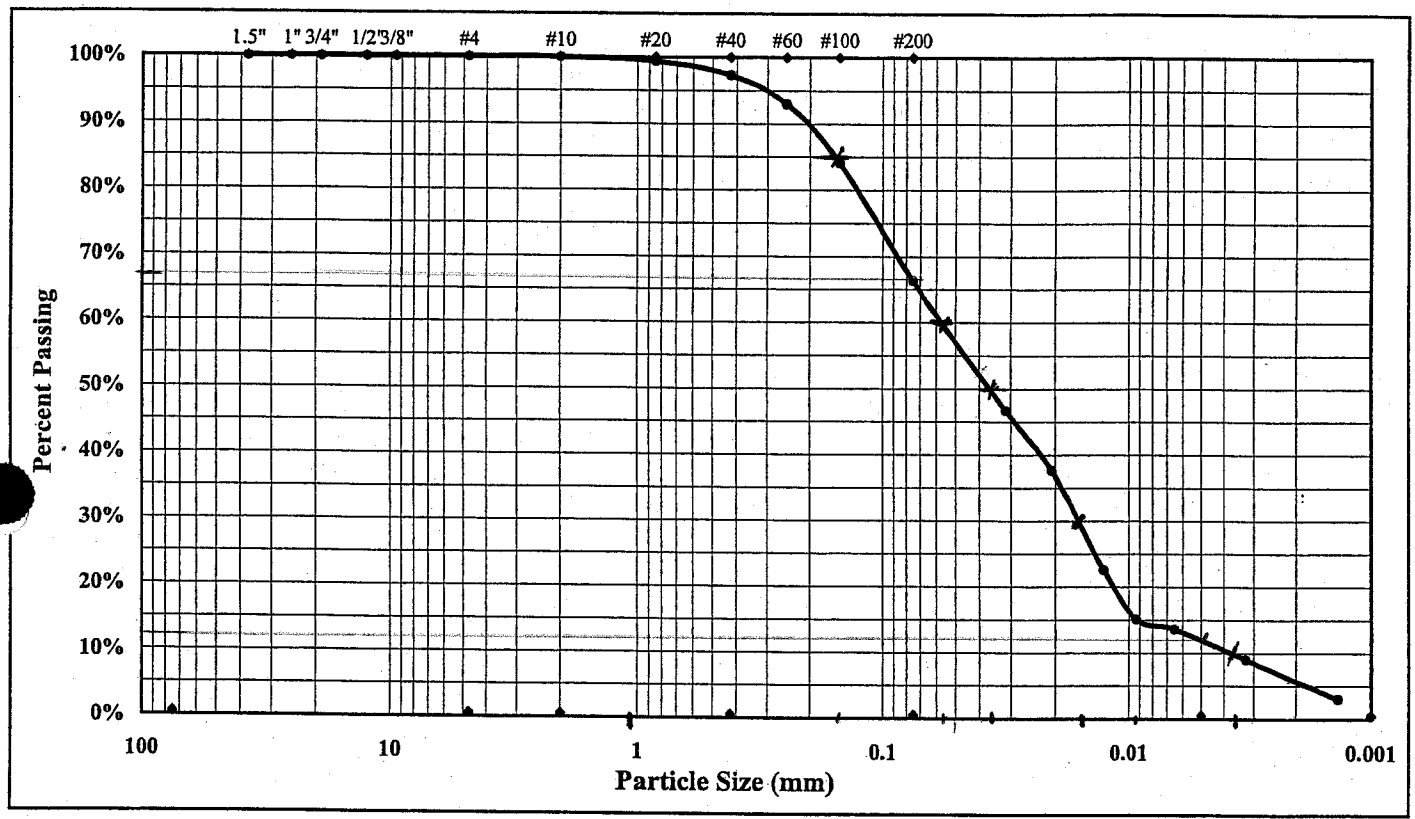
Particle Size Analysis of Soils

ASTM D 422

S&ME

ME Project #:	**1264-02-576**	Report Date:	January 22, 2003
Project Name:	**Duke Power-Belews Creek Landfill**	Test Date(s):	1/15/03-1/22/03
Client Name:	Duke Energy Corp.		
Client Address:	3195 Pine Hall Road, Belews Creek, NC 27009		

Boring #:	BC-12	Sample #:	SS-6	Sample Date:	NA
Location:		Offset:		Depth (ft):	18.5 - 20.0
Sample Description:	Tan Fine Sandy Clayey SILT (ML)				



Cobbles	< 300 mm (12") and > 75 mm (3")	Fine Sand	< 0.425 mm and > 0.075 mm (#200)
Gravel	< 75 mm and > 4.75 mm (#4)	Silt	< 0.075 and > 0.005 mm
Coarse Sand	< 4.75 mm and >2.00 mm (#10)	Clay	< 0.005 mm
Medium Sand	< 2.00 mm and > 0.425 mm (#40)	Colloids	< 0.001 mm

Moisture Content		Gravel	0.0%	Liquid Limit	45
Silt & Clay (% Passing #200)	66.3%	Sand	33.7%	Plastic Limit	36
Maximum Particle Size	#10	Silt	53.8%	Plastic Index	9
Specific Gravity	2.757	Clay	12.5%	Colloids	3.0%

Description of Sand & Gravel Particles: Rounded ☐ Angular ☐ Hard & Durable ☐ Soft ☐ Weathered & Friable

ASTM D 422: Particle Size Analysis of Soils Dispersing Agent: Sodium Hexametaphosphate: 40 g./ Liter
Mechanical Stirring Apparatus (A) Length of Dispersion Period: 1 min.

References: ASTM D 854: Specific Gravity of Soils
M D 421: Dry Preparation of Soil Samples
M D 2216: Laboratory Determination of Water (Moisture) Content of Soil and Rock by Mass
M D 4318: Liquid Limit, Plastic Limit, & Plastic Index of Soils
ASTM D 2487: Classification of Soils for Engineering Purposes (Unified Soil Classification System)

Technical Responsibility / Position: Mal Krajan Laboratory Supervis

Laboratory Report Version 4.2

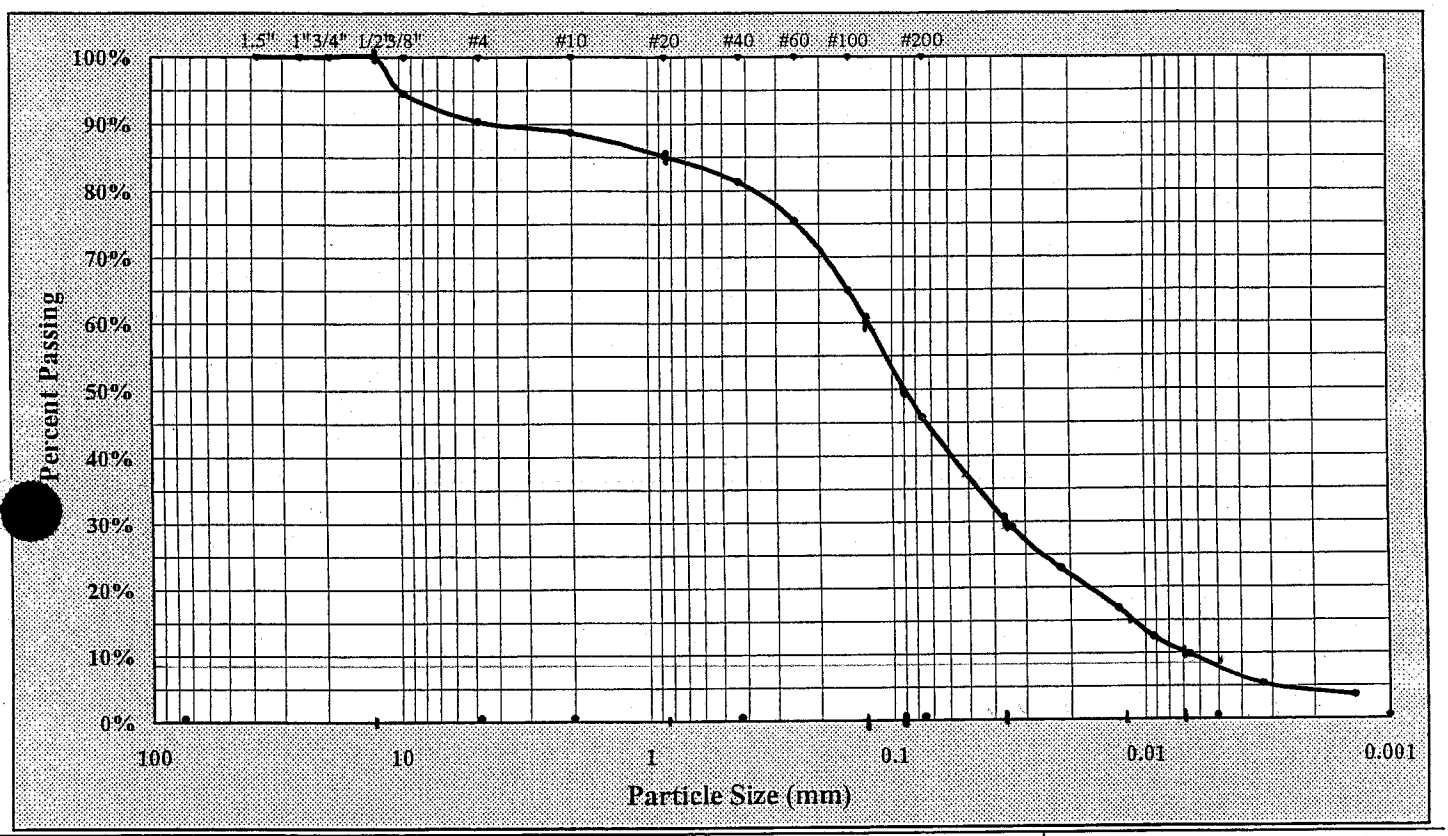
Particle Size Analysis of Soils

ASTM D 422

S&ME

S[redacted]E Project #:	**1054-04-955**	Report Date:	10/08/2004
P[redacted]ct Name:	**Belews Creek Steam Plant**	Test Date(s):	09/28 - 10/07/2004
Client Name:	Duke Energy		
Client Address:			

Boring #:	BC-16	Sample #:	SS-3	Sample Date:	9/1/04
Location:	Site Borehole	Offset:	NA	Depth (ft):	13.5 - 15'
Sample Description:	Purple and Tan Clayey SAND with Gravel (SC)				



Cobbles	< 300 mm (12") and > 75 mm (3")	Fine Sand	< 0.425 mm and > 0.075 mm (#200)
Gravel	< 75 mm and > 4.75 mm (#4)	Silt	< 0.075 and > 0.005 mm
Coarse Sand	< 4.75 mm and >2.00 mm (#10)	Clay	< 0.005 mm
Medium Sand	< 2.00 mm and > 0.425 mm (#40)	Colloids	< 0.001 mm

Maximum Particle Size	1/2"	Gravel	9.7%	Silt	38.1%
Silt & Clay (% Passing #200)	45.6%	Sand	44.7%	Clay	7.5%
Apparent Relative Density	2.677	Moisture Content		Colloids	3.5%
Liquid Limit	36	Plastic Limit	24	Plastic Index	12

Description of Sand & Gravel Particles Rounded ☐ Angular ☒ Hard & Durable ☐ Soft ☒ Weathered & Friable ☐

Mechanical Stirring Apparatus (A) Length of Dispersion Period: 1 min. Dispersing Agent: Sodium Hexametaphosphate: 40 g./ Liter

[redacted]rences: ASTM D 422: Particle Size Analysis of Soils
ASTM D 421: Dry Preparation of Soil Samples
[redacted]D 4318: Liquid Limit, Plastic Limit, & Plastic Index of Soils
ASTM D 854: Specific Gravity of Soils
[redacted]D 2487: Classification of Soils for Engineering Purposes (Unified Soil Classification System)

Technical Responsibility: Mal Krajan — Laboratory Supervisor

Signature Position

S&ME, INC. 3109 Spring Forest Road, Raleigh, N.C. 27616 BC-16 SS-3 Classification.xls

Laboratory Report Version 4.0

Particle Size Analysis of Soils

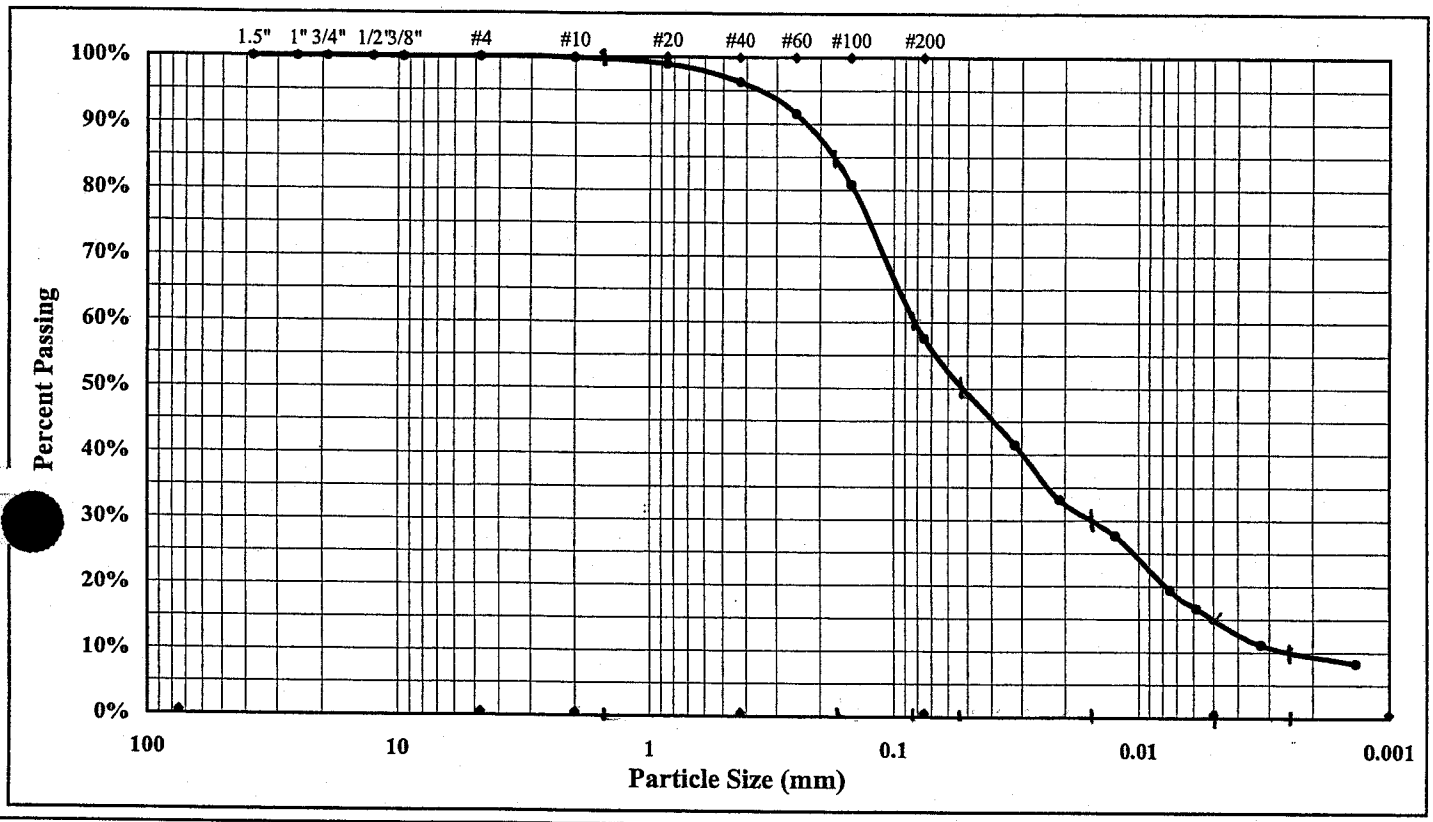
ASTM D 422

S&ME

[redacted] Project #: **1264-02-576**
Project Name: **Duke Power - Belews Creek Landfill**
Client Name: Duke Energy Corp.
Client Address: 526 South Church S, Charlotte, NC 28202

Report Date: January 9, 2003
Test Date(s): 1/2 - 1/9/03

Boring #:	BC-3	Sample #:		Sample Date:	NA
Location:	Scrubber Waste L.F.	Offset:		Depth (ft):	0.0 - 10.0
Sample Description:	Brown Fine Sandy Silty CLAY (CL)				



Cobbles	< 300 mm (12") and > 75 mm (3")	Fine Sand	< 0.425 mm and > 0.075 mm (#200)
Gravel	< 75 mm and > 4.75 mm (#4)	Silt	< 0.075 and > 0.005 mm
Coarse Sand	< 4.75 mm and >2.00 mm (#10)	Clay	< 0.005 mm
Medium Sand	< 2.00 mm and > 0.425 mm (#40)	Colloids	< 0.001 mm

Moisture Content	5.8%	Gravel	0.0%	Liquid Limit	37
Silt & Clay (% Passing #200)	57.6%	Sand	42.4%	Plastic Limit	20
Maximum Particle Size	#10	Silt	42.7%	Plastic Index	17
Specific Gravity	2.761	Clay	14.9%	Colloids	8.3%

Description of Sand & Gravel Particles: Rounded ☐ Angular ☐ Hard & Durable ☐ Soft ☐ Weathered & Friable ☐

ASTM D 422: Particle Size Analysis of Soils — Dispersing Agent: Sodium Hexametaphosphate: 40 g./ Liter; Mechanical Stirring Apparatus (A); Length of Dispersion Period: 1 min.

References: ASTM D 854: Specific Gravity of Soils
[redacted]M D 421: Dry Preparation of Soil Samples
[redacted]M D 2216: Laboratory Determination of Water (Moisture) Content of Soil and Rock by Mass
A[redacted]D 4318: Liquid Limit, Plastic Limit, & Plastic Index of Soils
A[redacted]D 2487: Classification of Soils for Engineering Purposes (Unified Soil Classification System)

Technical Responsibility / Position: Mal Krajan — Laboratory Supervisor

Laboratory Report Version 4.0

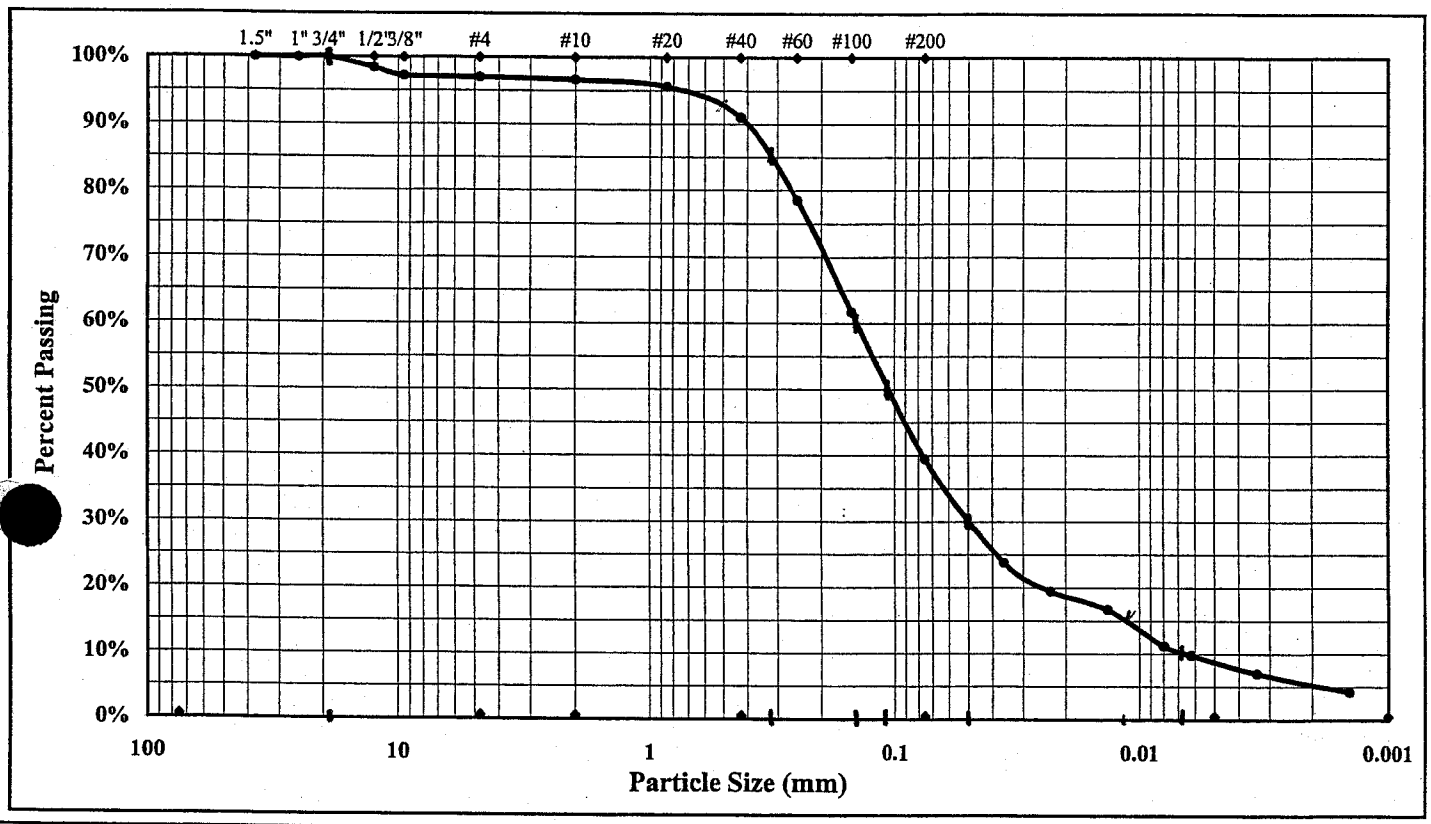
Particle Size Analysis of Soils

ASTM D 422

S&ME

[redacted]E Project #:	**1264-02-576**	Report Date:	January 9, 2003
Project Name:	**Duke Power - Belews Creek Landfill**	Test Date(s):	1/2 - 1/7/03
Client Name:	Duke Energy Corp.		
Client Address:	526 South Church Street, Charlotte, NC 28202		

Boring #:	BC-10	Sample #:		Sample Date:	NA
Location:	Scrubber Waste L.F.	Offset:		Depth (ft):	0.0 - 10.0
Sample Description:	Yellow Clayey Silty Fine SAND with a trace of Gravel				



Cobbles	< 300 mm (12") and > 75 mm (3")	Fine Sand	< 0.425 mm and > 0.075 mm (#200)
Gravel	< 75 mm and > 4.75 mm (#4)	Silt	< 0.075 and > 0.005 mm
Coarse Sand	< 4.75 mm and >2.00 mm (#10)	Clay	< 0.005 mm
Medium Sand	< 2.00 mm and > 0.425 mm (#40)	Colloids	< 0.001 mm

Moisture Content	9.3%	Gravel	3.0%	Liquid Limit	24
Silt & Clay (% Passing #200)	39.5%	Sand	57.5%	Plastic Limit	21
Maximum Particle Size	3/4"	Silt	31.6%	Plastic Index	3
Specific Gravity	2.652	Clay	7.9%	Colloids	4.2%

Description of Sand & Gravel Particles: Rounded ☐ Angular ☐ Hard & Durable ☐ Soft ☐ Weathered & Friable ☐

ASTM D 422: Particle Size Analysis of Soils — Dispersing Agent: Sodium Hexametaphosphate: 40 g./ Liter; Mechanical Stirring Apparatus (A); Length of Dispersion Period: 1 min.

References: ASTM D 854: Specific Gravity of Soils
[redacted]M D 421: Dry Preparation of Soil Samples
[redacted]M D 2216: Laboratory Determination of Water (Moisture) Content of Soil and Rock by Mass
[redacted]D 4318: Liquid Limit, Plastic Limit, & Plastic Index of Soils
ASTM D 2487: Classification of Soils for Engineering Purposes (Unified Soil Classification System)

Technical Responsibility / Position: Mal Krajan — Laboratory Superviso

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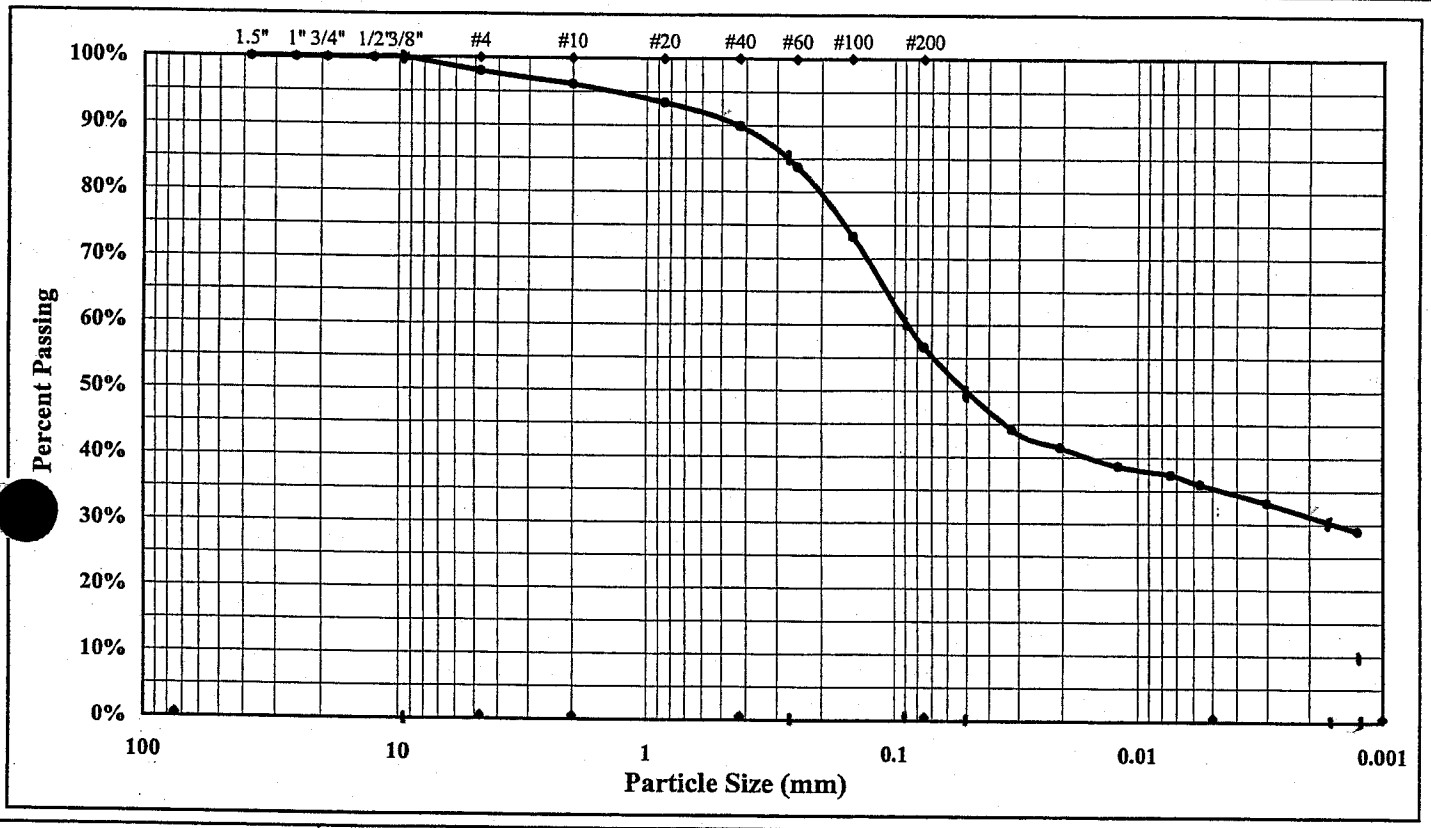
Particle Size Analysis of Soils

ASTM D 422

S&ME

S&ME Project #:	1264-02-576	Report Date:	January 9, 2003
Project Name:	**Duke Power - Belews Co. Landfill**	Test Date(s):	37623
Client Name:	Duke Energy Corp.		
Client Address:	526 S. Church St.		

Boring #:	BC-11	Sample #:		Sample Date:	not reported
Location:	Scrubber Waste	Offset:		Depth (ft):	0-10
Sample Description:	Red Silty Clayey Sand with a trace of Gravel				



Cobbles	< 300 mm (12") and > 75 mm (3")	Fine Sand	< 0.425 mm and > 0.075 mm (#200)
Gravel	< 75 mm and > 4.75 mm (#4)	Silt	< 0.075 and > 0.005 mm
Coarse Sand	< 4.75 mm and >2.00 mm (#10)	Clay	< 0.005 mm
Medium Sand	< 2.00 mm and > 0.425 mm (#40)	Colloids	< 0.001 mm

Moisture Content	19.4%	Gravel	2.0%	Liquid Limit	47
Silt & Clay (% Passing #200)	56.6%	Sand	41.4%	Plastic Limit	26
Maximum Particle Size	#4	Silt		Plastic Index	21
Specific Gravity	2.716	Clay		Colloids	29.0%

Description of Sand & Gravel Particles: Rounded ☐ Angular ☐ Hard & Durable ☐ Soft ☐ Weathered & Friable ☐

ASTM D 422: Particle Size Analysis of Soils

Dispersing Agent: Sodium Hexametaphosphate: 40 g./ Liter

Mechanical Stirring Apparatus (A) Length of Dispersion Period: 1 min.

References: ASTM D 854: Specific Gravity of Soils

ASTM D 421: Dry Preparation of Soil Samples

ASTM D 2216: Laboratory Determination of Water (Moisture) Content of Soil and Rock by Mass

ASTM D 4318: Liquid Limit, Plastic Limit, & Plastic Index of Soils

ASTM D 2487: Classification of Soils for Engineering Purposes (Unified Soil Classification System)

Technical Responsibility / Position: Mal Krajan — Laboratory Supervisor

Laboratory Report Version 4.0

Particle Size Analysis of Soils

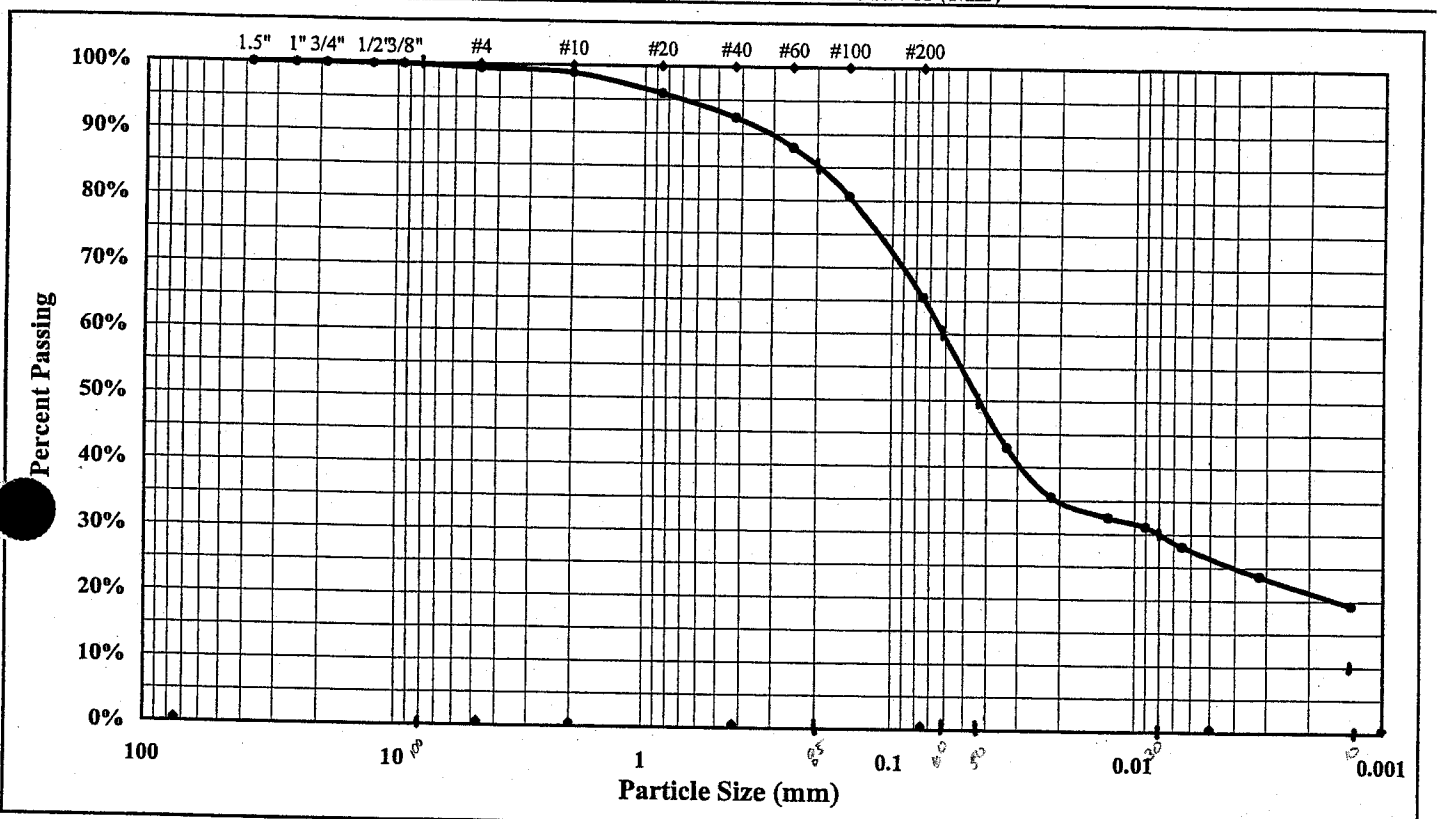
ASTM D 422

S&ME

S&ME Project #:	1264-02-576	Report Date:	January 9, 2003
Project Name:	**Duke Power - Belews Co. Landfill**	Test Date(s):	1/2 - 1/8/03
Client Name:	Duke Energy Corp.		
Client Address:	526 S. Church St.		

Boring #:	BC-13	Sample #:	Bag-1	Sample Date:	NA
Location:	Scrubber Waste L.F.	Offset:		Depth (ft):	0.0 - 10.0

Sample Description: Red Fine Sandy Clayey SILT with a Trace of Gravel (ML)



Cobbles	< 300 mm (12") and > 75 mm (3")	Fine Sand	< 0.425 mm and > 0.075 mm (#200)
Gravel	< 75 mm and > 4.75 mm (#4)	Silt	< 0.075 and > 0.005 mm
Coarse Sand	< 4.75 mm and >2.00 mm (#10)	Clay	< 0.005 mm
Medium Sand	< 2.00 mm and > 0.425 mm (#40)	Colloids	< 0.001 mm

Moisture Content	11.1%	Gravel	0.6%	Liquid Limit	38
Silt & Clay (% Passing #200)	65.5%	Sand	33.9%	Plastic Limit	28
Maximum Particle Size	3/8"	Silt		Plastic Index	10
Specific Gravity	2.672	Clay		Colloids	19.3%

Description of Sand & Gravel Particles: Rounded ☐ Angular ☐ Hard & Durable ☐ Soft ☐ Weathered & Friable ☐

ASTM D 422: Particle Size Analysis of Soils — Dispersing Agent: Sodium Hexametaphosphate: 40 g./ Liter; Mechanical Stirring Apparatus (A); Length of Dispersion Period: 1 min.

References: ASTM D 854: Specific Gravity of Soils
ASTM D 421: Dry Preparation of Soil Samples
ASTM D 2216: Laboratory Determination of Water (Moisture) Content of Soil and Rock by Mass
ASTM D 4318: Liquid Limit, Plastic Limit, & Plastic Index of Soils
ASTM D 2487: Classification of Soils for Engineering Purposes (Unified Soil Classification System)

Technical Responsibility / Position: Mal Krajan — Quality System Coordinator

Laboratory Report Version 4.2

Particle Size Analysis of Soils

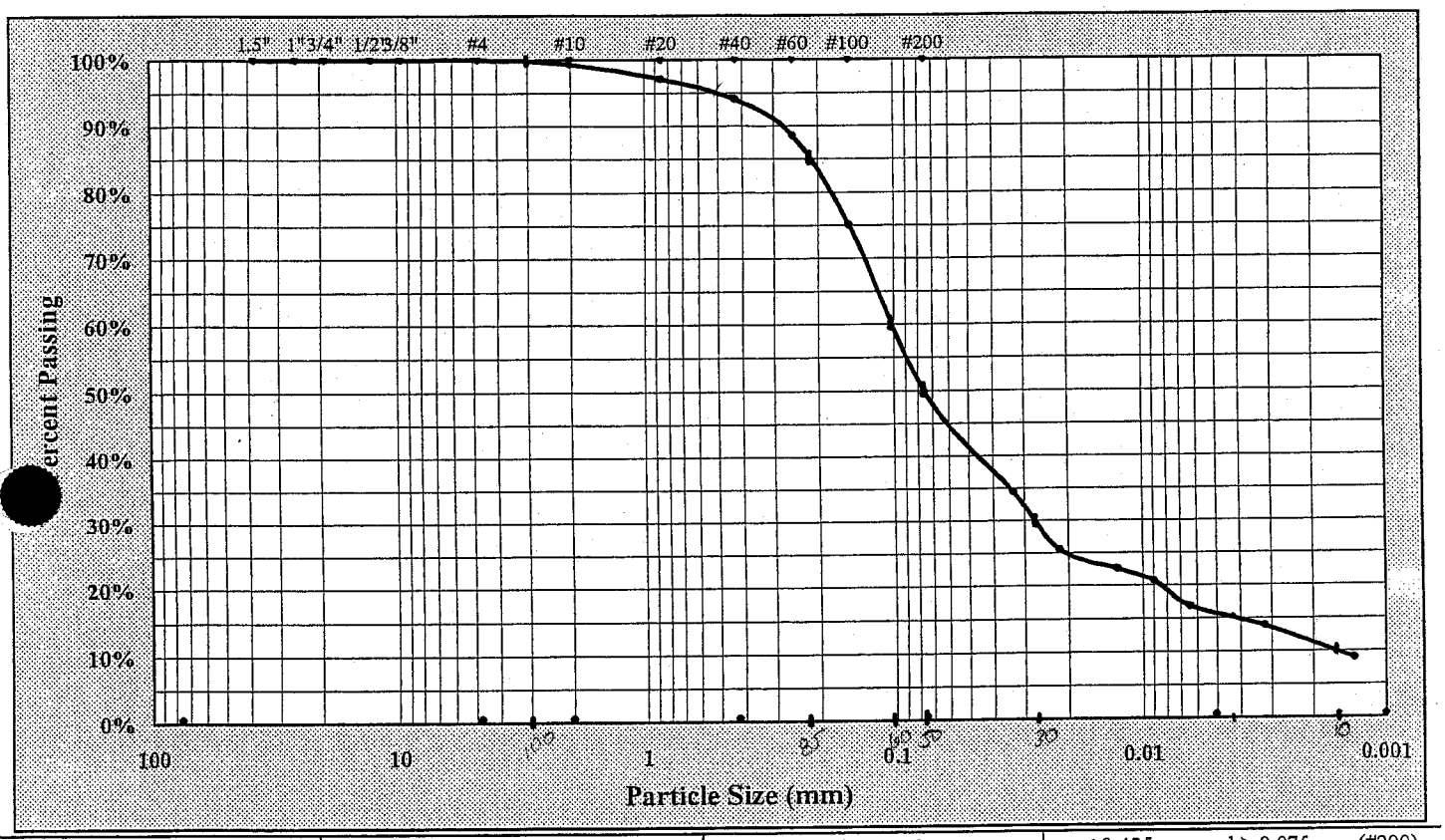
S&ME

ASTM D 422

S&ME Project #: **1054-04-955**
Project Name: **Belews Creek Steam Plant**
Client Name: Duke Energy
Client Address:

Report Date: 09/21/2004
Test Date(s): 09/13 - 09/17/2004

Boring #:	BC-16	Sample #:	Bag-1	Sample Date:	9/1/04
Location:	Site Borehole	Offset:	NA	Depth (ft):	0 - 5'
Sample Description:	Tan-Brown Silty SAND (SM)				



Cobbles	< 300 mm (12") and > 75 mm (3")	Fine Sand	< 0.425 mm and > 0.075 mm (#200)
Gravel	< 75 mm and > 4.75 mm (#4)	Silt	< 0.075 and > 0.005 mm
Coarse Sand	< 4.75 mm and >2.00 mm (#10)	Clay	< 0.005 mm
Medium Sand	< 2.00 mm and > 0.425 mm (#40)	Colloids	< 0.001 mm

Maximum Particle Size	#4	Gravel	0.0%	Silt	34.1%
Silt & Clay (% Passing #200)	49.7%	Sand	50.3%	Clay	15.6%
Apparent Relative Density	2.708	Moisture Content	42.1%	Colloids	8.8%
Liquid Limit	30	Plastic Limit	22	Plastic Index	8

Description of Sand & Gravel Particles Rounded ☐ Angular ☐ Hard & Durable ☐ Soft ☐ Weathered & Friable ☐

Mechanical Stirring Apparatus (A) Length of Dispersion Period: 1 min. Dispersing Agent: Sodium Hexametaphosphate: 40 g./ Liter

References: ASTM D 422: Particle Size Analysis of Soils
ASTM D 421: Dry Preparation of Soil Samples
ASTM D 4318: Liquid Limit, Plastic Limit, & Plastic Index of Soils
ASTM D 854: Specific Gravity of Soils
ASTM D 2487: Classification of Soils for Engineering Purposes (Unified Soil Classification System)

Technical Responsibility: Mal Krajan — Laboratory Supervisor
(Signature) (Position)
S&ME, INC. 3109 Spring Forest Road, Raleigh, N.C. 27616
BC-16 Bag-1 Classification.xls

Laboratory Report Version 4.2

Particle Size Analysis of Soils

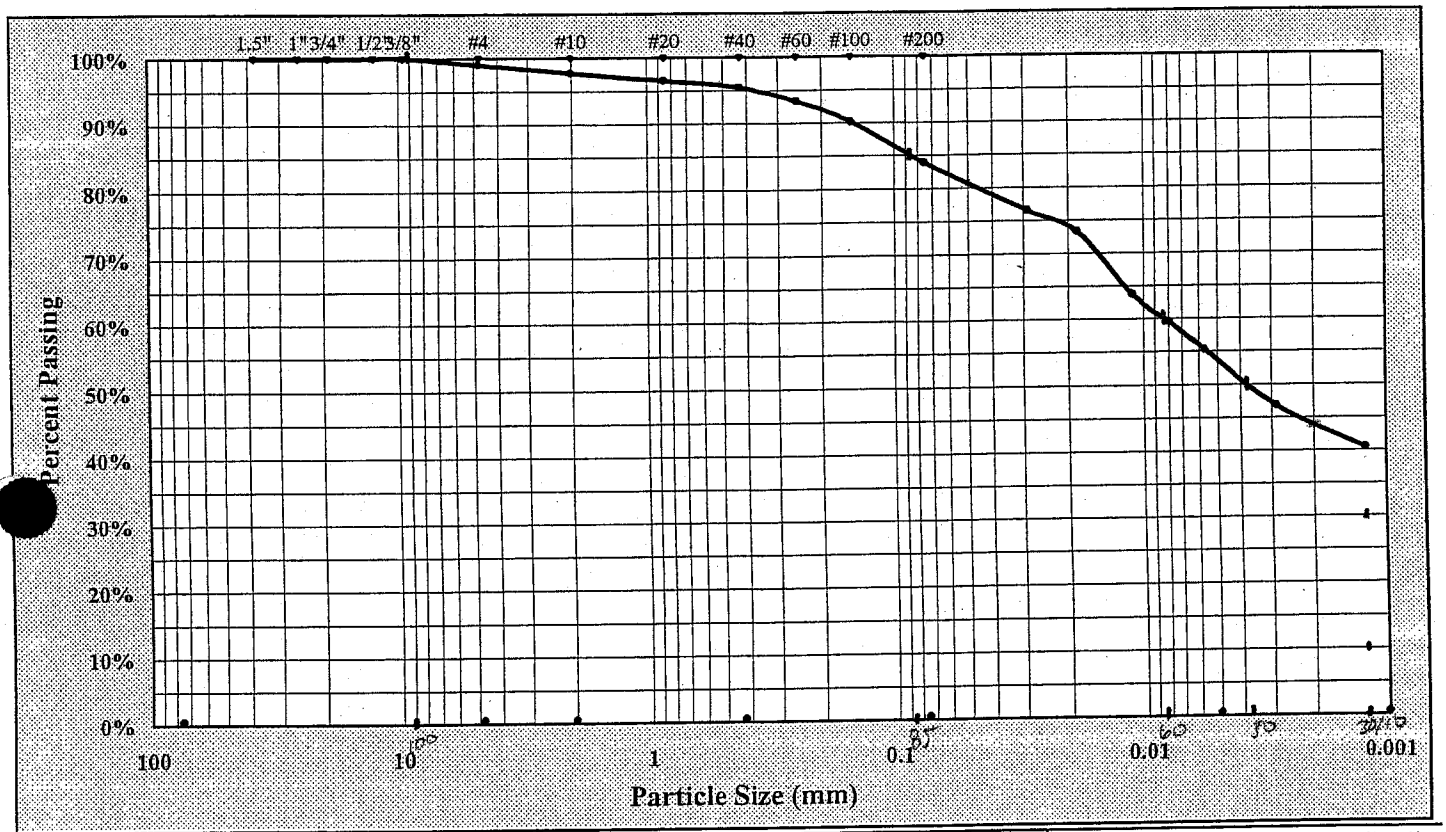
ASTM D 422

S&ME

[redacted] Project #: **1054-04-955**
Project Name: **Belew Creek Steam Plant**
Client Name: Duke Energy
Client Address:

Report Date: 10/08/2004
Test Date(s): 09/28 - 10/07/2004

Boring #:	BC-19	Sample #:	Bag-1	Sample Date:	9/1/04
Location:	Site Borehole	Offset:	NA	Depth (ft):	2 - 7'
Sample Description:	Orange-Brown Sandy SILT (MH)				



Cobbles	< 300 mm (12") and > 75 mm (3")	Fine Sand	< 0.425 mm and > 0.075 mm (#200)
Gravel	< 75 mm and > 4.75 mm (#4)	Silt	< 0.075 and > 0.005 mm
Coarse Sand	< 4.75 mm and >2.00 mm (#10)	Clay	< 0.005 mm
Medium Sand	< 2.00 mm and > 0.425 mm (#40)	Colloids	< 0.001 mm

Maximum Particle Size	3/8"	Gravel	1.0%	Silt	30.8%
Silt & Clay (% Passing #200)	83.8%	Sand	15.2%	Clay	53.0%
Apparent Relative Density	2.735	Moisture Content	37.1%	Colloids	40.3%
Liquid Limit	72	Plastic Limit	44	Plastic Index	28

Description of Sand & Gravel Particles Rounded ☐ Angular ☐ Hard & Durable ☐ Soft ☐ Weathered & Friable

Mechanical Stirring Apparatus (A) Length of Dispersion Period: 1 min. Dispersing Agent: Sodium Hexametaphosphate: 40 g./ Liter

References: ASTM D 422: Particle Size Analysis of Soils; ASTM D 421: Dry Preparation of Soil Samples
ASTM D 4318: Liquid Limit, Plastic Limit, & Plastic Index of Soils; ASTM D 854: Specific Gravity of Soils
ASTM D 2487: Classification of Soils for Engineering Purposes (Unified Soil Classification System)

Technical Responsibility: Mal Krajan — Laboratory Supervisor
S&ME, INC. — 3109 Spring Forest Road, Raleigh, N.C. 27616 — BC-19 Bag-1 Classification.xls

Particle Size Analysis of Soils

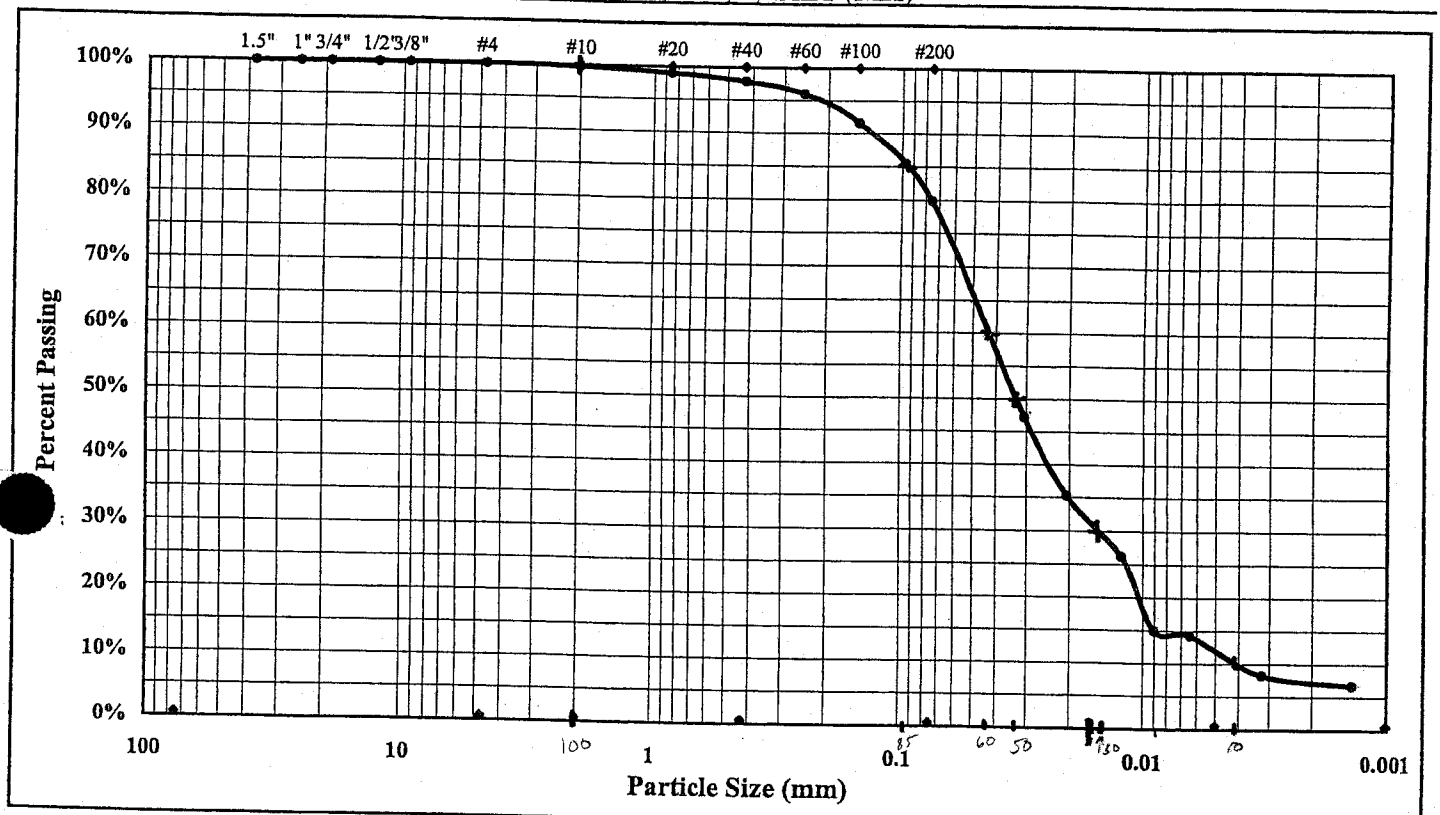
ASTM D 422

S&ME

ME Project #: **1264-02-576**
Project Name: **Duke Power-Belews Creek Landfill**
Client Name:
Client Address:

Report Date: April 1, 2003
Test Date(s): 3/26 - 4/1/03

Boring #: BC-10 Sample #: UD-1 Sample Date: NA
Location: Offset: Depth (ft): 12.0 - 14.0
Sample Description: Yellow and Tan-Brown Sandy Clayey SILT (MH)



Cobbles	< 300 mm (12") and > 75 mm (3")	Fine Sand	< 0.425 mm and > 0.075 mm (#200)
Gravel	< 75 mm and > 4.75 mm (#4)	Silt	< 0.075 and > 0.005 mm
Coarse Sand	< 4.75 mm and >2.00 mm (#10)	Clay	< 0.005 mm
Medium Sand	< 2.00 mm and > 0.425 mm (#40)	Colloids	< 0.001 mm

Moisture Content		Gravel	0.0%	Liquid Limit	59
Silt & Clay (% Passing #200)	79.9%	Sand	20.1%	Plastic Limit	45
Maximum Particle Size	#4	Silt	67.9%	Plastic Index	14
Specific Gravity	2.797	Clay	12.0%	Colloids	6.7%

Description of Sand & Gravel Particles: Rounded ☐ Angular ☐ Hard & Durable ☐ Soft ☐ Weathered & Friable ☐

ASTM D 422: Particle Size Analysis of Soils
Dispersing Agent: Sodium Hexametaphosphate: 40 g./ Liter
Mechanical Stirring Apparatus (A) Length of Dispersion Period: 1 min.

References: ASTM D 854: Specific Gravity of Soils
ASTM D 421: Dry Preparation of Soil Samples
ASTM D 2216: Laboratory Determination of Water (Moisture) Content of Soil and Rock by Mass
ASTM D 4318: Liquid Limit, Plastic Limit, & Plastic Index of Soils
ASTM D 2487: Classification of Soils for Engineering Purposes (Unified Soil Classification System)

Technical Responsibility / Position: Mal Krajan — Laboratory Supervisor

Particle Size Analysis of Soils

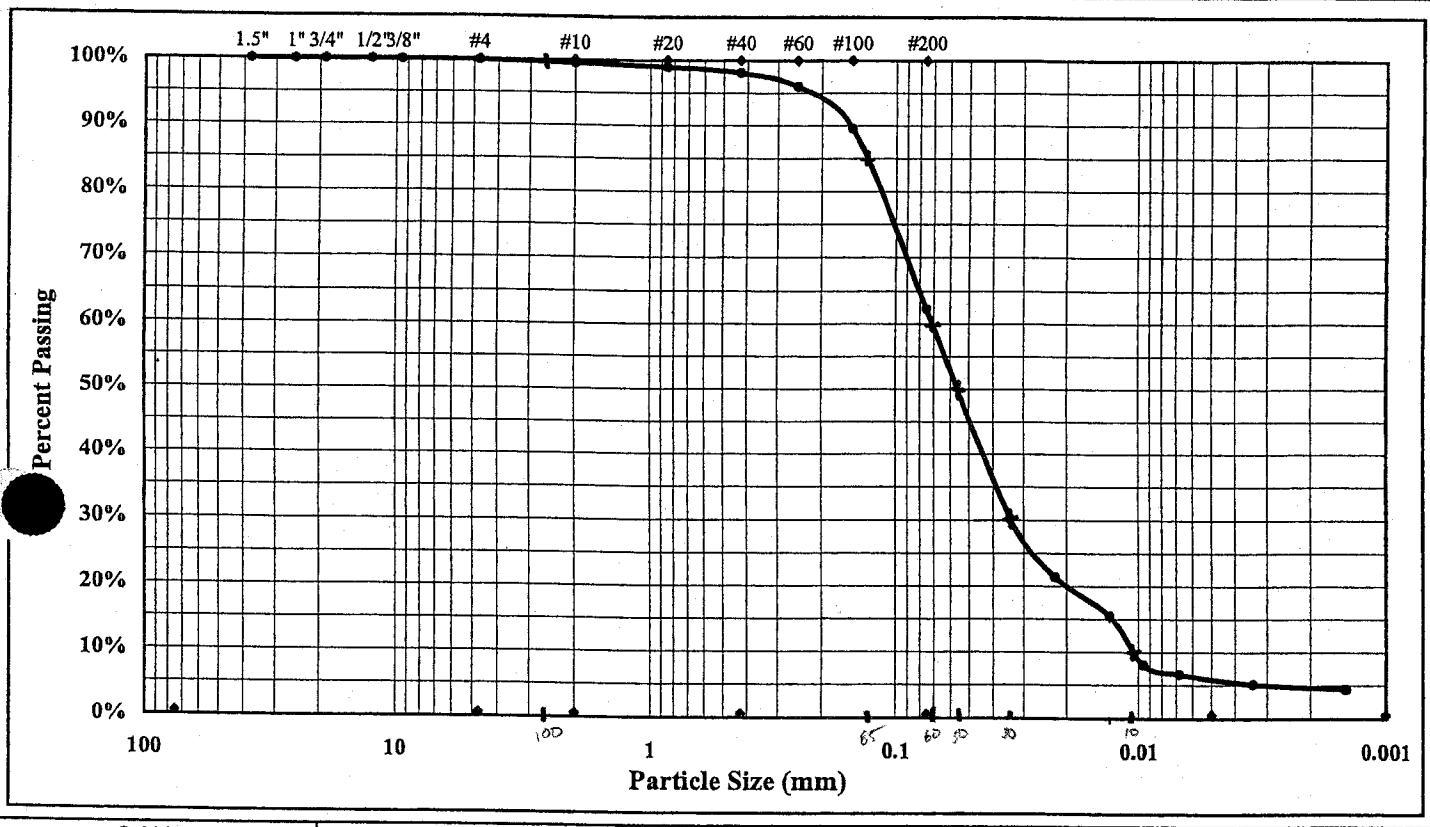
ASTM D 422

S&ME

ME Project #: 1264-02-576
Project Name: Duke Power-Belew's Creek Landfill
Client Name:
Client Address:

Report Date: April 1, 2003
Test Date(s): 3/25/2003-4/1/03

Boring #: BC-12 Sample #: UD-1 Sample Date: NA
Location: Offset: Depth (ft): 27.0 - 28.0
Sample Description: Tan Sandy Clayey SILT (ML)



Cobbles	< 300 mm (12") and > 75 mm (3")	Fine Sand	< 0.425 mm and > 0.075 mm (#200)
Gravel	< 75 mm and > 4.75 mm (#4)	Silt	< 0.075 and > 0.005 mm
Coarse Sand	< 4.75 mm and >2.00 mm (#10)	Clay	< 0.005 mm
Medium Sand	< 2.00 mm and > 0.425 mm (#40)	Colloids	< 0.001 mm

Moisture Content		Gravel	0.1%	Liquid Limit	32
Silt & Clay (% Passing #200)	62.3%	Sand	37.6%	Plastic Limit	26
Maximum Particle Size	#20	Silt	56.3%	Plastic Index	6
Specific Gravity	2.654	Clay	6.0%	Colloids	4.4%

Description of Sand & Gravel Particles: Rounded ☐ Angular ☐ Hard & Durable ☐ Soft ☐ Weathered & Friable ☐

ASTM D 422: Particle Size Analysis of Soils Dispersing Agent: Sodium Hexametaphosphate: 40 g./ Liter
Mechanical Stirring Apparatus (A) Length of Dispersion Period: 1 min.

References: ASTM D 854: Specific Gravity of Soils
ASTM D 421: Dry Preparation of Soil Samples
ASTM D 2216: Laboratory Determination of Water (Moisture) Content of Soil and Rock by Mass
ASTM D 4318: Liquid Limit, Plastic Limit, & Plastic Index of Soils
ASTM D 2487: Classification of Soils for Engineering Purposes (Unified Soil Classification System)

Technical Responsibility / Position: Mal Krajan — Laboratory Supervisor

Geonet Design

- Discussion and Results of Geonet Design
- Calculation for Geonet for Liner System of Landfill
- Calculation for Geonet for Cover System of Landfill
- Copies of References

Geonet Design – Discussion and Results

The following pages present the design for the geonet layer to be used with the liner system and with the cover system for the landfill.

Results

The results of the design are presented in the following table.

	Minimum Transmissivity (m^2/sec)	Normal Stress (pounds. ft^2)
Geonet used with Liner	3.7×10^{-4}	8500
Geonet used with Cover	1.7×10^{-4}	1000

HELP Analyses - Summary of Volume for Leachate and Precipitation Collected by Leachate Collection System (LCS)
September 12, 2005

Case Number	HELP Run Title	Case Description	Area (acres)	Peak Daily Runoff (cubic feet)	Peak Daily Drainage into LCS (cubic feet)	Total Peak Daily Flow into LCS (cubic feet)	Total Peak Daily Flow into LCS (gal/ac/day)
1	Belews Creek FGD Landfill 10' FGD Waste - Cell 1 Only	Cell 1 Open - 10 feet FGD waste placed in cell	5.7	23,879	3,270	27,149	35,627
2	Belews Creek FGD Landfill FGD Waste - Cell 1 + Cell 2	Cell 1 and Cell 2 - Height of waste = 66.3' (max. height from 3H:1V slope on side slopes)	12.7	26,431	5,067	31,498	18,552
3	Belews Creek FGD Landfill - Waste to 890' - All Cells-Open	Waste in all cells up to final grades; no synthetic cap	24	49,949	6,856	56,805	17,704
4	Belews Creek FGD Landfill Closed Condition	Waste in all cells up to final grades; synthetic cap in place	24				
		Synthetic Cap (on top of landfill)		N/A	N/A	N/A	
		Synthetic Liner (beneath waste)		N/A	6,023	6,023	1,877

Belews Creek FGD Residue Landfill	**Job No.**	**046119**			
Geonet Transmissivity Calculations	**Comp. By**	**WMM**	**Date**	**9/28/2005**	
Bottom of Landfill	**Chkd. By**	DED	**Date**	9/30/05	
	Sheet	1	**of**	3	

Objective: Verify that Geonet Provides Adequate Planar Flow
Along bottom of landfill

References: 1) *Geosynthetic Design Guidance for Hazardous Waste Landfills and Surface Impoundments*, Richardson, Example 3.1

2) *Designing With Geosynthetics*, 3rd Edition, Koerner, 1994.

Calculate Minimum Allowable Geonet Transmissivity (Reference 1)

$$\Theta_{min} = \frac{B^2 q}{hmax + B\sin\Phi}$$

Θ = geonet transmissivity
B = pipe spacing ÷ 2, ft
q = leachate quantity, gal/acre/day (From HELP)
hmax = allowable head on liner, ft
Φ = slope of geonet, degrees

B =	75	ft	
q =	35,627	gal/ac/day	
hmax =	1.0	ft	
Φ =	1% =	0.5729	degrees

convert q into $ft^3/ft^2/day$

q =	35,627	gal/ac/day
q =	0.109	$ft^3/ft^2/day$

Θ_{min} =	111.8	ft^2/day
Θ_{min} =	1.20E-04	m^2/sec

Belews Creek FGD Residue Landfill	**Job No.**	**046119**			
Geonet Transmissivity Calculations	**Comp. By**	**WMM**	**Date**	**9/28/2005**	
Bottom of Landfill	**Chkd. By**	DEB	**Date**	9/30/05	
	Sheet	2	**of**	3	

Calculate the Normal Stress

Operational Cover:

Density	125	lb/ft^3	(assumed)
Thickness	1	ft	

Waste:

Density	89	lb/ft^3	
Thickness	89	ft	measured from top of operational cover

Final Cover

Density	125	lb/ft^3	(assumed)
Thickness	3.5	ft	

Normal Stress = height of material x unit weight of material

Normal Stress = 8483.5 lb/ft^2

Calculate Hydraulic Gradient

$$i = \frac{hmax + B\sin\Phi}{B/\cos\Phi}$$

i = 0.023

Estimate Actual Transmissivity

from Figure 4.8, Reference 2, the flow rate considering the normal stress

is = 0.38 gal/min-ft

$$q_{allowable} = \frac{1}{FSin \times FScr \times FScc\ FSbc} \times q_{ult}$$ Reference 2, Table 4.2

$$q_{allowable} = \frac{1}{1.75 \times 1.7 \times 1.75 \times 1.75} \times .38$$

$q_{allowable}$ = 0.042 gal/min-ft

Belews Creek FGD Residue Landfill
Geonet Transmissivity Calculations
Bottom of Landfill

Job No.	**046119**		
Comp. By	**WMM**	**Date**	**9/28/2005**
Chkd. By	DEB	**Date**	9/30/05
Sheet	3	**of**	3

Convert to Transmissivity : $q/w = i \times \Theta$

$\Theta_{actual} = q/(w \times i)$

w = width, in this case 1 ft.
Reference 2, Eqn 4.7

$$\Theta_{actual} = \frac{0.042}{0.023} \text{ gal/min-ft}$$

$\Theta_{actual} =$ 1.79 gal/min-ft
$=$ 3.72E-04 m^2/sec

Check $\Theta_{actual} > \Theta_{min}$

$\Theta_{actual} =$ 3.72E-04 m^2/sec

$\Theta_{min} =$ 1.20E-04 m^2/sec

OK

Factor of Safety

$$FS = \frac{\Theta_{actual}}{\Theta_{min}}$$

$$FS = \frac{3.72E\text{-}04 \text{ } m^2/sec}{1.20E\text{-}04 \text{ } m^2/sec}$$

FS = 3.1

Belews Creek FGD Residue Landfill
Geonet Transmissivity Calculations
Cover of Landfill

Job No.	**046119**		
Comp. By	**WMM**	**Date**	**9/28/2005**
Chkd. By	**DEb**	**Date**	**9/30/05**
Sheet	**1**	**of**	**3**

Objective: Verify that Geonet Provides Adequate Planar Flow Geonet for Cover of Landfill

References: 1) *Geosynthetic Design Guidance for Hazardous Waste Landfills and Surface Impoundments*, Richardson, Example 3.1

2) *Designing With Geosynthetics*, 3rd Edition, Koerner, 1994.

Calculate Minimum Allowable Geonet Transmissivity (Reference 1)

$$\Theta_{min} = \frac{B^2 q}{hmax + B\sin\Phi}$$

Θ = geonet transmissivity
B = pipe spacing ÷ 2, ft
q = leachate quantity, gal/acre/day (From HELP)
hmax = allowable head on liner, ft
Φ = slope of geonet, degrees

B =	120	ft	
q =	1,877	gal/ac/day	
hmax =	1.0	ft	
Φ =	25% =	14.04	degrees

convert q into $ft^3/ft^2/day$

q =	1,877	gal/ac/day
q =	0.006	$ft^3/ft^2/day$

Θ_{min} =	1.3	ft^2/day
Θ_{min} =	1.43E-06	m^2/sec

Belews Creek FGD Residue Landfill
Geonet Transmissivity Calculations
Cover of Landfill

Job No.	**046119**			
Comp. By	**WMM**	**Date**	**9/28/2005**	
Chkd. By	DEB	**Date**	9/30/05	
Sheet	2	**of**	3	

Calculate the Normal Stress

Final Cover

Density 125 lb/ft^3 (assumed)
Thickness 3.5 ft

Normal Stress = height of material x unit weight of material

Normal Stress = 438 lb/ft^2

Calculate Hydraulic Gradient

$$i = \frac{hmax + B\sin\Phi}{B/\cos\Phi}$$

i = 0.243

Estimate Actual Transmissivity

from Figure 4.8, Reference 2, the flow rate considering the normal stress

is = 1.8 gal/min-ft

$$q_{allowable} = \frac{1}{FSin \times FScr \times FScc\ FSbc} \times q_{ult}$$ Reference 2, Table 4.2

$$q_{allowable} = \frac{1}{1.75 \times 1.7 \times 1.75 \times 1.75} \times 1.80$$

$q_{allowable}$ = 0.198 gal/min-ft

Convert to Transmissivity : $q/w = i \times \Theta$

$\Theta_{actual} = q/(w \times i)$ w = width, in this case 1 ft.
Reference 2, Eqn 4.7

$$\Theta_{actual} = \frac{0.198}{0.243}\ gal/min\text{-}ft$$

Belews Creek FGD Residue Landfill
Geonet Transmissivity Calculations
Cover of Landfill

Job No.	**046119**		
Comp. By	**WMM**	**Date**	**9/28/2005**
Chkd. By	DEB	**Date**	9/30/05
Sheet	3	**of**	3

Θ_{actual} = 0.81 gal/min-ft
= 1.69E-04 m^2/sec

Check $\Theta_{actual} > \Theta_{min}$

Θ_{actual} = 1.69E-04 m^2/sec

Θ_{min} = 1.43E-06 m^2/sec

OK

Factor of Safety

$$FS = \frac{\Theta_{actual}}{\Theta_{min}}$$

$$FS = \frac{1.69E\text{-}04\ m^2/sec}{1.43E\text{-}06\ m^2/sec}$$

FS = 118.3

Cell Component: LEACHATE COLLECTION/REMOVAL SYSTEMS

Consideration: TRANSMISSIVITY, VERIFY THAT LCR PROVIDES ADEQUATE PLANAR FLOW.

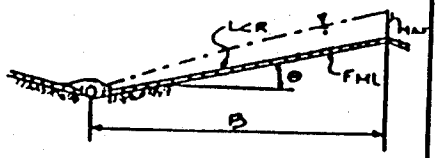
Required Material Properties	Range	Test	Standard
PLANAR FLOW CAPACITY	3×10^{-5} * (m^2/s)	TRANSMISSIVITY	ASTM D4631

* Draft MTG Minimum

Analysis Procedure:

(1) DEFINE MINIMUM TRANSMISSIVITY
- STATUTORY - 3×10^{-5} m^2/s
- 1-FOOT HEAD

$$T = \frac{B^2 \cdot q}{h_{max} + B \sin\theta}$$



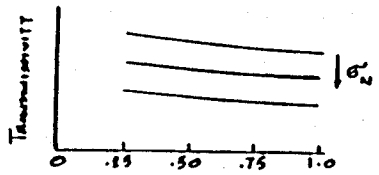
q = LEACHATE INFLOW RATE
h_{max} = 1 FOOT OF HEAD

(2) CALCULATE MAXIMUM NORMAL STRESS

$$\sigma_N = D \cdot \gamma$$

D = FINAL DEPTH CAP TO LCR
γ = UNIT WEIGHT OF FILL

(3) OBTAIN LABORATORY TRANSMISSIVITY DATA



FIELD GRADIENT, i

$$i = \Delta h/L = \frac{h_{max} + B \sin\theta}{(B/\cos\theta)}$$

(4) DEFINE DESIGN RATIO

$$DR = \frac{T_{LCR}}{T_{REQUIRED}}$$

Design Ratio:

$DR_{min} \geq 10$ DUE TO COMPRESSIVE CREEP IN SYNTHETIC LCR

References:

DEMETRACOPOULOUS (1984)
WONG (1977)
SCHARCH (1981)

Example:

GIVEN:
- LANDFILL HT, D = 120'
- UNIT WT. WASTE, γ = 80 lb/ft^3
- SLOPE ANGLE, θ = 4°
- SLOPE LENGTH, B = 60'
- LEACHATE INFLOW RATE, q = 0.01 ft/day
- LABORATORY TRANSMISSIVITY DATA

(1) DEFINE MINIMUM TRANSMISSIVITY (FLOW CRITERIA)

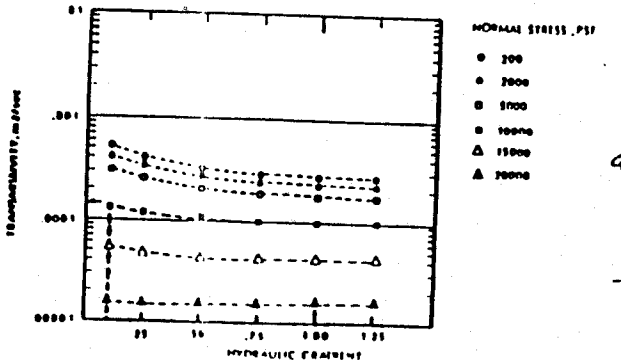
$$T = \frac{60^2 * 0.01}{1 + 60 \sin 4^\circ} = 6.94 \text{ ft}^2/\text{day} = 7.46 \times 10^{-6} \text{ m}^2/\text{sec}$$

$$\therefore T_{REQ} = 7.46 \times 10^{-6} \text{ m}^2/\text{s} \quad (> 3 \times 10^{-5} \text{ m}^2/\text{s})$$

(2) CALCULATE MAXIMUM NORMAL STRESS, σ_N

$$\sigma_N = 120 \cdot 80 = 9600 \text{ PSF}$$

(3) OBTAIN LCR TRANSMISSIVITY FROM LAB. DATA, T_{LCR}



GRADIENT, $i = \frac{1 \cdot 60 \sin 4^\circ}{(60/\cos 4^\circ)}$

$i = .086$

$T_{LCR} = .00014 \text{ m}^2/\text{s}$

(4) CALCULATE DESIGN RATIOS

$$DR_{FLOW} = \frac{T_{LCR}}{T_{REQ.}} = \frac{.00014}{7.46 \times 10^{-6}} = 18.8 \quad \text{OK}$$

$$DR_{MTG} = \frac{T_{LCR}}{T_{MTG}} = \frac{.00014}{3 \times 10^{-5}} = 4.67 \quad \text{MARGINAL}$$

Example No. 3.1

REF: Richardson, Koerner, Geosynthetic Design Guidance

Reference 1

Reference 2

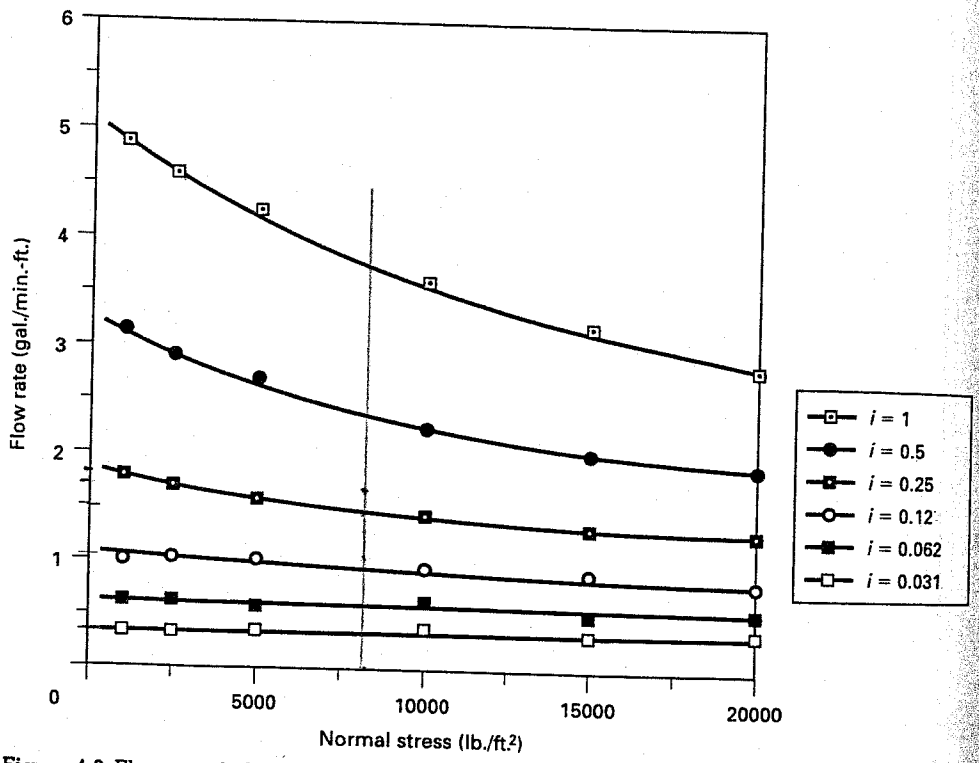


Figure 4.8 Flow rate behavior of a 0.25-in. (6.3-mm) geonet sandwiched between a 16-oz./yd.² (540-g/m²) nonwoven needle-punched geotextile with clay above and a 60-mil (1.5-mm) HDPE geomembrane below.

met with the typical flow regime in a geonet. Yet current EPA Leak Detection regulations [2] state the following:

- For landfills and waste piles, the geonet's transmissivity must be

$$\theta \geq 3 \times 10^{-5} \text{ m}^2/\text{sec}.$$

- For surface impoundments, the geonet's transmissivity must be

$$\theta \geq 3 \times 10^{-4} \text{ m}^2/\text{sec}.$$

One converts from flow rate per unit width to transmissivity as follows:

$$q = kiA \tag{4.1}$$

$$q = ki(t \times W)$$

$$q/W = i(k \times t)$$

$$q/W = i\theta \tag{4.2}$$

generated value is an ultimate value which, using ASTM D4716 for flow rate determination, must be reduced before use in design; that is,

$$q_{allow} < q_{ult}$$

One way of doing this is to ascribe partial factors of safety on each of the items not adequately assessed in the laboratory test. For example,

$$q_{allow} = q_{ult}\left[\frac{1}{FS_{IN} \times FS_{CR} \times FS_{CC} \times FS_{BC}}\right] \tag{4.5}$$

or if all of the partial factors of safety are lumped together,

$$q_{allow} = q_{ult}\left[\frac{1}{\Sigma\, FS_p}\right] \tag{4.6}$$

where q_{ult} = the flow rate determined from ASTM D4716 for short-term tests between solid plates using water as the transported liquid under laboratory test temperatures,

q_{allow} = the allowable flow rate to be used in Equation 4.3 for final design purposes,

FS_{IN} = the factor of safety for elastic deformation, or intrusion, of the adjacent geosynthetics into the geonet's core space,

FS_{CR} = the factor of safety for creep deformation of the geonet and/or adjacent geosynthetics into the geonet's core space,

FS_{CC} = the factor of safety for chemical clogging and/or precipitation of chemicals in the geonet's core space,

FS_{BC} = the factor of safety for biological clogging in the geonet's core space, and

$\Sigma\, FS_p$ = the *product* of all partial factors of safety for the site-specific conditions.

Some guidelines for various factors of safety to be used in different situations are given in Table 4.2. Example problems follow, which illustrate the use of geonets and point out that high factors of safety are warranted in critical situations. Please note that these values are based on preliminary and relatively sparse information. Other factors of safety, such as installation damage, temperature effects, and liquid turbidity, could also have been included. If needed they can be included on a site-specific basis. On the other hand, if the test method has included the particular item, it would appear in the foregoing formulation as a value of unity.

Example:

What is the allowable geonet flow rate to be used in the design of a capillary break beneath a roadway to prevent frost heave? Assume that laboratory testing was

Reference 2

Table 4.2 Recommended preliminary factor of safety values for determining allowable flow rate or transmissivity of geonets

	Partial Factor of Safety Value in Equation 4.5			
Application Area	FS_{IN}	FS_{CR}*	FS_{CC}	FS_{BC}
Sport fields	1.0 to 1.2	1.0 to 1.5	1.0 to 1.2	1.1 to 1.3
Capillary breaks	1.1 to 1.3	1.0 to 1.2	1.1 to 1.5	1.1 to 1.3
Roof and plaza decks	1.2 to 1.4	1.0 to 1.2	1.0 to 1.2	1.1 to 1.3
Retaining walls, seeping rock and soil slopes	1.3 to 1.5	1.2 to 1.4	1.1 to 1.5	1.0 to 1.5
Drainage blankets	1.3 to 1.5	1.2 to 1.4	1.0 to 1.2	1.0 to 1.2
Surface water drains for landfill caps	1.3 to 1.5	1.2 to 1.4	1.0 to 1.2	1.2 to 1.5
Secondary leachate collection (landfills)	1.5 to 2.0	1.4 to 2.0	1.5 to 2.0	1.5 to 2.0
Primary leachate collection (landfills)	1.5 to 2.0	1.4 to 2.0	1.5 to 2.0	1.5 to 2.0

*These values assume that the q_{ult} value was obtained using an applied normal pressure of 1.5 to 2 times the field-anticipated maximum value. If not, values must be increased.

done at the proper design load and hydraulic gradient and that this testing yielded a short-term between-rigid-plates value of 1.2 gal./min.-ft.

Solution: Since better information is not known, average values from Table 4.2 are used.

$$q_{allow} = q_{ult}\left[\frac{1}{FS_{IN} \times FS_{CR} \times FS_{CC} \times FS_{BC}}\right] \quad (4.5)$$

$$= 1.2\left[\frac{1}{1.1 \times 1.1 \times 1.1 \times 1.2}\right]$$

$$= 1.2\left[\frac{1}{1.60}\right]$$

$$= 0.75 \text{ gal./min.-ft.}$$

Example:

What is the allowable geonet flow rate to be used in the design of a secondary leachate collection system? Assume that laboratory testing at proper design load and proper hydraulic gradient gave a short-term between-rigid-plates value of 1.2 gal./min.-ft.

Solution: Average values from Table 4.2 are used; however, note the large reduction.

Anchor Trench Design

- Calculations for Anchor Trench Design
- Copies of References

Belews Creek FGD Residue Landfill
Anchor Trench Calculations

Job No.	**046119**		
Comp. By	**WMM**	**Date**	**9/28/2005**
Chkd. By	DEB	**Date**	9/28/05
Sheet	1	**of**	3

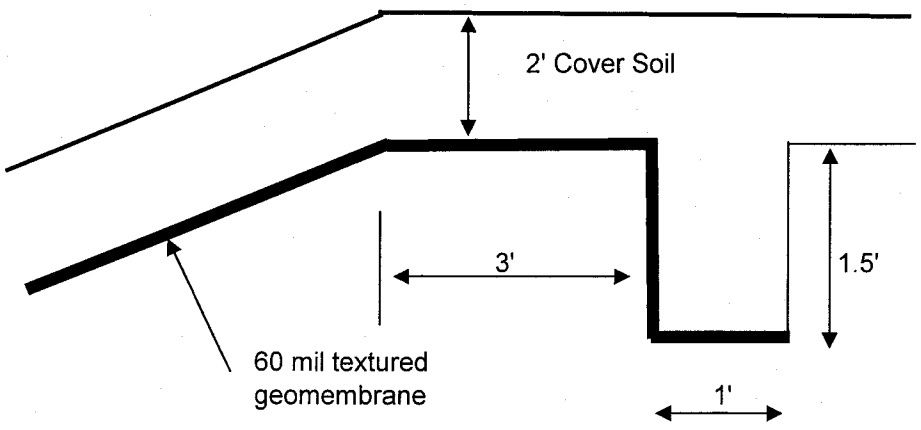
Objective: Determine Anchor Design Requirements

References: 1) *Geosynthetic Design Guidance for Hazardous Waste Landfills and Surface Impoundments*, Richardson, Example 3.17

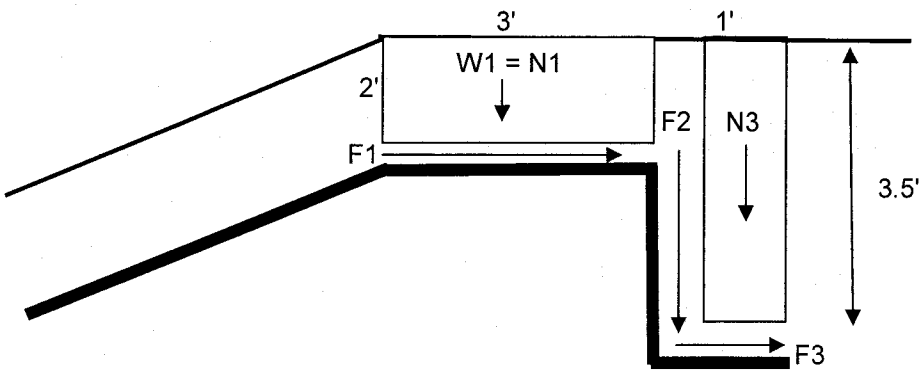
2) *Civil Engineering Reference Manual,* 4th Edition, Lindeburg

Given:	unit weight soil cover, γ =	110	lb/ft^3	(assumed)
	soil cover Φ =	30	degrees	
	textured geomembrane to subgrade friction angle, φ = (60 mil textured geomembrane)	22	degrees	

Detail of Anchor Trench



Resisting Forces:



Belews Creek FGD Residue Landfill	**Job No.**	**046119**			
Anchor Trench Calculations	**Comp. By**	**WMM**	**Date**	**9/28/2005**	
	Chkd. By	**DEB**	**Date**	**9/28/05**	
	Sheet	**2**	**of**	**3**	

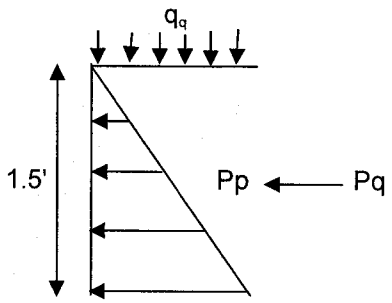
F1: Normal Force from Soil Block (2' x 3')

N1 tanφ = 2' x 3' x unit wt soil x tan φ

F1 = N1 tanφ = 2' x 3' x unit wt soil x tan φ

F1 = 266.7 lb/ft

F2: Friction From Normal Force of Surcharge Loaded Retaining Wall (Passive State)



q_q = 2' x unit weight of soil

q_q = 220 lb/ft2

$P_{horizontal}$ (passive) = $P_{vertical}$ $\tan^2(45 + \theta/2)$ Reference 2 , Eqn 10.55

$Pq = q_q H \tan^2(45 + \theta/2)$

soil cover Φ = 30 degrees

H = 1.5 ft

Pq = 990.0 lb/ft

$Pp = \frac{\gamma H^2}{2} \tan^2(45 + \theta/2)$

unit weight soil cover, γ = 110 lb/ft^3

Pp = 371.25 lb/ft

F2 = (Pq + Pp) tanφ

F2 = 550.0 lb/ft

F3: Normal Force from Soil Block (1' x 3.5')

F3 = N3 tanφ = 1' x 3.5' x unit wt soil x tan φ

F3 = 156 lb/ft

Belews Creek FGD Residue Landfill
Anchor Trench Calculations

Job No.	**046119**		
Comp. By	**WMM**	**Date**	**9/28/2005**
Chkd. By	DCB	**Date**	9/28/05
Sheet	3	**of**	3

Total Force Available = F1 + F2 +F3

F_{total} = 972.2 lb/ft

Conclusion:

As described in the Liner Stability calculations,
the liner system is at rest with no tension forces present.
Therefore, total force available from the anchor trench forces
is adequate.

Dimensions of anchor trench shown on detail 7-5 on Sheet C-5 are acceptable.

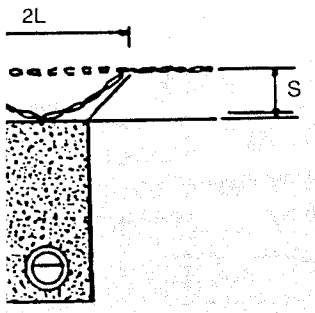


Figure 3-12). The vehicle much larger than the dead

Reference 1

Cell Component: FLEXIBLE MEMBRANE LINING

Consideration: FML ANCHORAGE - CALCULATE ANCHOR CAPACITY FOR FML PLACED IN VARIOUS ANCHORAGE CONFIGURATIONS.

Required Material Properties	Range	Test	Standard
· SOIL/FML FRICTION ANGLE	12-20°	DIRECT SHEAR	ASTM PROPOSED
· SOIL FRICTION ANGLE	25-38°	TRIAXIAL	ASTM PROPOSED
Draft MIG Minimum			

Analysis Procedure:

(1) DEFINE ANCHOR VARIABLES

- GEOMETRY
  - SLOPE ANGLE β
  - EMBEDMENT LENGTH, L
  - SOIL COVER, d_{cs}
  - ANCHOR BURIAL, d_{AT}
- MATERIAL
  - SOIL FRICTION ANGLE, ϕ
  - SOIL/FML FRICTION ANGLE, δ
  - SOIL UNIT WEIGHT, γ_{LS}

(2) SOLVE FOR ANCHOR CAPACITY

HORIZONTAL ANCHOR

$$T = \frac{qL\tan\delta}{(DR)\cos\beta - \sin\beta\tan\delta}$$

"V" ANCHOR

$$T = \frac{\tan\delta\left[q\left(L-(L_V+\frac{L_V}{\cos\beta})\right)+\frac{d_q L_V \gamma_{cs}}{2\cos\beta}\right]}{(DR)\cos\beta - \sin\beta\tan\delta}$$

CONCRETE ANCHOR

$$T = \frac{qL\tan\delta\,(K_p - K_a)\left[.5\gamma_{cs}d_{AT}^2 + qd_{AT}\right]}{(DR)\cos\beta - \sin\beta\tan\delta}$$

ANCHOR TRENCH

$$T = \frac{qL\tan\delta + (K' + K_a)\tan\delta\left[.5\gamma_{cs}d_{AT}^2 + qd_{AT}\right]}{(DR)\cos\beta - \sin\beta\tan\delta}$$

$K' = K_p$ OR K_o

REF. FIGURES 3.9 & 3.10

Design Ratio: NOT APPLICABLE

References:

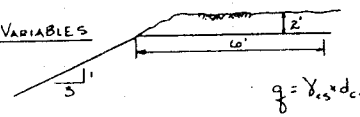
Example:

GIVEN:

GEOMETRY
- $\beta = 18.4°$ (3:1 SLOPE)
- L = 6 FT
- d_{cs} = 2'-0
- DR = 1.5 MINIMUM

SOIL
- $\phi = 35°$
- $\delta = 13°$
- $\gamma_{cs} = 120$ PCF

(1) DEFINE VARIABLES



$q = \gamma_{cs} \times d_{cs} = 120 \times 2 = 240$ PSF

(2) SOLVE FOR ANCHOR CAPACITIES

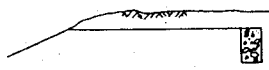
- HORIZONTAL (AS SHOWN)

$$T = \frac{240 \times 6 \times \tan 13°}{1.5\cos 18.4° - \sin 18.4°\tan 13°} = 248 \text{ lb/ft}$$

- "V" TRENCH
  - L_V = 4'
  - d_V = 1'
  - $\beta = 26.5°$

$$T = \frac{\tan 13°\left[240\left(6.4°\frac{4}{\cos 26.5°}\right) - \frac{1.0 \times 120}{2\cos 26.5°}\right]}{1.339} = 313 \text{ lb/ft}$$

- CONCRETE ANCHOR
  - d_{AT} = 2'-0"
  - $\phi = 35° \Rightarrow K_A = .27$ $K_P = 3.7$



$$T = \frac{240 \times 6 \times \tan 13° + (3.7 - .27)\left[0.5 \times 120 \times 2^2 + 240 \times 2\right]}{1.339} = 2070 \text{ lb/ft}$$

- ANCHOR TRENCH
  - d_{AT} = 2'-0"
  - $\phi = 35° \Rightarrow K_o = .426$

$$[T]_{K_P} = \frac{[240 \times 6 \times \tan 13° + (3.7 - .27)\tan 13°\left[.5 \times 120 \times 2^2 + 240 \times 2\right]}{1.339} = 735 \text{ lb/ft}$$

$$[T]_{K_o} = \frac{240 \times 6 \times \tan 13° + (.426 - .27)\tan 13°\left[.5 \times 120 \times 2^2 + 240 \times 2\right]}{1.339} = 334 \text{ lb/ft}$$

Example No. 3.17

C. COHESIONLESS SAND ($c = 0$)

The maximum slope angle for cohesionless sand is the angle of internal friction, ϕ.

23 EARTH PRESSURE THEORIES

The general equation for horizontal *active earth pressure* is:

$$p_{\text{horizontal}} = p_{\text{vertical}} \tan^2\left(45° - \frac{\phi}{2}\right) - 2c \tan\left(45° - \frac{\phi}{2}\right) \qquad 10.51$$

c in equation 10.51 is the soil's cohesion.

p_{vertical} can be due to surcharge, externally applied loads, or the soil's own mass.

If $\phi = 0°$, as in the limiting case for saturated clay, then

$$p_{\text{horizontal}} = p_{\text{vertical}} - 2c \qquad 10.52$$

If $c = 0$, as in the limiting case for drained sand,

$$p_{\text{horizontal}} = p_{\text{vertical}} \left[\tan^2\left(45° - \frac{\phi}{2}\right)\right] \qquad 10.53$$

The quantity in brackets in equation 10.53 is known as the *coefficient of active earth pressure.*

$$k_A = \tan^2\left(45° - \frac{\phi}{2}\right) = \frac{1 - \sin\phi}{1 + \sin\phi} \qquad 10.54$$

The general equation for horizontal *passive earth pressure* is:

$$p_{\text{horizontal}} = p_{\text{vertical}} \tan^2\left(45° + \frac{\phi}{2}\right) + 2c \tan\left(45° + \frac{\phi}{2}\right) \qquad 10.55$$

The *coefficient of passive earth pressure* for sand is

$$k_p = \frac{1}{k_A} = \tan^2\left(45° + \frac{\phi}{2}\right) = \frac{1 + \sin\phi}{1 - \sin\phi} \qquad 10.56$$

A. THE RANKINE THEORY

If it is assumed that the backfill soil is dry, cohesionless sand, then the *Rankine theory* can be used. At any depth, H, the vertical pressure is

$$p_{\text{vertical}} = \rho H \qquad 10.57$$

The horizontal pressure depends on the *coefficient of earth pressure at rest*, k_o, which varies from 0.4 to 0.5 for untamped sand.[18]

$$p_{\text{horizontal}} = k_o \rho H \qquad 10.58$$

$$k_0 \approx 1 - \sin\phi \qquad 10.59$$

Equations 10.57 and 10.58 apply only to a sand deposit of infinite depth and extent. For sand that is compressed or tensioned (as in around a retaining wall) the reactions are given by equations 10.60 and 10.61.

$$R_A = \frac{1}{2} k_A \rho H^2 \qquad 10.60$$

$$R_P = \frac{1}{2} k_P \rho H^2 \qquad 10.61$$

R_A and R_P are horizontal if the soil above the heel and toe is horizontal.

B. WEDGE THEORIES

The Rankine theory is based on infinite, cohesionless soil. It also requires that the soil above the heel be level. Modifications can be made to lift these restrictions, as well as to allow a water table above the foundation base. Several modifications are known as *wedge theories. Coulomb's earth-pressure theory* is one such wedge theory.

The wedge methods are based on the observation that retaining walls fail when the active soil shears. Although the shear plane is actually a slightly curved surface, it is assumed to be linear (line $*$–$*$ in figure 10.15). However, since the actual shear plane is not known in advance, several trial planes need to be taken. This is known as the *trial wedge method.*

24 SLOPED AND BROKEN SLOPE BACKFILL

It is possible to derive equations for the active force with a sloped backfill, as shown in figure 10.16. However, the complexity of these equations usually makes a graphical solution a better choice.

With sloped or broken slope backfill, the active force is not horizontal. Appendix A and appendix B provide a method of evaluating the horizontal and vertical earth pressure coefficients.

$$R_{A,h} = \frac{1}{2} k_h \rho H^2 \qquad 10.62$$

$$R_{A,v} = \frac{1}{2} k_v \rho H^2 \qquad 10.63$$

[18] It is appropriate to use the at-rest soil case whenever the wall does not move. Bridge abutments and basement walls are examples where movement is essentially nonexistent.

Leachate Generation

- Help Model, Discussion and Results

Belews Creek FGD Residue Landfill

HELP Model

Description, Discussion, and Results

1.0 Regulatory Requirements

The regulatory requirements for the design of a solid waste, industrial landfill are found in North Carolina Administrative Code, Title 15A, Chapter 13 Solid Waste Management, Section .0503 Siting and Design Requirements for Disposal Sites. In particular, Section .0504(2)(d)(ii) requires that:

> *(A) a design that will ensure that the ground water standards established under 15A NCAC 2L will not be exceeded in the uppermost aquifer at the compliance boundary established by the Division in accordance with 15A NCAC 2L. The design shall be based upon modeling methods acceptable to the Division, which shall include, at a minimum, the following factors:*
>
> > *(I) the hydrogeologic characteristics of the facility and surrounding lands;*
> > *(II) the climatic factors of the area; and*
> > *(III) the volume and physical and chemical characteristics of the leachate; or*
>
> *(B) a design with a leachate collection system, a closure cap system, and a composite liner system consisting of two components: the upper component shall consist of a minimum 30-ml flexible membrane (FML), and the lower components shall consist of at least a two-foot layer of compacted soil with a hydraulic conductivity of no more than 1 X 10^{-7}cm/sec. FML components consisting of high density polyethylene (HDPE) shall be at least 60-ml thick. The FML component shall be installed in direct and uniform contact with the compacted soil component.*

The proposed landfill design calls for a leachate collection system, a closure cap system, and a liner system composed of a 60 mil HDPE geomembrane, underlain by site soils. The HELP model is used to develop leakage/infiltration values that are used in a fate and transport model that is used to demonstrate compliance with 2L groundwater standards.

2.0 HELP Model Description

2.1 Description of HELP Program

The Hydological Evaluation of Landfill Performance (HELP) program is a quasi-two-dimensional hydrologic model for conducting water balance analyses of landfills, cover systems, and other solid waste containment facilities. The model accepts weather, soil and design data and uses solution techniques that account for the effects of surface storage, snowmelt, runoff, infiltration, evapotranspiration, vegetative growth, soil moisture storage, lateral subsurface drainage, leachate recirculation, unsaturated vertical drainage, and leakage through soil, geomembrane or composite liners.

Landfill systems including various combinations of vegetation, cover soils, waste cells, lateral drain layers, low permeability barrier soils, and synthetic geomembrane liners may be modeled. The model facilitates rapid estimation of the amounts of runoff, evapotranspiration, drainage, leachate collection and liner leakage that may be expected to result from the operation of a wide variety of landfill designs.

The primary purpose of the model is to assist in the comparison of design alternatives. The model is a tool for both designers and permit writers. [Reference 1]

The HELP model requires general climate data for computing potential evapotranspiration; daily climatologic data; soil characteristics; and design specifications to perform the analysis. The required general climate data include growing season, average annual wind speed, average quarterly relative humidities, normal mean monthly temperatures, maximum leaf area index, evaporative zone depth and latitude. Default values for these parameters were compiled or developed from the "Climates of the States" (Ruffner, 1985) and "Climatic Atlas of the United States" (National Oceanic and Atmospheric Administration, 1974) for 183 U.S. cities. Daily climatologic (weather) data requirements include precipitation, mean temperature and total global solar radiation. Daily rainfall data may be input by the user, generated stochastically, or taken from the model's historical data base. The model contains parameters for generating synthetic precipitation for 139 U.S. cities. The historical data base contains five years of daily precipitation data for 102 U.S. cities. Daily temperature and solar radiation data are generated stochastically or may be input by the user.

Necessary soil data include porosity, field capacity, wilting point, saturated hydraulic conductivity, initial moisture storage, and Soil Conservation Service (SCS) runoff curve number for antecedent moisture condition II. The model contains default soil characteristics for 42 material types for use when measurements or site-specific estimates are not available. The porosity, field capacity, wilting point and saturated hydraulic conductivity are used to estimate the soil water evaporation coefficient and Brooks-Corey soil moisture retention parameters. Design specifications include such items as the slope and maximum drainage distance for lateral drainage layers; layer thickness; layer description; area; leachate recirculation procedure; subsurface inflows; surface characteristics; and geomembrane characteristics.

HELP was used to develop values for leakage through the landfill liner and to develop values for leachate generation.

2.2 HELP Model Version:

The HELP model used for this demonstration is HELP Version 3.07.[1] HELP was developed by the USAE Waterways Experiment Station. Also used as reference material were the HELP User's Guide for Version 3, [Reference 2] and the HELP Engineering Documentation for Version 3 [Reference 1].

[1] The weather files used to generate the temperature and rainfall data are dated 1999.

2.3 Cases Modeled with HELP

Four cases were modeled with HELP to simulate the landfill conditions during the Operational Period and the Closed Period. These periods are described below:

- Operational Period – models the infiltration that occurs during the period of gypsum placement.
- Closed Period – models the infiltration that occurs after placement of the engineered cover.

The four HELP cases are described below:

Table 2-1

HELP Case #	Operational Cells	Description of Conditions	Other
1	Cell1	Cell1 – 10 feet of FGD gypsum waste	CN = 81. LAI = 0 Model period 10 years
2	Cell1 + Cell 2	Cell1 + Cell 2 – 66 feet of FGD gypsum waste. Max height for 3(H):1(V) side slopes in Cell 1 and Cell2	CN = 81. LAI = 0 Model period 10 years
3	All	All cells operational. FGD waste to Elevation 858' (89' of waste).	CN = 61 LAI = 0 Model period 10 years
4	All Cells Closed	Engineered Cover in place.	CN = 61 LAI = 2 Model period 100 years 8 holes/acre in geomembranes

Diagrams showing the configuration of the modeled layers for the case listed above are shown at the end of this section.

3.0 HELP Model Inputs - Soil and FGD Residue Properties

The soil and gypsum properties that are used in the HELP analyses were developed from tests performed on soil samples collected during the site investigation (Reference 3) and from gypsum samples provided by the scrubber equipment vendor.

3.1 Properties for Vegetative Soil Layer

The soil properties from borings located within the landfill footprint or adjacent to the footprint were used to determine porosity, field capacity, wilting point, and hydraulic conductivity parameters for the vegetative soil layer. These values are shown in Tables Table 3-1, Table 3-2, Table 3-3, Table 3-4, and Table 3-5.

The vegetative soil layer will be constructed from material excavated from the landfill footprint area and compacted as the vegetative cover layer. Therefore, the remolded samples reflect the properties for this layer.

HELP requires values for field capacity and wilting point. These values are calculated based on the percentages of sand and clay present in the soil.

Table 3-1

Find % of Sand , Silt, and Clay (USDA) for Calculation of Wilting Point and Field Capacity

Soil Cover Material

		% Passing				USDA Classification		
Well/Boring	Sample ID	2.0 mm	0.05 mm	0.002 mm	Comment	% Sand	%Silt	% Clay
BC-12	SS-6	100%	55%	6%	In cut	45%	49%	6%
BC-16	SS-3	88%	37%	4%	In cut	51%	33%	4%
BC-3	Bag 0'-10'	100%	49%	9%	In cut	51%	40%	9%
BC-10	Bag 0'-10'	97%	30%	5%	In cut	67%	25%	5%
BC-11	Bag 0'-10'	96%	50%	32%	In cut	46%	18%	32%
BC-13	Bag 0'-10'	98%	55%	22%	In cut	43%	33%	22%
BC-16	Bag 0' - 5'	99%	42%	12%	In cut	57%	30%	12%
BC-19	Bag 2' - 7'	97%	81%	44%	In cut	16%	37%	44%
BC-19	SS-4	97%	47%	5%	In cut	50%	42%	5%
BC-10	UD-1	100%	65%	7%	In cut	35%	58%	7%
BC-12	UD-1	100%	45%	5%		55%	40%	5%
						% Sand	%Silt	% Clay
					Average	47%	37%	14%

Table 3-2

Find Average Porosity Values and K's

Cover Soil

Boring	Sample Depth (ft)	Porosity (%) Initial Sample Conditions	Final Sample Conditions	K (cm/sec)
BC-3	0 - 10	42.8	46.2	2.40E-05
BC-7	0 - 10	45.4	46.3	3.42E-06
BC-10	0 - 10	39.6	37.1	9.06E-05
BC-11	0 - 10	39.2	40.4	1.67E-07
BC-12	5 - 10	40.2	40.2	5.61E-06
BC-13	0 - 10	43.9	45.0	4.81E-06
BC-13	10 - 20	41.8	41.8	8.65E-06
BC-15	0 - 5	41.8	41.8	7.61E-06
BC-16	1 - 5	36.9	36.9	2.30E-05
BC-17	0 - 6	40.6	40.6	1.82E-05
BC-19	0 - 10	51.3	51.3	1.79E-05
BC-20	0 - 5	37.3	37.3	2.03E-06
BC-21	0 - 5	36.2	36.2	3.95E-06
BC-22	0 - 5	37.5	37.4	2.87E-04
BC-23	1 - 6	34.3	34.1	4.39E-05
	Average Values	40.6	40.8	3.61E-05

Table 3-3

Calculate Field Capacity and Wilting Point :

The equations used for these calculations are found in the HELP Users Manual, Appendix A.

Cover Soil Properties

	% Sand	USDA Classification % Clay
Average	46.9%	13.7%

Field Capacity (HELP Eqn A-3)	Wilting Point (HELP Eqn A-4)
0.286	0.098

Table 3-4

Values Used in HELP for Soil

Cover Soil - Values Used in HELP

	Hydraulic Conductivity, k (cm/sec)	Porosity	Field Capacity (HELP Eqn A-3)	Wilting Point (HELP Eqn A-4)
Upper Vegetative Layer (see Table 1-5)	1.1E-04	0.41	0.286	0.098
Lower Vegetative Layer	3.61E-05	0.41	0.286	0.098

Table 3-5

Vegetative Soil Layer - Modified Saturated Hydraulic Conductivity in Upper Half of Evaporative Zone

Since the soil properties are not default HELP properties,
the saturatred hydraulic conductivity in the top half of the evaporative zone
must be adjusted to account for root penetration by surface vegetation.

LAI = Leaf Area Index

Ks = Saturated hydraulic conductivity in vegetative layer soil, cm/sec

(Ks)v = Saturated hydraulic conductivity in top half of evaporative zone, cm/sec

Eqn. A-10 HELP Manual

Ksv = (1.0 + 0.5966*LAI + 0.132659 *(LAI)^2 + 0.1123454*(LAI)^3 - 0.04777627 (LAI)^4 +(0.004325035 (LAI)^5)* Ks

LAI = 2.0

Ks = 3.6E-05 cm/sec

Ksv = 1.1E-04 cm/sec

3.2 Properties for Gypsum Layer

Samples of the gypsum material were sent to the geotechnical firm S&ME Inc. for testing. The results of the lab analyses on these samples is found in the report, FGD Scrubber Sludge Testing, (Revision 1) Duke Power Coal Fired Steam Stations in North Carolina, S&ME Project No. 1264-03-57, February 2004 (Reference 4). A copy of this report is included as Attachment 1.

Lab analyses were performed on these samples to determine the following properties:

- Grain size Distribution
- Atterburg Limits
- Specific Gravity
- Natural Moisture
- pH
- Standard Proctor Compaction
- Remolded Permeability
- Consolidation Properties (C_c)
- Total Porosity
- Triaxial Shear Strength

The grain size analyses, void ratio, porosity, and saturated hydraulic conductivity's from these analyses were used in the HELP model analyses. These values used in HELP are shown in Table 3-7.

Table 3-6

FGD Residue - Hydraulic Conductivity and Void Ratio

Remolded Sample Conditions	Porosity (at 8 ksf)	Hydraulic Conductivity, k (cm/sec)
90% compaction @ 12% moisture	0.43	5.09E-04
92% compaction @ 25% moisture	0.41	2.25E-04
95% compaction @ 19% moisture	0.4	2.53E-04

Table 3-7

FGD Residue – Values Used In HELP

Hydraulic Conductivity, k (cm/sec)	Porosity[2], n	Field Capacity (HELP Eqn A-3)	Wilting Point (HELP Eqn A-4)
2.53E-04	0.47	0.180	0.055

[2] NCDENR requested that a porosity of 0.47 be used with 90% compaction in the HELP analysis for the Duke Power Marshall Steam Station FGD landfill permit application.

3.3 Properties for HDPE Liner

The landfill design calls for a liner system composed of a 60 mil HDPE membrane liner, underlain by site soil.

A total of 8 holes per acre in the HDPE liner were used in the HELP analysis.

3.4 Properties for Soil Layer Beneath the Landfill

The soil properties from borings located within the landfill footprint or adjacent to the footprint were used to determine porosity, field capacity, wilting point, and hydraulic conductivity parameters for the layer of soil located beneath the liner system. These values are shown in Table 3-8, Table 3-9, Table 3-10, and Table 3-11.

Table 3-8

Find % of Sand , Silt, and Clay (USDA) for Calculation of Wilting Point and Field Capacity

Soil Below Liner

		% Passing				USDA Classification		
Well/Boring	Sample ID	2.0 mm	0.05 mm	0.002 mm	Comment	% Sand	%Silt	% Clay
BC-19	UD-1	98%	35%	4%	Below/At Liner	63%	31%	4%
BC-10	SS-7	98%	47%	3%	Below Liner	51%	44%	3%
BC-18	SS-4	88%	24%	2%	Below Liner	64%	22%	2%
BC-11	UD-1	100%	42%	6%	Below Liner	58%	36%	6%
						% Sand	%Silt	% Clay
					Average	59%	33%	4%

Table 3-9

Calculate Field Capacity and Wilting Point :

The equations used for these calculations are found in the HELP Users Manual, Appendix A.

Soil Below Liner

	% Sand	USDA Classification % Clay
Average	59.0%	3.8%

Field Capacity (HELP Eqn A-3)	Wilting Point (HELP Eqn A-4)
0.149	0.052

Table 3-10

Porosity Values and K's for Soil Below Liner

As shown in Table 8(Reference 3), only 2 Shelby Tube samples were collected at the elevation of the bottom gradefor the landfill. These samples were collected at borings BC-12 and BC-19.
The most conservative value for hydraulic conductivity and porosity is used.

K (cm/sec) =	1.23E-04
Porosity (%) =	45

Table 3-11

Soil Below Liner - Values Used in HELP

Hydraulic Conductivity, k (cm/sec)	Porosity	Field Capacity (HELP Eqn A-3)	Wilting Point (HELP Eqn A-4)
1.23E-04	0.45	0.149	0.052

4.0 HELP Model Results

4.1.1 Percolation/Leakage Results

The HELP model results for percolation/leakage through the liner system and into the soil beneath the liner are summarized in Table 4-1. The maximum value for percolation/leakage is 0.03" per year, occurring during the operational period for Cell1.

The average annual HELP results for leakage through the liner system, after placement of the engineered cover, are 0.00034". This leakage value is equivalent to 0.025 gallons/ac/day.

4.1.2 Head on Liner Membrane and Cover Membrane Results

The HELP model results for peak and average heads in the liner membrane and cover membrane are shown in Table 4-2. As shown in this table, the maximum peak daily head on the liner system is 4.8".

4.1.3 Volume of Leachate and Precipitation

The HELP model results for the volume of leachate and precipitation collected by the LCRS is shown in Table 4-3. The maximum peak daily drainage (runoff and drainage) is 108,098 cubic feet. The maximum average annual flow (runoff and drainage) into the LCRS is 998,091 cubic feet.

Table 4-1

Percolation/Leakage Values From HELP Analyses

Case Number	HELP Run Title	Case Description	Number of Years for Analysis	Average Annual Percolation/Leakage Through Liner System (inches)
1	Belews Creek FGD Landfill 10' FGD Waste - Cell 1 Only	Cell 1 Open - 10 feet FGD waste placed in cell	10	[0.02184] 0.03
2	Belews Creek FGD Landfill FGD Waste - Cell 1 + Cell 2	Cell 1 and Cell 2 - Height of waste = 66.3' (max. height from 3H:1V slope on side slopes)	10	[0.01755] 0.02
3	Belews Creek FGD Landfill - Waste to 858' - All Cells-Open	Waste in all cells up to final grades; no synthetic cap	10	[0.01534] 0.02
4	Belews Creek FGD Landfill Closed Condition	Waste in all cells up to final grades; synthetic cap in place	100	[0.00034] 0.001

NOTES:

(1) The values enclosed in [brackets] are the actual values from the HELP analyses.
The values not in brackets are the recommended values for use in evaluating the performance of the liner system.

(2) These analyses are based on gypsum being the sole waste placed in the landfill.

Table 4-2

HELP Analyses - Summary of Peak Daily and Annual Average Heads on Liners

Case Number	HELP Run Title	Case Description	Number of Years for Analyses	Peak Daily Average Head on Liner (inches)	Peak Daily Maximum Head on Liner (inches)	Average Annual Head on Liner (inches)
1	Belews Creek FGD Landfill 10' FGD Waste - Cell 1 Only	Cell 1 Open - 10 feet FGD waste placed in cell	10	4.026	4.803	0.054
2	Belews Creek FGD Landfill FGD Waste - Cell 1 + Cell 2	Cell 1 and Cell 2 - Height of waste = 66.3' (max. height from 3H:1V slope on side slopes)	10	0.104	0.193	0.037
3	Belews Creek FGD Landfill - Waste to 858' - All Cells-Open	Waste in all cells up to final grades; no synthetic cap	10	0.10	0.186	0.031
4	Belews Creek FGD Landfill Closed Condition	Waste in all cells up to final grades; synthetic cap in place	100	-	-	-
		Synthetic Cap (on top of landfill)	-	0.081	0.156	0.002
		Synthetic Liner (beneath waste)	-	0.069	0.13	0.002

Table 4-3

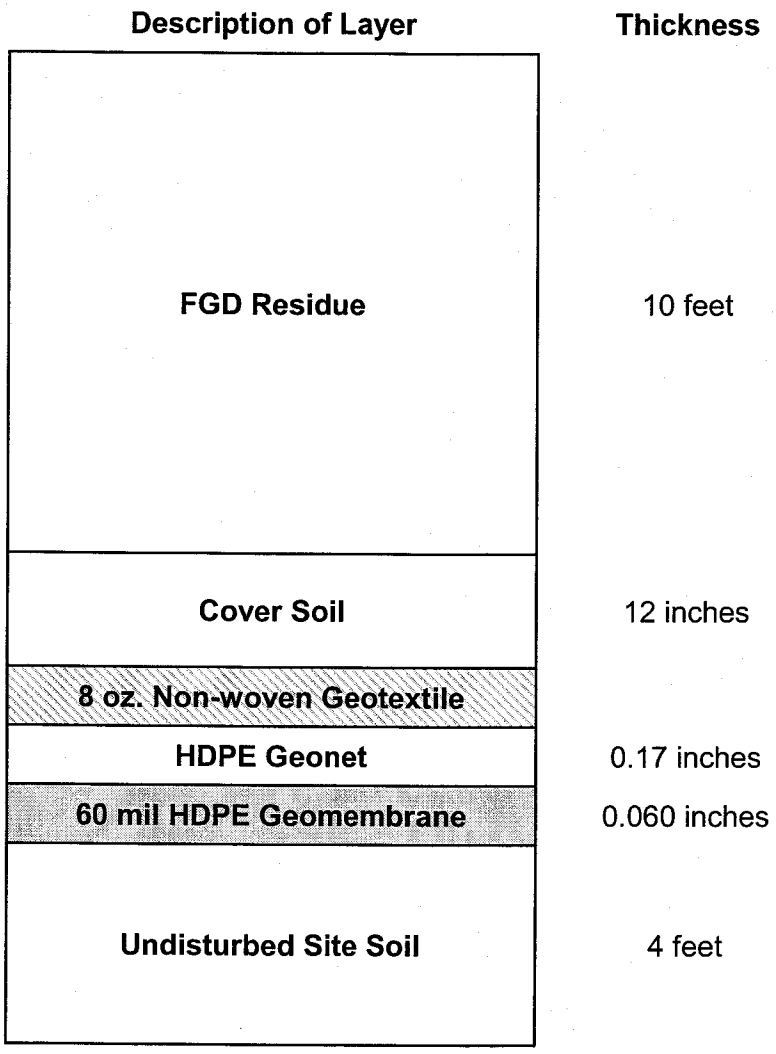
HELP Analyses - Summary of Volume for Leachate and Precipitation Collected by Leachate Collection System (LCS)

Case Number	HELP Run Title	Case Description	Number of Years for Analyses		Peak Daily Runoff (cubic feet)	Peak Daily Drainage into LCS (cubic feet)	Total Peak Daily Flow into LCS (cubic feet)		Average Annual Runoff (cubic feet)	Average Annual Drainage into LCS (cubic feet)	Total Annual Average Flow into LCS (cubic feet)
1	Belews Creek FGD Landfill 10' FGD Waste - Cell 1 Only	Cell 1 Open - 10 feet FGD waste placed in cell	10		36,013	3,153	39,166		33,258	259,799	293,057
2	Belews Creek FGD Landfill FGD Waste - Cell 1 + Cell 2	Cell 1 and Cell 2 - Height of waste = 66.3' (max. height from 3H:1V slope on side slopes)	10		53,582	3,775	57,357		23,262	484,617	507,879
3	Belews Creek FGD Landfill - Waste to 858' - All Cells-Open	Waste in all cells up to final grades; no synthetic cap	10		101,258	6,840	108,098		43,960	777,513	821,473
4	Belews Creek FGD Landfill Closed Condition	Waste in all cells up to final grades; synthetic cap in place	100								
		Synthetic Cap (on top of landfill)			N/A	N/A	N/A		N/A	N/A	N/A
		Synthetic Liner (beneath waste)			N/A	77,061	77,061		N/A	998,091	998,091

Diagrams of Modeled HELP Cases

Belews Creek FGD Residue Landfill HELP Model Discussion and Results	Comp. By	WMM	Job No.	046119
	Chkd. By		Date	9/30/2005
	Sheet		Date	
			of	

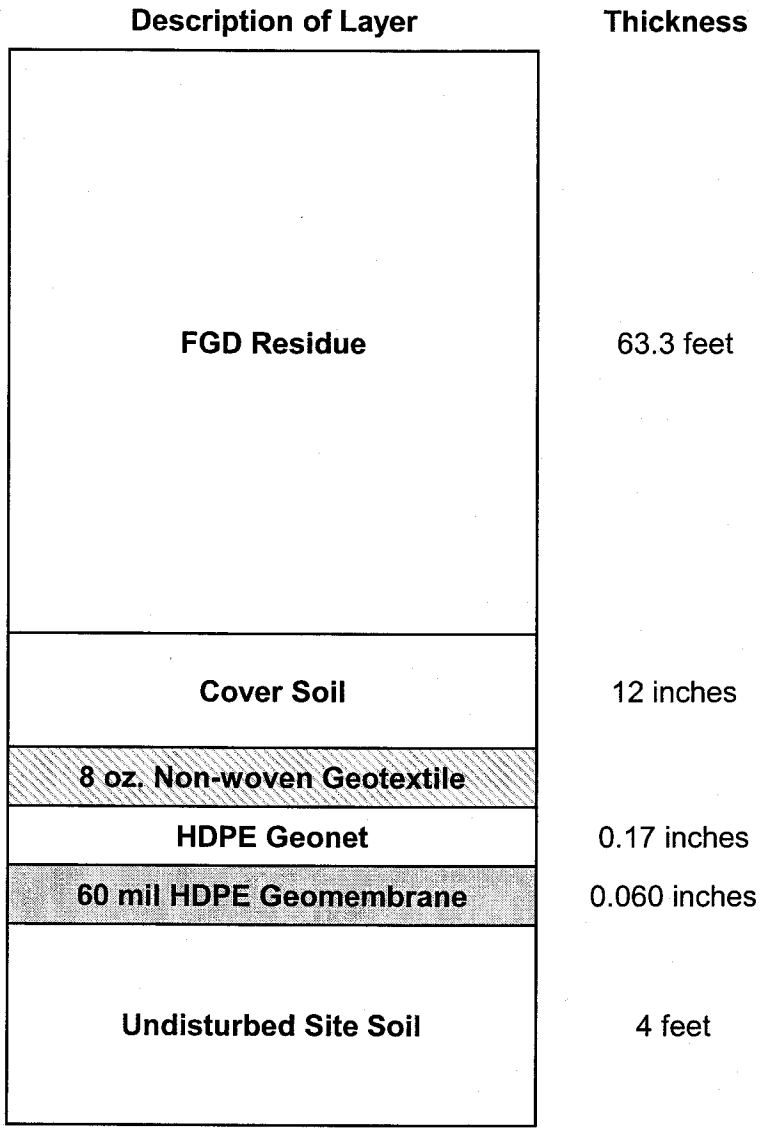
Case 1
Operations - Cell 1 - 10' FGD Residue
(no scale)



Notes:

1. 8 oz. Non-woven Geotextile is not modeled as a separate layer.

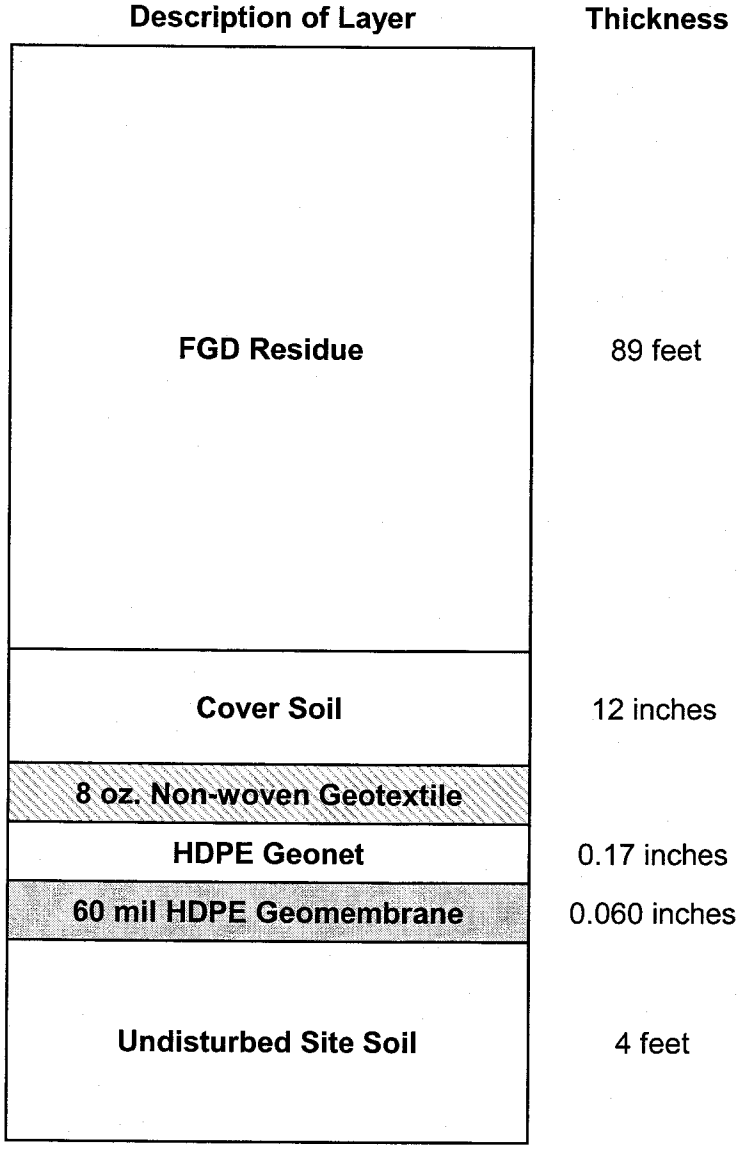
Case 2
Operations – Cell 1 + Cell 2
(no scale)



Notes:

1. 8 oz. Non-woven Geotextile is not modeled as a separate layer.

Case 3
Operations - Waste to El. 858' -
All Cells Open
(no scale)



Notes:

1. 8 oz. Non-woven Geotextile is not modeled as a separate layer.

Case 4
Closed Condition
(no scale)

Description of Layer	Thickness
Upper Vegetated Layer	0.75 ft
Lower Vegetated Layer	1.25 ft
HDPE Geonet	0.0164 ft
Low Density Polyethylene Geomembrane	0.040in
Compacted Site Soil	1.5 ft
FGD Residue	89 feet
Cover Soil	12 inches
8 oz. Non-woven Geotextile	
HDPE Geonet	0.17 inches
60 mil HDPE Geomembrane	0.060 inches
Undisturbed Site Soil	4 feet

Notes:

1) 8 oz. Non-woven Geotextile is not modeled as a separate layer.

References

Note: The information and sources listed below were used in the preparation of this report. The parties listed in the references prepared these reports and documents. The engineering certification on the cover page does not imply that the engineering certification of this report includes certification of these particular references.

1 THE HYDROLOGIC EVALUATION OF LANDFILL PERFORMANCE (HELP) MODEL, *ENGINEERING DOCUMENTATION FOR VERSION 3,* Environmental Laboratory, U.S. Army Corps of Engineers, Waterways Experiment Station, Vicksburg, MS.

2 *The Hydrologic Evaluation of Landfill Performance (HELP) Model – User's Guide for Version 3*, USEPA September 1994.

3 *Geologic and Hydrologic Siting Report, FGD Scrubber Residue Disposal Site, Belews Creek Steam Station, 3195 Pine Hall Road, Belews Creek, North Carolina, S&ME Project No: 1054-04-955, February 2005*

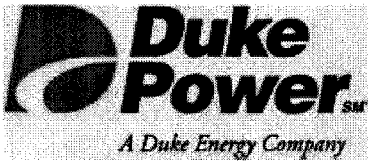
4 FGD Scrubber Sludge Testing, (Revision 1) Duke Power Coal Fired Steam Stations in North Carolina, S&ME Project No. 1264-03-57, February 2004.

Belews Creek FGD Residue Landfill
HELP Model Description and Results

Attachment 1
FGD Residue Material Geotechnical Test Results

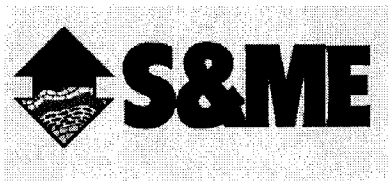
FGD SCRUBBER SLUDGE TESTING
(REVISION 1)
DUKE POWER
COAL FIRED STEAM STATIONS
IN NORTH CAROLINA
S&ME Project 1264-03-057

Prepared For:



Charlotte, North Carolina

Prepared By:



Spartanburg, South Carolina

February 2004

February 2, 2004

Mr. William M. (Bill) Miller, P.E.
Mail Code EC12ZB
Duke Energy
526 Church Street
Charlotte, North Carolina 28202

Reference: **FGD SCRUBBER SLUDGE TESTING (REVISION 1)**
Duke Power Coal-Fired Steam Stations
In North Carolina
S&ME Project 1264-03-057

Ladies and Gentlemen:

S&ME is pleased to submit the results of laboratory testing performed on a sample of flue gas desulfurization (FGD) scrubber sludge. This letter presents a brief discussion of the project, the testing performed, the test results and their comparison to published reference literature, and stability analyses of conceptual FGD scrubber sludge slopes. This submittal supercedes the April 30, 2003 submittal in that consistent earthquake loadings have been applied throughout the conceptual slope stability analyses, the minimum factors of safety have been updated accordingly, and the earthquake references have been included in Appendix B.

PROJECT INFORMATION

In response to the Clean Air Act of 1990 and its subsequent amendments mandating the reduction of sulfur dioxide (SO_2) emissions from power plants, Duke Power is evaluating installation of flue gas desulfurization (FGD) scrubber systems at several of its existing coal-fired power plants. Literature suggests the present FGD technology most favorable, particularly in terms of by-product produced, is limestone wet scrubbing with oxidation, which is the process being evaluated by Duke Power. These processes, followed by drying via vacuum filter, produce a calcium sulfate dihydrate scrubber sludge, or gypsum, that can potentially be used as raw

S&ME, Inc.
155 Tradd Street
Spartanburg, South Carolina 29301
(864) 574-2360
(864) 576-8730 fax
(864) 232-8987 Greenville
www.smeinc.com

product for the production of wallboard and/or exhibits more favorable engineering properties for landfill disposal than some other scrubbing process by-products.

In considering development of disposal landfills for the output product, in the absence of wallboard production or should wall board production not fully utilize the output product, Duke Energy obtained a sample of calcium sulfate dihydrate (gypsum) from a confidential source powerplant. We understand that the confidential powerplant comprises FGD scrubber equipment by the same manufacturer contracted by Duke Power to install FGD scrubber equipment at their facilities. Therefore, it is anticipated that the scrubber sludge produced at the confidential powerplant will at least be similar to that produced at the Duke Power facilities. Duke Energy requested S&ME to subject the scrubber sludge sample to selected geotechnical laboratory testing in an effort to gain understanding of its engineering properties relative to disposal practices akin to conventional landfill disposal, referred to as "dry-stacking" for this material. This letter presents the testing performed and the results, including comparison to engineering properties referenced in available literature. Furthermore, toward the consideration of landfill slope height and inclination, we performed stability analyses of conceptual FGD scrubber sludge slopes, the results of which are also presented herein.

SELECTED LABORATORY TESTING PERFORMED

Duke Energy and S&ME agreed the for this initial round of laboratory testing to comprise the following:

- Grain Size Distribution
- Atterberg Limits
- Specific Gravity
- Natural Moisture
- pH
- Standard Proctor Compaction
- Remolded Permeability
- Consolidation
- Total Porosity
- Triaxial Shear Strength

LABORATORY TESTING RESULTS

The laboratory results are summarized on the attached *Table 1, Summary of Laboratory Testing Results – Index Properties* and *Table 2 – Summary of Laboratory Testing Results – Engineering Properties* along with published data for comparison. Laboratory test reports are provided in Appendix A. A brief discussion of the testing results and comparison to published literature is provided below.

Grain Size Distribution

The confidential powerplant sample exhibited approximately 97.2 percent of its particle sizes in the silt size range [0.005 to 0.075 millimeters (mm)], 2.5 percent of its particle size in the clay/colloid range (<0.005-mm), and only 0.2 percent of its particle size in the fine sand range (> 0.075 mm). Published literature references calcium sulfate (gypsum) particle sizes predominately in the silt size range, with some samples exhibiting a greater particle size percentage in the fine sand range, a bit coarser than the confidential powerplant sample. For visual comparison of grain size distribution between the confidential powerplant sample and published literature, we produced the attached *Plate 1 - Grain Size Distribution Comparison.*

Atterberg Limits

The confidential powerplant sample tested as largely non-plastic, exhibiting a Liquid Limit of 27, a Plastic Limit of 26, and a resulting Plasticity Index of only 1. This non-plastic characteristic agrees with the published literature. It is noteworthy that the Liquid Limit, the moisture content at which the material begins to act as a liquid (flow), is relatively low at only 27 percent. Comparatively, the Liquid Limit for residual sandy silts from the Piedmont Geologic Province may be expected on the order of 30 to 50.

Specific Gravity

The confidential powerplant sample solids exhibited a specific gravity of 2.35, an exact match to the specific gravity for gypsum sludge cited in the literature of 2.35. This specific gravity means the gypsum material is lighter than soil, which often exhibits a solids specific gravity on the order of 2.6 to 2.7.

Natural Moisture

The "natural" moisture of the confidential powerplant sample as delivered was 11.8 percent. It is our understanding that the materials "natural" moisture will be a function of the drying process, anticipated being by vacuum filter, that follows the scrubber reaction.

pH

The confidential powerplant sample's pH was near neutral, at 6.5. This pH is less than that most cited in the referenced literature at or above 10.

Standard Proctor Compaction

The confidential powerplant sample exhibited a standard Proctor maximum dry density (MDD) of 88.5 pounds per cubic foot (pcf) at an optimum moisture content (OMC) of 19.5 percent. These values are within the ranges cited in the published literature for gypsum sludge MDDs ranging from 76 to 97 pcf and OMCs ranging from 13 to 32 percent. For visual comparison of standard Proctor compaction between the confidential powerplant sample and several standard Proctor test results from the published literature, we produced the attached *Plate 2 – Standard Proctor Comparison.*

Remolded Permeability

Based on the standard Proctor compaction results, we selected the following three remolded conditions to test permeability and obtained the following results:

Percent Compaction	Moisture Content	Comments	Permeability (cm/sec)
90	12	Dry of OMC and relatively low compaction.	5.09×10^{-4}
92	25	Wet of OMC and correspondingly lower compaction	2.25×10^{-4}
95	19	At typical compaction requirement and OMC.	2.53×10^{-4}

Confining pressures were set at approximately 2.5 pounds per square inch (psi), just to stabilize the specimen inside the test chamber.

The less compacted and dryer specimen (90 percent at 12 percent moisture) exhibited a higher (faster) permeability than the two better-compacted and wetter specimens. The permeability variability between the two better compacted and wetter specimens was low. As discussed in the published literature, we would expect permeability to decrease (be slower) with increased overburden (confining) pressure, but the lower confining pressure is considered more conservative for a fate and transport modeling scenario.

The measured permeabilities are within the range of those referenced in the literature of 1×10^{-3} to 1×10^{-6} cm/sec, but all are toward the faster end of this spectrum.

Consolidation

Consolidation tests were performed at the same three remolded conditions as the permeability testing. For variability and evaluation however, two of the consolidation tests (on samples remolded to 90 percent compaction at 12 percent moisture and 95 percent compaction at 19 percent moisture) involved saturating the specimen during the test and one of the consolidation tests (on sample remolded to 92 percent compaction at 25 percent moisture) involved not saturating the sample during the test.

The two saturated specimens represent the more conventional analysis and more conventional results. That is, primary consolidation occurs as water is expelled from the internal structure voids. The more unconventional unsaturated test analysis exhibited more unconventional results, as consolidation in this unsaturated sample is likely occurring as a combination of water displacement, air displacement, particle realignment, and potentially secondary consolidation, or creep.

In any event, the compression index, Cc, ranged from 0.03 to 0.12 for the consolidation tests on the confidential powerplant sample. These values are less than those cited in the reference literature ranging from 0.2 to 0.3. Relatively speaking, the confidential powerplant samples, exhibiting lower compression indices, would undergo less consolidation than the gypsum cited in the referenced literature exhibiting higher compression indices. Related to compression indices cited for soil, Cc between 0 and 0.19 are considered to represent a material of "slight or low permeability" (Sowers, 1979). Both the laboratory tests and literature suggest that primary consolidation will occur relatively rapidly. Contrary to what might be expected, Cc did not vary inversely with the percent compaction, being highest (0.12) for the sample remolded to 95 percent compaction, lowest (0.03) for the sample remolded to 92% compaction, and between the two (0.09) for the sample remolded to 90 percent compaction.

Total Porosity

Total porosity was computed from the void ratios measured during the consolidation testing at the initial remolded conditions and at the maximum loading conditions [8 kips per square foot (ksf)] as follows:

Percent Compaction	Moisture Content	Initial Total Porosity	Total Porosity at 8 ksf Loading
90	12	0.47	0.43
92	25	0.44	0.41
95	19	0.44	0.40

Total porosity for pure calcium sulfate was not cited in the referenced literature.

Triaxial Shear Strength

Triaxial Shear Strength tests were performed at the same three remolded conditions as the permeability and consolidation testing. Again for variability and evaluation however, we varied the triaxial shear strength test conditions. We initially analyzed two of the specimens (samples remolded to 90 percent compaction at 12 percent moisture and 95 percent compaction at 19 percent moisture) under consolidated undrained (CU) test conditions with pore water pressure measurements and one of the specimens (sample remolded to 92 percent compaction at 25 percent moisture) to unconsolidated undrained (UU) test conditions. Theoretically, the UU test more closely models the initial placement conditions and the CU test more closely models conditions longer term after placement. The reference literature and initial test results confirmed that the UU condition was perhaps the more conservative from a strength standpoint (i.e., lower angle of internal friction). Therefore, we elected to retest two specimens remolded to 90 percent compaction at 12 percent moisture and 95 percent compaction at 19 percent moisture under the UU test conditions. A summary of the test results is provided below.

Percent Compaction	Moisture Content	Test Conditions	Total Stresses		Effective Stresses	
			Cohesion (psf)	Internal Friction Angle	Cohesion (psf)	Internal Friction Angle
90	12	UU	250	36.0	---	---
92	25	UU	470	32.1	---	---
95	19	UU	340	37.9	---	---
90	12	CU	1920	47.3	0	39.3
95	19	CU	3680	51.2	0	39.7

Overall, the triaxial shear results are the same order of magnitude as the ranges cited in the published literature, which reference one UU test condition sample exhibiting total stresses of 8 psi [1,152 pounds per square foot (psf)] cohesion and a 39 degree friction angle, and a range of effective stresses from 0 to 5 psi (720 psf) cohesion and 31 to 39 degree friction angles. Interestingly, the compaction conditions for the cited literature data are not specified.

CONCEPTUAL SLOPE STABILITY

Complimentary to the laboratory testing data obtained, a preliminary yet primary consideration is the ability to landfill the gypsum in terms of potential development heights and slopes. While any and all site-specific landfill development cross-section(s) will need to be analyzed individually, we took this opportunity to evaluate several generalized cross-section heights and slopes for static and dynamic stability for preliminary planning purposes.

Slope stability analyses were performed using a computerized version (PCSTABL6H) of the Bishop's Modified Method of slope stability analysis. These analyses were performed for a typical cross-section of 100 feet in height (the approximate maximum being considered for the proposed gypsum disposal landfills) at inclinations of 2:1, 3:1, and 4:1 (horizontal to vertical) using undrained and drained strength parameters (total and effective stresses) with and without seismic loading. In determining the seismic loading, the probabilistic ground motions were obtained for the locations of the Cliffside, Marshall, and Belews Creek Steam Station facilities based on latitude and longitude. From the USGS seismic data, it was determined that the peak ground acceleration (PGA) for the Cliffside facility was most critical (0.18g) and therefore was used in the conceptual slope stability analysis.

The results of the stability analysis are summarized in the following table. In addition, a graphical representation of the critical failure surface for each trial section is presented in Appendix B, along with USGS Seismic Hazard Maps and probabilistic ground motions for each site.

Slope Height	Slope Inclination	Minimum Safety Factor without Seismic Loading		Minimum Safety Factor with Seismic Loading	
		Total Stresses	Effective Stresses	Total Stresses	Effective Stresses
100	2:1	1.72	1.64	1.14	1.02
100	3:1	2.35	2.45	1.34	1.37
100	4:1	2.94	3.16	1.49	1.56

ADDITIONAL CONSIDERATIONS

Our initial geotechnical testing provides an indication that the confidential powerplant FGD scrubber sludge is at least similar to that cited in the published reference literature. Our slope stability analyses suggest that, under ideal conditions (i.e., conditions in the field similar to those of the laboratory tests), slopes on the order of 100 feet in height (as desired) can be constructed and should, at least in concept, be relatively stable against deep-seated failure. Even though our analyses suggest allowable slope inclinations of as steep as 2:1 (horizontal to vertical) with marginal factors of safety, we recommend that slope inclinations not exceed 3:1 horizontal to vertical considering surface sloughing of steeper embankments, the unfamiliarity of constructing and maintaining slopes with this FGD scrubber sludge material, access for maintenance, and increased factors of safety.

We caution again that any and all landfill development scenarios must be analyzed individually for slope stability using the site specific development cross-section(s) considering actual slope heights, inclinations, benches for access roadways and stormwater management, liner materials, cover material, regulatory requirements, and site-specific loading conditions, etc., that are not specifically included in the conceptual slope stability analysis conducted herein.

There are many aspects of landfill design and operation that are not represented by these individual laboratory tests. These include, but are certainly not limited to, material handling, moisture conditioning, erodibility, wet-weather operations, the gypsum's sensitivity to moisture contents near or above the liquid limit of 27 percent, the gypsum's response to wetting and drying cycles, etc. Accordingly, we recommend that if at all possible, an operational dry-stacking gypsum landfill be located and visited to glean additional information regarding these and other intangible aspects of the landfill development, operation, and maintenance.

Two aspects of the gypsum that we have discussed but not specifically analyzed are the potentials for liquefaction and/or swelling. One characteristic of the gypsum that is an indicator of liquefaction potential is its poor or gap gradation (i.e., most all of the material is of similar grain size). ASTM (1997), however, specifically states that "...FGD material are not susceptible to liquefaction". Later in the same document, however, it recommends "Some structural fills

constructed of...FGD material must be well drained because of the sensitivity of the material to the flow of water (that is, piping). Problems such as ...liquefaction...that may result from saturation...are thus avoided". If saturation is avoided and the FGD scrubber sludge is well compacted (avoiding a low relative density), it is our opinion that liquefaction can be avoided.

Literature (ENR 1999) and our experience indicate that some coal combustion by-products (CCBs), or combinations thereof, can be expansive. At least one combination of calcium, sulfate, aluminum, and water can produce a hydrated mineral termed "ettringite" that is expansive. Based on our discussions, we believe the aluminum component derives from blending FGD gypsum sludge (calcium sulfate) with fly ash that we understand is not anticipated at the project facilities. Nonetheless, it may be prudent to test the FGD scrubber sludge for swelling. It is noteworthy that ASTM (1997) references that "Reactions producing the expansive properties to not commence for a period of more than 30 days after initial ash hydration". Therefore, this test could be set and run as plans for landfill development continue.

CLOSURE

S&ME certainly appreciates this and every opportunity we have to be of service to Duke Energy and Duke Power. We trust this information is responsive to your needs at this time. If you have questions regarding this report, or if we can be of further assistance, please to not hesitate to contact us at your convenience.

Sincerely,

S&ME, Inc.

NORTH CAROLINA PROFESSIONAL SEAL 17092 ENGINEER LARRY T. ARMSTRONG

Larry Armstrong, P.E.
Senior Engineer/Project Manager

M. Howard Perry, P.E.
Senior Engineer/Branch Manager

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TABLE 1

SUMMARY OF LABORATORY TESTING RESULTS - INDEX PROPERTIES

DUKE ENERGY FGD SCRUBBER SLUDGE

CHARLOTTE, NORTH CAROLINA

S&ME Project 1264-03-057

Sample Type or Remolded Conditions	Grain Size Distribution			Atterberg Limits			USCS Classification		Standard Proctor		Natural Moisture	Specific Gravity	pH
	Clay (%)	Silt (%)	Fine Sand (%)	Liquid Limit	Plastic Limit	Plastic Index	Symbol	Description	MDD (pcf)	OMC (%)	(%)		
Bulk	2.5	97.3	0.2	27	26	1	ML	Silt	88.5	19.5	11.8	2.35	6.5
Literature	2.2	81.3	16.5	Non-Plastic			ML	Silt	76 to 97	13 to 32	No Reference	2.35	10

NOTES

1. USCS = Unified Soil Classification System
2. MDD = Maximum Dry Density
3. OMC = Optimum Moisture Content
4. pcf = pounds per cubic foot

TABLE 2

SUMMARY OF LABORATORY TESTING RESULTS - ENGINEERING PROPERTIES

DUKE ENERGY FGD SCRUBBER SLUDGE

CHARLOTTE, NORTH CAROLINA

S&ME Project 1264-03-057

Remolded Sample Conditions	Permeability		Consolidation Index, Cc	Total Porosity		Triaxial Shear				
						Test Type	Total		Effective	
	Confining Pressure (psi)	Permeability (cm/sec)		Initial	8 ksf		c (psf)	ϕ (degrees)	c' (psf)	ϕ' (degrees)
90%@12%	2.5	5.09×10^{-4}	0.09	0.47	0.43	UU	250	36.0	---	---
92%@25%	2.5	2.25×10^{-4}	0.03	0.44	0.41	UU	470	32.1	---	---
95%@19%	2.5	2.53×10^{-4}	0.12	0.44	0.40	UU	340	37.9	---	---
90%@12%	---	---	---	---	---	CU	1920	47.3	0	39.3
95%@19%	---	---	---	---	---	CU	3680	51.2	0	39.7
Literature	No Reference	1×10^{-3} to 1×10^{-6}	0.2 to 0.3	No Reference		UU	1152	39	---	---
						CU	---	---	0 to 720	31 to 39

NOTES

1. ksf = kips per square foot
2. psi = pounds per square inch
3. cm/sec = centimeters per second
4. psf = pounds per square foot
5. CU = consolidated, undrained
6. UU = unconsolidated, undrained
7. Remolded Sample Conditions not specified for Literature values.

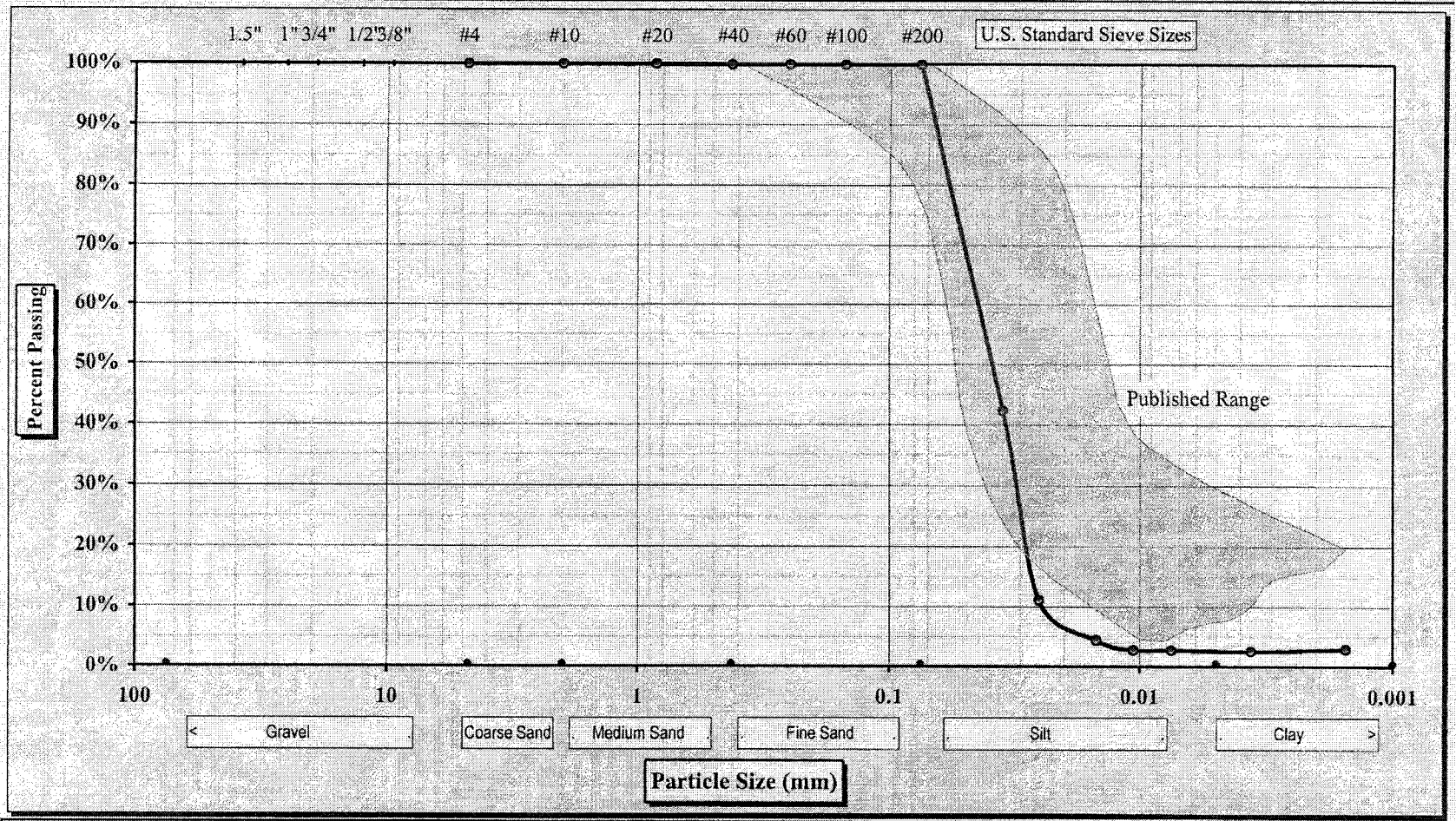
Plate 1 - Grain Size Distribution Comparison

ASTM D 422

S&ME

S&ME Project #:	**1264-03-057**	Report Date:	**April 10, 2003**
Project Name:	**Duke Energy FGD Scrubber Output Testing**	Test Date(s):	2/20/2003-2/24/2003

Boring #:	Bucket Sample	Sample #:		Sample Date:	
Location:	Confidential Powerplant	Offset:		Elevation:	
Sample Description:	FGD Scrubber Sludge				



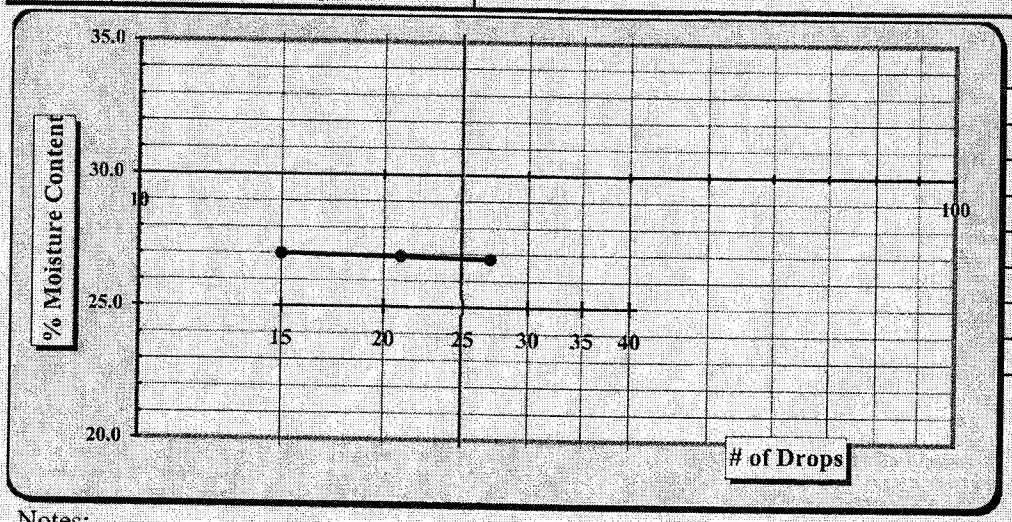
Moisture Content:	11.8%	Gravel:	0.0%	Liquid Limit:	27
Silt & Clay (% Passing #200):	99.8%	Sand:	0.2%	Plastic Limit:	26
Maximum Particle Size:	0.075 mm	Silt:	97.2%	Plastic Index:	1
Specific Gravity:	2.350	Clay:	2.6%	Colloids:	2.6%

Liquid Limit, Plastic Limit, and Plastic Index

S&ME

Project #:	1264-03-057	Report Date:	February 24, 2003
Project Name:	Duke Energy FGD Scrubber Output Testing	Test Date(s):	2-20/2-21
Client Name:	Duke Energy		
Client Address:	Charlotte, North Carolina		
Boring #:	Bucket Sample	Lab No.	103
Sample Date:			
Location:	Confidential Powerplant	Offset:	
Elevation:			
Sample Description:	FGD Scrubber Sludge		

Pan #		Liquid Limit						Plastic Limit		
	Test #	1	2	3	4	5	6	1	2	3
	Tare #	S-2	18	32				1	S-5	
A	Tare Weight	10.90	12.11	11.52				12.12	10.83	
B	Wet Soil Weight + A	27.31	28.71	26.95				21.91	19.05	
C	Dry Soil Weight + A	23.84	25.19	23.67				19.88	17.34	
D	Water Weight (B-C)	3.47	3.52	3.28				2.03	1.71	
E	Dry Soil Weight (C-A)	12.94	13.08	12.15				7.76	6.51	
F	% Moisture Content (D/E)*100	26.8%	26.9%	27.0%				26.2%	26.3%	
N	# OF DROPS	27	21	15						
LL	LL = F * FACTOR									
Ave.	Average							26.2%		



One Point Liquid Limit			
N	Factor	N	Factor
20	0.974	26	1.005
21	0.979	27	1.009
22	0.985	28	1.014
23	0.990	29	1.018
24	0.995	30	1.022
25	1.000		

Notes:

Estimate the % Retained on the #40 Sieve 0.01

Special Sampling Methods:

Sample Preparation:	Wet Preparation ☐	Dry Preparation ☐	Air Dried ☒
Liquid limit Test:	Multipoint Method ☒	One-point Method ☐	
Classification:	ASTM D 2487 ☒	AASHTO M 145 ☐	
Liquid limit Test:	ASTM D 4318 ☒	AASHTO T 89 ☐	
Plastic limit Test:	ASTM D 4318 ☒	AASHTO T 90 ☐	

% Passing #200 Sieve	99.9
NP, Non-Plastic	☐
Liquid Limit	27
Plastic Limit	26
Plastic Index	1
Group Symbol	ML

Technician Name / Certification #: P. Birchall

Technical Responsibility / Position: William Schwartz / Construction Services Manager

Moisture - Density Report

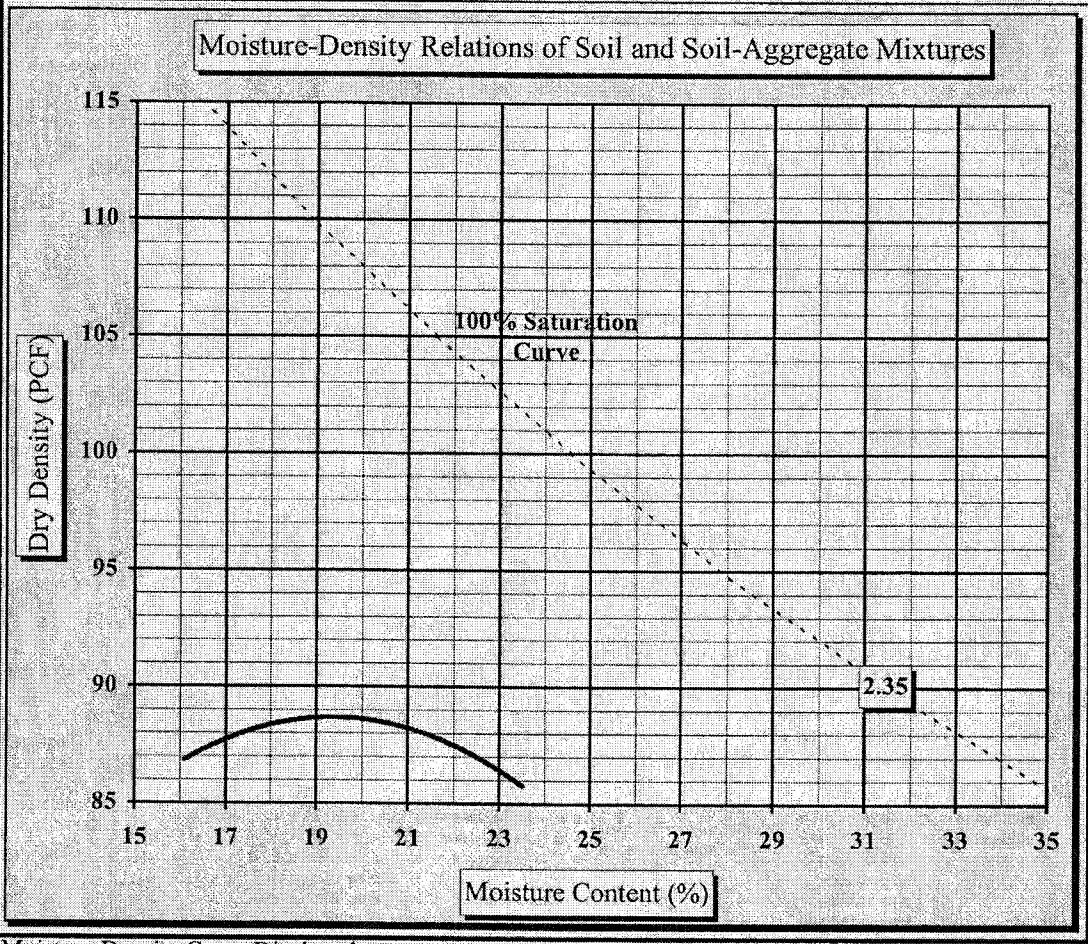
S&ME

S&ME Project #:	**1264-03-057**	Report Date:	April 10, 2003
Project Name:	**Duke Energy FGD Scrubber Output Testing**	Test Date(s):	2/20/03 - 2/21/03
Client Name:	Duke Energy		
Client Address:	Charlotte, North Carolina		

Boring #: Bucket Sample Lab No. 103 Sample Date:
Location: Confidential Powerplant Offset: Depth: 1/0/1900
Sample Description: FGD Scrubber Sludge

Maximum Dry Density 88.5 PCF. **Optimum Moisture Content 19.5 %**

ASTM D558 ☐ ASTM D 698 ☒ ASTM D1557 ☐ AASHTO T99 ☐ AASHTO T180 ☐
Method A ☒ Method B ☐ Method C ☐ Method D (1978) ☐ AASHTO Method D ☐



Soil Properties

Natural Moisture Content	11.8%
Liquid Limit:	27
Plastic Limit:	26
Plastic Index:	1
Specific Gravity:	2.350

% Passing

3/4"	
1/2"	
3/8"	
#4	
#10	
#40	
#60	
#200	99.8

Oversize Fraction

Bulk Sp. Gravity
% Moisture
Oversize Fraction

Moisture-Density Curve Displayed: Fine Fraction ☒ Corrected for Oversize Fraction (ASTM D 4718) ☐
Sieve Size used to separate the Oversize Fraction: #4 Sieve ☒ 3/8 inch Sieve ☐ 3/4 inch Sieve ☐
Mechanical Hammer ☐ Manual Hammer ☒ Moist Preparation ☒ Dry Preparation ☐

References: ASTM D 698: Laboratory Compaction Characteristics of Soil Using Standard Effort

ASTM D 2216: Laboratory Determination of Water (Moisture) Content of Soil and Rock by Mass

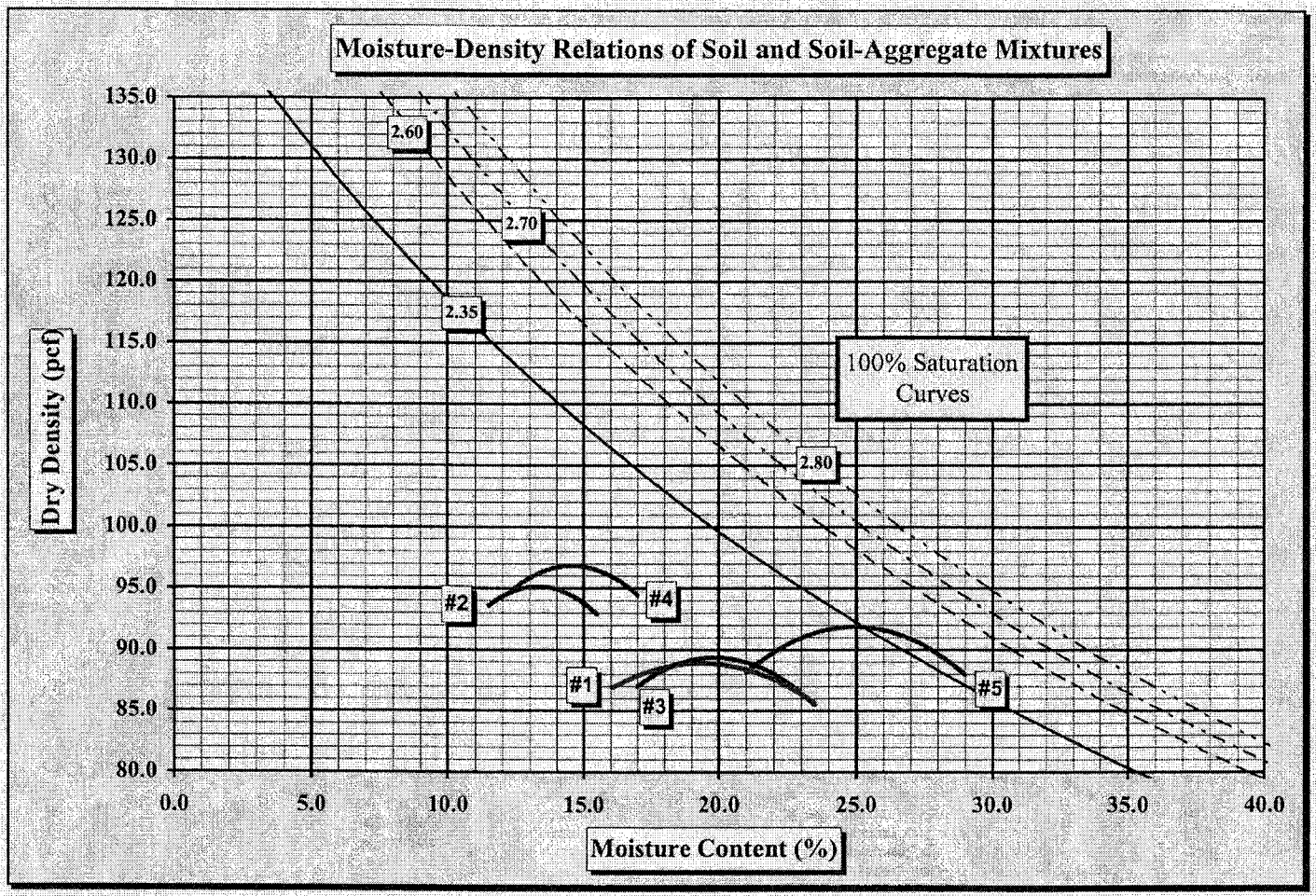
Technical Responsibility: William Schwartz Construction Services Manager

Plate 2 - Standard Proctor Comparison

Project #: 1264-03-057
Project Name: Duke Energy FGD Scrubber Output Testing
Client Name: Duke Energy
Client Address: Charlotte, North Carolina

Test Date: **April 10, 2003**
Test Date: 2/20/03 - 2/21/03

ASTM D 698 ☒ **ASTM D1557** ☐ **AASHTO T180** ☐ **AASHTO T99** ☐



Curve #	Soil Description	Method	Opt. MC	MDD	LL	PI	Nat.%W
1	FGD Scrubber Sludge	D 698A	19.50%	88.5	27	1	11.8%
2	Krizek, Chu and Atmatzidis (undated)						
3	Krizek, Chu and Atmatzidis (undated)						
4	Krizek, Chu and Atmatzidis (undated)						
5	Krizek, Chu and Atmatzidis (undated)						

References: ASTM D 698: Laboratory Compaction Characteristics of Soil Using Standard Effort
ASTM D 2487: Classification of Soils for Engineering Purposes (Unified Soil Classification System)
ASTM D 4318: Liquid Limit, Plastic Limit, & Plastic Index of Soils AASHTO T90: Determining the Plastic Limit & Plastic Index of Soi

Name (Technical Responsibility) Position

CONSTANT HEAD PERMEABILITY

ASTM D5084

S&ME Project Number: 1264-03-057 | Project Name: FGD SCRUBBER WASTE TESTING | Date: 3/17/03

Sample Name: BULK 90%

Sample Description: FGD SCRUBBER SLUDGE

Type of Sample: Remolded

Sample Data		Molding Moisture Content		Target Molding Conditon			Additonal Data	
D (cm)	7.297	Tare No.	O	γ_{wet} (pcf)	88.7		Panel No	7
L (cm)	7.62	WWS+ Tare (gm)	284.15	γ_{dry} (pcf)	79.2		Cell P (psi)	83.5
A (cm^2)	41.82	WDS+Tare (gm)	262.07	w (%)	12.0		Soil P (psi)	81.0
V (cm^3)	318.66	WT of Water (gm)	22.08	Actual Molding Condition			Back P (psi)	80.0
WWS (gm)	452.3	WT of TARE (gm)	81.21	γ_{wet} (pcf)	88.6		Gs	2.35
		WT Dry Soil (gm)	180.86	γ_{dry} (pcf)	79.0		n	0.28
		w (%)	12.2	w (%)	12.2			

$K = (Q \times L) / (t \times H \times A)$

Run	Temp. (^{0}C)	Hour	Minute	Time, t (sec)	BURRETTE 1 (cc)	BURRETTE 2 (cc)	BURRETTE 3 (cc)	Q (cc)	Δ H (cm)	Temp Corr.	K(cm/sec
1		0	0		16.1	0.2	24.0				
	21	0	2	120	16.1	24.0	0.0	23.9	70.4	0.9760	5.03E-04
2		0	0		16.1	0.1	24.3				
	21	0	2	120	16.1	24.0	0.1	24.1	70.4	0.9760	5.06E-04
3		0	0		16.1	0.1	24.5				
	21	0	2	120	16.2	24.1	0.2	24.2	70.4	0.9760	5.09E-04
4		0	0		16.2	0.2	24.3				
	21	0	2	120	16.2	24.4	0.1	24.2	70.4	0.9760	5.10E-04
5		0	0		16.2	0.2	24.7				
	21	0	2	120	16.2	24.5	0.3	24.4	70.4	0.9760	5.13E-04
6											
7										AVG. K	5.09E-04
8										(Runs 2 thru 5)	
9											
10											

CONSTANT HEAD PERMEABILITY

ASTM D5084

S&ME Project Number: 1264-03-057 | Project Name: FGD SCRUBBER WASTE TESTING | Date: 3/25/03

Sample Name: BULK 92%

Sample Description: FGD SCRUBBER SLUDGE

Type of Sample: Remolded

Sample Data		Molding Moisture Content		Target Molding Conditon			Additonal Data	
D (cm)	7.341	Tare No.	G-10	γ_{wet} (pcf)	101.3		Panel No	7
L (cm)	7.541	WWS+ Tare (gm)	486.8	γ_{dry} (pcf)	81		Cell P (psi)	83.5
A (cm^2)	42.33	WDS+Tare (gm)	407.5	w (%)	25.0		Soil P (psi)	81.0
V (cm^3)	319.18	WT of Water (gm)	79.3	Actual Molding Condition			Back P (psi)	80.0
WWS (gm)	515.2	WT of TARE (gm)	81.51	γ_{wet} (pcf)	100.8		Gs	2.35
		WT Dry Soil (gm)	325.99	γ_{dry} (pcf)	81.1		n	0.35
		w (%)	24.3	w (%)	24.3			

$K = (Q \times L) / (t \times H \times A)$

Run	Temp. (^{0}C)	Hour	Minute	Time, t (sec)	BURRETTE 1 (cc)	BURRETTE 2 (cc)	BURRETTE 3 (cc)	Q (cc)	Δ H (cm)	Temp Corr.	K(cm/sec
1		0	0		17.7	0.2	24.5				
	21	0	4	240	17.7	24.5	0.2	24.3	70.4	0.9760	2.50E-04
2		0	0		17.7	0.0	24.6				
	21	0	4	240	17.7	22.0	2.5	22.1	70.4	0.9760	2.27E-04
3		0	0		17.7	0.1	24.8				
	21	0	4	240	17.7	22.3	2.4	22.3	70.4	0.9760	2.30E-04
4		0	0		17.7	0.2	24.7				
	21	0	4	240	17.7	21.3	3.5	21.2	70.4	0.9760	2.18E-04
5		0	0		17.7	0.0	24.8				
	21	0	4	240	17.7	22.0	2.6	22.1	70.4	0.9760	2.28E-04
6											
7										AVG. K	2.25E-04
8										(Runs 2 thru 5)	
9											
10											

CONSTANT HEAD PERMEABILITY

ASTM D5084

S&ME Project Number: 1264-03-057 **Project Name: FGD SCRUBBER WASTE TESTING** **Date: 3/26/03**

Sample Name: BULK 95%

Sample Description: FGD SCRUBBER SLUDGE

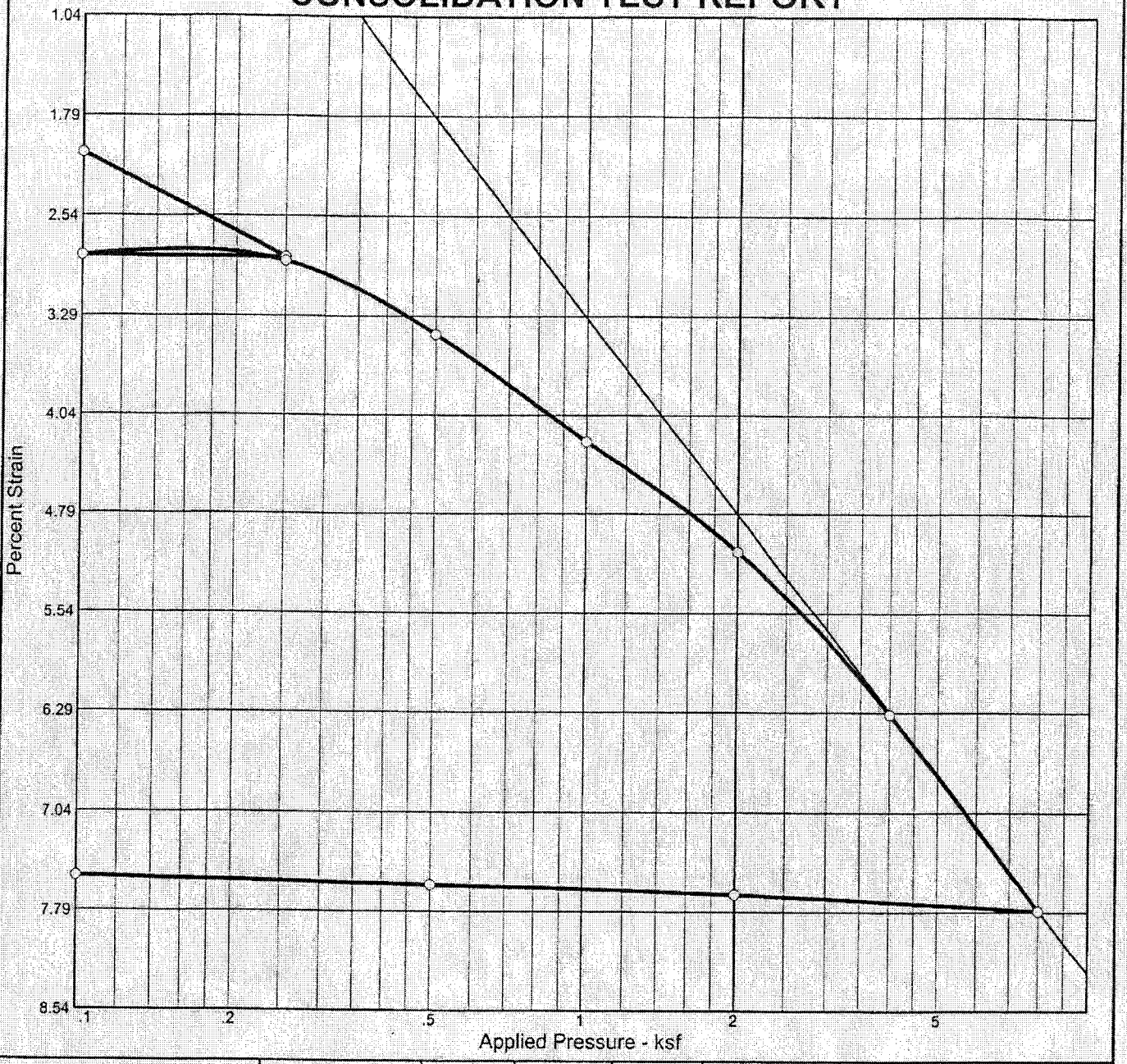
Type of Sample: Remolded

Sample Data		Molding Moisture Content		Target Molding Conditon			Additonal Data	
D (cm)	7.277	Tare No.	O	γ_{wet} (pcf)	99.5		Panel No	8
L (cm)	7.62	WWS+ Tare (gm)	261.88	γ_{dry} (pcf)	83.6		Cell P (psi)	83.5
A (cm^2)	41.59	WDS+Tare (gm)	232.68	w (%)	19.0		Soil P (psi)	81.0
V (cm^3)	316.92	WT of Water (gm)	29.2	**Actual Molding Condition**			Back P (psi)	80.0
WWS (gm)	507.4	WT of TARE (gm)	81.21	γ_{wet} (pcf)	99.9		Gs	2.35
		WT Dry Soil (gm)	151.47	γ_{dry} (pcf)	83.8		n	0.31
		w (%)	19.3	w (%)	19.3			

K = (Q x L) / (t x H x A)

Run	Temp. (^{0}C)	Hour	Minute	Time, t (sec)	BURRETTE 1 (cc)	BURRETTE 2 (cc)	BURRETTE 3 (cc)	Q (cc)	Δ H (cm)	Temp Corr.	K(cm/sec
1		0	0		17.8	0.0	24.8				
	22	0	4	240	17.9	24.8	0.0	24.8	70.4	0.9530	2.56E-04
2		0	0		17.9	0.3	24.8				
	22	0	4	240	17.9	24.8	0.1	24.6	70.4	0.9530	2.54E-04
3		0	0		17.9	0.1	24.8				
	22	0	4	240	18	24.6	0.1	24.6	70.4	0.9530	2.54E-04
4		0	0		18	0.1	24.7				
	21	0	4	240	18	24.5	0.1	24.5	70.4	0.9760	2.59E-04
5		0	0		18	0.1	24.7				
	21	0	4	240	18	23.0	1.5	23.1	70.4	0.9760	2.44E-04
6											
7										AVG. K	2.53E-04
8										(Runs 2 thru 5)	
9											
10											

CONSOLIDATION TEST REPORT



Natural		Dry Dens. (pcf)	LL	PI	Sp. Gr.	P_c (ksf)	C_c	Initial Void Ratio
Saturation	Moisture							
32.2 %	12.1 %	79.1	27	1	2.35	2.43	0.09	0.885

MATERIAL DESCRIPTION	USCS	AASHTO
FGD SCRUBBER SLUDGE	ML	

Project No. 1264-03-057 **Client:** DUKE ENERGY

Project: FGD SCRUBBER WASTE TESTING

Location: CONFIDENTIAL POWERPLANT

Remarks:
SATURATED/REMOLDED

CONSOLIDATION TEST REPORT

S & ME, INC.

SAMPLE S-1,(90%)

CONSOLIDATION TEST DATA

Client: DUKE ENERGY
Project: FGD SCRUBBER WASTE TESTING
Project Number: 1264-03-057

Sample Data

Source:
Sample No.: S-1 (90%)
Elev. or Depth:
Sample Length (in./cm.):
Location: CONFIDENTIAL POWERPLANT
Description: FGD SCRUBBER SLUDGE
Liquid Limit: 27
Plasticity Index: 1
USCS: ML **AASHTO:**
Figure No.: S-1,(90%)
Testing Remarks: SATURATED/REMOLDED

Test Specimen Data

TOTAL SAMPLE	BEFORE TEST	AFTER TEST
Wet w+t = 294.72 g.	Consolidometer # = 6	Wet w+t = 239.73 g.
Dry w+t = 271.73 g.		Dry w+t = 211.17 g.
Tare Wt. = 82.28 g.	Spec. Gravity = 2.35	Tare Wt. = 111.36 g.
Height = 1.00 in.	Height = 1.00 in.	
Diameter = 2.49 in.	Diameter = 2.49 in.	
Weight = 113.59 g.	Defl. Table = Consolidometer #6	
Moisture = 12.1 %	Ht. Solids = 0.5310 in.	Moisture = 28.6 %
Wet Den. = 88.7 pcf	Dry Wt. = 101.40 g.	Dry Wt. = 99.81 g.*
Dry Den. = 79.1 pcf	Void Ratio = 0.885	Void Ratio = 0.743
	Saturation = 32.2 %	

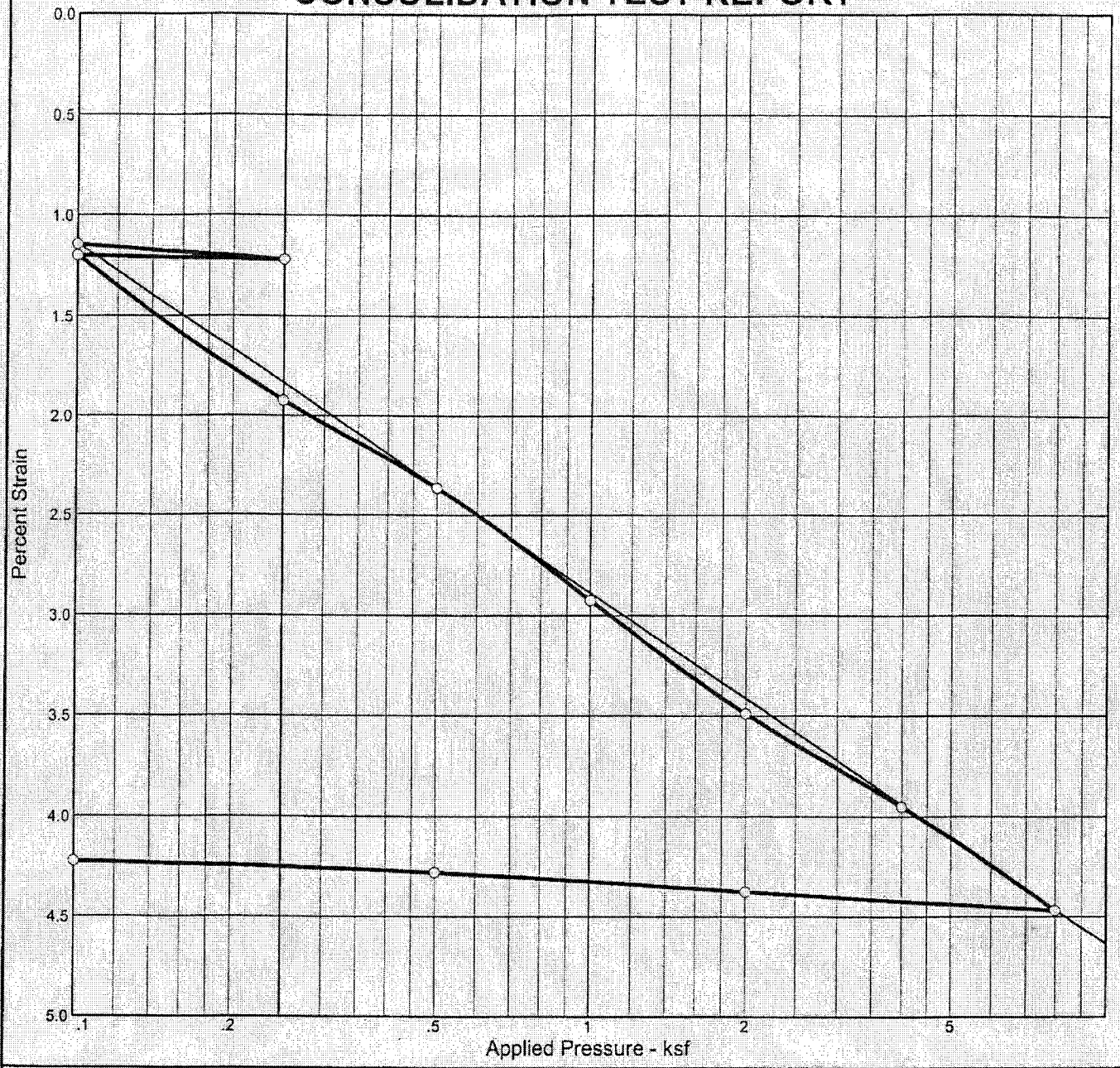
* Final dry weight used in calculations

End-of-Load Summary

Pressure (ksf)	Final Dial (in.)	Machine Defl. (in.)	C_v (ft.^{2}/day)	C_α	Void Ratio	% Compression /Swell
start	0.22390				0.885	
0.10	0.20320	0.00000	5.89		0.846	2.1 Comprs.
0.25	0.19540	0.00000	0.74		0.832	2.8 Comprs.
0.10	0.19550	0.00000	0.14		0.832	2.8 Comprs.
0.25	0.19510	0.00000	0.66		0.831	2.9 Comprs.
0.50	0.18960	0.00000	0.76		0.821	3.4 Comprs.
1.00	0.18145	0.00000	0.65		0.805	4.2 Comprs.
2.00	0.17310	0.00000	0.68		0.790	5.1 Comprs.
4.00	0.16080	0.00000	0.76		0.766	6.3 Comprs.
8.00	0.14600	0.00000	0.11		0.739	7.8 Comprs.
2.00	0.14725	0.00000	0.62		0.741	7.7 Comprs.
0.50	0.14795	0.00000	0.08		0.742	7.6 Comprs.
0.10	0.14855	0.00000	0.59		0.743	7.5 Comprs.

C_c = 0.09 P_c = 2.43 ksf C_r = 0.00

CONSOLIDATION TEST REPORT



Natural		Dry Dens. (pcf)	LL	PI	Sp. Gr.	P_c (ksf)	C_c	Initial Void Ratio
Saturation	Moisture							
74.5 %	24.8 %	81.7	27	1	2.35	0.01	0.03	0.783

MATERIAL DESCRIPTION	USCS	AASHTO
FGD SCRUBBER SLUDGE	ML	

Project No. 1264-03-057 **Client:** DUKE ENERGY

Project: FGD SCRUBBER WASTE TESTING

Location: CONFIDENTIAL POWERPLANT

Remarks:
UNSATURATED/REMOLDED

CONSOLIDATION TEST REPORT

S & ME, INC.

SAMPLE S-3,(92%)

CONSOLIDATION TEST DATA

Client: DUKE ENERGY
Project: FGD SCRUBBER WASTE TESTING
Project Number: 1264-03-057

Sample Data

Source:
Sample No.: S-3 (92%)
Elev. or Depth: **Sample Length (in./cm.):**
Location: CONFIDENTIAL POWERPLANT
Description: FGD SCRUBBER SLUDGE
Liquid Limit: 27 **Plasticity Index:** 1
USCS: ML **AASHTO:** **Figure No.:** S-3,(92%)
Testing Remarks: UNSATURATED/REMOLDED

Test Specimen Data

TOTAL SAMPLE		BEFORE TEST		AFTER TEST	
Wet w+t	= 375.53 g.	Consolidometer #	= 6	Wet w+t	= 218.86 g.
Dry w+t	= 317.04 g.			Dry w+t	= 216.79 g.
Tare Wt.	= 81.49 g.	Spec. Gravity	= 2.35	Tare Wt.	= 111.37 g.
Height	= 1.00 in.	Height	= 1.00 in.		
Diameter	= 2.49 in.	Diameter	= 2.49 in.		
Weight	= 130.69 g.	Defl. Table	= Consolidometer #6		
Moisture	= 24.8 %	Ht. Solids	= 0.5608 in.	Moisture	= 2.0 %
Wet Den.	= 102.0 pcf	Dry Wt.	= 104.69 g.	Dry Wt.	= 105.42 g.*
Dry Den.	= 81.7 pcf	Void Ratio	= 0.783	Void Ratio	= 0.708
		Saturation	= 74.5 %		

* Final dry weight used in calculations

End-of-Load Summary

Pressure (ksf)	Final Dial (in.)	Machine Defl. (in.)	C_v (ft.^{2}/day)	C_α	Void Ratio	% Compression /Swell
start	0.24230				0.783	
0.10	0.23085	0.00000	0.10		0.763	1.1 Comprs.
0.25	0.23010	0.00000	0.24		0.761	1.2 Comprs.
0.10	0.23030	0.00000	0.09		0.762	1.2 Comprs.
0.25	0.22305	0.00000	0.11		0.749	1.9 Comprs.
0.50	0.21865	0.00000	0.44		0.741	2.4 Comprs.
1.00	0.21305	0.00000	0.18		0.731	2.9 Comprs.
2.00	0.20740	0.00000	0.10		0.721	3.5 Comprs.
4.00	0.20280	0.00000	0.12		0.713	3.9 Comprs.
8.00	0.19760	0.00000	0.22		0.703	4.5 Comprs.
2.00	0.19855	0.00000	0.04		0.705	4.4 Comprs.
0.50	0.19950	0.00000	5.87		0.707	4.3 Comprs.
0.10	0.20005	0.00000	5.88		0.708	4.2 Comprs.

C_c = 0.03 P_c = 0.01 ksf C_r = 0.01

CONSOLIDATION TEST REPORT

Natural		Dry Dens. (pcf)	LL	PI	Sp. Gr.	P_c (ksf)	C_c	Initial Void Ratio
Saturation	Moisture							
57.5 %	19.4 %	83.3	27	1	2.35	1.96	0.12	0.791

MATERIAL DESCRIPTION	USCS	AASHTO
FGD SCRUBBER SLUDGE	ML	

Project No. 1264-03-057 **Client:** DUKE ENERGY

Project: FGD SCRUBBER WASTE TESTING

Location: CONFIDENTIAL POWERPLANT

Remarks:
SATURATED/REMOLDED

CONSOLIDATION TEST REPORT

S & ME, INC.

SAMPLE S-2,(95%)

CONSOLIDATION TEST DATA

Client: DUKE ENERGY
Project: FGD SCRUBBER WASTE TESTING
Project Number: 1264-03-057

Sample Data

Source:
Sample No.: S-2 (95%)
Elev. or Depth:
Sample Length (in./cm.):
Location: CONFIDENTIAL POWERPLANT
Description: FGD SCRUBBER SLUDGE
Liquid Limit: 27
Plasticity Index: 1
USCS: ML
AASHTO:
Figure No.: S-2,(95%)
Testing Remarks: SATURATED/REMOLDED

Test Specimen Data

TOTAL SAMPLE	BEFORE TEST	AFTER TEST
Wet w+t = 431.50 g.	Consolidometer # = 5	Wet w+t = 239.00 g.
Dry w+t = 374.81 g.		Dry w+t = 211.75 g.
Tare Wt. = 81.97 g.	Spec. Gravity = 2.35	Tare Wt. = 106.27 g.
Height = 1.00 in.	Height = 1.00 in.	
Diameter = 2.50 in.	Diameter = 2.50 in.	
Weight = 127.96 g.	Defl. Table = Consolidometer #5	
Moisture = 19.4 %	Ht. Solids = 0.5584 in.	Moisture = 25.8 %
Wet Den. = 99.4 pcf	Dry Wt. = 107.21 g.	Dry Wt. = 105.48 g.*
Dry Den. = 83.3 pcf	Void Ratio = 0.791	Void Ratio = 0.660
	Saturation = 57.5 %	

*** Final dry weight used in calculations**

End-of-Load Summary

Pressure (ksf)	Final Dial (in.)	Machine Defl. (in.)	C_v (ft.^{2}/day)	C_α	Void Ratio	% Compression /Swell
start	0.24200				0.791	
0.10	0.23400	0.00000	0.53		0.776	0.8 Comprs.
0.25	0.22630	0.00000	6.13		0.762	1.6 Comprs.
0.10	0.22650	0.00000	0.00		0.763	1.6 Comprs.
0.25	0.22350	0.00000	0.22		0.757	1.9 Comprs.
0.50	0.22030	0.00000	0.01		0.752	2.2 Comprs.
1.00	0.21245	0.00000	0.17		0.738	3.0 Comprs.
2.00	0.20020	0.00000	0.03		0.716	4.2 Comprs.
4.00	0.18530	0.00000	0.07		0.689	5.7 Comprs.
8.00	0.16470	0.00000	0.17		0.652	7.7 Comprs.
2.00	0.16595	0.00000	4.80		0.654	7.6 Comprs.
0.50	0.16675	0.00000	2.35		0.656	7.5 Comprs.
0.10	0.16925	0.00000	1.30		0.660	7.3 Comprs.

C_c = 0.12 P_c = 1.96 ksf C_r = 0.01

Project and Sample Data

Date: 4/13/2003
Client: DUKE ENERGY
Project: FGD SCRUBBER WASTE TESTING
Sample location: CONFIDENTIAL POWERPLANT
Sample description: FGD SCRUB. SLUDGE
Remarks: BULK-90% @12%

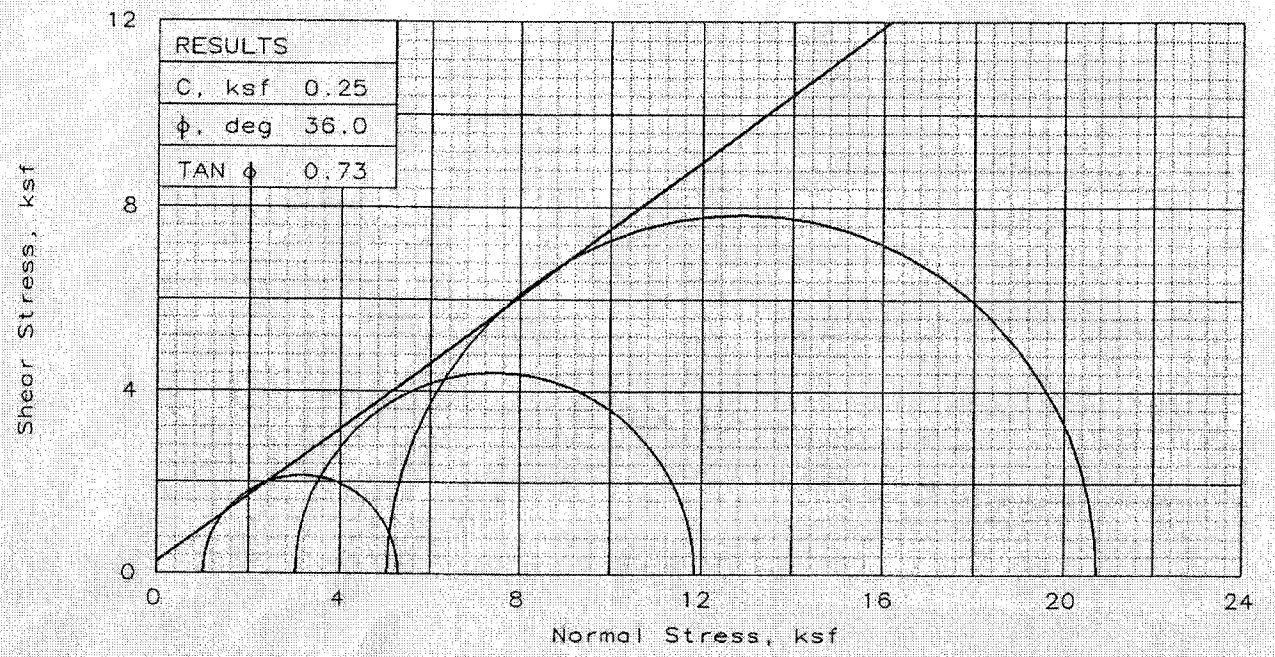
Fig no.: BULK-90% 2nd page Fig no. (if applicable):
Type of sample: REMOLDED
Specific gravity= 2.35 LL= 27 PL= 26 PI= 1
Test method: ASTM - Method A

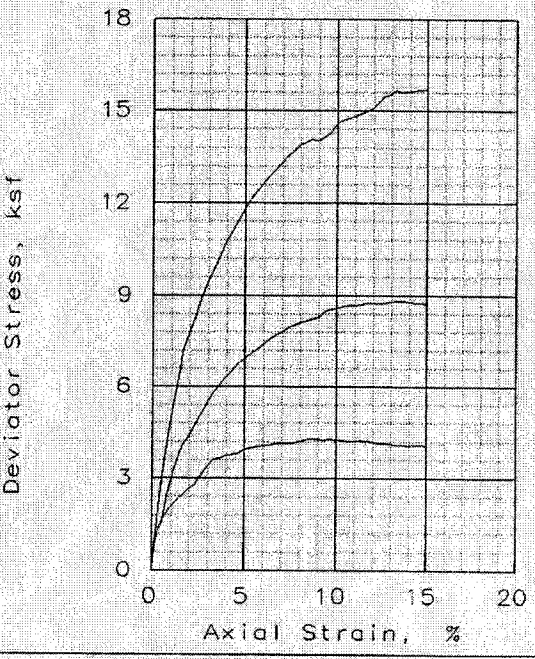
Specimen Parameters for Specimen No. 1

Specimen Parameter	Initial	Saturated	Final
Wt. moist soil and tare:	338.960		622.500
Wt. dry soil and tare:	310.480		570.100
Wt. of tare:	81.840		153.260
Weight, gms:	908.2		
Diameter, in:	2.873	2.873	
Area, in²:	6.483	6.483	
Height, in:	6.000	6.000	
Net decrease in height, in:		0.000	
% Moisture:	12.5	12.6	12.6
Wet density, pcf:	89.0	89.0	
Dry density, pcf:	79.1	79.1	
Void ratio:	0.8547	0.8547	
% Saturation:	34.2	34.6	

Test Readings Data for Specimen No. 1

Deformation dial constant= 0.001 in per input unit
Primary load ring constant= 1 lbs per input unit
Secondary load ring constant= 1 lbs per input unit
Crossover reading for secondary load ring= 45 input units
Cell pressure = 7.00 psi = 1.01 ksf
Back pressure = 0.00 psi = 0.00 ksf
Effective confining stress = 1.01 ksf
Strain rate, in/min = 0.0040
FAIL. STRESS = 4.28 ksf at reading no. 31
ULT. STRESS = not selected





SAMPLE NO.:		1	2	3
INITIAL	WATER CONTENT, %	12.5	12.5	12.5
	DRY DENSITY, pcf	79.1	79.0	78.4
	SATURATION, %	34.2	34.1	33.6
	VOID RATIO	0.855	0.858	0.871
	DIAMETER, in	2.87	2.87	2.87
	HEIGHT, in	6.00	6.00	6.03
AT TEST	WATER CONTENT, %	12.6	12.5	12.1
	DRY DENSITY, pcf	79.1	79.0	78.4
	SATURATION, %	34.6	34.3	32.7
	VOID RATIO	0.855	0.858	0.871
	DIAMETER, in	2.87	2.87	2.87
	HEIGHT, in	6.00	6.00	6.03
Strain rate, in/min		0.0040	0.0040	0.0040
BACK PRESSURE, ksf		0.0	0.0	0.0
CELL PRESSURE, ksf		1.0	3.0	5.0
FAIL. STRESS, ksf		4.3	8.8	15.7
STRAIN, %		8.7	13.7	14.9
ULT. STRESS, ksf				
STRAIN, %				
σ_1 FAILURE, ksf		5.3	11.9	20.7
σ_3 FAILURE, ksf		1.0	3.0	5.0

TYPE OF TEST:
Unconsolidated Undrained
SAMPLE TYPE: REMOLDED
DESCRIPTION: FGD SCRUB. SLUDGE

LL= 27 PL= 26 PI= 1
SPECIFIC GRAVITY= 2.35
REMARKS: BULK-90% @12%

Fig. No.: BULK-90%

CLIENT: DUKE ENERGY

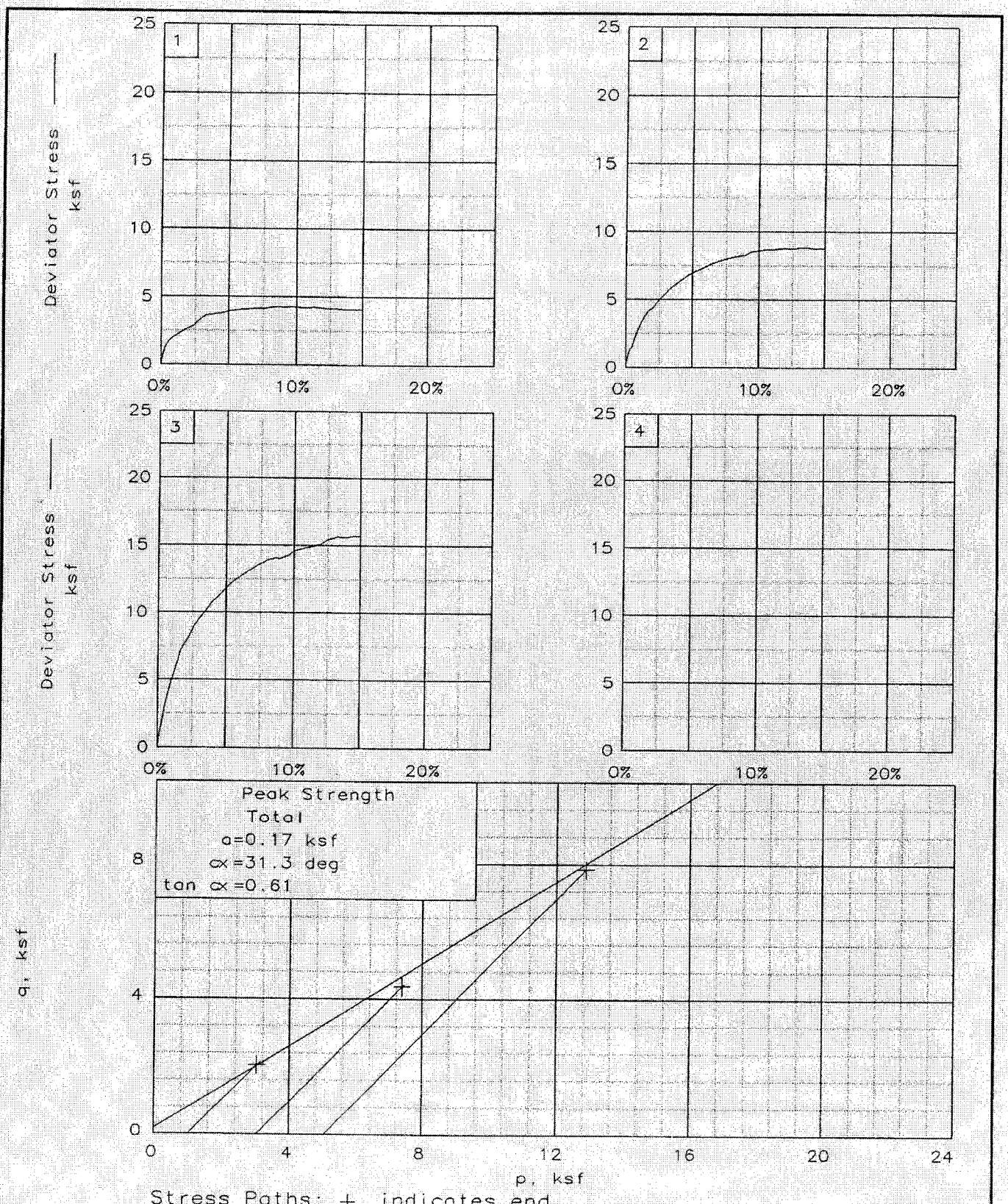
PROJECT: FGD SCRUBBER WASTE TESTING

SAMPLE LOCATION: CONFIDENTIAL POWERPLANT

PROJ. NO.: 1264-03-057 DATE: 4/13/2003

TRIAXIAL SHEAR TEST REPORT

S & ME, INC.



Stress Paths: + indicates end

Client: DUKE ENERGY

Project: FGD SCRUBBER WASTE TESTING

Location: CONFIDENTIAL POWERPLANT

File: FGD-90% Project No.: 1264-03-057 Fig. No.: ____

Project and Sample Data

Date: 3-27-03
Client: DUKE ENERGY
Project: FGD SCRUBBER WASTE TESTING
Sample location: CONFIDENTIAL POWERPLANT
Sample description: FGD SCRUB. SLUDGE
Remarks: BULK-92% @ 25%

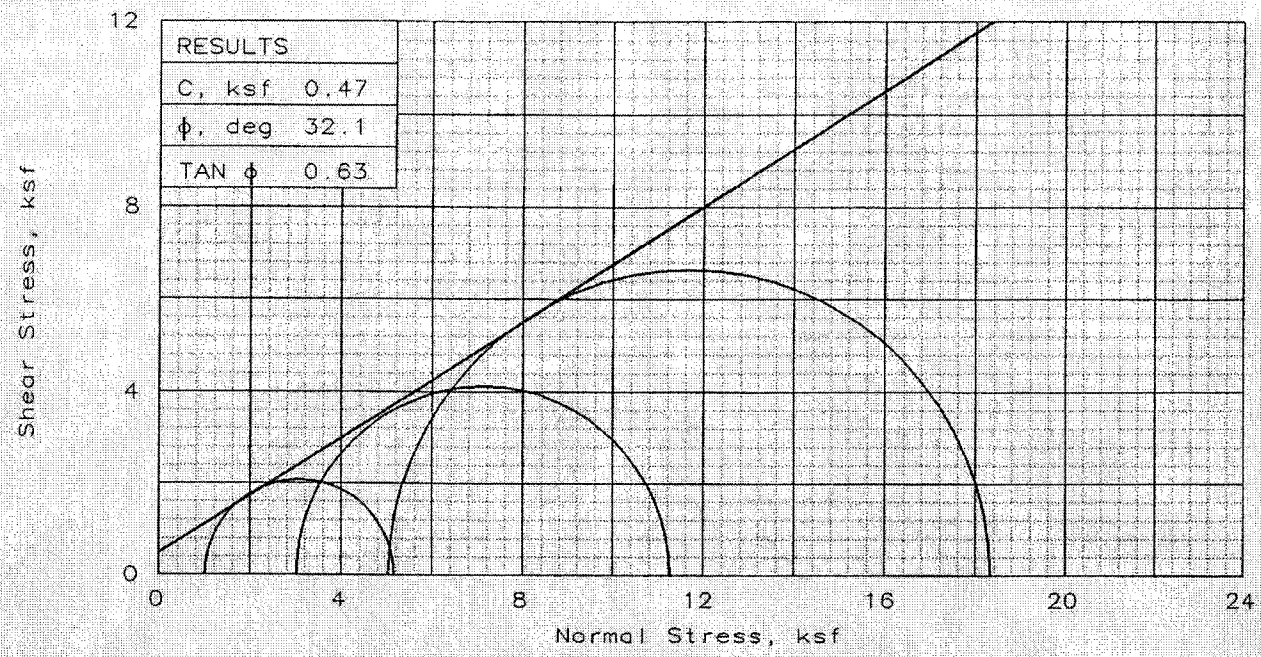
Fig no.: BULK-92% 2nd page Fig no. (if applicable):
Type of sample: REMOLDED
Specific gravity= 2.35 LL= 27 PL= 26 PI= 1
Test method: ASTM - Method A

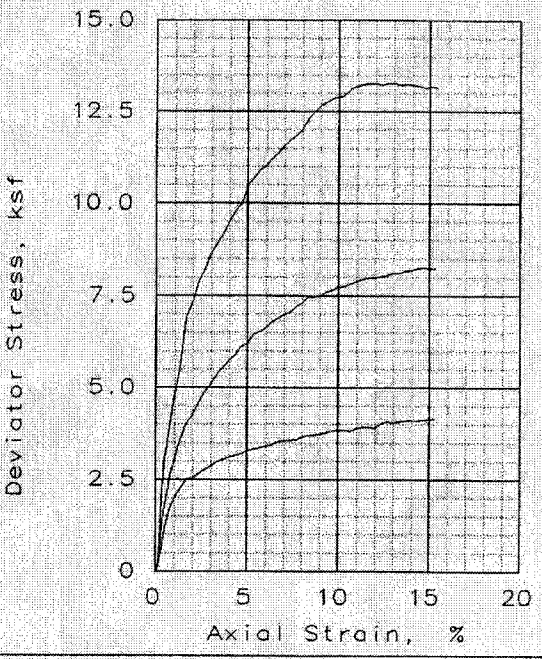
Specimen Parameters for Specimen No. 1

Specimen Parameter	Initial	Saturated	Final
Wt. moist soil and tare:	358.150		691.600
Wt. dry soil and tare:	304.650		591.000
Wt. of tare:	81.570		153.660
Weight, gms:	1034.0		
Diameter, in:	2.872	2.872	
Area, in²:	6.478	6.478	
Height, in:	6.000	6.000	
Net decrease in height, in:		0.000	
% Moisture:	24.0	23.0	23.0
Wet density, pcf:	101.3	100.5	
Dry density, pcf:	81.7	81.7	
Void ratio:	0.7948	0.7948	
% Saturation:	70.9	68.0	

Test Readings Data for Specimen No. 1

Deformation dial constant= 0.001 in per input unit
Primary load ring constant= 1 lbs per input unit
Secondary load ring constant= 1 lbs per input unit
Crossover reading for secondary load ring= 45 input units
Cell pressure = 7.00 psi = 1.01 ksf
Back pressure = 0.00 psi = 0.00 ksf
Effective confining stress = 1.01 ksf
Strain rate, in/min = 0.0040
FAIL. STRESS = 4.16 ksf at reading no. 50
ULT. STRESS = not selected





SAMPLE NO. :		1	2	3
INITIAL	WATER CONTENT, %	24.0	24.0	24.0
	DRY DENSITY, pcf	81.7	81.4	81.8
	SATURATION, %	70.9	70.2	71.1
	VOID RATIO	0.795	0.803	0.793
	DIAMETER, in	2.87	2.88	2.88
	HEIGHT, in	6.00	6.00	5.97
AT TEST	WATER CONTENT, %	23.0	25.6	25.6
	DRY DENSITY, pcf	81.7	81.4	81.8
	SATURATION, %	68.0	74.9	75.7
	VOID RATIO	0.795	0.803	0.793
	DIAMETER, in	2.87	2.88	2.88
	HEIGHT, in	6.00	6.00	5.97
Strain rate, in/min		0.0040	0.0040	0.0040
BACK PRESSURE, ksf		0.0	0.0	0.0
CELL PRESSURE, ksf		1.0	3.0	5.0
FAIL. STRESS, ksf		4.2	8.2	13.3
STRAIN, %		15.3	14.7	13.1
ULT. STRESS, ksf				
STRAIN, %				
σ_1 FAILURE, ksf		5.2	11.3	18.3
σ_3 FAILURE, ksf		1.0	3.0	5.0

TYPE OF TEST:
Unconsolidated Undrained
SAMPLE TYPE: REMOLDED
DESCRIPTION: FGD SCRUB. SLUDGE

LL= 27 PL= 26 PI= 1
SPECIFIC GRAVITY= 2.35
REMARKS: BULK-92% @ 25%

Fig. No.: BULK-92%

CLIENT: DUKE ENERGY

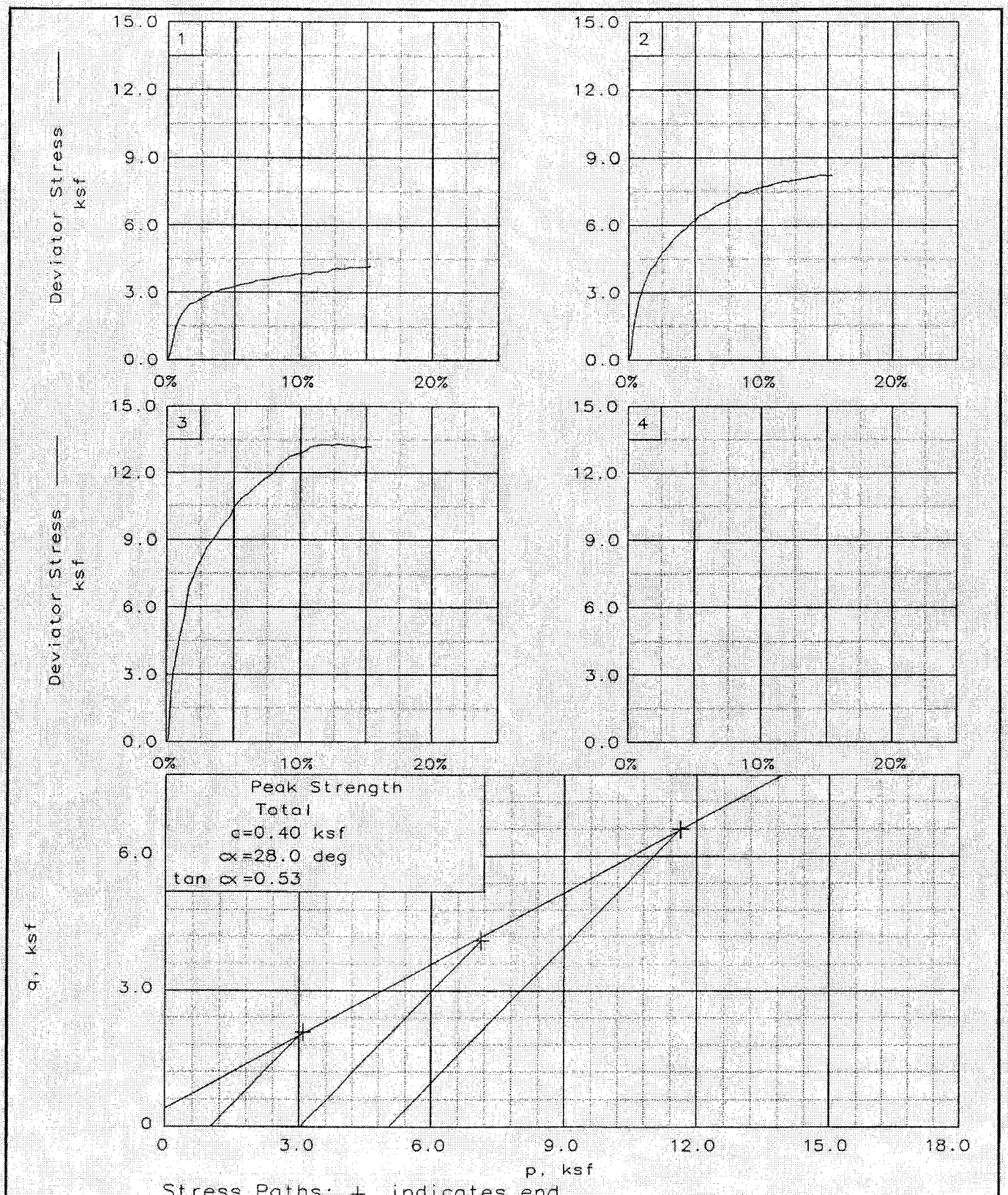
PROJECT: FGD SCRUBBER WASTE TESTING

SAMPLE LOCATION: CONFIDENTIAL POWERPLANT

PROJ. NO.: 1264-03-057 DATE: 3-27-03

TRIAXIAL SHEAR TEST REPORT

S & ME, INC.



Client: DUKE ENERGY

Project: FGD SCRUBBER WASTE TESTING

Location: CONFIDENTIAL POWERPLANT

File: BULK 92% Project No.: 1264-03-057 Fig. No.: ____

Project and Sample Data

Date: 4/16/03
Client: DUKE ENERGY
Project: FGD SCRUBBER WASTE TESTING
Sample location: CONFIDENTIAL POWERPLANT
Sample description: FGD SCRUB. SLUDGE
Remarks: BULK-95% @ 19%

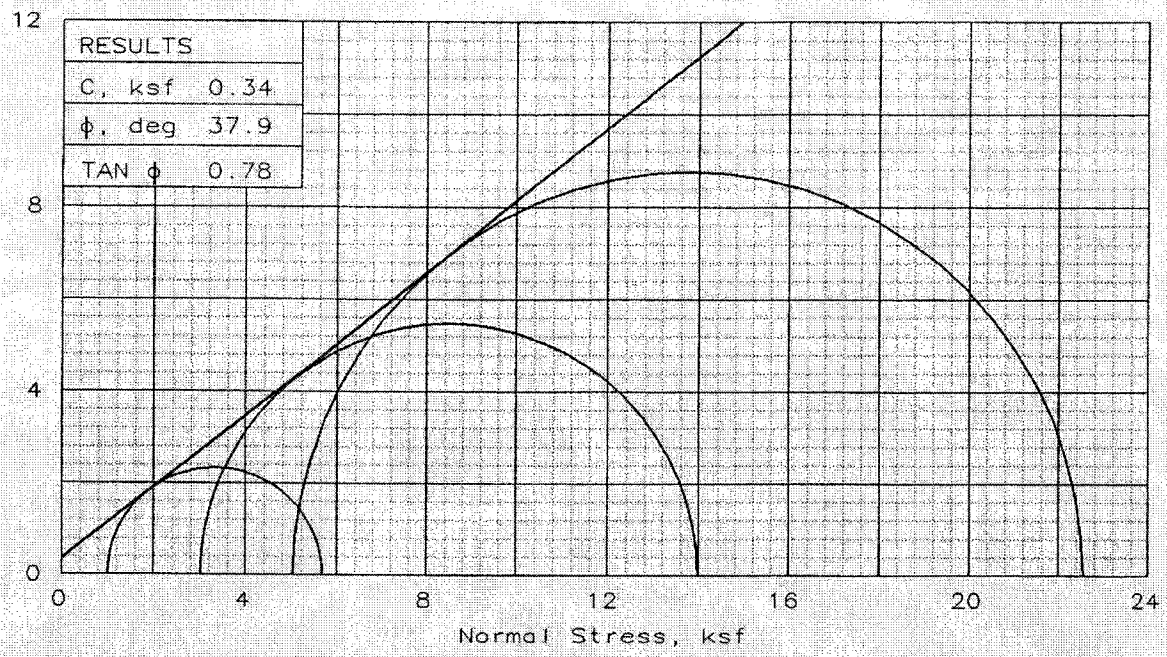
Fig no.: BULK-95% 2nd page Fig no. (if applicable):
Type of sample: REMOLDED
Specific gravity= 2.35 LL= 27 PL= 26 PI= 1
Test method: ASTM - Method A

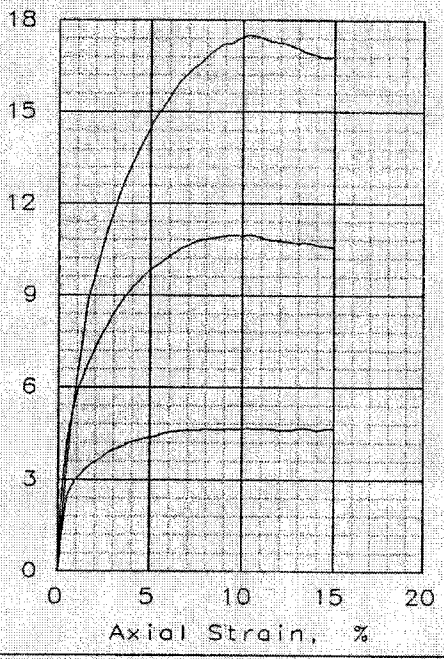
Specimen Parameters for Specimen No. 1

Specimen Parameter	Initial	Saturated	Final
Wt. moist soil and tare:	255.000		669.800
Wt. dry soil and tare:	226.510		574.300
Wt. of tare:	81.290		73.210
Weight, gms:	1015.3		
Diameter, in:	2.873	2.873	
Area, in²:	6.483	6.483	
Height, in:	6.000	6.000	
Net decrease in height, in:		0.000	
% Moisture:	19.6	19.1	19.1
Wet density, pcf:	99.4	99.0	
Dry density, pcf:	83.1	83.1	
Void ratio:	0.7648	0.7648	
% Saturation:	60.3	58.6	

Test Readings Data for Specimen No. 1

Deformation dial constant= 0.001 in per input unit
Primary load ring constant= 1 lbs per input unit
Secondary load ring constant= 1 lbs per input unit
Crossover reading for secondary load ring= 45 input units
Cell pressure = 7.00 psi = 1.01 ksf
Back pressure = 0.00 psi = 0.00 ksf
Effective confining stress = 1.01 ksf
Strain rate, in/min = 0.0040
FAIL. STRESS = 4.66 ksf at reading no. 36
ULT. STRESS = not selected





SAMPLE NO.:		1	2	3
INITIAL	WATER CONTENT, %	19.6	19.6	19.2
	DRY DENSITY, pcf	83.1	83.2	83.6
	SATURATION, %	60.3	60.4	59.6
	VOID RATIO	0.765	0.763	0.755
	DIAMETER, in	2.87	2.87	2.87
	HEIGHT, in	6.00	6.00	6.00
AT TEST	WATER CONTENT, %	19.1	18.6	18.3
	DRY DENSITY, pcf	83.1	83.2	83.6
	SATURATION, %	58.6	57.1	57.1
	VOID RATIO	0.765	0.763	0.755
	DIAMETER, in	2.87	2.87	2.87
	HEIGHT, in	6.00	6.00	6.00
Strain rate, in/min		0.0040	0.0040	0.0040
BACK PRESSURE, ksf		0.0	0.0	0.0
CELL PRESSURE, ksf		1.0	3.0	5.0
FAIL. STRESS, ksf		4.7	11.0	17.5
STRAIN, %		10.3	10.3	10.3
ULT. STRESS, ksf				
STRAIN, %				
σ_1 FAILURE, ksf		5.7	14.0	22.6
σ_3 FAILURE, ksf		1.0	3.0	5.0

TYPE OF TEST:
Unconsolidated Undrained
SAMPLE TYPE: REMOLDED
DESCRIPTION: FGD SCRUB. SLUDGE

LL= 27 PL= 26 PI= 1
SPECIFIC GRAVITY= 2.35
REMARKS: BULK-95% @ 19%

Fig. No.: BULK-95%

CLIENT: DUKE ENERGY

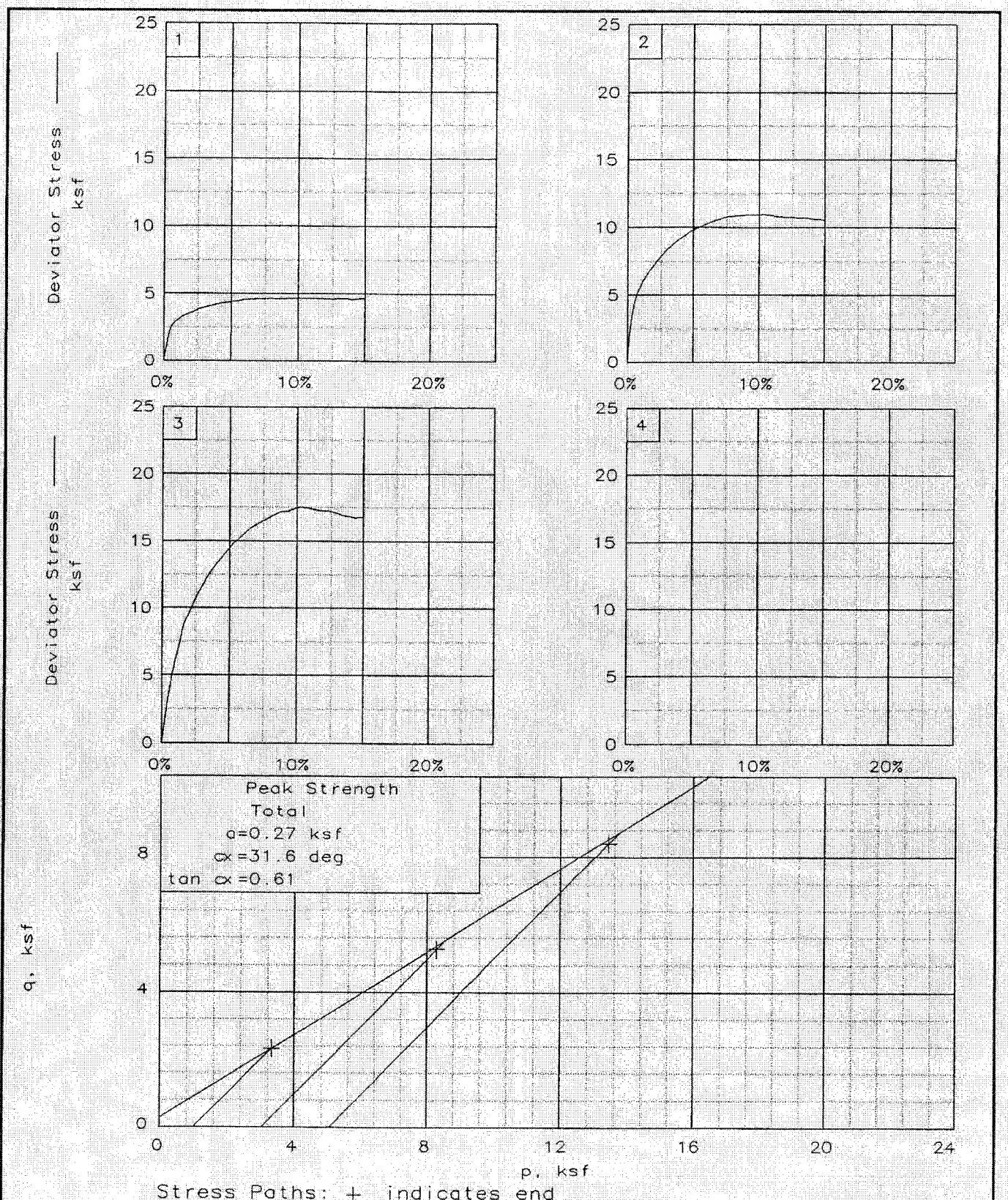
PROJECT: FGD SCRUBBER WASTE TESTING

SAMPLE LOCATION: CONFIDENTIAL POWERPLANT

PROJ. NO.: 1264-03-057 DATE: 4/16/03

TRIAXIAL SHEAR TEST REPORT

S & ME, INC.



Client: DUKE ENERGY

Project: FGD SCRUBBER WASTE TESTING

Location: CONFIDENTIAL POWERPLANT

File: FGD95 Project No.: 1264-03-057 Fig. No.: _____

Project and Sample Data

Date: 3/18/03
Client: DUKE ENERGY
Project: FGD SCRUBBER WASTE TESTING
Sample location: CONFIDENTIAL POWERPLANT
Sample description: FGD SCRUB. SLUDGE
Remarks: BULK-90% @ 12%

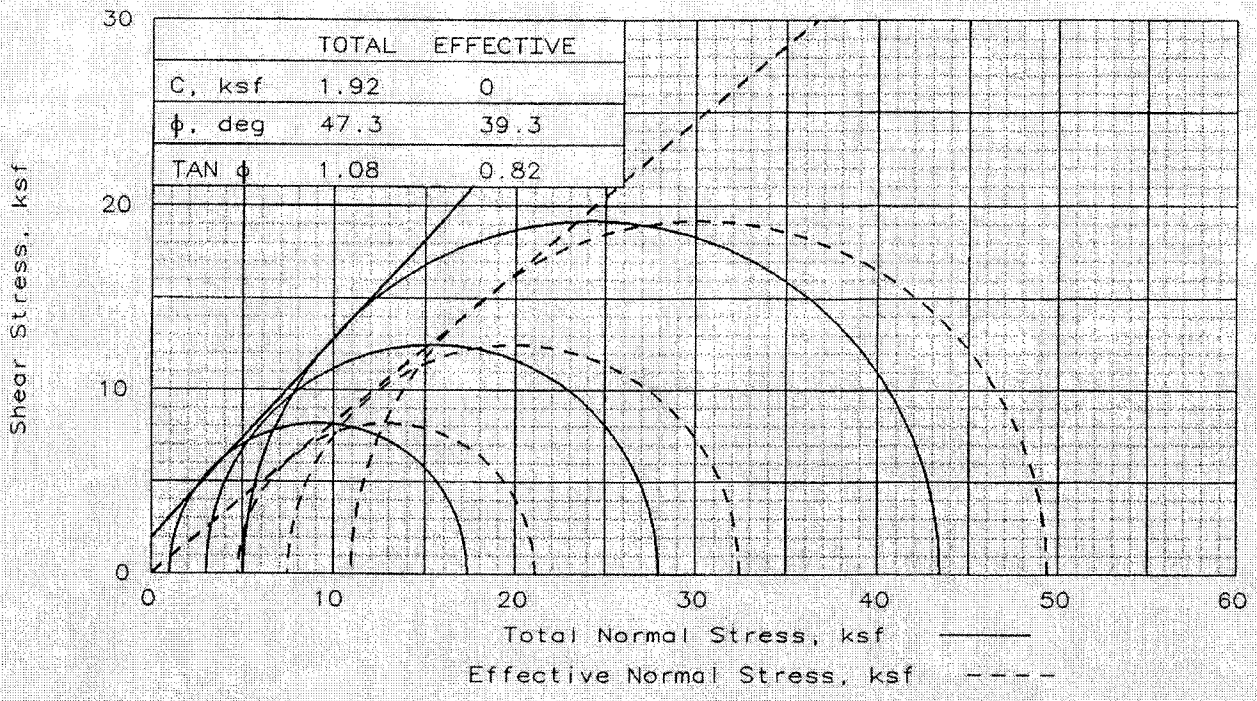
Fig no.: BULK-90% 2nd page Fig no. (if applicable):
Type of sample: REMOLDED
Specific gravity= 2.35 LL= 27 PL= 26 PI= 1
Test method: Corps of Eng. - uniform strain

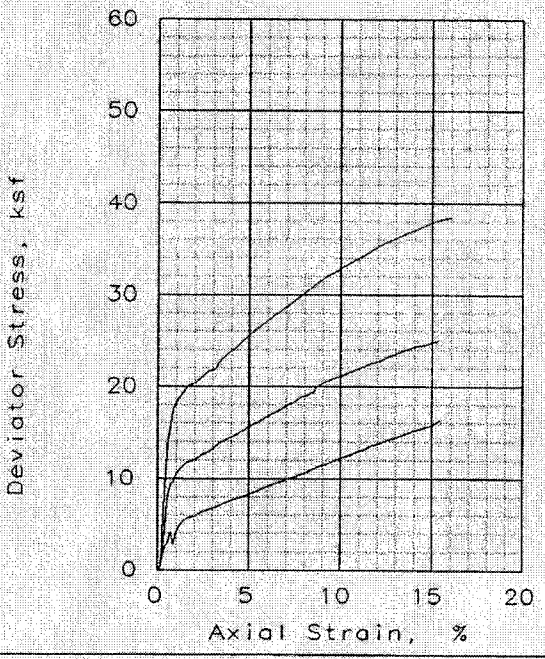
Specimen Parameters for Specimen No. 1

Specimen Parameter	Initial	Saturated	Consolidated	Final
Wt. moist soil and tare:	284.150			656.600
Wt. dry soil and tare:	262.070			518.400
Wt. of tare:	81.210			81.540
Weight, gms:	905.9			
Diameter, in:	2.874	2.874	2.795	
Area, in²:	6.487	6.487	6.135	
Height, in:	6.000	6.000	5.837	
Net decrease in height, in:		0.000	0.163	
Net decrease in water volume, cc:				
% Moisture:	12.2	36.5	30.1	31.6
Wet density, pcf:	88.7	107.8	111.8	
Dry density, pcf:	79.0	79.0	85.9	
Void ratio:	0.8566	0.8566	0.7083	
% Saturation:	33.5	100.0	100.0	

Test Readings Data for Specimen No. 1

Deformation dial constant= 0.001 in per input unit
Primary load ring constant= 1 lbs per input unit
Secondary load ring constant= 1 lbs per input unit
Crossover reading for secondary load ring= 45 input units
Consolidation cell pressure = 87.00 psi = 12.53 ksf
Consolidation back pressure = 80.00 psi = 11.52 ksf
Consolidation effective confining stress = 1.01 ksf
Strain rate, in/min = 0.0040
FAIL. STRESS = 16.36 ksf at reading no. 50
ULT. STRESS = not selected





SAMPLE NO.:		1	2	3
INITIAL	WATER CONTENT, %	12.2	12.2	12.3
	DRY DENSITY, pcf	79.0	78.8	79.4
	SATURATION, %	33.5	33.3	34.0
	VOID RATIO	0.857	0.862	0.847
	DIAMETER, in	2.87	2.87	2.87
	HEIGHT, in	6.00	6.03	5.97
AT TEST	WATER CONTENT, %	30.1	30.6	27.2
	DRY DENSITY, pcf	85.9	85.4	89.4
	SATURATION, %	100.0	100.0	100.0
	VOID RATIO	0.708	0.719	0.640
	DIAMETER, in	2.79	2.80	2.76
	HEIGHT, in	5.84	5.87	5.74
Strain rate, in/min		0.0040	0.0040	0.0040
EFF CELL PRESSURE, ksf		1.0	3.0	5.0
FAIL. STRESS, ksf		16.4	25.0	38.4
TOTAL PORE PR., ksf		7.8	7.1	5.6
STRAIN, %		15.4	15.3	16.0
ULT. STRESS, ksf				
TOTAL PORE PR., ksf				
STRAIN, %				
$\bar{\sigma}_1$ FAILURE, ksf		21.1	32.5	49.4
$\bar{\sigma}_3$ FAILURE, ksf		4.8	7.5	11.0

TYPE OF TEST:
CU with Pore Pressures
SAMPLE TYPE: REMOLDED
DESCRIPTION: FGD SCRUB. SLUDGE

LL= 27 PL= 26 PI= 1
SPECIFIC GRAVITY= 2.35
REMARKS: BULK-90% @ 12%

Fig. No.: BULK-90%

CLIENT: DUKE ENERGY

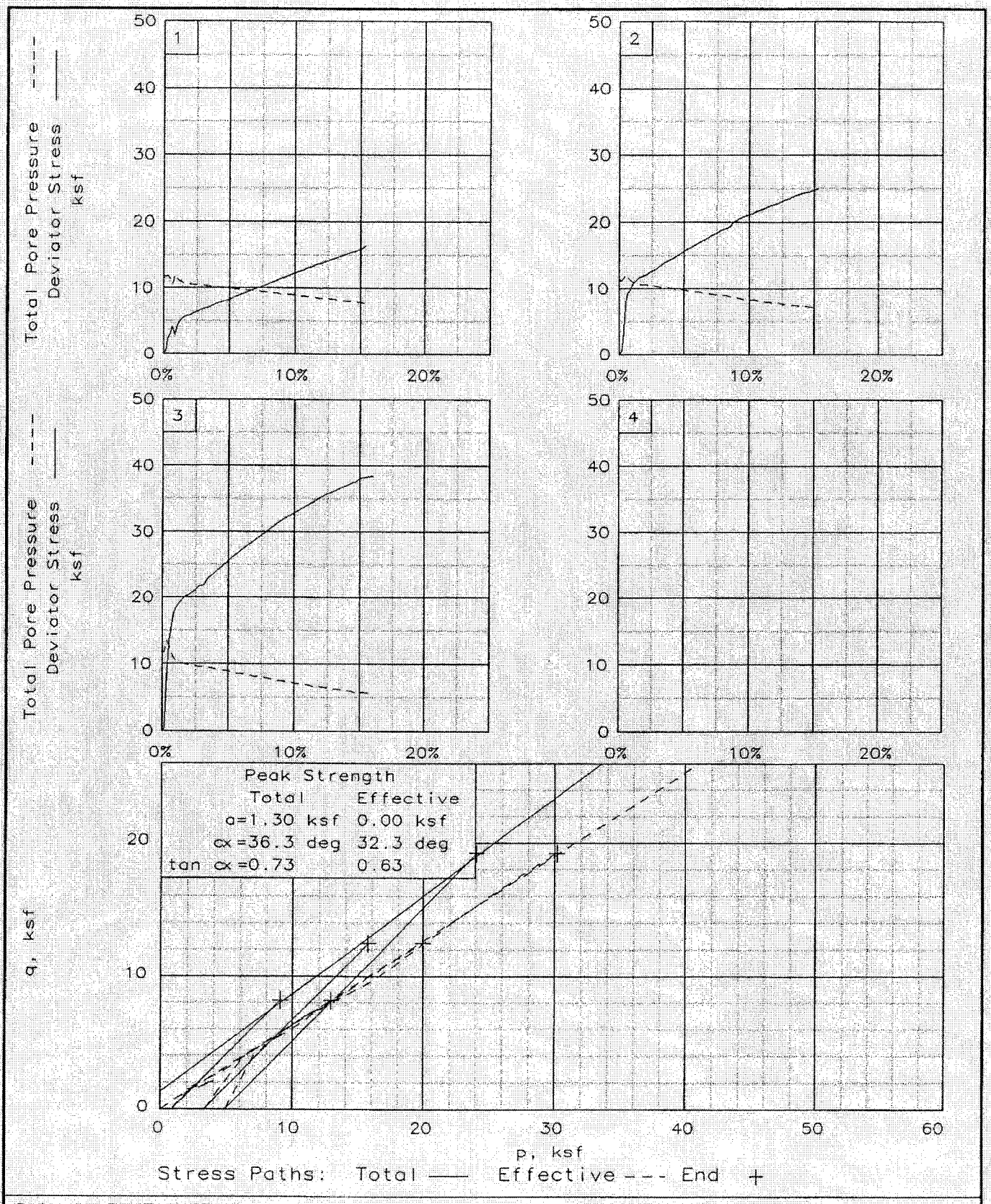
PROJECT: FGD SCRUBBER WASTE TESTING

SAMPLE LOCATION: CONFIDENTIAL POWERPLANT

PROJ. NO.: 1264-03-057 DATE: 3/18/03

TRIAXIAL SHEAR TEST REPORT

S & ME, INC.



Client: DUKE ENERGY

Project: FGD SCRUBBER WASTE TESTING

Location: CONFIDENTIAL POWERPLANT

File: BULK-90% Project No.: 1264-03-057 Fig. No.: ____

Project and Sample Data

Date: 3-25-03
Client: DUKE ENERGY
Project: FGD SCRUBBER WASTE TESTING
Sample location: CONFIDENTIAL POWERPLANT
Sample description: FGD SCRUB. SLUDGE
Remarks: BULK 95% @ 19%

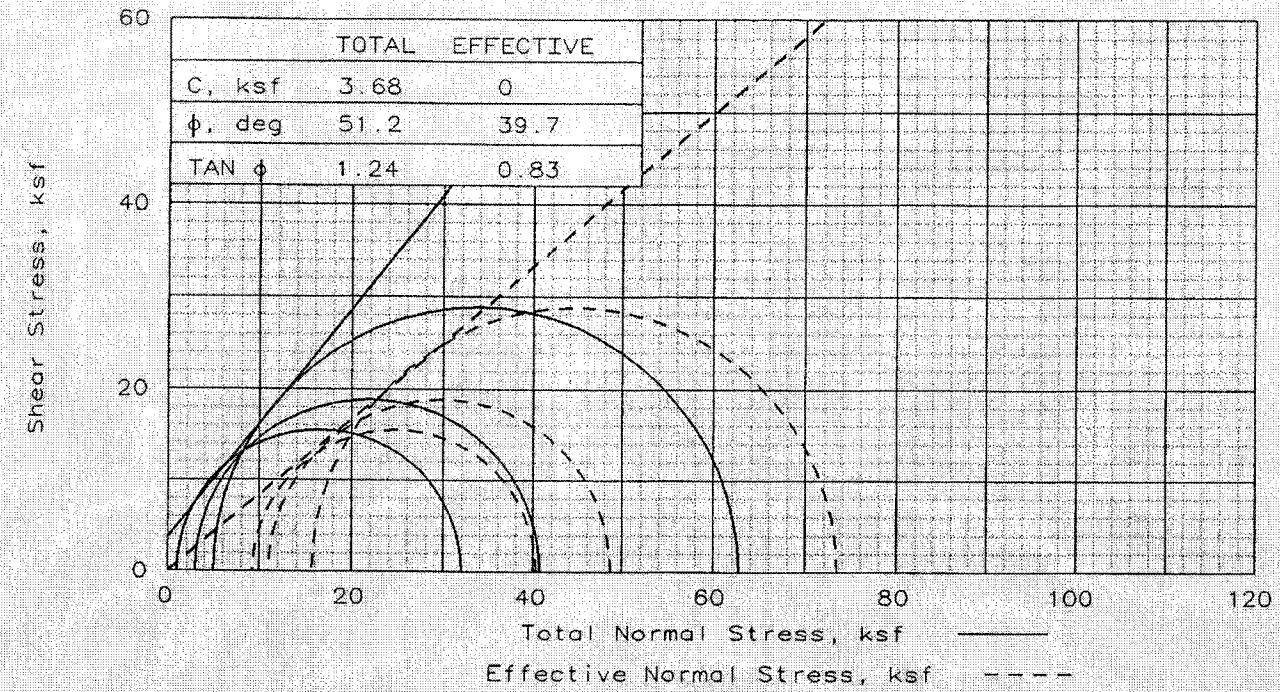
Fig no.: BULK-95% 2nd page Fig no. (if applicable):
Type of sample: REMOLDED
Specific gravity= 2.35 LL= 27 PL= 26 PI= 1
Test method: Corps of Eng. - uniform strain

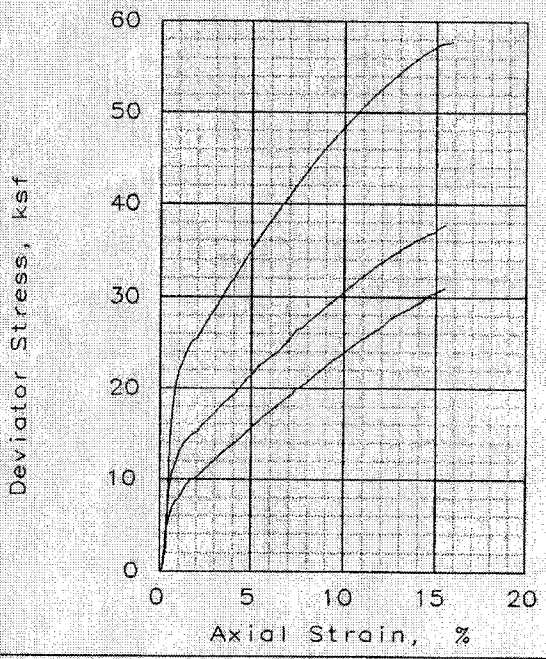
Specimen Parameters for Specimen No. 1

Specimen Parameter	Initial	Saturated	Consolidated	Final
Wt. moist soil and tare:	350.000			638.500
Wt. dry soil and tare:	306.540			532.100
Wt. of tare:	81.790			152.860
Weight, gms:	1016.6			
Diameter, in:	2.873	2.873	2.841	
Area, in²:	6.483	6.483	6.338	
Height, in:	6.000	6.000	5.933	
Net decrease in height, in:		0.000	0.067	
Net decrease in water volume, cc:				
% Moisture:	19.3	32.3	29.8	28.1
Wet density, pcf:	99.6	110.4	112.0	
Dry density, pcf:	83.4	83.4	86.3	
Void ratio:	0.7584	0.7584	0.7001	
% Saturation:	59.9	100.0	100.0	

Test Readings Data for Specimen No. 1

Deformation dial constant= 0.001 in per input unit
Primary load ring constant= 1 lbs per input unit
Secondary load ring constant= 1 lbs per input unit
Crossover reading for secondary load ring= 45 input units
Consolidation cell pressure = 87.00 psi = 12.53 ksf
Consolidation back pressure = 80.00 psi = 11.52 ksf
Consolidation effective confining stress = 1.01 ksf
Strain rate, in/min = 0.0040
FAIL. STRESS = 31.02 ksf at reading no. 51
ULT. STRESS = not selected





SAMPLE NO.:		1	2	3
INITIAL	WATER CONTENT, %	19.3	19.6	19.3
	DRY DENSITY, pcf	83.4	83.2	83.8
	SATURATION, %	59.9	60.3	60.5
	VOID RATIO	0.758	0.763	0.751
	DIAMETER, in	2.87	2.87	2.87
	HEIGHT, in	6.00	6.00	5.97
AT TEST	WATER CONTENT, %	29.8	29.6	25.3
	DRY DENSITY, pcf	86.3	86.5	92.0
	SATURATION, %	100.0	100.0	100.0
	VOID RATIO	0.700	0.695	0.595
	DIAMETER, in	2.84	2.84	2.78
	HEIGHT, in	5.93	5.92	5.79
Strain rate, in/min		0.0040	0.0040	0.0040
EFF CELL PRESSURE, ksf		1.0	3.0	5.0
FAIL. STRESS, ksf		31.0	37.7	57.7
TOTAL PORE PR., ksf		3.2	3.5	0.8
STRAIN, %		15.5	15.5	15.9
ULT. STRESS, ksf				
TOTAL PORE PR., ksf				
STRAIN, %				
σ_1 FAILURE, ksf		40.3	48.8	73.5
σ_3 FAILURE, ksf		9.3	11.0	15.7

TYPE OF TEST:
CU with Pore Pressures
SAMPLE TYPE: REMOLDED
DESCRIPTION: FGD SCRUB. SLUDGE

LL= 27 PL= 26 PI= 1
SPECIFIC GRAVITY= 2.35
REMARKS: BULK 95% @ 19%

Fig. No.: BULK 95%

CLIENT: DUKE ENERGY

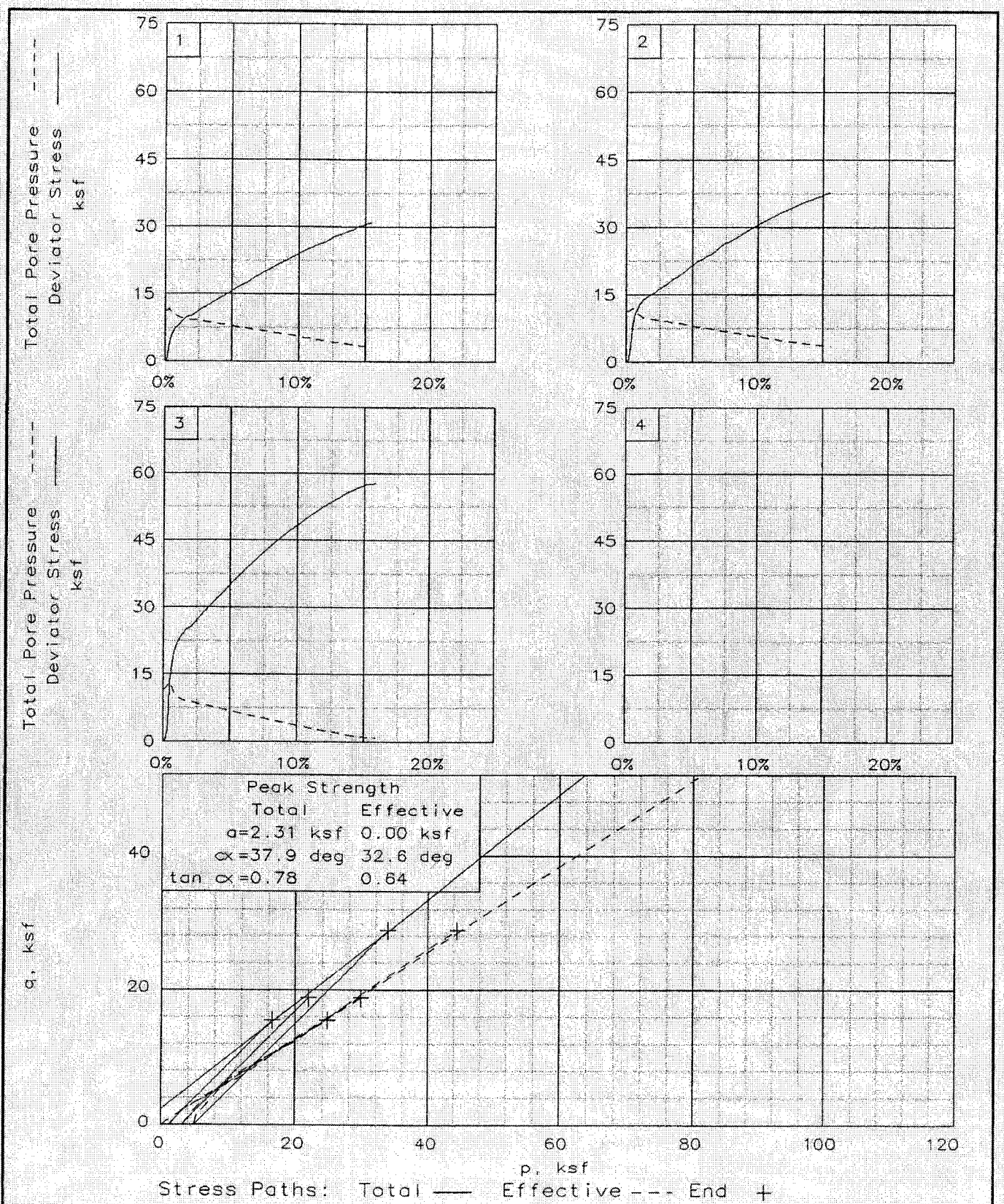
PROJECT: FGD SCRUBBER WASTE TESTING

SAMPLE LOCATION: CONFIDENTIAL POWERPLANT

PROJ. NO.: 1264-03-057 DATE: 3-25-03

TRIAXIAL SHEAR TEST REPORT

S & ME, INC.



Client: DUKE ENERGY

Project: FGD SCRUBBER WASTE TESTING

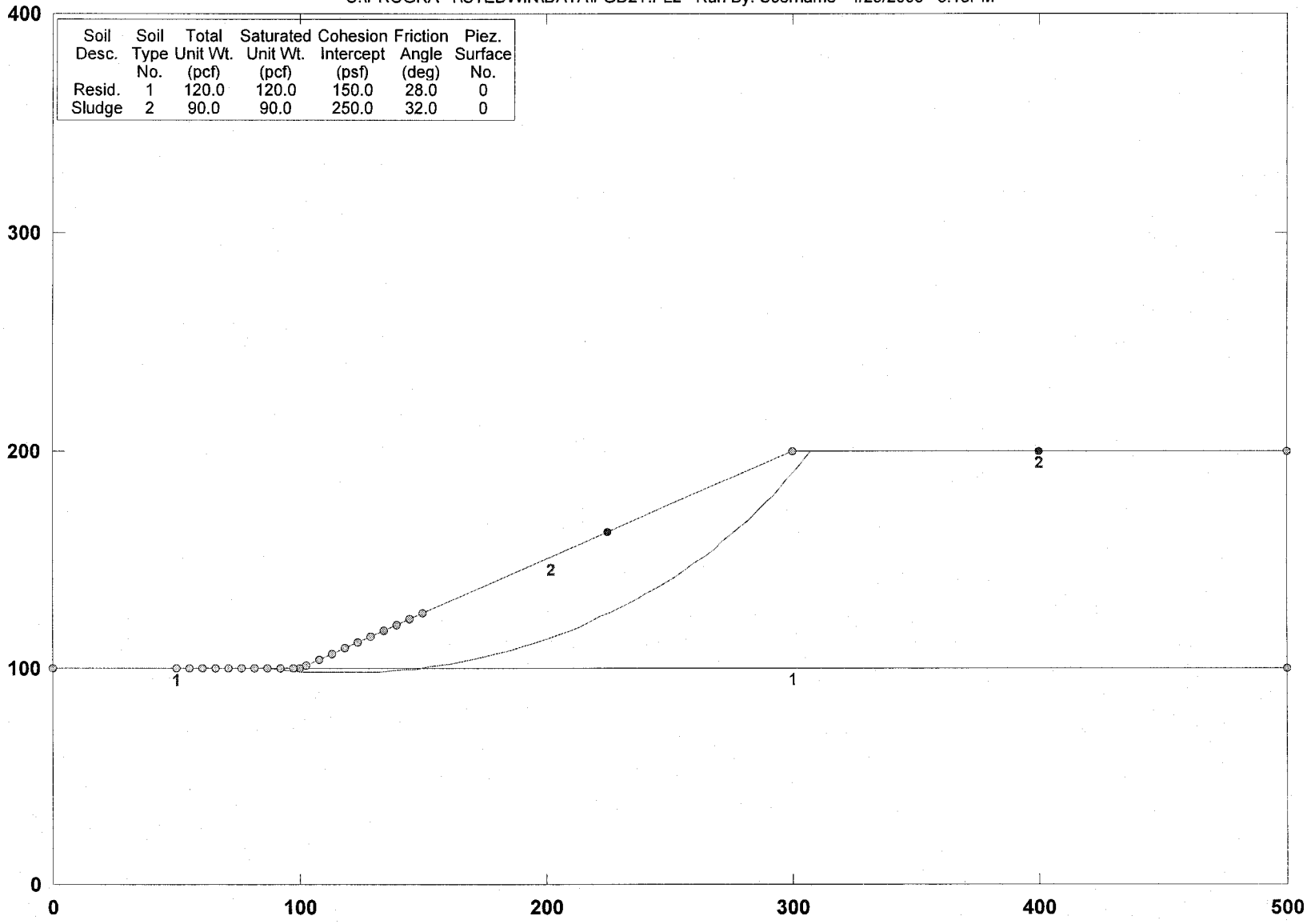
Location: CONFIDENTIAL POWERPLANT

File: SCRUBBER Project No.: 1264-03-057 Fig. No.: ____

Duke FGD Scrubber Sludge Testing 100'/ 2:1 Slope (Total Stresses w/o EQ)

C:\PROGRA~1\STEDWIN\DATA\FGD2T.PL2 Run By: Username 4/29/2003 3:18PM

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Piez. Surface No.
Resid.	1	120.0	120.0	150.0	28.0	0
Sludge	2	90.0	90.0	250.0	32.0	0



STABL6H FSmin=1.72

Safety Factors Are Calculated By The Modified Bishop Method

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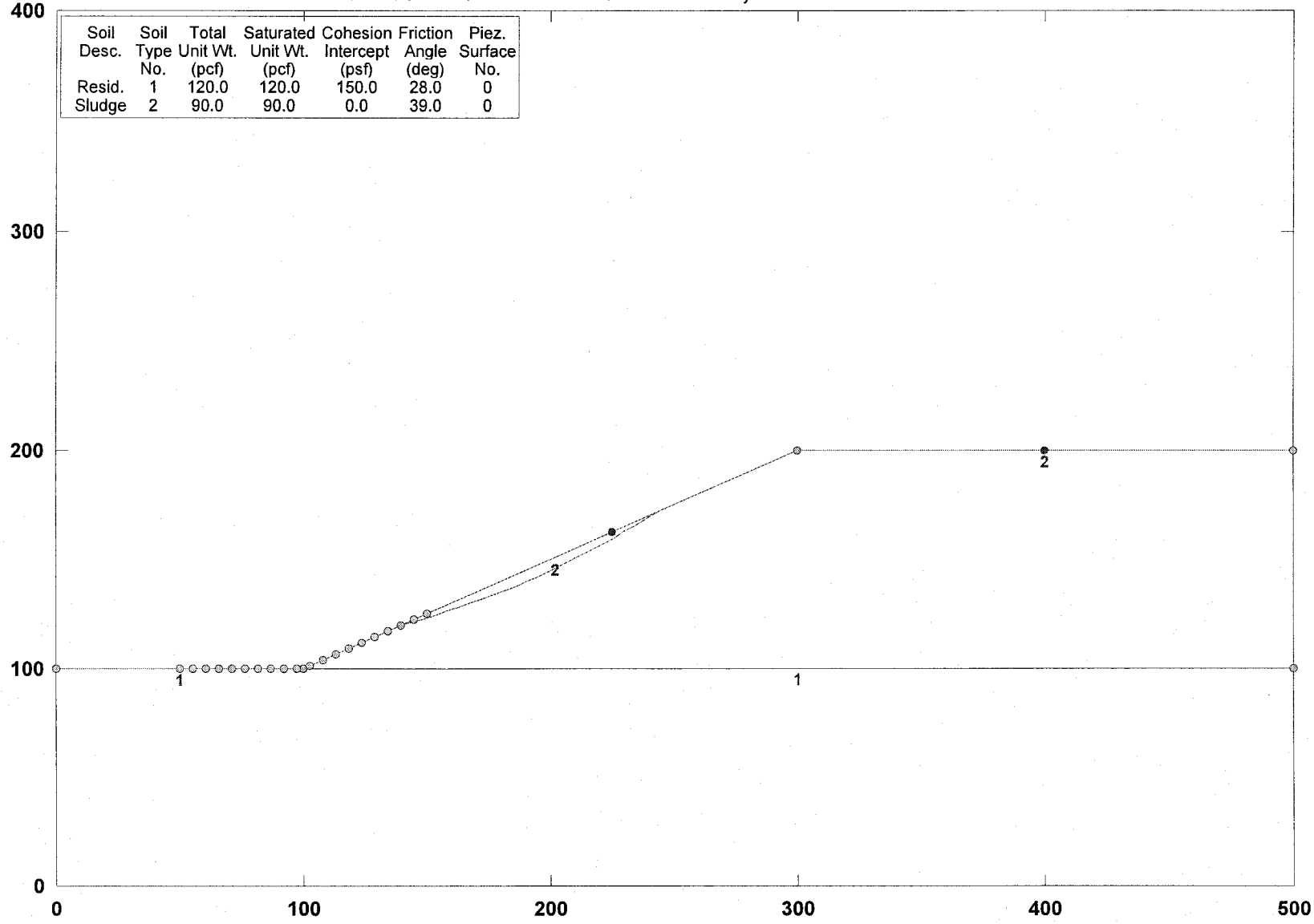
Duke FGD Scrubber Sludge Testing 100'/ 2:1 Slope (Eff. Stresses w/o EQ)

C:\PROGRA~1\STEDWIN\DATA\FGD2E.PL2 Run By: Username 4/10/2003 10:50AM

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Piez. Surface No.
Resid.	1	120.0	120.0	150.0	28.0	0
Sludge	2	90.0	90.0	0.0	39.0	0



STABL6H FSmin=1.64

Safety Factors Are Calculated By The Modified Bishop Method

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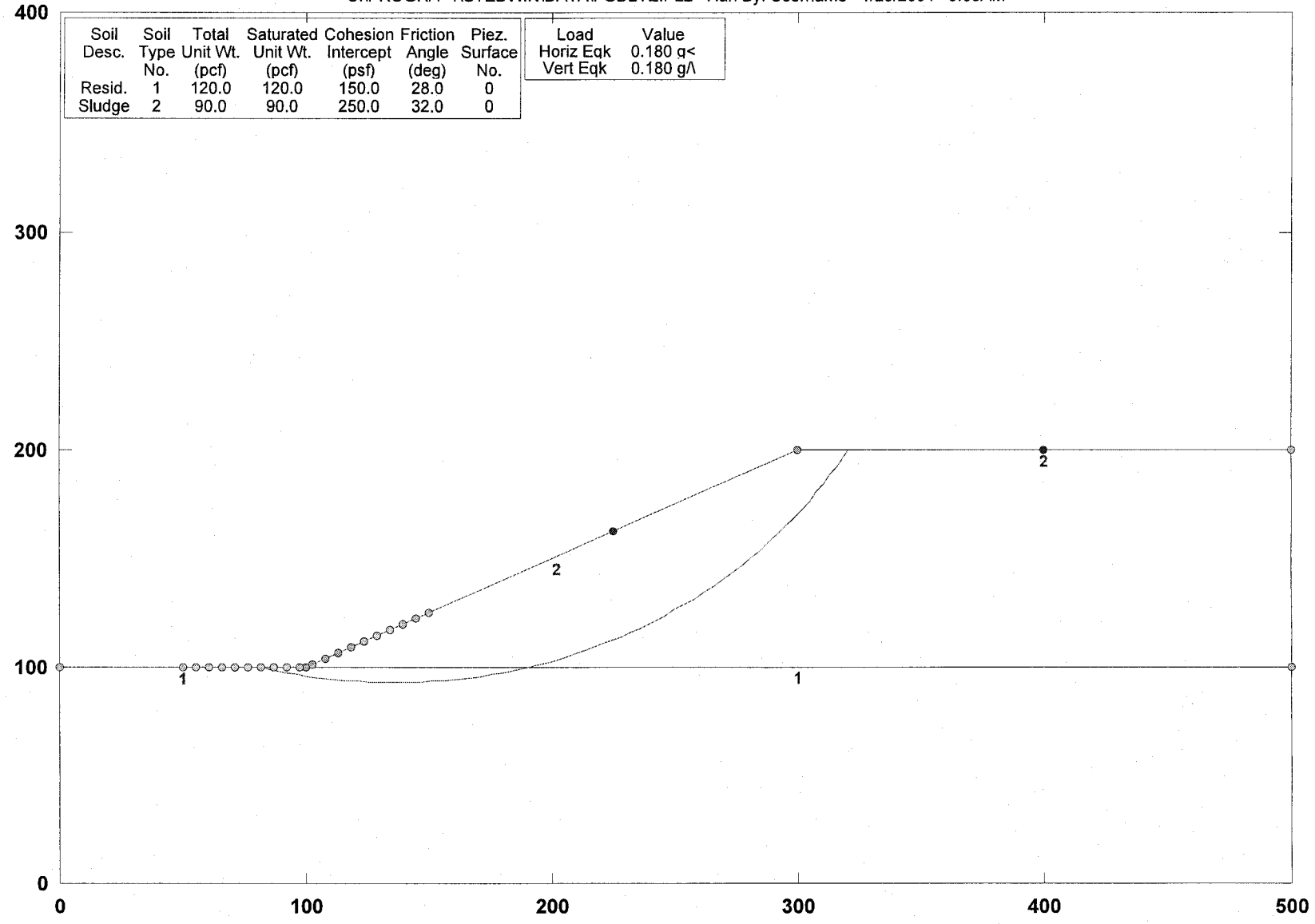
Duke FGD Scrubber Sludge Testing 100'/ 2:1 Slope (Total Stresses w/ EQ)

C:\PROGRA~1\STEDWIN\DATA\FGD2TE.PL2 Run By: Username 1/29/2004 9:06AM

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Piez. Surface No.
Resid.	1	120.0	120.0	150.0	28.0	0
Sludge	2	90.0	90.0	250.0	32.0	0

Load	Value
Horiz Eqk	0.180 g<
Vert Eqk	0.180 g/\



STABL6H FSmin=1.14

Safety Factors Are Calculated By The Modified Bishop Method

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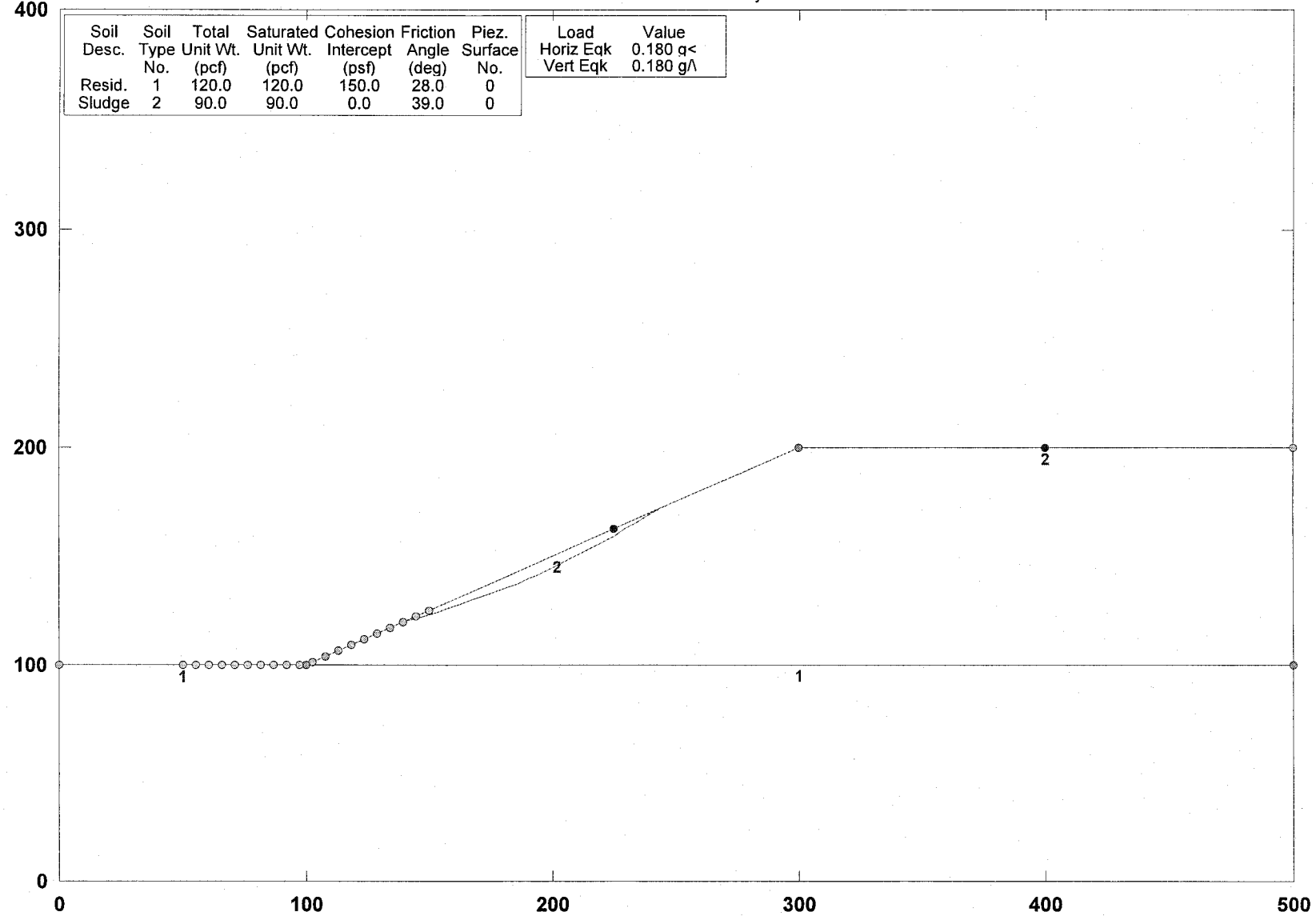
Duke FGD Scrubber Sludge Testing 100'/ 2:1 Slope (Eff Stresses w/ EQ)

C:\PROGRA~1\STEDWIN\DATA\FGD2EE.PL2 Run By: Username 1/29/2004 9:07AM

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Piez. Surface No.
Resid.	1	120.0	120.0	150.0	28.0	0
Sludge	2	90.0	90.0	0.0	39.0	0

Load	Value
Horiz Eqk	0.180 g<
Vert Eqk	0.180 g/\



STABL6H FSmin=1.02

Safety Factors Are Calculated By The Modified Bishop Method

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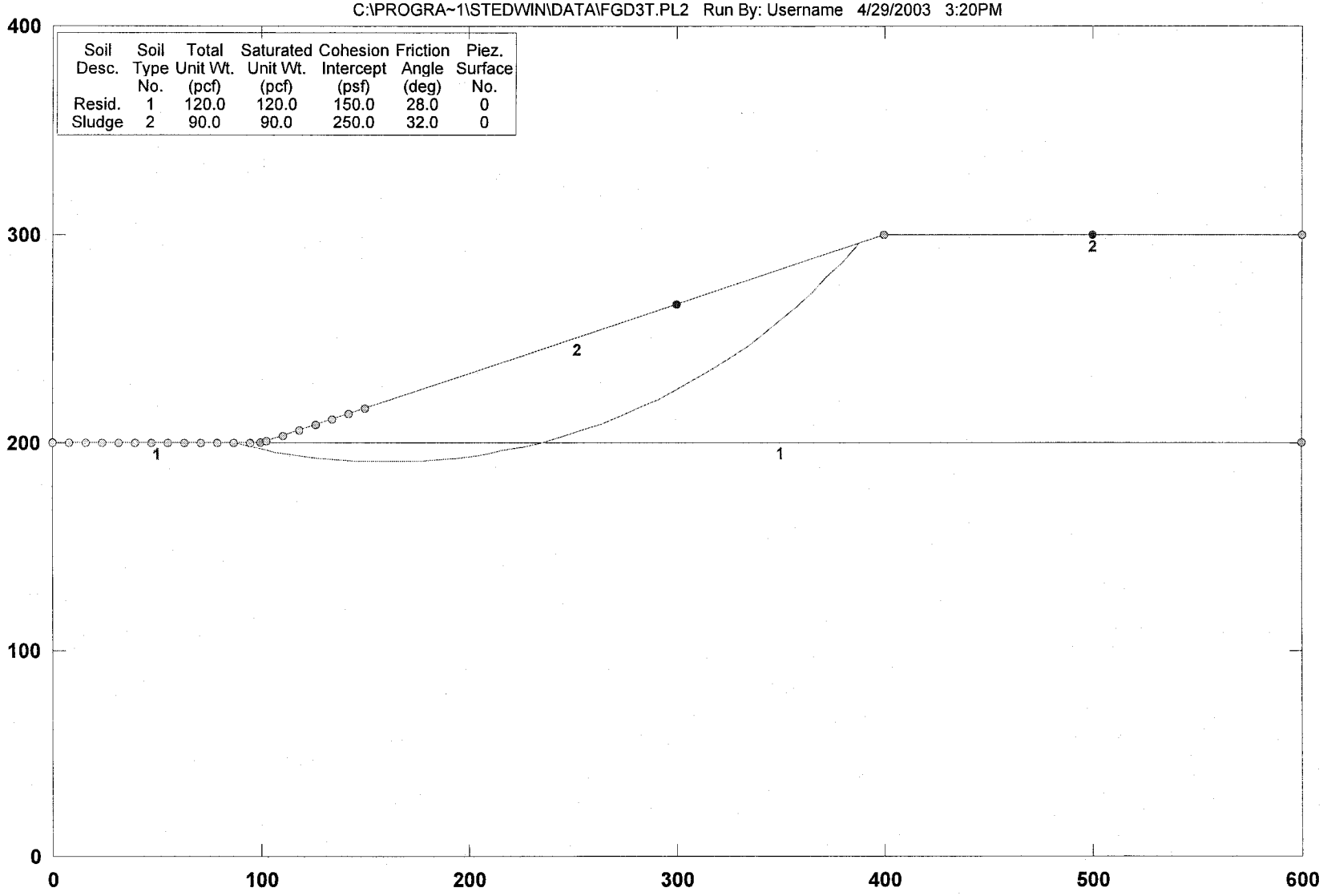
Duke FGD Scrubber Sludge Testing 100'/ 3:1 Slope (Total Stresses w/o EQ)

C:\PROGRA~1\STEDWIN\DATA\FGD3T.PL2 Run By: Username 4/29/2003 3:20PM



STABL6H FSmin=2.35

Safety Factors Are Calculated By The Modified Bishop Method

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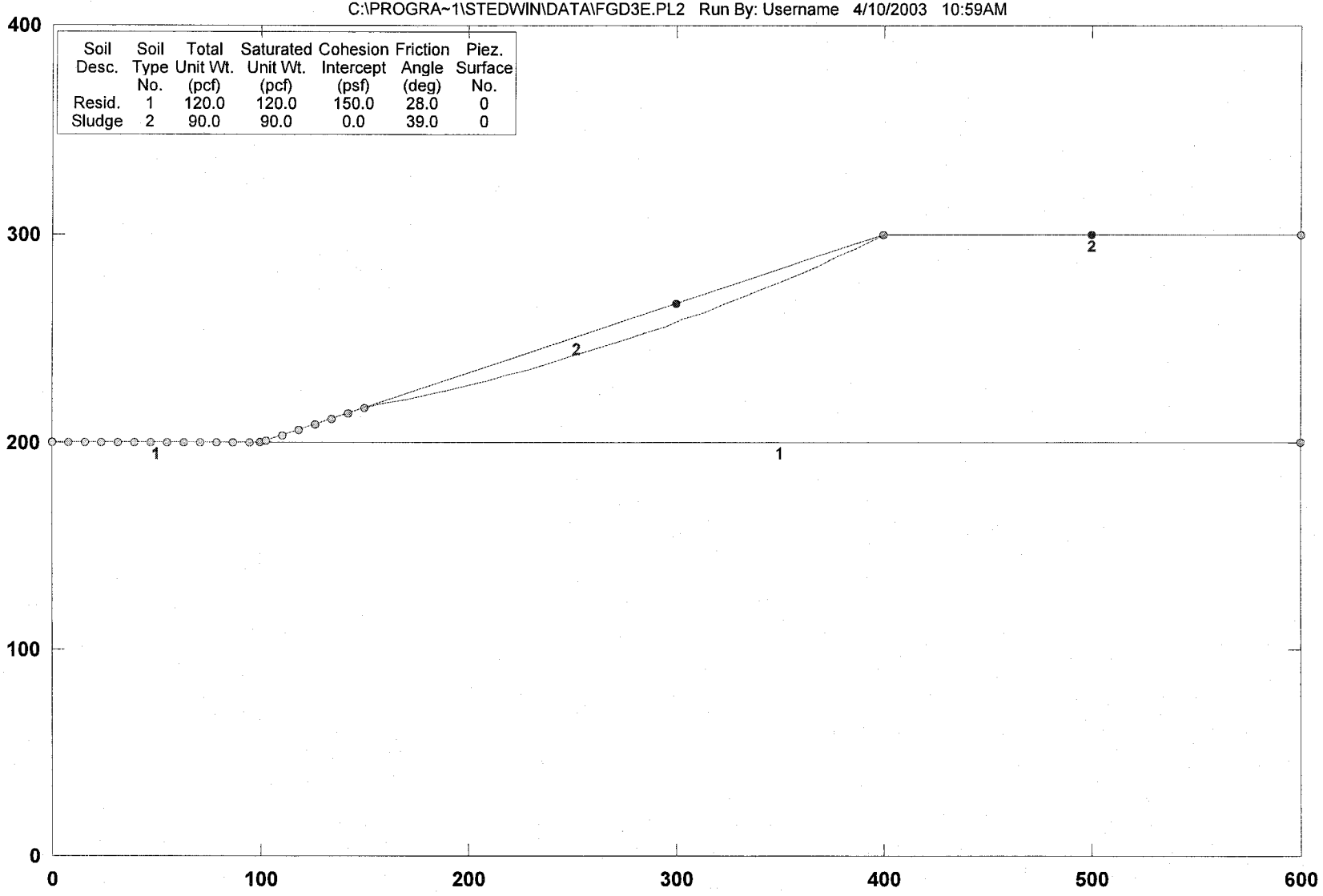
Duke FGD Scrubber Sludge Testing 100'/ 3:1 Slope (Eff Stresses w/o EQ)

C:\PROGRA~1\STEDWIN\DATA\FGD3E.PL2 Run By: Username 4/10/2003 10:59AM

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Piez. Surface No.
Resid.	1	120.0	120.0	150.0	28.0	0
Sludge	2	90.0	90.0	0.0	39.0	0



STABL6H FSmin=2.45

Safety Factors Are Calculated By The Modified Bishop Method

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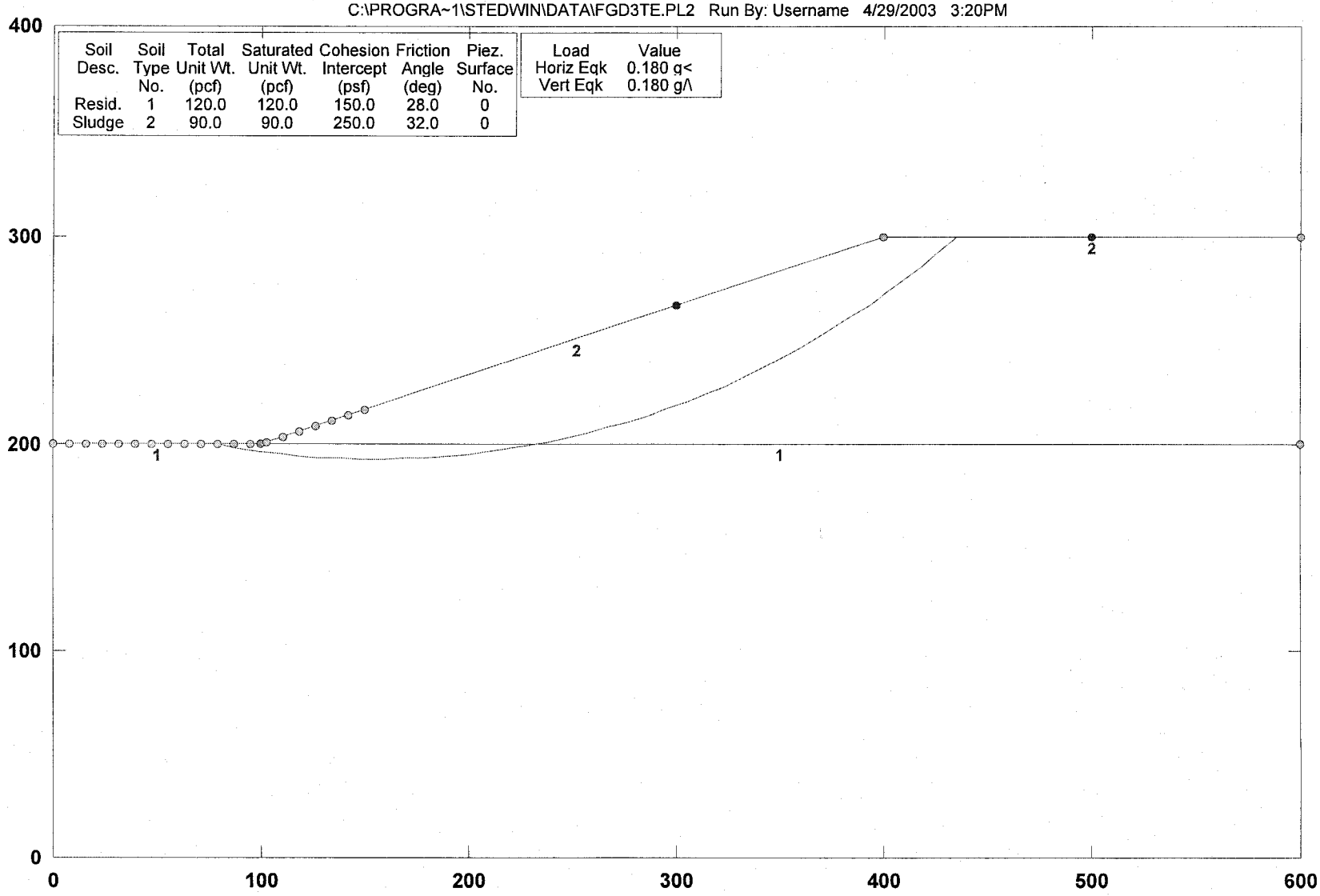
Duke FGD Scrubber Sludge Testing 100'/ 3:1 Slope (Total Stresses w/ EQ)

C:\PROGRA~1\STEDWIN\DATA\FGD3TE.PL2 Run By: Username 4/29/2003 3:20PM

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Piez. Surface No.
Resid.	1	120.0	120.0	150.0	28.0	0
Sludge	2	90.0	90.0	250.0	32.0	0

Load	Value
Horiz Eqk	0.180 g<
Vert Eqk	0.180 g/\



STABL6H FSmin=1.34

Safety Factors Are Calculated By The Modified Bishop Method

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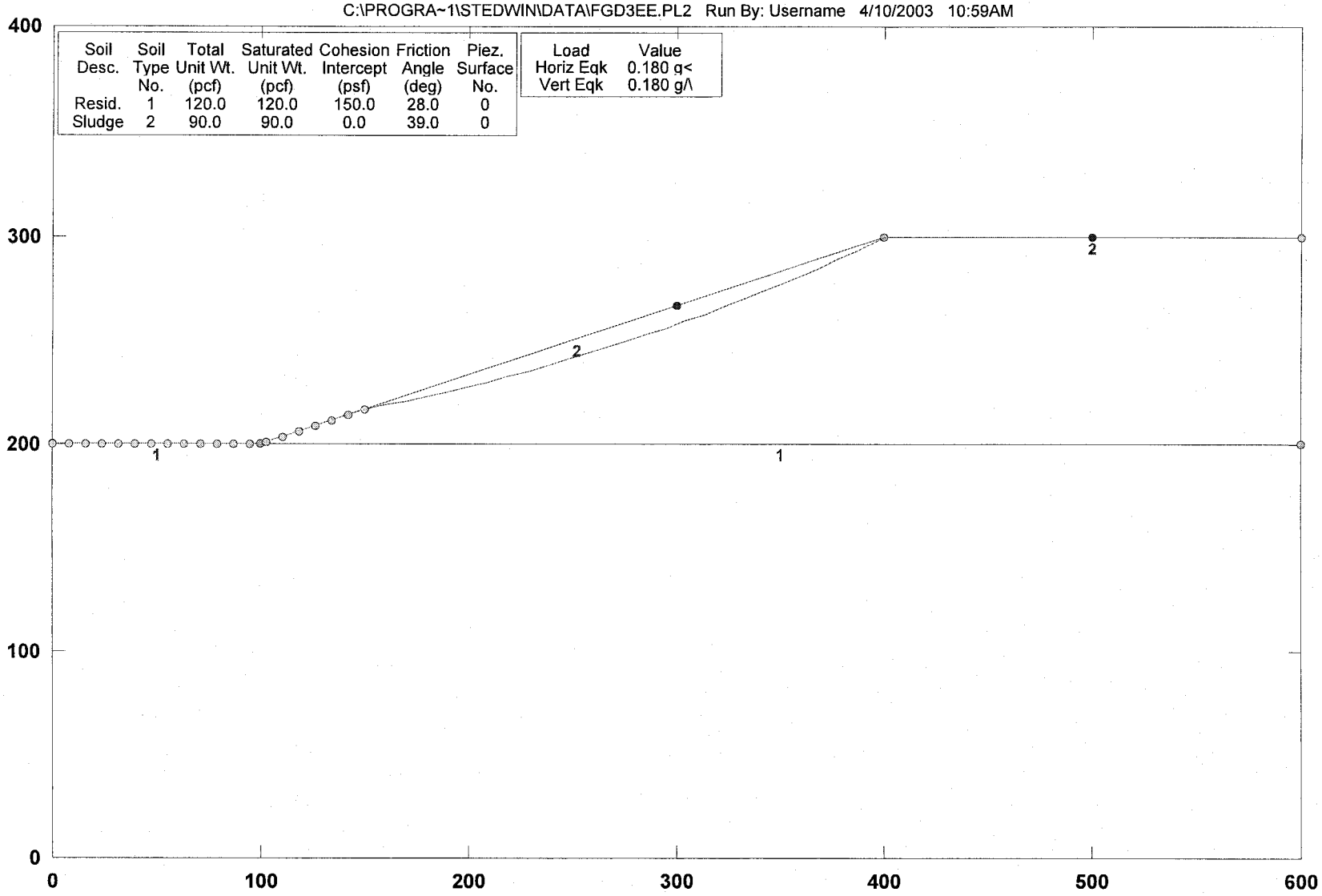
Duke FGD Scrubber Sludge Testing 100'/ 3:1 Slope (Eff Stresses w/ EQ)

C:\PROGRA~1\STEDWIN\DATA\FGD3EE.PL2 Run By: Username 4/10/2003 10:59AM

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Piez. Surface No.
Resid.	1	120.0	120.0	150.0	28.0	0
Sludge	2	90.0	90.0	0.0	39.0	0

Load	Value
Horiz Eqk	0.180 g<
Vert Eqk	0.180 g/\



STABL6H FSmin=1.37

Safety Factors Are Calculated By The Modified Bishop Method

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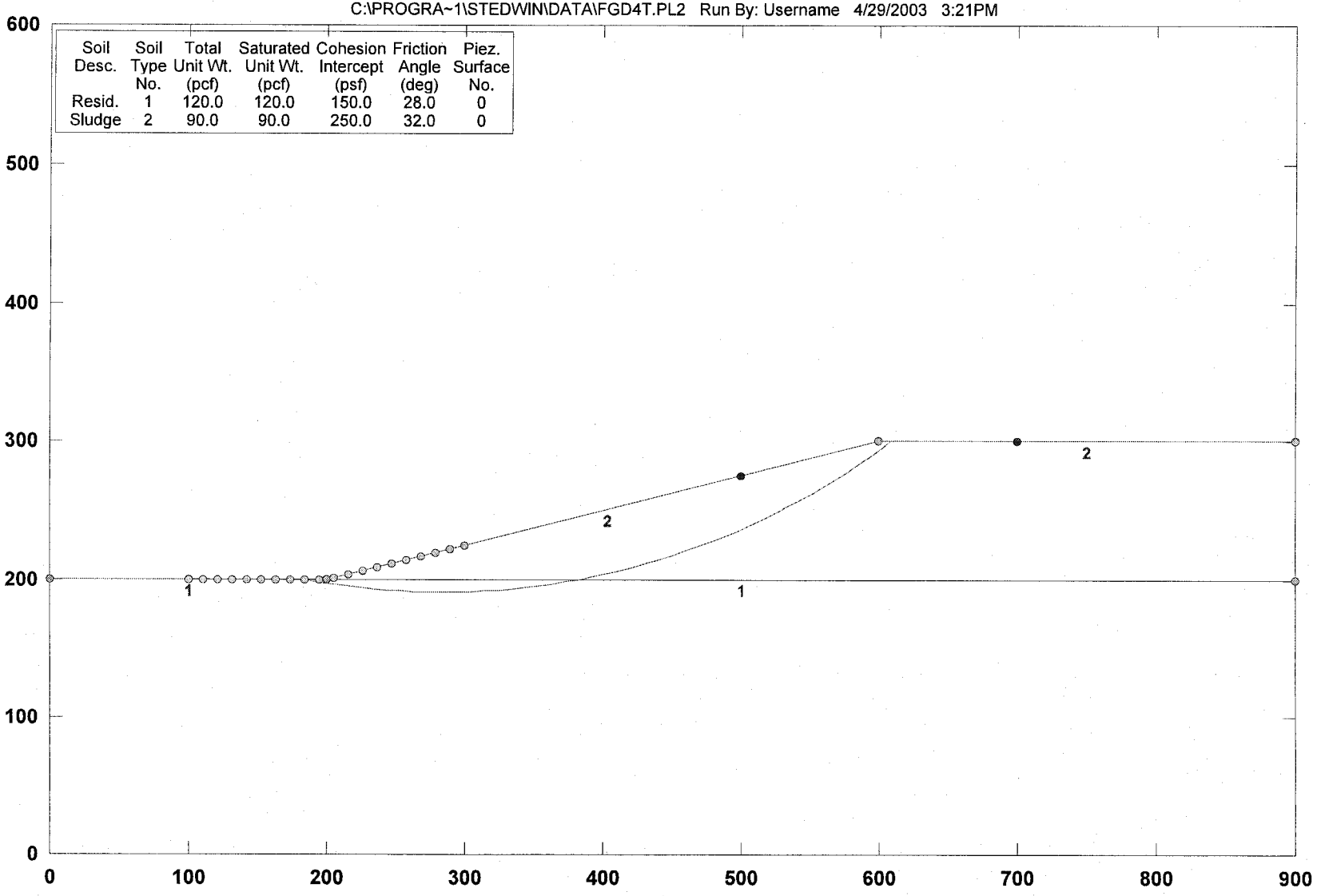
Duke FGD Scrubber Sludge Testing 100'/ 4:1 Slope (Total Stresses w/o EQ)

C:\PROGRA~1\STEDWIN\DATA\FGD4T.PL2 Run By: Username 4/29/2003 3:21PM

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Piez. Surface No.
Resid.	1	120.0	120.0	150.0	28.0	0
Sludge	2	90.0	90.0	250.0	32.0	0



STABL6H FSmin=2.94

Safety Factors Are Calculated By The Modified Bishop Method

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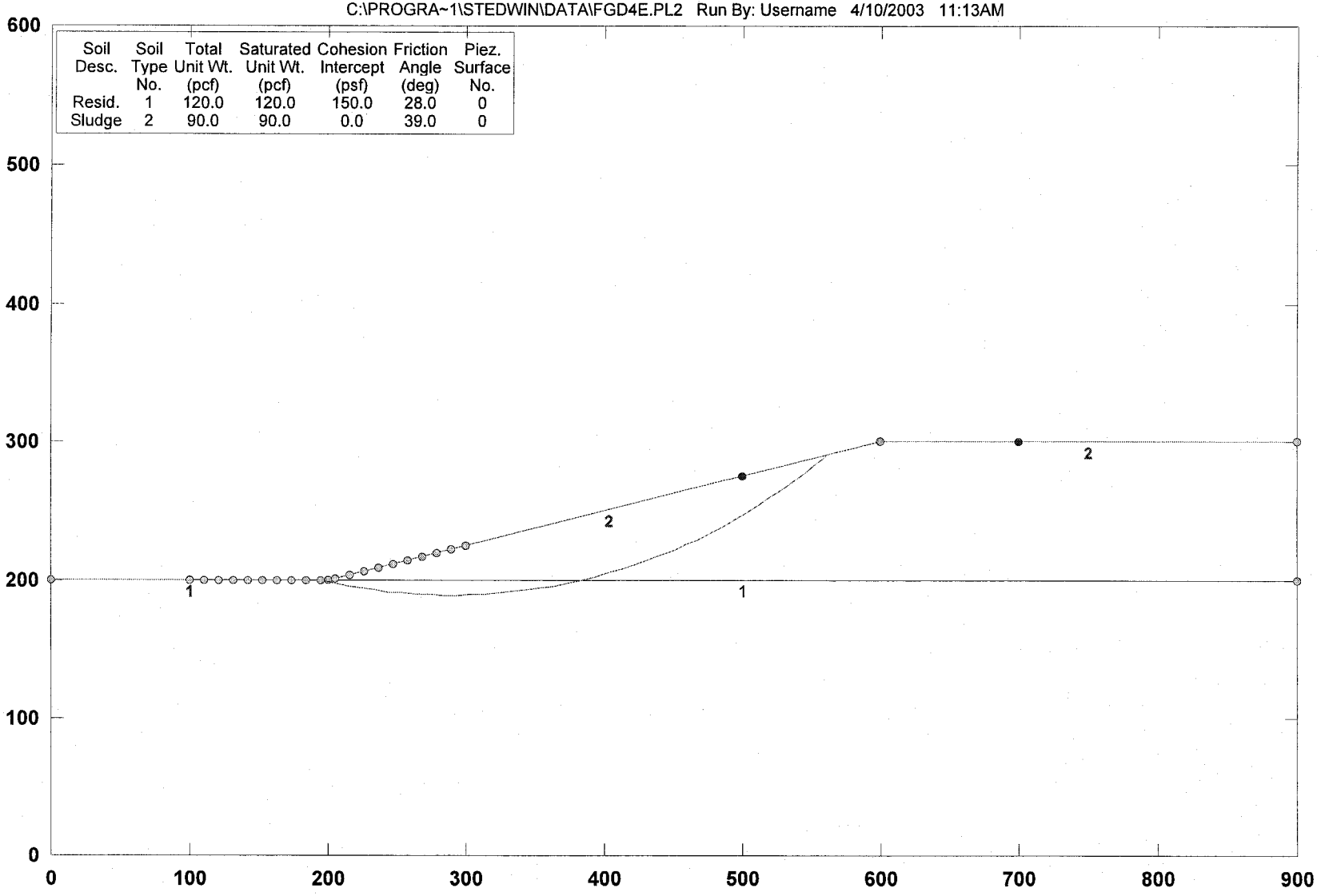
Duke FGD Scrubber Sludge Testing 100'/ 4:1 Slope (Eff Stresses w/o EQ)

C:\PROGRA~1\STEDWIN\DATA\FGD4E.PL2 Run By: Username 4/10/2003 11:13AM

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Piez. Surface No.
Resid.	1	120.0	120.0	150.0	28.0	0
Sludge	2	90.0	90.0	0.0	39.0	0



STABL6H FSmin=3.16

Safety Factors Are Calculated By The Modified Bishop Method

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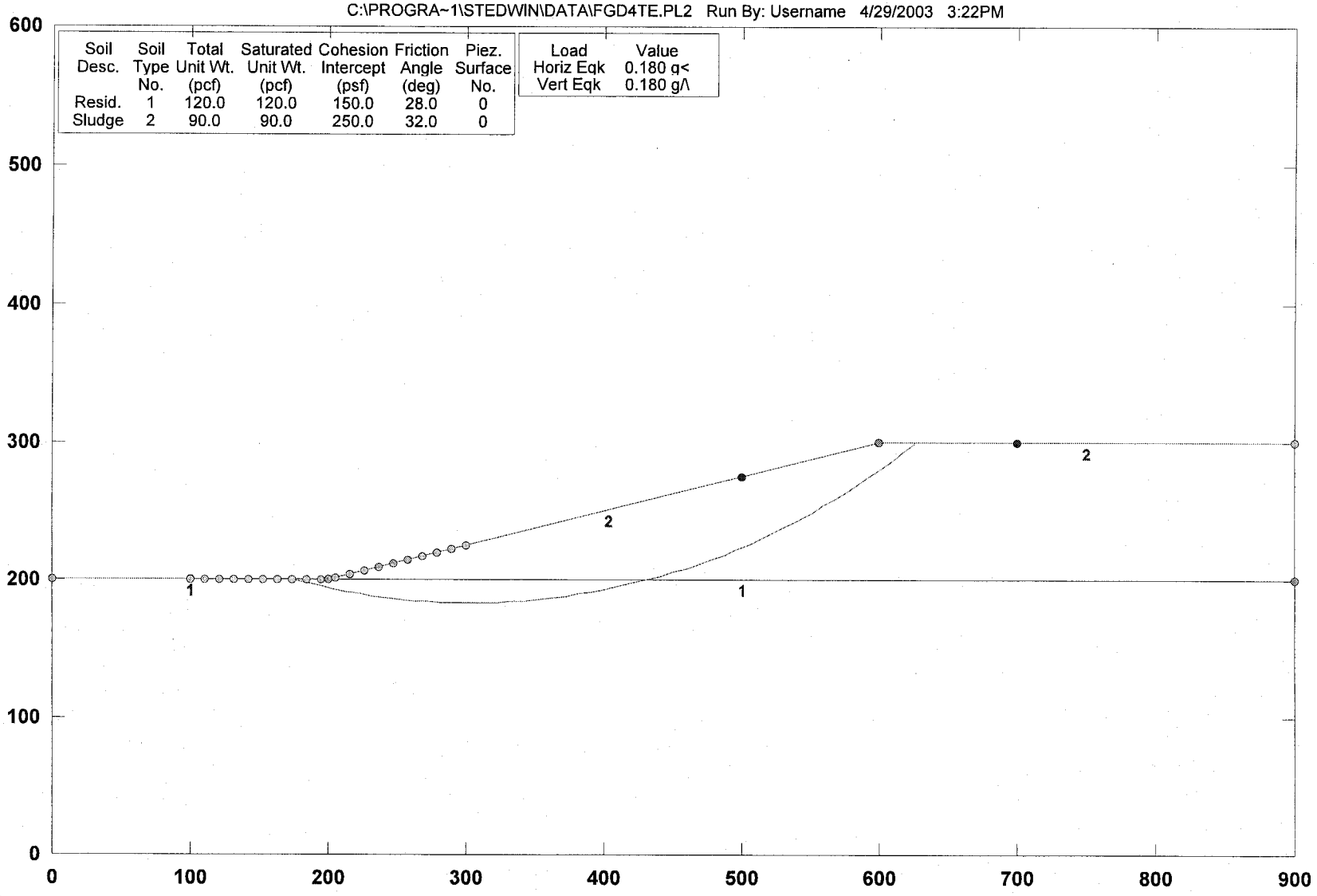
Duke FGD Scrubber Sludge Testing 100'/ 4:1 Slope (Total Stresses w/ EQ)

C:\PROGRA~1\STEDWIN\DATA\FGD4TE.PL2 Run By: Username 4/29/2003 3:22PM

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Piez. Surface No.
Resid.	1	120.0	120.0	150.0	28.0	0
Sludge	2	90.0	90.0	250.0	32.0	0

Load	Value
Horiz Eqk	0.180 g<
Vert Eqk	0.180 g/\



STABL6H FSmin=1.49

Safety Factors Are Calculated By The Modified Bishop Method

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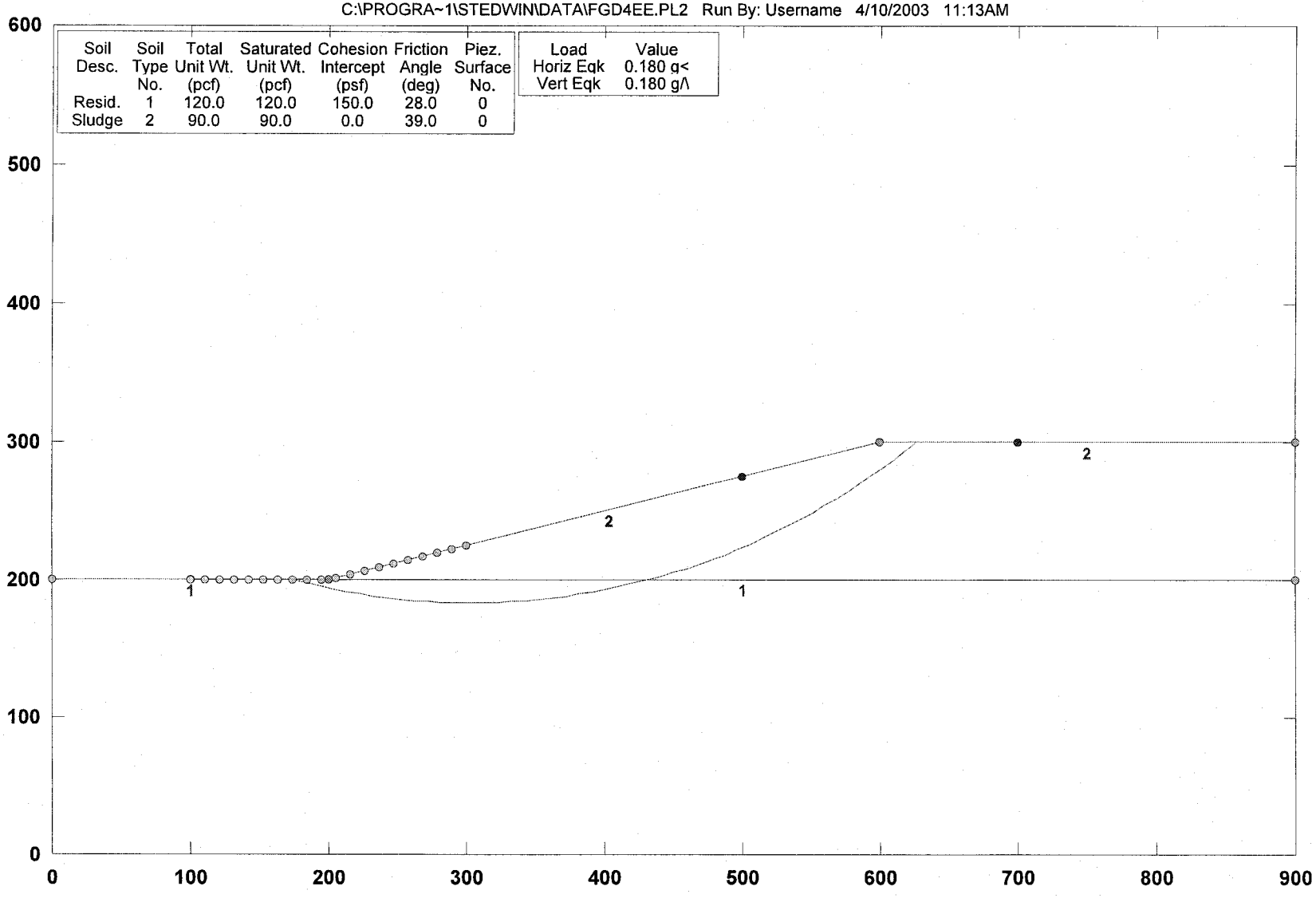
Duke FGD Scrubber Sludge Testing 100'/ 4:1 Slope (Eff Stresses w/ EQ)

C:\PROGRA~1\STEDWIN\DATA\FGD4EE.PL2 Run By: Username 4/10/2003 11:13AM

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Piez. Surface No.
Resid.	1	120.0	120.0	150.0	28.0	0
Sludge	2	90.0	90.0	0.0	39.0	0

Load	Value
Horiz Eqk	0.180 g<
Vert Eqk	0.180 g/\



STABL6H FSmin=1.56

Safety Factors Are Calculated By The Modified Bishop Method

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GROUND MOTION VALUES

SITE: **CLIFFSIDE**
LATITUDE: 35.21367
LONGITUDE: 81.76253

PROBABILISTIC GROUND MOTION VALUES, %g

	10% PE in 50 years	5% PE in 50 years	2% PE in 50 years
PGA	6.475299	10.02965	17.87137
0.2 sec SA	14.83405	22.97971	38.22041
0.3 sec SA	12.08536	18.49899	30.39903
1.0 sec SA	5.282979	8.438171	13.71333

SITE: **MARSHALL**
LATITUDE: 35.60267
LONGITUDE: 80.97572

PROBABILISTIC GROUND MOTION VALUES, %g

	10% PE in 50 years	5% PE in 50 years	2% PE in 50 years
PGA	5.351426	8.719715	14.61557
0.2 sec SA	12.54198	19.22603	33.45272
0.3 sec SA	10.93671	17.00028	27.52791
1.0 sec SA	4.851488	7.907398	13.09891

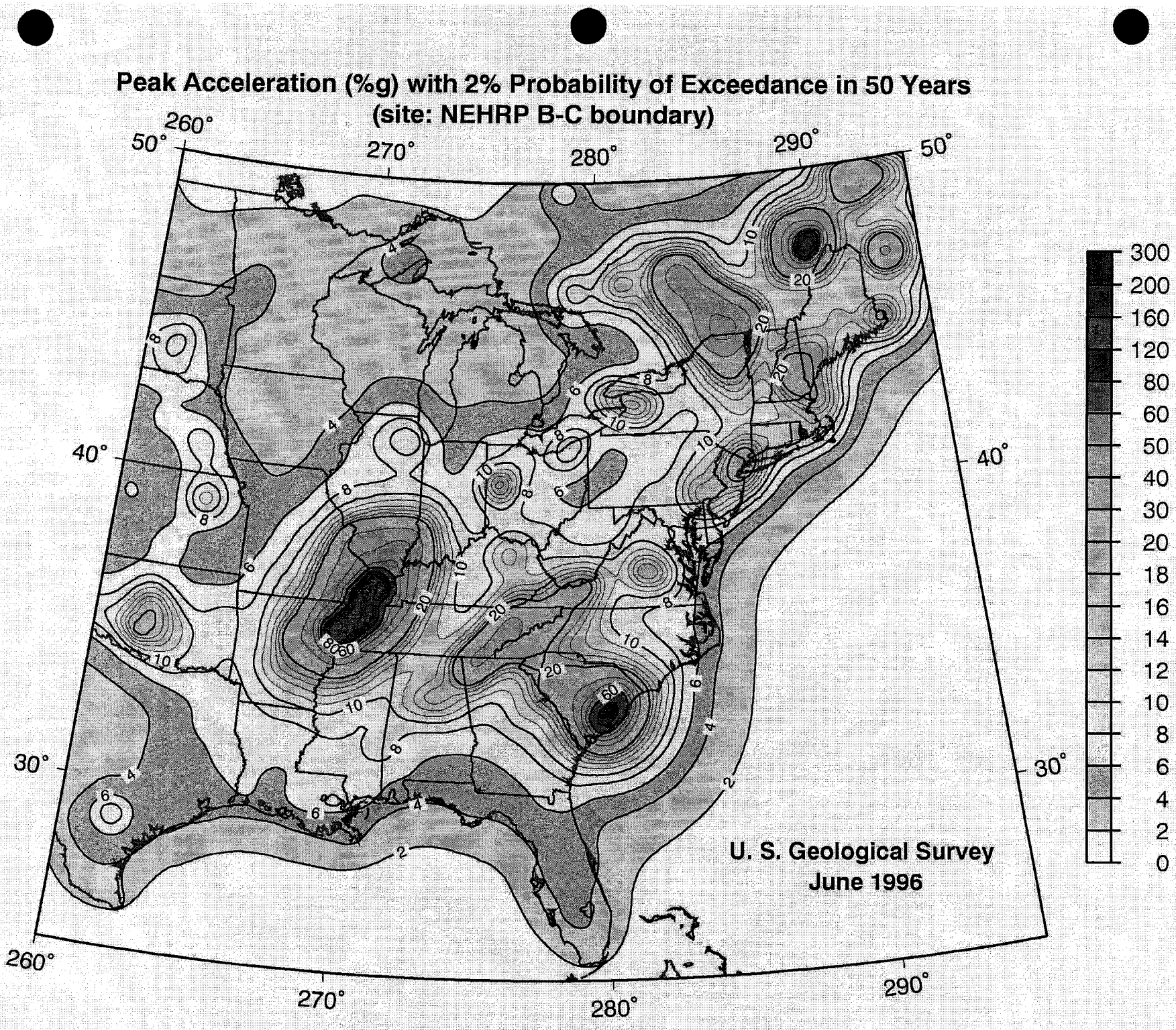
SITE: **BELEWS CREEK**
LATITUDE: 36.27578
LONGITUDE: 80.05647

PROBABILISTIC GROUND MOTION VALUES, %g

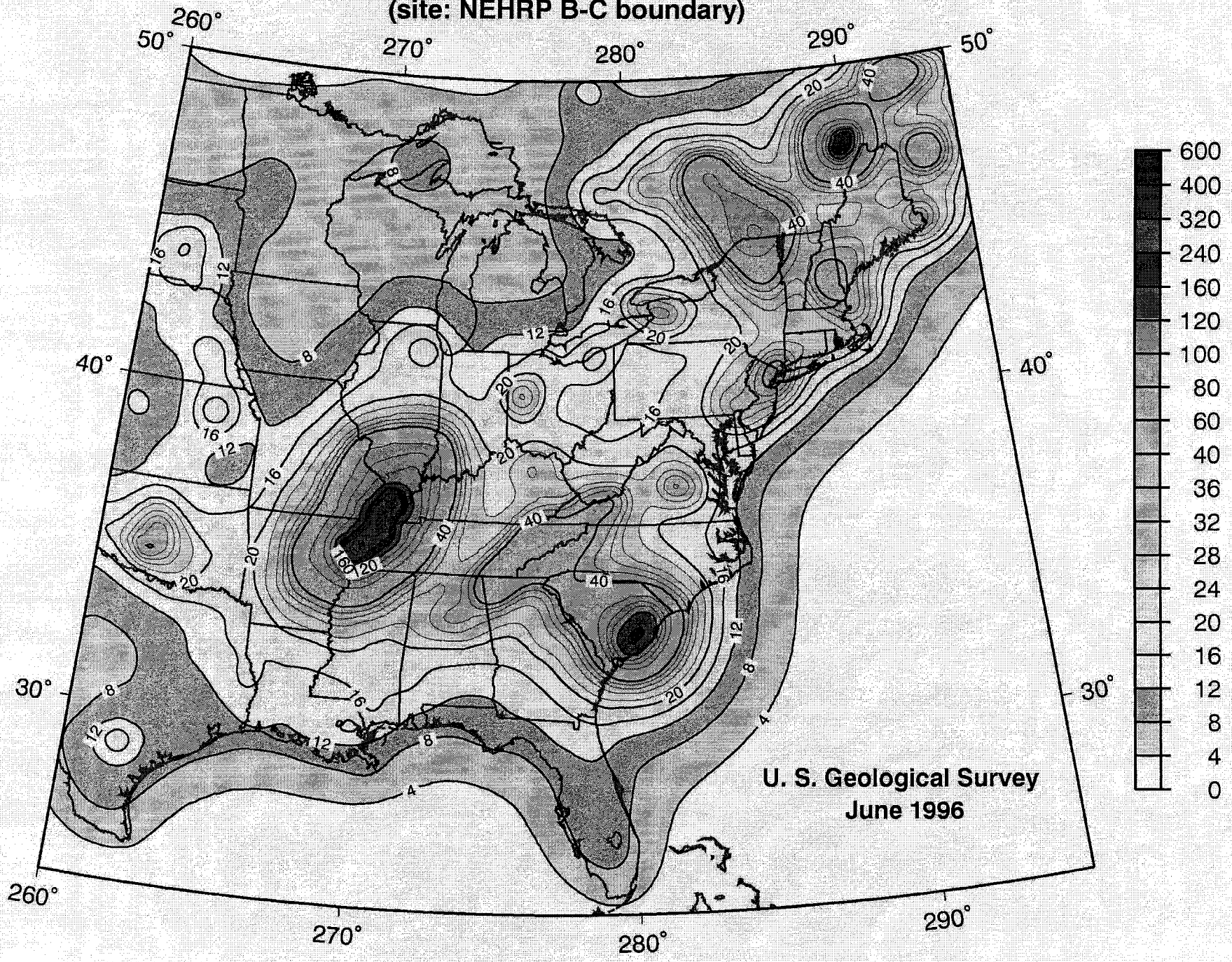
	10% PE in 50 years	5% PE in 50 years	2% PE in 50 years
PGA	4.357762	6.874004	12.07349
0.2 sec SA	10.41264	16.23185	26.81065
0.3 sec SA	8.516776	13.45018	23.06256
1.0 sec SA	4.067841	6.413146	11.15028

Notes:
1. PE – Probability of Exceedance
2. PGA – Peak Ground Acceleration
3. SA – Spectral Acceleration
4.

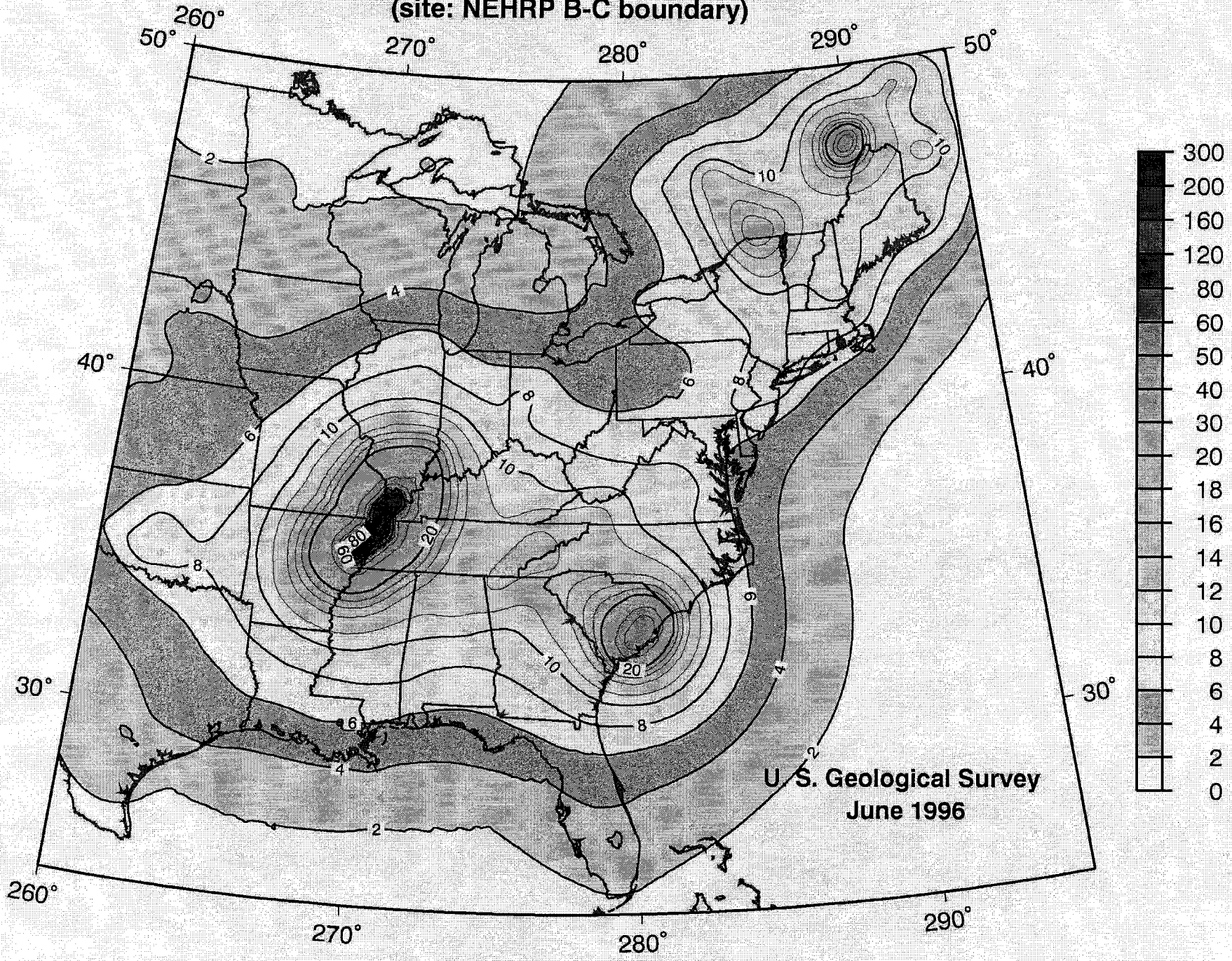
Reference: USGS Seismic Hazard Maps, 1996


Peak Acceleration (%g) with 2% Probability of Exceedance in 50 Years
(site: NEHRP B-C boundary)
260°
270°
280°
290°
50°
40°
30°
300
200
160
120
80
60
50
40
30
20
18
16
14
12
10
8
6
4
2
0
U. S. Geological Survey
June 1996

0.2 sec Spectral Acceleration (%g) with 2% Probability of Exceedance in 50 Years
(site: NEHRP B-C boundary)



1.0 sec Spectral Acceleration (%g) with 2% Probability of Exceedance in 50 Years
(site: NEHRP B-C boundary)



Belews Creek FGD Residue Landfill
HELP Model Description and Results

Attachment 2

Particle Size Analysis of Soils

Particle Size Analysis for "Cut" Material

Particle Size Analysis of Soils

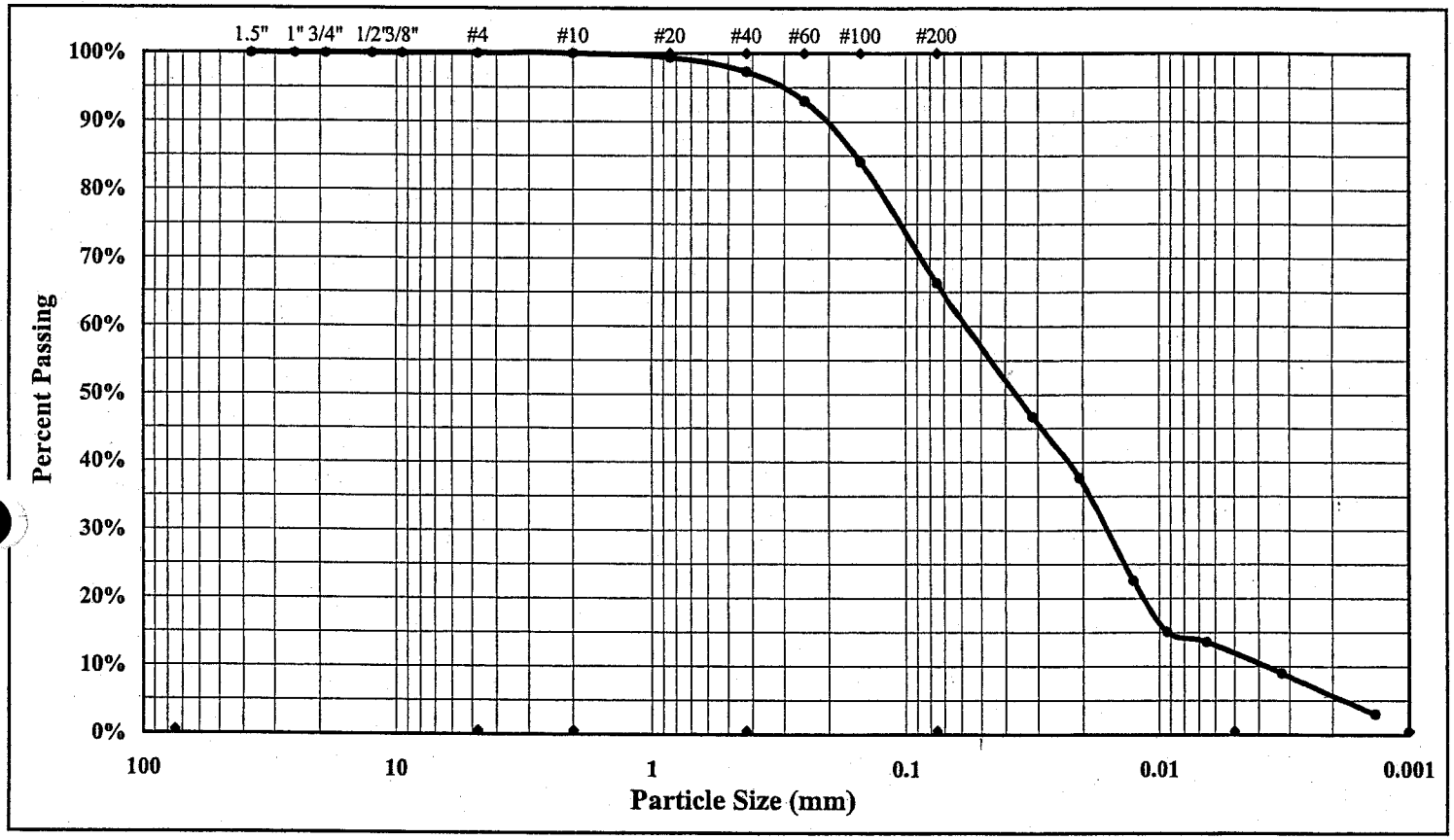
ASTM D 422

S&ME

S&ME Project #: **1264-02-576**
Project Name: **Duke Power-Belews Creek Landfill**
Client Name: Duke Energy Corp.
Client Address: 3195 Pine Hall Road, Belews Creek, NC 27009

Report Date: January 22, 2003
Test Date(s): 1/15/03-1/22/03

Boring #:	BC-12	Sample #:	SS-6	Sample Date:	NA
Location:		Offset:		Depth (ft):	18.5 - 20.0
Sample Description:	Tan Fine Sandy Clayey SILT (ML)				



Cobbles	< 300 mm (12") and > 75 mm (3")	Fine Sand	< 0.425 mm and > 0.075 mm (#200)
Gravel	< 75 mm and > 4.75 mm (#4)	Silt	< 0.075 and > 0.005 mm
Coarse Sand	< 4.75 mm and >2.00 mm (#10)	Clay	< 0.005 mm
Medium Sand	< 2.00 mm and > 0.425 mm (#40)	Colloids	< 0.001 mm

Moisture Content		Gravel	0.0%	Liquid Limit	45
Silt & Clay (% Passing #200)	66.3%	Sand	33.7%	Plastic Limit	36
Maximum Particle Size	#10	Silt	53.8%	Plastic Index	9
Specific Gravity	2.757	Clay	12.5%	Colloids	3.0%

Description of Sand & Gravel Particles: Rounded ☐ Angular ☐ Hard & Durable ☐ Soft ☐ Weathered & Friable ☐

ASTM D 422: Particle Size Analysis of Soils — Dispersing Agent: Sodium Hexametaphosphate: 40 g./ Liter; Mechanical Stirring Apparatus (A); Length of Dispersion Period: 1 min.

References: ASTM D 854: Specific Gravity of Soils
ASTM D 421: Dry Preparation of Soil Samples
ASTM D 2216: Laboratory Determination of Water (Moisture) Content of Soil and Rock by Mass
ASTM D 4318: Liquid Limit, Plastic Limit, & Plastic Index of Soils
ASTM D 2487: Classification of Soils for Engineering Purposes (Unified Soil Classification System)

Technical Responsibility / Position: Mal Krajan — Laboratory Supervisor

Particle Size Analysis of Soils

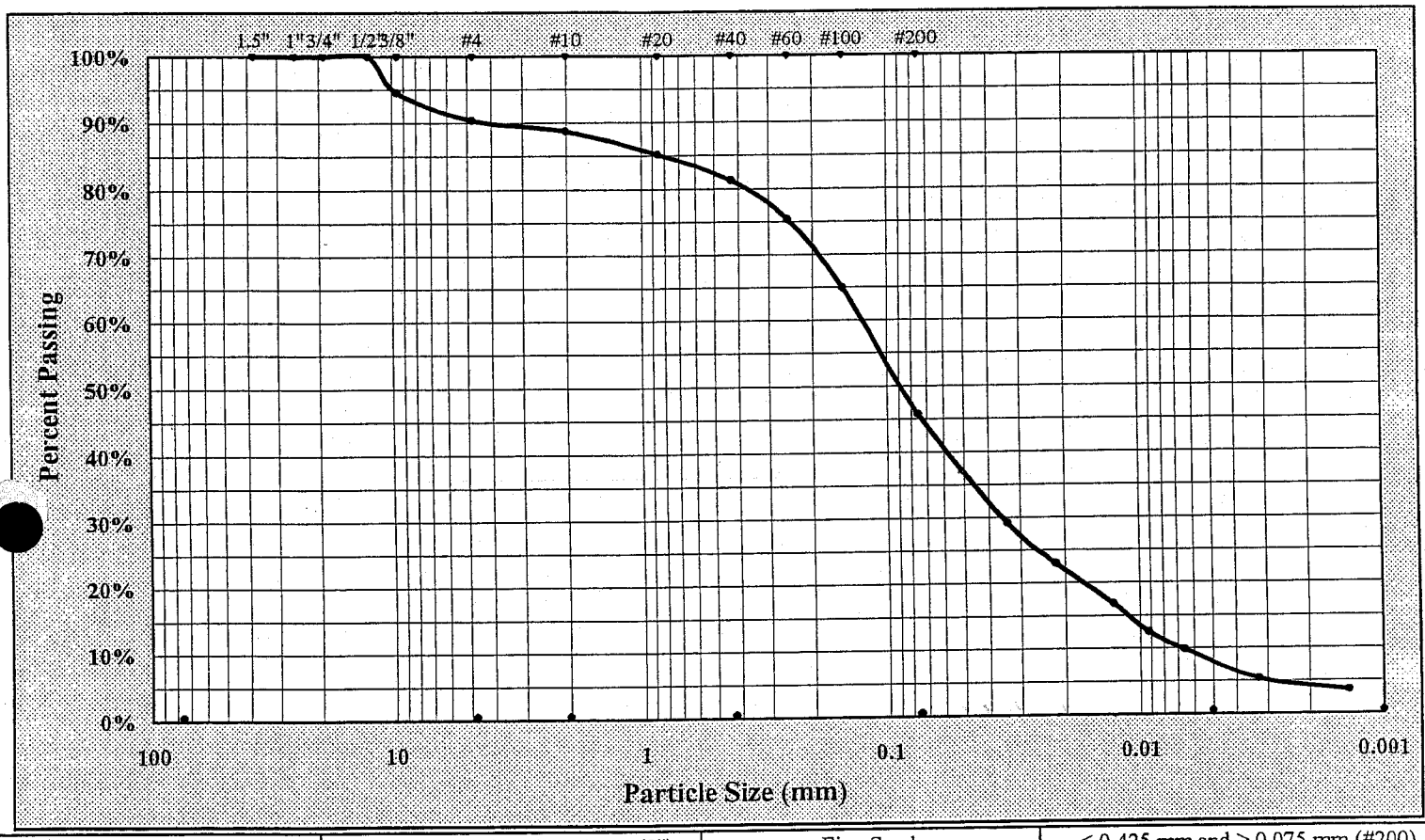
ASTM D 422

S&ME

S&ME Project #: **1054-04-955**
Project Name: **Belews Creek Steam Plant**
Client Name: Duke Energy
Client Address:

Report Date: 10/08/2004
Test Date(s): 09/28 - 10/07/2004

Boring #:	BC-16	Sample #:	SS-3	Sample Date:	9/1/04
Location:	Site Borehole	Offset:	NA	Depth (ft):	13.5 - 15'
Sample Description:	Purple and Tan Clayey SAND with Gravel (SC)				



Cobbles	< 300 mm (12") and > 75 mm (3")	Fine Sand	< 0.425 mm and > 0.075 mm (#200)
Gravel	< 75 mm and > 4.75 mm (#4)	Silt	< 0.075 and > 0.005 mm
Coarse Sand	< 4.75 mm and >2.00 mm (#10)	Clay	< 0.005 mm
Medium Sand	< 2.00 mm and > 0.425 mm (#40)	Colloids	< 0.001 mm

Maximum Particle Size	1/2"	Gravel	9.7%	Silt	38.1%
Silt & Clay (% Passing #200)	45.6%	Sand	44.7%	Clay	7.5%
Apparent Relative Density	2.677	Moisture Content		Colloids	3.5%
Liquid Limit	36	Plastic Limit	24	Plastic Index	12

Description of Sand & Gravel Particles Rounded ☐ Angular ☒ Hard & Durable ☐ Soft ☒ Weathered & Friable ☐

Mechanical Stirring Apparatus (A) Length of Dispersion Period: 1 min. Dispersing Agent: Sodium Hexametaphosphate: 40 g./ Liter

References: ASTM D 422: Particle Size Analysis of Soils
ASTM D 4318: Liquid Limit, Plastic Limit, & Plastic Index of Soils
ASTM D 2487: Classification of Soils for Engineering Purposes (Unified Soil Classification System)
ASTM D 421: Dry Preparation of Soil Samples
ASTM D 854: Specific Gravity of Soils

Technical Responsibility: Mal Krajan — Laboratory Supervisor
(Signature) (Position)

S&ME, INC.
3109 Spring Forest Road, Raleigh, N.C. 27616

Particle Size Analysis of Soils

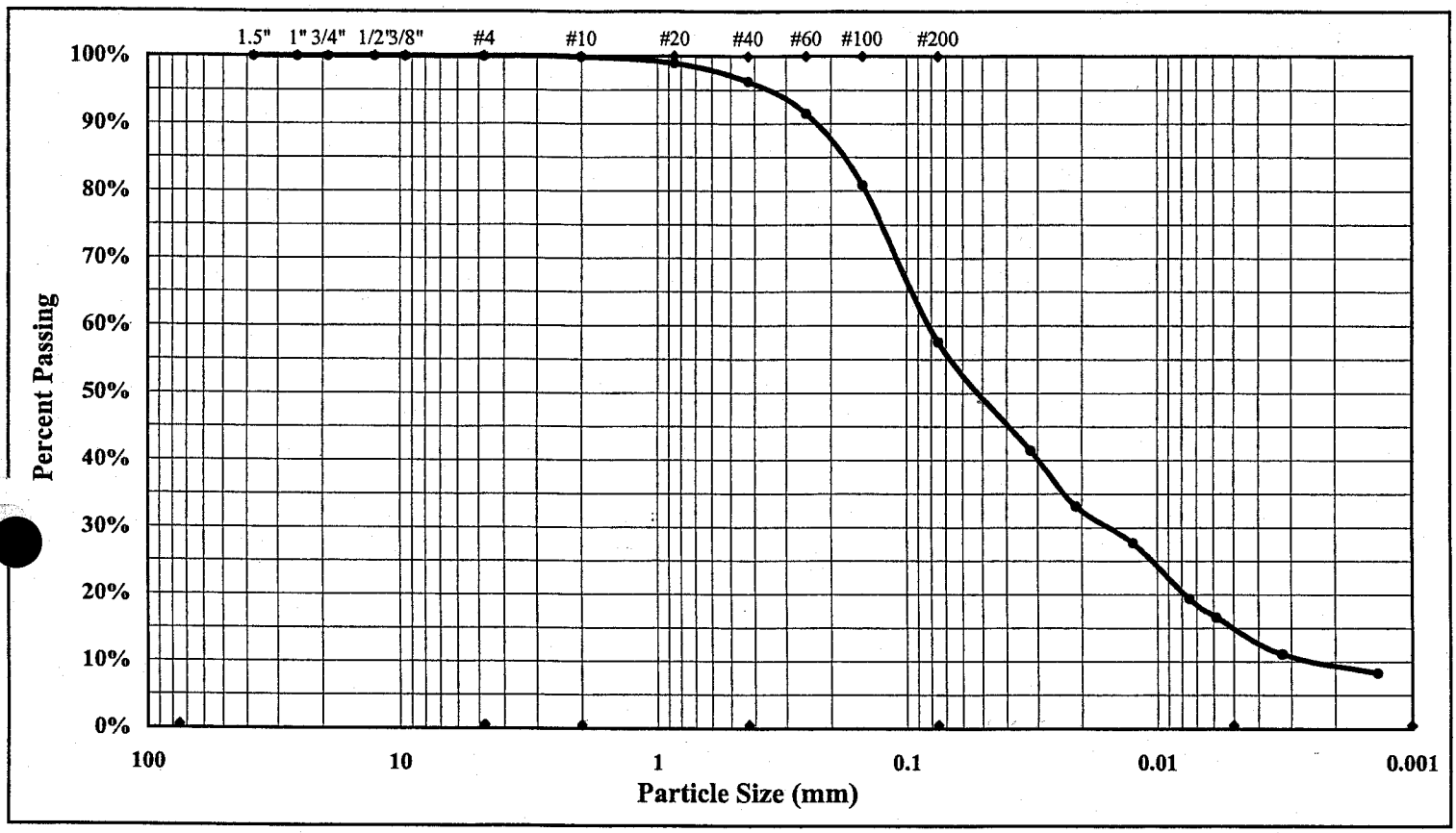
ASTM D 422

S&ME

S&ME Project #: 1264-02-576
Project Name: Duke Power - Belews Creek Landfill
Client Name: Duke Energy Corp.
Client Address: 526 South Church S, Charlotte, NC 28202

Report Date: January 9, 2003
Test Date(s): 1/2 - 1/9/03

Boring #:	BC-3	Sample #:		Sample Date:	NA
Location:	Scrubber Waste L.F.	Offset:		Depth (ft):	0.0 - 10.0
Sample Description:	Brown Fine Sandy Silty CLAY (CL)				



Cobbles	< 300 mm (12") and > 75 mm (3")	Fine Sand	< 0.425 mm and > 0.075 mm (#200)
Gravel	< 75 mm and > 4.75 mm (#4)	Silt	< 0.075 and > 0.005 mm
Coarse Sand	< 4.75 mm and >2.00 mm (#10)	Clay	< 0.005 mm
Medium Sand	< 2.00 mm and > 0.425 mm (#40)	Colloids	< 0.001 mm

Moisture Content	5.8%	Gravel	0.0%	Liquid Limit	37
Silt & Clay (% Passing #200)	57.6%	Sand	42.4%	Plastic Limit	20
Maximum Particle Size	#10	Silt	42.7%	Plastic Index	17
Specific Gravity	2.761	Clay	14.9%	Colloids	8.3%

Description of Sand & Gravel Particles: Rounded ☐ Angular ☐ Hard & Durable ☐ Soft ☐ Weathered & Friable ☐

ASTM D 422: Particle Size Analysis of Soils

Dispersing Agent: Sodium Hexametaphosphate: 40 g./ Liter
Mechanical Stirring Apparatus (A) Length of Dispersion Period: 1 min.

References: ASTM D 854: Specific Gravity of Soils
ASTM D 421: Dry Preparation of Soil Samples
ASTM D 2216: Laboratory Determination of Water (Moisture) Content of Soil and Rock by Mass
ASTM D 4318: Liquid Limit, Plastic Limit, & Plastic Index of Soils
ASTM D 2487: Classification of Soils for Engineering Purposes (Unified Soil Classification System)

Technical Responsibility / Position: Mal Krajan — Laboratory Supervisor

Particle Size Analysis of Soils

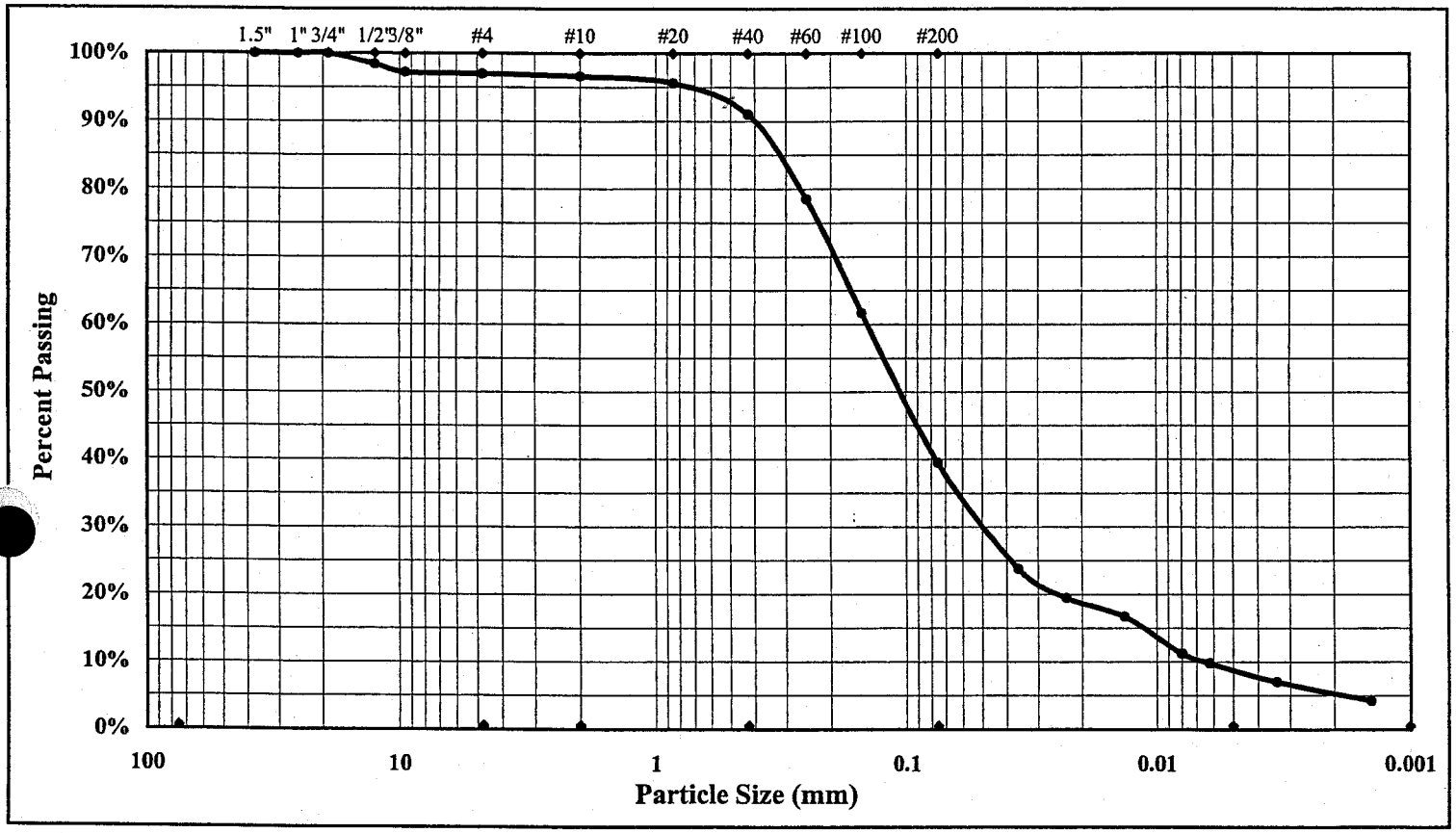
ASTM D 422

S&ME

ME Project #: **1264-02-576**
ject Name: **Duke Power - Belews Creek Landfill**
Client Name: Duke Energy Corp.
Client Address: 526 South Church Street, Charlotte, NC 28202

Report Date: January 9, 2003
Test Date(s): 1/2 - 1/7/03

Boring #:	BC-10	Sample #:		Sample Date:	NA
Location:	Scrubber Waste L.F.	Offset:		Depth (ft):	0.0 - 10.0
Sample Description:	Yellow Clayey Silty Fine SAND with a trace of Gravel				



Cobbles	< 300 mm (12") and > 75 mm (3")	Fine Sand	< 0.425 mm and > 0.075 mm (#200)
Gravel	< 75 mm and > 4.75 mm (#4)	Silt	< 0.075 and > 0.005 mm
Coarse Sand	< 4.75 mm and >2.00 mm (#10)	Clay	< 0.005 mm
Medium Sand	< 2.00 mm and > 0.425 mm (#40)	Colloids	< 0.001 mm

Moisture Content	9.3%	Gravel	3.0%	Liquid Limit	24
Silt & Clay (% Passing #200)	39.5%	Sand	57.5%	Plastic Limit	21
Maximum Particle Size	3/4"	Silt	31.6%	Plastic Index	3
Specific Gravity	2.652	Clay	7.9%	Colloids	4.2%

Description of Sand & Gravel Particles: Rounded ☐ Angular ☐ Hard & Durable ☐ Soft ☐ Weathered & Friable ☒

ASTM D 422: Particle Size Analysis of Soils Dispersing Agent: Sodium Hexametaphosphate: 40 g./ Liter
Mechanical Stirring Apparatus (A) Length of Dispersion Period: 1 min.

References: ASTM D 854: Specific Gravity of Soils
TM D 421: Dry Preparation of Soil Samples
M D 2216: Laboratory Determination of Water (Moisture) Content of Soil and Rock by Mass
ASTM D 4318: Liquid Limit, Plastic Limit, & Plastic Index of Soils
ASTM D 2487: Classification of Soils for Engineering Purposes (Unified Soil Classification System)

Technical Responsibility / Position: Mal Krajan — Laboratory Supervisor

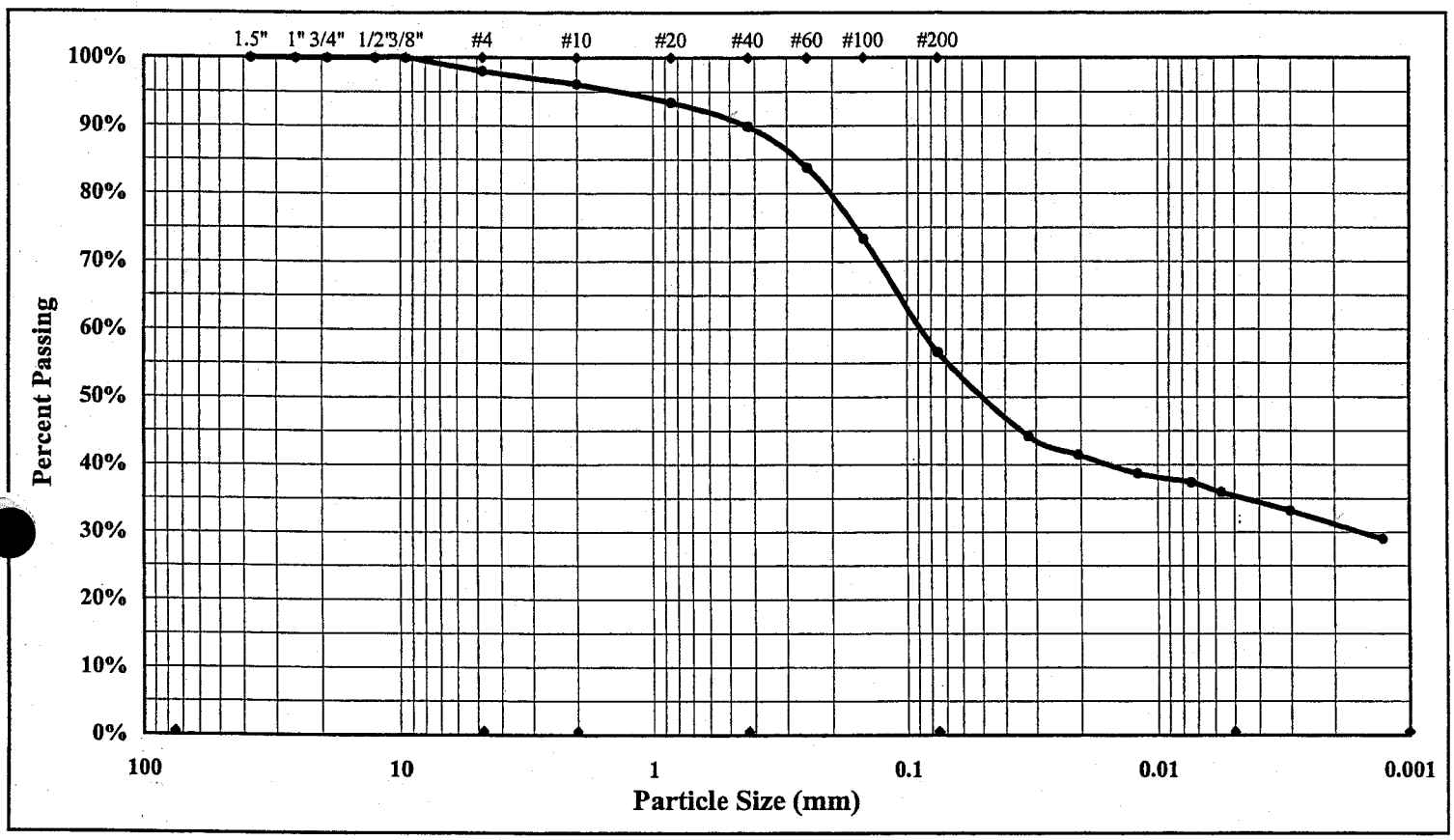
Particle Size Analysis of Soils

ASTM D 422

S&ME

ME Project #:	**1264-02-576**	Report Date:	January 9, 2003
ject Name:	**Duke Power - Belews Co. Landfill**	Test Date(s):	37623
Client Name:	Duke Energy Corp.		
Client Address:	526 S. Church St.		

Boring #:	BC-11	Sample #:		Sample Date:	not reported
Location:	Scrubber Waste	Offset:		Depth (ft):	0-10
Sample Description:	Red Silty Clayey Sand with a trace of Gravel				



Cobbles	< 300 mm (12") and > 75 mm (3")	Fine Sand	< 0.425 mm and > 0.075 mm (#200)
Gravel	< 75 mm and > 4.75 mm (#4)	Silt	< 0.075 and > 0.005 mm
Coarse Sand	< 4.75 mm and >2.00 mm (#10)	Clay	< 0.005 mm
Medium Sand	< 2.00 mm and > 0.425 mm (#40)	Colloids	< 0.001 mm

Moisture Content	19.4%	Gravel	2.0%	Liquid Limit	47
Silt & Clay (% Passing #200)	56.6%	Sand	41.4%	Plastic Limit	26
Maximum Particle Size	#4	Silt		Plastic Index	21
Specific Gravity	2.716	Clay		Colloids	29.0%

Description of Sand & Gravel Particles: Rounded ☐ Angular ☐ Hard & Durable ☐ Soft ☐ Weathered & Friable ☐

ASTM D 422: Particle Size Analysis of Soils Dispersing Agent: Sodium Hexametaphosphate: 40 g./ Liter

Mechanical Stirring Apparatus (A) Length of Dispersion Period: 1 min.

References: ASTM D 854: Specific Gravity of Soils

TM D 421: Dry Preparation of Soil Samples

M D 2216: Laboratory Determination of Water (Moisture) Content of Soil and Rock by Mass

ASTM D 4318: Liquid Limit, Plastic Limit, & Plastic Index of Soils

ASTM D 2487: Classification of Soils for Engineering Purposes (Unified Soil Classification System)

Technical Responsibility / Position: Mal Krajan — Laboratory Supervisor

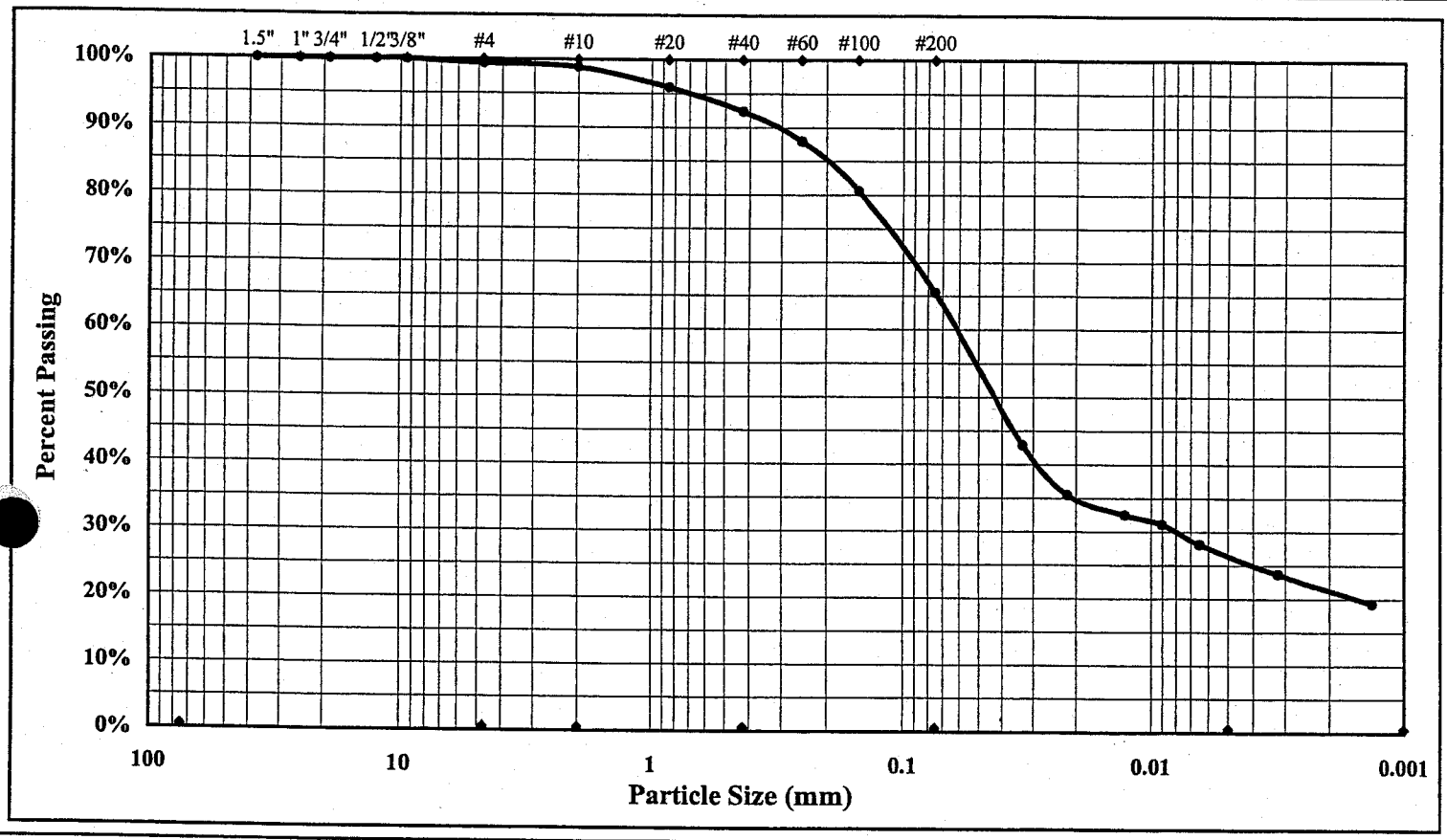
Particle Size Analysis of Soils

ASTM D 422

S&ME

ME Project #:	1264-02-576	Report Date:	January 9, 2003
ject Name:	**Duke Power - Belews Co. Landfill**	Test Date(s):	1/2 - 1/8/03
Client Name:	Duke Energy Corp.		
Client Address:	526 S. Church St.		

Boring #:	BC-13	Sample #:	Bag-1	Sample Date:	NA
Location:	Scrubber Waste L.F.	Offset:		Depth (ft):	0.0 - 10.0
Sample Description:	Red Fine Sandy Clayey SILT with a Trace of Gravel (ML)				



Cobbles	< 300 mm (12") and > 75 mm (3")	Fine Sand	< 0.425 mm and > 0.075 mm (#200)
Gravel	< 75 mm and > 4.75 mm (#4)	Silt	< 0.075 and > 0.005 mm
Coarse Sand	< 4.75 mm and >2.00 mm (#10)	Clay	< 0.005 mm
Medium Sand	< 2.00 mm and > 0.425 mm (#40)	Colloids	< 0.001 mm

Moisture Content	11.1%	Gravel	0.6%	Liquid Limit	38
Silt & Clay (% Passing #200)	65.5%	Sand	33.9%	Plastic Limit	28
Maximum Particle Size	3/8"	Silt		Plastic Index	10
Specific Gravity	2.672	Clay		Colloids	19.3%

Description of Sand & Gravel Particles: Rounded ☐ Angular ☐ Hard & Durable ☐ Soft ☐ Weathered & Friable ☐

ASTM D 422: Particle Size Analysis of Soils Dispersing Agent: Sodium Hexametaphosphate: 40 g./ Liter

Mechanical Stirring Apparatus (A) Length of Dispersion Period: 1 min.

References: ASTM D 854: Specific Gravity of Soils

M D 421: Dry Preparation of Soil Samples

M D 2216: Laboratory Determination of Water (Moisture) Content of Soil and Rock by Mass

ASTM D 4318: Liquid Limit, Plastic Limit, & Plastic Index of Soils

ASTM D 2487: Classification of Soils for Engineering Purposes (Unified Soil Classification System)

Technical Responsibility / Position: Mal Krajan — Quality System Coordinator

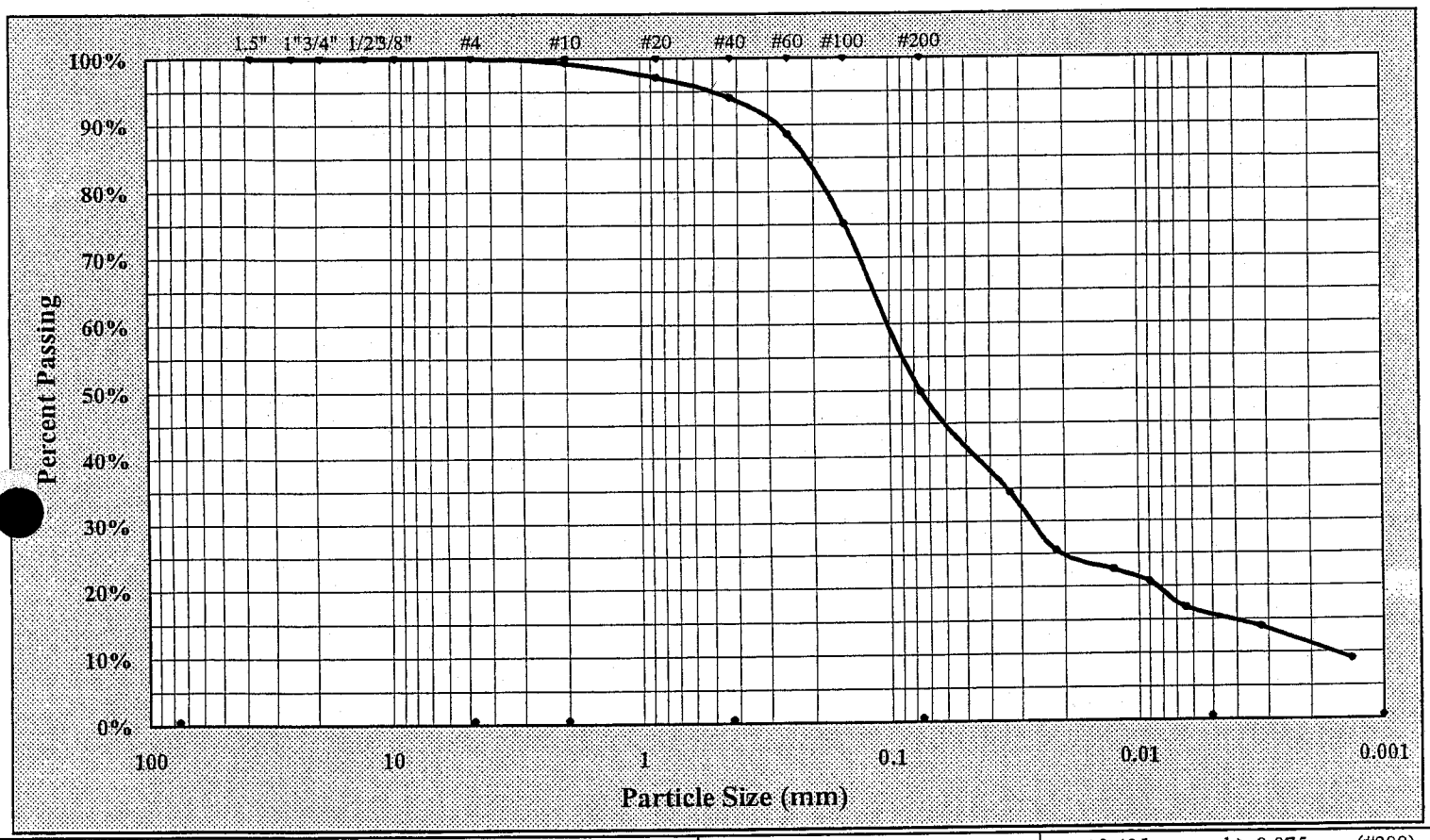
Particle Size Analysis of Soils

ASTM D 422

S&ME Project #: **1054-04-955**
Project Name: **Belews Creek Steam Plant**
Client Name: Duke Energy
Client Address:

Report Date: 09/21/2004
Test Date(s): 09/13 - 09/17/2004

Boring #:	BC-16	Sample #:	Bag-1	Sample Date:	9/1/04
Location:	Site Borehole	Offset:	NA	Depth (ft):	0 - 5'
Sample Description:	Tan-Brown Silty SAND (SM)				



Cobbles	< 300 mm (12") and > 75 mm (3")	Fine Sand	< 0.425 mm and > 0.075 mm (#200)
Gravel	< 75 mm and > 4.75 mm (#4)	Silt	< 0.075 and > 0.005 mm
Coarse Sand	< 4.75 mm and >2.00 mm (#10)	Clay	< 0.005 mm
Medium Sand	< 2.00 mm and > 0.425 mm (#40)	Colloids	< 0.001 mm

Maximum Particle Size	#4	Gravel	0.0%	Silt	34.1%
Silt & Clay (% Passing #200)	49.7%	Sand	50.3%	Clay	15.6%
Apparent Relative Density	2.708	Moisture Content	42.1%	Colloids	8.8%
Liquid Limit	30	Plastic Limit	22	Plastic Index	8

Description of Sand & Gravel Particles Rounded ☐ Angular ☐ Hard & Durable ☐ Soft ☐ Weathered & Friable ☐

Mechanical Stirring Apparatus (A) Length of Dispersion Period: 1 min. Dispersing Agent: Sodium Hexametaphosphate: 40 g./ Liter

References: ASTM D 422: Particle Size Analysis of Soils; ASTM D 421: Dry Preparation of Soil Samples
ASTM D 4318: Liquid Limit, Plastic Limit, & Plastic Index of Soils; ASTM D 854: Specific Gravity of Soils
ASTM D 2487: Classification of Soils for Engineering Purposes (Unified Soil Classification System)

Technical Responsibility: Mal Krajan — Laboratory Supervisor
(Signature) (Position)

Particle Size Analysis of Soils

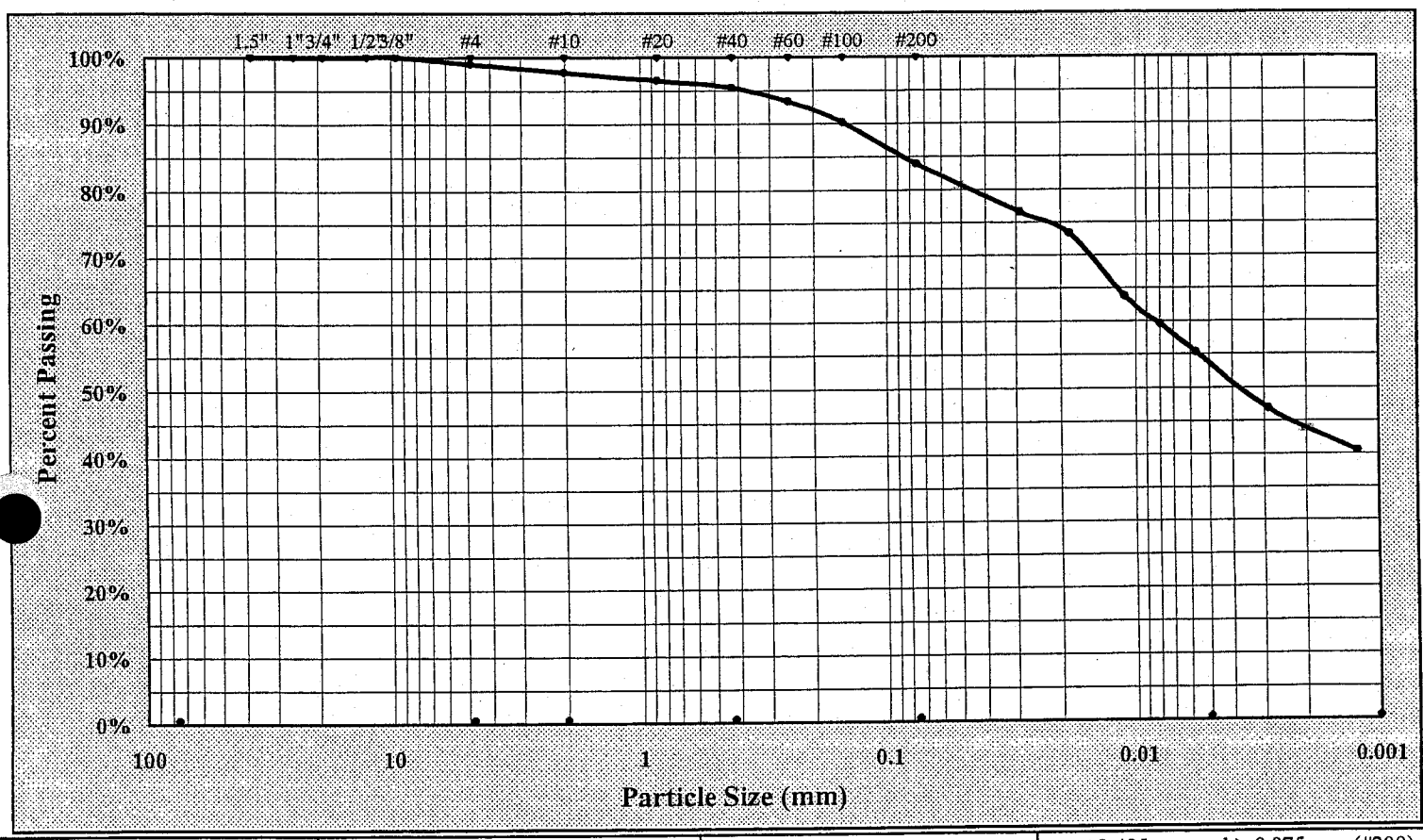
S&ME

ASTM D 422

S&ME Project #: **1054-04-955**
Project Name: **Belew Creek Steam Plant**
Client Name: Duke Energy
Client Address:

Report Date: 10/08/2004
Test Date(s): 09/28 - 10/07/2004

Boring #:	BC-19	Sample #:	Bag-1	Sample Date:	9/1/04
Location:	Site Borehole	Offset:	NA	Depth (ft):	2 - 7'
Sample Description:	Orange-Brown Sandy SILT (MH)				



Cobbles	< 300 mm (12") and > 75 mm (3")	Fine Sand	< 0.425 mm and > 0.075 mm (#200)
Gravel	< 75 mm and > 4.75 mm (#4)	Silt	< 0.075 and > 0.005 mm
Coarse Sand	< 4.75 mm and >2.00 mm (#10)	Clay	< 0.005 mm
Medium Sand	< 2.00 mm and > 0.425 mm (#40)	Colloids	< 0.001 mm

Maximum Particle Size	3/8"	Gravel	1.0%	Silt	30.8%
Silt & Clay (% Passing #200)	83.8%	Sand	15.2%	Clay	53.0%
Apparent Relative Density	2.735	Moisture Content	37.1%	Colloids	40.3%
Liquid Limit	72	Plastic Limit	44	Plastic Index	28

Description of Sand & Gravel Particles Rounded ☐ Angular ☐ Hard & Durable ☐ Soft ☐ Weathered & Friable ☐

Mechanical Stirring Apparatus (A) Length of Dispersion Period: 1 min. Dispersing Agent: Sodium Hexametaphosphate: 40 g./ Liter

References: ASTM D 422: Particle Size Analysis of Soils
ASTM D 421: Dry Preparation of Soil Samples
ASTM D 4318: Liquid Limit, Plastic Limit, & Plastic Index of Soils
ASTM D 854: Specific Gravity of Soils
ASTM D 2487: Classification of Soils for Engineering Purposes (Unified Soil Classification System)

Technical Responsibility: Mal Krajan — Laboratory Supervisor
Signature — Position

Particle Size Analysis of Soils

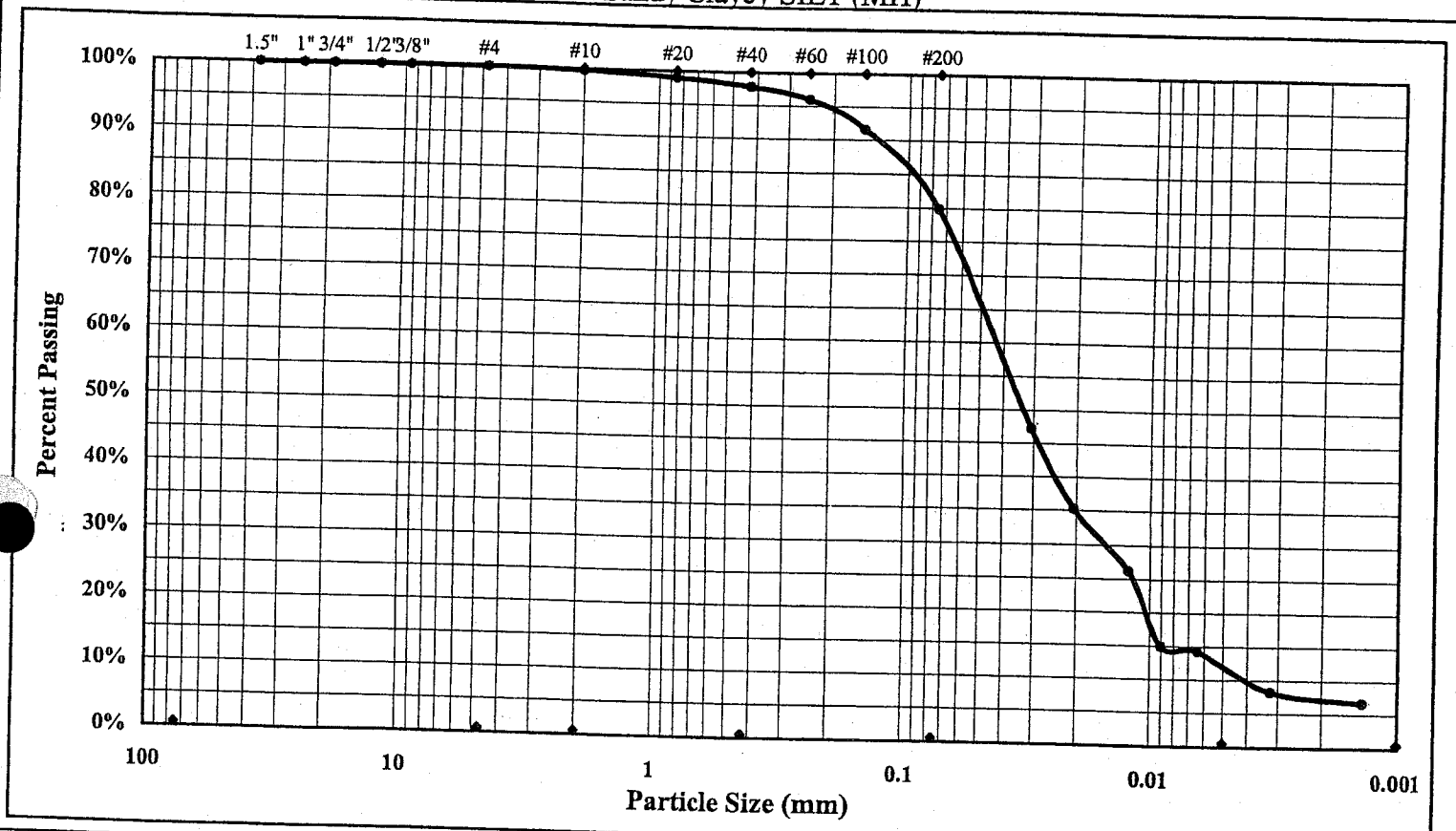
ASTM D 422

S&ME

ME Project #: 1264-02-576
Project Name: Duke Power-Belews Creek Landfill
Client Name:
Client Address:

Report Date: April 1, 2003
Test Date(s): 3/26 - 4/1/03

Boring #: BC-10 Sample #: UD-1 Sample Date: NA
Location: Offset: Depth (ft): 12.0 - 14.0
Sample Description: Yellow and Tan-Brown Sandy Clayey SILT (MH)



Cobbles	< 300 mm (12") and > 75 mm (3")	Fine Sand	< 0.425 mm and > 0.075 mm (#200)
Gravel	< 75 mm and > 4.75 mm (#4)	Silt	< 0.075 and > 0.005 mm
Coarse Sand	< 4.75 mm and >2.00 mm (#10)	Clay	< 0.005 mm
Medium Sand	< 2.00 mm and > 0.425 mm (#40)	Colloids	< 0.001 mm

Moisture Content		Gravel	0.0%	Liquid Limit	59
Silt & Clay (% Passing #200)	79.9%	Sand	20.1%	Plastic Limit	45
Maximum Particle Size	#4	Silt	67.9%	Plastic Index	14
Specific Gravity	2.797	Clay	12.0%	Colloids	6.7%

Description of Sand & Gravel Particles: Rounded ☐ Angular ☐ Hard & Durable ☐ Soft ☐ Weathered & Friable ☐

ASTM D 422: Particle Size Analysis of Soils

Dispersing Agent: Sodium Hexametaphosphate: 40 g./ Liter
Mechanical Stirring Apparatus (A) Length of Dispersion Period: 1 min.

References: ASTM D 854: Specific Gravity of Soils
ASTM D 421: Dry Preparation of Soil Samples
ASTM D 2216: Laboratory Determination of Water (Moisture) Content of Soil and Rock by Mass
ASTM D 4318: Liquid Limit, Plastic Limit, & Plastic Index of Soils
ASTM D 2487: Classification of Soils for Engineering Purposes (Unified Soil Classification System)

Technical Responsibility / Position: Mal Krajan — Laboratory Supervisor

Particle Size Analysis of Soils

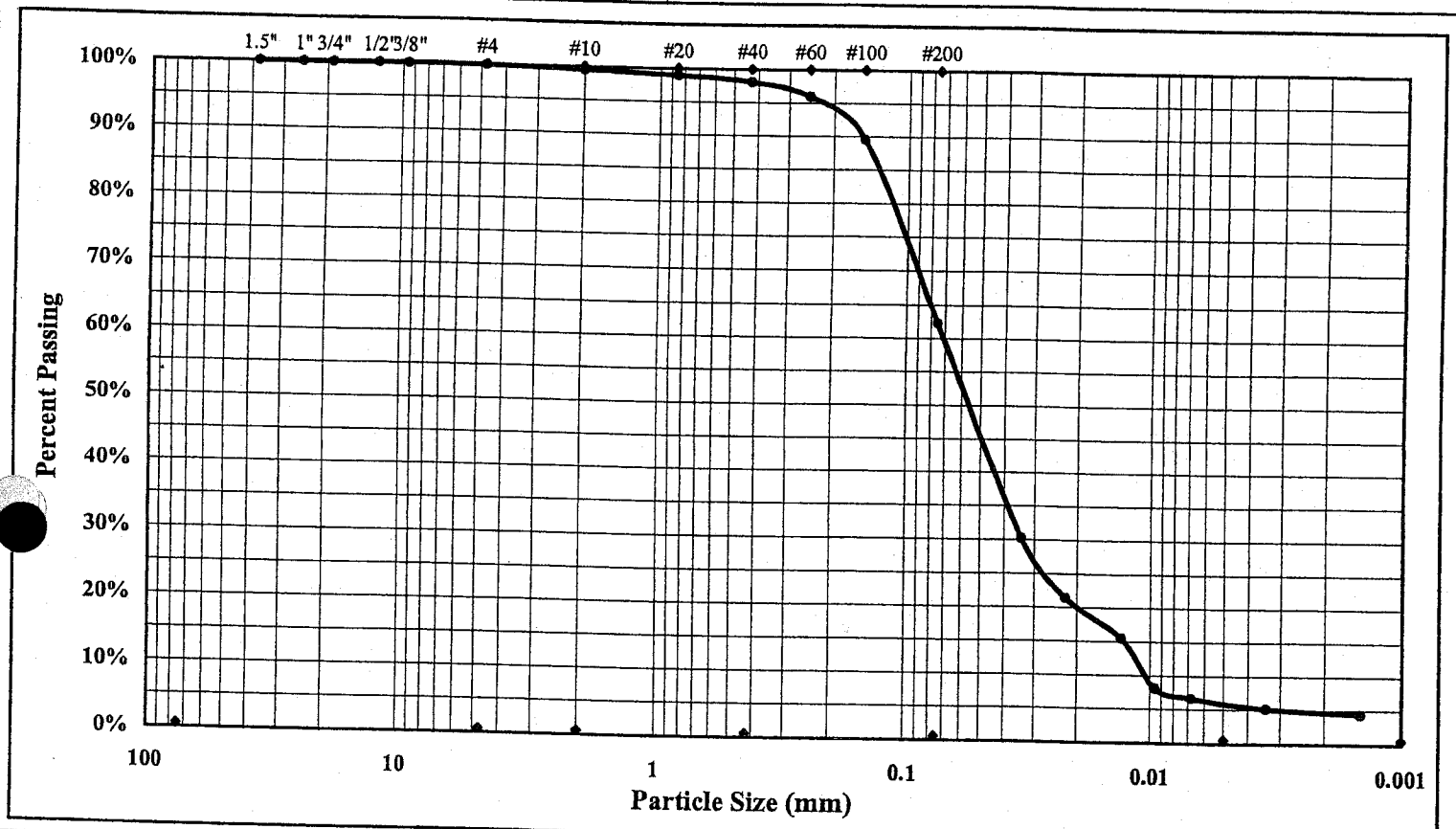
ASTM D 422

S&ME

S&ME Project #: 1264-02-576
Project Name: Duke Power-Belew's Creek Landfill
Client Name:
Client Address:

Report Date: April 1, 2003
Test Date(s): 3/25/2003-4/1/03

Boring #: BC-12 Sample #: UD-1 Sample Date: NA
Location: Offset: Depth (ft): 27.0 - 28.0
Sample Description: Tan Sandy Clayey SILT (ML)



Cobbles	< 300 mm (12") and > 75 mm (3")	Fine Sand	< 0.425 mm and > 0.075 mm (#200)
Gravel	< 75 mm and > 4.75 mm (#4)	Silt	< 0.075 and > 0.005 mm
Coarse Sand	< 4.75 mm and >2.00 mm (#10)	Clay	< 0.005 mm
Medium Sand	< 2.00 mm and > 0.425 mm (#40)	Colloids	< 0.001 mm

Moisture Content		Gravel	0.1%	Liquid Limit	32
Silt & Clay (% Passing #200)	62.3%	Sand	37.6%	Plastic Limit	26
Maximum Particle Size	#20	Silt	56.3%	Plastic Index	6
Specific Gravity	2.654	Clay	6.0%	Colloids	4.4%

Description of Sand & Gravel Particles: Rounded ☐ Angular ☐ Hard & Durable ☐ Soft ☐ Weathered & Friable ☐

ASTM D 422: Particle Size Analysis of Soils
Dispersing Agent: Sodium Hexametaphosphate: 40 g./ Liter
Mechanical Stirring Apparatus (A) Length of Dispersion Period: 1 min.

References: ASTM D 854: Specific Gravity of Soils
ASTM D 421: Dry Preparation of Soil Samples
ASTM D 2216: Laboratory Determination of Water (Moisture) Content of Soil and Rock by Mass
ASTM D 4318: Liquid Limit, Plastic Limit, & Plastic Index of Soils
ASTM D 2487: Classification of Soils for Engineering Purposes (Unified Soil Classification System)

Technical Responsibility / Position: Mal Krajan — Laboratory Supervisor

Particle Size Analysis of Soils

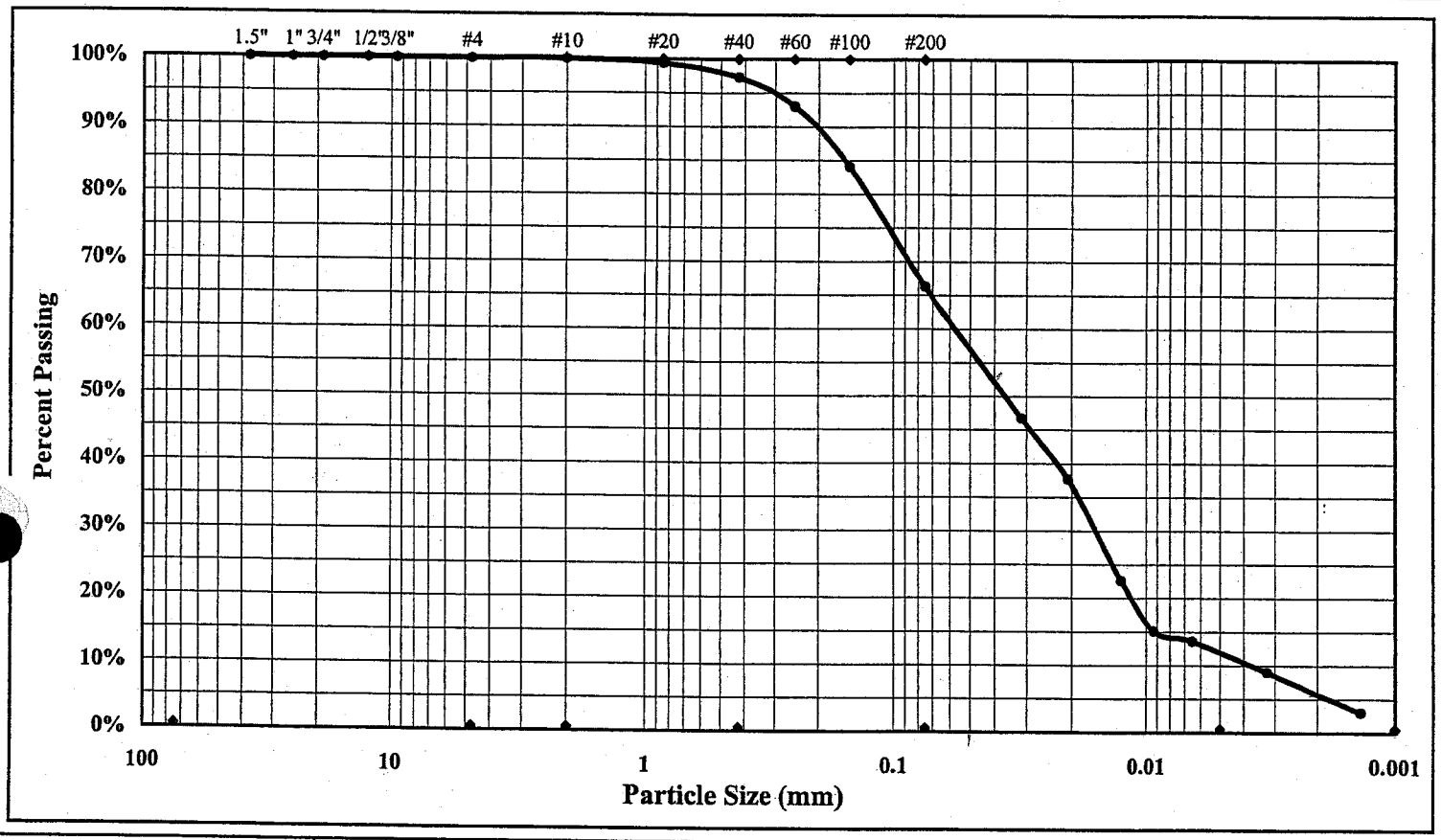
ASTM D 422

S&ME

S&ME Project #: **1264-02-576**
Project Name: **Duke Power-Belews Creek Landfill**
Client Name: Duke Energy Corp.
Client Address: 3195 Pine Hall Road, Belews Creek, NC 27009

Report Date: January 22, 2003
Test Date(s): 1/15/03-1/22/03

Boring #: BC-12 Sample #: SS-6 Sample Date: NA
Location: Offset: Depth (ft): 18.5 - 20.0
Sample Description: Tan Fine Sandy Clayey SILT (ML)



Cobbles	< 300 mm (12") and > 75 mm (3")	Fine Sand	< 0.425 mm and > 0.075 mm (#200)
Gravel	< 75 mm and > 4.75 mm (#4)	Silt	< 0.075 and > 0.005 mm
Coarse Sand	< 4.75 mm and >2.00 mm (#10)	Clay	< 0.005 mm
Medium Sand	< 2.00 mm and > 0.425 mm (#40)	Colloids	< 0.001 mm

Moisture Content		Gravel	0.0%	Liquid Limit	45
Silt & Clay (% Passing #200)	66.3%	Sand	33.7%	Plastic Limit	36
Maximum Particle Size	#10	Silt	53.8%	Plastic Index	9
Specific Gravity	2.757	Clay	12.5%	Colloids	3.0%

Description of Sand & Gravel Particles: Rounded ☐ Angular ☐ Hard & Durable ☐ Soft ☐ Weathered & Friable ☐

ASTM D 422: Particle Size Analysis of Soils
Dispersing Agent: Sodium Hexametaphosphate: 40 g./ Liter
Mechanical Stirring Apparatus (A) Length of Dispersion Period: 1 min.

References: ASTM D 854: Specific Gravity of Soils
ASTM D 421: Dry Preparation of Soil Samples
ASTM D 2216: Laboratory Determination of Water (Moisture) Content of Soil and Rock by Mass
ASTM D 4318: Liquid Limit, Plastic Limit, & Plastic Index of Soils
ASTM D 2487: Classification of Soils for Engineering Purposes (Unified Soil Classification System)

Technical Responsibility / Position: Mal Krajan — Laboratory Supervisor

Particle Size Analysis for Material at Grade or Slightly Below Grade

Particle Size Analysis of Soils

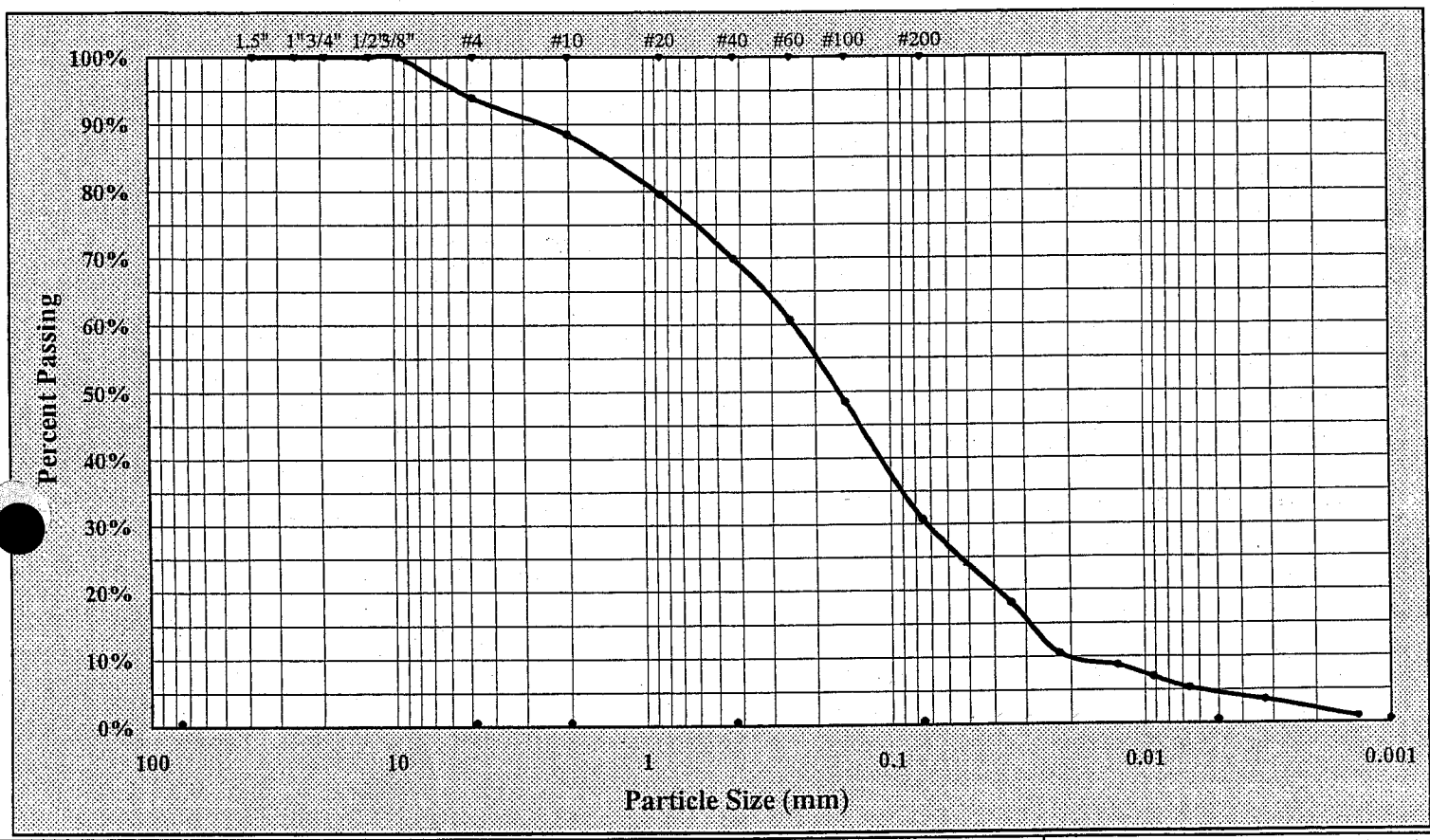
S&ME

ASTM D 422

S&ME Project #: **1054-04-955**
Project Name: **Belews Creek Steam Plant**
Client Name: Duke Energy
Client Address:

Report Date: 09/21/2004
Test Date(s): 09/10 - 09/15/2004

Boring #:	BC-18	Sample #:	SS-4	Sample Date:	9/1/04
Location:	Site Borehole	Offset:	NA	Depth (ft):	18.5'
Sample Description:	Brown Clayey SAND (SC)				



Cobbles	< 300 mm (12") and > 75 mm (3")	Fine Sand	< 0.425 mm and > 0.075 mm (#200)
Gravel	< 75 mm and > 4.75 mm (#4)	Silt	< 0.075 and > 0.005 mm
Coarse Sand	< 4.75 mm and >2.00 mm (#10)	Clay	< 0.005 mm
Medium Sand	< 2.00 mm and > 0.425 mm (#40)	Colloids	< 0.001 mm

Maximum Particle Size	3/8"	Gravel	6.2%	Silt	26.0%
Silt & Clay (% Passing #200)	30.7%	Sand	63.1%	Clay	4.7%
Apparent Relative Density	2.759	Moisture Content		Colloids	0.9%
Liquid Limit	32	Plastic Limit	23	Plastic Index	9

Description of Sand & Gravel Particles Rounded ☐ Angular ☐ Hard & Durable ☐ Soft ☐ Weathered & Friable ☐

Mechanical Stirring Apparatus (A) Length of Dispersion Period: 1 min. Dispersing Agent: Sodium Hexametaphosphate: 40 g./ Liter

References: ASTM D 422: Particle Size Analysis of Soils
ASTM D 421: Dry Preparation of Soil Samples
ASTM D 4318: Liquid Limit, Plastic Limit, & Plastic Index of Soils
ASTM D 854: Specific Gravity of Soils
ASTM D 2487: Classification of Soils for Engineering Purposes (Unified Soil Classification System)

Technical Responsibility: Mal Krajan — Signature — Laboratory Supervisor — Position

Particle Size Analysis of Soils

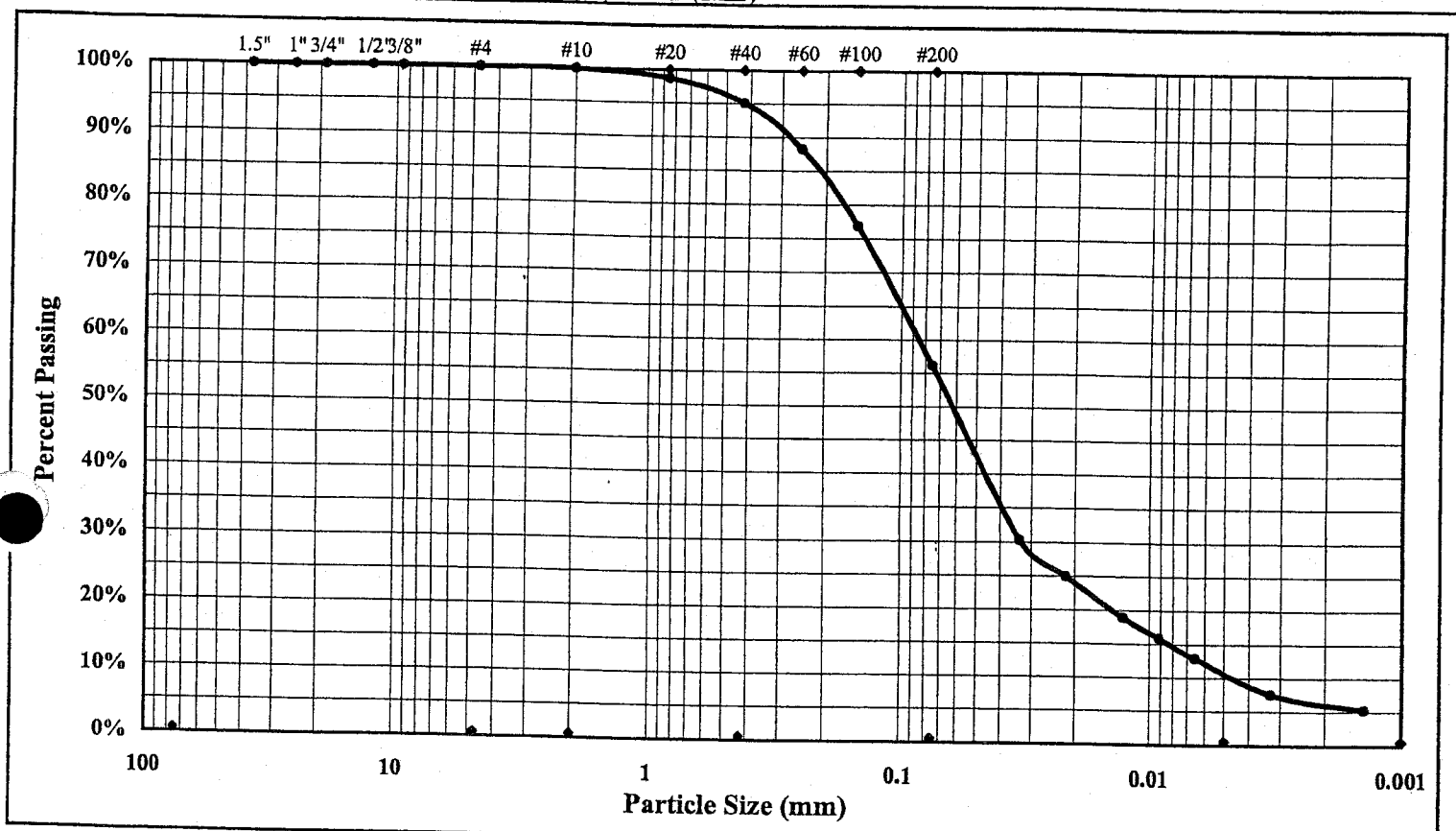
ASTM D 422

S&ME

S&ME Project #: 1264-02-576
Project Name: Duke Power-Belews Creek Landfill
Client Name:
Client Address:

Report Date: April 1, 2003
Test Date(s): 3/25 - 4/1/03

Boring #:	BC-11	Sample #:	UD-1	Sample Date:	NA
Location:		Offset:		Depth (ft):	23.0 - 25.0
Sample Description:	Tan-Brown Sandy Clayey SILT (ML)				



Cobbles	< 300 mm (12") and > 75 mm (3")	Fine Sand	< 0.425 mm and > 0.075 mm (#200)
Gravel	< 75 mm and > 4.75 mm (#4)	Silt	< 0.075 and > 0.005 mm
Coarse Sand	< 4.75 mm and >2.00 mm (#10)	Clay	< 0.005 mm
Medium Sand	< 2.00 mm and > 0.425 mm (#40)	Colloids	< 0.001 mm

Moisture Content		Gravel	0.0%	Liquid Limit	44
Silt & Clay (% Passing #200)	56.2%	Sand	43.8%	Plastic Limit	37
Maximum Particle Size	#4	Silt	46.0%	Plastic Index	7
Specific Gravity	2.745	Clay	10.2%	Colloids	5.3%

Description of Sand & Gravel Particles: Rounded ☐ Angular ☐ Hard & Durable ☐ Soft ☐ Weathered & Friable ☐

ASTM D 422: Particle Size Analysis of Soils

Dispersing Agent: Sodium Hexametaphosphate: 40 g./ Liter
Mechanical Stirring Apparatus (A) Length of Dispersion Period: 1 min.

References: ASTM D 854: Specific Gravity of Soils
ASTM D 421: Dry Preparation of Soil Samples
ASTM D 2216: Laboratory Determination of Water (Moisture) Content of Soil and Rock by Mass
ASTM D 4318: Liquid Limit, Plastic Limit, & Plastic Index of Soils
ASTM D 2487: Classification of Soils for Engineering Purposes (Unified Soil Classification System)

Technical Responsibility / Position: Mal Krajan — Laboratory Supervisor

Particle Size Analysis of Soils

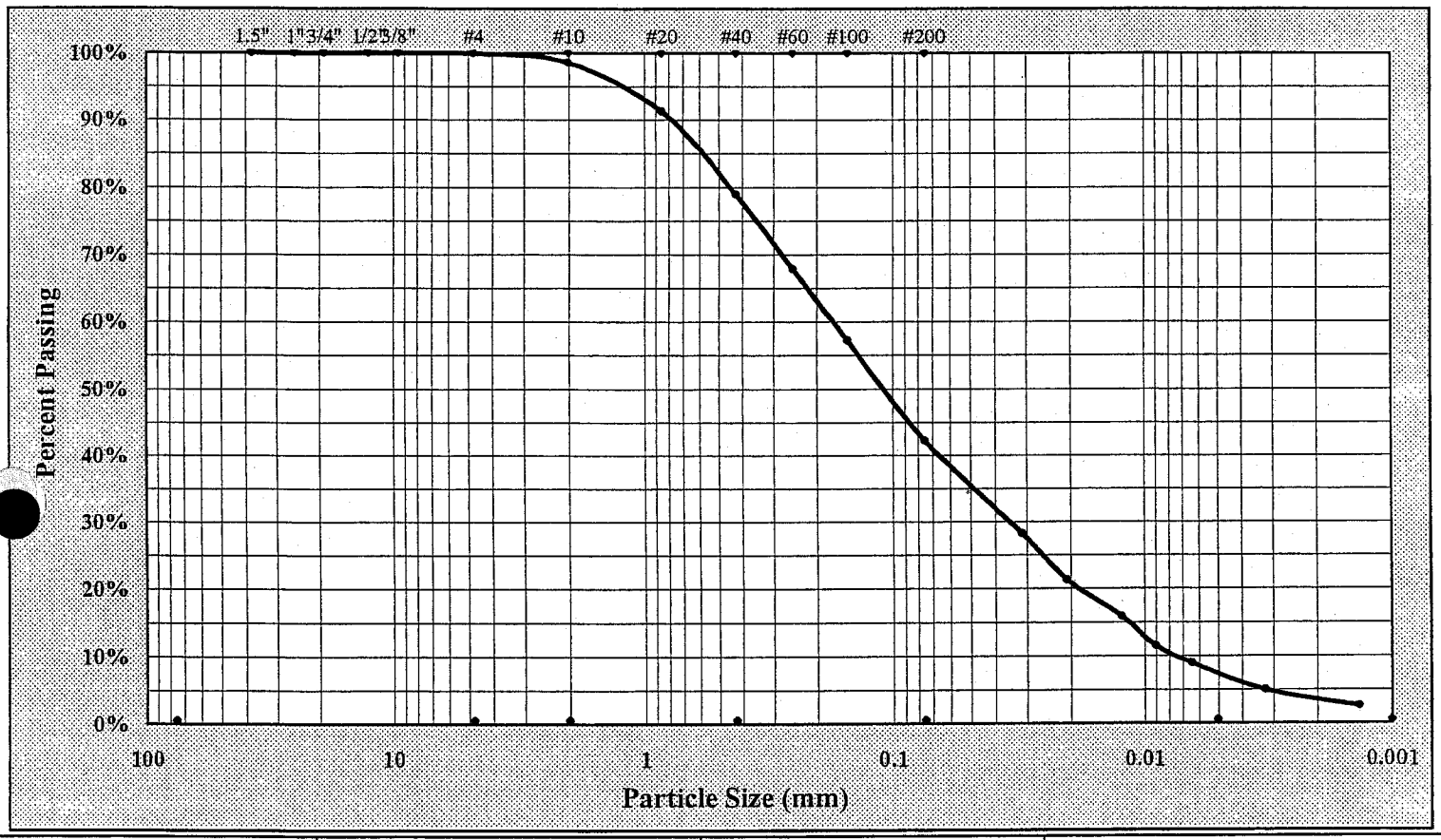
S&ME

ASTM D 422

S&ME Project #: **1054-04-955**
Project Name: **Belews Creek Steam Plant**
Client Name: Duke Energy
Client Address:

Report Date: 10/08/2004
Test Date(s): 09/30 - 10/07/204

Boring #:	BC-19	Sample #:	UD-1	Sample Date:	9/1/04
Location:	Site Borehole	Offset:	NA	Depth (ft):	27 - 29'
Sample Description:	Brown Clayey SAND (SC)				



Cobbles	< 300 mm (12") and > 75 mm (3")	Fine Sand	< 0.425 mm and > 0.075 mm (#200)
Gravel	< 75 mm and > 4.75 mm (#4)	Silt	< 0.075 and > 0.005 mm
Coarse Sand	< 4.75 mm and >2.00 mm (#10)	Clay	< 0.005 mm
Medium Sand	< 2.00 mm and > 0.425 mm (#40)	Colloids	< 0.001 mm

Maximum Particle Size	3/8"	Gravel	0.1%	Silt	34.9%
Silt & Clay (% Passing #200)	42.1%	Sand	57.8%	Clay	7.2%
Apparent Relative Density	2.665	Moisture Content		Colloids	2.5%
Liquid Limit	31	Plastic Limit	22	Plastic Index	9

Description of Sand & Gravel Particles Rounded ☐ Angular ☐ Hard & Durable ☐ Soft ☐ Weathered & Friable ☐

Mechanical Stirring Apparatus (A) Length of Dispersion Period: 1 min. Dispersing Agent: Sodium Hexametaphosphate: 40 g./ Liter

References: ASTM D 422: Particle Size Analysis of Soils ASTM D 421: Dry Preparation of Soil Samples
ASTM D 4318: Liquid Limit, Plastic Limit, & Plastic Index of Soils ASTM D 854: Specific Gravity of Soils
ASTM D 2487: Classification of Soils for Engineering Purposes (Unified Soil Classification System)

Technical Responsibility: Mal Krajan Laboratory Supervisor
(Signature) (Position)
S&ME, INC. 3109 Spring Forest Road, Raleigh, N.C. 27616 BC-19 UD-1 Classification.xls

Particle Size Analysis of Soils

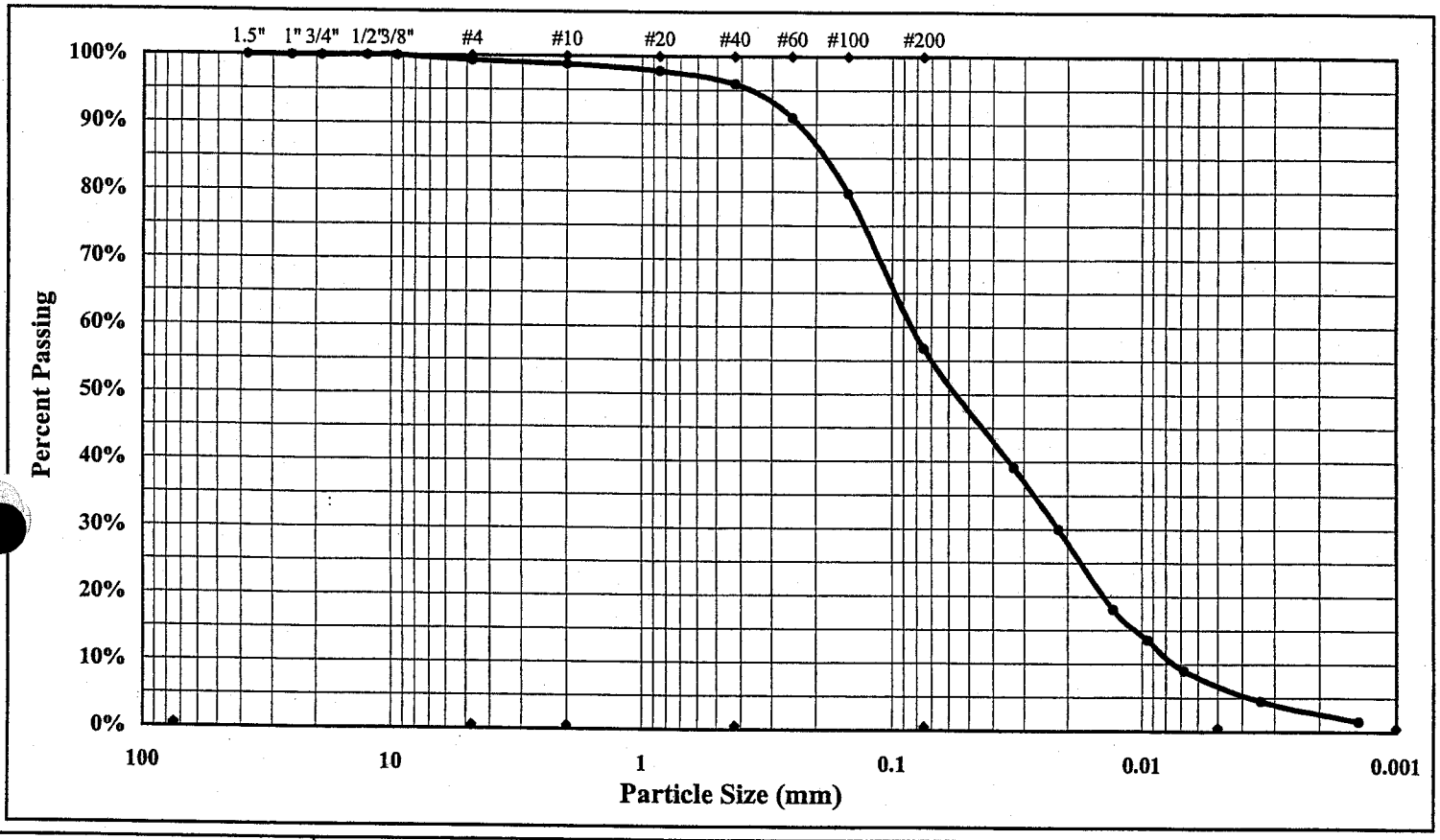
ASTM D 422

S&ME

S&ME Project #: **1264-02-576**
Project Name: **Duke Power-Belews Creek Landfill**
Client Name: Duke Energy Corp.
Client Address: 3195 Pine Hall Road, Belews Creek, NC 27009

Report Date: January 22, 2003
Test Date(s): 1/15/03-1/22/03

Boring #:	BC-10	Sample #:	SS-7	Sample Date:	NA
Location:		Offset:		Depth (ft):	NA
Sample Description:	Tan Clayey Fine Sandy SILT (ML)				



Cobbles	< 300 mm (12") and > 75 mm (3")	Fine Sand	< 0.425 mm and > 0.075 mm (#200)
Gravel	< 75 mm and > 4.75 mm (#4)	Silt	< 0.075 and > 0.005 mm
Coarse Sand	< 4.75 mm and >2.00 mm (#10)	Clay	< 0.005 mm
Medium Sand	< 2.00 mm and > 0.425 mm (#40)	Colloids	< 0.001 mm

Moisture Content		Gravel	0.7%	Liquid Limit	35
Silt & Clay (% Passing #200)	56.8%	Sand	42.5%	Plastic Limit	28
Maximum Particle Size	3/8"	Silt	50.4%	Plastic Index	7
Specific Gravity	2.666	Clay	6.4%	Colloids	1.5%

Description of Sand & Gravel Particles: Rounded ☐ Angular ☐ Hard & Durable ☐ Soft ☐ Weathered & Friable ☒

ASTM D 422: Particle Size Analysis of Soils
Dispersing Agent: Sodium Hexametaphosphate: 40 g./ Liter
Mechanical Stirring Apparatus (A) Length of Dispersion Period: 1 min.

References: ASTM D 854: Specific Gravity of Soils
ASTM D 421: Dry Preparation of Soil Samples
ASTM D 2216: Laboratory Determination of Water (Moisture) Content of Soil and Rock by Mass
ASTM D 4318: Liquid Limit, Plastic Limit, & Plastic Index of Soils
ASTM D 2487: Classification of Soils for Engineering Purposes (Unified Soil Classification System)

Technical Responsibility / Position: Mal Krajan — Laboratory Supervisor

Belews Creek FGD Residue Landfill
HELP Model Description and Results

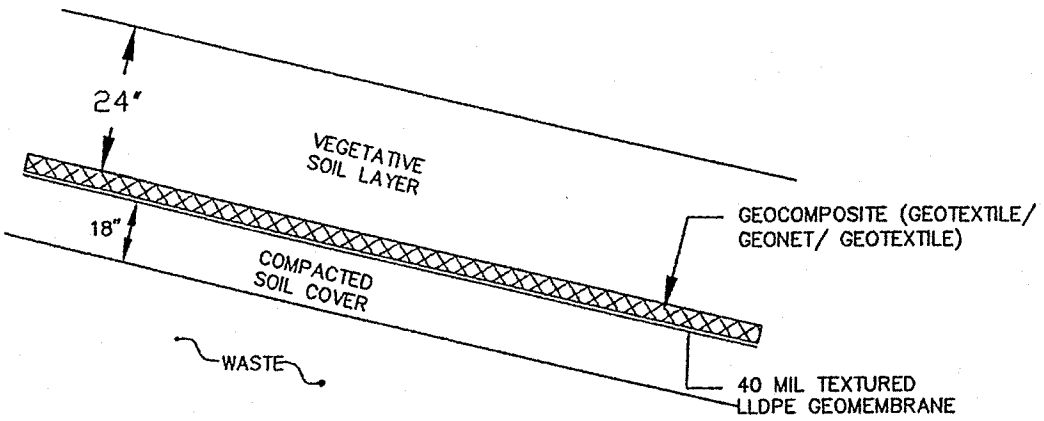
Attachment 3

Miscellaneous Information

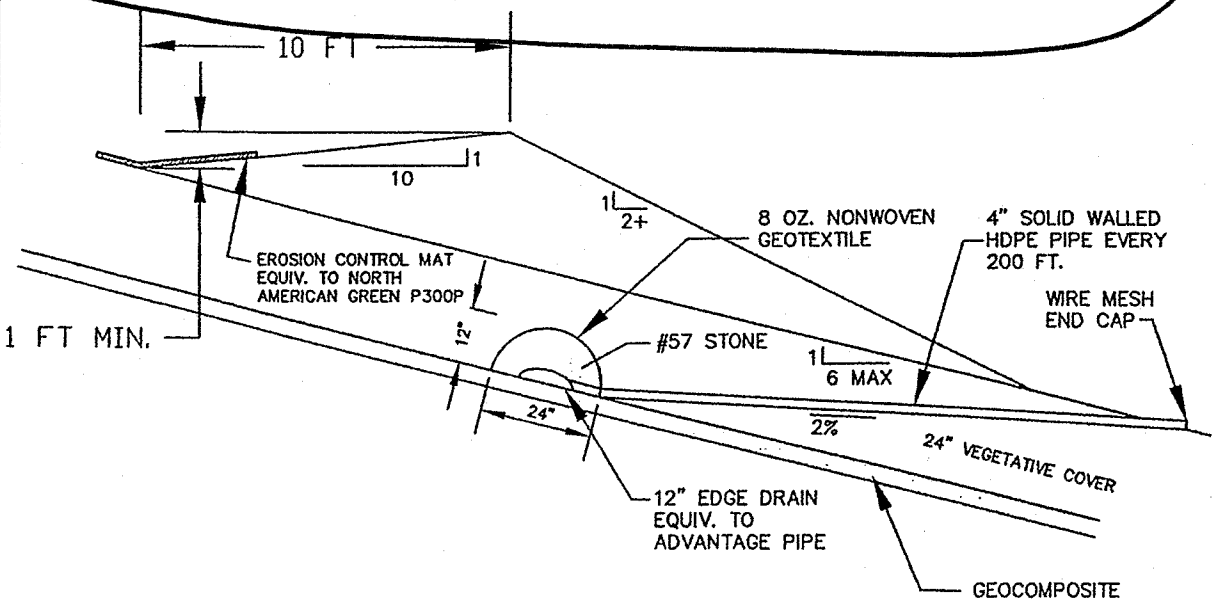
Belews Creek FGD Residue Landfill
Areas of Cells

Cell 1	5.7 ac
Cell 2	7.0 ac
Cell 3	5.6 ac
Cell 4	5.7 ac

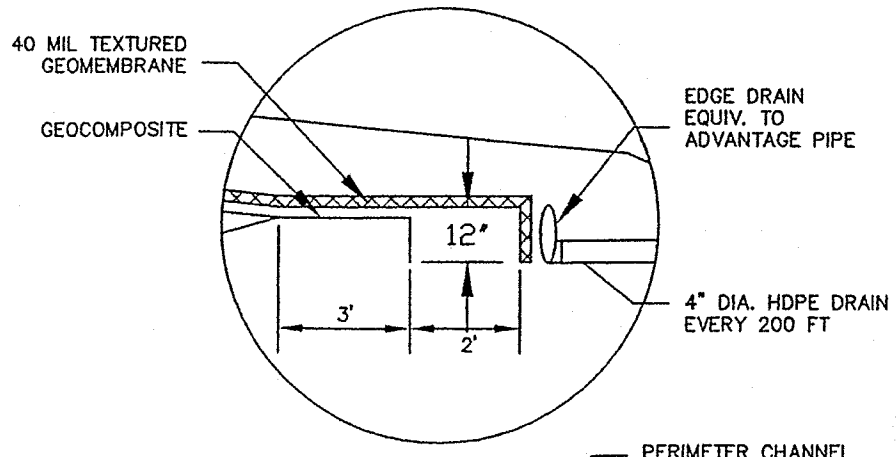
VICINITY MAP
NTS

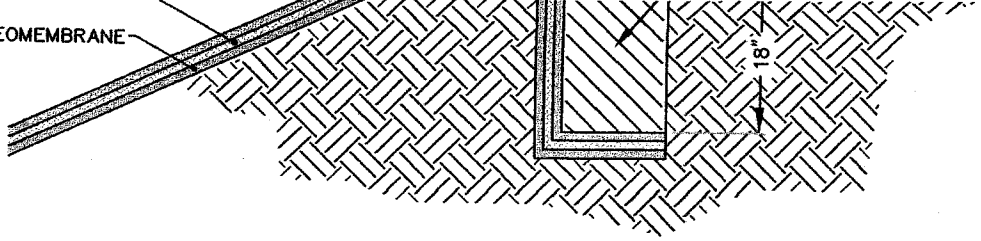


FINAL COVER SECTION
NTS

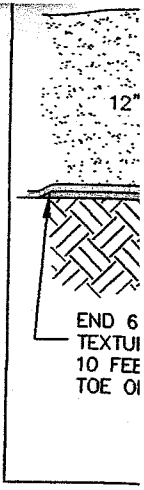


EROSION CONTROL BENCH
NTS

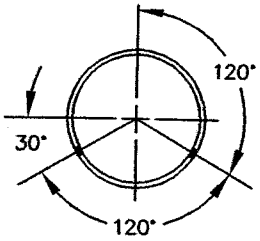




7/5 ANCHOR TRENCH
NTS

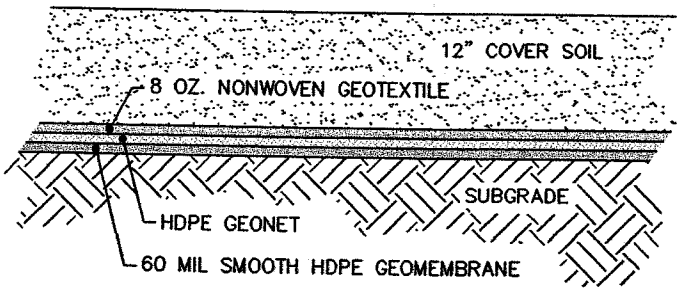


" DIAMETER HOLES



PROFILE

PERFORATIONS
NTS

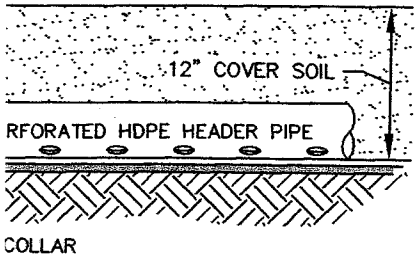


1/5 BOTTOM LINER SYSTEM
NTS

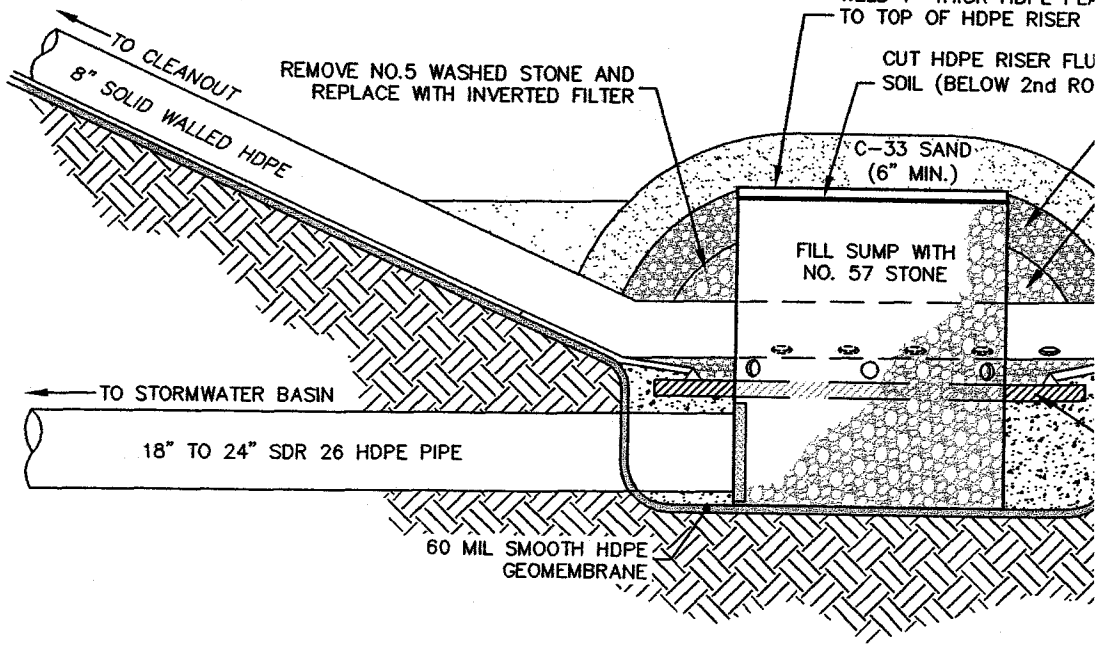
12" COVER
SOIL

9/5

'E.



COLLAR



8A INACTIVE LEACHATE S

Table 8.03b
Runoff Curve Numbers (CN)

Land Use/Cover	Average % Impervious	Hydrologic Soil Group A	B	C	D
Cultivated land					
without conservation		72	81	88	91
with conservation		62	71	78	81
* Pasture land					
poor condition		68	79	86	89
fair condition		49	69	79	84
good condition		39	61	74	80
Meadow					
good condition		30	58	71	78
Wood or forest land					
Thin stand - poor cover, no mulch		45	66	77	83
Good stand - good cover		25	55	70	77
* Open spaces, lawns, parks, golf courses, cemeteries, etc.					
good condition:					
grass cover on 75%					
or more of the area		39	61	74	80
fair condition:					
grass cover on 50					
to 75% of the area		49	69	79	84
Commercial and business					
areas (85% impervious)		89	92	94	95
Industrial districts					
(72% impervious)		81	88	91	93
Residential:[1] Development completed and vegetation established					
Average lot size	Average % Impervious				
1/8 acre or less	65	77	85	90	92
1/4 acre	38	61	75	83	87
1/3 acre	30	57	72	81	86
1/2 acre	25	54	70	80	85
1 acre	20	51	68	79	84
2 acre	15	47	66	77	81
Paved parking lots, roofs,					
driveways, etc.		98	98	98	98
Streets and roads					
paved with curbs and					
storm sewers		98	98	98	98
gravel		76	85	89	91
dirt		72	82	87	89
* Newly graded area		81	89	93	95
Residential: Development underway and no vegetation					
Lot sizes of 1/4 acre		88	93	95	97
Lot sizes of 1/2 acre		85	91	94	96
Lot sizes of 1 acre		82	90	93	95
Lot sizes of 2 acres		81	89	92	94

[1]Curve numbers are computed assuming the runoff from the house and driveway is directed toward the street.

source: USDA-SCS

CHAS. H. SELLS, INC.
Consulting Engineers, Surveyors & Photogrammetrists

Comp. By ..... Date .....
Chkd. By ..... Date .....

Project Belews Creek FGD Landfill
Subject HELP Analysis
Approx Ht of Cell 1+2 Fill

Sheet No. ..... of .....
Job No. .....

- Find the Approximate Height of Fill for Operation of Cells 1 + 2

As shown on Sheet C-4, the approximate width of Cells 1 and 2 (combined) is:

Cell 1	200' wide
Cell 2	180' wide
combined	390' wide

The operations for these cells will call for a 3H:1V slope on the side slopes. So the maximum height for the combined Cell 1 + Cell 2 is:

$$\text{mid point width} = \frac{390'}{2} = 180'$$

for 3H:1V, the maximum ht $= \frac{180'}{3} \simeq 63'$

Belews Creek FGD Residue Landfill
HELP Model Description and Results

Attachment 4

Copies of HELP Analyses

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

PRECIPITATION DATA FILE:     C:\wmm\help3-7\bcs\OP-PRCIP.D4
TEMPERATURE DATA FILE:       C:\wmm\help3-7\bcs\OP-TEMP.D7
SOLAR RADIATION DATA FILE:   C:\wmm\help3-7\bcs\OP-S-RAD.D13
EVAPOTRANSPIRATION DATA:     C:\wmm\help3-7\bcs\OP-ET.D11
SOIL AND DESIGN DATA FILE:   C:\wmm\help3-7\bcs\CEL1NGCL.D10
OUTPUT DATA FILE:            C:\WMM\HELP3-7\bcs\cel1ngcl.OUT

TIME:  14:29     DATE:   9/30/2005

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

     TITLE:  Belews Creek FGD Landfill-10'FGD Waste-Cell 1 Only-No GCL

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

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


                                LAYER  1
                                --------

                  TYPE 1 - VERTICAL PERCOLATION LAYER
                      MATERIAL TEXTURE NUMBER  44
          THICKNESS                     =    120.00   INCHES
          POROSITY                      =      0.4700 VOL/VOL
          FIELD CAPACITY                =      0.1800 VOL/VOL
          WILTING POINT                 =      0.0550 VOL/VOL
          INITIAL SOIL WATER CONTENT    =      0.2398 VOL/VOL
          EFFECTIVE SAT. HYD. COND.     =  0.253000006000E-03 CM/SEC
```

LAYER 2

TYPE 1 - VERTICAL PERCOLATION LAYER
MATERIAL TEXTURE NUMBER 62

THICKNESS	=	12.00	INCHES
POROSITY	=	0.4100	VOL/VOL
FIELD CAPACITY	=	0.2860	VOL/VOL
WILTING POINT	=	0.0980	VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.2860	VOL/VOL
EFFECTIVE SAT. HYD. COND.	=	0.360999984000E-04	CM/SEC

LAYER 3

TYPE 2 - LATERAL DRAINAGE LAYER
MATERIAL TEXTURE NUMBER 61

THICKNESS	=	0.17	INCHES
POROSITY	=	0.8500	VOL/VOL
FIELD CAPACITY	=	0.0100	VOL/VOL
WILTING POINT	=	0.0050	VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.0720	VOL/VOL
EFFECTIVE SAT. HYD. COND.	=	0.899999976000	CM/SEC
SLOPE	=	1.00	PERCENT
DRAINAGE LENGTH	=	65.0	FEET

LAYER 4

TYPE 4 - FLEXIBLE MEMBRANE LINER
MATERIAL TEXTURE NUMBER 35

THICKNESS	=	0.06	INCHES
POROSITY	=	0.0000	VOL/VOL
FIELD CAPACITY	=	0.0000	VOL/VOL
WILTING POINT	=	0.0000	VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.0000	VOL/VOL
EFFECTIVE SAT. HYD. COND.	=	0.199999996000E-12	CM/SEC
FML PINHOLE DENSITY	=	4.00	HOLES/ACRE
FML INSTALLATION DEFECTS	=	4.00	HOLES/ACRE
FML PLACEMENT QUALITY	=	3 - GOOD	

LAYER 5

TYPE 1 - VERTICAL PERCOLATION LAYER
MATERIAL TEXTURE NUMBER 60

THICKNESS	=	48.00	INCHES
POROSITY	=	0.4500	VOL/VOL
FIELD CAPACITY	=	0.1490	VOL/VOL
WILTING POINT	=	0.0520	VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.1490	VOL/VOL
EFFECTIVE SAT. HYD. COND.	=	0.123000005000E-03	CM/SEC

GENERAL DESIGN AND EVAPORATIVE ZONE DATA

NOTE: SCS RUNOFF CURVE NUMBER WAS USER-SPECIFIED.

SCS RUNOFF CURVE NUMBER	=	81.00	
FRACTION OF AREA ALLOWING RUNOFF	=	100.0	PERCENT
AREA PROJECTED ON HORIZONTAL PLANE	=	5.700	ACRES
EVAPORATIVE ZONE DEPTH	=	9.0	INCHES
INITIAL WATER IN EVAPORATIVE ZONE	=	2.236	INCHES
UPPER LIMIT OF EVAPORATIVE STORAGE	=	4.230	INCHES
LOWER LIMIT OF EVAPORATIVE STORAGE	=	0.495	INCHES
INITIAL SNOW WATER	=	0.000	INCHES
INITIAL WATER IN LAYER MATERIALS	=	39.372	INCHES
TOTAL INITIAL WATER	=	39.372	INCHES
TOTAL SUBSURFACE INFLOW	=	0.00	INCHES/YEAR

EVAPOTRANSPIRATION AND WEATHER DATA

NOTE: EVAPOTRANSPIRATION DATA WAS OBTAINED FROM
GREENSBORO NORTH CAROLINA

STATION LATITUDE	=	35.13	DEGREES
MAXIMUM LEAF AREA INDEX	=	0.00	
START OF GROWING SEASON (JULIAN DATE)	=	90	
END OF GROWING SEASON (JULIAN DATE)	=	305	
EVAPORATIVE ZONE DEPTH	=	9.0	INCHES
AVERAGE ANNUAL WIND SPEED	=	7.60	MPH
AVERAGE 1ST QUARTER RELATIVE HUMIDITY	=	66.00	%
AVERAGE 2ND QUARTER RELATIVE HUMIDITY	=	68.00	%
AVERAGE 3RD QUARTER RELATIVE HUMIDITY	=	74.00	%
AVERAGE 4TH QUARTER RELATIVE HUMIDITY	=	70.00	%

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

NORMAL MEAN MONTHLY PRECIPITATION (INCHES)

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

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

NORMAL MEAN MONTHLY TEMPERATURE (DEGREES FAHRENHEIT)

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

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

****

AVERAGE MONTHLY VALUES IN INCHES FOR YEARS 1 THROUGH 10

	JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
PRECIPITATION						
TOTALS	3.01	3.14	4.14	2.51	3.56	4.49
	5.44	5.30	3.69	2.84	2.09	3.82
STD. DEVIATIONS	2.22	1.31	1.75	1.15	1.59	3.13
	1.77	2.81	2.02	1.83	1.29	1.71
RUNOFF						
TOTALS	0.061	0.064	0.046	0.050	0.057	0.225
	0.210	0.274	0.293	0.189	0.000	0.140
STD. DEVIATIONS	0.107	0.133	0.086	0.157	0.119	0.609
	0.366	0.479	0.548	0.281	0.000	0.344
EVAPOTRANSPIRATION						
TOTALS	1.491	1.834	2.924	2.630	2.904	3.384
	4.135	3.453	2.612	2.060	1.442	1.236
STD. DEVIATIONS	0.263	0.406	0.474	0.744	0.983	2.042
	0.812	1.209	0.974	0.916	0.402	0.316
LATERAL DRAINAGE COLLECTED FROM LAYER 3						
TOTALS	0.7850	1.3908	1.3105	1.1088	1.2897	0.8868
	0.9248	0.8805	1.0361	1.0013	0.9923	0.9496
STD. DEVIATIONS	0.4025	0.9037	0.6763	0.6490	0.2747	0.3110
	0.5603	0.6084	0.8925	0.6709	0.5419	0.4210
PERCOLATION/LEAKAGE THROUGH LAYER 4						
TOTALS	0.0013	0.0033	0.0021	0.0018	0.0021	0.0015
	0.0016	0.0015	0.0018	0.0016	0.0016	0.0016
STD. DEVIATIONS	0.0005	0.0048	0.0010	0.0009	0.0003	0.0004
	0.0007	0.0009	0.0014	0.0009	0.0007	0.0006
PERCOLATION/LEAKAGE THROUGH LAYER 5						
TOTALS	0.0017	0.0021	0.0030	0.0023	0.0027	0.0021
	0.0020	0.0018	0.0021	0.0019	0.0020	0.0021
STD. DEVIATIONS	0.0006	0.0012	0.0030	0.0011	0.0005	0.0005
	0.0008	0.0010	0.0015	0.0007	0.0007	0.0007

AVERAGES OF MONTHLY AVERAGED DAILY HEADS (INCHES)

DAILY AVERAGE HEAD ON TOP OF LAYER 4

AVERAGES	0.0323	0.1637	0.0589	0.0471	0.0530	0.0377
	0.0380	0.0382	0.0555	0.0412	0.0421	0.0390
STD. DEVIATIONS	0.0165	0.3463	0.0414	0.0276	0.0113	0.0132
	0.0230	0.0292	0.0626	0.0276	0.0230	0.0173

****

AVERAGE ANNUAL TOTALS & (STD. DEVIATIONS) FOR YEARS 1 THROUGH 10

	INCHES	CU. FEET	PERCENT
PRECIPITATION	44.04 (7.015)	911190.2	100.00
RUNOFF	1.607 (1.0457)	33258.13	3.650
EVAPOTRANSPIRATION	30.105 (2.7383)	622893.31	68.360
LATERAL DRAINAGE COLLECTED FROM LAYER 3	12.55613 (4.13082)	259798.906	28.51204
PERCOLATION/LEAKAGE THROUGH LAYER 4	0.02184 (0.00873)	451.976	0.04960
AVERAGE HEAD ON TOP OF LAYER 4	0.054 (0.041)		
PERCOLATION/LEAKAGE THROUGH LAYER 5	0.02589 (0.00735)	535.785	0.05880
CHANGE IN WATER STORAGE	-0.256 (2.0350)	-5295.86	-0.581

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PEAK DAILY VALUES FOR YEARS 1 THROUGH 10

	(INCHES)	(CU. FT.)
PRECIPITATION	3.73	77177.430
RUNOFF	1.741	36012.9062
DRAINAGE COLLECTED FROM LAYER 3	0.15238	3152.92725
PERCOLATION/LEAKAGE THROUGH LAYER 4	0.001888	39.05513
AVERAGE HEAD ON TOP OF LAYER 4	4.026	
MAXIMUM HEAD ON TOP OF LAYER 4	4.803	
LOCATION OF MAXIMUM HEAD IN LAYER 3 (DISTANCE FROM DRAIN)	31.0 FEET	
PERCOLATION/LEAKAGE THROUGH LAYER 5	0.000477	9.87108
SNOW WATER	1.73	35808.4609

MAXIMUM VEG. SOIL WATER (VOL/VOL)	0.4174
MINIMUM VEG. SOIL WATER (VOL/VOL)	0.0550

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

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

****

****

FINAL WATER STORAGE AT END OF YEAR 10

LAYER	(INCHES)	(VOL/VOL)
1	26.2460	0.2187
2	3.4320	0.2860
3	0.0245	0.1444
4	0.0000	0.0000
5	7.1101	0.1481
SNOW WATER	0.000	

****
****

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


PRECIPITATION DATA FILE:     C:\wmm\help3-7\bcs\OP-PRCIP.D4
TEMPERATURE DATA FILE:       C:\wmm\help3-7\bcs\OP-TEMP.D7
SOLAR RADIATION DATA FILE:   C:\wmm\help3-7\bcs\OP-S-RAD.D13
EVAPOTRANSPIRATION DATA:     C:\wmm\help3-7\bcs\OP-ET.D11
SOIL AND DESIGN DATA FILE:   C:\wmm\help3-7\bcs\C1-2NGCL.D10
OUTPUT DATA FILE:            C:\WMM\HELP3-7\bcs\C1-2CGCL.OUT


TIME:  14:36     DATE:   9/30/2005


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

     TITLE:  Belews Creek FGD Landfill-FGD Waste-Cell 1+Cell 2-No GCL

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


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


                                  LAYER  1
                                  --------

                  TYPE 1 - VERTICAL PERCOLATION LAYER
                      MATERIAL TEXTURE NUMBER  44
          THICKNESS                     =    760.00   INCHES
          POROSITY                      =      0.4700 VOL/VOL
          FIELD CAPACITY                =      0.1800 VOL/VOL
          WILTING POINT                 =      0.0550 VOL/VOL
          INITIAL SOIL WATER CONTENT    =      0.1921 VOL/VOL
          EFFECTIVE SAT. HYD. COND.     =  0.253000006000E-03 CM/SEC
```

LAYER 2

TYPE 1 - VERTICAL PERCOLATION LAYER
MATERIAL TEXTURE NUMBER 62

THICKNESS	=	12.00	INCHES
POROSITY	=	0.4100	VOL/VOL
FIELD CAPACITY	=	0.2860	VOL/VOL
WILTING POINT	=	0.0980	VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.2854	VOL/VOL
EFFECTIVE SAT. HYD. COND.	=	0.360999984000E-04	CM/SEC

LAYER 3

TYPE 2 - LATERAL DRAINAGE LAYER
MATERIAL TEXTURE NUMBER 61

THICKNESS	=	0.17	INCHES
POROSITY	=	0.8500	VOL/VOL
FIELD CAPACITY	=	0.0100	VOL/VOL
WILTING POINT	=	0.0050	VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.0104	VOL/VOL
EFFECTIVE SAT. HYD. COND.	=	0.899999976000	CM/SEC
SLOPE	=	1.00	PERCENT
DRAINAGE LENGTH	=	65.0	FEET

LAYER 4

TYPE 4 - FLEXIBLE MEMBRANE LINER
MATERIAL TEXTURE NUMBER 35

THICKNESS	=	0.06	INCHES
POROSITY	=	0.0000	VOL/VOL
FIELD CAPACITY	=	0.0000	VOL/VOL
WILTING POINT	=	0.0000	VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.0000	VOL/VOL
EFFECTIVE SAT. HYD. COND.	=	0.199999996000E-12	CM/SEC
FML PINHOLE DENSITY	=	4.00	HOLES/ACRE
FML INSTALLATION DEFECTS	=	4.00	HOLES/ACRE
FML PLACEMENT QUALITY	=	3 - GOOD	

LAYER 5

TYPE 1 - VERTICAL PERCOLATION LAYER
MATERIAL TEXTURE NUMBER 60

THICKNESS	=	48.00	INCHES
POROSITY	=	0.4500	VOL/VOL
FIELD CAPACITY	=	0.1490	VOL/VOL
WILTING POINT	=	0.0520	VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.1490	VOL/VOL
EFFECTIVE SAT. HYD. COND.	=	0.123000005000E-03	CM/SEC

GENERAL DESIGN AND EVAPORATIVE ZONE DATA

NOTE: SCS RUNOFF CURVE NUMBER WAS USER-SPECIFIED.

SCS RUNOFF CURVE NUMBER	=	61.00	
FRACTION OF AREA ALLOWING RUNOFF	=	100.0	PERCENT
AREA PROJECTED ON HORIZONTAL PLANE	=	12.700	ACRES
EVAPORATIVE ZONE DEPTH	=	9.0	INCHES
INITIAL WATER IN EVAPORATIVE ZONE	=	2.236	INCHES
UPPER LIMIT OF EVAPORATIVE STORAGE	=	4.230	INCHES
LOWER LIMIT OF EVAPORATIVE STORAGE	=	0.495	INCHES
INITIAL SNOW WATER	=	0.000	INCHES
INITIAL WATER IN LAYER MATERIALS	=	156.555	INCHES
TOTAL INITIAL WATER	=	156.555	INCHES
TOTAL SUBSURFACE INFLOW	=	0.00	INCHES/YEAR

EVAPOTRANSPIRATION AND WEATHER DATA

NOTE: EVAPOTRANSPIRATION DATA WAS OBTAINED FROM
GREENSBORO NORTH CAROLINA

STATION LATITUDE	=	35.13	DEGREES
MAXIMUM LEAF AREA INDEX	=	0.00	
START OF GROWING SEASON (JULIAN DATE)	=	90	
END OF GROWING SEASON (JULIAN DATE)	=	305	
EVAPORATIVE ZONE DEPTH	=	9.0	INCHES
AVERAGE ANNUAL WIND SPEED	=	7.60	MPH
AVERAGE 1ST QUARTER RELATIVE HUMIDITY	=	66.00	%
AVERAGE 2ND QUARTER RELATIVE HUMIDITY	=	68.00	%
AVERAGE 3RD QUARTER RELATIVE HUMIDITY	=	74.00	%
AVERAGE 4TH QUARTER RELATIVE HUMIDITY	=	70.00	%

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

NORMAL MEAN MONTHLY PRECIPITATION (INCHES)

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

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

NORMAL MEAN MONTHLY TEMPERATURE (DEGREES FAHRENHEIT)

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

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

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AVERAGE MONTHLY VALUES IN INCHES FOR YEARS 1 THROUGH 10

	JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
PRECIPITATION						
TOTALS	3.01	3.14	4.14	2.51	3.56	4.49
	5.44	5.30	3.69	2.84	2.09	3.82
STD. DEVIATIONS	2.22	1.31	1.75	1.15	1.59	3.13
	1.77	2.81	2.02	1.83	1.29	1.71
RUNOFF						
TOTALS	0.000	0.012	0.002	0.018	0.003	0.119
	0.049	0.068	0.139	0.049	0.000	0.045
STD. DEVIATIONS	0.000	0.037	0.008	0.057	0.007	0.372
	0.100	0.150	0.356	0.076	0.000	0.136
EVAPOTRANSPIRATION						
TOTALS	1.490	1.833	2.921	2.637	2.897	3.386
	4.137	3.455	2.596	2.054	1.453	1.236
STD. DEVIATIONS	0.265	0.408	0.472	0.739	0.987	2.041
	0.816	1.214	0.987	0.911	0.401	0.316
LATERAL DRAINAGE COLLECTED FROM LAYER 3						
TOTALS	0.6980	0.6161	0.8376	0.6935	1.1387	0.9931
	0.9308	0.7348	0.8731	0.8522	0.9710	1.1732
STD. DEVIATIONS	0.3968	0.4823	0.5653	0.5364	0.5861	0.5357
	0.5371	0.3320	0.3442	0.5434	0.5299	0.5795
PERCOLATION/LEAKAGE THROUGH LAYER 4						
TOTALS	0.0012	0.0011	0.0014	0.0012	0.0018	0.0016
	0.0015	0.0013	0.0015	0.0014	0.0016	0.0019
STD. DEVIATIONS	0.0006	0.0007	0.0008	0.0008	0.0008	0.0007
	0.0008	0.0005	0.0005	0.0008	0.0008	0.0008
PERCOLATION/LEAKAGE THROUGH LAYER 5						
TOTALS	0.0015	0.0014	0.0016	0.0013	0.0020	0.0017
	0.0017	0.0017	0.0019	0.0016	0.0020	0.0020
STD. DEVIATIONS	0.0007	0.0010	0.0010	0.0009	0.0009	0.0006
	0.0008	0.0005	0.0008	0.0008	0.0009	0.0008

AVERAGES OF MONTHLY AVERAGED DAILY HEADS (INCHES)

DAILY AVERAGE HEAD ON TOP OF LAYER 4

AVERAGES	0.0287	0.0278	0.0344	0.0295	0.0468	0.0422
	0.0383	0.0302	0.0371	0.0350	0.0412	0.0482
STD. DEVIATIONS	0.0163	0.0216	0.0232	0.0228	0.0241	0.0228
	0.0221	0.0136	0.0146	0.0223	0.0225	0.0238

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AVERAGE ANNUAL TOTALS & (STD. DEVIATIONS) FOR YEARS 1 THROUGH 10

	INCHES	CU. FEET	PERCENT
PRECIPITATION	44.04 (7.015)	2030195.9	100.00
RUNOFF	0.505 (0.4503)	23262.19	1.146
EVAPOTRANSPIRATION	30.094 (2.7539)	1387375.37	68.337
LATERAL DRAINAGE COLLECTED FROM LAYER 3	10.51206 (4.17207)	484616.562	23.87043
PERCOLATION/LEAKAGE THROUGH LAYER 4	0.01755 (0.00618)	809.239	0.03986
AVERAGE HEAD ON TOP OF LAYER 4	0.037 (0.015)		
PERCOLATION/LEAKAGE THROUGH LAYER 5	0.02058 (0.00639)	948.686	0.04673
CHANGE IN WATER STORAGE	2.907 (7.0438)	133992.91	6.600

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PEAK DAILY VALUES FOR YEARS 1 THROUGH 10

	(INCHES)	(CU. FT.)
PRECIPITATION	3.73	171956.734
RUNOFF	1.162	53582.1797
DRAINAGE COLLECTED FROM LAYER 3	0.08189	3775.31372
PERCOLATION/LEAKAGE THROUGH LAYER 4	0.000113	5.21799
AVERAGE HEAD ON TOP OF LAYER 4	0.104	
MAXIMUM HEAD ON TOP OF LAYER 4	0.193	
LOCATION OF MAXIMUM HEAD IN LAYER 3 (DISTANCE FROM DRAIN)	4.7 FEET	
PERCOLATION/LEAKAGE THROUGH LAYER 5	0.000272	12.54381
SNOW WATER	1.73	79783.7734

MAXIMUM VEG. SOIL WATER (VOL/VOL)	0.4628
MINIMUM VEG. SOIL WATER (VOL/VOL)	0.0550

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

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

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FINAL WATER STORAGE AT END OF YEAR 10

LAYER	(INCHES)	(VOL/VOL)
1	175.0183	0.2303
2	3.4320	0.2860
3	0.0484	0.2844
4	0.0000	0.0000
5	7.1215	0.1484
SNOW WATER	0.000	

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**                                                                  **
**              HYDROLOGIC EVALUATION OF LANDFILL PERFORMANCE        **
**                HELP MODEL VERSION 3.07  (1 NOVEMBER 1997)        **
**                  DEVELOPED BY ENVIRONMENTAL LABORATORY           **
**                    USAE WATERWAYS EXPERIMENT STATION             **
**             FOR USEPA RISK REDUCTION ENGINEERING LABORATORY       **
**                                                                  **
**                                                                  **
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PRECIPITATION DATA FILE:     C:\wmm\help3-7\bcs\OP-PRCIP.D4
TEMPERATURE DATA FILE:       C:\wmm\help3-7\bcs\OP-TEMP.D7
SOLAR RADIATION DATA FILE:   C:\wmm\help3-7\bcs\OP-S-RAD.D13
EVAPOTRANSPIRATION DATA:     C:\wmm\help3-7\bcs\OP-ET.D11
SOIL AND DESIGN DATA FILE:   C:\wmm\help3-7\bcs\OP-NOGCL.D10
OUTPUT DATA FILE:            C:\WMM\HELP3-7\bcs\OP-NOGCL.OUT

TIME:  16:17     DATE:   9/30/2005

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     TITLE:  BCS - Waste to 859'-All Cells-Open- 4' E-04

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


                                LAYER  1
                                --------

                  TYPE 1 - VERTICAL PERCOLATION LAYER
                      MATERIAL TEXTURE NUMBER  44
          THICKNESS                     =   1068.00   INCHES
          POROSITY                      =      0.4700 VOL/VOL
          FIELD CAPACITY                =      0.1800 VOL/VOL
          WILTING POINT                 =      0.0550 VOL/VOL
          INITIAL SOIL WATER CONTENT    =      0.1886 VOL/VOL
          EFFECTIVE SAT. HYD. COND.     = 0.253000006000E-03 CM/SEC
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LAYER 2

TYPE 1 - VERTICAL PERCOLATION LAYER
MATERIAL TEXTURE NUMBER 62

THICKNESS	=	12.00	INCHES
POROSITY	=	0.4100	VOL/VOL
FIELD CAPACITY	=	0.2860	VOL/VOL
WILTING POINT	=	0.0980	VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.2844	VOL/VOL
EFFECTIVE SAT. HYD. COND.	=	0.360999984000E-04	CM/SEC

LAYER 3

TYPE 2 - LATERAL DRAINAGE LAYER
MATERIAL TEXTURE NUMBER 61

THICKNESS	=	0.17	INCHES
POROSITY	=	0.8500	VOL/VOL
FIELD CAPACITY	=	0.0100	VOL/VOL
WILTING POINT	=	0.0050	VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.0106	VOL/VOL
EFFECTIVE SAT. HYD. COND.	=	0.899999976000	CM/SEC
SLOPE	=	1.00	PERCENT
DRAINAGE LENGTH	=	65.0	FEET

LAYER 4

TYPE 4 - FLEXIBLE MEMBRANE LINER
MATERIAL TEXTURE NUMBER 35

THICKNESS	=	0.06	INCHES
POROSITY	=	0.0000	VOL/VOL
FIELD CAPACITY	=	0.0000	VOL/VOL
WILTING POINT	=	0.0000	VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.0000	VOL/VOL
EFFECTIVE SAT. HYD. COND.	=	0.199999996000E-12	CM/SEC
FML PINHOLE DENSITY	=	4.00	HOLES/ACRE
FML INSTALLATION DEFECTS	=	4.00	HOLES/ACRE
FML PLACEMENT QUALITY	=	3 - GOOD	

LAYER 5

TYPE 1 - VERTICAL PERCOLATION LAYER
MATERIAL TEXTURE NUMBER 60

THICKNESS	=	48.00	INCHES
POROSITY	=	0.4500	VOL/VOL
FIELD CAPACITY	=	0.1490	VOL/VOL
WILTING POINT	=	0.0520	VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.1490	VOL/VOL
EFFECTIVE SAT. HYD. COND.	=	0.123000005000E-03	CM/SEC

GENERAL DESIGN AND EVAPORATIVE ZONE DATA

NOTE: SCS RUNOFF CURVE NUMBER WAS USER-SPECIFIED.

SCS RUNOFF CURVE NUMBER	=	61.00	
FRACTION OF AREA ALLOWING RUNOFF	=	100.0	PERCENT
AREA PROJECTED ON HORIZONTAL PLANE	=	24.000	ACRES
EVAPORATIVE ZONE DEPTH	=	9.0	INCHES
INITIAL WATER IN EVAPORATIVE ZONE	=	2.236	INCHES
UPPER LIMIT OF EVAPORATIVE STORAGE	=	4.230	INCHES
LOWER LIMIT OF EVAPORATIVE STORAGE	=	0.495	INCHES
INITIAL SNOW WATER	=	0.000	INCHES
INITIAL WATER IN LAYER MATERIALS	=	212.013	INCHES
TOTAL INITIAL WATER	=	212.013	INCHES
TOTAL SUBSURFACE INFLOW	=	0.00	INCHES/YEAR

EVAPOTRANSPIRATION AND WEATHER DATA

NOTE: EVAPOTRANSPIRATION DATA WAS OBTAINED FROM
GREENSBORO NORTH CAROLINA

STATION LATITUDE	=	35.13	DEGREES
MAXIMUM LEAF AREA INDEX	=	0.00	
START OF GROWING SEASON (JULIAN DATE)	=	90	
END OF GROWING SEASON (JULIAN DATE)	=	305	
EVAPORATIVE ZONE DEPTH	=	9.0	INCHES
AVERAGE ANNUAL WIND SPEED	=	7.60	MPH
AVERAGE 1ST QUARTER RELATIVE HUMIDITY	=	66.00	%
AVERAGE 2ND QUARTER RELATIVE HUMIDITY	=	68.00	%
AVERAGE 3RD QUARTER RELATIVE HUMIDITY	=	74.00	%
AVERAGE 4TH QUARTER RELATIVE HUMIDITY	=	70.00	%

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

NORMAL MEAN MONTHLY PRECIPITATION (INCHES)

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

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

NORMAL MEAN MONTHLY TEMPERATURE (DEGREES FAHRENHEIT)

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

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

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AVERAGE MONTHLY VALUES IN INCHES FOR YEARS 1 THROUGH 10

	JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
PRECIPITATION						
TOTALS	3.01	3.14	4.14	2.51	3.56	4.49
	5.44	5.30	3.69	2.84	2.09	3.82
STD. DEVIATIONS	2.22	1.31	1.75	1.15	1.59	3.13
	1.77	2.81	2.02	1.83	1.29	1.71
RUNOFF						
TOTALS	0.000	0.012	0.002	0.018	0.003	0.119
	0.049	0.068	0.139	0.049	0.000	0.045
STD. DEVIATIONS	0.000	0.037	0.008	0.057	0.007	0.372
	0.100	0.150	0.356	0.076	0.000	0.136
EVAPOTRANSPIRATION						
TOTALS	1.490	1.833	2.921	2.637	2.897	3.386
	4.137	3.455	2.596	2.054	1.453	1.236
STD. DEVIATIONS	0.265	0.408	0.472	0.739	0.987	2.041
	0.816	1.214	0.987	0.911	0.401	0.316
LATERAL DRAINAGE COLLECTED FROM LAYER 3						
TOTALS	0.5935	0.5498	0.6913	0.5357	0.9616	0.8740
	0.8390	0.6079	0.7282	0.7259	0.7934	1.0240
STD. DEVIATIONS	0.3457	0.4630	0.5267	0.4152	0.5293	0.5480
	0.5281	0.3623	0.3614	0.5395	0.4906	0.5234
PERCOLATION/LEAKAGE THROUGH LAYER 4						
TOTALS	0.0011	0.0010	0.0012	0.0010	0.0016	0.0015
	0.0014	0.0011	0.0013	0.0012	0.0013	0.0017
STD. DEVIATIONS	0.0006	0.0007	0.0008	0.0007	0.0008	0.0008
	0.0008	0.0005	0.0006	0.0008	0.0007	0.0008
PERCOLATION/LEAKAGE THROUGH LAYER 5						
TOTALS	0.0014	0.0013	0.0014	0.0012	0.0021	0.0017
	0.0016	0.0014	0.0017	0.0015	0.0017	0.0021
STD. DEVIATIONS	0.0008	0.0010	0.0009	0.0010	0.0011	0.0008
	0.0008	0.0007	0.0009	0.0009	0.0008	0.0009

AVERAGES OF MONTHLY AVERAGED DAILY HEADS (INCHES)

DAILY AVERAGE HEAD ON TOP OF LAYER 4

AVERAGES	0.0244	0.0248	0.0284	0.0227	0.0395	0.0371
	0.0345	0.0250	0.0309	0.0298	0.0337	0.0421
STD. DEVIATIONS	0.0142	0.0208	0.0216	0.0176	0.0218	0.0233
	0.0217	0.0149	0.0154	0.0222	0.0208	0.0215

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AVERAGE ANNUAL TOTALS & (STD. DEVIATIONS) FOR YEARS 1 THROUGH 10

	INCHES	CU. FEET	PERCENT
PRECIPITATION	44.04 (7.015)	3836590.7	100.00
RUNOFF	0.505 (0.4503)	43960.04	1.146
EVAPOTRANSPIRATION	30.094 (2.7539)	2621811.75	68.337
LATERAL DRAINAGE COLLECTED FROM LAYER 3	8.92462 (4.27670)	777513.062	20.26573
PERCOLATION/LEAKAGE THROUGH LAYER 4	0.01534 (0.00648)	1336.529	0.03484
AVERAGE HEAD ON TOP OF LAYER 4	0.031 (0.015)		
PERCOLATION/LEAKAGE THROUGH LAYER 5	0.01916 (0.00762)	1669.632	0.04352
CHANGE IN WATER STORAGE	4.495 (7.4178)	391636.12	10.208

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PEAK DAILY VALUES FOR YEARS 1 THROUGH 10

	(INCHES)	(CU. FT.)
PRECIPITATION	3.73	324957.594
RUNOFF	1.162	101257.6640
DRAINAGE COLLECTED FROM LAYER 3	0.07851	6840.09961
PERCOLATION/LEAKAGE THROUGH LAYER 4	0.000110	9.54949
AVERAGE HEAD ON TOP OF LAYER 4	0.100	
MAXIMUM HEAD ON TOP OF LAYER 4	0.186	
LOCATION OF MAXIMUM HEAD IN LAYER 3 (DISTANCE FROM DRAIN)	4.6 FEET	
PERCOLATION/LEAKAGE THROUGH LAYER 5	0.000263	22.87083
SNOW WATER	1.73	150772.4840

MAXIMUM VEG. SOIL WATER (VOL/VOL)	0.4628
MINIMUM VEG. SOIL WATER (VOL/VOL)	0.0550

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

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

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FINAL WATER STORAGE AT END OF YEAR 10

LAYER	(INCHES)	(VOL/VOL)
1	246.3708	0.2307
2	3.4320	0.2860
3	0.0508	0.2986
4	0.0000	0.0000
5	7.1135	0.1482
SNOW WATER	0.000	

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**                                                                  **
**              HYDROLOGIC EVALUATION OF LANDFILL PERFORMANCE        **
**                HELP MODEL VERSION 3.07  (1 NOVEMBER 1997)        **
**                  DEVELOPED BY ENVIRONMENTAL LABORATORY           **
**                    USAE WATERWAYS EXPERIMENT STATION             **
**             FOR USEPA RISK REDUCTION ENGINEERING LABORATORY       **
**                                                                  **
**                                                                  **
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PRECIPITATION DATA FILE:     C:\wmm\help3-7\bcs\OP-PRCIP.D4
TEMPERATURE DATA FILE:       C:\wmm\help3-7\bcs\OP-TEMP.D7
SOLAR RADIATION DATA FILE:   C:\wmm\help3-7\bcs\OP-S-RAD.D13
EVAPOTRANSPIRATION DATA:     C:\wmm\help3-7\bcs\CLOSED.D11
SOIL AND DESIGN DATA FILE:   C:\wmm\help3-7\bcs\CLOSED-1.D10
OUTPUT DATA FILE:            C:\WMM\HELP3-7\bcs\CLOSED-1.OUT

TIME:  16:33     DATE:   9/30/2005

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     TITLE:  Belews Creek FGD Landfill - Closed Condition-No GCL

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     NOTE:  INITIAL MOISTURE CONTENT OF THE LAYERS AND SNOW WATER
              WERE SPECIFIED BY THE USER.

                                LAYER  1
                                --------

                  TYPE 1 - VERTICAL PERCOLATION LAYER
                      MATERIAL TEXTURE NUMBER  63
          THICKNESS                     =      6.00   INCHES
          POROSITY                      =      0.4100 VOL/VOL
          FIELD CAPACITY                =      0.2800 VOL/VOL
          WILTING POINT                 =      0.0900 VOL/VOL
          INITIAL SOIL WATER CONTENT    =      0.2700 VOL/VOL
          EFFECTIVE SAT. HYD. COND.     =  0.110000001000E-03 CM/SEC
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LAYER 2

TYPE 1 - VERTICAL PERCOLATION LAYER
MATERIAL TEXTURE NUMBER 62

THICKNESS	=	18.00	INCHES
POROSITY	=	0.4100	VOL/VOL
FIELD CAPACITY	=	0.2800	VOL/VOL
WILTING POINT	=	0.0900	VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.3100	VOL/VOL
EFFECTIVE SAT. HYD. COND.	=	0.360999984000E-04	CM/SEC

LAYER 3

TYPE 2 - LATERAL DRAINAGE LAYER
MATERIAL TEXTURE NUMBER 61

THICKNESS	=	0.17	INCHES
POROSITY	=	0.8500	VOL/VOL
FIELD CAPACITY	=	0.0100	VOL/VOL
WILTING POINT	=	0.0050	VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.0100	VOL/VOL
EFFECTIVE SAT. HYD. COND.	=	0.899999976000	CM/SEC
SLOPE	=	33.00	PERCENT
DRAINAGE LENGTH	=	100.0	FEET

LAYER 4

TYPE 4 - FLEXIBLE MEMBRANE LINER
MATERIAL TEXTURE NUMBER 36

THICKNESS	=	0.04	INCHES
POROSITY	=	0.0000	VOL/VOL
FIELD CAPACITY	=	0.0000	VOL/VOL
WILTING POINT	=	0.0000	VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.0000	VOL/VOL
EFFECTIVE SAT. HYD. COND.	=	0.399999993000E-12	CM/SEC
FML PINHOLE DENSITY	=	4.00	HOLES/ACRE
FML INSTALLATION DEFECTS	=	4.00	HOLES/ACRE
FML PLACEMENT QUALITY	=	3 - GOOD	

LAYER 5

TYPE 1 - VERTICAL PERCOLATION LAYER
MATERIAL TEXTURE NUMBER 62

THICKNESS	=	18.00	INCHES
POROSITY	=	0.4100	VOL/VOL
FIELD CAPACITY	=	0.2800	VOL/VOL
WILTING POINT	=	0.0900	VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.2700	VOL/VOL
EFFECTIVE SAT. HYD. COND.	=	0.360999984000E-04	CM/SEC

LAYER 6

TYPE 1 - VERTICAL PERCOLATION LAYER
MATERIAL TEXTURE NUMBER 44

THICKNESS	=	1068.00	INCHES
POROSITY	=	0.4700	VOL/VOL
FIELD CAPACITY	=	0.1800	VOL/VOL
WILTING POINT	=	0.0550	VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.2300	VOL/VOL
EFFECTIVE SAT. HYD. COND.	=	0.253000006000E-03	CM/SEC

LAYER 7

TYPE 1 - VERTICAL PERCOLATION LAYER
MATERIAL TEXTURE NUMBER 62

THICKNESS	=	12.00	INCHES
POROSITY	=	0.4100	VOL/VOL
FIELD CAPACITY	=	0.2860	VOL/VOL
WILTING POINT	=	0.0980	VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.2900	VOL/VOL
EFFECTIVE SAT. HYD. COND.	=	0.360999984000E-04	CM/SEC

LAYER 8

TYPE 2 - LATERAL DRAINAGE LAYER
MATERIAL TEXTURE NUMBER 61

THICKNESS	=	0.17	INCHES
POROSITY	=	0.8500	VOL/VOL
FIELD CAPACITY	=	0.0100	VOL/VOL
WILTING POINT	=	0.0050	VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.1900	VOL/VOL
EFFECTIVE SAT. HYD. COND.	=	0.899999976000	CM/SEC
SLOPE	=	1.00	PERCENT
DRAINAGE LENGTH	=	65.0	FEET

LAYER 9

TYPE 4 - FLEXIBLE MEMBRANE LINER
MATERIAL TEXTURE NUMBER 35

THICKNESS	=	0.06	INCHES
POROSITY	=	0.0000	VOL/VOL
FIELD CAPACITY	=	0.0000	VOL/VOL
WILTING POINT	=	0.0000	VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.0000	VOL/VOL
EFFECTIVE SAT. HYD. COND.	=	0.199999996000E-12	CM/SEC
FML PINHOLE DENSITY	=	4.00	HOLES/ACRE
FML INSTALLATION DEFECTS	=	4.00	HOLES/ACRE
FML PLACEMENT QUALITY	=	3 - GOOD	

LAYER 10

TYPE 1 - VERTICAL PERCOLATION LAYER
MATERIAL TEXTURE NUMBER 60

THICKNESS	=	48.00	INCHES
POROSITY	=	0.4500	VOL/VOL
FIELD CAPACITY	=	0.1490	VOL/VOL
WILTING POINT	=	0.0520	VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.1500	VOL/VOL
EFFECTIVE SAT. HYD. COND.	=	0.123000005000E-03	CM/SEC

GENERAL DESIGN AND EVAPORATIVE ZONE DATA

NOTE: SCS RUNOFF CURVE NUMBER WAS USER-SPECIFIED.

SCS RUNOFF CURVE NUMBER	=	61.00	
FRACTION OF AREA ALLOWING RUNOFF	=	100.0	PERCENT
AREA PROJECTED ON HORIZONTAL PLANE	=	24.000	ACRES
EVAPORATIVE ZONE DEPTH	=	12.0	INCHES
INITIAL WATER IN EVAPORATIVE ZONE	=	3.480	INCHES
UPPER LIMIT OF EVAPORATIVE STORAGE	=	4.920	INCHES
LOWER LIMIT OF EVAPORATIVE STORAGE	=	1.080	INCHES
INITIAL SNOW WATER	=	0.000	INCHES
INITIAL WATER IN LAYER MATERIALS	=	268.414	INCHES
TOTAL INITIAL WATER	=	268.414	INCHES
TOTAL SUBSURFACE INFLOW	=	0.00	INCHES/YEAR

EVAPOTRANSPIRATION AND WEATHER DATA

NOTE: EVAPOTRANSPIRATION DATA WAS OBTAINED FROM
GREENSBORO NORTH CAROLINA

STATION LATITUDE	=	35.13	DEGREES
MAXIMUM LEAF AREA INDEX	=	2.00	
START OF GROWING SEASON (JULIAN DATE)	=	90	
END OF GROWING SEASON (JULIAN DATE)	=	305	
EVAPORATIVE ZONE DEPTH	=	12.0	INCHES
AVERAGE ANNUAL WIND SPEED	=	7.60	MPH
AVERAGE 1ST QUARTER RELATIVE HUMIDITY	=	66.00	%
AVERAGE 2ND QUARTER RELATIVE HUMIDITY	=	68.00	%
AVERAGE 3RD QUARTER RELATIVE HUMIDITY	=	74.00	%
AVERAGE 4TH QUARTER RELATIVE HUMIDITY	=	70.00	%

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

NORMAL MEAN MONTHLY PRECIPITATION (INCHES)

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

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

NORMAL MEAN MONTHLY TEMPERATURE (DEGREES FAHRENHEIT)

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

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

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AVERAGE MONTHLY VALUES IN INCHES FOR YEARS 1 THROUGH 100

	JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
PRECIPITATION						
TOTALS	3.46	3.46	4.14	3.02	3.20	3.83
	4.43	4.33	3.83	3.11	2.68	3.18
STD. DEVIATIONS	1.75	1.75	1.93	1.43	1.66	2.04
	2.01	2.19	2.50	2.02	1.61	1.98
RUNOFF						
TOTALS	0.067	0.187	0.061	0.005	0.017	0.034
	0.018	0.061	0.170	0.109	0.041	0.115
STD. DEVIATIONS	0.300	0.606	0.269	0.046	0.110	0.183
	0.127	0.216	0.487	0.315	0.289	0.398
EVAPOTRANSPIRATION						
TOTALS	1.314	1.673	2.907	3.170	3.381	3.562
	4.077	3.722	2.551	1.490	1.339	1.156
STD. DEVIATIONS	0.229	0.314	0.436	0.786	1.131	1.480
	1.463	1.248	1.083	0.485	0.251	0.193
LATERAL DRAINAGE COLLECTED FROM LAYER 3						
TOTALS	1.9470	1.8510	1.6145	0.4969	0.3494	0.1840
	0.2682	0.3763	0.6186	1.0345	1.0999	1.6163
STD. DEVIATIONS	1.6311	1.4415	1.3726	0.7771	0.7314	0.4377
	0.6013	0.7357	0.9313	1.2432	1.3075	1.3786
PERCOLATION/LEAKAGE THROUGH LAYER 4						
TOTALS	0.0001	0.0001	0.0000	0.0000	0.0000	0.0000
	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
STD. DEVIATIONS	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
LATERAL DRAINAGE COLLECTED FROM LAYER 8						
TOTALS	0.0484	0.0462	0.0508	0.0476	0.0477	0.0447
	0.0449	0.0437	0.0412	0.0416	0.0393	0.0397
STD. DEVIATIONS	0.1691	0.1727	0.1990	0.1866	0.1858	0.1728
	0.1721	0.1665	0.1561	0.1564	0.1470	0.1477
PERCOLATION/LEAKAGE THROUGH LAYER 9						

TOTALS	0.0001	0.0001	0.0001	0.0001	0.0001	0.0001
	0.0001	0.0001	0.0001	0.0001	0.0001	0.0001
STD. DEVIATIONS	0.0003	0.0003	0.0003	0.0003	0.0003	0.0003
	0.0003	0.0003	0.0003	0.0003	0.0003	0.0003

PERCOLATION/LEAKAGE THROUGH LAYER 10

TOTALS	0.0002	0.0001	0.0001	0.0002	0.0003	0.0002
	0.0002	0.0001	0.0001	0.0001	0.0001	0.0001
STD. DEVIATIONS	0.0013	0.0004	0.0005	0.0007	0.0016	0.0015
	0.0009	0.0004	0.0004	0.0004	0.0004	0.0004

AVERAGES OF MONTHLY AVERAGED DAILY HEADS (INCHES)

DAILY AVERAGE HEAD ON TOP OF LAYER 4

AVERAGES	0.0041	0.0043	0.0034	0.0011	0.0007	0.0004
	0.0006	0.0008	0.0014	0.0022	0.0024	0.0034
STD. DEVIATIONS	0.0035	0.0033	0.0029	0.0017	0.0016	0.0010
	0.0013	0.0016	0.0020	0.0026	0.0029	0.0029

DAILY AVERAGE HEAD ON TOP OF LAYER 9

AVERAGES	0.0020	0.0021	0.0021	0.0020	0.0020	0.0019
	0.0018	0.0018	0.0018	0.0017	0.0017	0.0016
STD. DEVIATIONS	0.0069	0.0078	0.0082	0.0079	0.0076	0.0073
	0.0071	0.0068	0.0066	0.0064	0.0062	0.0061

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AVERAGE ANNUAL TOTALS & (STD. DEVIATIONS) FOR YEARS 1 THROUGH 100

	INCHES	CU. FEET	PERCENT
PRECIPITATION	42.67 (6.674)	3717393.2	100.00
RUNOFF	0.885 (1.0378)	77061.48	2.073
EVAPOTRANSPIRATION	30.342 (3.1524)	2643355.00	71.108
LATERAL DRAINAGE COLLECTED FROM LAYER 3	11.45651 (4.38844)	998091.375	26.84923
PERCOLATION/LEAKAGE THROUGH LAYER 4	0.00034 (0.00012)	29.925	0.00080
AVERAGE HEAD ON TOP OF LAYER 4	0.002 (0.001)		
LATERAL DRAINAGE COLLECTED FROM LAYER 8	0.53582 (2.02530)	46680.234	1.25572
PERCOLATION/LEAKAGE THROUGH LAYER 9	0.00107 (0.00372)	92.999	0.00250
AVERAGE HEAD ON TOP OF LAYER 9	0.002 (0.007)		
PERCOLATION/LEAKAGE THROUGH LAYER 10	0.00188 (0.00869)	163.890	0.00441
CHANGE IN WATER STORAGE	-0.551 (2.3385)	-47959.61	-1.290

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PEAK DAILY VALUES FOR YEARS 1 THROUGH 100

	(INCHES)	(CU. FT.)
PRECIPITATION	5.93	516621.594
RUNOFF	3.088	269054.0940
DRAINAGE COLLECTED FROM LAYER 3	1.22524	106742.57800
PERCOLATION/LEAKAGE THROUGH LAYER 4	0.000030	2.60906
AVERAGE HEAD ON TOP OF LAYER 4	0.081	
MAXIMUM HEAD ON TOP OF LAYER 4	0.156	
LOCATION OF MAXIMUM HEAD IN LAYER 3 (DISTANCE FROM DRAIN)	0.0 FEET	
DRAINAGE COLLECTED FROM LAYER 8	0.05382	4688.79590
PERCOLATION/LEAKAGE THROUGH LAYER 9	0.000082	7.16359
AVERAGE HEAD ON TOP OF LAYER 9	0.069	
MAXIMUM HEAD ON TOP OF LAYER 9	0.130	
LOCATION OF MAXIMUM HEAD IN LAYER 8 (DISTANCE FROM DRAIN)	3.6 FEET	
PERCOLATION/LEAKAGE THROUGH LAYER 10	0.000545	47.50722
SNOW WATER	4.65	405084.9060

MAXIMUM VEG. SOIL WATER (VOL/VOL)	0.4050
MINIMUM VEG. SOIL WATER (VOL/VOL)	0.0900

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

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

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FINAL WATER STORAGE AT END OF YEAR 100

LAYER	(INCHES)	(VOL/VOL)
1	1.0267	0.1711
2	4.8584	0.2699
3	0.0017	0.0101
4	0.0000	0.0000
5	4.8166	0.2676
6	192.2400	0.1800
7	3.3001	0.2750
8	0.0017	0.0100
9	0.0000	0.0000
10	7.1186	0.1483
SNOW WATER	0.000	

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Leachate Collection and Removal Pipe

- LCR Pipe Spacing
- LCR Pipe Flow Calculations
- LCR Pipe Stresses

CHAS. H. SELLS, INC.
Consulting Engineers, Surveyors & Photogrammetrists

Comp. By DEB Date 8/22/05 | Project Belews Creek FGD Landfill | Sheet No. 1 of 1
Chkd. By WMM Date 9/8/05 | Subject LCRS Pipe Spacing | Job No. 046119

Objective: Determine spacing for LCRS pipe across cell floor.

Flow from 25 yr Storm Event

25 yr, 24 hr precipitation = 5.6" (Ref: Fig. 8.03k)

Runoff Depth = 4.4" for CN = 90

$$q = 4.4" \left(\frac{1\,ft}{12"}\right)\left(43{,}560\ ft^2/ac\right) = 15{,}972\ ft^3/ac/day$$

$$= 119{,}470\ gal/ac/day$$

$$= \underline{4.24 \times 10^{-6}\ ft/sec}$$

Pipe Spacing Flow Model

$$L = \frac{2\,h_{max}}{\sqrt{C}\left[\frac{(\tan(\phi))^2}{C} + 1 - \frac{\tan\phi}{C}\left(\sqrt{(\tan(\phi))^2 + C}\right)\right]}$$

L = pipe spacing, ft

h_{max} = max head on liner = 1 ft

$$C = \frac{q}{k} = \frac{\text{infiltration rate}}{\text{hydraulic conductivity}} = \frac{4.24 \times 10^{-6}\ ft/sec}{0.005\ ft/sec} = 0.00085$$

ϕ = avg. slope of liner = 1.33% = 0.76°

$$L = \frac{2(1\,ft)}{\sqrt{0.00085}\left[\frac{(\tan(0.76))^2}{0.00085} + 1 - \frac{\tan 0.76}{0.00085}\left(\sqrt{(\tan(0.76))^2 + 0.00085}\right)\right]}$$

$$= \underline{97\ ft}$$

Use $\boxed{L = 75\ ft}$ FS = $\frac{97}{75} = 1.3$

Belews Creek FGD Residue Landfill
LCRS Pipe - Flow Calculations

Job No.	**046119**		
Comp. By	**WMM**	**Date**	**9/30/2005**
Chkd. By	**DEB**	**Date**	**9/30/05**
Sheet	**1**	**of**	**5**

Objective: Verify flow capability of 6" and 8" diameter pipes used in LCRS
Check flow of 24 hour - 25 year storm

References: 1) *Elements of Urban Stormwater Design, Malcolm, NCSU.*

2) Performance Pipe - Pipe Dimesnion Table

Check flow capacity of 6" dia. pipe

q from 24 hr, 25 yr storm = 119,470 gal/ac/day

pipe spacing of LCRS piping = 75 ft

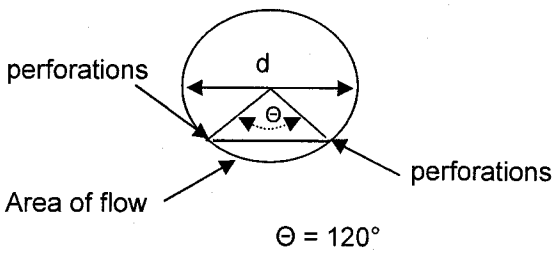
maximum length of 6" collector pipe = 310 ft

area that contributes to pipe = 23250 ft^2
= 0.53 ac

q_{max} = (q from 24 hr, 25 yr storm) x area that contributes to pipe
q_{max} = 0.0987 ft^3/sec

compute $Q_{available}$ in 6" dia. pipe (Manning)

$$Q = \frac{1.49}{n} A R^{2/3} S^{1/2}$$



Calculate flow in lower region of pipe:

Calculate area of flow:

$$A = \frac{d^2}{8} [\pi\Theta/180 - \sin\Theta]$$

outside diameter pipe = 6.625 in Reference 2
wall thickness = 0.39 in Reference 2

inside diameter, d = 5.845 in
= 0.487 ft^2

Θ = 120 degrees

A = 0.0364 ft^2

Calculate hydraulic radius:

$$R = \frac{45d}{\pi\Theta}\left[\pi\Theta/180 - \sin\Theta\right]$$

$$R = 0.0714 \text{ ft}$$

Calculate available flow capacity in lower portion of pipe"

$$Q_{available} = \frac{1.49}{n} A R^{2/3} S^{1/2}$$

$n = 0.009$

$S = 0.01$ ft/ft

$Q_{available} = 0.10$ ft^3/sec

Check against actual flow:

$q_{max} = 0.0987$ ft^3/sec

$Q_{available} > q_{max}$ **OK**

The 6" pipe has adequate flow capacity

Belews Creek FGD Residue Landfill
LCRS Pipe - Flow Calculations

Job No.	**046119**		
Comp. By	**WMM**	**Date**	**9/30/2005**
Chkd. By	DEB	**Date**	9/30/05
Sheet	3	**of**	5

Check flow capacity of 8" dia. pipe

Cell 2 has largest area	=	7.0	ac	
q_{max}, from 24 hr, 25	=	119,470	gal/ac/day	
q_{max}	=	1.29	ft^3/sec	
S, slope of pipe	=	0.015	ft/ft	

for pipes flowing just full:

$$D = 16\,[Qn/s^{1/2}]^{3/8}$$ Reference 1, Eqn I-9

solving for the available flow capacity, Qa:

$$Qa = \frac{D^{8/3}\, s^{1/2}}{16^{8/3} n}$$

D	=	OD - 2* wall thickness		
tw	=	0.507	in	Reference 2
OD	=	8.625	in	Reference 2
D	=	7.611	in	
n	=	0.009		
Qa	=	1.88	ft^3/sec	

Check against q_{max} from 24 hour, 25 year storm:

available flow capacity, Qa:	=	1.88	ft^3/sec	
q_{max}	=	1.29	ft^3/sec	
the available flow capacity, Qa	>	q_{max}		**OK**

The 8" pipe has adequate flow capacity

Check the Hole Spacing and Hole Diameter:

Number of holes per foot of pipe = 4

diameter of holes = 0.5 in

The flow for each hole can be calculated by the orifice equation:

$Q = Cd\ A\ (2gH)^{1/2}$

where

Cd = orifice coefficient
A = area of hole
g = gravity
H = head
=

Cd = 0.6
area of 0.5 in hole = 0.00136 ft^2
g = 32.2 ft/sec^2
H = 1 ft

Q = 6.57E-03 ft^3/sec

Calculate the flow per foot of pipe:

Flow per foot of pipe = 6.57E-03 ft^3/sec x 4 holes/ft
= 2.63E-02 ft^3/sec/foot of pipe length

In Cell 2, if 1 foot of head occurred, there would be 170 ft of pipe experiencing the head

Calculate the flow into the pipe (through the holes) and compare to the required flow (from the 24 hr, 25 year storm):

Flow per foot of pipe = 2.63E-02 ft^3/sec/foot of pipe length

Total flow capacity of pipe experiencing head = 4.46 ft^3/sec

q_{max}, from 24 hr, 25 = 1.29 ft^3/sec **OK**

Belews Creek FGD Residue Landfill
LCRS Pipe - Flow Calculations

Job No.	**046119**		
Comp. By	**WMM**	**Date**	**9/30/2005**
Chkd. By	DEB	**Date**	9/30/05
Sheet	5	**of**	5

Check the Hole Spacing and Hole Diameter for All Cells:

Use the calculation process to evaluate the flow capacity for all Cells.

q_{max}, from 24 hr, 25 yr storm = 119,470 gal/ac/day

Cell	Area (ac)	Qreq (ft^3/sec) (1)	Length of pipe w/1' head (ft)	Flow into holes (ft^3/sec) (2)	Sufficient Flow Capacity Available (1) < (2) ?
1	5.7	1.05	130	3.41	**OK**
2	7	1.29	170	4.46	**OK**
3	5.6	1.04	90	2.36	**OK**
4	5.7	1.05	92	2.42	**OK**

Flow capacity provided by hole diameters and spacing is sufficient.

Reference 1

Application of Manning Equation

The Manning equation can be stated as:

$$Q = \frac{1.486}{n} A R^{2/3} S^{1/2} \qquad \text{(I-8)}$$

in which

Q = Discharge (cfs).

n = Manning roughness coefficient (dimensionless), an experientially determined value which is a function of the nature of the channel lining.

A = Cross-sectional area of flow (square feet), the area through which flow takes place.

R = Hydraulic radius (ft), found by dividing cross-sectional area, A (sq ft), by wetted perimeter, P (ft). Wetted perimeter is the distance along the perimeter of the cross section against which water is flowing. It does not include the free water surface.

s = Longitudinal slope of the water surface (ft fall/ft run). If flow is uniform, it is also the slope of the invert of the channel.

The constant, 1.486, is frequently rounded to 1.49, respecting that other parameters, most notably roughness, are rarely known to more than two significant figures. The constant has units associated with it. The value given here requires that the specified units be used for the values entered into the equation.

Long pipe segments operating under gravity flow are frequently sized to flow just full with the Manning equation. Usual source data are the discharge, slope and roughness coefficient. The pipe diameter for full flow is computed, and average cross-sectional velocity is often of interest. Another recurring problem is to find the capacity (discharge) of a pipe at a given slope.

Such problems may be solved with the aid of a Manning chart. (For an example, see AISI manual for corrugated-metal pipe and the ACPA manual for concrete pipe.) However, since the advent of electronic calculators, it may be more efficient to make direct calculations.

For <u>circular pipes flowing just full</u>, the Manning equation can be reformulated as:

$$D = 16\left[\frac{Q\,n}{\sqrt{s}}\right]^{3/8} \qquad \text{(I-9)}$$

where:

D = Theoretical pipe diameter (in) for just-full flow.

Q = Discharge (cfs).

n = Manning roughness coefficient (dimensionless).

s = Longitudinal slope (ft/ft).

PERFORMANCE PIPE
A Division of Chevron Phillips Chemical Company LP

PERFORMANCE PIPE Municipal & Industrial Series/IPS Pipe Data

Pipe weights are calculated in accordance with PPI TR-7. Average inside diameter calculated using nominal OD and minimum wall plus 6% for use in estimating fluid flows. Actual ID will vary. When designing components to fit the pipe ID, refer to pipe dimensions and tolerances in applicable pipe specifications.

Pressure Ratings are for water at 73.4 °F. For other fluid and service temperature, ratings may differ. Refer to Engineering Manual for Chemical and Environmental Considerations.

Pressure Rating		100 psi DR 17.0			80 psi DR 21.0			65 psi DR 26.0			50 psi DR 32.5			
IPS Pipe Size	Nominal OD (in)	Minimum Wall (in)	Average ID (in)	Weight (lbs/ft)	Minimum Wall (in)	Average ID (in)	Weight (lbs/ft)	Minimum Wall (in)	Average ID (in)	Weight (lbs/ft)	Minimum Wall (in)	Average ID (in)	Weight (lbs/ft)	IPS Pipe Size
1 1/4"	1.660													1 1/4"
1 1/2"	1.900													1 1/2"
2"	2.375	0.140	2.078	0.43										2"
3"	3.500	0.206	3.063	0.93										3"
4"	4.500	0.265	3.938	1.54	0.214	4.046	1.26							4"
5 3/8"	5.375	0.316	4.705	2.20	0.256	4.832	1.80	0.207	4.936	1.47				5 3/8"
5"	5.563	0.327	4.870	2.35	0.265	5.001	1.93	0.214	5.109	1.57				5"
6"	6.625	0.390	5.798	3.34	0.315	5.957	2.73	0.255	6.084	2.23	0.204	6.193	1.80	6"
7 1/8"	7.125	0.419	6.237	3.86	0.339	6.406	3.16	0.274	6.544	2.58	0.219	6.661	2.08	7 1/8"
8"	8.625	0.507	7.550	5.65	0.411	7.754	4.64	0.332	7.921	3.79	0.265	8.063	3.05	8"
10"	10.750	0.632	9.410	8.78	0.512	9.665	7.21	0.413	9.874	5.87	0.331	10.048	4.75	10"
12"	12.750	0.750	11.160	12.36	0.607	11.463	10.13	0.490	11.711	8.26	0.392	11.919	6.67	12"
13 3/8"	13.375	0.787	11.707	13.61	0.637	12.025	11.15	0.514	12.285	9.09	0.412	12.502	7.35	13 3/8"
14"	14.000	0.824	12.253	14.91	0.667	12.586	12.22	0.538	12.859	9.96	0.431	13.086	8.05	14"
16"	16.000	0.941	14.005	19.46	0.762	14.385	15.96	0.615	14.696	13.01	0.492	14.957	10.50	16"
18"	18.000	1.059	15.755	24.64	0.857	16.183	20.19	0.692	16.533	16.47	0.554	16.826	13.30	18"
20"	20.000	1.176	17.507	30.41	0.952	17.982	24.93	0.769	18.370	20.34	0.615	18.696	16.41	20"
22"	22.000	1.294	19.257	36.80	1.048	19.778	30.18	0.846	20.206	24.61	0.677	20.565	19.86	22"
24"	24.000	1.412	21.007	43.81	1.143	21.577	35.91	0.923	22.043	29.30	0.738	22.435	23.62	24"
26"	26.000	1.529	22.759	51.39	1.238	23.375	42.14	1.000	23.880	34.39	0.800	24.304	27.74	26"
28"	28.000	1.647	24.508	59.62	1.333	25.174	48.86	1.077	25.717	39.88	0.862	26.173	32.19	28"
30"	30.000	1.765	26.258	68.45	1.429	26.971	56.12	1.154	27.554	45.78	0.923	28.043	36.93	30"
32"	32.000	1.882	28.010	77.86	1.524	28.769	63.84	1.231	29.390	52.10	0.985	29.912	42.04	32"
34"	34.000	2.000	29.760	87.91	1.619	30.568	72.06	1.308	31.227	58.81	1.046	31.782	47.43	34"
36"	36.000	2.118	31.510	98.57	1.714	32.366	80.78	1.385	33.064	65.94	1.108	33.651	53.20	36"
42"	42.000	2.471	36.761	134.16	2.000	37.760	109.97	1.615	38.576	89.71	1.292	39.261	72.37	42"
48"	48.000	2.824	42.013	175.23	2.286	43.154	143.65	1.846	44.086	117.18	1.477	44.869	94.56	48"
54"	54.000				2.571	48.549	181.75	2.077	49.597	148.33	1.662	50.477	119.70	54"

Performance Pipe can produce to specialized pipe dimensions. Check with your Performance Pipe contact for availability of dimensions not listed.

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Reference 2

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Pipe Stress Evaluation

- Discussion and Conclusions
- 6" Diameter Pipe Construction Loads
- 8" Diameter Pipe Construction Loads
- 6" Diameter Pipe Operational Loads
- 8" Diameter Pipe Operational Loads
- Copy of Reference Material

Discussion of Pipe Stress Evaluation and Conclusions

An evaluation of stresses was performed on the 6" diameter and 8" diameter HDPE leachate collection and removal system piping due to stress imposed from construction and operational loads. The calculations for the evaluation are presented on the following pages.

The following cases were evaluated:

Condition	Pipe Diameter	SDR	Thickness of Material Over Top of Pipe	Equipment Used for Live Load
Construction	6"	17	6"	Cat D-5 LGP Dozer
Construction	8"	17	10"	Cat D-5 LGP Dozer
Operations	6"	17	1 foot of gypsum over 6"of soil cover	20,000 lb tire load
Operations	8"	17	1 foot of gypsum over 10"of soil cover	20,000 lb tire load
Operations	6"	17	89 feet of gypsum over Operational Soil Cover	20,000 lb tire load
Operations	8"	17	89 feet of gypsum over Operational Soil Cover	20,000 lb tire load

Conclusion
Stresses in the LCRS pipe are acceptable for the conditions evaluated in the table presented above.

6" Diameter Pipe - Construction Loads
Calculate Stresses in 6" Dia. Pipe From D-5, LGP Dozer
With Minimal Cover (6")

Objective: **To determine the pipe stresses for the perforated leachate collection pipes during construction.**

References: 1) Driscopipe Material (copies attached)

Design Data: Operational Cover:

Density	125	lb/ft^3	(assumed)
Thickness	1	ft	(measured from bottom of pipe)

Waste:

Density	89	lb/ft^3	
Thickness	0	ft	(assumed)

Equipment Load : Cat D5 Low Ground Pressure Dozer

Diameter of pipe = 6 "

Calculate Stresses in Pipe:

From Reference 1:

Pt = Ps + Plive + Pi

Pt =Total Pipe Load, psi, adjusted for pipe perforations
Ps = stress, psi, from static load
Plive, = stress, psi, from live load
Pi = stress, psi, apparent external pressure due to external vacuum = 0

Load from Soil and Waste over Pipe = = unit weight of material * height of material

Load from Soil and Waste over Pipe = 125 psf
= 0.9 psi

Static Load:

Ps = Operational Cover = depth of cover to top of pipe * density of cover
Ps = 125 psf
Ps = 0.9 psi

6" Diameter Pipe - Construction Loads
Calculate Stresses in 6" Dia. Pipe From D-5, LGP Dozer
With Minimal Cover (6")

Load from Equipment: (live load increased 50% for dynamic load)

$$Plive = \frac{3 \times W \times H^3}{2 \times \pi \times R^5} \times 1.5$$

where:

W = load/tire
H = vertical distance from top of pipe to load

R = straight line distance from point of load to top of pipe
= $(H^2 + x^2)^{0.5}$
H = vertical distance from top of pipe to load
x = horizontal distance from point of load application to point directly over pipe crown

Width between tracks = 60 in (assumed)

W = 4,550 lbs
(use entire load from one track)

H = 0.5 ft
x = 0.0 ft
R = 0.5 ft

Load Factor = 1.5

Plive = 13,035 psf
= 90.52 psi

This is the load from 1 track. Add load for other track.

H = 0.5 ft
x = 5.0 ft
R = 5.0 ft

Plive = 0.1 psf
0.0 psi

Ps + Plive = 91.39 psi

6" Diameter Pipe - Construction Loads
Calculate Stresses in 6" Dia. Pipe From D-5, LGP Dozer
With Minimal Cover (6")

Due to perforations in the pipe, Pt is increased:

$$Pt = \frac{12 * Pt}{12 - Lp}$$

Where: Lp = Hp * Hd * R

Hp = Number of holes per foot of pipe
Hd = Diameter of each perforation
R = Number of rows of holes

Hp	=	2	holes per foot of pipe
Hd	=	0.5	inches
R	=	1	
Lp	=	1	
Pt	=	99.7 psi	

Wall Crushing Check for Operational Loads

S_A = Compressive Strength

$$S_A = \frac{SDR - 1}{2} * Pt$$

$$SF = \frac{1500}{S_A}$$

Pt = 99.7 psi

SDR = 17

$$S_A = \frac{SDR - 1}{2} * Pt$$

S_A = 798 psi

Factor of safety, FS:

$$SF = \frac{1500}{S_A}$$

= 1.9 **OK**

6" Diameter Pipe - Construction Loads
Calculate Stresses in 6" Dia. Pipe From D-5, LGP Dozer
With Minimal Cover (6")

Critical Collapse

Calculate the critical-collapse differential pressure, Pc

$$Pc = \frac{2.32 * E}{(SDR)^3}$$

From Chart 25, Reference 1,
S_A = 798 psi
to find :
E = Modulus of Elasticity

Since duration is short, use 1 year.

E = 20,000 psi
SDR = 17

$$Pc = \frac{2.32 * E}{(SDR)^3}$$

Pc = 9.4 psi

Calculate the critical buckling pressure at the top of the pipe, Pcb

$$Pcb = 0.8 (E' \times Pc)^{1/2}$$

from Reference 1, for angular, granular materials with moderate compaction

E' = 3000.0 psi

Pcb = 135 psi

Calculate the Factor of Safety, SF

SF = Pcb ÷ Pt
SF = 1 **OK**

This is acceptable since the loading condition is for a short duration.

6" Diameter Pipe - Construction Loads
Calculate Stresses in 6" Dia. Pipe From D-5, LGP Dozer
With Minimal Cover (6")

Soil Modulus Required to Resist Buckling

$$E' = \frac{Pcb^2}{0.64 * Pc}$$

Assume Pcb = Pt

E' = 3000psi

Apply Safety Factor (SF) to E'

SF = 2

E' min = SF x E'

E' min = 6000psi

Check Ring Deflection

$$\% \text{ Soil Strain} = \varepsilon_s = \frac{Pt * 100}{E'min}$$

$$\varepsilon_s = \frac{Pt \times 100}{E'min}$$

ε_s = 1.7%

Check against Allowable Ring Deflection

From Reference 1, Chart 27

SDR = 17

Allowable Ring Deflection (%) = 4.2% **OK**

6" diameter pipe with SDR = 17 is acceptable for construction loads from low pressure dozer.

8" Diameter Pipe - Construction Loads
Calculate Stresses in 8" Dia. Pipe From D-5, LGP Dozer
With Minimal Cover (10")

Objective: **To determine the pipe stresses for the perforated leachate collection pipes during construction.**

References: 1) Driscopipe Material (copies attached)

Design Data: Operational Cover:

Density	125 lb/ft^3	(assumed)
Thickness	1.5 ft	(measured from bottom of pipe)

Waste:

Density	89 lb/ft^3	
Thickness	0 ft	(assumed)

Equipment Load : Cat D5 Low Ground Pressure Dozer

Diameter of pipe = 8 "

Calculate Stresses in Pipe:

From Reference 1:

Pt = Ps + Plive + Pi

Pt =Total Pipe Load, psi, adjusted for pipe perforations
Ps = stress, psi, from static load
Plive, = stress, psi, from live load
Pi = stress, psi, apparent external pressure due to external vacuum = 0

Load from Soil and Waste over Pipe = = unit weight of material * height of material

Load from Soil and Waste over Pipe = 125 psf
= 0.9 psi

Static Load:

Ps = Operational Cover = depth of cover to top of pipe * density of cover
Ps = 187.5 psf
Ps = 1.3 psi

8" Diameter Pipe - Construction Loads
Calculate Stresses in 8" Dia. Pipe From D-5, LGP Dozer
With Minimal Cover (10")

Load from Equipment: (live load increased 50% for dynamic load)

$$Plive = \frac{3 \times W \times H^3}{2 \times \pi \times R^5} \times 1.5$$

where:

W = load/tire
H = vertical distance from top of pipe to load

R = straight line distance from point of load to top of pipe
= $(H^2 + x^2)^{0.5}$
H = vertical distance from top of pipe to load
x = horizontal distance from point of load application to point directly over pipe crown

Width between tracks = 60 in (assumed)

W = 4,550 lbs
(use entire load from one track)

H = 0.83 ft
x = 0.0 ft
R = 0.8 ft

Load Factor = 1.5

Plive = 4,693 psf
= 32.6 psi

This is the load from 1 track. Add load for other track.

H = 0.83 ft
x = 5.0 ft
R = 5.1 ft

Plive = 0.6 psf
0.0 psi

Ps + Plive = 33.89 psi

8" Diameter Pipe - Construction Loads
Calculate Stresses in 8" Dia. Pipe From D-5, LGP Dozer
With Minimal Cover (10")

Due to perforations in the pipe, Pt is increased:

$$Pt = \frac{12 * Pt}{12 - Lp}$$

Where: Lp = Hp * Hd * R

Hp = Number of holes per foot of pipe
Hd = Diameter of each perforation
R = Number of rows of holes

Hp	=	2	holes per foot of pipe
Hd	=	0.5	inches
R	=	1	
Lp =	=	1	
Pt	=	37 psi	

Wall Crushing Check for Operational Loads

S_A = Compressive Strength

$$S_A = \frac{SDR - 1}{2} * Pt$$

$$SF = \frac{1500}{S_A}$$

Pt = 37 psi

SDR = 17

$$S_A = \frac{SDR - 1}{2} * Pt$$

S_A = 296 psi

Factor of safety, FS:

$$SF = \frac{1500}{S_A}$$

= 5.1 **OK**

8" Diameter Pipe - Construction Loads
Calculate Stresses in 8" Dia. Pipe From D-5, LGP Dozer
With Minimal Cover (10")

Critical Collapse

Calculate the critical-collapse differential pressure, Pc

$$Pc = \frac{2.32 * E}{(SDR)^3}$$

From Chart 25, Reference 1,
S_A = 296 psi
to find :
E = Modulus of Elasticity

Since duration is short, use 1 year.

E = 35,000 psi
SDR = 17

$$Pc = \frac{2.32 * E}{(SDR)^3}$$

Pc = 16.5 psi

Calculate the critical buckling pressure at the top of the pipe, Pcb

$$Pcb = 0.8 (E' \times Pc)^{1/2}$$

from Reference 1, for angular, granular materials with moderate compaction

E' = 3000 psi

Pcb = 178 psi

Calculate the Factor of Safety, SF

SF = Pcb ÷ Pt
SF = 5 **OK**

8" Diameter Pipe - Construction Loads
Calculate Stresses in 8" Dia. Pipe From D-5, LGP Dozer With Minimal Cover (10")

Soil Modulus Required to Resist Buckling

$$E' = \frac{Pcb^2}{0.64 * Pc}$$

Assume Pcb = Pt

E' = 3000 psi

Apply Safety Factor (SF) to E'

SF = 2

E' min = SF x E'

E' min = 6000psi

Check Ring Deflection

$$\% \text{ Soil Strain} = \varepsilon_s = \frac{Pt * 100}{E'_{min}}$$

$$\varepsilon_s = \frac{Pt \times 100}{E'min}$$

ε_s = 0.6%

Check against Allowable Ring Deflection

From Reference 1, Chart 27

SDR = 17

Allowable Ring Deflection (%) = 4.2% **OK**

8" diameter pipe with SDR = 17 is acceptable for construction loads from low pressure dozer.

6" Diameter Pipe - Operational Loads
Calculate Stresses in 6" Dia. Pipe From Operational Loads
Depth of Gypsum over Operational Cover = 1 foot

Objective: **To determine the pipe stresses for the perforated leachate collection pipes during operation.**

References: 1) Driscopipe Material (copies attached)

Design Data: Operational Cover:

Density	125	lb/ft^3	(assumed)
Thickness	1	ft	(measured from bottom of pipe)

Waste:

Density	89	lb/ft^3	
Thickness	1	ft	measured from top of operational cover

Equipment Load : Tire load of 20000 lbs (single tire load)

Diameter of pipe = 6 "

Calculate Stresses in Pipe:

From Reference 1:

Pt = Ps + Plive + Pi

Pt =Total Pipe Load, psi, adjusted for pipe perforations
Ps = stress, psi, from static load
Plive, = stress, psi, from live load
Pi = stress, psi, apparent external pressure due to external vacuum = 0

Load from Soil and Waste over Pipe = = unit weight of material * height of material

Load from Soil and Waste over Pipe = 214 psf
= 1.5 psi

Static Load:

Ps = Operational Cover = depth of cover to top of pipe * density of cover
Ps = 214 psf
Ps = 1.5 psi

6" Diameter Pipe - Operational Loads
Calculate Stresses in 6" Dia. Pipe From Operational Loads
Depth of Gypsum over Operational Cover = 1 foot

Load from Equipment: (live load increased 50% for dynamic load)

$$Plive = \frac{3 \times W \times H^3}{2 \times \pi \times R^5} \times 1.5$$

where:

W = load/tire
H = vertical distance from top of pipe to load

R = straight line distance from point of load to top of pipe
= $(H^2 + x^2)^{0.5}$
H = vertical distance from top of pipe to load
x = horizontal distance from point of load application to point directly over pipe crown

Width between tracks = 60 in (assumed)

W = 20,000 lbs

H = 1.5 ft
x = 0.0 ft
R = 1.5 ft

Load Factor = 1.5

Plive = 6,366 psf
= 44.21 psi

This is the load from 1 track. Add load for other track.

H = 1.5 ft
x = 5.0 ft
R = 5.2 ft

Plive = 12.5 psf
0.1 psi

Ps + Plive = 45.78 psi

6" Diameter Pipe - Operational Loads
Calculate Stresses in 6" Dia. Pipe From Operational Loads
Depth of Gypsum over Operational Cover = 1 foot

Due to perforations in the pipe, Pt is increased:

$$Pt = \frac{12 * Pt}{12 - Lp}$$

Where: Lp = Hp * Hd * R

Hp = Number of holes per foot of pipe
Hd = Diameter of each perforation
R = Number of rows of holes

Hp	=	2	holes per foot of pipe
Hd	=	0.5	inches
R	=	1	
Lp	=	1	
Pt	=	49.9 psi	

Wall Crushing Check for Operational Loads

SA = Compressive Strength

$$SA = \frac{SDR - 1}{2} * Pt$$

$$SF = \frac{1500}{SA}$$

Pt = 49.9 psi

SDR = 17

$$SA = \frac{SDR - 1}{2} * Pt$$

SA = 400 psi

Factor of safety, FS:

$$SF = \frac{1500}{SA}$$

= 3.8 **OK**

6" Diameter Pipe - Operational Loads
Calculate Stresses in 6" Dia. Pipe From Operational Loads
Depth of Gypsum over Operational Cover = 1 foot

Critical Collapse

Calculate the critical-collapse differential pressure, Pc

$$Pc = \frac{2.32 * E}{(SDR)^3}$$

From Chart 25, Reference 1,
S_A = 400 psi
to find :
E = Modulus of Elasticity

E = 21,000 psi
SDR = 17

$$Pc = \frac{2.32 * E}{(SDR)^3}$$

Pc = 9.9 psi

Calculate the critical buckling pressure at the top of the pipe, Pcb

$$Pcb = 0.8 (E' \times Pc)^{1/2}$$

from Reference 1, for angular, granular materials with moderate compaction

E' = 3000 psi

Pcb = 138 psi

Calculate the Factor of Safety, SF

SF = Pcb ÷ Pt
SF = 3 **OK**

6" Diameter Pipe - Operational Loads
Calculate Stresses in 6" Dia. Pipe From Operational Loads
Depth of Gypsum over Operational Cover = 1 foot

Soil Modulus Required to Resist Buckling

$$E' = \frac{Pcb^2}{0.64 * Pc}$$

Assume Pcb = Pt

E' = 3000 psi

Apply Safety Factor (SF) to E'

SF = 2

E' min = SF x E'
E' min = 6000psi

Check Ring Deflection

$$\% \text{ Soil Strain} = \varepsilon_s = \frac{Pt * 100}{E'_{min}}$$

$$\varepsilon_s = \frac{Pt \times 100}{E'min}$$

ε_s = 0.8%

Check against Allowable Ring Deflection

From Reference 1, Chart 27

SDR = 17

Allowable Ring Deflection (%) = 4.2% **OK**

6" diameter pipe with SDR = 17 is acceptable for operational loads with 1 ft of gypsum over operational cover.

8" Diameter Pipe - Operational Loads
Calculate Stresses in 8" Dia. Pipe From Operational Loads
Depth of Gypsum over Operational Cover = 1 foot

Objective: **To determine the pipe stresses for the perforated leachate collection pipes during operation.**

References: 1) Driscopipe Material (copies attached)

Design Data: Operational Cover:

Density	125	lb/ft^3	(assumed)
Thickness	1	ft	(measured from bottom of pipe)

Waste:

Density	89	lb/ft^3	
Thickness	1	ft	measured from top of operational cover

Equipment Load : Tire load of 20000 lbs (single tire load)

Diameter of pipe = 8 "

Calculate Stresses in Pipe:

From Reference 1:

Pt = Ps + Plive + Pi

Pt =Total Pipe Load, psi, adjusted for pipe perforations
Ps = stress, psi, from static load
Plive, = stress, psi, from live load
Pi = stress, psi, apparent external pressure due to external vacuum = 0

Load from Soil and Waste over Pipe = = unit weight of material * height of material

Load from Soil and Waste over Pipe = 214 psf
= 1.5 psi

Static Load:

Ps = Operational Cover = depth of cover to top of pipe * density of cover
Ps = 214 psf
Ps = 1.5 psi

8" Diameter Pipe - Operational Loads
Calculate Stresses in 8" Dia. Pipe From Operational Loads
Depth of Gypsum over Operational Cover = 1 foot

Load from Equipment: (live load increased 50% for dynamic load)

$$Plive = \frac{3 \times W \times H^3}{2 \times \pi \times R^5} \times 1.5$$

where:

W = load/tire
H = vertical distance from top of pipe to load

R = straight line distance from point of load to top of pipe
= $(H^2 + x^2)^{0.5}$
H = vertical distance from top of pipe to load
x = horizontal distance from point of load application to point directly over pipe crown

Width between tracks = 60 in (assumed)

W = 20,000 lbs

H = 1.33333 ft
x = 0.0 ft
R = 1.3 ft

Load Factor = 1.5

Plive = 8,057 psf
= 55.95 psi

This is the load from 1 track. Add load for other track.

H = 1.3 ft
x = 5.0 ft
R = 5.2 ft

Plive = 9.2 psf
0.1 psi

Ps + Plive = 57.50 psi

8" Diameter Pipe - Operational Loads
Calculate Stresses in 8" Dia. Pipe From Operational Loads
Depth of Gypsum over Operational Cover = 1 foot

Due to perforations in the pipe, Pt is increased:

$$Pt = \frac{12 * Pt}{12 - Lp}$$

Where: Lp = Hp * Hd * R

Hp = Number of holes per foot of pipe
Hd = Diameter of each perforation
R = Number of rows of holes

Hp	=	2	holes per foot of pipe
Hd	=	0.5	inches
R	=	1	
Lp	=	1	
Pt	=	62.7 psi	

Wall Crushing Check for Operational Loads

SA = Compressive Strength

$$SA = \frac{SDR - 1}{2} * Pt$$

$$SF = \frac{1500}{SA}$$

Pt = 62.7 psi

SDR = 17

$$SA = \frac{SDR - 1}{2} * Pt$$

SA = 502 psi

Factor of safety, FS:

$$SF = \frac{1500}{SA}$$

= 3.0 **OK**

8" Diameter Pipe - Operational Loads
Calculate Stresses in 8" Dia. Pipe From Operational Loads
Depth of Gypsum over Operational Cover = 1 foot

Critical Collapse

Calculate the critical-collapse differential pressure, Pc

$$Pc = \frac{2.32 * E}{(SDR)^3}$$

From Chart 25, Reference 1,
S_A = 502 psi
to find :
E = Modulus of Elasticity

E = 19,000 psi
SDR = 17

$$Pc = \frac{2.32 * E}{(SDR)^3}$$

Pc = 9.0 psi

Calculate the critical buckling pressure at the top of the pipe, Pcb

$$Pcb = 0.8 (E' \times Pc)^{1/2}$$

from Reference 1, for angular, granular materials with moderate compaction

E' = 3000 psi

Pcb = 131 psi

Calculate the Factor of Safety, SF
SF = Pcb ÷ Pt
SF = 2 **OK**

8" Diameter Pipe - Operational Loads
Calculate Stresses in 8" Dia. Pipe From Operational Loads
Depth of Gypsum over Operational Cover = 1 foot

Soil Modulus Required to Resist Buckling

$$E' = \frac{Pcb^2}{0.64 * Pc}$$

Assume Pcb = Pt

$E' = 3000$ psi

Apply Safety Factor (SF) to E'

SF = 2

E' min = SF x E'
E' min = 6000psi

Check Ring Deflection

$$\% \text{ Soil Strain} = \varepsilon_s = \frac{Pt * 100}{E'_{min}}$$

$$\varepsilon_s = \frac{Pt \times 100}{E'min}$$

$\varepsilon_s = 1.0\%$

Check against Allowable Ring Deflection

From Reference 1, Chart 27

SDR = 17

Allowable Ring Deflection (%) = 4.2% **OK**

8" diameter pipe with SDR = 17 is acceptable for operational loads with 1 ft of gypsum over operational cover.

6" Diameter Pipe - Operational Loads
Calculate Stresses in 6" Dia. Pipe From Operational Loads
Depth of Gypsum over Operational Cover = 89 feet

Objective: **To determine the pipe stresses for the perforated leachate collection pipes during operation.**

References: 1) Driscopipe Material (copies attached)

Design Data: Operational Cover:

Density	125	lb/ft^3	(assumed)
Thickness	1	ft	(measured from bottom of pipe)

Waste:

Density	89	lb/ft^3	
Thickness	89	ft	measured from top of operational cover

Equipment Load : Tire load of 20000 lbs (single tire load)

Diameter of pipe = 6 "

Calculate Stresses in Pipe:

From Reference 1:

Pt = Ps + Plive + Pi

Pt =Total Pipe Load, psi, adjusted for pipe perforations
Ps = stress, psi, from static load
Plive, = stress, psi, from live load
Pi = stress, psi, apparent external pressure due to external vacuum = 0

Load from Soil and Waste over Pipe = = unit weight of material * height of material

Load from Soil and Waste over Pipe = 8046 psf
= 55.9 psi

Static Load:

Ps = Operational Cover = depth of cover to top of pipe * density of cover
Ps = 8046 psf
Ps = 55.9 psi

6" Diameter Pipe - Operational Loads
Calculate Stresses in 6" Dia. Pipe From Operational Loads
Depth of Gypsum over Operational Cover = 89 feet

Load from Equipment: (live load increased 50% for dynamic load)

$$Plive = \frac{3 \times W \times H^3}{2 \times \pi \times R^5} \times 1.5$$

where:

W = load/tire
H = vertical distance from top of pipe to load

R = straight line distance from point of load to top of pipe
= $(H^2 + x^2)^{0.5}$
H = vertical distance from top of pipe to load
x = horizontal distance from point of load application to point directly over pipe crown

Width between tracks = 60 in (assumed)

W	=	20,000	lbs
H	=	89.5	ft
x	=	0.0	ft
R	=	89.5	ft
Load Factor	=	1.5	
Plive	=	2	psf
	=	0.01	psi

This is the load from 1 track. Add load for other track.

H	=	89.5	ft
x	=	5.0	ft
R	=	89.6	ft
Plive	=	1.8	psf
		0.0	psi
Ps + Plive =		55.90	psi

6" Diameter Pipe - Operational Loads
Calculate Stresses in 6" Dia. Pipe From Operational Loads
Depth of Gypsum over Operational Cover = 89 feet

Due to perforations in the pipe, Pt is increased:

$$Pt = \frac{12 * Pt}{12 - Lp}$$

Where: Lp = Hp * Hd * R

Hp = Number of holes per foot of pipe
Hd = Diameter of each perforation
R = Number of rows of holes

Hp	=	2	holes per foot of pipe
Hd	=	0.5	inches
R	=	1	
Lp	=	1	
Pt	=	61.0 psi	

Wall Crushing Check for Operational Loads

S_A = Compressive Strength

$$S_A = \frac{SDR - 1}{2} * Pt$$

$$SF = \frac{1500}{S_A}$$

Pt = 61.0 psi

SDR = 17

$$S_A = \frac{SDR - 1}{2} * Pt$$

S_A = 488 psi

Factor of safety, FS:

$$SF = \frac{1500}{S_A}$$

= 3.1 **OK**

6" Diameter Pipe - Operational Loads
Calculate Stresses in 6" Dia. Pipe From Operational Loads
Depth of Gypsum over Operational Cover = 89 feet

Critical Collapse

Calculate the critical-collapse differential pressure, Pc

$$Pc = \frac{2.32 * E}{(SDR)^3}$$

From Chart 25, Reference 1,
S_A = 488 psi
to find :
E = Modulus of Elasticity

E = 19,000 psi
SDR = 17

$$Pc = \frac{2.32 * E}{(SDR)^3}$$

Pc = 9.0 psi

Calculate the critical buckling pressure at the top of the pipe, Pcb

$$Pcb = 0.8 (E' \times Pc)^{1/2}$$

from Reference 1, for angular, granular materials with moderate compaction

E' = 3000 psi

Pcb = 131 psi

Calculate the Factor of Safety, SF

SF = Pcb ÷ Pt
SF = 2 **OK**

6" Diameter Pipe - Operational Loads
Calculate Stresses in 6" Dia. Pipe From Operational Loads
Depth of Gypsum over Operational Cover = 89 feet

Soil Modulus Required to Resist Buckling

$$E' = \frac{Pcb^2}{0.64 * Pc}$$

Assume Pcb = Pt

$E' = 3000$ psi

Apply Safety Factor (SF) to E'

SF = 2

E' min = SF x E'
E' min = 6000psi

Check Ring Deflection

$$\% \text{ Soil Strain} = \varepsilon_s = \frac{Pt * 100}{E'min}$$

$$\varepsilon_s = \frac{Pt \times 100}{E'min}$$

$\varepsilon_s = 1.0\%$

Check against Allowable Ring Deflection

From Reference 1, Chart 27

SDR = 17

Allowable Ring Deflection (%) = 4.2% **OK**

6" diameter pipe with SDR = 17 is acceptable for operational loads with 89 ft of gypsum over operational cover.

8" Diameter Pipe - Operational Loads
Calculate Stresses in 8" Dia. Pipe From Operational Loads
Depth of Gypsum over Operational Cover = 89 feet

Objective: **To determine the pipe stresses for the perforated leachate collection pipes during operation.**

References: 1) Driscopipe Material (copies attached)

Design Data: Operational Cover:

Density	125	lb/ft^3	(assumed)
Thickness	1	ft	(measured from bottom of pipe)

Waste:

Density	89	lb/ft^3	
Thickness	89	ft	measured from top of operational cover

Equipment Load : Tire load of 20000 lbs (Single tire load)

Diameter of pipe = 8 "

Calculate Stresses in Pipe:

From Reference 1:

Pt = Ps + Plive + Pi

Pt =Total Pipe Load, psi, adjusted for pipe perforations
Ps = stress, psi, from static load
Plive, = stress, psi, from live load
Pi = stress, psi, apparent external pressure due to external vacuum = 0

Load from Soil and Waste over Pipe = = unit weight of material * height of material

Load from Soil and Waste over Pipe = 8046 psf
= 55.9 psi

Static Load:

Ps = Operational Cover = depth of cover to top of pipe * density of cover
Ps = 8046 psf
Ps = 55.9 psi

8" Diameter Pipe - Operational Loads
Calculate Stresses in 8" Dia. Pipe From Operational Loads
Depth of Gypsum over Operational Cover = 89 feet

Load from Equipment: (live load increased 50% for dynamic load)

$$Plive = \frac{3 \times W \times H^3}{2 \times \pi \times R^5} \times 1.5$$

where:

W = load/tire
H = vertical distance from top of pipe to load

R = straight line distance from point of load to top of pipe
= $(H^2 + x^2)^{0.5}$
H = vertical distance from top of pipe to load
x = horizontal distance from point of load application to point directly over pipe crown

Width between tracks = 60 in (assumed)

W = 20,000 lbs

H = 89.3333 ft
x = 0.0 ft
R = 89.3 ft

Load Factor = 1.5

Plive = 2 psf
= 0.01 psi

This is the load from 1 track. Add load for other track.

H = 89.3333 ft
x = 5.0 ft
R = 89.5 ft

Plive = 1.8 psf
0.0 psi

Ps + Plive = 55.90 psi

8" Diameter Pipe - Operational Loads
Calculate Stresses in 8" Dia. Pipe From Operational Loads
Depth of Gypsum over Operational Cover = 89 feet

Due to perforations in the pipe, Pt is increased:

$$Pt = \frac{12 * Pt}{12 - Lp}$$

Where: Lp = Hp * Hd * R

Hp = Number of holes per foot of pipe
Hd = Diameter of each perforation
R = Number of rows of holes

Hp	=	2	holes per foot of pipe
Hd	=	0.5	inches
R	=	1	
Lp	=	1	
Pt =		61.0 psi	

Wall Crushing Check for Operational Loads

S_A = Compressive Strength

$$S_A = \frac{SDR - 1}{2} * Pt$$

$$SF = \frac{1500}{S_A}$$

Pt = 61.0 psi

SDR = 17

$$S_A = \frac{SDR - 1}{2} * Pt$$

S_A = 488 psi

Factor of safety, FS:

$$SF = \frac{1500}{S_A}$$

= 3.1 **OK**

8" Diameter Pipe - Operational Loads
Calculate Stresses in 8" Dia. Pipe From Operational Loads
Depth of Gypsum over Operational Cover = 89 feet

Critical Collapse

Calculate the critical-collapse differential pressure, Pc

$$Pc = \frac{2.32 * E}{(SDR)^3}$$

From Chart 25, Reference 1,
S_A = 488 psi
to find :
E = Modulus of Elasticity

E = 19,000 psi
SDR = 17

$$Pc = \frac{2.32 * E}{(SDR)^3}$$

Pc = 9.0 psi

Calculate the critical buckling pressure at the top of the pipe, Pcb

$$Pcb = 0.8 (E' \times Pc)^{1/2}$$

from Reference 1, for angular, granular materials with moderate compaction

E' = 3000 psi

Pcb = 131 psi

Calculate the Factor of Safety, SF

SF = Pcb ÷ Pt
SF = 2 **OK**

8" Diameter Pipe - Operational Loads
Calculate Stresses in 8" Dia. Pipe From Operational Loads
Depth of Gypsum over Operational Cover = 89 feet

Soil Modulus Required to Resist Buckling

$$E' = \frac{Pcb^2}{0.64 * Pc}$$

Assume Pcb = Pt

E' = 3000 psi

Apply Safety Factor (SF) to E'

SF = 2

E' min = SF x E'
E' min = 6000psi

Check Ring Deflection

$$\% \text{ Soil Strain} = \varepsilon_s = \frac{Pt * 100}{E'_{min}}$$

$$\varepsilon_s = \frac{Pt \times 100}{E'min}$$

ε_s = 1.0%

Check against Allowable Ring Deflection

From Reference 1, Chart 27

SDR = 17

Allowable Ring Deflection (%) = 4.2% **OK**

8" diameter pipe with SDR = 17 is acceptable for operational loads with 89 ft of gypsum over operational cover.

Simplified Burial Design: A conservative estimate of the ability of Driscopipe pipelines to perform in a buried environment is found in Chart 24. It is based on a minimum 2:1 safety factor and 50 year design service life. A detailed burial design starts on page 37. The detailed design should be used for critical or marginal applications or whenever a more precise solution is desired.

Detailed Burial Design:
Design by Wall Crushing: Wall crushing would theoretically occur when the stress in a pipe wall, due to the external vertical pressure, exceeded the long-term compressive strength of the pipe material. To ensure that the Driscopipe wall is strong enough to endure the external pressure the following check should be made:

$$S_A = \frac{(SDR - 1)}{2} P_T$$

Values of E'

Based on Soil Type (ASTM D2321) and Degree of Compaction

Soil Type of Initial Backfill Embedment Material	Description	E' (psi) for Degree of Compaction (Proctor Density, %)			
		Loose	Slight (70-85%)	Moderate (85-95%)	High (95%)
I	Manufactured angular, granular materials (crushed stone or rock, broken coral, cinders, etc.)	1,000	3,000	3,000	3,000
II	Coarse grained soils with little or no fines	N.R.	1,000	2,000	3,000
III	Coarse grained soils with fines	N.R.	N.R.	1,000	2,000
IV	Fine-grained soils	N.R.	N.R.	N.R.	N.R.
V	Organic soils (peat, muck, clay, etc.)	N.R.	N.R.	N.R.	N.R.

N.R. = Not Recommended for use by ASTM D2321 for pipe wall support

Chart 24

SDR	Maximum Burial Depth, ft. in dry soil of 100 lbs/cu. ft. Soil Modulus, psi*			Maximum External Pressure psi Soil Modulus, psi*			Maximum Deflection, % after installation Soil Modulus, psi*		
	1000	2000	3000	1000	2000	3000	1000	2000	3000
32.5	25	32	37	17	22	26	1.7	0.9	0.6
26	33	45	52	23	31	36	2.3	1.2	0.8
21	46	61	71	32	42	49	3.2	1.6	1.1
19	52	69	81	36	48	56	3.6	1.8	1.2
17	61	121	181	42	84	126	4.2	2.1	1.4
15.5	56	112	168	39	78	117	3.9	2.0	1.3
13.5	49	98	147	34	68	102	3.4	1.7	1.1
11	39	78	117	27	54	81	2.7	1.4	0.9
9.3	33	68	101	23	47	70	2.3	1.2	0.8
8.3	30	61	89	21	42	62	2.1	1.1	0.7
7.3	26	52	79	18	36	55	1.8	0.9	0.6

*assumes no external loads

Where: S_A = Actual compressive stress, psi
SDR = Standard Dimension Ratio
P_T = External Pressure, psi

Safety Factor = 1500 psi ÷ S_A where 1500 psi is the Compressive Yield Strength of Driscopipe.

Design by Wall Buckling: Local wall buckling is a longitudinal wrinkling of the pipe wall. Tests of *non-pressurized* Driscopipe show that buckling and collapse do not occur when the soil envelope is in full contact with the pipe and is compacted to a dense state. However, it can be forced to occur over the long term in *non-pressurized* pipe if the total external soil pressure, P_t, is allowed to exceed the pipe-soil system's critical buckling pressure, P_{cb}. If $P_t > P_{cb}$, gradual collapse may occur over the long term. A calculated, conservative value for the critical buckling pressure may be obtained by the following approximate formula. All pipe diameters with the same SDR in the same burial situation have the same critical collapse and critical buckling endurance

$$P_{cb} = 0.8\sqrt{E' \times P_c}$$

Where:

P_t = Total vertical soil pressure at the top of the pipe, psi

P_{cb} = Critical buckling soil pressure at the top of the pipe, psi

E' = Soil modulus in psi calculated as the ratio of the vertical soil pressure to vertical soil strain at a specified density

P_C = Hydrostatic, critical-collapse differential pressure, psi

$$P_C = \frac{2E\,(t/D)^3\,(D_{MIN}/D_{MAX})^3}{(1-\mu^2)}$$

$$P_c = \frac{2.32\,E}{(SDR)^3}$$

Where: $(D_{MIN}/D_{MAX}) = .95$
μ = Poission's Ratio
μ = .45 for Driscopipe
E = stress and time dependent tensile modulus of elasticity, psi

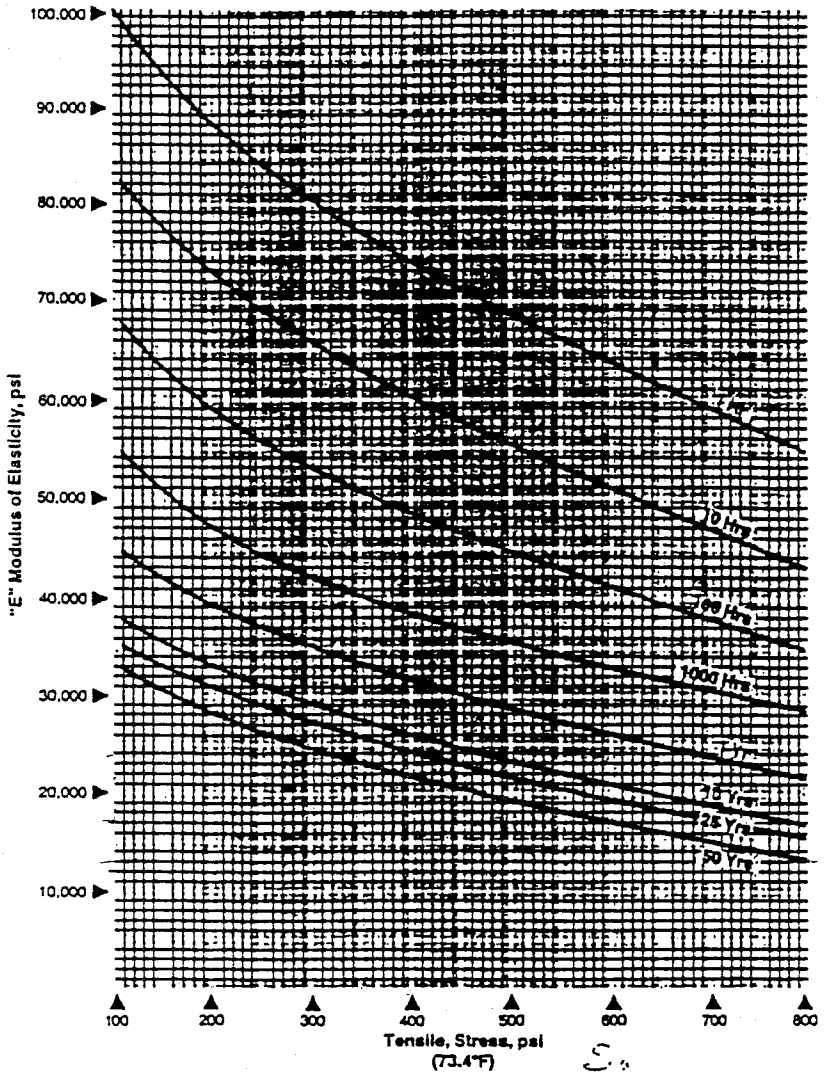
In a direct burial pressurized pipeline, the internal pressure is usually great enough to exceed the external critical-buckling soil pressure. When a pressurized line is to be shut down for a period, wall buckling should be examined.

Design by Wall Buckling Guidelines:
Although wall buckling is seldom the limiting factor in the design of a Driscopipe system, a check of non-pressurized pipelines can be made according to the following steps to insure $P_t < P_{cb}$.

1. Calculate or estimate the total soil pressure, P_t, at the top of the pipe.
2. Calculate the stress "S_A" in the pipe wall according to the formula:
$$S_A = \frac{(SDR - 1)\,P_t}{2}$$
3. Based upon the stress "S_A" and the estimated time duration of non-pressurization, use Chart 25 to find the value of the pipe's modulus of elasticity, E, in psi.

Chart 25

Time Dependent Modulus of Elasticity for Polyethylene Pipe vs. Stress Intensity (73.4°F)



NOTE: The short term modulus of elasticity of Driscopipe per ASTM D 638 is approximately 100,000 psi. Due to the cold flow (creep) characteristic of the pipe material, this modulus is dependent upon the stress intensity and the time duration of the applied stress.

4. Based upon the pipe SDR and the value of the polyethylene modulus of elasticity, E, calculate the pipe's hydrostatic, critical-collapse differential pressure, P_C:

$$P_C = \frac{2.32\,(E)}{(SDR)^3}$$

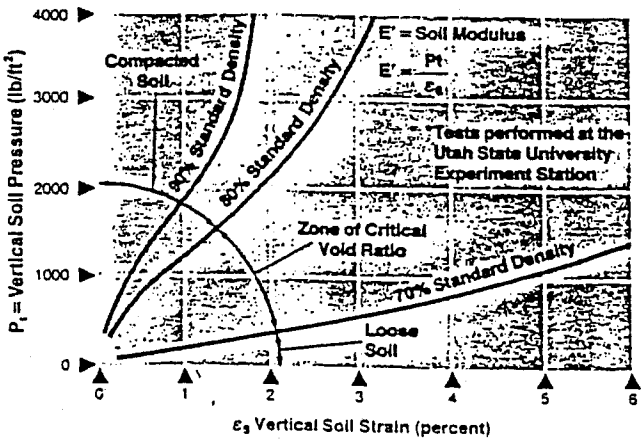
5. Calculate the soil modulus, E', by plotting the total external soil pressure, P_t, against a specified soil density to derive the soil strain as shown in the example problem on Chart 26.
6. Calculate the critical buckling pressure at the top of the pipe by the formula:

$$P_{CB} = 0.8\sqrt{E' \times P_C}$$

7. Calculate the Safety Factor: S.F. $= P_{CB} \div P_t$
   In burial applications, a safety factor of 1.0 may be considered a minimum because of the margin of safety provided by the arching action of the soil. However, Driscopipe endorses using a more conservative value approaching or exceeding a 2.0 safety factor.
8. The above procedures could be reversed to derive the minimum pipe SDR required for a given soil pressure and an estimated soil density. However, this procedure should permit the engineer to optimize the system design quickly by examining several combinations.

Chart 26

Plot of Vertical Stress-Strain Data for Typical Trench Backfill (Except Clay) from Actual Tests*



EXAMPLE
Find: E' @ 2000 PSF and 80% Density
Formula: $E' = P_t / \varepsilon_s$
Calculations: E' = 2000 PSF / .018 = 111111 PSF = 771 psi

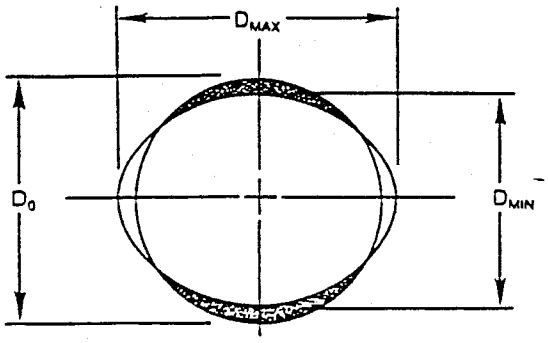
Note: The curves shown on this chart are sample curves for a granular soil. If other types of soil are used for backfill, such as clay or clay loam, curves should be developed from laboratory test data for the material used. Soil pressures greater than 4000 psf may be examined by extrapolating the slope of the curve or by generating curves by testing at those higher soil pressures. Probable error of curves is about half the distance between adjacent lines.

Design by Ring Deflection: Ring deflection is defined as the ratio of the vertical change in diameter to the original diameter. It is often expressed as a percentage. Ring deflection for buried Driscopipe is conservatively the same as (no more than) the vertical compression of the soil envelope around the pipe. Design by ring deflection matches the ability of Driscopipe to accommodate, without structural distress, the vertical compression of the soil enveloping the buried pipeline. *Design by ring deflection comprises a calculation of vertical soil strain to ensure it will be less than the allowable ring deflection of the pipe.* See Chart 27. The tabulation shows that with lower values of SDR, the allowable deflection is less. For installations which require this thicker wall to resist the external soil pressure, actual ring deflection can easily be limited to the tabular values by proper compaction of the backfill around the pipe. The recommended allowable deflection for the various SDR s are:

Chart 27

SDR	Allowable Ring Deflection
32.5	8.1%
26.0	6.5%
21.0	5.2%
19.0	4.7%
17.0	4.2%
15.5	3.9%
13.5	3.4%
11.0	2.7%

The allowable ring deflection of polyethylene pipe is a function of the allowable tangential strain in the outer surface of the pipe wall. A conservative limit of 1-1½% tangential strain in the outer surface of the pipe wall due to vertical deflection of the pipe "ring" by soil compression can be understood by comparing two pipes of the same diameter but different wall thickness.



$$\% \text{ Ring Deflection} = \left(1 - \frac{D_{MIN}}{D_0}\right) \times 100\%$$

NOTE: 5% deflection decreases flow-area by ¼%. 10% deflection decreases flow-area by 1%.

P_v: Apparent External Pressure Due To Vacuum:

3 psi × 144 = 432 psf

TOTAL EXTERNAL PRESSURE: $P_t = P_S + P_L + P_v$

P_t = 1390 psf + 99 psf + 432 psf = 1921 psf = 13.34 psi

Burial Design Guidelines: By combining the Burial Design Considerations with the Total External Soil Pressure, calculated by components, the designer can select the proper pipe SDR and specify the soil density to engineer into the pipeline the desired performance of the "pipe-soil" system. The following guidelines are presented for evaluation when designing a specific Driscopipe system. Because various parameters are available, in different situations, the guidelines may be approached in a mixed order or the equations may require mathematical rearrangement. These guidelines, along with the following notes and sample problem, should be helpful:

1. Calculate by components the total external soil pressure, P_t, at the top of the pipe.
2. Examine Short Term Wall Crushing by calculating the compressive stress in the wall of the pipe at the springline:

$$S_A = \frac{(SDR - 1)\,P_t}{2}$$

(a) If $S_A < 1500$ psi proceed to #3
(b) If $S_A > 1500$ psi consider a heavier pipe wall

3. Calculate the critical-collapse pressure, P_C, from this formula using the time dependent modulus of elasticity, E, rated at the stress level calculated above in #2 (see Chart 25).

$$P_C = \frac{2.32\,E}{(SDR)^3}$$

4. Examine Wall Buckling of the pipe-soil system. By assuming the critical buckling pressure, P_{cb}, equals the pressure at the top of the pipe. P_t (see #1), and by using the critical pressure, P_C, calculated in #3, the basic soil modulus, E', required to resist buckling can be calculated by:

$$E' = \frac{(P_{cb})^2}{.64\,(P_C)}$$

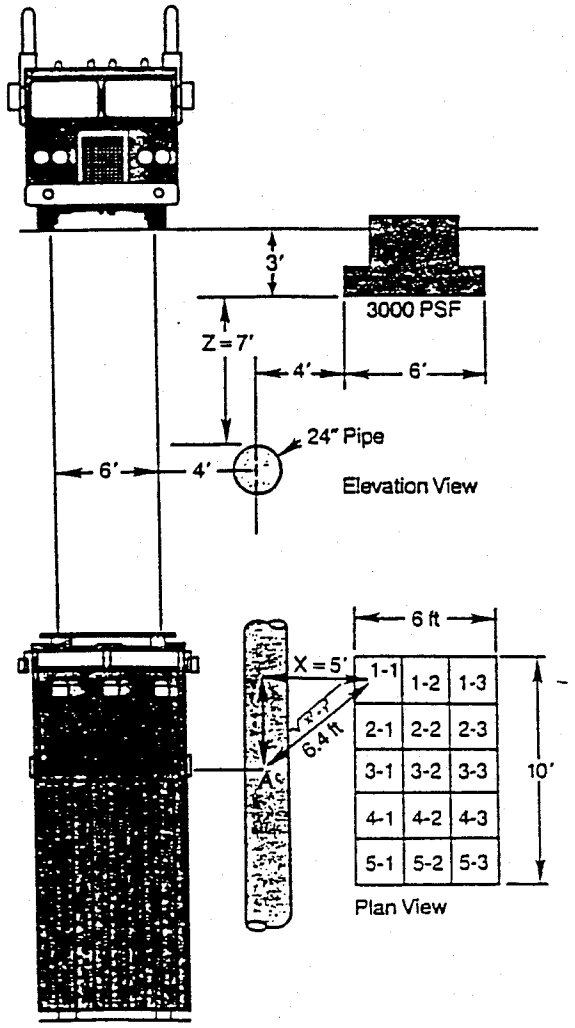
5. To safeguard against wall buckling, multiply E' by a reasonable safety factor (S.F.) equal to or greater than 2.0.

$$E'_{MIN} = (E')(S.F.)$$

6. Calculate pipe deflection based upon the principle that its deflection will be the same as the backfill surrounding the pipe under the influence of the soil pressure at the top of the pipe:

$$\%\ \text{Soil Strain} = \xi_S = \frac{P_t}{E'_{MIN}} \times 100$$

7. Examine allowable Ring Deflection for the specific SDR under consideration to insure the pipe deflection (#6) is less than the allowable deflection for that SDR (See Chart 27).
   - If the actual deflection exceeds the permissible value, increase E', the soil strength modulus, and recalculate #6. The other alternative is to consider another SDR at #1.
   - If the predicted deflection is less than the allowable value, proceed to #8.
8. Specify the primary backfill soil density matched to the soil pressure, P_t, and the calculated soil strain, ξ_S, by using soil stress/strain curves developed from field specimens or from Chart 26 on page 38.



Stormwater Management

- Stormwater Basin Detention Calculations
- Cell Riser Calculations
- Stormwater Culvert Calculations
- Outlet Protection Apron Calculations
- Permanent Diversion Channel Calculations

Note: Erosion and Sediment Control Calculations for the Belews Creek FGD Residue Landfill and the Soil Stockpile Storage Areas were submitted to NCDENR Land Quality Section. Erosion and Sediment Control Permits # Stoke 2005-007 and Stoke 2006-001 were issued as a result. Copies of these calculations can be furnished if requested.

STORMWATER BASIN DETENTION CALCULATIONS

FOR

BELEWS CREEK GYPSUM LANDFILL

Project Number: 046121

Prepared For:

DUKE ENERGY

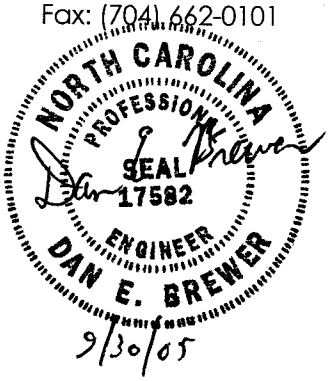
CHAS. H. SELLS, INC.

Consulting Engineers, Surveyors & Photogrammetrists

128 Overhill Drive, Suite 105, Mooresville, NC 28117

Phone: (704) 662-0100

Fax: (704) 662-0101



STORMWATER PEAK RUNOFF SUMMARY
STORMWATER BASIN TO SERVE
BELEW'S CREEK FGD GYPSUM LANDFILL

PROJECT NO. 046119

Design Storm	Runoff Into Pond Cells 1&2 (cfs)	Outflow From Pond Riser Structure (cfs)	Outflow From Pond Riser Structure (GPM)
2 year - 24 hour	46.2	0.7	314
25 year - 24 hour	96.2	1.0	449
100 year - 24 hour	118.7	2.5	1122

SET PUMP CAPACITY = 500 GPM TO MEET 25 YEAR STORM EVENT

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MASTER DESIGN STORM SUMMARY

Network Storm Collection: CMLDS-24HR

Return Event	Total Depth in	Rainfall Type	RNF ID
2	3.1200	Synthetic Curve	TypeII 24hr
25	5.7600	Synthetic Curve	TypeII 24hr
100	6.9600	Synthetic Curve	TypeII 24hr

MASTER NETWORK SUMMARY
SCS Unit Hydrograph Method

(*Node=Outfall; +Node=Diversion;)
(Trun= HYG Truncation: Blank=None; L=Left; R=Rt; LR=Left&Rt)

Node ID		Type	Return Event	HYG Vol ac-ft	Trun	Qpeak hrs	Qpeak cfs	Max WSEL ft	Max Pond Storage ac-ft
*OUT 10		JCT	2	2.728		16.4500	.72		
*OUT 10		JCT	25	5.776		19.6500	.96		
*OUT 10		JCT	100	6.965		15.8000	2.52		
POND 10	IN	POND	2	2.728		11.9500	46.24		
POND 10	IN	POND	25	5.902		11.9500	96.22		
POND 10	IN	POND	100	7.373		11.9500	118.66		
POND 10	OUT	POND	2	2.728		16.4500	.72	755.01	1.974
POND 10	OUT	POND	25	5.776		19.6500	.96	757.52	4.729
POND 10	OUT	POND	100	6.965		15.8000	2.52	758.19	5.545
SUBAREA 10		AREA	2	2.728		11.9500	46.24		
SUBAREA 10		AREA	25	5.902		11.9500	96.22		
SUBAREA 10		AREA	100	7.373		11.9500	118.66		

$.96(60)(7.481) = 430.91$ gpm

NETWORK SUMMARY -- NODES
(Trun.= HYG Truncation: Blank=None; L=Left; R=Rt; LR=Left & Rt)

DEFAULT Design Storm File,ID = CMLDS-24HR

Storm Tag Name = 2

Data Type, File, ID = Synthetic Storm TypeII 24hr
Storm Frequency = 2 yr
Total Rainfall Depth= 3.1200 in
Duration Multiplier = 1
Resulting Duration = 24.0000 hrs
Resulting Start Time= .0000 hrs Step= .1000 hrs End= 24.0000 hrs

	Node ID		Type	HYG Vol ac-ft	Trun.	Qpeak hrs	Qpeak cfs	Max WSEL ft
Outfall	OUT 10		JCT	2.728		16.4500	.72	
	POND 10	IN	POND	2.728		11.9500	46.24	
	POND 10	OUT	POND	2.728		16.4500	.72	755.01
	SUBAREA 10		AREA	2.728		11.9500	46.24	

NETWORK SUMMARY -- NODES
(Trun.= HYG Truncation: Blank=None; L=Left; R=Rt; LR=Left & Rt)

DEFAULT Design Storm File,ID = CMLDS-24HR

Storm Tag Name = 25

Data Type, File, ID = Synthetic Storm TypeII 24hr
Storm Frequency = 25 yr
Total Rainfall Depth= 5.7600 in
Duration Multiplier = 1
Resulting Duration = 24.0000 hrs
Resulting Start Time= .0000 hrs Step= .1000 hrs End= 24.0000 hrs

Node ID		Type	HYG Vol ac-ft	Trun.	Qpeak hrs	Qpeak cfs	Max WSEL ft
Outfall OUT 10		JCT	5.776		19.6500	.96	
POND 10	IN	POND	5.902		11.9500	96.22	
POND 10	OUT	POND	5.776		19.6500	.96	757.52
SUBAREA 10		AREA	5.902		11.9500	96.22	

NETWORK SUMMARY -- NODES
(Trun.= HYG Truncation: Blank=None; L=Left; R=Rt; LR=Left & Rt)

DEFAULT Design Storm File,ID = CMLDS-24HR

Storm Tag Name = 100

Data Type, File, ID = Synthetic Storm TypeII 24hr
Storm Frequency = 100 yr
Total Rainfall Depth= 6.9600 in
Duration Multiplier = 1
Resulting Duration = 24.0000 hrs
Resulting Start Time= .0000 hrs Step= .1000 hrs End= 24.0000 hrs

Node ID		Type	HYG Vol ac-ft	Trun.	Qpeak hrs	Qpeak cfs	Max WSEL ft
Outfall OUT 10		JCT	6.965		15.8000	2.52	
POND 10	IN	POND	7.373		11.9500	118.66	
POND 10	OUT	POND	6.965		15.8000	2.52	758.19
SUBAREA 10		AREA	7.373		11.9500	118.66	

NETWORK RUNOFF NODE SEQUENCE

Runoff Data	Apply to Node	Receiving Link
SCS UH SUBAREA 10	Subarea SUBAREA 10	Add Hyd SUBAREA 10

NETWORK ROUTING SEQUENCE

Link Operation	UPstream Node	DNstream Node
Add Hyd ADDLINK 10	Subarea SUBAREA 10	Pond POND 10 IN
POND ROUTE TOTAL OUTFLOW...		
Total Pond Outflow	Pond POND 10 IN	Outflow POND 10 OUT
SET POND ROUTING LINK TO TOTAL POND OUTFLOW...		
Outlet ROUTE 10	Outflow POND 10 OUT	Jct OUT 10

File.... J:\PROJECT\2004\046119BelewsCreek\
Title... Project Date: 8/4/2004
Project Engineer: Chas H. Sells
Project Title: Belew's Creek Landfill
Project Comments:

DESIGN STORMS SUMMARY

Design Storm File,ID = CMLDS-24HR

Storm Tag Name = 2

Data Type, File, ID = Synthetic Storm TypeII 24hr
Storm Frequency = 2 yr
Total Rainfall Depth= 3.1200 in
Duration Multiplier = 1
Resulting Duration = 24.0000 hrs
Resulting Start Time= .0000 hrs Step= .1000 hrs End= 24.0000 hrs

Storm Tag Name = 25

Data Type, File, ID = Synthetic Storm TypeII 24hr
Storm Frequency = 25 yr
Total Rainfall Depth= 5.7600 in
Duration Multiplier = 1
Resulting Duration = 24.0000 hrs
Resulting Start Time= .0000 hrs Step= .1000 hrs End= 24.0000 hrs

Storm Tag Name = 100

Data Type, File, ID = Synthetic Storm TypeII 24hr
Storm Frequency = 100 yr
Total Rainfall Depth= 6.9600 in
Duration Multiplier = 1
Resulting Duration = 24.0000 hrs
Resulting Start Time= .0000 hrs Step= .1000 hrs End= 24.0000 hrs

DESIGN STORMS SUMMARY

Design Storm File,ID = CMLDS-24HR

Storm Tag Name = 2

Data Type, File, ID = Synthetic Storm TypeII 24hr
Storm Frequency = 2 yr
Total Rainfall Depth= 3.1200 in
Duration Multiplier = 1
Resulting Duration = 24.0000 hrs
Resulting Start Time= .0000 hrs Step= .1000 hrs End= 24.0000 hrs

Storm Tag Name = 25

Data Type, File, ID = Synthetic Storm TypeII 24hr
Storm Frequency = 25 yr
Total Rainfall Depth= 5.7600 in
Duration Multiplier = 1
Resulting Duration = 24.0000 hrs
Resulting Start Time= .0000 hrs Step= .1000 hrs End= 24.0000 hrs

Storm Tag Name = 100

Data Type, File, ID = Synthetic Storm TypeII 24hr
Storm Frequency = 100 yr
Total Rainfall Depth= 6.9600 in
Duration Multiplier = 1
Resulting Duration = 24.0000 hrs
Resulting Start Time= .0000 hrs Step= .1000 hrs End= 24.0000 hrs

File.... J:\PROJECT\2004\046119BelewsCreek\CELLS 3-4.PPW

TIME OF CONCENTRATION CALCULATOR

Segment #1: Tc: TR-55 Sheet

Mannings n	.0110	
Hydraulic Length	227.00	ft
2yr, 24hr P	3.1200	in
Slope	.330000	ft/ft
Avg.Velocity	4.91	ft/sec

Segment #1 Time: .0128 hrs

Segment #2: Tc: Length & Vel.

Hydraulic Length	566.00	ft
Avg.Velocity	4.00	ft/sec

Segment #2 Time: .0393 hrs

Segment #3: Tc: TR-55 Shallow

Hydraulic Length	647.00	ft
Slope	.020000	ft/ft
Unpaved		
Avg.Velocity	2.28	ft/sec

Segment #3 Time: .0788 hrs

Total Tc: .1309 hrs

File.... J:\PROJECT\2004\046119BelewsCreek\CELLS 3-4.PPW

Tc Equations used...

==== SCS TR-55 Sheet Flow ==========

Tc = (.007 * ((n * Lf)**0.8)) / ((P**.5) * (Sf**.4))

Where: Tc = Time of concentration, hrs
n = Mannings n
Lf = Flow length, ft
P = 2yr, 24hr Rain depth, inches
Sf = Slope, %

==== SCS TR-55 Shallow Concentrated Flow ==========

Unpaved surface:
V = 16.1345 * (Sf**0.5)

Paved surface:
V = 20.3282 * (Sf**0.5)

Tc = (Lf / V) / (3600sec/hr)

Where: V = Velocity, ft/sec
Sf = Slope, ft/ft
Tc = Time of concentration, hrs
Lf = Flow length, ft

==== User Defined Length & Velocity ==========

Tc = (Lf / V) / (3600sec/hr)

Where: Tc = Time of concentration, hrs
Lf = Flow length, ft
V = Velocity, ft/sec

File.... J:\PROJECT\2004\046119BelewsCreek\CELLS 3-4.PPW

RUNOFF CURVE NUMBER DATA

Soil/Surface Description	CN	Area acres	Impervious Adjustment %C	%UC	Adjusted CN
Newly graded area (pervious only)	91	15.000			91.00
COMPOSITE AREA & WEIGHTED CN --->		15.000			91.00 (91)

SCS UNIT HYDROGRAPH METHOD
(Computational Notes)

DEFINITION OF TERMS: ----------

At = Total area (acres): At = Ai+Ap
Ai = Impervious area (acres)
Ap = Pervious area (acres)
CNi = Runoff curve number for impervious area
CNp = Runoff curve number for pervious area
fLoss = f loss constant infiltration (depth/time)
gKs = Saturated Hydraulic Conductivity (depth/time)
Md = Volumetric Moisture Deficit
Psi = Capillary Suction (length)
hK = Horton Infiltration Decay Rate (time^-1)
fo = Initial Infiltration Rate (depth/time)
fc = Ultimate(capacity)Infiltration Rate (depth/time)
Ia = Initial Abstraction (length)
dt = Computational increment (duration of unit excess rainfall)
Default dt is smallest value of 0.1333Tc, rtm, and th
(Smallest dt is then adjusted to match up with Tp)
UDdt = User specified override computational main time increment
(only used if UDdt is => .1333Tc)
D(t) = Point on distribution curve (fraction of P) for time step t

K = 2 / (1 + (Tr/Tp)): default K = 0.75: (for Tr/Tp = 1.67)
Ks = Hydrograph shape factor
= Unit Conversions * K:
= ((1hr/3600sec) * (1ft/12in) * ((5280ft)**2/sq.mi)) * K
Default Ks = 645.333 * 0.75 = 484

Lag = Lag time from center of excess runoff (dt) to Tp: Lag = 0.6Tc
P = Total precipitation depth, inches
Pa(t) = Accumulated rainfall at time step t
Pi(t) = Incremental rainfall at time step t
qp = Peak discharge (cfs) for 1in. runoff, for 1hr, for 1 sq.mi.
= (Ks * A * Q) / Tp (where Q = 1in. runoff, A=sq.mi.)
Qu(t) = Unit hydrograph ordinate (cfs) at time step t
Q(t) = Final hydrograph ordinate (cfs) at time step t
Rai(t) = Accumulated runoff (inches) at time step t for impervious area
Rap(t) = Accumulated runoff (inches) at time step t for pervious area
Rii(t) = Incremental runoff (inches) at time step t for impervious area
Rip(t) = Incremental runoff (inches) at time step t for pervious area
R(t) = Incremental weighted total runoff (inches)
Rtm = Time increment for rainfall table
Si = S for impervious area: Si = (1000/CNi) - 10
Sp = S for pervious area: Sp = (1000/CNp) - 10
t = Time step (row) number
Tc = Time of concentration
Tb = Time (hrs) of entire unit hydrograph: Tb = Tp + Tr
Tp = Time (hrs) to peak of a unit hydrograph: Tp = (dt/2) + Lag
Tr = Time (hrs) of receding limb of unit hydrograph: Tr = ratio of Tp

SCS UNIT HYDROGRAPH METHOD
(Computational Notes)

PRECIPITATION: ----------
Column (1): Time for time step t
Column (2): D(t) = Point on distribution curve for time step t
Column (3): Pi(t) = Pa(t) - Pa(t-1): Col.(4) - Preceding Col.(4)
Column (4): Pa(t) = D(t) x P: Col.(2) x P

PERVIOUS AREA RUNOFF (using SCS Runoff CN Method) ----------
Column (5): Rap(t) = Accumulated pervious runoff for time step t
If (Pa(t) is <= 0.2Sp) then use: Rap(t) = 0.0
If (Pa(t) is > 0.2Sp) then use:

Rap(t) = (Col.(4)-0.2Sp)**2 / (Col.(4)+0.8Sp)

Column (6): Rip(t) = Incremental pervious runoff for time step t
Rip(t) = Rap(t) - Rap(t-1)
Rip(t) = Col.(5) for current row - Col.(5) for preceding row.

IMPERVIOUS AREA RUNOFF ----------
Column (7 & 8)... Did not specify to use impervious areas.

INCREMENTAL WEIGHTED RUNOFF: ----------
Column (9): R(t) = (Ap/At) x Rip(t) + (Ai/At) x Rii(t)
R(t) = (Ap/At) x Col.(6) + (Ai/At) x Col.(8)

SCS UNIT HYDROGRAPH METHOD: ----------
Column (10): Q(t) is computed with the SCS unit hydrograph method
using R() and Qu().

SCS UNIT HYDROGRAPH METHOD

STORM EVENT: 2 year storm
Duration = 24.0000 hrs Rain Depth = 3.1200 in
Rain Dir = J:\PROJECT\2004\046119BelewsCreek\
Rain File -ID = - TypeII 24hr
Unit Hyd Type = Default Curvilinear
HYG Dir = J:\PROJECT\2004\046119BelewsCreek\
HYG File - ID = - SUBAREA 10 2
Tc = .1309 hrs
Drainage Area = 15.000 acres Runoff CN= 91

==========
Computational Time Increment = .01745 hrs
Computed Peak Time = 11.9564 hrs
Computed Peak Flow = 46.32 cfs

Time Increment for HYG File = .0500 hrs
Peak Time, Interpolated Output = 11.9500 hrs
Peak Flow, Interpolated Output = 46.24 cfs
==========

DRAINAGE AREA

ID:SUBAREA 10
CN = 91
Area = 15.000 acres
S = .9890 in
0.2S = .1978 in

Cumulative Runoff

2.1833 in
2.729 ac-ft

HYG Volume... 2.728 ac-ft (area under HYG curve)

***** SCS UNIT HYDROGRAPH PARAMETERS *****

Time Concentration, Tc = .13091 hrs (ID: SUBAREA 10)
Computational Incr, Tm = .01745 hrs = 0.20000 Tp

Unit Hyd. Shape Factor = 483.432 (37.46% under rising limb)
K = 483.43/645.333, K = .7491 (also, K = 2/(1+(Tr/Tp))
Receding/Rising, Tr/Tp = 1.6698 (solved from K = .7491)

Unit peak, qp = 129.83 cfs
Unit peak time Tp = .08727 hrs
Unit receding limb, Tr = .34909 hrs
Total unit time, Tb = .43637 hrs

SCS UNIT HYDROGRAPH METHOD

STORM EVENT: 25 year storm
Duration = 24.0000 hrs Rain Depth = 5.7600 in
Rain Dir = J:\PROJECT\2004\046119BelewsCreek\
Rain File -ID = - TypeII 24hr
Unit Hyd Type = Default Curvilinear
HYG Dir = J:\PROJECT\2004\046119BelewsCreek\
HYG File - ID = - SUBAREA 10 25
Tc = .1309 hrs
Drainage Area = 15.000 acres Runoff CN= 91

==========
Computational Time Increment = .01745 hrs
Computed Peak Time = 11.9390 hrs
Computed Peak Flow = 96.29 cfs

Time Increment for HYG File = .0500 hrs
Peak Time, Interpolated Output = 11.9500 hrs
Peak Flow, Interpolated Output = 96.22 cfs
==========

DRAINAGE AREA

ID:SUBAREA 10
CN = 91
Area = 15.000 acres
S = .9890 in
0.2S = .1978 in

Cumulative Runoff

4.7225 in
5.903 ac-ft

HYG Volume... 5.902 ac-ft (area under HYG curve)

***** SCS UNIT HYDROGRAPH PARAMETERS *****

Time Concentration, Tc = .13091 hrs (ID: SUBAREA 10)
Computational Incr, Tm = .01745 hrs = 0.20000 Tp

Unit Hyd. Shape Factor = 483.432 (37.46% under rising limb)
K = 483.43/645.333, K = .7491 (also, K = 2/(1+(Tr/Tp))
Receding/Rising, Tr/Tp = 1.6698 (solved from K = .7491)

Unit peak, qp = 129.83 cfs
Unit peak time Tp = .08727 hrs
Unit receding limb, Tr = .34909 hrs
Total unit time, Tb = .43637 hrs

SCS UNIT HYDROGRAPH METHOD

STORM EVENT: 100 year storm
Duration = 24.0000 hrs Rain Depth = 6.9600 in
Rain Dir = J:\PROJECT\2004\046119BelewsCreek\
Rain File -ID = - TypeII 24hr
Unit Hyd Type = Default Curvilinear
HYG Dir = J:\PROJECT\2004\046119BelewsCreek\
HYG File - ID = - SUBAREA 10 100
Tc = .1309 hrs
Drainage Area = 15.000 acres Runoff CN= 91

==========
Computational Time Increment = .01745 hrs
Computed Peak Time = 11.9390 hrs
Computed Peak Flow = 118.83 cfs

Time Increment for HYG File = .0500 hrs
Peak Time, Interpolated Output = 11.9500 hrs
Peak Flow, Interpolated Output = 118.66 cfs
==========

DRAINAGE AREA

ID:SUBAREA 10
CN = 91
Area = 15.000 acres
S = .9890 in
0.2S = .1978 in

Cumulative Runoff

5.8994 in
7.374 ac-ft

HYG Volume... 7.373 ac-ft (area under HYG curve)

***** SCS UNIT HYDROGRAPH PARAMETERS *****

Time Concentration, Tc = .13091 hrs (ID: SUBAREA 10)
Computational Incr, Tm = .01745 hrs = 0.20000 Tp

Unit Hyd. Shape Factor = 483.432 (37.46% under rising limb)
K = 483.43/645.333, K = .7491 (also, K = 2/(1+(Tr/Tp))
Receding/Rising, Tr/Tp = 1.6698 (solved from K = .7491)

Unit peak, qp = 129.83 cfs
Unit peak time Tp = .08727 hrs
Unit receding limb, Tr = .34909 hrs
Total unit time, Tb = .43637 hrs

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Elevation (ft)	Planimeter (sq.in)	Area (sq.ft)	A1+A2+sqr(A1*A2) (sq.ft)	Volume (ac-ft)	Volume Sum (ac-ft)
751.50	-----	100	0	.000	.000
752.00	-----	6816	7742	.030	.030
754.00	-----	40109	63459	.971	1.001
756.00	-----	46903	130385	1.995	2.996
758.00	-----	54079	151345	2.316	5.313
760.00	-----	61629	173439	2.654	7.967

POND VOLUME EQUATIONS

* Incremental volume computed by the Conic Method for Reservoir Volumes.

Volume = (1/3) * (EL2-EL1) * (Area1 + Area2 + sq.rt.(Area1*Area2))

where: EL1, EL2 = Lower and upper elevations of the increment
Area1,Area2 = Areas computed for EL1, EL2, respectively
Volume = Incremental volume between EL1 and EL2

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REQUESTED POND WS ELEVATIONS:

Min. Elev.= 751.50 ft
Increment = .10 ft
Max. Elev.= 760.00 ft

****
OUTLET CONNECTIVITY
****

---> Forward Flow Only (UpStream to DnStream)
<--- Reverse Flow Only (DnStream to UpStream)
<---> Forward and Reverse Both Allowed

Structure	No.		Outfall	E1, ft	E2, ft
Stand Pipe	R1	--->	CV	758.000	760.000
Orifice-Circular	O1	--->	CV	751.500	760.000
Culvert-Circular	CV	--->	TW	751.500	760.000

TW SETUP, DS Channel

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OUTLET STRUCTURE INPUT DATA

Structure ID	= R1	
Structure Type	= Stand Pipe	
# of Openings	= 1	
Invert Elev.	= 758.00	ft
Diameter	= 2.0000	ft
Orifice Area	= 3.1416	sq.ft
Orifice Coeff.	= .600	
Weir Length	= 6.28	ft
Weir Coeff.	= 3.000	
K, Submerged	= .000	
K, Reverse	= 1.000	
Kb,Barrel	= .000000	(per ft of full flow)
Barrel Length	= .00	ft
Mannings n	= .0000	

Structure ID	= O1	
Structure Type	= Orifice-Circular	
# of Openings	= 1	
Invert Elev.	= 751.50	ft
Diameter	= .3300	ft
Orifice Coeff.	= .600	

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OUTLET STRUCTURE INPUT DATA

Structure ID = CV
Structure Type = Culvert-Circular

No. Barrels	=	1	
Barrel Diameter	=	1.5000	ft
Upstream Invert	=	751.50	ft
Dnstream Invert	=	750.97	ft
Horiz. Length	=	53.00	ft
Barrel Length	=	53.00	ft
Barrel Slope	=	.01000	ft/ft

OUTLET CONTROL DATA...

Mannings n	=	.0240	
Ke	=	.2000	(forward entrance loss)
Kb	=	.062076	(per ft of full flow)
Kr	=	.2000	(reverse entrance loss)
HW Convergence	=	.001	+/- ft

INLET CONTROL DATA...

Equation form	=	1
Inlet Control K	=	.0078
Inlet Control M	=	2.0000
Inlet Control c	=	.03790
Inlet Control Y	=	.6900
T1 ratio (HW/D)	=	1.131
T2 ratio (HW/D)	=	1.291
Slope Factor	=	-.500

Use unsubmerged inlet control Form 1 equ. below T1 elev.
Use submerged inlet control Form 1 equ. above T2 elev.

In transition zone between unsubmerged and submerged inlet control, interpolate between flows at T1 & T2...
At T1 Elev = 753.20 ft ---> Flow = 7.58 cfs
At T2 Elev = 753.44 ft ---> Flow = 8.66 cfs

Structure ID = TW
Structure Type = TW SETUP, DS Channel

FREE OUTFALL CONDITIONS SPECIFIED

CONVERGENCE TOLERANCES...

Maximum Iterations=	30	
Min. TW tolerance =	.01	ft
Max. TW tolerance =	.01	ft
Min. HW tolerance =	.01	ft
Max. HW tolerance =	.01	ft
Min. Q tolerance =	.10	cfs
Max. Q tolerance =	.10	cfs

File.... J:\PROJECT\2004\046119BelewsCreek\CELLS 3-4.PPW

RATING TABLE FOR ONE OUTLET TYPE

Structure ID = R1 (Stand Pipe)

Upstream ID = (Pond Water Surface)
DNstream ID = CV (Culvert-Circular)

Pond WS. Elev. ft	Device Q cfs	(into) HW HGL ft	Converge DS HGL ft	Next DS HGL ft	DS HGL Error +/-ft	Q SUM Error +/-cfs	DS Chan. TW ft	TW Error +/-ft
751.50	.00	...	...	...	...	...	Free Outfall	
		WS below an invert; no flow.						
751.60	.00	...	...	...	...	...	Free Outfall	
		WS below an invert; no flow.						
751.70	.00	...	...	...	...	...	Free Outfall	
		WS below an invert; no flow.						
751.80	.00	...	...	...	...	...	Free Outfall	
		WS below an invert; no flow.						
751.90	.00	...	...	...	...	...	Free Outfall	
		WS below an invert; no flow.						
752.00	.00	...	...	...	...	...	Free Outfall	
		WS below an invert; no flow.						
752.10	.00	...	...	...	...	...	Free Outfall	
		WS below an invert; no flow.						
752.20	.00	...	...	...	...	...	Free Outfall	
		WS below an invert; no flow.						
752.30	.00	...	...	...	...	...	Free Outfall	
		WS below an invert; no flow.						
752.40	.00	...	...	...	...	...	Free Outfall	
		WS below an invert; no flow.						
752.50	.00	...	...	...	...	...	Free Outfall	
		WS below an invert; no flow.						
752.60	.00	...	...	...	...	...	Free Outfall	
		WS below an invert; no flow.						
752.70	.00	...	...	...	...	...	Free Outfall	
		WS below an invert; no flow.						
752.80	.00	...	...	...	...	...	Free Outfall	
		WS below an invert; no flow.						
752.90	.00	...	...	...	...	...	Free Outfall	
		WS below an invert; no flow.						
753.00	.00	...	...	...	...	...	Free Outfall	
		WS below an invert; no flow.						
753.10	.00	...	...	...	...	...	Free Outfall	
		WS below an invert; no flow.						
753.20	.00	...	...	...	...	...	Free Outfall	
		WS below an invert; no flow.						

File.... J:\PROJECT\2004\046119BelewsCreek\CELLS 3-4.PPW

RATING TABLE FOR ONE OUTLET TYPE

Structure ID = R1 (Stand Pipe)

Upstream ID = (Pond Water Surface)
DNstream ID = CV (Culvert-Circular)

Pond WS. Elev. ft	Device Q cfs	(into) HW HGL ft	Converge DS HGL ft	Next DS HGL ft	DS HGL Error +/-ft	Q SUM Error +/-cfs	DS Chan. TW ft	TW Error +/-ft
753.30	.00	...	...	...	...	...	Free Outfall	
		WS below an invert; no flow.						
753.40	.00	...	...	...	...	...	Free Outfall	
		WS below an invert; no flow.						
753.50	.00	...	...	...	...	...	Free Outfall	
		WS below an invert; no flow.						
753.60	.00	...	...	...	...	...	Free Outfall	
		WS below an invert; no flow.						
753.70	.00	...	...	...	...	...	Free Outfall	
		WS below an invert; no flow.						
753.80	.00	...	...	...	...	...	Free Outfall	
		WS below an invert; no flow.						
753.90	.00	...	...	...	...	...	Free Outfall	
		WS below an invert; no flow.						
754.00	.00	...	...	...	...	...	Free Outfall	
		WS below an invert; no flow.						
754.10	.00	...	...	...	...	...	Free Outfall	
		WS below an invert; no flow.						
754.20	.00	...	...	...	...	...	Free Outfall	
		WS below an invert; no flow.						
754.30	.00	...	...	...	...	...	Free Outfall	
		WS below an invert; no flow.						
754.40	.00	...	...	...	...	...	Free Outfall	
		WS below an invert; no flow.						
754.50	.00	...	...	...	...	...	Free Outfall	
		WS below an invert; no flow.						
754.60	.00	...	...	...	...	...	Free Outfall	
		WS below an invert; no flow.						
754.70	.00	...	...	...	...	...	Free Outfall	
		WS below an invert; no flow.						
754.80	.00	...	...	...	...	...	Free Outfall	
		WS below an invert; no flow.						
754.90	.00	...	...	...	...	...	Free Outfall	
		WS below an invert; no flow.						
755.00	.00	...	...	...	...	...	Free Outfall	
		WS below an invert; no flow.						

RATING TABLE FOR ONE OUTLET TYPE

Structure ID = R1 (Stand Pipe)

Upstream ID = (Pond Water Surface)
DNstream ID = CV (Culvert-Circular)

Pond WS. Elev. ft	Device Q cfs	(into) HW HGL ft	Converge DS HGL ft	Next DS HGL ft	DS HGL Error +/-ft	Q SUM Error +/-cfs	DS Chan. TW ft	TW Error +/-ft
755.10	.00	...	...	...	...	...	Free Outfall	
		WS below an invert; no flow.						
755.20	.00	...	...	...	...	...	Free Outfall	
		WS below an invert; no flow.						
755.30	.00	...	...	...	...	...	Free Outfall	
		WS below an invert; no flow.						
755.40	.00	...	...	...	...	...	Free Outfall	
		WS below an invert; no flow.						
755.50	.00	...	...	...	...	...	Free Outfall	
		WS below an invert; no flow.						
755.60	.00	...	...	...	...	...	Free Outfall	
		WS below an invert; no flow.						
755.70	.00	...	...	...	...	...	Free Outfall	
		WS below an invert; no flow.						
755.80	.00	...	...	...	...	...	Free Outfall	
		WS below an invert; no flow.						
755.90	.00	...	...	...	...	...	Free Outfall	
		WS below an invert; no flow.						
756.00	.00	...	...	...	...	...	Free Outfall	
		WS below an invert; no flow.						
756.10	.00	...	...	...	...	...	Free Outfall	
		WS below an invert; no flow.						
756.20	.00	...	...	...	...	...	Free Outfall	
		WS below an invert; no flow.						
756.30	.00	...	...	...	...	...	Free Outfall	
		WS below an invert; no flow.						
756.40	.00	...	...	...	...	...	Free Outfall	
		WS below an invert; no flow.						
756.50	.00	...	...	...	...	...	Free Outfall	
		WS below an invert; no flow.						
756.60	.00	...	...	...	...	...	Free Outfall	
		WS below an invert; no flow.						
756.70	.00	...	...	...	...	...	Free Outfall	
		WS below an invert; no flow.						
756.80	.00	...	...	...	...	...	Free Outfall	
		WS below an invert; no flow.						

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RATING TABLE FOR ONE OUTLET TYPE

Structure ID = R1 (Stand Pipe)

Upstream ID = (Pond Water Surface)
DNstream ID = CV (Culvert-Circular)

Pond WS. Elev. ft	Device Q cfs	(into) HW HGL ft	Converge DS HGL ft	Next DS HGL ft	DS HGL Error +/-ft	Q SUM Error +/-cfs	DS Chan. TW ft	TW Error +/-ft
756.90	.00	...	...	...	...	...	Free Outfall	
		WS below an invert; no flow.						
757.00	.00	...	...	...	...	...	Free Outfall	
		WS below an invert; no flow.						
757.10	.00	...	...	...	...	...	Free Outfall	
		WS below an invert; no flow.						
757.20	.00	...	...	...	...	...	Free Outfall	
		WS below an invert; no flow.						
757.30	.00	...	...	...	...	...	Free Outfall	
		WS below an invert; no flow.						
757.40	.00	...	...	...	...	...	Free Outfall	
		WS below an invert; no flow.						
757.50	.00	...	...	...	...	...	Free Outfall	
		WS below an invert; no flow.						
757.60	.00	...	...	...	...	...	Free Outfall	
		WS below an invert; no flow.						
757.70	.00	...	...	...	...	...	Free Outfall	
		WS below an invert; no flow.						
757.80	.00	...	...	...	...	...	Free Outfall	
		WS below an invert; no flow.						
757.90	.00	...	...	...	...	...	Free Outfall	
		WS below an invert; no flow.						
758.00	.00	...	...	...	...	...	Free Outfall	
		WS below an invert; no flow.						
758.10	.60	758.10	Free	752.19	.000	.000	Free Outfall	
		Weir: H =.10						
758.20	1.69	758.20	Free	752.41	.000	.000	Free Outfall	
		Weir: H =.20						
758.30	3.10	758.30	Free	752.67	.000	.000	Free Outfall	
		Weir: H =.30						
758.40	4.77	758.40	Free	752.97	.000	.000	Free Outfall	
		Weir: H =.40						
758.50	6.66	758.50	Free	753.50	.000	.000	Free Outfall	
		Weir: H =.50						
758.60	8.76	758.60	Free	754.37	.000	.000	Free Outfall	
		Weir: H =.60						

RATING TABLE FOR ONE OUTLET TYPE

Structure ID = R1 (Stand Pipe)

Upstream ID = (Pond Water Surface)
DNstream ID = CV (Culvert-Circular)

Pond WS. Elev. ft	Device Q cfs	(into) HW HGL ft	Converge DS HGL ft	Next DS HGL ft	DS HGL Error +/-ft	Q SUM Error +/-cfs	DS Chan. TW ft	TW Error +/-ft
758.70	11.04	758.70	Free	755.49	.000	.000	Free Outfall	
		Weir: H =.70						
758.80	13.49	758.80	Free	756.84	.000	.000	Free Outfall	
		Weir: H =.80						
758.90	14.34	758.90	Free	757.36	.000	.000	Free Outfall	
		Orifice: H =.90						
759.00	15.12	759.00	Free	757.85	.000	.000	Free Outfall	
		Orifice: H =1.00						
759.10	16.22	759.10	759.10	759.10	.000	.000	Free Outfall	
		DS HGL+Loss > crest: Flow set to Downstream outlet.						
759.20	16.83	759.20	759.20	759.20	.000	.000	Free Outfall	
		DS HGL+Loss > crest: Flow set to Downstream outlet.						
759.30	17.51	759.30	759.30	759.30	.000	.000	Free Outfall	
		DS HGL+Loss > crest: Flow set to Downstream outlet.						
759.40	17.63	759.40	759.40	759.40	.000	.000	Free Outfall	
		DS HGL+Loss > crest: Flow set to Downstream outlet.						
759.50	17.76	759.50	759.50	759.50	.000	.000	Free Outfall	
		DS HGL+Loss > crest: Flow set to Downstream outlet.						
759.60	17.89	759.60	759.60	759.60	.000	.000	Free Outfall	
		DS HGL+Loss > crest: Flow set to Downstream outlet.						
759.70	18.01	759.70	759.70	759.70	.000	.000	Free Outfall	
		DS HGL+Loss > crest: Flow set to Downstream outlet.						
759.80	18.13	759.80	759.80	759.80	.000	.000	Free Outfall	
		DS HGL+Loss > crest: Flow set to Downstream outlet.						
759.90	18.26	759.90	759.90	759.90	.000	.000	Free Outfall	
		DS HGL+Loss > crest: Flow set to Downstream outlet.						
760.00	18.38	760.00	760.00	760.00	.000	.000	Free Outfall	
		DS HGL+Loss > crest: Flow set to Downstream outlet.						

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RATING TABLE FOR ONE OUTLET TYPE

Structure ID = O1 (Orifice-Circular)

Upstream ID = (Pond Water Surface)
DNstream ID = CV (Culvert-Circular)

Pond WS. Elev. ft	Device Q cfs	(into) HW HGL ft	Converge DS HGL ft	Next DS HGL ft	DS HGL Error +/-ft	Q SUM Error +/-cfs	DS Chan. TW ft	TW Error +/-ft
751.50	.00	...	...	...	...	...	Free Outfall	
		WS below an invert; no flow.						
751.60	.02	751.60	751.57	751.57	.000	.000	Free Outfall	
		CRIT.DEPTH	CONTROL	Vh= .027ft	Dcr= .072ft		CRIT.DEPTH	
751.70	.07	751.70	751.64	751.64	.000	.000	Free Outfall	
		CRIT.DEPTH	CONTROL	Vh= .055ft	Dcr= .144ft		CRIT.DEPTH	
751.80	.14	751.80	751.69	751.69	.000	.000	Free Outfall	
		CRIT.DEPTH	CONTROL	Vh= .089ft	Dcr= .211ft		CRIT.DEPTH	
751.90	.18	751.90	751.71	751.72	.006	.000	Free Outfall	
		H =.19						
752.00	.21	752.00	751.74	751.74	.005	.000	Free Outfall	
		H =.26						
752.10	.24	752.10	751.76	751.76	.004	.000	Free Outfall	
		H =.34						
752.20	.27	752.20	751.77	751.77	.004	.000	Free Outfall	
		H =.43						
752.30	.30	752.30	751.78	751.79	.001	.000	Free Outfall	
		H =.52						
752.40	.32	752.40	751.80	751.80	.002	.000	Free Outfall	
		H =.60						
752.50	.34	752.50	751.81	751.81	.003	.000	Free Outfall	
		H =.69						
752.60	.36	752.60	751.82	751.82	.002	.000	Free Outfall	
		H =.78						
752.70	.39	752.70	751.82	751.83	.002	.000	Free Outfall	
		H =.88						
752.80	.40	752.80	751.83	751.84	.001	.000	Free Outfall	
		H =.97						
752.90	.42	752.90	751.84	751.84	.001	.000	Free Outfall	
		H =1.06						
753.00	.44	753.00	751.85	751.85	.000	.000	Free Outfall	
		H =1.15						
753.10	.46	753.10	751.86	751.86	.001	.000	Free Outfall	
		H =1.24						
753.20	.48	753.20	751.86	751.86	.001	.000	Free Outfall	
		H =1.34						

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RATING TABLE FOR ONE OUTLET TYPE

Structure ID = O1 (Orifice-Circular)

Upstream ID = (Pond Water Surface)
DNstream ID = CV (Culvert-Circular)

Pond WS. Elev. ft	Device Q cfs	(into) HW HGL ft	Converge DS HGL ft	Next DS HGL ft	DS HGL Error +/-ft	Q SUM Error +/-cfs	DS Chan. TW ft	TW Error +/-ft
753.30	.49	753.30	751.87	751.87	.002	.000	Free Outfall	
		H =1.43						
753.40	.51	753.40	751.88	751.88	.001	.000	Free Outfall	
		H =1.52						
753.50	.52	753.50	751.88	751.88	.000	.000	Free Outfall	
		H =1.62						
753.60	.54	753.60	751.89	751.89	.001	.000	Free Outfall	
		H =1.71						
753.70	.55	753.70	751.89	751.89	.001	.000	Free Outfall	
		H =1.81						
753.80	.57	753.80	751.90	751.90	.001	.000	Free Outfall	
		H =1.90						
753.90	.58	753.90	751.90	751.90	.000	.000	Free Outfall	
		H =2.00						
754.00	.60	754.00	751.91	751.91	.000	.000	Free Outfall	
		H =2.09						
754.10	.61	754.10	751.91	751.91	.000	.000	Free Outfall	
		H =2.19						
754.20	.62	754.20	751.92	751.92	.000	.000	Free Outfall	
		H =2.28						
754.30	.63	754.30	751.92	751.92	.002	.000	Free Outfall	
		H =2.38						
754.40	.65	754.40	751.93	751.93	.001	.000	Free Outfall	
		H =2.47						
754.50	.66	754.50	751.93	751.93	.000	.000	Free Outfall	
		H =2.57						
754.60	.67	754.60	751.94	751.94	.000	.000	Free Outfall	
		H =2.66						
754.70	.68	754.70	751.94	751.94	.001	.000	Free Outfall	
		H =2.76						
754.80	.70	754.80	751.94	751.94	.001	.000	Free Outfall	
		H =2.86						
754.90	.71	754.90	751.95	751.95	.001	.000	Free Outfall	
		H =2.95						
755.00	.72	755.00	751.95	751.95	.000	.000	Free Outfall	
		H =3.05						

RATING TABLE FOR ONE OUTLET TYPE

Structure ID = O1 (Orifice-Circular)

Upstream ID = (Pond Water Surface)
DNstream ID = CV (Culvert-Circular)

Pond WS. Elev. ft	Device Q cfs	(into) HW HGL ft	Converge DS HGL ft	Next DS HGL ft	DS HGL Error +/-ft	Q SUM Error +/-cfs	DS Chan. TW ft	TW Error +/-ft
755.10	.73	755.10	751.95	751.95	.000	.000	Free Outfall	
		H =3.15						
755.20	.74	755.20	751.96	751.96	.000	.000	Free Outfall	
		H =3.24						
755.30	.75	755.30	751.96	751.96	.000	.000	Free Outfall	
		H =3.34						
755.40	.76	755.40	751.96	751.96	.001	.000	Free Outfall	
		H =3.44						
755.50	.77	755.50	751.97	751.97	.000	.000	Free Outfall	
		H =3.53						
755.60	.78	755.60	751.97	751.97	.000	.000	Free Outfall	
		H =3.63						
755.70	.79	755.70	751.98	751.97	.010	.000	Free Outfall	
		H =3.72						
755.80	.80	755.80	751.98	751.98	.002	.000	Free Outfall	
		H =3.82						
755.90	.81	755.90	751.98	751.98	.001	.000	Free Outfall	
		H =3.92						
756.00	.82	756.00	751.98	751.98	.001	.000	Free Outfall	
		H =4.02						
756.10	.83	756.10	752.00	751.99	.010	.000	Free Outfall	
		H =4.10						
756.20	.84	756.20	752.00	751.99	.009	.000	Free Outfall	
		H =4.20						
756.30	.85	756.30	751.99	751.99	.001	.000	Free Outfall	
		H =4.31						
756.40	.86	756.40	752.00	751.99	.001	.000	Free Outfall	
		H =4.40						
756.50	.87	756.50	752.01	752.00	.010	.000	Free Outfall	
		H =4.49						
756.60	.88	756.60	752.01	752.00	.008	.000	Free Outfall	
		H =4.59						
756.70	.89	756.70	752.00	752.00	.000	.000	Free Outfall	
		H =4.70						
756.80	.90	756.80	752.02	752.01	.010	.000	Free Outfall	
		H =4.78						

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RATING TABLE FOR ONE OUTLET TYPE

Structure ID = O1 (Orifice-Circular)

Upstream ID = (Pond Water Surface)
DNstream ID = CV (Culvert-Circular)

Pond WS. Elev. ft	Device Q cfs	(into) HW HGL ft	Converge DS HGL ft	Next DS HGL ft	DS HGL Error +/-ft	Q SUM Error +/-cfs	DS Chan. TW ft	TW Error +/-ft
756.90	.91	756.90	752.02	752.01	.010	.000	Free Outfall	
		H =4.88						
757.00	.92	757.00	752.02	752.01	.008	.000	Free Outfall	
		H =4.98						
757.10	.93	757.10	752.02	752.01	.009	.000	Free Outfall	
		H =5.08						
757.20	.94	757.20	752.03	752.02	.009	.000	Free Outfall	
		H =5.17						
757.30	.95	757.30	752.03	752.02	.010	.000	Free Outfall	
		H =5.27						
757.40	.95	757.40	752.02	752.02	.001	.000	Free Outfall	
		H =5.38						
757.50	.96	757.50	752.02	752.03	.001	.000	Free Outfall	
		H =5.48						
757.60	.97	757.60	752.04	752.03	.009	.000	Free Outfall	
		H =5.56						
757.70	.98	757.70	752.04	752.03	.009	.000	Free Outfall	
		H =5.66						
757.80	.99	757.80	752.04	752.03	.008	.000	Free Outfall	
		H =5.76						
757.90	1.00	757.90	752.04	752.03	.008	.000	Free Outfall	
		H =5.86						
758.00	1.00	758.00	752.05	752.04	.009	.000	Free Outfall	
		H =5.95						
758.10	1.00	758.10	752.20	752.19	.009	.000	Free Outfall	
		H =5.90						
758.20	.99	758.20	752.41	752.41	.000	.000	Free Outfall	
		H =5.79						
758.30	.98	758.30	752.67	752.67	.000	.000	Free Outfall	
		H =5.63						
758.40	.96	758.40	752.97	752.97	.000	.000	Free Outfall	
		H =5.43						
758.50	.92	758.50	753.51	753.50	.004	.000	Free Outfall	
		H =4.99						
758.60	.85	758.60	754.37	754.37	.005	.000	Free Outfall	
		H =4.23						

RATING TABLE FOR ONE OUTLET TYPE

Structure ID = O1 (Orifice-Circular)

Upstream ID = (Pond Water Surface)
DNstream ID = CV (Culvert-Circular)

Pond WS. Elev. ft	Device Q cfs	(into) HW HGL ft	Converge DS HGL ft	Next DS HGL ft	DS HGL Error +/-ft	Q SUM Error +/-cfs	DS Chan. TW ft	TW Error +/-ft
758.70	.74	758.70	755.49	755.49	.002	.000	Free Outfall	
		H =3.21						
758.80	.58	758.80	756.85	756.84	.003	.000	Free Outfall	
		H =1.95						
758.90	.51	758.90	757.37	757.36	.006	.000	Free Outfall	
		H =1.53						
759.00	.44	759.00	757.85	757.85	.001	.000	Free Outfall	
		H =1.15						
759.10	.00	759.10	759.10	759.10	.000	.000	Free Outfall	
				Full riser flow. Q=0 this opening.				
759.20	.00	759.20	759.20	759.20	.000	.000	Free Outfall	
				Full riser flow. Q=0 this opening.				
759.30	.00	759.30	759.30	759.30	.000	.000	Free Outfall	
				Full riser flow. Q=0 this opening.				
759.40	.00	759.40	759.40	759.40	.000	.000	Free Outfall	
				Full riser flow. Q=0 this opening.				
759.50	.00	759.50	759.50	759.50	.000	.000	Free Outfall	
				Full riser flow. Q=0 this opening.				
759.60	.00	759.60	759.60	759.60	.000	.000	Free Outfall	
				Full riser flow. Q=0 this opening.				
759.70	.00	759.70	759.70	759.70	.000	.000	Free Outfall	
				Full riser flow. Q=0 this opening.				
759.80	.00	759.80	759.80	759.80	.000	.000	Free Outfall	
				Full riser flow. Q=0 this opening.				
759.90	.00	759.90	759.90	759.90	.000	.000	Free Outfall	
				Full riser flow. Q=0 this opening.				
760.00	.00	760.00	760.00	760.00	.000	.000	Free Outfall	
				Full riser flow. Q=0 this opening.				

Type.... Individual Outlet Curves
Name.... Outlet 1

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RATING TABLE FOR ONE OUTLET TYPE

Structure ID = CV (Culvert-Circular)

Mannings open channel maximum capacity: 6.12 cfs
UPstream ID's= R1, O1
DNstream ID = TW (Pond Outfall)

Pond WS. Elev. ft	Device Q cfs	(into) HW HGL ft	Converge DS HGL ft	Next DS HGL ft	DS HGL Error +/-ft	Q SUM Error +/-cfs	DS Chan. TW ft	TW Error +/-ft
751.50	.00	...	...	...	...	...	Free Outfall	
		WS below an invert; no flow.						
751.60	.02	751.57	Free	Free	.000	.000	Free Outfall	
		BACKWATER CONTROL..		Vh= .008ft	hwDi= .062ft		Lbw= 53.0ft	
751.70	.07	751.64	Free	Free	.000	.000	Free Outfall	
		BACKWATER CONTROL..		Vh= .018ft	hwDi= .115ft		Lbw= 53.0ft	
751.80	.14	751.69	Free	Free	.000	.000	Free Outfall	
		BACKWATER CONTROL..		Vh= .028ft	hwDi= .161ft		Lbw= 53.0ft	
751.90	.18	751.72	Free	Free	.000	.000	Free Outfall	
		BACKWATER CONTROL..		Vh= .033ft	hwDi= .182ft		Lbw= 53.0ft	
752.00	.21	751.74	Free	Free	.000	.000	Free Outfall	
		BACKWATER CONTROL..		Vh= .036ft	hwDi= .198ft		Lbw= 53.0ft	
752.10	.24	751.76	Free	Free	.000	.000	Free Outfall	
		BACKWATER CONTROL..		Vh= .039ft	hwDi= .212ft		Lbw= 53.0ft	
752.20	.27	751.77	Free	Free	.000	.000	Free Outfall	
		BACKWATER CONTROL..		Vh= .042ft	hwDi= .223ft		Lbw= 53.0ft	
752.30	.30	751.79	Free	Free	.000	.000	Free Outfall	
		BACKWATER CONTROL..		Vh= .045ft	hwDi= .232ft		Lbw= 53.0ft	
752.40	.32	751.80	Free	Free	.000	.000	Free Outfall	
		BACKWATER CONTROL..		Vh= .047ft	hwDi= .241ft		Lbw= 53.0ft	
752.50	.34	751.81	Free	Free	.000	.000	Free Outfall	
		BACKWATER CONTROL..		Vh= .049ft	hwDi= .250ft		Lbw= 53.0ft	
752.60	.36	751.82	Free	Free	.000	.000	Free Outfall	
		BACKWATER CONTROL..		Vh= .051ft	hwDi= .258ft		Lbw= 53.0ft	
752.70	.39	751.83	Free	Free	.000	.000	Free Outfall	
		BACKWATER CONTROL..		Vh= .052ft	hwDi= .264ft		Lbw= 53.0ft	
752.80	.40	751.84	Free	Free	.000	.000	Free Outfall	
		BACKWATER CONTROL..		Vh= .054ft	hwDi= .271ft		Lbw= 53.0ft	
752.90	.42	751.84	Free	Free	.000	.000	Free Outfall	
		BACKWATER CONTROL..		Vh= .055ft	hwDi= .277ft		Lbw= 53.0ft	
753.00	.44	751.85	Free	Free	.000	.000	Free Outfall	
		BACKWATER CONTROL..		Vh= .057ft	hwDi= .282ft		Lbw= 53.0ft	
753.10	.46	751.86	Free	Free	.000	.000	Free Outfall	
		BACKWATER CONTROL..		Vh= .058ft	hwDi= .288ft		Lbw= 53.0ft	
753.20	.48	751.86	Free	Free	.000	.000	Free Outfall	
		BACKWATER CONTROL..		Vh= .060ft	hwDi= .293ft		Lbw= 53.0ft	

RATING TABLE FOR ONE OUTLET TYPE

Structure ID = CV (Culvert-Circular)

Mannings open channel maximum capacity: 6.12 cfs
UPstream ID's= R1, O1
DNstream ID = TW (Pond Outfall)

Pond WS. Elev. ft	Device Q cfs	(into) HW HGL ft	Converge DS HGL ft	Next DS HGL ft	DS HGL Error +/-ft	Q SUM Error +/-cfs	DS Chan. TW ft	TW Error +/-ft
753.30	.49	751.87	Free	Free	.000	.000	Free Outfall	
		BACKWATER	CONTROL..	Vh= .061ft	hwDi= .299ft		Lbw= 53.0ft	
753.40	.51	751.88	Free	Free	.000	.000	Free Outfall	
		BACKWATER	CONTROL..	Vh= .061ft	hwDi= .304ft		Lbw= 53.0ft	
753.50	.52	751.88	Free	Free	.000	.000	Free Outfall	
		BACKWATER	CONTROL..	Vh= .063ft	hwDi= .308ft		Lbw= 53.0ft	
753.60	.54	751.89	Free	Free	.000	.000	Free Outfall	
		BACKWATER	CONTROL..	Vh= .063ft	hwDi= .313ft		Lbw= 53.0ft	
753.70	.55	751.89	Free	Free	.000	.000	Free Outfall	
		BACKWATER	CONTROL..	Vh= .065ft	hwDi= .316ft		Lbw= 53.0ft	
753.80	.57	751.90	Free	Free	.000	.000	Free Outfall	
		BACKWATER	CONTROL..	Vh= .066ft	hwDi= .319ft		Lbw= 53.0ft	
753.90	.58	751.90	Free	Free	.000	.000	Free Outfall	
		BACKWATER	CONTROL..	Vh= .066ft	hwDi= .325ft		Lbw= 53.0ft	
754.00	.60	751.91	Free	Free	.000	.000	Free Outfall	
		BACKWATER	CONTROL..	Vh= .067ft	hwDi= .328ft		Lbw= 53.0ft	
754.10	.61	751.91	Free	Free	.000	.000	Free Outfall	
		BACKWATER	CONTROL..	Vh= .068ft	hwDi= .332ft		Lbw= 53.0ft	
754.20	.62	751.92	Free	Free	.000	.000	Free Outfall	
		BACKWATER	CONTROL..	Vh= .069ft	hwDi= .335ft		Lbw= 53.0ft	
754.30	.63	751.92	Free	Free	.000	.000	Free Outfall	
		BACKWATER	CONTROL..	Vh= .070ft	hwDi= .339ft		Lbw= 53.0ft	
754.40	.65	751.93	Free	Free	.000	.000	Free Outfall	
		BACKWATER	CONTROL..	Vh= .071ft	hwDi= .342ft		Lbw= 53.0ft	
754.50	.66	751.93	Free	Free	.000	.000	Free Outfall	
		BACKWATER	CONTROL..	Vh= .072ft	hwDi= .345ft		Lbw= 53.0ft	
754.60	.67	751.94	Free	Free	.000	.000	Free Outfall	
		BACKWATER	CONTROL..	Vh= .073ft	hwDi= .347ft		Lbw= 53.0ft	
754.70	.68	751.94	Free	Free	.000	.000	Free Outfall	
		BACKWATER	CONTROL..	Vh= .074ft	hwDi= .350ft		Lbw= 53.0ft	
754.80	.70	751.94	Free	Free	.000	.000	Free Outfall	
		BACKWATER	CONTROL..	Vh= .074ft	hwDi= .354ft		Lbw= 53.0ft	
754.90	.71	751.95	Free	Free	.000	.000	Free Outfall	
		BACKWATER	CONTROL..	Vh= .076ft	hwDi= .357ft		Lbw= 53.0ft	
755.00	.72	751.95	Free	Free	.000	.000	Free Outfall	
		BACKWATER	CONTROL..	Vh= .076ft	hwDi= .360ft		Lbw= 53.0ft	

RATING TABLE FOR ONE OUTLET TYPE

Structure ID = CV (Culvert-Circular)

Mannings open channel maximum capacity: 6.12 cfs
UPstream ID's= R1, O1
DNstream ID = TW (Pond Outfall)

Pond WS. Elev. ft	Device Q cfs	(into) HW HGL ft	Converge DS HGL ft	Next DS HGL ft	DS HGL Error +/-ft	Q SUM Error +/-cfs	DS Chan. TW ft	TW Error +/-ft
755.10	.73	751.95	Free	Free	.000	.000	Free Outfall	
		BACKWATER CONTROL..		Vh= .076ft	hwDi= .363ft		Lbw= 53.0ft	
755.20	.74	751.96	Free	Free	.000	.000	Free Outfall	
		BACKWATER CONTROL..		Vh= .077ft	hwDi= .366ft		Lbw= 53.0ft	
755.30	.75	751.96	Free	Free	.000	.000	Free Outfall	
		BACKWATER CONTROL..		Vh= .078ft	hwDi= .368ft		Lbw= 53.0ft	
755.40	.76	751.96	Free	Free	.000	.000	Free Outfall	
		BACKWATER CONTROL..		Vh= .078ft	hwDi= .371ft		Lbw= 53.0ft	
755.50	.77	751.97	Free	Free	.000	.000	Free Outfall	
		BACKWATER CONTROL..		Vh= .078ft	hwDi= .374ft		Lbw= 53.0ft	
755.60	.78	751.97	Free	Free	.000	.000	Free Outfall	
		BACKWATER CONTROL..		Vh= .079ft	hwDi= .377ft		Lbw= 53.0ft	
755.70	.79	751.97	Free	Free	.000	.000	Free Outfall	
		BACKWATER CONTROL..		Vh= .080ft	hwDi= .378ft		Lbw= 53.0ft	
755.80	.80	751.98	Free	Free	.000	.000	Free Outfall	
		BACKWATER CONTROL..		Vh= .080ft	hwDi= .382ft		Lbw= 53.0ft	
755.90	.81	751.98	Free	Free	.000	.000	Free Outfall	
		BACKWATER CONTROL..		Vh= .081ft	hwDi= .382ft		Lbw= 53.0ft	
756.00	.82	751.98	Free	Free	.000	.000	Free Outfall	
		BACKWATER CONTROL..		Vh= .082ft	hwDi= .386ft		Lbw= 53.0ft	
756.10	.83	751.99	Free	Free	.000	.000	Free Outfall	
		BACKWATER CONTROL..		Vh= .083ft	hwDi= .388ft		Lbw= 53.0ft	
756.20	.84	751.99	Free	Free	.000	.000	Free Outfall	
		BACKWATER CONTROL..		Vh= .083ft	hwDi= .390ft		Lbw= 53.0ft	
756.30	.85	751.99	Free	Free	.000	.000	Free Outfall	
		BACKWATER CONTROL..		Vh= .084ft	hwDi= .392ft		Lbw= 53.0ft	
756.40	.86	751.99	Free	Free	.000	.000	Free Outfall	
		BACKWATER CONTROL..		Vh= .084ft	hwDi= .394ft		Lbw= 53.0ft	
756.50	.87	752.00	Free	Free	.000	.000	Free Outfall	
		BACKWATER CONTROL..		Vh= .084ft	hwDi= .398ft		Lbw= 53.0ft	
756.60	.88	752.00	Free	Free	.000	.000	Free Outfall	
		BACKWATER CONTROL..		Vh= .085ft	hwDi= .400ft		Lbw= 53.0ft	
756.70	.89	752.00	Free	Free	.000	.000	Free Outfall	
		BACKWATER CONTROL..		Vh= .086ft	hwDi= .401ft		Lbw= 53.0ft	
756.80	.90	752.01	Free	Free	.000	.000	Free Outfall	
		BACKWATER CONTROL..		Vh= .085ft	hwDi= .404ft		Lbw= 53.0ft	

File.... J:\PROJECT\2004\046119BelewsCreek\CELLS 3-4.PPW

RATING TABLE FOR ONE OUTLET TYPE

Structure ID = CV (Culvert-Circular)

Mannings open channel maximum capacity: 6.12 cfs
UPstream ID's= R1, O1
DNstream ID = TW (Pond Outfall)

Pond WS. Elev. ft	Device Q cfs	(into) HW HGL ft	Converge DS HGL ft	Next DS HGL ft	DS HGL Error +/-ft	Q SUM Error +/-cfs	DS Chan. TW ft	TW Error +/-ft
756.90	.91	752.01	Free	Free	.000	.000	Free Outfall	
		BACKWATER CONTROL..	Vh= .087ft	hwDi= .405ft	Lbw= 53.0ft			
757.00	.92	752.01	Free	Free	.000	.000	Free Outfall	
		BACKWATER CONTROL..	Vh= .086ft	hwDi= .409ft	Lbw= 53.0ft			
757.10	.93	752.01	Free	Free	.000	.000	Free Outfall	
		BACKWATER CONTROL..	Vh= .087ft	hwDi= .410ft	Lbw= 53.0ft			
757.20	.94	752.02	Free	Free	.000	.000	Free Outfall	
		BACKWATER CONTROL..	Vh= .088ft	hwDi= .412ft	Lbw= 53.0ft			
757.30	.95	752.02	Free	Free	.000	.000	Free Outfall	
		BACKWATER CONTROL..	Vh= .088ft	hwDi= .413ft	Lbw= 53.0ft			
757.40	.95	752.02	Free	Free	.000	.000	Free Outfall	
		BACKWATER CONTROL..	Vh= .089ft	hwDi= .416ft	Lbw= 53.0ft			
757.50	.96	752.03	Free	Free	.000	.000	Free Outfall	
		BACKWATER CONTROL..	Vh= .090ft	hwDi= .418ft	Lbw= 53.0ft			
757.60	.97	752.03	Free	Free	.000	.000	Free Outfall	
		BACKWATER CONTROL..	Vh= .090ft	hwDi= .419ft	Lbw= 53.0ft			
757.70	.98	752.03	Free	Free	.000	.000	Free Outfall	
		BACKWATER CONTROL..	Vh= .090ft	hwDi= .422ft	Lbw= 53.0ft			
757.80	.99	752.03	Free	Free	.000	.000	Free Outfall	
		BACKWATER CONTROL..	Vh= .090ft	hwDi= .424ft	Lbw= 53.0ft			
757.90	1.00	752.03	Free	Free	.000	.000	Free Outfall	
		BACKWATER CONTROL..	Vh= .091ft	hwDi= .425ft	Lbw= 53.0ft			
758.00	1.00	752.04	Free	Free	.000	.000	Free Outfall	
		BACKWATER CONTROL..	Vh= .090ft	hwDi= .428ft	Lbw= 53.0ft			
758.10	1.60	752.19	Free	Free	.000	.000	Free Outfall	
		BACKWATER CONTROL..	Vh= .118ft	hwDi= .544ft	Lbw= 53.0ft			
758.20	2.68	752.41	Free	Free	.000	.000	Free Outfall	
		BACKWATER CONTROL..	Vh= .156ft	hwDi= .724ft	Lbw= 53.0ft			
758.30	4.07	752.67	Free	Free	.000	.000	Free Outfall	
		BACKWATER CONTROL..	Vh= .191ft	hwDi= .937ft	Lbw= 53.0ft			
758.40	5.73	752.97	Free	Free	.000	.000	Free Outfall	
		BACKWATER CONTROL..	Vh= .221ft	hwDi= 1.203ft	Lbw= 53.0ft			
758.50	7.58	753.50	Free	Free	.000	.000	Free Outfall	
		FULL FLOW...Lfull=20.39ft	Vh=.286ft	HL=.706ft				
758.60	9.61	754.37	Free	Free	.000	.000	Free Outfall	
		FULL FLOW...Lfull=44.36ft	Vh=.459ft	HL=1.816ft				

RATING TABLE FOR ONE OUTLET TYPE

Structure ID = CV (Culvert-Circular)

Mannings open channel maximum capacity: 6.12 cfs
UPstream ID's= R1, O1
DNstream ID = TW (Pond Outfall)

Pond WS. Elev. ft	Device Q cfs	(into) HW HGL ft	Converge DS HGL ft	Next DS HGL ft	DS HGL Error +/-ft	Q SUM Error +/-cfs	DS Chan. TW ft	TW Error +/-ft
758.70	11.78	755.49	Free	Free	.000	.000	Free Outfall	
		FULL FLOW...Lfull=50.48ft			Vh=.690ft	HL=2.991ft		
758.80	14.06	756.84	Free	Free	.000	.000	Free Outfall	
		FULL FLOW...Lfull=52.08ft			Vh=.984ft	HL=4.363ft		
758.90	14.85	757.36	Free	Free	.000	.000	Free Outfall	
		FULL FLOW...Lfull=52.36ft			Vh=1.098ft	HL=4.887ft		
759.00	15.56	757.85	Free	Free	.000	.000	Free Outfall	
		FULL FLOW...Lfull=52.53ft			Vh=1.205ft	HL=5.376ft		
759.10	16.22	759.10	Free	Free	.000	.000	Free Outfall	
		FULL FLOW...Lfull=52.78ft			Vh=1.481ft	HL=6.628ft		
759.20	16.83	759.20	Free	Free	.000	.000	Free Outfall	
		FULL FLOW...Lfull=52.78ft			Vh=1.503ft	HL=6.727ft		
759.30	17.51	759.30	Free	Free	.000	.000	Free Outfall	
		FULL FLOW...Lfull=52.79ft			Vh=1.525ft	HL=6.827ft		
759.40	17.63	759.40	Free	Free	.000	.000	Free Outfall	
		FULL FLOW...Lfull=52.79ft			Vh=1.547ft	HL=6.927ft		
759.50	17.76	759.50	Free	Free	.000	.000	Free Outfall	
		FULL FLOW...Lfull=52.80ft			Vh=1.570ft	HL=7.029ft		
759.60	17.89	759.60	Free	Free	.000	.000	Free Outfall	
		FULL FLOW...Lfull=52.80ft			Vh=1.592ft	HL=7.128ft		
759.70	18.01	759.70	Free	Free	.000	.000	Free Outfall	
		FULL FLOW...Lfull=52.78ft			Vh=1.615ft	HL=7.228ft		
759.80	18.13	759.80	Free	Free	.000	.000	Free Outfall	
		FULL FLOW...Lfull=52.82ft			Vh=1.636ft	HL=7.328ft		
759.90	18.26	759.90	Free	Free	.000	.000	Free Outfall	
		FULL FLOW...Lfull=52.82ft			Vh=1.659ft	HL=7.429ft		
760.00	18.38	760.00	Free	Free	.000	.000	Free Outfall	
		FULL FLOW...Lfull=52.83ft			Vh=1.681ft	HL=7.528ft		

Type.... Composite Rating Curve
Name.... Outlet 1

File.... J:\PROJECT\2004\046119BelewsCreek\CELLS 3-4.PPW

***** COMPOSITE OUTFLOW SUMMARY ****

WS Elev, Total Q		Notes		
		Converge		
Elev. ft	Q cfs	TW Elev ft	Error +/-ft	Contributing Structures
751.50	.00	Free Outfall		(no Q: R1,O1,CV)
751.60	.02	Free Outfall	O1,CV	(no Q: R1)
751.70	.07	Free Outfall	O1,CV	(no Q: R1)
751.80	.14	Free Outfall	O1,CV	(no Q: R1)
751.90	.18	Free Outfall	O1,CV	(no Q: R1)
752.00	.21	Free Outfall	O1,CV	(no Q: R1)
752.10	.24	Free Outfall	O1,CV	(no Q: R1)
752.20	.27	Free Outfall	O1,CV	(no Q: R1)
752.30	.30	Free Outfall	O1,CV	(no Q: R1)
752.40	.32	Free Outfall	O1,CV	(no Q: R1)
752.50	.34	Free Outfall	O1,CV	(no Q: R1)
752.60	.36	Free Outfall	O1,CV	(no Q: R1)
752.70	.39	Free Outfall	O1,CV	(no Q: R1)
752.80	.40	Free Outfall	O1,CV	(no Q: R1)
752.90	.42	Free Outfall	O1,CV	(no Q: R1)
753.00	.44	Free Outfall	O1,CV	(no Q: R1)
753.10	.46	Free Outfall	O1,CV	(no Q: R1)
753.20	.48	Free Outfall	O1,CV	(no Q: R1)
753.30	.49	Free Outfall	O1,CV	(no Q: R1)
753.40	.51	Free Outfall	O1,CV	(no Q: R1)
753.50	.52	Free Outfall	O1,CV	(no Q: R1)
753.60	.54	Free Outfall	O1,CV	(no Q: R1)
753.70	.55	Free Outfall	O1,CV	(no Q: R1)
753.80	.57	Free Outfall	O1,CV	(no Q: R1)
753.90	.58	Free Outfall	O1,CV	(no Q: R1)
754.00	.60	Free Outfall	O1,CV	(no Q: R1)
754.10	.61	Free Outfall	O1,CV	(no Q: R1)
754.20	.62	Free Outfall	O1,CV	(no Q: R1)
754.30	.63	Free Outfall	O1,CV	(no Q: R1)
754.40	.65	Free Outfall	O1,CV	(no Q: R1)
754.50	.66	Free Outfall	O1,CV	(no Q: R1)
754.60	.67	Free Outfall	O1,CV	(no Q: R1)
754.70	.68	Free Outfall	O1,CV	(no Q: R1)
754.80	.70	Free Outfall	O1,CV	(no Q: R1)
754.90	.71	Free Outfall	O1,CV	(no Q: R1)
755.00	.72	Free Outfall	O1,CV	(no Q: R1)
755.10	.73	Free Outfall	O1,CV	(no Q: R1)
755.20	.74	Free Outfall	O1,CV	(no Q: R1)

Type.... Composite Rating Curve
Name.... Outlet 1

File.... J:\PROJECT\2004\046119BelewsCreek\CELLS 3-4.PPW

***** COMPOSITE OUTFLOW SUMMARY ****

WS Elev, Total Q		Converge		Notes
Elev. ft	Q cfs	TW Elev ft	Error +/-ft	Contributing Structures
755.30	.75	Free Outfall		O1,CV (no Q: R1)
755.40	.76	Free Outfall		O1,CV (no Q: R1)
755.50	.77	Free Outfall		O1,CV (no Q: R1)
755.60	.78	Free Outfall		O1,CV (no Q: R1)
755.70	.79	Free Outfall		O1,CV (no Q: R1)
755.80	.80	Free Outfall		O1,CV (no Q: R1)
755.90	.81	Free Outfall		O1,CV (no Q: R1)
756.00	.82	Free Outfall		O1,CV (no Q: R1)
756.10	.83	Free Outfall		O1,CV (no Q: R1)
756.20	.84	Free Outfall		O1,CV (no Q: R1)
756.30	.85	Free Outfall		O1,CV (no Q: R1)
756.40	.86	Free Outfall		O1,CV (no Q: R1)
756.50	.87	Free Outfall		O1,CV (no Q: R1)
756.60	.88	Free Outfall		O1,CV (no Q: R1)
756.70	.89	Free Outfall		O1,CV (no Q: R1)
756.80	.90	Free Outfall		O1,CV (no Q: R1)
756.90	.91	Free Outfall		O1,CV (no Q: R1)
757.00	.92	Free Outfall		O1,CV (no Q: R1)
757.10	.93	Free Outfall		O1,CV (no Q: R1)
757.20	.94	Free Outfall		O1,CV (no Q: R1)
757.30	.95	Free Outfall		O1,CV (no Q: R1)
757.40	.95	Free Outfall		O1,CV (no Q: R1)
757.50	.96	Free Outfall		O1,CV (no Q: R1)
757.60	.97	Free Outfall		O1,CV (no Q: R1)
757.70	.98	Free Outfall		O1,CV (no Q: R1)
757.80	.99	Free Outfall		O1,CV (no Q: R1)
757.90	1.00	Free Outfall		O1,CV (no Q: R1)
758.00	1.00	Free Outfall		O1,CV (no Q: R1)
758.10	1.60	Free Outfall		R1,O1,CV
758.20	2.68	Free Outfall		R1,O1,CV
758.30	4.07	Free Outfall		R1,O1,CV
758.40	5.73	Free Outfall		R1,O1,CV
758.50	7.58	Free Outfall		R1,O1,CV
758.60	9.61	Free Outfall		R1,O1,CV
758.70	11.78	Free Outfall		R1,O1,CV
758.80	14.06	Free Outfall		R1,O1,CV
758.90	14.85	Free Outfall		R1,O1,CV
759.00	15.56	Free Outfall		R1,O1,CV

Type.... Composite Rating Curve
Name.... Outlet 1

File.... J:\PROJECT\2004\046119BelewsCreek\CELLS 3-4.PPW

***** COMPOSITE OUTFLOW SUMMARY ****

WS Elev, Total Q		Converge		Notes
Elev. ft	Q cfs	TW Elev ft	Error +/-ft	Contributing Structures
759.10	16.22	Free Outfall		R1,CV (no Q: O1)
759.20	16.83	Free Outfall		R1,CV (no Q: O1)
759.30	17.51	Free Outfall		R1,CV (no Q: O1)
759.40	17.63	Free Outfall		R1,CV (no Q: O1)
759.50	17.76	Free Outfall		R1,CV (no Q: O1)
759.60	17.89	Free Outfall		R1,CV (no Q: O1)
759.70	18.01	Free Outfall		R1,CV (no Q: O1)
759.80	18.13	Free Outfall		R1,CV (no Q: O1)
759.90	18.26	Free Outfall		R1,CV (no Q: O1)
760.00	18.38	Free Outfall		R1,CV (no Q: O1)

Type.... Pond E-V-Q Table
Name.... POND 10
File.... J:\PROJECT\2004\046119BelewsCreek\CELLS 3-4.PPW

LEVEL POOL ROUTING DATA

HYG Dir = J:\PROJECT\2004\046119BelewsCreek\
Inflow HYG file = NONE STORED - POND 10 IN 2
Outflow HYG file = NONE STORED - POND 10 OUT 2

Pond Node Data = POND 10
Pond Volume Data = POND 10
Pond Outlet Data = Outlet 1

No Infiltration

INITIAL CONDITIONS

Starting WS Elev	=	751.50 ft
Starting Volume	=	.000 ac-ft
Starting Outflow	=	.00 cfs
Starting Infiltr.	=	.00 cfs
Starting Total Qout	=	.00 cfs
Time Increment	=	.0500 hrs

Elevation ft	Outflow cfs	Storage ac-ft	Area sq.ft	Infilt. cfs	Q Total cfs	2S/t + O cfs
751.50	.00	.000	100	.00	.00	.00
751.60	.02	.001	601	.00	.02	.37
751.70	.07	.003	1523	.00	.07	1.56
751.80	.14	.008	2866	.00	.14	4.03
751.90	.18	.017	4631	.00	.18	8.19
752.00	.21	.030	6816	.00	.21	14.55
752.10	.24	.046	7822	.00	.24	22.70
752.20	.27	.066	8898	.00	.27	32.02
752.30	.30	.087	10043	.00	.30	42.56
752.40	.32	.112	11258	.00	.32	54.41
752.50	.34	.139	12541	.00	.34	67.65
752.60	.36	.169	13894	.00	.36	82.34
752.70	.39	.203	15316	.00	.39	98.59
752.80	.40	.240	16807	.00	.40	116.45
752.90	.42	.280	18369	.00	.42	136.01
753.00	.44	.324	19998	.00	.44	157.33
753.10	.46	.372	21697	.00	.46	180.50
753.20	.48	.424	23466	.00	.48	205.61
753.30	.49	.480	25304	.00	.49	232.71
753.40	.51	.540	27212	.00	.51	261.91

Type.... Pond E-V-Q Table
Name.... POND 10
File.... J:\PROJECT\2004\046119BelewsCreek\CELLS 3-4.PPW

LEVEL POOL ROUTING DATA

HYG Dir = J:\PROJECT\2004\046119BelewsCreek\
Inflow HYG file = NONE STORED - POND 10 IN 2
Outflow HYG file = NONE STORED - POND 10 OUT 2

Pond Node Data = POND 10
Pond Volume Data = POND 10
Pond Outlet Data = Outlet 1

No Infiltration

INITIAL CONDITIONS

Starting WS Elev = 751.50 ft
Starting Volume = .000 ac-ft
Starting Outflow = .00 cfs
Starting Infiltr. = .00 cfs
Starting Total Qout= .00 cfs
Time Increment = .0500 hrs

Elevation ft	Outflow cfs	Storage ac-ft	Area sq.ft	Infilt. cfs	Q Total cfs	2S/t + O cfs
753.50	.52	.605	29188	.00	.52	293.24
753.60	.54	.674	31233	.00	.54	326.81
753.70	.55	.748	33349	.00	.55	362.71
753.80	.57	.827	35532	.00	.57	400.97
753.90	.58	.911	37787	.00	.58	441.73
754.00	.60	1.001	40109	.00	.60	485.00
754.10	.61	1.093	40436	.00	.61	529.75
754.20	.62	1.187	40765	.00	.62	574.89
754.30	.63	1.280	41094	.00	.63	620.37
754.40	.65	1.375	41425	.00	.65	666.24
754.50	.66	1.471	41758	.00	.66	712.46
754.60	.67	1.567	42091	.00	.67	759.04
754.70	.68	1.664	42427	.00	.68	806.02
754.80	.70	1.762	42763	.00	.70	853.35
754.90	.71	1.860	43101	.00	.71	901.08
755.00	.72	1.960	43440	.00	.72	949.16
755.10	.73	2.060	43780	.00	.73	997.61
755.20	.74	2.161	44122	.00	.74	1046.48
755.30	.75	2.262	44465	.00	.75	1095.69
755.40	.76	2.365	44809	.00	.76	1145.32

Type.... Pond E-V-Q Table
Name.... POND 10
File.... J:\PROJECT\2004\046119BelewsCreek\CELLS 3-4.PPW

LEVEL POOL ROUTING DATA

HYG Dir = J:\PROJECT\2004\046119BelewsCreek\
Inflow HYG file = NONE STORED - POND 10 IN 2
Outflow HYG file = NONE STORED - POND 10 OUT 2

Pond Node Data = POND 10
Pond Volume Data = POND 10
Pond Outlet Data = Outlet 1

No Infiltration

INITIAL CONDITIONS

Starting WS Elev	=	751.50 ft
Starting Volume	=	.000 ac-ft
Starting Outflow	=	.00 cfs
Starting Infiltr.	=	.00 cfs
Starting Total Qout	=	.00 cfs
Time Increment	=	.0500 hrs

Elevation ft	Outflow cfs	Storage ac-ft	Area sq.ft	Infilt. cfs	Q Total cfs	2S/t + O cfs
755.50	.77	2.468	45155	.00	.77	1195.30
755.60	.78	2.572	45502	.00	.78	1245.66
755.70	.79	2.677	45850	.00	.79	1296.44
755.80	.80	2.783	46200	.00	.80	1347.57
755.90	.81	2.889	46551	.00	.81	1399.13
756.00	.82	2.996	46903	.00	.82	1451.05
756.10	.83	3.104	47250	.00	.83	1503.35
756.20	.84	3.213	47598	.00	.84	1556.07
756.30	.85	3.323	47947	.00	.85	1609.15
756.40	.86	3.433	48297	.00	.86	1662.65
756.50	.87	3.545	48649	.00	.87	1716.50
756.60	.88	3.657	49002	.00	.88	1770.75
756.70	.89	3.770	49357	.00	.89	1825.42
756.80	.90	3.883	49712	.00	.90	1880.46
756.90	.91	3.998	50069	.00	.91	1935.92
757.00	.92	4.113	50427	.00	.92	1991.75
757.10	.93	4.229	50787	.00	.93	2047.97
757.20	.94	4.346	51147	.00	.94	2104.63
757.30	.95	4.464	51509	.00	.95	2161.66
757.40	.95	4.583	51873	.00	.95	2219.12

Type.... Pond E-V-Q Table
Name.... POND 10
File.... J:\PROJECT\2004\046119BelewsCreek\CELLS 3-4.PPW

LEVEL POOL ROUTING DATA

HYG Dir = J:\PROJECT\2004\046119BelewsCreek\
Inflow HYG file = NONE STORED - POND 10 IN 2
Outflow HYG file = NONE STORED - POND 10 OUT 2

Pond Node Data = POND 10
Pond Volume Data = POND 10
Pond Outlet Data = Outlet 1

No Infiltration

INITIAL CONDITIONS

Starting WS Elev	=	751.50 ft
Starting Volume	=	.000 ac-ft
Starting Outflow	=	.00 cfs
Starting Infiltr.	=	.00 cfs
Starting Total Qout	=	.00 cfs
Time Increment	=	.0500 hrs

Elevation ft	Outflow cfs	Storage ac-ft	Area sq.ft	Infilt. cfs	Q Total cfs	2S/t + O cfs
757.50	.96	4.702	52237	.00	.96	2276.95
757.60	.97	4.823	52603	.00	.97	2335.19
757.70	.98	4.944	52970	.00	.98	2393.87
757.80	.99	5.066	53338	.00	.99	2452.93
757.90	1.00	5.189	53708	.00	1.00	2512.43
758.00	1.00	5.313	54079	.00	1.00	2572.30
758.10	1.60	5.437	54445	.00	1.60	2633.17
758.20	2.68	5.563	54812	.00	2.68	2694.97
758.30	4.07	5.689	55180	.00	4.07	2757.46
758.40	5.73	5.816	55550	.00	5.73	2820.65
758.50	7.58	5.944	55920	.00	7.58	2884.42
758.60	9.61	6.073	56292	.00	9.61	2948.77
758.70	11.78	6.202	56665	.00	11.78	3013.72
758.80	14.06	6.333	57040	.00	14.06	3079.16
758.90	14.85	6.464	57416	.00	14.85	3143.56
759.00	15.56	6.596	57792	.00	15.56	3208.25
759.10	16.22	6.730	58170	.00	16.22	3273.32
759.20	16.83	6.864	58550	.00	16.83	3338.80
759.30	17.51	6.998	58930	.00	17.51	3404.72
759.40	17.63	7.134	59312	.00	17.63	3470.57

Type.... Pond E-V-Q Table
Name.... POND 10
File.... J:\PROJECT\2004\046119BelewsCreek\CELLS 3-4.PPW

LEVEL POOL ROUTING DATA

HYG Dir = J:\PROJECT\2004\046119BelewsCreek\
Inflow HYG file = NONE STORED - POND 10 IN 2
Outflow HYG file = NONE STORED - POND 10 OUT 2

Pond Node Data = POND 10
Pond Volume Data = POND 10
Pond Outlet Data = Outlet 1

No Infiltration

INITIAL CONDITIONS

Starting WS Elev	=	751.50 ft
Starting Volume	=	.000 ac-ft
Starting Outflow	=	.00 cfs
Starting Infiltr.	=	.00 cfs
Starting Total Qout	=	.00 cfs
Time Increment	=	.0500 hrs

Elevation ft	Outflow cfs	Storage ac-ft	Area sq.ft	Infilt. cfs	Q Total cfs	2S/t + O cfs
759.50	17.76	7.271	59695	.00	17.76	3536.79
759.60	17.89	7.408	60079	.00	17.89	3603.44
759.70	18.01	7.547	60465	.00	18.01	3670.56
759.80	18.13	7.686	60852	.00	18.13	3738.07
759.90	18.26	7.826	61240	.00	18.26	3806.04
760.00	18.38	7.967	61629	.00	18.38	3874.41

SUMMARY FOR HYDROGRAPH ADDITION
at Node: POND 10 IN

HYG Directory: J:\PROJECT\2004\046119BelewsCreek\

Upstream Link ID	Upstream Node ID	HYG file	HYG ID	HYG tag
ADDLINK 10	SUBAREA 10		SUBAREA 10	2

INFLOWS TO: POND 10 IN

HYG file	HYG ID	HYG tag	Volume ac-ft	Peak Time hrs	Peak Flow cfs
	SUBAREA 10	2	2.728	11.9500	46.24

TOTAL FLOW INTO: POND 10 IN

HYG file	HYG ID	HYG tag	Volume ac-ft	Peak Time hrs	Peak Flow cfs
	POND 10 IN	2	2.728	11.9500	46.24

TOTAL NODE INFLOW...
HYG file =
HYG ID = POND 10 IN
HYG Tag = 2

Peak Discharge	46.24 cfs
Time to Peak	11.9500 hrs
HYG Volume	2.728 ac-ft

HYDROGRAPH ORDINATES (cfs)
Output Time increment = .0500 hrs
Time on left represents time for first value in each row.

Time hrs					
5.1000	.00	.00	.01	.01	.01
5.3500	.02	.02	.03	.03	.03
5.6000	.04	.04	.05	.05	.06
5.8500	.06	.06	.07	.07	.08
6.1000	.08	.09	.09	.10	.10
6.3500	.11	.11	.12	.12	.12
6.6000	.13	.13	.14	.14	.15
6.8500	.15	.16	.17	.17	.18
7.1000	.18	.19	.19	.20	.20
7.3500	.21	.21	.22	.22	.23
7.6000	.23	.24	.25	.25	.26
7.8500	.26	.27	.27	.28	.29
8.1000	.29	.30	.31	.32	.34
8.3500	.35	.36	.37	.39	.40
8.6000	.41	.43	.44	.46	.47
8.8500	.48	.50	.52	.53	.55
9.1000	.56	.57	.58	.59	.59
9.3500	.60	.61	.61	.62	.63
9.6000	.64	.66	.68	.70	.73
9.8500	.75	.78	.80	.83	.86
10.1000	.89	.92	.96	.99	1.03
10.3500	1.07	1.11	1.15	1.19	1.23
10.6000	1.29	1.34	1.40	1.46	1.53
10.8500	1.60	1.67	1.74	1.82	1.90
11.1000	2.01	2.13	2.28	2.43	2.60
11.3500	2.76	2.94	3.11	3.31	4.02
11.6000	5.67	8.02	11.75	15.94	21.49
11.8500	28.79	40.45	46.24	43.47	36.06
12.1000	22.71	13.55	9.65	7.87	6.84
12.3500	6.19	5.62	5.16	4.66	4.27
12.6000	3.91	3.68	3.51	3.39	3.26

HYDROGRAPH ORDINATES (cfs)
Output Time increment = .0500 hrs
Time on left represents time for first value in each row.

Time hrs					
12.8500	3.16	3.04	2.94	2.83	2.73
13.1000	2.64	2.56	2.49	2.44	2.37
13.3500	2.32	2.26	2.20	2.14	2.08
13.6000	2.03	1.98	1.94	1.90	1.85
13.8500	1.81	1.77	1.73	1.68	1.65
14.1000	1.61	1.59	1.57	1.55	1.54
14.3500	1.52	1.51	1.49	1.48	1.46
14.6000	1.45	1.43	1.42	1.41	1.39
14.8500	1.37	1.36	1.35	1.33	1.32
15.1000	1.30	1.29	1.27	1.26	1.24
15.3500	1.23	1.21	1.20	1.18	1.17
15.6000	1.15	1.14	1.12	1.11	1.09
15.8500	1.08	1.06	1.05	1.03	1.02
16.1000	1.01	1.00	.99	.99	.98
16.3500	.98	.97	.96	.96	.95
16.6000	.95	.94	.94	.93	.93
16.8500	.92	.92	.91	.91	.90
17.1000	.90	.89	.88	.88	.87
17.3500	.87	.86	.86	.85	.85
17.6000	.84	.84	.83	.83	.82
17.8500	.81	.81	.80	.80	.79
18.1000	.79	.78	.78	.77	.77
18.3500	.76	.76	.75	.75	.74
18.6000	.73	.73	.72	.72	.71
18.8500	.71	.70	.70	.69	.69
19.1000	.68	.68	.67	.66	.66
19.3500	.65	.65	.64	.64	.63
19.6000	.63	.62	.62	.61	.60
19.8500	.60	.59	.59	.58	.58
20.1000	.57	.57	.57	.57	.57
20.3500	.57	.56	.56	.56	.56
20.6000	.56	.56	.56	.56	.56
20.8500	.55	.55	.55	.55	.55
21.1000	.55	.55	.55	.55	.55
21.3500	.55	.54	.54	.54	.54
21.6000	.54	.54	.54	.54	.53
21.8500	.53	.53	.53	.53	.53
22.1000	.53	.53	.53	.53	.52
22.3500	.52	.52	.52	.52	.52
22.6000	.52	.52	.52	.52	.51
22.8500	.51	.51	.51	.51	.51
23.1000	.51	.51	.51	.50	.50
23.3500	.50	.50	.50	.50	.50
23.6000	.50	.50	.49	.49	.49
23.8500	.49	.49	.49	.49	.42

HYDROGRAPH ORDINATES (cfs)
Output Time increment = .0500 hrs
Time on left represents time for first value in each row.

Time hrs					
24.1000	.22	.09	.03	.01	.00
24.3500	.00	.00			

SUMMARY FOR HYDROGRAPH ADDITION
at Node: POND 10 IN

HYG Directory: J:\PROJECT\2004\046119BelewsCreek\

Upstream Link ID	Upstream Node ID	HYG file	HYG ID	HYG tag
ADDLINK 10	SUBAREA 10		SUBAREA 10	25

INFLOWS TO: POND 10 IN

HYG file	HYG ID	HYG tag	Volume ac-ft	Peak Time hrs	Peak Flow cfs
	SUBAREA 10	25	5.902	11.9500	96.22

TOTAL FLOW INTO: POND 10 IN

HYG file	HYG ID	HYG tag	Volume ac-ft	Peak Time hrs	Peak Flow cfs
	POND 10 IN	25	5.902	11.9500	96.22

TOTAL NODE INFLOW...
HYG file =
HYG ID = POND 10 IN
HYG Tag = 25

Peak Discharge =	96.22 cfs
Time to Peak =	11.9500 hrs
HYG Volume =	5.902 ac-ft

HYDROGRAPH ORDINATES (cfs)
Output Time increment = .0500 hrs
Time on left represents time for first value in each row.

Time hrs					
3.0500	.00	.00	.01	.02	.03
3.3000	.04	.04	.05	.06	.07
3.5500	.08	.09	.10	.10	.11
3.8000	.12	.13	.14	.15	.16
4.0500	.16	.17	.18	.19	.20
4.3000	.21	.22	.23	.24	.25
4.5500	.26	.27	.28	.29	.30
4.8000	.31	.32	.33	.34	.35
5.0500	.36	.37	.38	.39	.40
5.3000	.41	.42	.43	.44	.46
5.5500	.47	.48	.49	.50	.51
5.8000	.52	.53	.54	.55	.57
6.0500	.58	.59	.60	.61	.62
6.3000	.63	.65	.66	.67	.68
6.5500	.69	.70	.72	.73	.74
6.8000	.75	.76	.77	.79	.80
7.0500	.81	.82	.83	.85	.86
7.3000	.87	.88	.89	.91	.92
7.5500	.93	.94	.95	.97	.98
7.8000	.99	1.00	1.01	1.03	1.04
8.0500	1.05	1.07	1.10	1.13	1.16
8.3000	1.19	1.22	1.26	1.29	1.33
8.5500	1.36	1.40	1.43	1.47	1.50
8.8000	1.54	1.58	1.62	1.65	1.69
9.0500	1.72	1.75	1.78	1.79	1.80
9.3000	1.81	1.83	1.84	1.85	1.86
9.5500	1.87	1.90	1.94	1.99	2.05
9.8000	2.11	2.16	2.22	2.28	2.34
10.0500	2.41	2.48	2.56	2.64	2.73
10.3000	2.82	2.91	3.00	3.09	3.19
10.5500	3.29	3.41	3.53	3.68	3.82

HYDROGRAPH ORDINATES (cfs)
Output Time increment = .0500 hrs
Time on left represents time for first value in each row.

Time hrs					
10.8000	3.98	4.13	4.29	4.44	4.61
11.0500	4.79	5.04	5.30	5.66	6.00
11.3000	6.38	6.74	7.13	7.49	7.92
11.5500	9.53	13.34	18.64	26.92	35.90
11.8000	47.50	62.32	85.66	96.22	89.20
12.0500	73.23	45.87	27.25	19.33	15.71
12.3000	13.61	12.30	11.15	10.23	9.24
12.5500	8.45	7.74	7.27	6.93	6.69
12.8000	6.44	6.23	6.00	5.80	5.57
13.0500	5.38	5.19	5.04	4.91	4.79
13.3000	4.67	4.56	4.43	4.32	4.20
13.5500	4.09	3.98	3.89	3.80	3.72
13.8000	3.63	3.55	3.46	3.38	3.29
14.0500	3.22	3.16	3.11	3.07	3.04
14.3000	3.01	2.98	2.95	2.92	2.89
14.5500	2.86	2.83	2.80	2.77	2.75
14.8000	2.71	2.68	2.65	2.63	2.59
15.0500	2.57	2.54	2.51	2.48	2.45
15.3000	2.42	2.39	2.36	2.33	2.30
15.5500	2.28	2.24	2.21	2.18	2.16
15.8000	2.12	2.10	2.07	2.04	2.01
16.0500	1.98	1.96	1.94	1.93	1.92
16.3000	1.91	1.90	1.88	1.87	1.86
16.5500	1.85	1.84	1.83	1.82	1.81
16.8000	1.80	1.79	1.78	1.77	1.76
17.0500	1.75	1.74	1.73	1.72	1.71
17.3000	1.70	1.69	1.67	1.66	1.65
17.5500	1.64	1.63	1.62	1.61	1.60
17.8000	1.59	1.58	1.57	1.56	1.55
18.0500	1.54	1.53	1.52	1.51	1.50
18.3000	1.49	1.48	1.46	1.45	1.44
18.5500	1.43	1.42	1.41	1.40	1.39
18.8000	1.38	1.37	1.36	1.35	1.34
19.0500	1.33	1.32	1.31	1.29	1.28
19.3000	1.27	1.27	1.25	1.24	1.23
19.5500	1.22	1.21	1.20	1.19	1.18
19.8000	1.17	1.16	1.15	1.14	1.13
20.0500	1.12	1.11	1.11	1.10	1.10
20.3000	1.10	1.10	1.09	1.09	1.09
20.5500	1.09	1.08	1.08	1.08	1.08
20.8000	1.07	1.07	1.07	1.07	1.07
21.0500	1.06	1.06	1.06	1.06	1.06
21.3000	1.05	1.05	1.05	1.05	1.05
21.5500	1.04	1.04	1.04	1.04	1.04
21.8000	1.03	1.03	1.03	1.03	1.02

HYDROGRAPH ORDINATES (cfs)
Output Time increment = .0500 hrs
Time on left represents time for first value in each row.

Time hrs				
22.0500	1.02	1.02	1.02	1.02	1.01
22.3000	1.01	1.01	1.01	1.01	1.00
22.5500	1.00	1.00	1.00	1.00	.99
22.8000	.99	.99	.99	.99	.98
23.0500	.98	.98	.98	.97	.97
23.3000	.97	.97	.97	.96	.96
23.5500	.96	.96	.95	.95	.95
23.8000	.95	.95	.95	.94	.94
24.0500	.80	.42	.16	.06	.02
24.3000	.01	.00	.00		

SUMMARY FOR HYDROGRAPH ADDITION
at Node: POND 10 IN

HYG Directory: J:\PROJECT\2004\046119BelewsCreek\

Upstream Link ID	Upstream Node ID	HYG file	HYG ID	HYG tag
ADDLINK 10	SUBAREA 10		SUBAREA 10	100

INFLOWS TO: POND 10 IN

HYG file	HYG ID	HYG tag	Volume ac-ft	Peak Time hrs	Peak Flow cfs
	SUBAREA 10	100	7.373	11.9500	118.66

TOTAL FLOW INTO: POND 10 IN

HYG file	HYG ID	HYG tag	Volume ac-ft	Peak Time hrs	Peak Flow cfs
	POND 10 IN	100	7.373	11.9500	118.66

TOTAL NODE INFLOW...
HYG file =
HYG ID = POND 10 IN
HYG Tag = 100

Peak Discharge = 118.66 cfs
Time to Peak = 11.9500 hrs
HYG Volume = 7.373 ac-ft

HYDROGRAPH ORDINATES (cfs)
Output Time increment = .0500 hrs
Time on left represents time for first value in each row.

Time hrs					
2.5500	.00	.00	.01	.02	.03
2.8000	.04	.05	.06	.07	.09
3.0500	.10	.11	.12	.13	.14
3.3000	.16	.17	.18	.19	.20
3.5500	.21	.22	.23	.25	.26
3.8000	.27	.28	.29	.30	.31
4.0500	.32	.33	.35	.36	.37
4.3000	.38	.40	.41	.42	.44
4.5500	.45	.46	.47	.49	.50
4.8000	.51	.53	.54	.55	.57
5.0500	.58	.59	.61	.62	.63
5.3000	.65	.66	.68	.69	.70
5.5500	.72	.73	.75	.76	.77
5.8000	.79	.80	.82	.83	.85
6.0500	.86	.87	.89	.90	.92
6.3000	.93	.95	.96	.97	.99
6.5500	1.00	1.02	1.03	1.05	1.06
6.8000	1.08	1.09	1.11	1.12	1.14
7.0500	1.15	1.16	1.18	1.19	1.21
7.3000	1.22	1.24	1.25	1.27	1.28
7.5500	1.30	1.31	1.33	1.34	1.36
7.8000	1.37	1.38	1.40	1.41	1.43
8.0500	1.45	1.47	1.50	1.54	1.58
8.3000	1.63	1.67	1.71	1.75	1.80
8.5500	1.84	1.89	1.93	1.98	2.03
8.8000	2.08	2.12	2.17	2.22	2.27
9.0500	2.31	2.34	2.37	2.39	2.40
9.3000	2.41	2.43	2.44	2.45	2.46
9.5500	2.48	2.51	2.56	2.63	2.69
9.8000	2.77	2.84	2.92	2.99	3.07
10.0500	3.15	3.25	3.34	3.45	3.56

HYDROGRAPH ORDINATES (cfs)
Output Time increment = .0500 hrs
Time on left represents time for first value in each row.

Time hrs					
10.3000	3.67	3.78	3.90	4.01	4.14
10.5500	4.26	4.41	4.56	4.75	4.93
10.8000	5.13	5.31	5.52	5.70	5.91
11.0500	6.14	6.45	6.78	7.23	7.65
11.3000	8.13	8.57	9.06	9.51	10.04
11.5500	12.06	16.85	23.49	33.82	44.96
11.8000	59.26	77.44	105.99	118.66	109.71
12.0500	89.91	56.25	33.39	23.67	19.22
12.3000	16.65	15.05	13.64	12.51	11.29
12.5500	10.33	9.46	8.88	8.47	8.17
12.8000	7.87	7.61	7.33	7.08	6.80
13.0500	6.57	6.34	6.16	5.99	5.85
13.3000	5.69	5.56	5.41	5.27	5.12
13.5500	4.99	4.86	4.75	4.63	4.53
13.8000	4.42	4.33	4.22	4.12	4.02
14.0500	3.93	3.85	3.79	3.74	3.70
14.3000	3.67	3.63	3.59	3.56	3.52
14.5500	3.49	3.45	3.41	3.38	3.35
14.8000	3.31	3.27	3.24	3.20	3.16
15.0500	3.13	3.09	3.06	3.02	2.98
15.3000	2.95	2.92	2.88	2.84	2.81
15.5500	2.77	2.73	2.70	2.66	2.63
15.8000	2.59	2.55	2.52	2.49	2.44
16.0500	2.41	2.39	2.37	2.35	2.33
16.3000	2.32	2.31	2.30	2.28	2.27
16.5500	2.26	2.24	2.23	2.22	2.21
16.8000	2.19	2.18	2.17	2.16	2.14
17.0500	2.13	2.12	2.11	2.09	2.08
17.3000	2.07	2.05	2.04	2.03	2.01
17.5500	2.00	1.99	1.98	1.96	1.95
17.8000	1.94	1.92	1.91	1.90	1.88
18.0500	1.87	1.86	1.85	1.83	1.82
18.3000	1.81	1.80	1.78	1.77	1.76
18.5500	1.75	1.73	1.72	1.71	1.69
18.8000	1.68	1.67	1.65	1.64	1.63
19.0500	1.62	1.60	1.59	1.58	1.56
19.3000	1.55	1.54	1.52	1.51	1.50
19.5500	1.49	1.47	1.46	1.45	1.44
19.8000	1.42	1.41	1.40	1.39	1.37
20.0500	1.36	1.35	1.35	1.34	1.34
20.3000	1.33	1.33	1.33	1.33	1.32
20.5500	1.32	1.32	1.32	1.31	1.31
20.8000	1.31	1.31	1.30	1.30	1.30
21.0500	1.29	1.29	1.29	1.29	1.28
21.3000	1.28	1.28	1.28	1.27	1.27

HYDROGRAPH ORDINATES (cfs)
Output Time increment = .0500 hrs
Time on left represents time for first value in each row.

Time hrs					
21.5500	1.27	1.27	1.26	1.26	1.26
21.8000	1.26	1.25	1.25	1.25	1.25
22.0500	1.24	1.24	1.24	1.24	1.23
22.3000	1.23	1.23	1.23	1.22	1.22
22.5500	1.22	1.22	1.21	1.21	1.21
22.8000	1.21	1.20	1.20	1.20	1.20
23.0500	1.19	1.19	1.19	1.18	1.18
23.3000	1.18	1.18	1.17	1.17	1.17
23.5500	1.17	1.16	1.16	1.16	1.16
23.8000	1.15	1.15	1.15	1.15	1.14
24.0500	.97	.51	.20	.08	.03
24.3000	.01	.00	.00		

LEVEL POOL ROUTING SUMMARY

HYG Dir = J:\PROJECT\2004\046119BelewsCreek\
Inflow HYG file = NONE STORED - POND 10 IN 2
Outflow HYG file = NONE STORED - POND 10 OUT 2

Pond Node Data = POND 10
Pond Volume Data = POND 10
Pond Outlet Data = Outlet 1

No Infiltration

INITIAL CONDITIONS

Starting WS Elev	=	751.50 ft
Starting Volume	=	.000 ac-ft
Starting Outflow	=	.00 cfs
Starting Infiltr.	=	.00 cfs
Starting Total Qout	=	.00 cfs
Time Increment	=	.0500 hrs

INFLOW/OUTFLOW HYDROGRAPH SUMMARY

Peak Inflow	=	46.24 cfs	at	11.9500 hrs
Peak Outflow	=	.72 cfs	at	16.4500 hrs
Peak Elevation	=	755.01 ft		
Peak Storage	=	1.974 ac-ft		

MASS BALANCE (ac-ft)

+ Initial Vol	=	.000
+ HYG Vol IN	=	2.728
- Infiltration	=	.000
- HYG Vol OUT	=	2.728
- Retained Vol	=	.000
Unrouted Vol	=	-.000 ac-ft (.002% of Inflow Volume)

LEVEL POOL ROUTING SUMMARY

HYG Dir = J:\PROJECT\2004\046119BelewsCreek\
Inflow HYG file = NONE STORED - POND 10 IN 25
Outflow HYG file = NONE STORED - POND 10 OUT 25

Pond Node Data = POND 10
Pond Volume Data = POND 10
Pond Outlet Data = Outlet 1

No Infiltration

INITIAL CONDITIONS

Starting WS Elev	=	751.50 ft
Starting Volume	=	.000 ac-ft
Starting Outflow	=	.00 cfs
Starting Infiltr.	=	.00 cfs
Starting Total Qout	=	.00 cfs
Time Increment	=	.0500 hrs

INFLOW/OUTFLOW HYDROGRAPH SUMMARY

Peak Inflow	=	96.22 cfs	at	11.9500 hrs
Peak Outflow	=	.97 cfs	at	19.6500 hrs
Peak Elevation	=	757.52 ft		
Peak Storage	=	4.729 ac-ft		

MASS BALANCE (ac-ft)

+ Initial Vol	=	.000
+ HYG Vol IN	=	5.902
- Infiltration	=	.000
- HYG Vol OUT	=	5.776
- Retained Vol	=	.126
Unrouted Vol	=	-.000 ac-ft (.006% of Inflow Volume)

LEVEL POOL ROUTING SUMMARY

HYG Dir = J:\PROJECT\2004\046119BelewsCreek\
Inflow HYG file = NONE STORED - POND 10 IN 100
Outflow HYG file = NONE STORED - POND 10 OUT 100

Pond Node Data = POND 10
Pond Volume Data = POND 10
Pond Outlet Data = Outlet 1

No Infiltration

INITIAL CONDITIONS

Starting WS Elev	=	751.50 ft
Starting Volume	=	.000 ac-ft
Starting Outflow	=	.00 cfs
Starting Infiltr.	=	.00 cfs
Starting Total Qout	=	.00 cfs
Time Increment	=	.0500 hrs

INFLOW/OUTFLOW HYDROGRAPH SUMMARY

Peak Inflow	=	118.66 cfs	at	11.9500 hrs
Peak Outflow	=	2.52 cfs	at	15.8000 hrs
Peak Elevation	=	758.19 ft		
Peak Storage	=	5.545 ac-ft		

MASS BALANCE (ac-ft)

+ Initial Vol	=	.000
+ HYG Vol IN	=	7.373
- Infiltration	=	.000
- HYG Vol OUT	=	6.965
- Retained Vol	=	.407
Unrouted Vol	=	-.000 ac-ft (.006% of Inflow Volume)

Index of Starting Page Numbers for ID Names

----- C -----
CMLDS-24HR... 3.01, 3.02

----- O -----
Outlet 1... 8.01, 8.04, 8.19

----- P -----
POND 10... 7.01, 9.01
POND 10 IN 2... 9.06, 9.10,
9.14, 9.18, 9.19, 9.20

----- S -----
SUBAREA 10... 4.01, 5.01, 6.03,
6.04, 6.05

----- W -----
Watershed... 1.01, 2.01, 2.02, 2.03,
2.04

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Consulting Engineers, Surveyors & Photogrammetrists

Comp. By DHB Date 12-6-04 Project BELEWS CREEK LANDFILL Sheet No. ..... of .....
Chkd. By DEB Date 12-7-04 Subject STILLING BASIN CALC. Job No. 046119

CALC. DETENTION TIMES:

2YR PEAK ELEV. = 755.01 SA. = 43,492 ±
25YR PEAK ELEV. = 757.52 SA. = 52,329 ±

$d^{4"} = .0873$ ±

$t^{2YR} = \frac{2(43,492)\sqrt{3.51}}{0.6(.0873)\sqrt{64.4}} = 387,690 \text{ sec} = 4.49 \text{ DAYS.}$

$t^{25YR} = \frac{2(52,329)\sqrt{6.02}}{0.6(.0873)\sqrt{64.4}} = 610,889 \text{ sec} = 7.07 \text{ DAYS.}$

0.96 qpm = 431 qpm

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Comp. By DHS Date 12-2-04 Project BELLEWS CREEK LANDFILL Sheet No. ..... of .....
Chkd. By ..... Date ..... Subject STILLING BASIN CALCS. Job No. 846119

ANTI-FLOTATION BLOCK:

$F\uparrow = \frac{\pi(2)^2}{4}(6.5)(62.4) = 1,274\ lb\uparrow$

VOL. CONC. $= \frac{1274}{150}(1.3) = 11.04\ ft^3.$

PROVIDE FTG. $3.5' \times 3.5' \times 1'$ DEEP $= 12.25\ ft^3.$

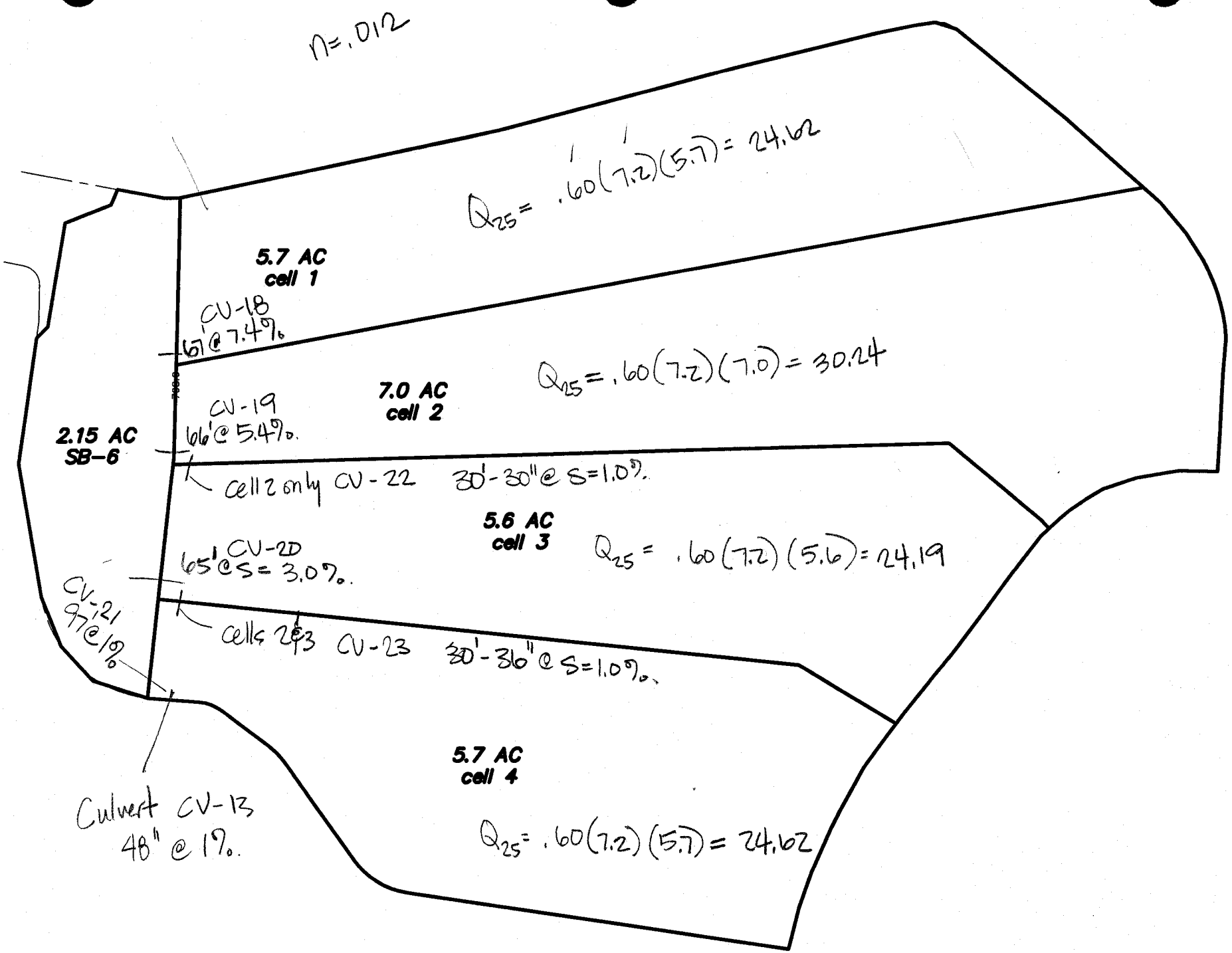
$F\downarrow = 12.25(150) = 1,837.5\ lb\downarrow$

$F.S. = \frac{1837.5}{1274} = 1.44$ OK.

Stormwater and Erosion and Sediment Control Calculations

- **Stormwater Calculations**

- **Note: Erosion and Sediment Control Calculations for the Belews Creek FGD Residue Landfill and the Soil Stockpile Storage Areas were submitted to NCDENR Land Quality Section. Erosion and Sediment Control Permits # Stoke 2005-007 and Stoke 2006-001 were issued as a result. Copies of these calculations can be furnished if requested.**



Size SWCR Culverts for Q_{25}

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Comp. By DHB Date 9-30-05 Project BELEWS CREEK LANDFILL Sheet No. ____ of ____
Chkd. By ____ Date ____ Subject CELL RISER CALCS. Job No. 046119

CELL 1 D.A. = 5.7 Ac.
C = .60
I = 7.2 in/hr

$Q_{25} = 0.6(7.2)(5.7) = 24.62$ cfs.

A_s @ 766.0 @ TOP OF LINER = 50,596 ± +/-
$h = 766 - 762.40 = 3.60$ ft.

$$A_o = \frac{A_s\sqrt{2h}}{T\,C_d\,(20{,}428)}$$ Assume T = 24 Hr.

$$A_o = \frac{50{,}596\sqrt{2(3.6)}}{24(.6)(20{,}428)} = 0.4615 \text{ ft} = 66.5 \text{ in}^2.$$

USE 3/4" ϕ HOLES $a = \frac{\pi(.75)^2}{4} = 0.442 \text{ in}^2$

HOLES REQ'D = 151

CIRCUMFERENCE OF RISER = $\pi(36") = 113.1$ in

SPACE HOLES 4" c/c E.W.

28 HOLES PER ROW x 6 ROWS = 168 HOLES = 74 in^2

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Comp. By DHB Date 9-30-05 Project BELEWS CREEK LANDFILL Sheet No. ____ of ____
Chkd. By ____ Date ____ Subject CELL RISER CALCS Job No. 046119

CELL 2 D.A. = 7.0 Ac.
C = .60
I = 7.2 in/hr.

$Q_{25} = 0.6(7.2)(7.0) = 30.24$ cfs.

A_s @ ELEV. 765 JUST ABOVE RISER CREST = 31,553 +/-
h = 765 - 761.22 = 3.78 ft.

$$A_o = \frac{31{,}553\sqrt{2(3.78)}}{24(.6)(20{,}428)} = 0.295 \text{ ft} < \text{CELL 1}$$

CELL 3 D.A. = 5.6 Ac.

$Q_{25} = 0.6(7.2)(5.6) = 24.19$ cfs.

A_s @ ELEV. 763 = 26106 +/-
h = 763 - 759.49 = 3.51 ft.

$$A_o = \frac{26106\sqrt{2(3.51)}}{24(.6)(20{,}428)} = 0.235 \text{ ft} < \text{CELL 1}$$

CELL 4 D.A. = 5.7 Ac.

$Q_{25} = 0.6(7.2)(5.7) = 24.62$ cfs.

A_s @ ELEV. 762.5 = 10937 +/-
h = 762.5 - 758.62 = 3.88 ft.

$$A_o = \frac{10{,}937\sqrt{2(3.88)}}{24(.6)(20{,}428)} = 0.104 \text{ ft} < \text{CELL 1}$$

Worksheet
Worksheet for Circular Channel

Project Description	
Worksheet	CV-18
Flow Element	Circular Chann
Method	Manning's For
Solve For	Channel Depth

Input Data		
Mannings Coeffic	0.012	
Slope	076000	ft/ft
Diameter	18	in
Discharge	24.62	cfs

Results		
Depth	1.00	ft
Flow Area	1.3	ft²
Wetted Perime	2.87	ft
Top Width	1.41	ft
Critical Depth	1.49	ft
Percent Full	66.7	%
Critical Slope	0.043215	ft/ft
Velocity	19.65	ft/s
Velocity Head	6.00	ft
Specific Energ	7.00	ft
Froude Numbe	3.68	
Maximum Disc	33.74	cfs
Discharge Full	31.37	cfs
Slope Full	0.046812	ft/ft
Flow Type	Supercritical	

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Worksheet
Worksheet for Circular Channel

Project Description	
Worksheet	CV-19
Flow Element	Circular Chann
Method	Manning's For
Solve For	Channel Depth

Input Data		
Mannings Coeffic	0.012	
Slope	052000	ft/ft
Diameter	24	in
Discharge	30.24	cfs

Results		
Depth	1.05	ft
Flow Area	1.7	ft²
Wetted Perime	3.24	ft
Top Width	2.00	ft
Critical Depth	1.87	ft
Percent Full	52.4	%
Critical Slope	0.013164	ft/ft
Velocity	18.14	ft/s
Velocity Head	5.11	ft
Specific Energ	6.16	ft
Froude Numbe	3.50	
Maximum Disc	60.11	cfs
Discharge Full	55.88	cfs
Slope Full	0.015227	ft/ft
Flow Type	Supercritical	

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Worksheet
Worksheet for Circular Channel

Project Description	
Worksheet	CV-20
Flow Element	Circular Chann
Method	Manning's For
Solve For	Channel Depth

Input Data		
Mannings Coeffic	0.012	
Slope	024000	ft/ft
Diameter	24	in
Discharge	24.19	cfs

Results		
Depth	1.16	ft
Flow Area	1.9	ft²
Wetted Perime	3.46	ft
Top Width	1.97	ft
Critical Depth	1.74	ft
Percent Full	58.0	%
Critical Slope	0.008883	ft/ft
Velocity	12.81	ft/s
Velocity Head	2.55	ft
Specific Energ	3.71	ft
Froude Numbe	2.31	
Maximum Disc	40.84	cfs
Discharge Full	37.96	cfs
Slope Full	0.009744	ft/ft
Flow Type	Supercritical	

Worksheet
Worksheet for Circular Channel

Project Description	
Worksheet	CV-21
Flow Element	Circular Chann
Method	Manning's For
Solve For	Channel Depth

Input Data		
Mannings Coeffic	0.012	
Slope	010000	ft/ft
Diameter	24	in
Discharge	24.62	cfs

Results		
Depth	1.65	ft
Flow Area	2.8	ft²
Wetted Perime	4.55	ft
Top Width	1.52	ft
Critical Depth	1.75	ft
Percent Full	82.4	%
Critical Slope	0.009129	ft/ft
Velocity	8.89	ft/s
Velocity Head	1.23	ft
Specific Energ	2.88	ft
Froude Numbe	1.16	
Maximum Disc	26.36	cfs
Discharge Full	24.51	cfs
Slope Full	0.010093	ft/ft
Flow Type	Supercritical	

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Worksheet
Worksheet for Circular Channel

Project Description	
Worksheet	CV-22
Flow Element	Circular Chann
Method	Manning's For
Solve For	Channel Depth

Input Data		
Mannings Coeffic	0.012	
Slope	010000	ft/ft
Diameter	30	in
Discharge	30.30	cfs

Results		
Depth	1.51	ft
Flow Area	3.1	ft²
Wetted Perime	4.46	ft
Top Width	2.44	ft
Critical Depth	1.88	ft
Percent Full	60.6	%
Critical Slope	0.005583	ft/ft
Velocity	9.74	ft/s
Velocity Head	1.47	ft
Specific Energ	2.99	ft
Froude Numbe	1.52	
Maximum Disc	47.80	cfs
Discharge Full	44.43	cfs
Slope Full	0.004650	ft/ft
Flow Type	Supercritical	

30' - 30" ϕ HDPE @ S = 1.0%

Worksheet
Worksheet for Circular Channel

Project Description	
Worksheet	CV-23
Flow Element	Circular Chann
Method	Manning's For
Solve For	Channel Depth

Input Data		
Mannings Coeffic	0.012	
Slope	010000	ft/ft
Diameter	36	in
Discharge	54.50	cfs

30'- 36" ϕ HDPE @ S = 1.0%.

Results		
Depth	1.95	ft
Flow Area	4.9	ft²
Wetted Perime	5.62	ft
Top Width	2.86	ft
Critical Depth	2.40	ft
Percent Full	64.9	%
Critical Slope	0.005970	ft/ft
Velocity	11.23	ft/s
Velocity Head	1.96	ft
Specific Energ	3.91	ft
Froude Numbe	1.52	
Maximum Disc	77.72	cfs
Discharge Full	72.25	cfs
Slope Full	0.005690	ft/ft
Flow Type	Supercritical	

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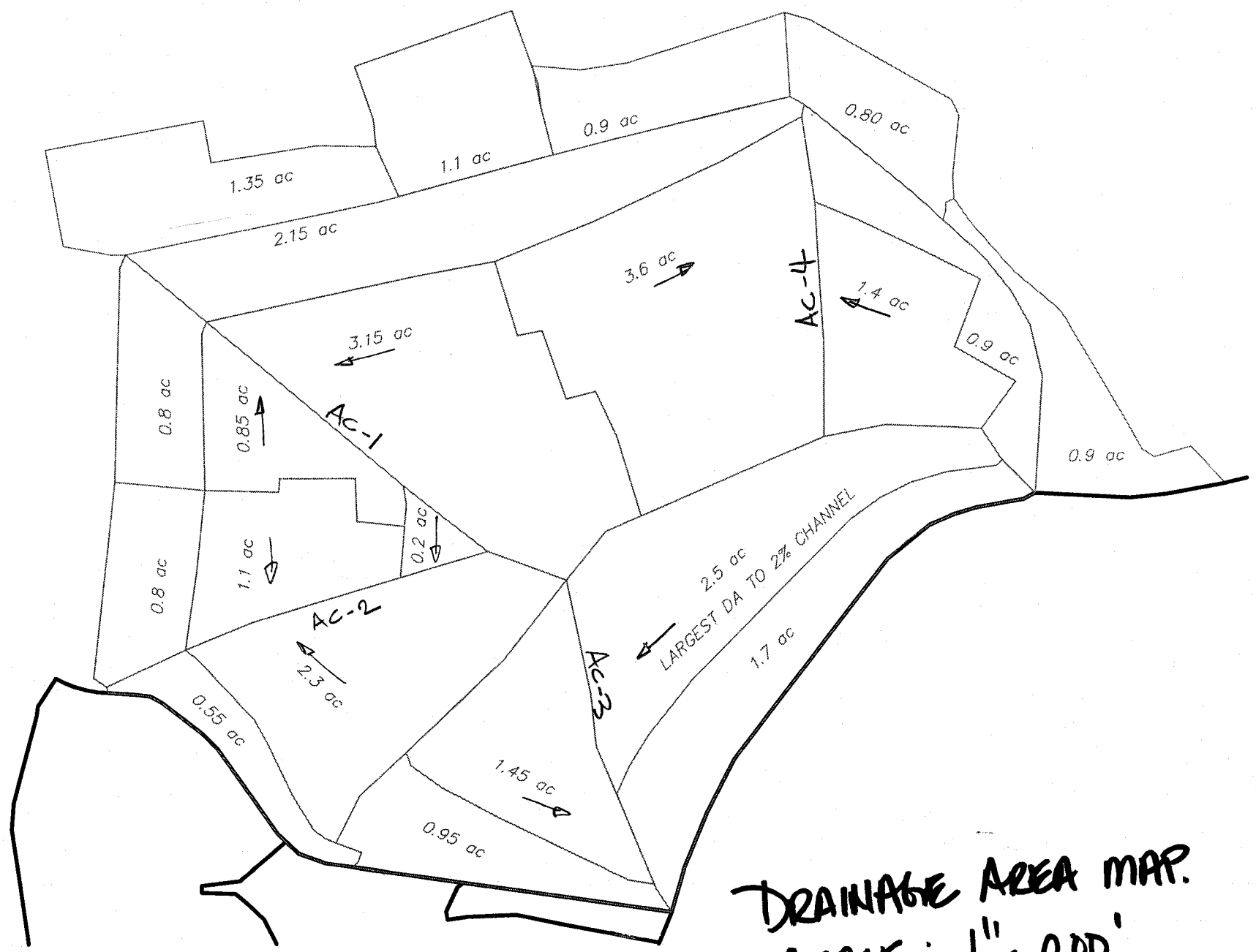

DRAINAGE AREA MAP.
SCALE: 1" = 200'

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Comp. By DHS Date 9-29-05 Project BELEWS CREEK LANDFILL Sheet No. ..... of .....
Chkd. By ..... Date ..... Subject ARMORED CHANNELS Job No. 046119

ARMORED CHANNEL AC-1
D.A. = 4.0 Ac.
C = .60
I = 6.8 IN/HR.
Q_{10} = .60 (6.8)(4.0) = 16.32 cfs
S = 16.4%.

ARMORED CHANNEL AC-2
D.A. = 3.6 Ac.
C = .60
I = 6.8 IN/HR
Q_{10} = .60 (6.8)(3.6) = 14.69 cfs.
S = 18.3%.

ARMORED CHANNEL AC-3
D.A. = 3.95 Ac.
C = .60
I = 6.8 IN/HR.
Q_{10} = .60 (6.8)(3.95) = 16.12 cfs
S = 13.9 %

ARMORED CHANNEL AC-4
D.A. = 5.0 Ac.
C = .60
I = 6.8 IN/HR.
Q_{10} = .60 (6.8)(5.0) = 20.4 cfs.

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Comp. By DHB Date 9-29-05 Project BELEWS CREEK LANDFILL Sheet No. ..... of .....
Chkd. By ..... Date ..... Subject ..... Job No. 046119
STORM DRAINAGE FLOW CALCS.

CULVERT @ AC-1
D.A. = AC-1 + 2.95 = 6.95 Ac.
C = .60
I = 7.2 in/hr.
Q_{25} = .60 (7.2)(6.95) = 30.02 cfs.

CULVERT @ AC-2
D.A = AC-2 + 2.30 = 5.9 Ac.
C = .60
I = 7.2 in/hr.
Q_{25} = .60(7.2)(5.9) = 25.48 cfs.

CULVERT @ AC-3
D.A. = AC-3 + 2.65 = 6.60 Ac
C = .60
I = 7.2 in/hr.
Q_{25} = .60 (7.2)(6.60) = 28.51 cfs

CULVERT @ AC-4
D.A. = AC-4 + 0.9 = 5.9 Ac.
C = .60
I = 7.2 in/hr.
Q_{25} = .60(7.2)(5.9) = 25.48 cfs.

Worksheet
Worksheet for Circular Channel

Project Description	
Worksheet	CULVERT @ A AC-1
Flow Element	Circular Chann
Method	Manning's For
Solve For	Channel Depth

Input Data		
Mannings Coeffic	0.013	
Slope	035000	ft/ft
Diameter	24	in
Discharge	30.02	cfs

Results		
Depth	1.24	ft
Flow Area	2.1	ft²
Wetted Perime	3.63	ft
Top Width	1.94	ft
Critical Depth	1.86	ft
Percent Full	62.2	%
Critical Slope	0.015230	ft/ft
Velocity	14.62	ft/s
Velocity Head	3.32	ft
Specific Energy	4.56	ft
Froude Numbe	2.50	
Maximum Disc	45.52	cfs
Discharge Full	42.32	cfs
Slope Full	0.017611	ft/ft
Flow Type	Supercritical	

Worksheet
Worksheet for Circular Channel

Project Description	
Worksheet	CULVERT @ A Ac-2
Flow Element	Circular Chann
Method	Manning's For
Solve For	Channel Depth

Input Data		
Mannings Coeffic	0.013	
Slope	040000	ft/ft
Diameter	24	in
Discharge	25.49	cfs

Results		
Depth	1.07	ft
Flow Area	1.7	ft²
Wetted Perime	3.29	ft
Top Width	1.99	ft
Critical Depth	1.77	ft
Percent Full	53.7	%
Critical Slope	0.011328	ft/ft
Velocity	14.83	ft/s
Velocity Head	3.42	ft
Specific Energ	4.49	ft
Froude Numbe	2.82	
Maximum Disc	48.67	cfs
Discharge Full	45.24	cfs
Slope Full	0.012697	ft/ft
Flow Type	Supercritical	

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Worksheet
Worksheet for Circular Channel

Project Description	
Worksheet	CULVERT @ AC-3
Flow Element	Circular Chann
Method	Manning's For
Solve For	Channel Depth

Input Data		
Mannings Coeffic	0.013	
Slope	020000	ft/ft
Diameter	24	in
Discharge	28.51	cfs

Results		
Depth	1.47	ft
Flow Area	2.5	ft²
Wetted Perime	4.12	ft
Top Width	1.76	ft
Critical Depth	1.84	ft
Percent Full	73.6	%
Critical Slope	0.013796	ft/ft
Velocity	11.51	ft/s
Velocity Head	2.06	ft
Specific Energ	3.53	ft
Froude Numbe	1.71	
Maximum Disc	34.41	cfs
Discharge Full	31.99	cfs
Slope Full	0.015884	ft/ft
Flow Type	Supercritical	

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Worksheet
Worksheet for Circular Channel

Project Description	
Worksheet	CULVERT @ A AC-4
Flow Element	Circular Chann
Method	Manning's For
Solve For	Channel Depth

Input Data		
Mannings Coeffic	0.013	
Slope	040000	ft/ft
Diameter	24	in
Discharge	25.48	cfs

Results		
Depth	1.07	ft
Flow Area	1.7	ft²
Wetted Perime	3.29	ft
Top Width	1.99	ft
Critical Depth	1.77	ft
Percent Full	53.7	%
Critical Slope	0.011321	ft/ft
Velocity	14.83	ft/s
Velocity Head	3.42	ft
Specific Energ	4.49	ft
Froude Numbe	2.82	
Maximum Disc	48.67	cfs
Discharge Full	45.24	cfs
Slope Full	0.012687	ft/ft
Flow Type	Supercritical	

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Comp. By DHB Date 9-29-05 Project BELEWS CREEK LANDFILL Sheet No. ____ of ____
Chkd. By ____ Date ____ Subject OUTLET PROT. APRONS. Job No. 046119

31
A-~~30~~ @ AC-1
DOWNSTREAM CULVERT DIA. = 24"
Q_{10} = 16.32 cfs.
L_A = 12 ft.
W = 14 ft.
d_{50} RIPRAP = 6"
d_{max} = 9"
t = 13.5"
H = 16"

32
A-~~31~~ @ 24" ϕ CULVERT OUTLET. D.S. OF DI-1
Q_{10} = .60 (6.8)(6.95) = 28.35 cfs.
L_A = 18 ft
W = 20 ft.
d_{50} RIPRAP = 9"
d_{max} = 13.5"
t = 20.25"
H = 16"

33
A-~~32~~ @ AC-2
DOWNSTREAM PIPE ϕ = 24"
Q_{10} = 14.69 cfs
L_A = 12 ft
W = 14 ft.
d_{50} RIPRAP = 6"
d_{max} = 9"
t = 13.5"
H = 16"

A-~~33~~ 34 @ 24" ∅ CULVERT OUTLET D.S. OF DI-2

$Q_{10} = 0.6(6.8)(5.9) = 24.07$ cfs.

$L_A = 15$ ft.

$W = 17$ ft.

d_{50} RIPRAP = 6"

$d_{max} = 9"$

$t = 13.5"$

$H = 16"$

A-~~34~~ 35 @ Ac-3

DOWNSTREAM PIPE ∅ = 24"

$Q_{10} = 16.12$ cfs.

$L_A = 12$ ft.

$W = 14$ ft.

d_{50} RIPRAP = 6"

$d_{max} = 9"$

$t = 13.5"$

$H = 24"$

A-~~35~~ 36 @ 24" ∅ CULVERT D.S. OF DI-3

$Q_{10} = 0.6(6.8)(6.60) = 26.93$ cfs

$L_A = 16$ ft.

$W = 18$ ft.

d_{50} RIPRAP = 9"

$d_{max} = 13.5"$

$t = 20.25"$

$H = 16"$

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Comp. By DHB Date 9-29-05 Project BELEWS CREEK LANDFILL Sheet No. ____ of ____
Chkd. By ____ Date ____ Subject OUTLET PROT. APRONS Job No. 046119

A-~~36~~ 37 @ AC-4
DOWNSTREAM PIPE ϕ = 24"
Q_{10} = 20.4 cfs.
L_A = 12 ft
W = 14 ft.
d_{50} RIPRAP = 6"
d_{max} = 9"
t = 13.5"
H = 16"

A-~~37~~ 38 @ 24" ϕ CULVERT OUTLET D.S. OF DI-4
Q_{10} = 0.6(6.8)(5.9) = 24.07 cfs
L_A = 15 ft
W = 17 ft.
d_{50} RIPRAP = 6"
d_{max} = 9"
t = 13.5"
H = 16"

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Comp. By DHB. Date 9-29-0 | Project BELEWS CREEK LANDFILL | Sheet No. ..... of .....
Chkd. By ..... Date ..... | Subject BENCH CHANNELS. | Job No. 046119

BENCH CHANNELS

$S = 2.0\%$ TYP.

LARGEST D.A. = 2.5 Ac.
$C = .60$
$I_{10} = 6.8$ in/hr.
$Q_{10} = 0.6(6.8)(2.5) = 10.2$ cfs.

SYNTHETIC MAT LINER $n = .025$, $D = 0.62$ ft.

$T_d = \gamma ds = 62.4(.62)(.02) = 0.77$ lb/ft
< 2.0 lb/ft OK.

GRASS LINER: $n = .062$ @ RET. CLASS D.

$R_h = \frac{5.3}{12.32} = 0.43$

$V_A R_h = 1.93(.43) = 0.83$.

FIG. 8.05c $\therefore$ $n = .062$ VALID

GEOMETRY $B = 0$
$Z_L = 10$
$Z_R = 4$
$D = 1.0'$ MIN.
$T = 15$ ft.

Worksheet
Worksheet for Trapezoidal Channel

Project Description	
Worksheet	BENCH CHANN
Flow Element	Trapezoidal Cha
Method	Manning's Formı
Solve For	Channel Depth

Input Data		
Mannings Coeffic	0.025	
Slope	020000	ft/ft
Left Side Slope	10.00	H : V
Right Side Slope	4.00	H : V
Bottom Width	0.00	ft
Discharge	10.20	cfs

Results		
Depth	0.62	ft
Flow Area	2.7	ft²
Wetted Perim	8.76	ft
Top Width	8.66	ft
Critical Depth	0.67	ft
Critical Slope	0.013350	ft/ft
Velocity	3.81	ft/s
Velocity Head	0.23	ft
Specific Energ	0.84	ft
Froude Numb	1.21	
Flow Type	Supercritical	

Worksheet
Worksheet for Trapezoidal Channel

Project Description	
Worksheet	BENCH CHANN
Flow Element	Trapezoidal Cha
Method	Manning's Formı
Solve For	Channel Depth

Input Data		
Mannings Coeffic	0.062	
Slope	020000	ft/ft
Left Side Slope	10.00	H : V
Right Side Slope	4.00	H : V
Bottom Width	0.00	ft
Discharge	10.20	cfs

Results		
Depth	0.87	ft
Flow Area	5.3	ft²
Wetted Perim	12.32	ft
Top Width	12.17	ft
Critical Depth	0.67	ft
Critical Slope	0.082108	ft/ft
Velocity	1.93	ft/s
Velocity Head	0.06	ft
Specific Energ	0.93	ft
Froude Numb	0.52	
Flow Type	Subcritical	

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Comp. By DHB Date 9-29-05 Project BELEWS CREEK LANDFILL Sheet No. ____ of ____
Chkd. By ____ Date ____ Subject PERM. DIVERSION CHANNELS. Job No. 04611.9

PD-51
D.A. = 2.15 Ac.
C = .60
I_2 = 4.8 in/hr
I_{10} = 6.8 in/hr.
Q_2 = 0.6 (4.8)(2.15) = 6.19 cfs.
Q_{10} = 0.6 (6.8)(2.15) = 8.77 cfs
S = 0.67 %

B=1
Z=3
D=1.5
T=10
STRAW W/ NET

PD-52
D.A. = 0.9 Ac.
C = .60
Q_2 = 0.6(4.8)(0.9) = 2.59 cfs.
Q_{10} = 0.6 (6.8)(0.9) = 3.67 cfs.
S = 1/12 = 8.33 %.

B=1
Z=3
D=1.0
T=7
SYNTH. MAT.

PD-53
D.A. = 1.5 Ac.
C = .60
Q_2 = 0.6(4.8)(1.5) = 4.32 cfs
Q_{10} = 0.6 (6.8)(1.5) = 6.12 cfs.
S = 5.5% 2/36

B=1
Z=3
D=1.5
T=10
SYNTH MAT.

PD-54.
D.A. = 1.7 Ac.
C = .60
Q_2 = 0.6(4.8)(1.7) = 4.90 cfs.
Q_{10} = 0.6 (6.8) (1.7) = 6.94 cfs.
S = 5.0 % 2/40

B=0
Z=3
D=1.5
T=9
STRAW W/ NET

Worksheet
Worksheet for Trapezoidal Channel

Project Description	
Worksheet	PD-51
Flow Element	Trapezoidal Cha
Method	Manning's Formı
Solve For	Channel Depth

Input Data		
Mannings Coeffic	0.033	
Slope	006700	ft/ft
Left Side Slope	3.00	H : V
Right Side Slope	3.00	H : V
Bottom Width	1.00	ft
Discharge	6.19	cfs

Results		
Depth	0.82	ft
Flow Area	2.8	ft²
Wetted Perim	6.18	ft
Top Width	5.91	ft
Critical Depth	0.62	ft
Critical Slope	0.023239	ft/ft
Velocity	2.19	ft/s
Velocity Head	0.07	ft
Specific Energ	0.89	ft
Froude Numb	0.56	
Flow Type	Subcritical	

STRAW W/ NET.

Worksheet
Worksheet for Trapezoidal Channel

Project Description	
Worksheet	PD-51
Flow Element	Trapezoidal Cha
Method	Manning's Formı
Solve For	Channel Depth

Input Data		
Mannings Coeffic	0.059	
Slope	006700	ft/ft
Left Side Slope	3.00	H : V
Right Side Slope	3.00	H : V
Bottom Width	1.00	ft
Discharge	8.77	cfs

Results		
Depth	1.22	ft
Flow Area	5.7	ft²
Wetted Perim	8.70	ft
Top Width	8.30	ft
Critical Depth	0.73	ft
Critical Slope	0.070937	ft/ft
Velocity	1.55	ft/s
Velocity Head	0.04	ft
Specific Energ	1.25	ft
Froude Numb	0.33	
Flow Type	Subcritical	

GRASS
RET. CLASS D.

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Worksheet
Worksheet for Trapezoidal Channel

Project Description	
Worksheet	PD-~~51~~ 52
Flow Element	Trapezoidal Cha
Method	Manning's Formı
Solve For	Channel Depth

Input Data		
Mannings Coeffic	0.036	
Slope	083000	ft/ft
Left Side Slope	3.00	H : V
Right Side Slope	3.00	H : V
Bottom Width	1.00	ft
Discharge	2.59	cfs

SYNTHETIC MAT.

Results		
Depth	0.32	ft
Flow Area	0.6	ft²
Wetted Perim	3.01	ft
Top Width	2.91	ft
Critical Depth	0.40	ft
Critical Slope	0.031069	ft/ft
Velocity	4.16	ft/s
Velocity Head	0.27	ft
Specific Energ	0.59	ft
Froude Numb	1.58	
Flow Type	Supercritical	

Worksheet
Worksheet for Trapezoidal Channel

Project Description	
Worksheet	PD-ST52
Flow Element	Trapezoidal Cha
Method	Manning's Formı
Solve For	Channel Depth

Input Data		
Mannings Coeffic	0.060	
Slope	083000	ft/ft
Left Side Slope	3.00	H : V
Right Side Slope	3.00	H : V
Bottom Width	1.00	ft
Discharge	3.67	cfs

Results		
Depth	0.48	ft
Flow Area	1.2	ft²
Wetted Perim	4.04	ft
Top Width	3.88	ft
Critical Depth	0.48	ft
Critical Slope	0.082355	ft/ft
Velocity	3.13	ft/s
Velocity Head	0.15	ft
Specific Energ	0.63	ft
Froude Numb	1.00	
Flow Type	Supercritical	

GRASS

RET. CLASS D.

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Worksheet
Worksheet for Trapezoidal Channel

Project Description	
Worksheet	PD-53
Flow Element	Trapezoidal Cha
Method	Manning's Formı
Solve For	Channel Depth

Input Data		
Mannings Coeffic	0.036	
Slope	055000	ft/ft
Left Side Slope	3.00	H : V
Right Side Slope	3.00	H : V
Bottom Width	1.00	ft
Discharge	4.32	cfs

Results		
Depth	0.45	ft
Flow Area	1.1	ft²
Wetted Perim	3.84	ft
Top Width	3.70	ft
Critical Depth	0.52	ft
Critical Slope	0.029009	ft/ft
Velocity	4.09	ft/s
Velocity Head	0.26	ft
Specific Energ	0.71	ft
Froude Numb	1.35	
Flow Type	Supercritical	

SYNTH. MAT.

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Worksheet
Worksheet for Trapezoidal Channel

Project Description	
Worksheet	PD-53
Flow Element	Trapezoidal Cha
Method	Manning's Formı
Solve For	Channel Depth

Input Data		
Mannings Coeffic	0.057	
Slope	055000	ft/ft
Left Side Slope	3.00	H : V
Right Side Slope	3.00	H : V
Bottom Width	1.00	ft
Discharge	6.12	cfs

Results		
Depth	0.65	ft
Flow Area	1.9	ft²
Wetted Perim	5.12	ft
Top Width	4.91	ft
Critical Depth	0.62	ft
Critical Slope	0.069436	ft/ft
Velocity	3.18	ft/s
Velocity Head	0.16	ft
Specific Ener	0.81	ft
Froude Numb	0.90	
Flow Type	Subcritical	

GRASS
RET. CLASS D.

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Worksheet
Worksheet for Trapezoidal Channel

Project Description	
Worksheet	PD-54
Flow Element	Trapezoidal Cha
Method	Manning's Formi
Solve For	Channel Depth

Input Data		
Mannings Coeffic	0.033	
Slope	050000	ft/ft
Left Side Slope	3.00	H : V
Right Side Slope	3.00	H : V
Bottom Width	0.00	ft
Discharge	4.90	cfs

STRAW W/ NET

Results		
Depth	0.61	ft
Flow Area	1.1	ft²
Wetted Perim	3.85	ft
Top Width	3.66	ft
Critical Depth	0.70	ft
Critical Slope	0.024178	ft/ft
Velocity	4.40	ft/s
Velocity Head	0.30	ft
Specific Energ	0.91	ft
Froude Numb	1.41	
Flow Type	Supercritical	

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Worksheet
Worksheet for Trapezoidal Channel

Project Description	
Worksheet	PD-54
Flow Element	Trapezoidal Cha
Method	Manning's Formı
Solve For	Channel Depth

Input Data		
Mannings Coeffic	0.052	
Slope	050000	ft/ft
Left Side Slope	3.00	H : V
Right Side Slope	3.00	H : V
Bottom Width	0.00	ft
Discharge	6.94	cfs

Results		
Depth	0.82	ft
Flow Area	2.0	ft²
Wetted Perim	5.21	ft
Top Width	4.94	ft
Critical Depth	0.80	ft
Critical Slope	0.057311	ft/ft
Velocity	3.41	ft/s
Velocity Head	0.18	ft
Specific Ener	1.00	ft
Froude Numb	0.94	
Flow Type	Subcritical	

GRASS
RET. CLASS D.

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B

List of Drawings

Description of Drawing	Drawing Number	Duke Document Number
FGD Residue Landfill Existing Conditions Plan	CP-1	BCM 6451.00-0007.001
FGD Residue Landfill Top of Liner Grading Plan	CP-2	BCM 6451.00-0008.001
FGD Residue Landfill Leachate Collection System	CP-3	BCM 6451.00-0009.001
FGD Residue Landfill Operational Cover Grading Plan	CP-4	BCM 6451.00-0010.001
FGD Residue Landfill Cell Details	CP-5	BCM 6451.00-0011.001
FGD Residue Landfill Stormwater Basin Plan and Details	CP-6	BCM 6451.00-0012.001
FGD Residue Landfill Pump Station Details	CP-7	BCM 6451.00-0013.001
FGD Residue Landfill Final Closure Plan and Details	CP-8	BCM 6451.00-0014.001
FGD Residue Landfill Final Closure Gas Venting System Plan	CP-9	BCM 6451.00-0015.001
FGD Erosion and Sediment Control Plan FGD Residue Landfill	CP-10	BCM 6451.00-0016.001
FGD Project Erosion and Sediment Control Plan Stockpile Areas	CP-11	BCM 6451.00-0017.001
FGD Project Erosion and Sediment Control Plan Details	CP-12	BCM 6451.00-0018.001
FGD Residue Landfill Profiles 1 of 2	CP-13	BCM 6451.00-0019.001
FGD Residue Landfill Profiles 2 of 2	CP-14	BCM 6451.00-0020.001

Drawing Under Seperate Cover