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January 25, 2006

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NC DEPT. OF ENVIRONMENT
AND NATURAL RESOURCES
RECEIVED
JAN 25 2006
MOORESVILLE REGIONAL OFFICE
DIVISION OF WASTE MANAGEMENT, SWS

Subject: Duke Power Marshall Steam Station, Catawba County
Construction Quality Assurance Program
Revision to Landfill Detail Drawing

Mr. Murray,
As we discussed at our meeting on January 24, 2006, on behalf of Duke Power, Sells is providing three copies of the Construction Quality Assurance Program for the Duke Power Marshall FGD Residue Landfill, dated January 16, 2006.

We are also providing you three copies of the revised drawing, MM6451.00-0007.001, showing the Cell 1 and Cell 2 Details. As you requested, we have revised Detail 1/7 for your comments.

If you have questions on this, please contact me at 704-662-0100.

Sincerely,

William M. Miller

William M. Miller, P.E.

cc Richard Baker, Duke Power
Pat McCabe, Duke Power

# CONSTRUCTION QUALITY ASSURANCE PROGRAM

**DUKE ENERGY CORP.**
**MARSHALL STEAM STATION**
**FGD RESIDUE LANDFILL**
**CATAWBA COUNTY, NC**

Prepared By:


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January 16, 2006

NORTH CAROLINA PROFESSIONAL SEAL 17582 ENGINEER DAN E. BREWER
1/16/06

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## 1.0 PROJECT TEAM AND RESPONSIBILITIES

Construction Quality Assurance (CQA) for the construction of the Marshall FGD Gypsum Landfill 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.

2.1.2 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 COMPACTED SOIL LAYER

## 3.1 Material

3.1.1 The Compacted Soil Layer is defined as a smooth soil layer for placement of the geomembrane.

3.1.2 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.

3.1.3 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.

3.1.4 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.

## 3.2 Construction

3.2.1 The Compacted Soil Layer 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.

3.2.2 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 (ASTM D-698) at moisture contents within 2 percent dry of optimum and 5 percent wet of optimum.

3.2.3 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. |

3.2.4 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.

Moisture content shall be wet of optimum moisture content at the density specified.

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.

3.2.5 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.

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

# 4.0 GEOSYNTHETIC CLAY LINER

## 4.1 Material

4.1.1 The geosynthetic clay liner (GCL) used on the landfill floor shall be equivalent to Claymax 600 CL. The GCL for the side slopes shall be stitched reinforced GCL equivalent to Bentomat DN.

4.1.2 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.

4.1.3 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.

4.1.4 The GCL shall be stored in a dry, well drained storage area. The GCL shall be stored under plastic sheets or tarpaulins to prevent hydration.

## 4.2 Geosynthetic Clay Liner Manufacturer and Contractor

4.2.1 The geomembrane installer is the party responsible for installation of the GCL. "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 Clay Liner Manufacturer to the ENGINEER:

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

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

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

- Qualifications and resumes of the GEOSYNTHETIC CONTRACTOR Superintendent and Foreman

- Two sets of proposed GCL field panel layout drawings

## 4.3 GCL Installation

4.3.1 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.

4.3.2 The geomembrane liner shall be constructed as soon as practical after completion and approval of the Compacted Soil Layer.

4.3.3 The top of the Compacted Soil Layer will be surveyed to ensure specified grades toward the collection sump area have been achieved. The GCL is to cover the bottom of the cell and the side slopes in accordance with the Contract Drawings.

4.3.4 Areas to receive GCL 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.

4.3.5 An anchor trench (as illustrated on the Contract Drawings) will be required to secure the GCL. No loose soil will be allowed to underlie the GCL 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 Technical Specifications, Section 3.3.10).

4.3.6 The GCL installed into 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.

4.3.7 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.

4.3.8 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.

4.3.9 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).

4.3.10 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.

4.3.11 Once the daily panels are placed and smooth, commence placement of the geomembrane in accordance with Section 5.0 GEOMEMBRANE. 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.

4.3.12 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.

4.3.13 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:

- Bentonite Mass/Area ASTM D 5993
- GCL Grab Strength ASTM D 4632, ASTM D 6768
- GCL Peel Strength ASTM D 4632, ASTM D 6496
- GCL Hydraulic Conductivity ASTM D 5887
- GCL Hydrated Internal Shear Strength, ASTM D 5321, ASTM D 6243

4.3.14 Placement of the GCL will be observed and recorded to assure conformation with the Technical Specifications and GEOSYNTHETIC MANUFACTURER'S recommendations. Any damaged material will be marked for repair and/or replacement.

4.3.15 During seaming of the GCL, the ENGINEER or his representative will observe the seams for the proper overlap and seaming techniques are employed.

# 5.0 GEOMEMBRANE

## 5.1 Meetings

5.1.1 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.

5.1.2 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.

## 5.2 Material

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

Smooth geomembrane shall be used on the landfill floor.

Textured geomembrane shall be used for the landfill side slopes. This textured geomembrane shall extend 10 feet past the toe of the slope, onto the landfill floor.

## 5.3 Geosynthetic Manufacturer and Contractor

5.3.1 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.

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

- Qualifications and resumes of the GEOSYNTHETIC CONTRACTOR Superintendent and Foreman

- Two sets of proposed geosynthetic field panel layout drawings.

5.3.3 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.

5.3.4 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.

## 5.4 Geomembrane Installation

5.4.1 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.

5.4.2 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.

5.4.3 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.

5.4.4 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)

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

5.4.6 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.

5.4.7 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.

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

5.4.9 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.

5.4.10 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.

## 5.5 Test Seams

5.5.1 The GEOSYNTHETIC CONTRACTOR will perform a test seam at least twice daily. 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.

5.5.2 The ENGINEER shall complete the Test Seam Form immediately after each test.

5.5.3 Requirements for test seams are as found in the Technical Specifications, Section 4.3.5.2.

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

5.5.5 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.

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

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

## 5.6 Non-Destructive Testing

5.6.1 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.

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

## 5.7 Field Destructive Samples

5.7.1 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, as specified in the Technical Specifications, Section 4.3.7. 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.

5.7.2 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.

5.7.3 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.

## 5.8 Laboratory Destructive Testing

5.8.1 Destructive seam samples will be laboratory tested by the ENGINEER as described in the Technical Specifications. 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.

5.8.2 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.

## 5.9 Documentation

Forms to be completed 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 to be completed by the CQAO:

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

**Table 5-1**

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

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

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

# 6.0 LEACHATE COLLECTION AND REMOVAL SYSTEM (LCRS)

## 6.1 Geotextile, Geonet, and Geocomposite

6.1.1 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. The cover soil thickness varies from one foot to greater than one foot on the side slopes. Within the operational cover on the landfill floor will be a series of leachate collection piping wrapped with gravel and geotextile.

6.1.2 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)

6.1.3 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)

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

6.1.5 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.

6.1.6 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.

6.1.7 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:

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

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

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

6.1.10 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.

## 6.2 Leachate Collection Pipe and Sump

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

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

6.2.3 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.

6.2.4 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.

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

6.2.6 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.

6.2.7 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 for field density testing.

## 6.3 Operational Cover

6.3.1 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 and gradation testing. Material must meet the gradation requirements described in the Technical Specifications, Section 6.2.1. Placement of the operational cover will be placed in minimum 12 inch thick lifts in a method to be approved by the ENGINEER.

6.3.2 The ENGINEER will monitor operational cover placement on a full-time basis.

6.3.3 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.

6.3.4 The final surface of the operational cover will be rolled with a non-vibratory smooth drum roller.

# 7.0 FINAL COVER SYSTEM

## 7.1 Compacted Soil Layer

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

7.1.2 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.

7.1.3 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.

7.1.4 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.

## 7.2 Geomembrane Cap

The geomembrane cap shall consist of a 40 mil textured LLDPE. The CQA procedures established for the landfill bottom geomembrane (Section 5.0 GEOMEMBRANE) will be modified and submitted to the Division of Waste Management for approval prior to closure.

## 7.3 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.

## 7.4 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.

## 7.5 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.

## 8.0 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.

## 9.0 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. Geosynthetic Clay Liner
3. Geomembrane Liner
4. Leachate Collection and Removal System
5. Operational Cover
6. 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 to NCDENR, Solid Waste Section, 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.

## 10.0 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 | Index 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 |

Carmen Johnson
Fac/Permit/Doc ID # 18-09 Date 1/25/12 Doc ID# DIN

# CONSTRUCTION PLAN APPLICATION

**DUKE POWER**
**MARSHALL STEAM STATION**
**CATAWBA COUNTY, NC**

**FLUE GAS DESULFURIZATION (FGD) GYPSUM LANDFILL**

RECEIVED
DEC 2 9 2005
NC DEPT OF ENVIRONMENT AND NATURAL RESOURCES
MOORESVILLE REGIONAL OFFICE

APPROVED
DIVISION OF WASTE MANAGEMENT
SOLID WASTE SECTION
DATE 6-5-06 BY

Prepared for:
NCDENR
Solid Waste Section
Raleigh, North Carolina

Prepared by:

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

128 Overhill Drive, Suite 105
Mooresville, NC 28117

Revised December 15, 2005

Document MM6451.00-0000.001, Revision 1
ERN No. FHS0004Z

**Volume 1**

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# CONSTRUCTION PLAN APPLICATION

**DUKE POWER**
**MARSHALL STEAM STATION**
**FLUE GAS DESULPHURIZATION (FGD) GYPSUM LANDFILL**
**CATAWBA COUNTY, NC**

Prepared for:
NCDENR
Solid Waste Section
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Revised December 15, 2005


NORTH CAROLINA PROFESSIONAL SEAL 17582 ENGINEER DAN E. BREWER
12/15/05

**Description of Revision 1 to Document MM6451.00-0000.001**

In March 2004, Duke Energy submitted Construction Plan Application - Document MM6451.00-0000.001 to NCDENR. That document presented information on a landfill to receive FGD residue (gypsum) generated at Marshall and other Duke plants located in NC. This landfill was to be constructed as an unlined landfill. At the end of 5 years, the landfill would be covered with an engineered cover system. NCDENR issued a Permit to Construct for Cell 1 of this landfill on January 14, 2005.

Subsequent to the issuance of this Permit to Construct, Duke performed additional leaching tests on the clarifier sludge filter cake, an additional waste stream to be placed in this landfill. This material is a filter cake generated by sludge from the FGD wastewater treatment system. Testing of this material indicated placement of this material in an unlined landfill was not acceptable and that a synthetic liner would be required for disposal of this material.

This CPA presents information pertaining to the revised design for this landfill. This design calls for an engineered synthetic liner system, a leachate collection and removal system, and an engineered cover system. Certain information and calculations contained in the previous CPA (Document MM6451.00-0000.001) remain valid and will be referenced in this CPA where appropriate.

In Document MM6451.00-0000.001, submitted March 2004, contained the following information:

| | |
|---|---|
| Section 1 | Construction Plan Application Report |
| Section 2 | Operations and Maintenance Plan |
| Section 3 | Closure/Post-Closure Plan |
| Section 4 | Technical Specifications For Closure<br>Site Preparation for Cell 1<br>Spec. MS-6451.00-00-0001<br><br>Site Preparation for Cell 2<br>Spec. MS-6451.00-00-0002 |
| Appendix | Calculations |

There are no changes to Section 2, Section 3, and Section 4 from Revision 1. Therefore, these sections are not included in this submittal.

**Drawings**

Certain drawings included in the March 2004 submittal also remain valid and are not revised for Revision 1 of the CPA. The following page presents a table showing which drawings are not revised for Revision 1 and listing drawings issued for this revision. Drawings that are not revised are not included in this submittal.

# LIST OF DRAWINGS

| | Duke Drawing # | Title |
|---|---|---|
| * | M-6024-02.00 | Cell 1 Excavation |
| * | M-6024-03.00 | Cell 1 Profile and Sections |
| * | M-6024-04.00 | Cell 1 Perimeter Road Realignment |
| | | |
| * | M-6024-08.00 | Cell 1 and Cell 2 Perimeter Road Sections |
| * | M-6024-09.00 | Cell 1 and Cell 2 Perimeter Road Profile |
| | | |
| * | MM6451.00-0002.001 | Gas Venting System Plan |
| * | MM6451.01-0001.001 | Cell 1 Erosion and Sediment Control Plan |
| * | MM6451.01-0002.001 | Cell 2 Erosion and Sediment Control Plan |
| * | MM6451.01-0003.001 | Final Closure Erosion and Sediment Control Plan |
| * | MM6451.01-0004.001 | Erosion Control Details |
| | | |
| | MM6451.00-0001.001 | Final Closure Details |
| | MM6451.00-0003.001 | Cell 1 Leachate Collection and Removal System |
| | MM6451.00-0004.001 | Cell 1 Operational Cover Grading Plan |
| | MM6451.00-0005.001 | Cell 1 Final Configuration |
| | MM6451.00-0006.001 | Cell 1 Stormwater Basin |
| | MM6451.00-0007.001 | Cell 1 and Cell 2 – Details |
| | MM6451.00-0008.001 | Cell 1 and Cell 2 Leachate Collection and Removal System |
| | | |
| | MM6451.00-0009.001 | Cell 2 Landfill Excavation |
| | MM6451.00-0010.001 | Cell 2 Leachate Collection and Removal System |
| | MM6451.00-0011.001 | Cell 2 Operational Cover Grading Plan |
| | MM6451.00-0012.001 | Cell 2 Stormwater Basin |
| | | |
| | MM6451.00-0013.001 | Cell 1 and Cell 2 Final Configuration |
| | MM6451.00-0014.001 | Cell 1 and Cell 2 Sections |
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* Drawing is not revised.

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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 (NOx) 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 Marshall Steam Station in Catawba 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 project, the Marshall FGD Residue landfill. Marshall Steam Station is located in Catawba County, on Highway NC 150, just west of Lake Norman. The FGD landfill will be located on Duke Power property, northwest of the Marshall plant and to the west of the Marshall Ash Basin.

The landfill footprint contains approximately 31.9 acres and will consist of two cells. Cell 1 which will be constructed and filled first, has a footprint of 14.8 acres. Cell 2 has a footprint of 17.1 acres. The landfill will begin receiving FGD residue in the fall of 2006. The net volumetric capacity of the landfill is 1.82 million cubic yards. The landfill will be constructed with an engineered cover system to minimize infiltration, a leachate collection and removal system, and an engineered liner system to prevent impacts to groundwater.

## 1.2 Site Conditions

The FGD landfill will be located on Duke Power property, northwest of the Marshall plant and to the west of the Marshall Ash Basin. The landfill is located to the east of a railroad line, running north-to-south. Located to the west of this railroad line is Sherrill's Ford Road, which runs north-south along a surface water divide. Located within landfill footprint and east of the railroad line is a surface water drainage feature. This feature drains to the south, to an intermittent stream that drains to Beaverdam Creek, which drains to Lake Norman. There is a topographic divide running NNW through the landfill footprint, along Steam Plant Road. Surface drainage to the west of Steam Plant Road drains to the surface drainage feature and surface drainage to the east of Steam Plant Road drains to the Marshall Ash Basin.

## 1.3 Description of Material Disposed in Landfill

The FGD gypsum landfill at Marshall will receive residue solely generated from the FGD processes at Marshall and from other Duke plants located in North Carolina. The material placed in the landfill will be:

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

beneficial use options. (i.e. gypsum produced from the FGD process).

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. This filter cake material will be dewatered.

The FGD residue and clarifier sludge filter cake will be conveyed to the landfill site by conveyor and truck, where the material will be spread and compacted.

## 2.0 WASTE STREAM

### 2.1 Types of Waste

As described in Section 1.3 Description of Material Disposed in Landfill, the waste disposed of in the Marshal FGD Residue 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. This filter cake material will be dewatered.

The FGD residue and clarifier sludge filter cake will be conveyed to the landfill site by conveyor and truck, where the material will be spread and compacted.

The FGD residue material placed in the landfill (1 and 2 from above) is primarily gypsum (93% to 95%). Typical parameters for the residue materials to be produced at Marshall 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% |

The clarifier sludge filter cake will be composed of fractions of the above constituents.

The clarifier filter cake material will be mixed with gypsum on a 1:1 ratio (by volume) to provide the strength required for stable slopes.

The following table presents the unit weights and other relevant information on these materials.

| | Gypsum | Clarifier Filter Cake | Clarifier (Proctor - Maximum Dry Density) | Clarifier:Gypsum Mixture (1:1) (Proctor - Maximum Dry Density) |
|---|---|---|---|---|
| pH (S.U.) | 6.0 to 8.3 | 9.89 | - | - |
| Specific Gravity | 2.35 | 2.0 to 2.3 | 2.32 | 2.33 |
| Unit Weigh (lb/ft$^3$) | 76 to 97 | 81 to 86 | 44.1 | 71.4 |
| % Moisture (wt. water/wt. solids) | 10% to 12% | 486 | 93.2 | 31.9 |

## 2.2 Disposal Rates

The FGD residue generation rate projected for this facility will be on the order of 350,000 dry tons (438,000 wet tons) of gypsum per year. The wastewater treatment system clarifier sludge filter cake that will be disposed in the landfill will have a disposal rate of 50,000 tons per year.

The material placed in the landfill will be gypsum, or a mixture of clarifier filter cake mixed with gypsum at a 1:1 ratio (by volume). The in-place density for the gypsum material is estimated to be 89 lbs/ft$^3$ (1.2 tons/yd$^3$). Although the filter cake material has a density of 81 to 86 lb/ft$^3$ as filter press material, compaction testing (proctor tests) on the material indicate that the in-place density of the filter cake material alone will be 44 lb/ft$^3$. Proctor tests on the mix of clarifier filter cake and gypsum (1:1) indicate that the density will be 71 lb/ft$^3$. The in-place unit weight of the clarifier:gypsum mix is estimated to be 60 lb/ft$^3$.

Future disposal rates may vary due to beneficial reuse or recycling efforts. The combined annual disposal of FGD residue and clarifier sludge is approximately 427,000 yd$^3$. The net volumetric capacity of the landfill is 1,824,160 yd$^3$. At this disposal rate, this landfill volume will provide approximately 4.3 years of disposal.

## 2.3 Service Area

The FGD gypsum landfill at Marshall Steam Station will receive FGD residue generated at the Marshall facility and from other Duke Power plants located in North Carolina.

## 2.4 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.

# 3.0 LANDFILL DESIGN

## 3.1 Site Development

The FGD gypsum landfill consists of two cells containing approximately 31.9 acres. Cell 1 which has a footprint of 14.8 acres. Cell 2 has a 17.1 acres footprint.

The landfill footprint is contained by excavation into existing ground on the north-east, east, and southern sides. An exterior perimeter berm will be constructed along the north-west and western sides. The perimeter ditch 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 250-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 1.5 percent base grade. The subgrade will be a minimum of four feet above the estimated seasonal high groundwater level and a minimum of four feet above bedrock. 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 will be lined with a 60 mil HDPE geomembrane. A geosynthetic clay liner (GCL) will be placed under the HDPE geomembrane. A leachate collection system is designed with a geocomposite and system of perforated HDPE piping to drain the leachate within each active cell to the downgradient stormwater storage basin. Stormwater and leachate will be collected and will be routed to the Marshall Ash Basin. A 12-inch thick layer of operational cover is designed over the geomembrane liner and leachate collection and removal system.

An existing road, Steam Plant Road, provides access along the eastern side of the landfill. A road will be constructed along the western and southern side of Cell 2 to provide maintenance access. The roads will be approximately 30 feet wide with side ditches to prevent erosion of the roadway surface. These roads will be surfaced with ABC stone to provide all weather access and limit dust. Temporary access roads will be constructed throughout the operation of the landfill.

Final cover elevations, as shown on the drawing, Cell 1 and Cell 2 Final Configuration, will be sloped no steeper than 6H:1V (horizontal to vertical) with erosion control benches. The final engineered cover system (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.

## 3.2 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 and Maintenance Plan, Section 2.

Closure of cells will commence 10 years after placement of waste. Partial closure of cells may be implemented during landfill operations. Site phasing diagrams are presented in the FGD Gypsum Landfill Operations Plan, Section 2.

## 3.3 Closure Design

The primary purpose of an engineered cover system is to minimize infiltration of water into the waste. The typical engineered cover cross section is presented on the drawing, Final Closure Details. 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. A gas collection and venting system is included in the design of the cap system.

## 3.4 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 drawings, Cell 1 Profile and Sections and Cell 1 and Cell 2 Sections, 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 bottom of the landfill excavation).

## 3.5 Engineering Considerations of Landfill Components

### 3.5.1 Landfill Subgrade

The excavated subgrade will be compacted and maintained to provide support for the overlying geomembrane. 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 boring OW-2.

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.

### 3.5.2 Geosynthetic Clay Liner

A geosynthetic clay liner (GCL) will be placed on top of the excavated surface and underneath the HDPE geomembrane. The geosynthetic clay liner (GCL) placed on the bottom grade will be equivalent to Claymax 600 CL. This will be a non-reinforced GCL consisting of two geotextiles encapsulating a layer of sodium bentonite with a composite laminate applied to one of the geotextiles.

The GCL for the side slopes shall be stitched reinforced GCL equivalent to Bentomat DN. The side slopes require a reinforced needlepunched GCL that not only has high internal shear strength, but also provides interface friction on both sides of the GCL.

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. Granular bentonite is used to ensure the seal at overlapped areas.

### 3.5.3 Geomembrane Liner

A 60-mil thick, high density polyethylene (HDPE), smooth geomembrane will be placed as the synthetic liner for the landfill 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 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. A textured geomembrane 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.

### 3.5.4 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 of the landfill will consist of a geonet/geotextile with perforated collector/header pipe that drains to the sump area. The slope on the geomembrane will be a minimum of 1.5 percent. The geonet is a polyethylene synthetic mesh with a transmissivity equal to or greater than $1 \times 10^{-4}$ $m^2$/sec. The geonet will transmit leachate to the leachate collection pipe.

A geocomposite will be used on the landfill sideslopes to collect and convey leachate downward to the LCRS located in the bottom of the landfill.

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 HELP model was used to verify that the pipe spacing and slope of the landfill lower grade is adequate to ensure that the maximum head on the liner system is less than 12 inches. The HELP model input includes the properties of the waste material and the leachate collection and removal system, as well as site specific meteorological information to calculate the quantities of leachate generated, as well as the head on the liner system.

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.

In Cell 1, the leachate collection header pipes drain directly to the Cell 1 Stormwater Basin. The perforations in the riser allow leachate to drain into the sump. The Cell 1 riser will be removed at completion of the cell. The leachate collection header piping will for Cell 1 will be continued into Cell 2.

The leachate collection header pipes are routed into the riser in Cell 2. After the surface water discharge function is no longer needed (at completion of the cell), the riser is filled with stone, and the riser is sealed. The leachate collection header pipes remain routed into the riser.

### 3.5.5 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 the landfill 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.

### 3.5.6 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.

### 3.5.7 Surface Water Collection and Removal (SWCR)

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.

Surface water run-on into the landfill will be prevented by installation of ditches around the landfill. Surface water falling inside the footprint will be collected in lined ditches constructed along the edges of the cells. In Cell 1, these ditches drain to a stormwater collection basin at the lower end of the operating cell. The discharge from this basin will be routed to the Ash Basin.

In Cell 2, the lined ditches drain to a sump area at the lower end of Cell 2. The sump then drains to a lined stormwater collection basin, located outside of the landfill footprint. The discharge from this basin is routed to the Marshall Ash Basin.

As Cell 2 construction begins, the Cell 1 stormwater basin will be abandoned. The HDPE riser will be remover, the 42"diameter pipe draining to the Ash Basin will be capped and the first 10 feet filled with grout. The Cell 1 stormwater basin will be filled with compacted soil to grade into the Cell 2 area excavation. The GCL, HDPE geomembrane, LCRS and operational cover will be then be placed over this area.

### 3.5.8 Geomembrane Cover- Geocomposite

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.

Placed over the LLDPE geomembrane will be a geocomposite to collect precipitation infiltrating through the vegetative soil cover. This geocomposite will have a minimum transmissivity equal to or greater than 1 x $10^{-4}$ $m^2$/sec. Drainage of infiltration collected in the geocomposite will enhanced by placement of edge drains. 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.

### 3.5.9 Gas Venting System

Based upon our past experience with gypsum landfills, hydrogen sulfide, $H_2S$, 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. The organic content of the clarifier sludge filter cake material will be minimal. 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 significant 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 drawing, Gas Venting System Plan.

The gas venting wells are to be constructed prior to final capping of the landfill. A total of 30 gas vents are distributed over the 31 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. These gas vents are connected by a system of gas venting trenches. These gas venting trenches are constructed of #57 stone and are used to collect gas and provide a pathway for transmission of the collected gas to the gas vent.

### 3.5.10 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.

### 3.5.11 Vegetative Cover

A grass cover is proposed for the landfill 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.

## 4.0 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 discharge to the Marshall Ash Basin.

### 4.1 Leachate Generation

Leachate generation rates are based on the results of the HELP Model. Leachate generation rates were developed using scenarios to approximate the operating conditions for the landfill. Cell1 and Cell 2 were modeled in HELP at three stages of waste placement (10 feet, mid-height of cell, and cell height at completion), and with the final cover in place.

| HELP Scenario | Area (ac) | Maximum Daily Rate (gallons/ac/day) | Average Annual Rate (gallons/ac/day) |
|---|---|---|---|
| Cell 1 – 10' Waste | 14.8 | 9,158 | 1,040 |
| Cell 2 – 10' Waste | 17.1 | 9,158 | 1,040 |
| Cell 1 and Cell 2 – Final Cover | 31.9 | 1,049 | 1,049 in year 2 after closure. This decreases to 0 after year 6. |

Closure conditions reveal leachate generation estimates of 1049 gallons per acre per day. The HELP analyses show leachate generation after closure conditions to diminish to zero over approximately a 6 year period.

# 5.0 ENVIRONMENTAL CONTROL AND MANAGEMENT

## 5.1 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.

## 5.2 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.

## 5.3 Wetlands and Waters of the US

There are no wetlands or streams affected by the construction of Cell 1. An intermittent stream and wetland area of 0.56 acres are located within the landfill footprint for Cell 2. The stream and wetland area were investigated by a qualified wetlands specialist. Representatives from the US Army Corps of Engineers (USACOE) and from the NCDENR Water Quality Section visited the site and confirmed the presence and jurisdictional boundary of the stream and wetland area (Waters of the U.S.). This was documented per USACOE notification action report ID: 2003300269. Subsequent to the delineation, the stream and wetland area was surveyed and documented on site drawings.

The intermittent stream will be re-routed south of the future Cell 2 area as shown on the Cell 2 Grading Plan. Prior to disturbance of the stream and wetland area, appropriate permitting will be completed.

# 6.0 LANDFILL CAPACITY AND BORROW SOIL

## 6.1 Design Parameters

Landfill capacity calculations were performed for proposed grading and closure plans using anticipated monthly disposal rate data presented previously. The calculations account for intermediate cover and the final cover. The calculations presented in the Appendix of the March 2004 CPA assume the waste generation rate of 350,000 dry tons (438,000 wet weight) per yea

## 6.2 Operating Capacity

The volumetric capacity for the landfill is 1.824 million cubic yards. As discussed in Section 2.2, the combined annual disposal of FGD residue and clarifier sludge is approximately 427,000 $yd^3$. The net volumetric capacity of the landfill is 1,824,160 $yd^3$. This volume will provide approximately 4.3 years of disposal.

## 6.3 Borrow Soil Quantities

Based on the borrow soil quantity calculations presented in the Appendix of the March 2004 CPA, about 525,800 cubic yards of soil will be required for construction, operation and closure of the landfill. Borrow soil from excavation within the landfill footprint will provide approximately 669,400 cubic yards of borrow soil. Excess soil will be stockpiled south of the landfill footprint.

# TECHNICAL SPECIFICATIONS
# FOR
# CONSTRUCTION

**DUKE ENERGY CORP.**
**MARSHALL STEAM STATION**
**FGD RESIDUE LANDFILL**
**CATAWBA COUNTY, NC**

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December 15, 2005

NORTH CAROLINA PROFESSIONAL SEAL 17582 ENGINEER DAN E. BREWER

# Technical Specifications for Construction
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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 Marshall FGD Residue Landfill, Cell 1 and Cell 2 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.

- Compaction: the process of increasing the density or unit weight of soil by rolling, tamping, vibrating, or other mechanical means. Compaction tests are performed 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.

- Geosynthetic Clay Liner (GCL): a manufactured hydraulic barrier consisting of clay bonded to a layer or layers of geosynthetics.

- 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 The landfill 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 The landfill shall be constructed and prepared for receipt of waste in the following sequence:

| SECTION | TITLE |
|---|---|
| 2.0 | EARTHWORK |
| 3.0 | GEOSYNTHETIC CLAY LINER |
| 4.0 | GEOMEMBRANE LINER |
| 5.0 | LEACHATE COLLECTION AND REMOVAL SYSTEM (LCRS) |
| 6.0 | OPERATIONAL COVER |
| 7.0 | STORMWATER COLLECTION AND REMOVAL (SWCR) SYSTEM |
| 8.0 | ROADWAYS |
| 9.0 | LANDFILL CLOSURE (Not In Contract) |

1.3.3. At or near completion of operation (filling of waste) of the landfill, 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 the landfill 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 Marshall FGD Residue Landfill.

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 Marshall FGD Residue Landfill, including, but not limited to excavation, hauling, scraping, stockpiling, and removing accumulated rainwater during construction. The landfill 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

2.2.4.1 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.

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

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

2.2.4.4 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.

2.2.4.5 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

The landfill 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.

Moisture content shall be wet of optimum moisture content at the density specified.

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 GEOSYNTHETIC CLAY LINER

## 3.1 Scope of Work

The GEOSYNTHETIC CONTRACTOR shall furnish all labor, materials, supervision and equipment to complete the Geosynthetic Clay Liner (GCL) for the Marshall FGD Residue Landfill, 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

3.2.1 Earthwork shall be in accordance with Section 2.2.

3.2.2 The geosynthetic clay liner (GCL) used on the landfill floor shall be equivalent to Claymax 600 CL. The GCL for the side slopes shall be stitched reinforced GCL equivalent to Bentomat DN.

3.2.3 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.

3.2.4 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.

3.2.5 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.

3.2.6 The GCL shall be stored in a dry, well drained storage area. The GCL shall be stored under plastic sheets or tarpaulins to prevent hydration.

## 3.3 Construction

3.3.1 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.

3.3.2 Areas to receive GCL 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.

3.3.3 An anchor trench (as illustrated on the Contract Drawings) will be required to secure the GCL. No loose soil will be allowed to underlie the GCL 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).

3.3.4 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.

3.3.5 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.

3.3.6 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.

3.3.7 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).

3.3.8 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.

3.3.9 Once the daily panels are placed and smooth, commence placement of the geomembrane in accordance with Section 4.0 GEOMEMBRANE LINER. 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.

3.3.10 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.

## 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 GCL liner excluding the anchor trench, scrap and overlap. 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.

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.

# 4.0 GEOMEMBRANE LINER

## 4.1 Scope of Work

The GEOSYNTHETIC CONTRACTOR shall furnish all labor, materials, supervision and equipment to complete the Geomembrane Liner for the Marshall FGD Residue Landfill, 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.

## 4.2 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 4-1.

Smooth geomembrane shall be used on the landfill floor. Textured geomembrane shall be used for the landfill side slopes. This textured geomembrane shall extend 10 feet past the toe of the slope, onto the landfill floor.

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.

## 4.3 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.

4.3.1 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.

4.3.2 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 4.3.11).

4.3.3 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.

4.3.4 <u>Geomembrane Field Seaming</u>

4.3.4.1 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.

4.3.4.2 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.

4.3.4.3 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.

4.3.4.4 The completed liner shall not exhibit <u>any</u> "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.

4.3.4.5 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.

4.3.4.6 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.

4.3.4.7 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. <u>Double hot wedge fusion welding shall be the predominant seaming method.</u> Additional concepts and requirements of proper field seaming include the following:

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

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

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

4.3.4.11 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.

4.3.4.12 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.

4.3.4.13 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.

4.3.4.14 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.

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

4.3.4.16 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.

4.3.4.17 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.

4.3.4.18 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.

4.3.5 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:

4.3.5.1 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.

4.3.5.2 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.

4.3.5.3 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.

4.3.6 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.

4.3.7 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.

4.3.8 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.

4.3.9 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.

4.3.10 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.

4.3.11 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.

4.3.12 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.

4.3.13 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.

4.3.14 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.

4.3.15 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

4.3.16 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.

4.3.17 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).

## 4.4 Measurement and Payment

4.4.1 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.

4.4.2 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.

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

Table 4-1
REQUIRED PHYSICAL PROPERTIES OF GEOMEMBRANE LINER (HDPE) SHEET

| Property | Test Method | Required Values 60 Mil HDPE |
|---|---|---|
| Thickness | ASTM D-5994 | 57/60 Mil* |
| Asperity Height (Textured HDPE) | GRI GM12 | 10 mils |
| Sheet Density | ASTM D-1505/D 792 | 0.940 g/cm$^3$ min. |
| Tensile Properties | | |
| Yield Strength | ASTM D 6693 | 126 lb/in |
| Break Strength | ASTM D 6693 | 90 lb/in |
| Yield Elongation, % | ASTM D 6693 | 12 |
| Break Elongation, % | ASTM D 6693 | 12 |
| Carbon Black Content | ASTM D-1603 | 2% min-3% max |
| Carbon Black Dispersion | ASTM D-5596 | 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 | ASTM D 4833 | 90 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 3 psi for 15 seconds minimum. |
| 2. Double Weld | Air Testing | Maintain 30 psi for not greater than 3 psi for last 30 minutes |

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

Table 4-2

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

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

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

# 5.0 LEACHATE COLLECTION AND REMOVAL SYSTEM (LCRS)

## 5.1 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, 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.

## 5.2 Materials

### 5.2.1 Pipe and Sump

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

PE piping inside the landfill 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.

Table 5-1 HDPE Pipe Material Requirements

| 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.

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.

5.2.2 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 5.2.3.

5.2.3 Nonwoven Geotextile

Nonwoven geotextile for the cell floor, side slopes, and leachate pipe shall be 8 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 geotextile material shall meet or exceed the following criteria using Minimum Average Values:

**Table 5-2 Nonwoven Geotextile Material Requirements**

| Property | ASTM Test Method | 8 oz Geotextile Required Value | 12 oz Geotextile Required Value |
|---|---|---|---|
| Fabric Weight | D3776 | 7.5 oz/yd2 | 11.5 oz/yd2 |
| 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 | D4751 | 100 Sieve | 70 Sieve |
| Size (AOS) | | | |
| 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.

5.2.4 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.

| 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 | D1505 or D792 | Every 50,000 $ft^2$ |
| Carbon Black Content | D1603 | Every 50,000 $ft^2$ |
| Transmissivity | D4716 | One Test Per Landfill Cell |

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.

5.2.5 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.

5.2.6 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.

## 5.3 Construction

### 5.3.1 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, as shown on the Contract drawings.

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 on the Contract Drawings.

### 5.3.2 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 and gravel surrounding 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.

### 5.3.3 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.

### 5.3.4 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.

5.3.5 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.

5.3.6 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.

## 5.4 Measurement and Payment

5.4.1 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.

5.4.2 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.

5.4.3 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.

5.4.4 Payment for geotextile, geonet, and geocomposite 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.

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

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

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

5.4.8 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.

# 6.0 OPERATIONAL COVER

## 6.1 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 for the Marshall FGD Residue Landfill, including but not limited to excavation, hauling, scraping, stockpiling, placement, maintenance, and removing accumulated rain water during construction. The operational cover shall be constructed to the lines and grades shown on the Contract Drawings.

## 6.2 Materials

6.2.1 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.

6.2.2 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.

6.2.3 The stormwater drainage channel 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.

## 6.3 Construction

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

6.3.2 An approved method of placing the operational cover on top of the LCRS 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.

6.3.3 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.

6.3.4 Construction of the operational cover 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.

6.3.5 The temporary cover liner shall be used to line the stormwater channels following thickness verification. The EARTHWORK CONTRACTOR shall be responsible for anchoring the temporary cover liners.

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

## 6.4 Measurement and Payment

6.4.1 Payment will be based on in-place quantities. Measurement of the operational cover 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.

6.4.2 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.

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

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

6.4.5 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.

6.4.6 Payment for the temporary cover liner used to line the stormwater channels will be made at the contract price per square foot of installed material. This price shall include furnishing, installing, and anchoring.

# 7.0 STORMWATER COLLECTION AND REMOVAL (SWCR) SYSTEM

## 7.1 Scope of Work

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

## 7.2 Materials

Pipe for the SWCR system shall be 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.

## 7.3 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 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.

## 7.4 Measurement and Payment

7.4.1 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.

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

# 8.0 ROADWAYS

## 8.1 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.

## 8.2 Materials

### 8.2.1 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.

## 8.3 Construction

### 8.3.1 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.

## 8.4 Measurement and Payment

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

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

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

8.4.4 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.

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

## 9.0 LANDFILL CLOSURE (NOT IN CONTRACT)

The scope of work and materials required to perform closure of the landfill is provided in the document, .Technical Specifications for Closure.

Drawings under Seperate Cover

Carmen Johnson

| Fac/Perm/Co ID # | Date | Doc ID# |
|---|---|---|
| 18-09 | 1/25/12 | DIN |

# CONSTRUCTION PLAN APPLICATION

**DUKE POWER**
**MARSHALL STEAM STATION**
**CATAWBA COUNTY, NC**

**FLUE GAS DESULFURIZATION (FGD) GYPSUM LANDFILL**

RECEIVED
DEC 2 9 2005
NC DEPT OF ENVIRONMENT
AND NATURAL RESOURCES
MOORESVILLE REGIONAL OFFICE

APPROVED
DIVISION OF WASTE MANAGEMENT
SOLID WASTE SECTION
DATE 6-5-06 BY

Prepared for:
NCDENR
Solid Waste Section
Raleigh, North Carolina

Prepared by:

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

128 Overhill Drive, Suite 105
Mooresville, NC 28117

Revised December 15, 2005

Document MM6451.00-0000.001, Revision 1
ERN No. FHS0004Z

**Volume 2**

# CALCULATIONS

**Landfill Capacity/Borrow Soil Quantities** *See March 2004 CPA*

**Stability**

- Stability of Side Slope Liner System
- Evaluation of Stresses Due to Liner Self Weight
- Slope Stability Evaluation

**Foundation Evaluation**

- Subgrade Bearing Capacity *See March 2004 CPA*
- Subgrade Settlement Evaluation *See March 2004 CPA*

**Geosynthetics**

- Geotextile Design
- Geonet Design
- Anchor Trench Design

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**Leachate Generation**

- Help Model, Discussion and Results

**Leachate Collection and Removal Pipe**

- LCR Pipe Flow Calculations
- LCR Pipe Stresses

**Stormwater Management**

- Cell 1 Stormwater Pond
- Cell 2 Stormwater Pond

*Also See March 2004 CPA for other stormwater calculations*

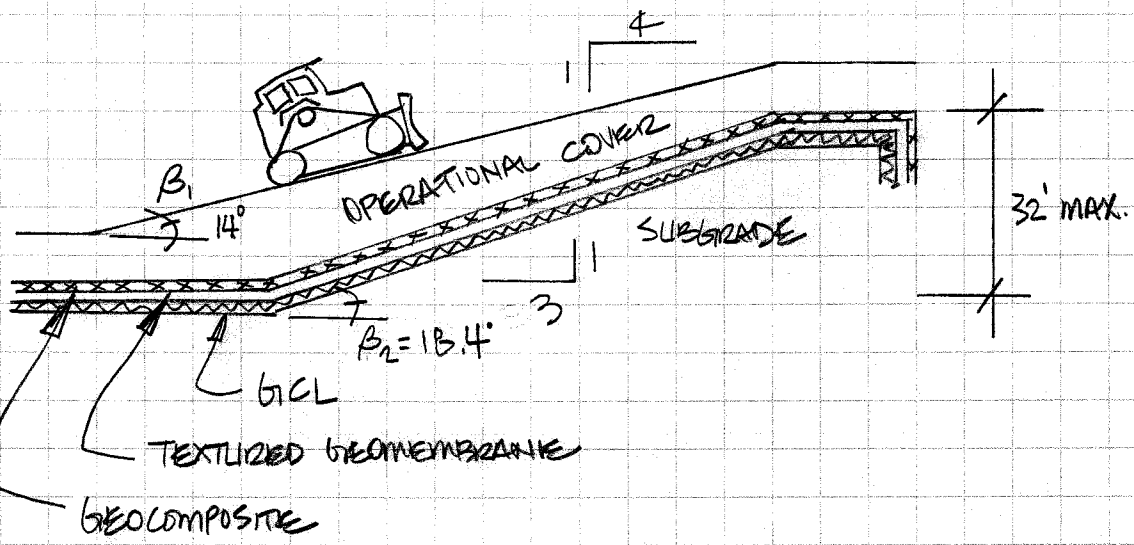
## Stability

- Stability of Side Slope Liner System
- Evaluation of Stresses Due to Liner Self Weight
- Slope Stability Evaluation

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

Comp. By DHB Date 12-13-05 Project MARSHALL EGD LANDFILL Sheet No. 1 of
Chkd. By DEB Date 12-20-05 Subject STABILITY OF SIDE SLOPE LINER SYSTEM Job No. 046003.001

EVALUATE STABILITY OF SIDE SLOPE LINER SYSTEM

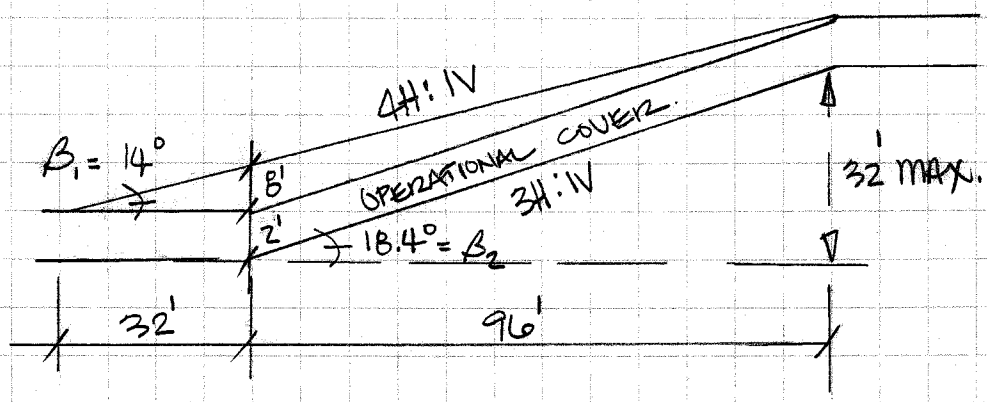



OPERATIONAL COVER SIDE SLOPE = 4H:1V, $\beta_1 = 14°$
MAX BERM HEIGHT = 32'
$\gamma$ GYPSUM = 30.7° (PER S&ME TRIAXIAL SHEAR REPORT)
$\gamma$ SOIL = 100 PCF, $\phi$ SOIL = 30°

LINER DESIGN: GEOCOMPOSITE
60 MIL. TEXTURED GEOMEMBRANE
GCL.

FRICTION ANGLES: OPERATIONAL COVER / GEOCOMPOSITE $\delta_1 = 34°$
GEOCOMPOSITE / TEXTURED GEOMEMBRANE $\delta_2 = 26°$
TEXTURED GEOMEMBRANE / GCL $\delta_3 = 15°$
GCL / SUBGRADE $\delta_4 = 23°$

$SF_{MIN.}$ AGAINST SLIDING = 1.5




ASSUME EQUIP. IS CAT LGP DOZER : 18,200 lb.
2 TRACKS, 2 FT. WIDE.

$$W_D = \frac{18{,}200 \text{ lb}}{2 \text{ TRACKS } (2 \text{ FT})} = 4{,}550 \frac{\text{lb}}{\text{ft.}}$$

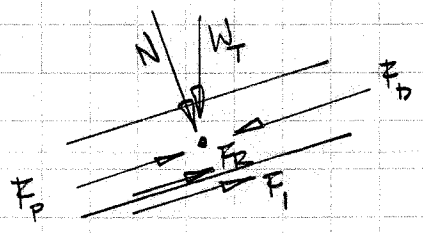
WEIGHT OF SOIL: $W_S$ FOR 3:1 SLOPE $= 100 \frac{\text{lb}}{\text{ft}^3} (96 \text{ ft})(2 \text{ ft}) = 19{,}200 \frac{\text{lb}}{\text{ft}}$

$W_S$ FOR 4:1 SLOPE $= 100 \frac{\text{lb}}{\text{ft}^3} \frac{(8 \text{ ft})(96 \text{ ft})}{2} = 38{,}400 \text{ lb/ft.}$

$W_S$ TOTAL $= 57{,}600$ lb/ft.

$W_T = W_S + W_D = 62{,}150$ lb/ft.

OPERATIONAL COVER/ GEOCOMPOSITE : $\delta_1 = 34°$




$N = W_T \cos \beta_2 = 62{,}150 \cos 18.4° = 58{,}973$ lb/ft.
$F_D = W_T \sin \beta_2 = 62{,}150 \sin 18.4° = 19{,}618$ lb/ft.
$F_P = \frac{1}{2} \gamma H^2 \tan^2 (45 + \phi/2) = \frac{1}{2} (100 \text{ lb/ft}^3)(10 \text{ ft})^2 \tan^2 (45 + 30/2)$
$= 15{,}000$ lb/ft.

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

Comp. By DHB Date 12-13-05 Project MARSHALL EGD LANDFILL Sheet No. 3 of
Chkd. By DEB Date 12/20/05 Subject STABILITY OF SIDE SLOPE LINER SYSTEM Job No. 046003.001

$F_1$ AVAILABLE $= N \tan \delta_1 = 58{,}973 \text{ lb/ft} \tan 34° = 39{,}778 \text{ lb/ft.}$

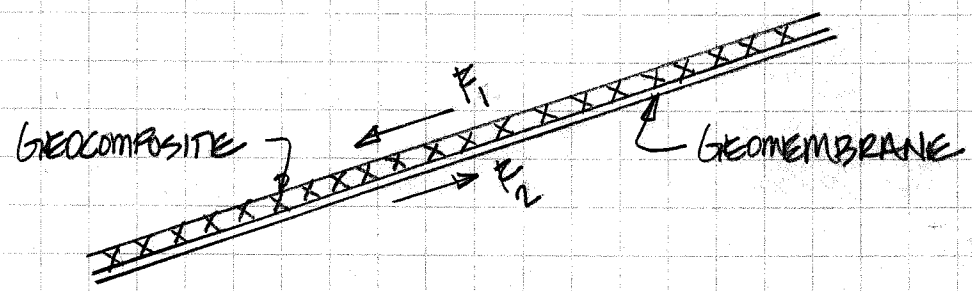
$F_R = F_D - F_P = 19{,}618 - 15{,}000 = 4{,}618 \text{ lb/ft.}$

$F_R < F_1$ AVAILABLE $\therefore$ AT REST, NO TENSION

$F_1 = F_R = 4{,}618 \text{ lb/ft.}$

$F.S. = \dfrac{39{,}778}{4{,}618} = 8.6$ OK

## GEOCOMPOSITE / TEXTURED GEOMEMBRANE

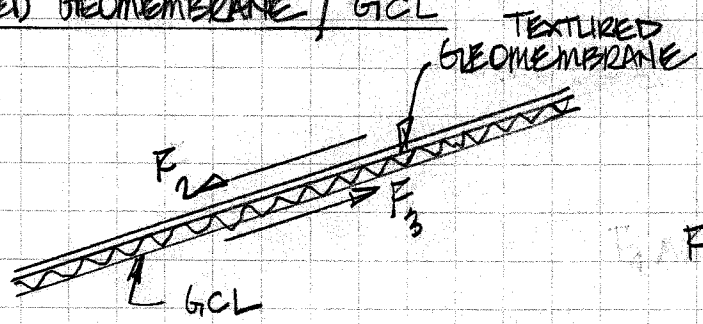



$F_1 = 4{,}618$ lb/ft.

$F_2 = N \tan \delta_2$
$= 58{,}973$ lb/ft $\tan 26°$
$= 28{,}763$ lb/ft. $\therefore F_1 < F_2$
AT REST - NO TENSION

$F.S. = \frac{28{,}763}{4{,}618} = \underline{\underline{6.2}}$ $\underline{\underline{OK}}$

## TEXTURED GEOMEMBRANE / GCL

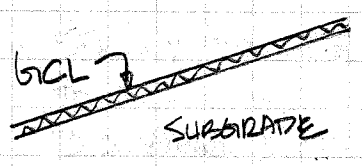



$F_2 = 4{,}618$ lb/ft.

$F_3 = N \tan \delta_3$
$= 58{,}973 \tan 15°$
$= 15{,}802$ lb/ft $\therefore F_2 < F_3$
AT REST - NO TENSION

$F.S. = \frac{15{,}802}{4{,}618} = \underline{\underline{3.4}}$ $\underline{\underline{OK}}$

## GCL / SUBGRADE




$F_3 = 4618$ lb/ft.

$F_4 = N \tan \delta_4$
$= 58{,}973 \tan 23°$
$= 24{,}956$ lb/ft $\therefore F_3 < F_4$
AT REST - NO TENSION

$F.S. = \frac{24{,}956}{4{,}618} = \underline{\underline{5.4}}$ $\underline{\underline{OK}}$

GSE™

# 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 | 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.*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*For environmental lining solutions...the world comes to GSE.™*

*A Gundle/SLT Environmental, Inc. Company*

Reference (1)



$c$ = the cohesion of soil,

$\delta$ = the friction angle of soil to fabric, and

$\phi$ = 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 $\phi$ = 30 deg. | Rounded sand $\phi$ = 28 deg. | Silty sand $\phi$ = 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]

YSTEMS

s. This may occur

rally has a high efficiency
; generally higher for wo-
nded nonwovens. This is
f soil-to-fabric interaction
factors or trends observed
iles include:

tiles decreases the shear
i et al., 1986).
tions produce different in-
ypically being lower (Ei-

ct on the interface friction
geotextiles (Eigenbrod and

due to the interlocking of
woven geotextiles (Eigen-

and high confining
e adhesion decrease
(Williams and Houli-

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

in openings or pores, the
gely dependent on whether
gh to push the failure plane
nto the adjacent soils, the

REFERENCE 1



**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, $\delta$ (deg) | Reported Clay Friction Angles (deg) (Efficiency) | Recommended Clay Friction Angles, $\delta$ (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.

REFERENCE 1



**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 | — |

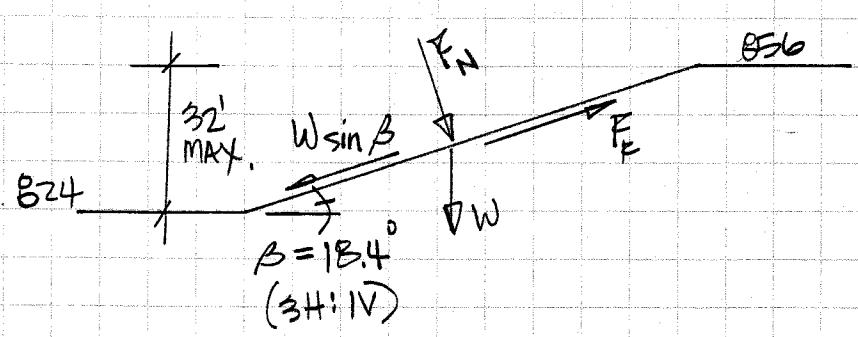
The testing conditions may also have a significant effect on results. Mitchell et al. (1990) noted that polishing of geomembrane surfaces by geotextiles reduced interface friction. Also, the orientation of geonet strands can affect the interface strength between geonets and geomembranes (Geotek, 1987; 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.

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

$T = W \sin\beta - F_F$ WHERE W = WEIGHT OF LINER SYSTEM, lb/ft
$\beta$ = ANGLE OF SLOPE, DEGREES.
$F_F$ = FRICTION FORCE.

W = WT. OF LINER SYSTEM X LENGTH ALONG SLOPE

$$W = \left[\left(0.941 \times 62.4\,\frac{lb}{ft^3}\left(\frac{0.060''}{12''/ft}\right)\right) + \left(0.20\ lb/ft^2\right)\right]\left(32'/\sin 18.4°\right)$$

[WEIGHT GEOMEMBRANE + WEIGHT GEONET] (LENGTH SLOPE).

$$= \left(0.294\ lb/ft^2 + .020\ lb/ft^2\right)(101.4\ ft) = 31.8\ lb/ft.$$

$F_N = W \cos \beta = 31.8 \frac{lb}{ft} \cos 18.4° = 30.2 \ lb/ft.$

$F_F = F_N \tan \delta_1$ WHERE $\delta_1$ = FRICTION ∠ BETWEEN SOIL & GEOMEMBRANE = 22°

$= 30.2 \frac{lb}{ft} \tan 22°$

$= 12.2 \ lb/ft.$

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.

$F_F = 12.2 \ lb/ft.$ $W \sin \beta = 31.8 \ lb/ft \sin 18.4° = 10.04$

$F_F > W \sin \beta$ ∴ 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 | $2 \times 10^{-4}$ |
| TexDrain 200 SS 8 | 0.48 | $1 \times 10^{-4}$ |
| TexDrain 200 DS 6 | 0.48 | $1 \times 10^{-4}$ |
| TexDrain 200 DS 8 | 0.19 | $4 \times 10^{-5}$ |
| TexDrain 250 SS 6 | 2.17 | $4.5 \times 10^{-4}$ |
| TexDrain 250 SS 8 | 1.2 | $2.5 \times 10^{-4}$ |
| TexDrain 250 DS 6 | 1.44 | $3 \times 10^{-4}$ |
| TexDrain 250 DS 8 | 0.84 | $1.75 \times 10^{-4}$ |

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

CETCO® LINING TECHNOLOGIES
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| | | | | |
|---|---|---|---|---|
| Immersion in standard liquids | ASTM D471, D543 | ASTM D471 | ASTM D543 | ASTM D471, D543 |
| Immersion in waste liquids | EPA 9090 | EPA 9090 | EPA 9090 | EPA 9090 |
| Soil burial | ASTM D3083 | ASTM D3083 | ASTM D3083 | ASTM D3083 |
| Outdoor exposure | ASTM D4364 | ASTM D4364 | ASTM D4364 | ASTM D4364 |
| Tub test | b | b | b | b |

[a]See reference (8).
[b]See reference (12).
NA = not applicable.
Source: Haxo, 1987

**Cell Component:** FLEXIBLE MEMBRANE LINER

**Consideration:** TENSILE STRESS - LINER WEIGHT; EVALUATE ABILITY OF FML TO SUPPORT ITS OWN WEIGHT ON THE SIDE SLOPES.

| Required Material Properties | Range | Test | Standard |
|---|---|---|---|
| FML SPECIFIC GRAVITY, G | 0.92 TO 1.4 | | |
| FRICTION ANGLE | | | |
| • FML-TO-LCR, $\delta_L$ | 10° TO 45° | DIRECT SHEAR | PROPOSED ASTM |
| FML THICKNESS, t | 30 TO 120 MIL | | |
| FML YIELD STRESS, $\sigma_Y$ Draft MTG Minimum | 1000 TO 5000 (P&L) | TENSILE | ASTM D638 |

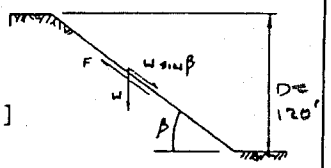
**Analysis Procedure:**

(1) CALCULATE FML TENSILE FORCE, T

$T = W \sin\beta - F$

WHERE $W$ = LINER WEIGHT $= [G \gamma_w t][1 \times D/\sin\beta]$

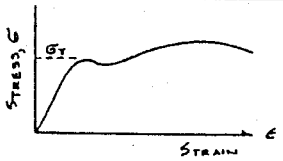
$F = W \cos\beta \tan\delta_L$




(2) CALCULATE FML TENSILE STRESS, $\sigma$

$\sigma = T/A$ WHERE A = AREA = 1' × t

(3) OBTAIN LABORATORY FML YIELD STRESS, $\sigma_Y$




(4) CALCULATE DESIGN RATIO

$DR = \sigma_Y / \sigma$

**Design Ratio:** $DR_{MIN} = 10$ ON YIELD

**References:**

**Example:**

GIVEN:
- 60 MIL HDPE
- FML SPECIFIC GRAVITY, G = 0.941
- FRICTION ANGLE FML-TO-LCR, $\delta_L = 20°$
- D = 120 FT
- $\beta = 30°$

(1) CALCULATE FML TENSILE FORCE, T

$W = [0.941 \times 62.4 \times \frac{.060}{12}] = [1 \times 120/\sin 30°]$
$= 70.5$ lb/FT

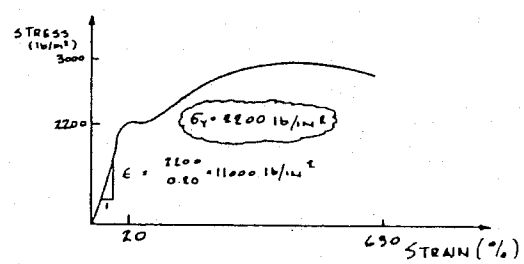
$F = 70.5 \cos 30° \tan 20°$
$= 22.2$ lb/FT

$T = 70.5 \sin 30° - 22.2$
$= \underline{13.0 \text{ lb/FT}}$

(2) CALCULATE FML TENSILE STRESS, $\sigma$

$\sigma = 13.0/(1 \times \frac{.060}{12}) = 2600$ lb/FT²
$= 18.0$ lb/IN²

(3) OBTAIN LABORATORY FML YIELD STRESS




(4) CALCULATE DESIGN RATIO

$DR = 2200/18 = 122$ OK

Example 1 5.14

# Marshall FGD Residue Landfill

# Slope Stability Evaluation

## 1.0 OBJECTIVE

The purpose of this report is to evaluate the slope stability for the Marshall FGD Residue Landfill.

## 2.0 BACKGROUND

The previous stability evaluation was performed by Shaw [Reference 1] and evaluated the slope stability of the landfill. This evaluation was based on gypsum being disposed in the landfill. The properties of the clarifier sludge material will be different from the properties of gypsum; therefore, the stability of the landfill containing the clarifier material, as well as a mix of clarifier sludge and gypsum (1 part each) will be evaluated.

## 3.0 ASSUMPTIONS

The following assumptions are used in performing the slope stability evaluations:

1. Clarifier Sludge material properties – the shear strength properties and unit weights were developed as a result of testing performed on material that Duke Power believes will be similar to that produced by the FGD system at Marshall.
2. Gypsum material properties - the shear strength properties and unit weights were developed as a result of testing performed on material that Duke Power believes will be similar to that produced by the FGD system at Marshall.
3. In-place Densities – It is assumed that the in-place densities for the clarifier sludge and gypsum materials will be achieved by placement of the material in accordance with the Operational Plan.
4. Site Soil material properties – the shear strength properties for the site soils are assumed, based on evaluation of SPT results.
5. Groundwater Surface Profile – the groundwater surface profile used in the analyses is that from the Estimated Seasonal High Groundwater information found in the site hydro-geo report [Reference 2].
6. The stability of the final cover system is not evaluated in this section. Separate calculations are performed for that evaluation. The layers underneath the waste (geomembrane and geosynthetic clay layer) are modeled in the evaluation.
7. The landfill is assumed to be built to the final configuration shown on the design drawings. The working face of the landfill is assumed to be no steeper that 3:1 (H:V).

## 4.0 METHOD

### General

The slope stability analysis is performed by use of the program, STABL FOR WINDOWS 2.0. The geometry of the landfill and subgrade layers are used along with the material unit weights and strength properties to determine the factors of safety against slope failure.

The trial slip circle surfaces are generated in STABL by the user defining the region of the slope where the surface initiates and terminates along the surface of the slope. As recommended in the STABL User's manual, the initial search for critical failure surfaces was first performed using the Bishop Simplified Method. Once the critical slope initiation and termination regions are defined using the Bishop Simplified Method, Spencer's Method was then run to determine the Critical Factor of Safety. [Reference 3]

The analysis includes both static and seismic analyses. The seismic analyses were performed using static methods (non-dynamic) with accelerations applied in the horizontal and vertical directions. The peak acceleration shown on the USGS Seismic Hazards Map[1], Oct 2003, for the areas where the landfill is located is between 0.12g and 0.14g [Reference 4]. A value of 0.15g (horizontal) and 0.10g vertical is used in the analysis.

## Landfill Geometry and Cross Sections Analyzed

The final configuration of the landfill is for Cell1 shown on Drawing M-6024-01.00. The final configuration for Cell 1 and Cell 2 is shown on Drawing M-6024-06.00. The final slopes range from approximately 7:1 (H:V) to 12:1. The excavated surface of the landfill is shown on drawing M-6024-02.00 (Cell 1) and M-6024-05.00 (Cell 2).

The working face of the landfill, as construction progresses in Cell 1, from north to south, and into the operation of Cell 2, is assumed to never be steeper than 3:1.

The maximum elevation of the landfill is Elevation 920', along an axis that generally runs north-south. The highest excavated grade for the landfill is at the north end of the landfill at approximately elevation 850'. The lowest excavated grade occurs on the southern end of the landfill at elevation 824'. The highest elevation of the landfill occurs during construction of Cell 2 at elevation 920'. The elevation of the excavated grade, underneath the location of the highest surface elevation, is approximately elevation 836'. This gives a maximum height of fill of 84 feet. The maximum elevation of Cell 1 is 910'.

The cross sections analyzed are identified by the stationing along the road around the landfill.

[1] Peak Acceleration (5) g with 2% Probability of Exceedance in 50 Years, USGS Map, Oct. 2002.

The following cross sections slopes were analyzed:

| Cross Section | Landfill Stage of Operation | Slope of Analyzed Face | Slope Facing |
|---|---|---|---|
| Station 22+00 | Cell1 at El. 910' | 3:1 slope of working face | South |
| Station 15+00 East | Cell 1 at El. 910' | 5:1 | East |
| Station 42+50 | Cell 2 Complete at El. 920' | 9:1 | South |

These slopes and configurations are considered to account for the slope conditions requiring evaluation.

## Material Properties

The material properties used in the analyses are presented in the following table. Actual saturated unit weights are only required for materials below the ground water surface. For materials located above the groundwater surface, the values for saturated unit weights are not used in the calculations.

| Material | Unit Weight Dry/Saturated (pcf) | Cohesion Intercept (psf) | Friction Angle (degrees) |
|---|---|---|---|
| Clarifier Sludge | 66/66 | 600 | 6 |
| Clarifier Sludge: Gypsum (1:1 mixture by volume) | 60/60 | See below | See below |
| UU - Unconsolidated – Undrained Test Results | - | 370 | 26.9 |
| UU - Unconsolidated – Undrained Test Results | - | 630 | 21.1 |
| HDPE Liner | 60/60 | 0 | 8 |
| GCL Layer | 60/60 | 500 | 0 |
| Site Soil (above auger refusal) | 85/118 | 0 | 25 |
| Site Soil (below auger refusal) | 85/118 | 0 | 38 |

Shear strength values for the materials were determined as described below.

Clarifier Sludge

Triaxial tests were performed on clarifier sludge. Only unconsolidated- undrained (UU) tests were performed. Shear strength values for the Clarifier Sludge are taken from the p-q plots. Copies of the triaxial test results are included at the end of this report.

Clarifier Sludge/Gypsum Mix
Triaxial tests were performed on a 1:1 mix of clarifier sludge and gypsum. Both unconsolidated- undrained (UU) and consolidated-undrained (CU) tests were performed. Shear strength values for the Clarifier Sludge/Gypsum mix are taken from the p-q plots. Copies of the triaxial test results are included at the end of this report.

Unconsolidated-undrained (UU) values were used for the stability analysis.

Site Soils- Above Auger Refusal
Shear strength values for site soils above auger refusal (N>50)were developed from correlating Standard Penetration Test blow counts with shear strength values found in Table 3-4 in Bowles [Reference 5]. Copies of the boring logs are included at the end of this report.

The minimum SPT blow counts for the borings in or near the landfill footprint were reviewed. The minimum blow count in these borings was 5.

| Location | Borings |
|---|---|
| Inside the Landfill Footprint | B-2, B-3, B-6, MS-7, MS-8, OW-1, OW-2 |
| Outside the Landfill Footprint | MS-6, MS-9, MS-10, MS-11, B-4 |

Table 3-4 in Bowles [Reference 5] provides a table of empirical values for Φ angle, relative density, and SPT values. The minimum value listed for Φ is 26°. A value for Φ of 25° is conservatively used in these analyses. The site soils are assumed to have zero cohesion strength.

Site Soils- Above Auger Refusal
The shear strength for site soils below auger refusal is taken to be equal to a Φ angle of 38° and zero cohesion.

## 5.0 RESULTS

The initial analyses were performed for the landfill considering clarifier sludge alone as the material placed in the landfill. Since only one sample was subjected to shear strength tests, and since the factor of safety for the seismic analysis was marginal (less than1.15), a second analysis was perform using slightly lower shear strength values. The purpose of this second analysis was to evaluate the sensitivity of the seismic factor of safety to lower strength values.,

The minimum factors of safety for the analyzed slopes are presented in the following tables.

**Factors of Safety - Clarifier Sludge Only**
**C = 500 psf, Φ = 5°**

| Cross Section | Landfill - Stage of Operation | Slope of Analyzed Face | Minimum Factor of Safety (Analysis Method) | Figure Showing Critical Surface |
|---|---|---|---|---|
| Station 22+00 | Cell1 at El 910' | 3:1 slope of working face | Static FS = 1.408 (Spencer)<br><br>Seismic FS = 0.971 (Spencer) | Figure 1<br><br>Figure 1a |

**Factors of Safety - Clarifier Sludge Only**
**C = 600 psf, Φ = 6°**

| Cross Section | Landfill - Stage of Operation | Slope of Analyzed Face | Minimum Factor of Safety (Analysis Method) | Figure Showing Critical Surface |
|---|---|---|---|---|
| Station 22+00 | Cell1 at El 910' | 3:1 slope of working face | Static FS = 1.671 (Spencer)<br><br>Seismic FS = 1.1212 (Spencer) | Figure 2<br><br>Figure 2a |

As noted, the Factor of Safety for the seismic case is less than 1.15. A factor of safety of 1.15 or greater is typically used as providing an acceptable margin of safety against slope failure for a seismic loading condition. Therefore, until additional test information is available, it is unacceptable to place clarifier sludge alone in the landfill.

The remainder of the slope stability analyses are performed by using the strength values for clarifier sludge mixed with gypsum at a 1:1 ratio.

**Factors of Safety - Clarifier Sludge Mixed with Gypsum (1 to 1 mix by volume)**

| Cross Section | Landfill - Stage of Operation | Slope of Analyzed Face | Minimum Factor of Safety (Analysis Method) | Figure Showing Critical Surface |
|---|---|---|---|---|
| Station 22+00 | Cell1 at El 910' | 3:1 slope of working face | Seismic FS = 1.265 (Spencer)<br><br>Static FS = 2.12 (Spencer)<br><br>Seismic FS = 1.627 (Sliding Block) | Figure 3<br><br>Figure 3a<br><br>Figure 3b |
| Station 15+00 East | Cell 1 at El 910' | 5:1 | Seismic FS = 1.704 (Spencer)<br><br>Static FS = 3.466 (Spencer) | Figure 4<br><br>Figure 4a |
| Station 42+50 South | Cell 2 Complete | 9:1 | Seismic FS = 1.728 (Spencer)<br><br>Static FS = 4.654 (Spencer) | Figure 5<br><br>Figure 5a |

Only one cross section was analyzed for stability by using a sliding block approach. Station 22+00 was analyzed using the sliding block approach. After reviewing the factor of safety from this approach and comparing the factors of safety using the circular surface search, it was determined that no other cross sections would be analyzed using the sliding block method.

Plots showing each of the cross sections and the locations of the critical surfaces found in the analyses are attached.

## 6.0 CONCLUSIONS

1. Seismic factors of safety for a 3:1 slope working face of clarifier sludge alone are at or less than 1.15. A safety factor of 1.15 is generally considered to be the minimum acceptable factor of safety for seismic conditions. Due to the limited information on the strength of the placed material, it is recommended that clarifier sludge alone not be placed, unless further testing and analysis determine that use of this material alone is acceptable.

2. Factors of safety for a material composed of 1 part clarifier sludge and 1 part gypsum are greater than 1.5 for static conditions and 1.15 for seismic conditions. Slopes constructed of this mixture of material is should be stable when placed at the densities used in these analyses.

3. Testing of in-place material should be performed to verify the shear strengths and in-place densities used in these analyses.

## 7.0 REFERENCES

1 Calculation MC-6451.00-00-0001, *Gypsum Landfill Stability*, Dated 3/29/04.

2 *Duke Power Marshall Steam Station, Catawba County, NC Flue Gas Disposal (FGD) Residue Landfill Permit Application Site Suitability Information, August 20, 2003, Hydrogeologic Study, S&ME Project No. 1264-02-578.*

3 *STABL FOR WINDOWS, Version 2.0 User's Manual.*

4 USGS Seismic Hazard Map – Eastern US – address:
*http://earthquake.usgs.gov/hazmaps/products_data/2002/2002October/CEUS/CEUSpga2500v3.pdf*

5 *Foundation Analysis and Design*, Bowles, 4th Edition, 1988.

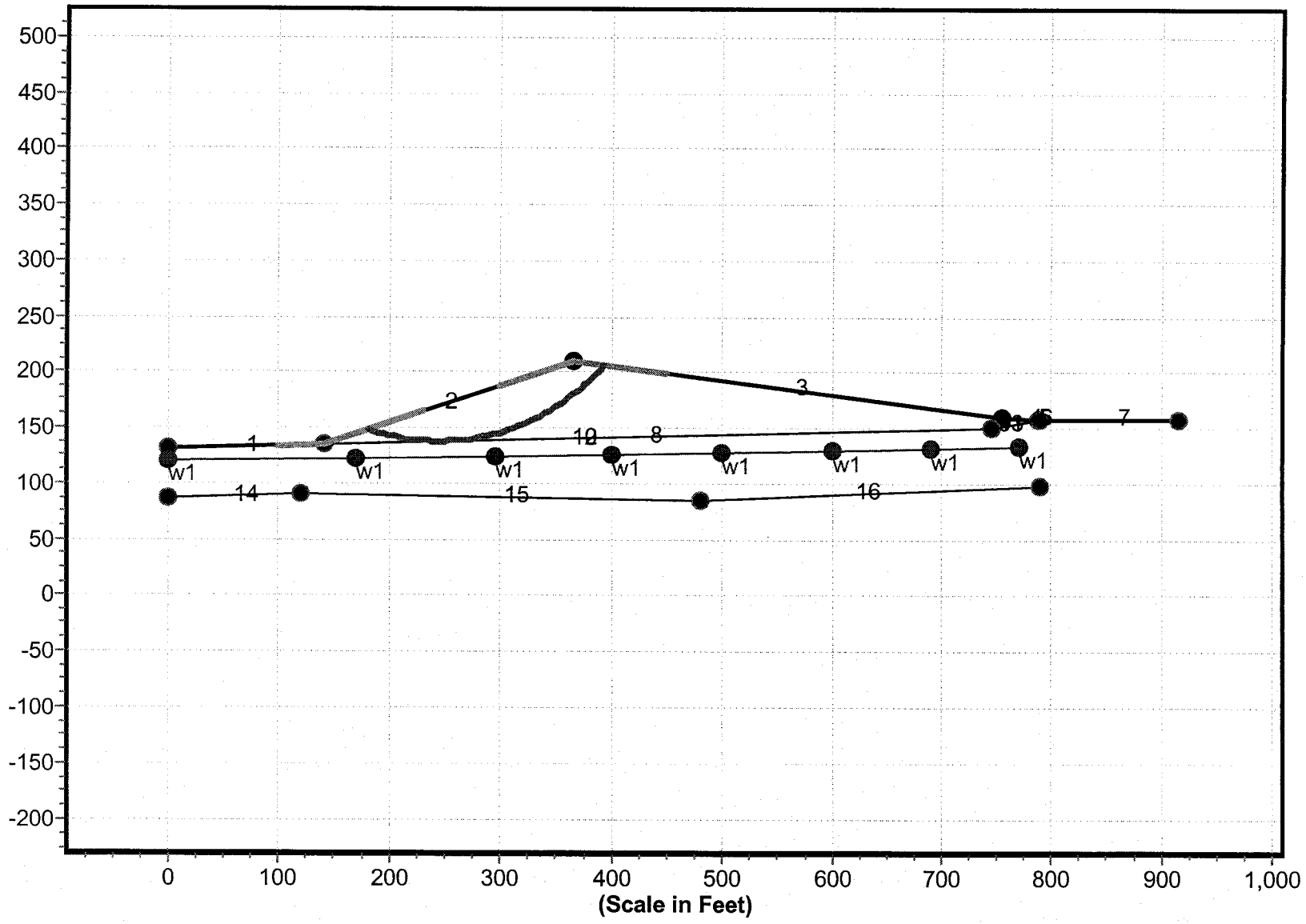
# Marshall FGD Residue Landfill

## Slope Stability Evaluation

## Critical Failure Surfaces

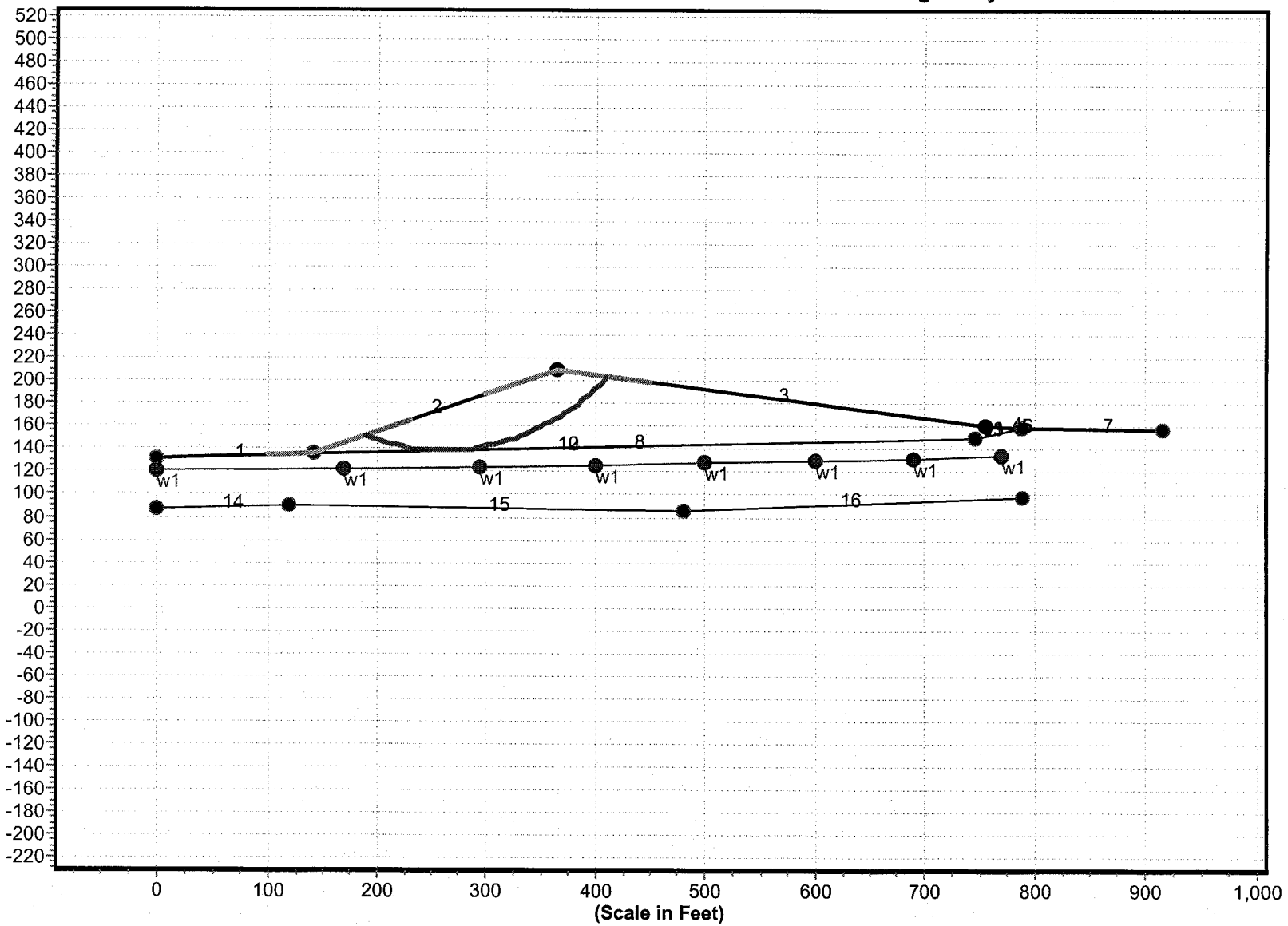
## Geometry and Boundary Conditions

**Problem: Figure 1 - MSS FGD LF - Station 22+00 - South Face - Clarifier Sludge Only - FS Min = 1.408**



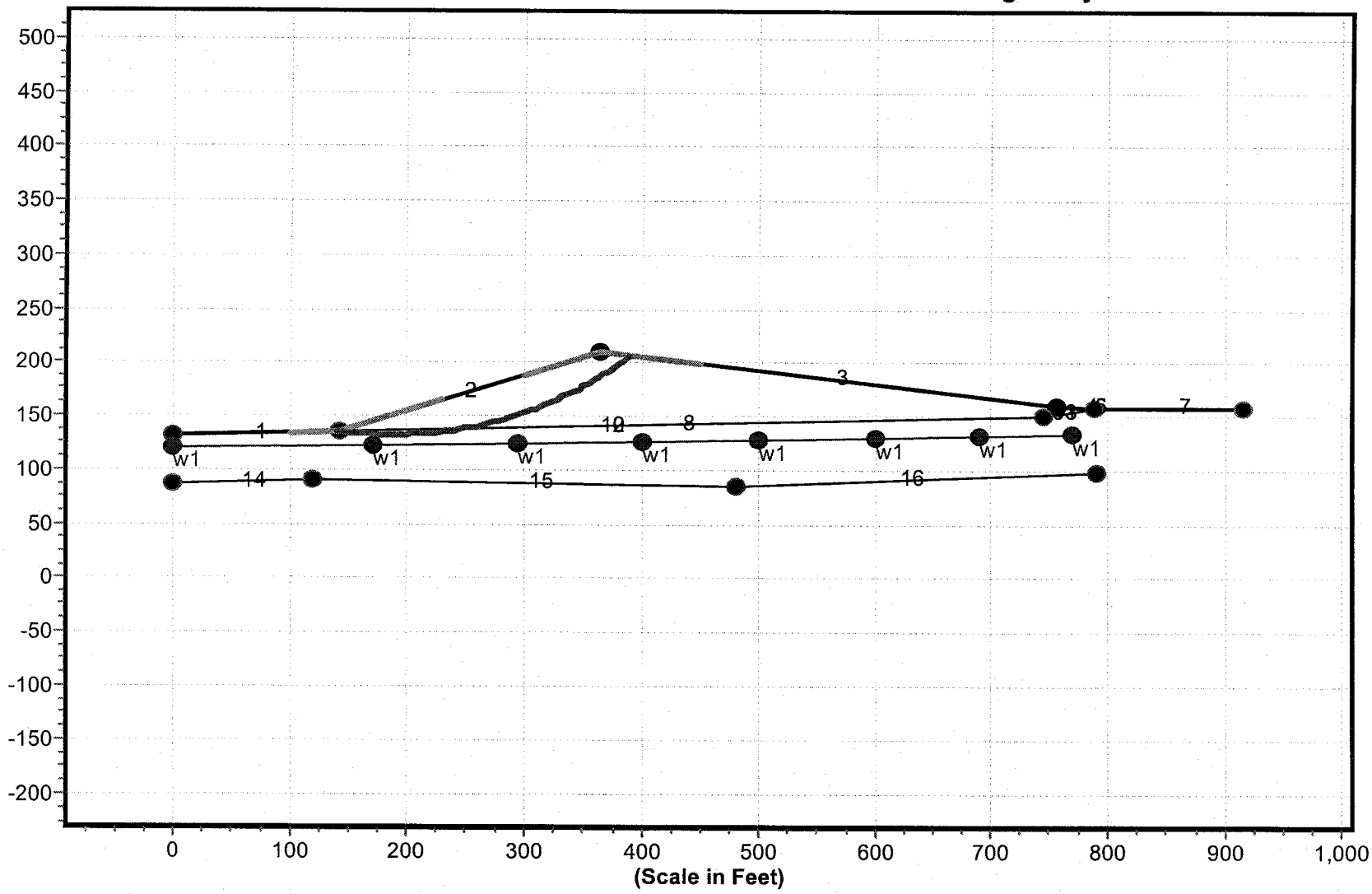
## Geometry and Boundary Conditions

**Problem: Figure 1a - MSS FGD LF - Station 22+00 - South Face - Clarifier Sludge Only - FS Min = 0.971**



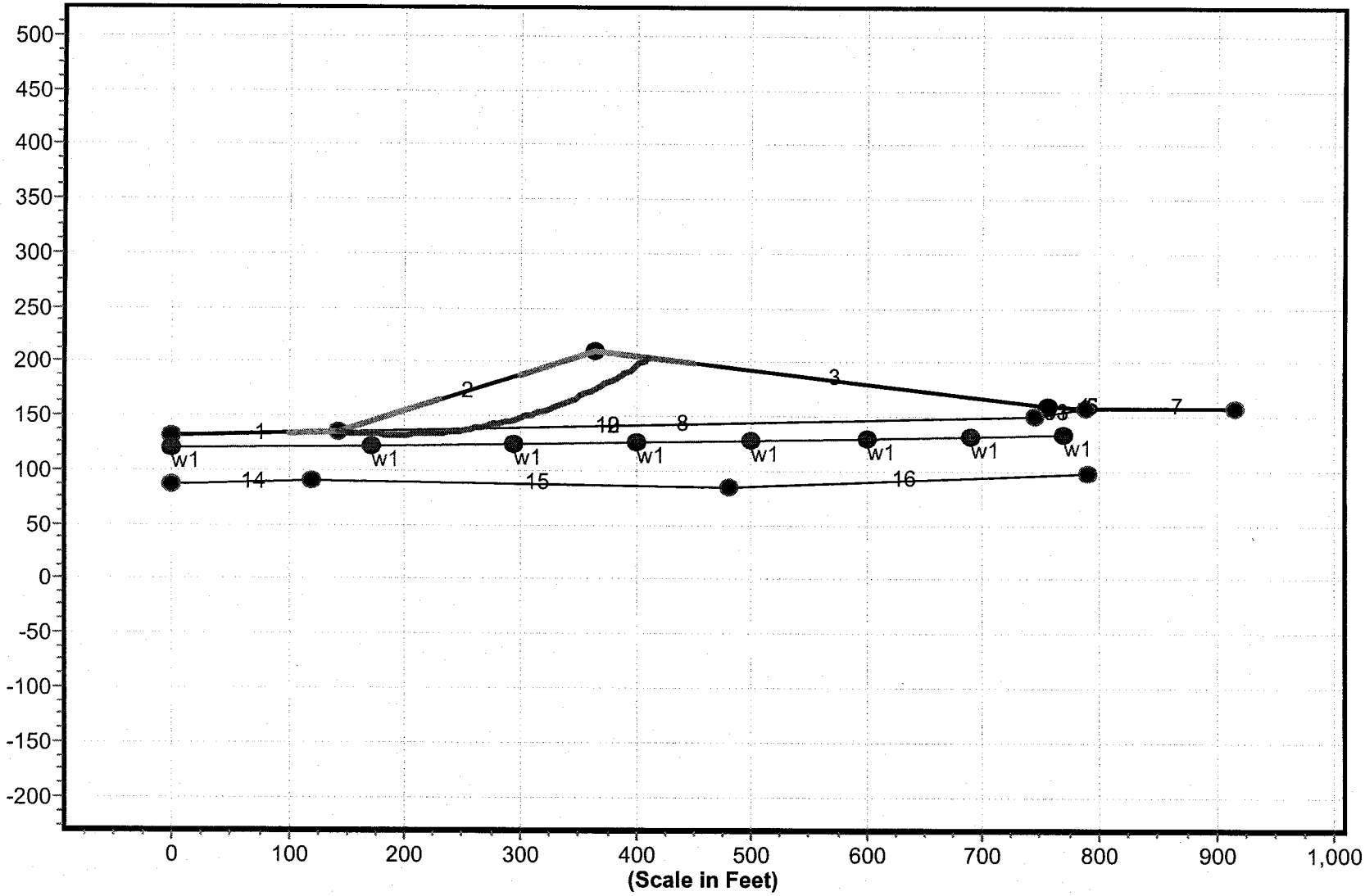
## Geometry and Boundary Conditions

**Problem: Figure 2 - MSS FGD LF - Station 22+00 - South Face - Clarifier Sludge Only - FS Min = 1.671**



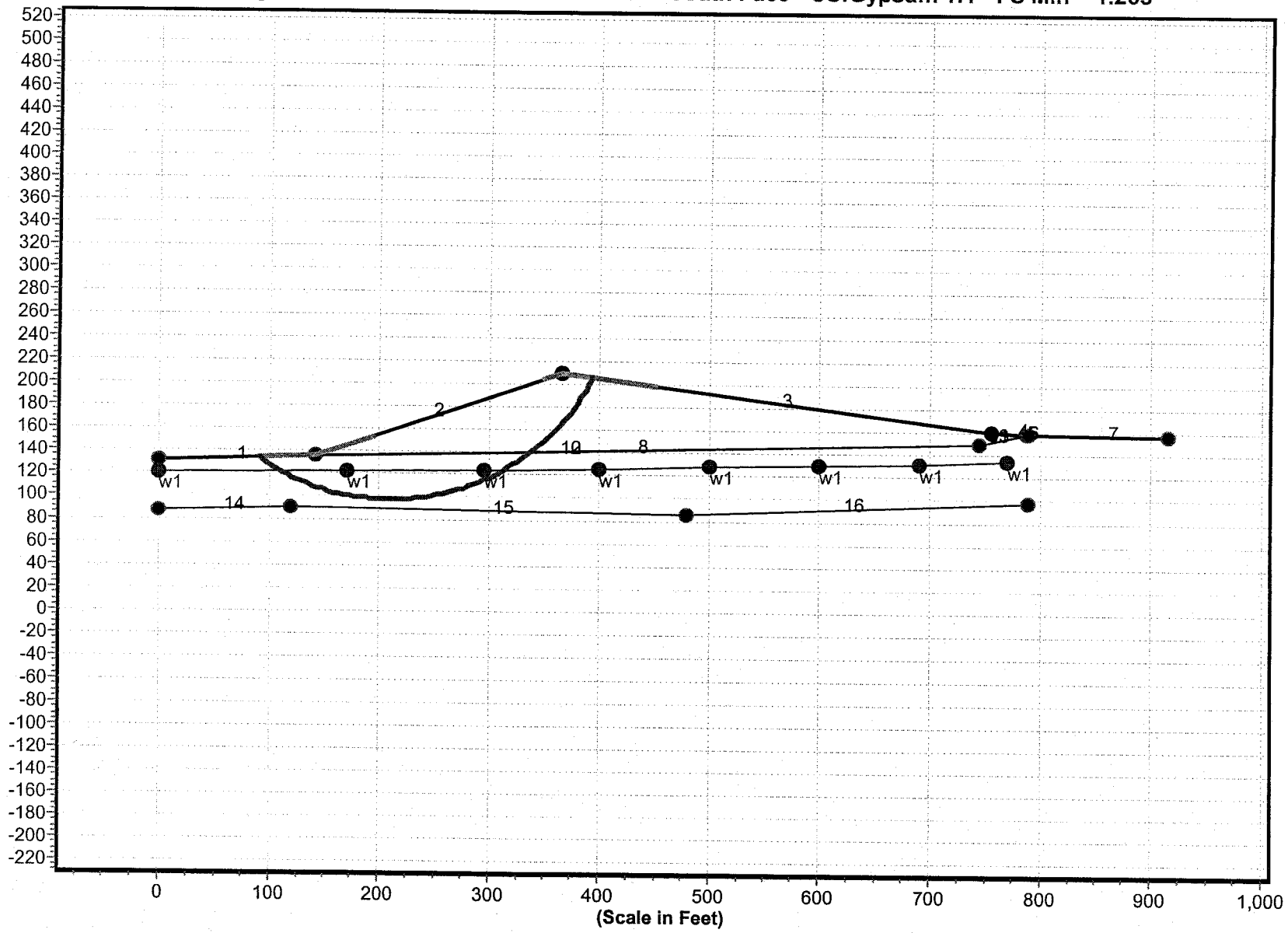
## Geometry and Boundary Conditions

**Problem: Figure 2a - MSS FGD LF - Station 22+00 - South Face - Clarifier Sludge Only - FS Min = 1.121**



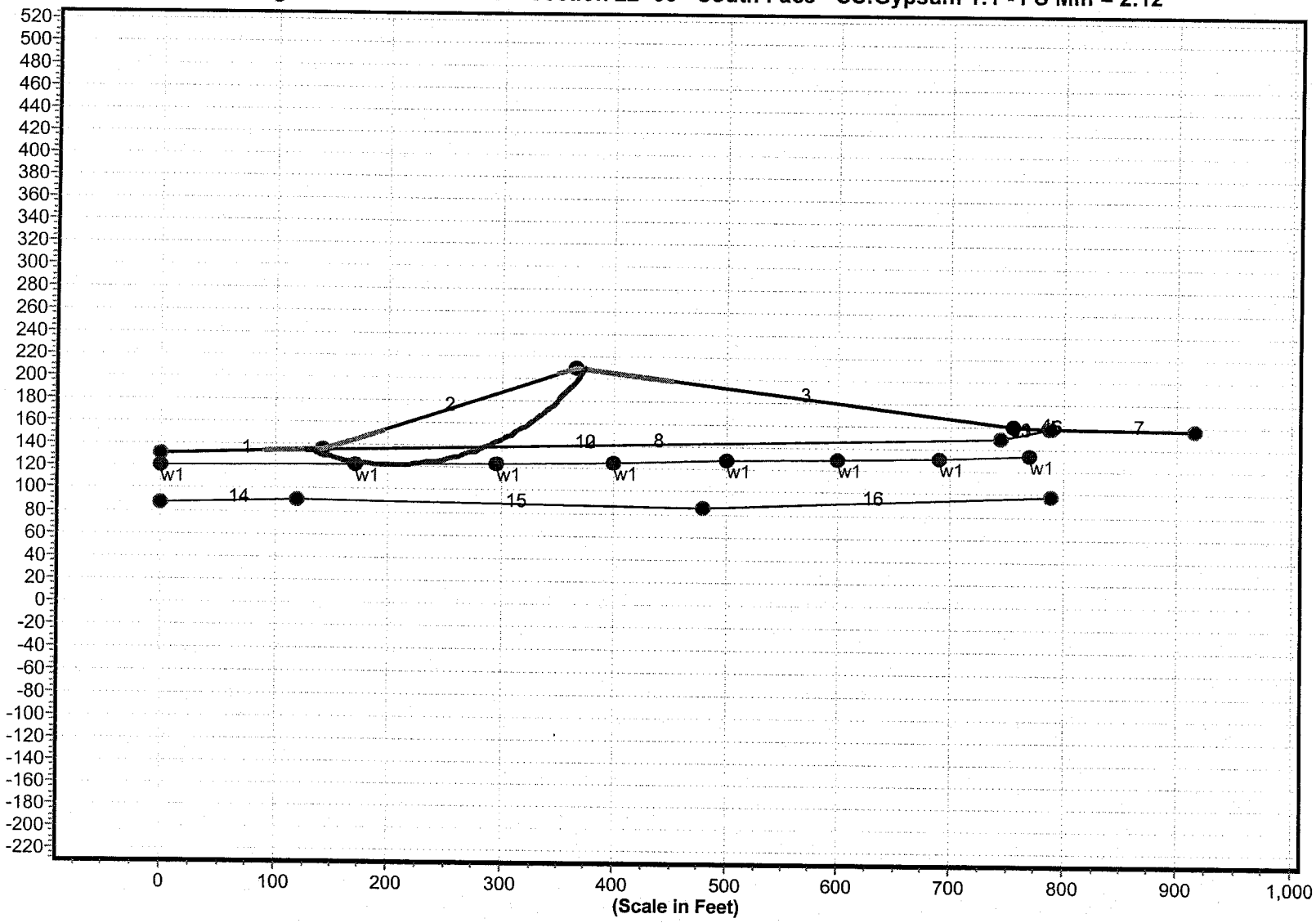
## Geometry and Boundary Conditions

**Problem: Figure 3 - MSS FGD LF - Section 22+00 - South Face - CS:Gypsum 1:1 - FS Min = 1.265**



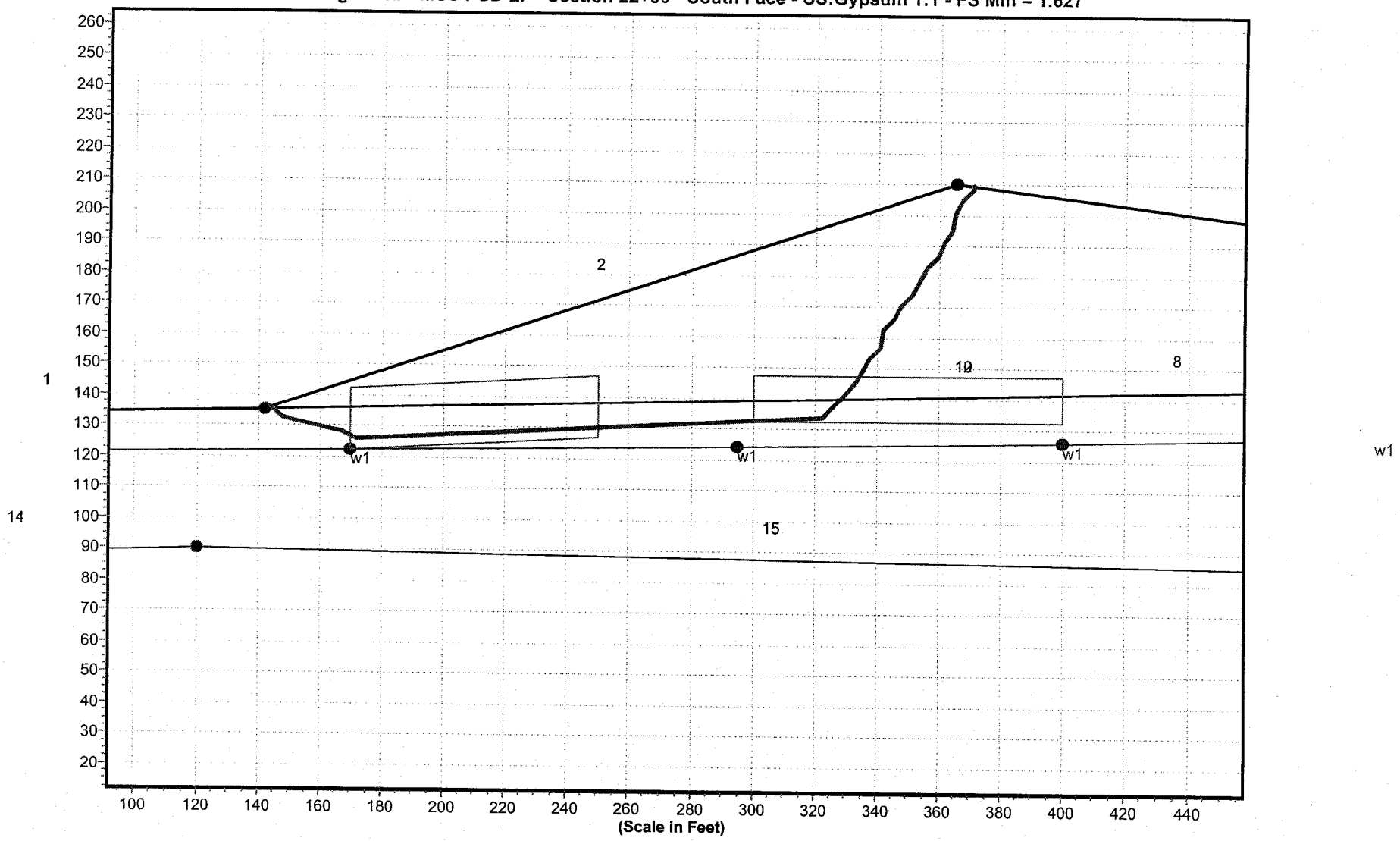
## Geometry and Boundary Conditions

**Problem: Figure 3a - MSS FGD LF - Section 22+00 - South Face - CS:Gypsum 1:1 - FS Min = 2.12**



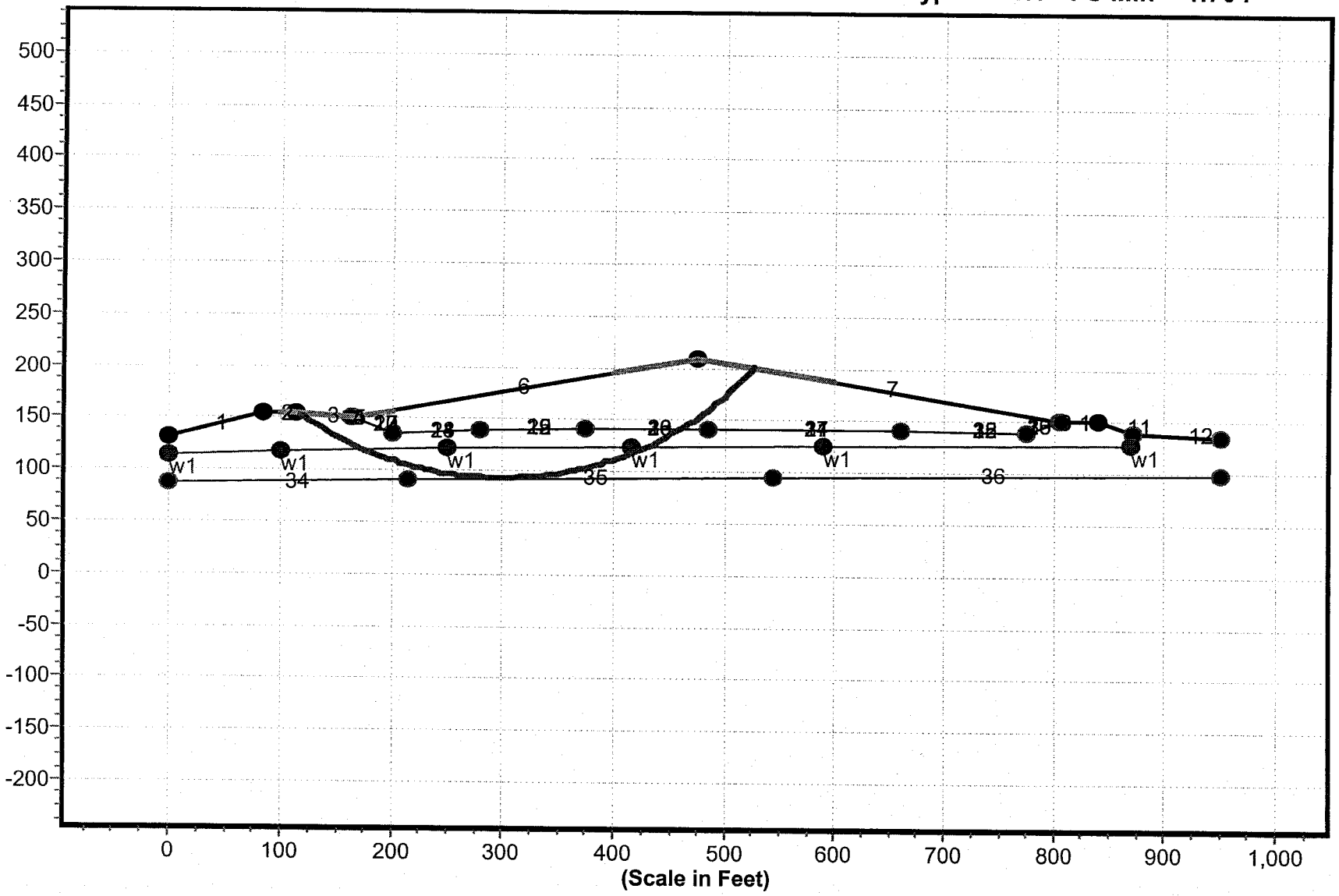
Geometry and Boundary Conditions

Problem: Figure 3b - MSS FGD LF - Section 22+00 - South Face - CS:Gypsum 1:1 - FS Min = 1.627



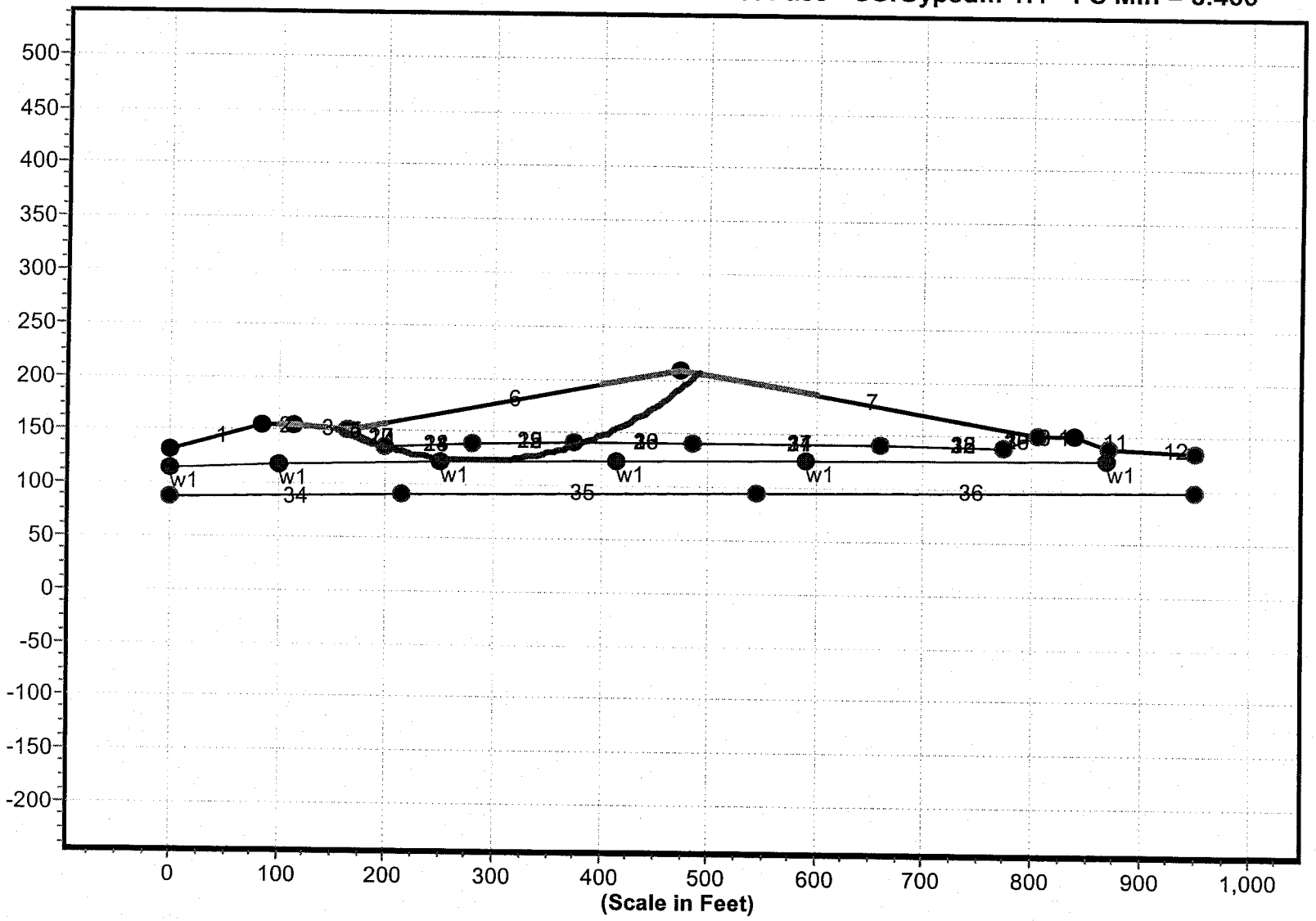
## Geometry and Boundary Conditions

**Problem: Figure 4 - MSS FGD LF - Section 15+00 - East Face - CS:Gypsum 1:1 - FS Min = 1.704**

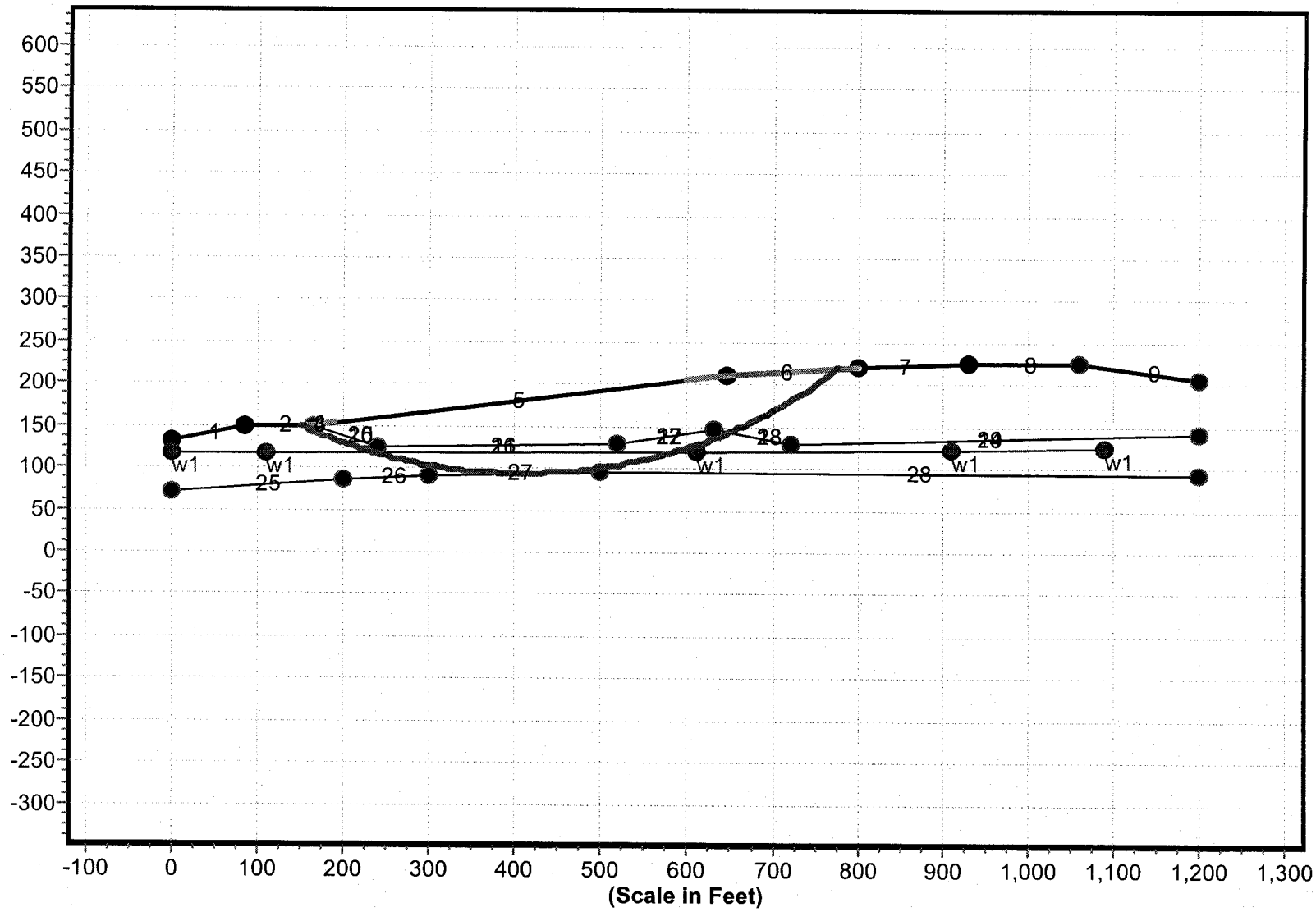


## Geometry and Boundary Conditions

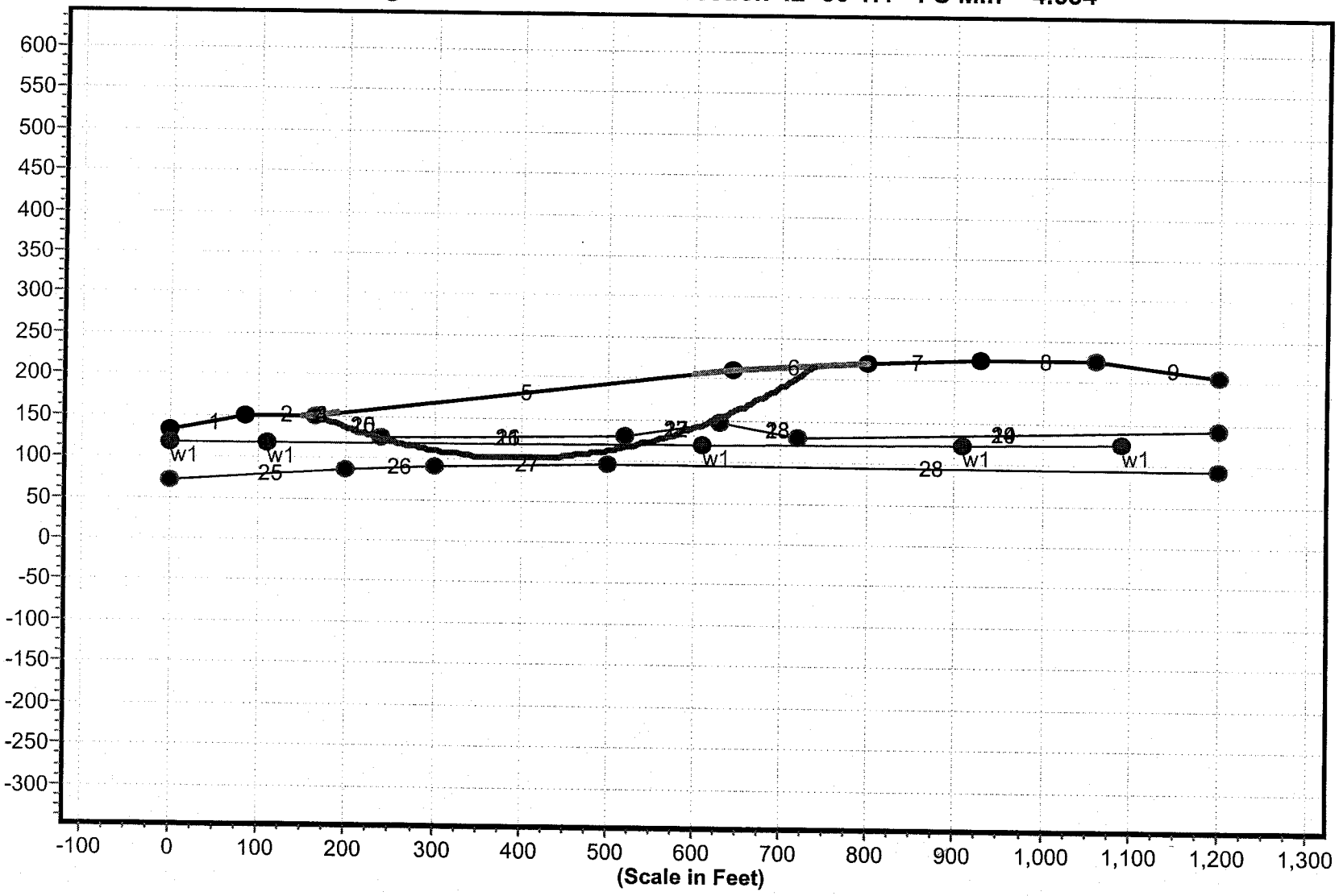
**Problem: Figure 4a - MSS FGD LF - Section 15+00 - East Face - CS:Gypsum 1:1 - FS Min = 3.466**



## Geometry and Boundary Conditions
## Problem: Figure 5 - MSS FGD LF - Section 42+50 1:1 - FS Min = 1.728



## Geometry and Boundary Conditions
## Problem: Figure 5a - MSS FGD LF - Section 42+50 1:1 - FS Min = 4.654



## SUMMARY OF LABORATORY TEST DATA
## TABLE 6

| Boring No. | Sample Depth (Ft) | Sample Type * | USCS Class. | Soil Description | Hydro-geologic Unit | Natural Moisture Content (%) | % Finer No. 200 | Atterberg Limits | | Specific Gravity | Unit Weight (pcf) | Hydraulic Conductivity k (cm/sec) | Proctor Data | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | | | | LL | PI | | | | Max. Dry Density (pcf) | Opt. Moisture Content (%) |
| B-5 | 8.5 - 10 | SS | SM | Clayey Silty Med/Fine Sand | Residuum | | 42.7 | NP | NP | | | | | |
| B-5 | 33.5 - 35 | SS | SM | Clayey Silty Med/Fine Sand | Saprolite | | 33 | 39 | NP | | | | | |
| B-5 | 0 - 10 | BAG | MH | Clayey Silt | Residuum | 30.8 | | 59 | 17 | 2.75 | 86.4 | See Lab Data Sheets | 90.9 | 27.4 |
| B-6 | 3.5 - 5.0 | SS | SM | Clayey Silty Med/Fine Sand | Residuum | | 44.9 | 48 | 3 | | | | | |
| B-6 | 58.5- 60 | SS | SM | Sl. Clayey Silty Med/Fine Sand | PWR | | 30.8 | NP | NP | | | | | |
| B-6 | 43 - 45 | UD | SM | Clayey Silty Med/Fine Sand | Saprolite | 39.4 | 31.9 | 37 | NP | 2.67 | 84.7 | See Lab Data Sheets | | |
| B-6 | 0 - 10 | BAG | MH | Clayey Silt | Residuum | 27.6 | | 52 | 11 | 2.66 | | | 93 | 25 |
| B-7 | 48.5 - 60 | SS | SM | Sl. Clayey Silty Med/Fine Sand | PWR | | 20.9 | NP | NP | 2.81 | | | | |
| B-7 | 30 - 32 | UD | SM | Sl. Clayey Silty Med/Fine Sand | Saprolite | 9.3 | 11.5 | NP | NP | 2.85 | 99.4 | See Lab Data Sheets | | |
| B-7 | 0 - 10 | BAG | MH | Clayey Silt | Residuum | 28.5 | | 62 | 14 | 2.77 | 80.7 | See Lab Data Sheets | 84.8 | 32.9 |

Note: Graphic Presentations of Results of Proctor, Grain Size, and other tests follow this summary in Appendix II

* SS = Split Spoon Sample (ASTM D-1586)
UD = Undisturbed Sample (ASTM D-1587)
BAG = Bulk Sample
NP = Non Plastic

JOB NAME: **Marshall Steam Station - FGD Scrubber Landfill**
LOCATION: **Terrell, North Carolina**
JOB NUMBER: **1264-02-578**
PAGE 2 of 3

## SUMMARY OF LABORATORY TEST DATA
## TABLE 6

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|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | | | | LL | PI | | | | Max. Dry Density (pcf) | Opt. Moisture Content (%) |
| MS-1 | 8.5 - 10 | SS | SM | Sl. Clayey Silty Med/Fine Sand | Saprolite | | 33.6 | 38 | NP | | | | | |
| MS-1 | 20 - 22 | UD | ML | Clayey Med/Fine Sandy Silt | Saprolite | 37.9 | 64.3 | 45 | 12 | 2.66 | 87.4 | See Lab Data Sheets | | |
| MS-3 | 15 - 17 | UD | SM | Sl. Clayey Silty Coarse/Fine Sand | Saprolite | 31.8 | 22.4 | NP | NP | 2.74 | 90.3 | See Lab Data Sheets | | |
| MS-4 | 3.5 - 5.0 | SS | MH | Sl. Sandy Clayey Silt | Residuum | | 82.5 | 83 | 33 | 2.72 | | | | |
| MS-5 | 32 - 34 | UD | SM | Sl. Clayey Silty Coarse/Fine Sand | Saprolite | 22.7 | 21.3 | NP | NP | 2.72 | 98.1 | See Lab Data Sheets | | |
| MS-7 | 8.5 - 10 | SS | CH | Med/Fine Sandy Silty Clay | Residuum | | 82.9 | 61 | 36 | | | | | |
| MS-7 | 18.5 - 20 | SS | SM | Sl. Clayey Silty Med/Fine Sand | Saprolite | | 36.8 | 41 | NP | | | | | |
| MS-8 | 23.5 - 25 | SS | SM | Sl. Clayey Silty Coarse/Fine Sand | Saprolite | | 31 | NP | NP | 2.67 | | | | |
| | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | |

Note: Graphic Presentations of Results of Proctor, Grain Size, and other tests follow this summary in Appendix II

* SS = Split Spoon Sample (ASTM D-1586)
UD = Undisturbed Sample (ASTM D-1587)
BAG = Bulk Sample
NP = Non Plastic

JOB NAME: **Marshall Steam Station - FGD Scrubber Landfill**
LOCATION: **Terrell, North Carolina**
JOB NUMBER: **1264-02-578**
PAGE 3 of 3

# Marshall FGD Residue Landfill

# Slope Stability Evaluation

## Copies of References

$$N'_{60} = 0.68 \times 20 \times 1.00 \times 1.0 \times 0.80 \times 1.05 = \underline{11}$$

$$N'_{70} = \frac{60}{70} \times 11 = \underline{9}$$

////

**Example 3-4**

Same as Example 3-2 but $E_r = 55$; $p'_o = 100$ kPa; 205 mm hollow stem auger; hole depth = 6 m

$$C_N = \left(\frac{95.76}{100}\right)^{1/2} = 0.98$$

$$\eta_1 = \frac{55}{70} = 0.79 \qquad \eta_2 = 0.95 \quad 6 < 10\,\text{m}$$

$$\eta_3 = 1.0 \quad \text{no liner} \qquad \eta_4 = 1 \quad \text{using hollow stem auger}$$

$$N'_{70} = 0.98 \times 20 \times 0.79 \times 0.95 \times 1.0 \times 1.0 = \underline{15}$$

$$N'_{60} = \frac{70}{60} \times 15 = \underline{17}$$

////

## 3-8 SPT CORRELATIONS

The SPT has been used in correlations for unit weight, relative density $D_r$, angle of internal friction $\phi$, and undrained compressive strength $q_u$. For reasons in the preceding section these will be of questionable value—particularly since many were suggested prior to extensive study of the large number of variables affecting the $N$ values. The data shown in Table 3-4 for $D_r$ and $\phi$ relate roughly to $N'_{70}$ and for borehole depths on the order of 4 to 6 m as for spread foundations.

TABLE 3-4 **Empirical values for $\phi$, $D_r$, and unit weight of granular soils based on the SPT at about 6 m depth and normally consolidated**

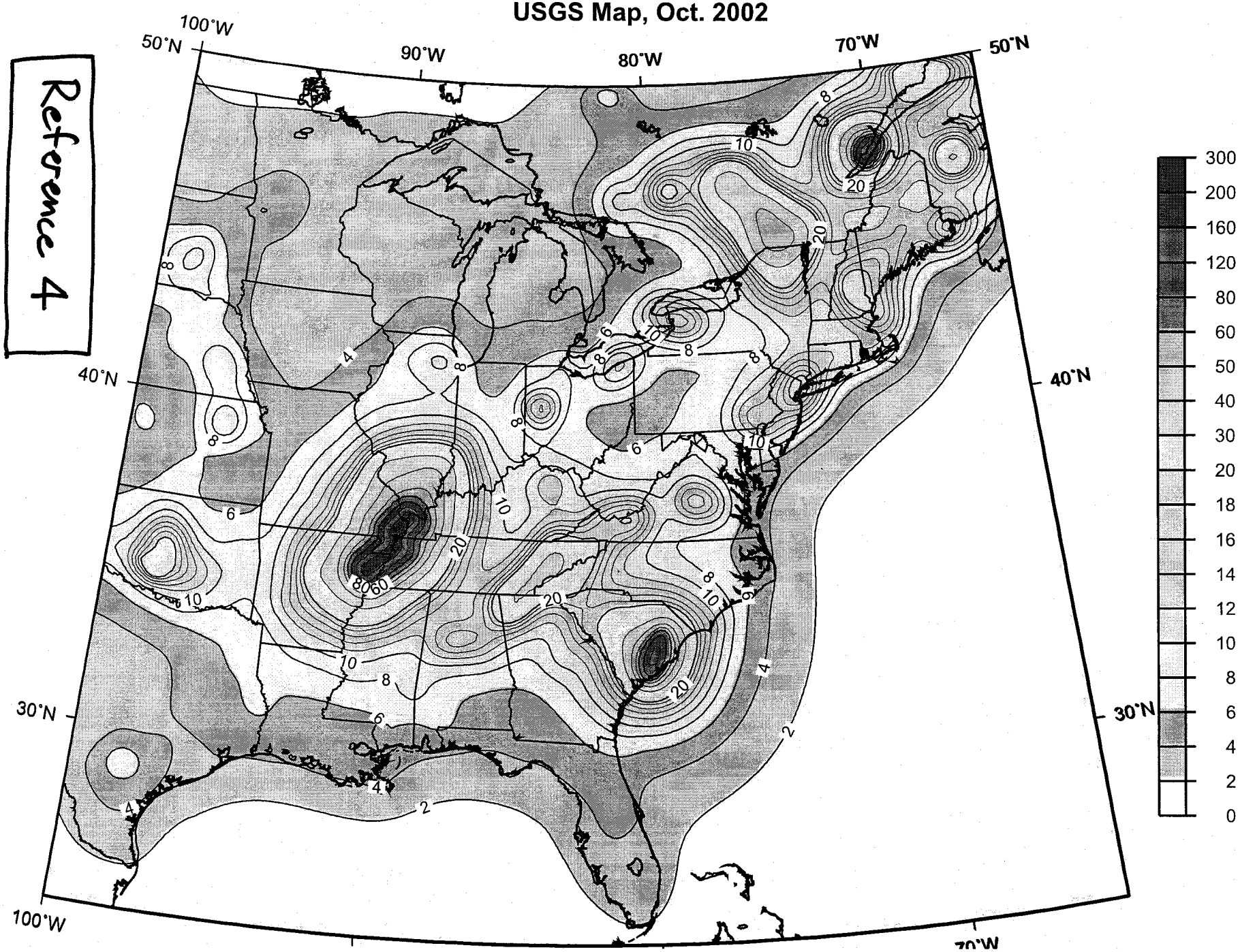
| Description | Very loose | Loose | Medium | Dense | Very dense |
|---|---|---|---|---|---|
| Relative density $D_r$ | 0 | 0.15 | 0.35 | 0.65 | 0.85 |
| SPT $N'_{70}$: fine | 1–2 | 3–6 | 7–15 | 16–30 | ? |
| medium | 2–3 | 4–7 | 8–20 | 21–40 | >40 |
| coarse | 3–6 | 5–9 | 10–25 | 26–45 | >45 |
| $\phi$: fine | 26–28 | 28–30 | 30–34 | 33–38 | |
| medium | 27–28 | 30–32 | 32–36 | 36–42 | <50 |
| coarse | 28–30 | 30–34 | 33–40 | 40–50 | |
| $\gamma_{wet}$, pcf | 70–100† | 90–115 | 110–130 | 110–140 | 130–150 |
| (kN/m$^3$) | (11–16) | (14–18) | (17–20) | (17–22) | (20–23) |

† Excavated soil or material dumped from a truck will weigh 11 to 14 kN/m$^3$ and must be quite dense to weigh much over 21 kN/m$^3$. No existing soil has a $D_r = 0.00$ nor a value of 1.00—common ranges are from 0.3 to 0.7.

Reference 3

Reference 4

# Peak Acceleration (%g) with 2% Probability of Exceedance in 50 Years
## USGS Map, Oct. 2002



# Marshall FGD Residue Landfill

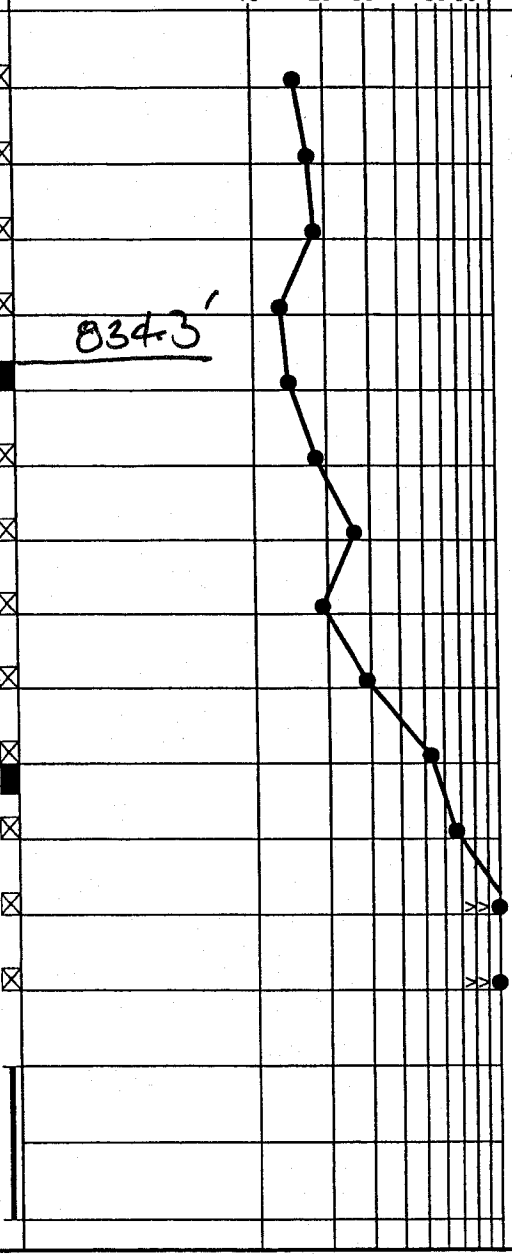
## Slope Stability Evaluation

## Copies of Boring Logs

| PROJECT: | Marshall Steam Station<br>Terrell, North Carolina<br>1264-02-578 | BORING LOG B-2 |
|---|---|---|
| DATE DRILLED: 12/16/02 | ELEVATION: 857.9 | NOTES: Bag samples collected every 10 ft to 60 ft bls.<br>~~Auger Refusal Encountered~~ at 70 ft bls.<br>Cut to ~834.3' |
| ILLING METHOD: 4¼" H.S.A. | BORING DEPTH: 80.0 | |
| GED BY: Julie Petersen | WATER LEVEL: 45.64 ft bls at 24 hrs. | |
| DRILLER: Jay Little, NC Cert No. 2717 | DRILL RIG: Diedrich D-50 | |

| DEPTH (feet) | MATERIAL DESCRIPTION |
|---|---|
| | Topsoil/Rootmat |
| 5 | RESIDUUM: Red Slightly Micaceous Fine Sandy Silty **CLAY** |
| 10 | RESIDUUM: Tannish Red Micaceous Slightly Sandy **SILT** |
| 15 | SAPROLITE: Tannish Red Very Micaceous Slightly Sandy **SILT** |
| 20 | SAPROLITE: Grey and White Very Micaceous Fine Sandy **SILT** |
| 25 | SAPROLITE: Reddish Tan and White Micaceous Silty Fine **SAND** with Manganese Staining |
| 30 | SAPROLITE: Greyish Tan and White Micaceous Fine Sandy **SILT** |
| 35 | SAPROLITE: Reddish Tan and White Fine Sandy **SILT** with Manganese Staining |
| 40 | SAPROLITE: Greyish Tan and White Very Micaceous Fine Sandy **SILT** with Manganese Staining |
| 45 | SAPROLITE: Greyish Tan and White Very Micaceous Silty Medium to Fine **SAND (SM)** |
| 50 | |
| 55 | SAPROLITE: Reddish Tan Micaceous Fine Sandy **SILT** with Manganese Staining |
| 60 | PARTIALLY WEATHERED ROCK: When Sampled Becomes Greyish Tan and White Very Micaceous Silty Fine **SAND** |
| 65 | PARTIALLY WEATHERED ROCK: When Sampled Becomes Reddish Tan Micaceous Fine to Medium Sandy **SILT** with Manganese Staining<br>**Auger Refusal at 70 ft bls** |
| 70 | BEDROCK: Sampled as Pink Granite |
| 75 | BEDROCK: Sampled as Fine Grained Granite-Gneiss (Very Weathered) |
| 80 | |

| ELEVATION (feet-MSL) | SAMPLE NO/TYPE | N VALUE |
|---|---|---|
| 852.9 | 1 | 15 |
| 847.9 | 2 | 17 |
| 842.9 | 3 | 18 |
| 837.9 | 4 | 13 |
| 832.9 | REC 100% 5 | 14 |
| 827.9 | 6 | 18 |
| 822.9 | 7 | 26 |
| 817.9 | 8 | 19 |
| 812.9 | 9 | 29 |
| 807.9 | 10 | 53 |
| 802.9 | REC 40% 11 | 67 |
| 797.9 | 12 | 50/5 |
| 792.9 | 13 | 50/2 |
| 787.9 | | |
| 782.9 | REC 86% RQD 58% | |
| 777.9 | | |




1. *BORING AND SAMPLING IS IN ACCORDANCE WITH ASTM D-1586.*
2. *PENETRATION (N-VALUE) IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I.D. SAMPLER 1 FT.*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BORING LOG WELLLOGS.GPJ S&ME.GDT 3/13/03

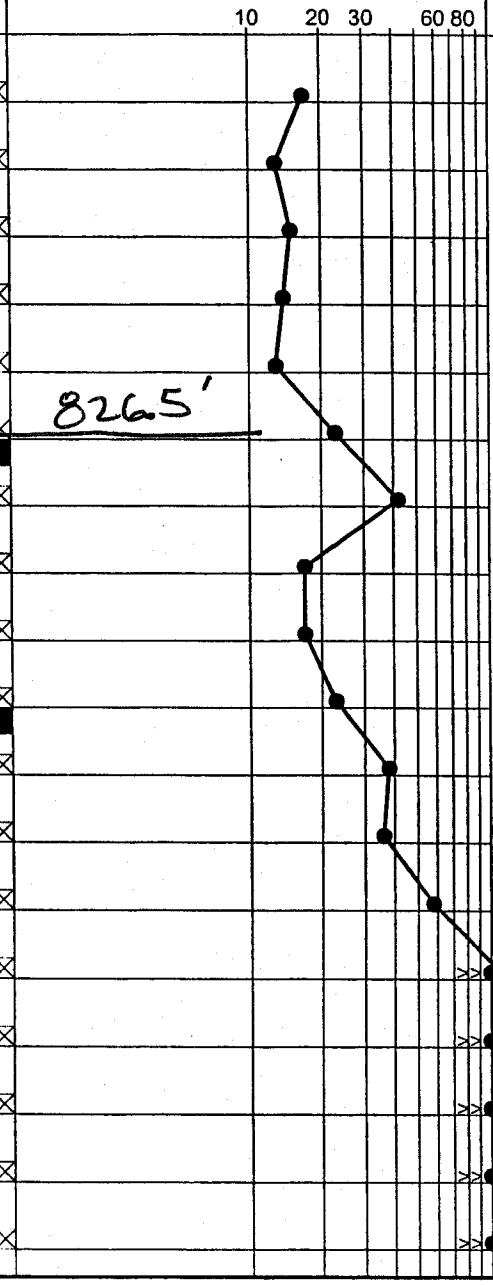
| PROJECT: Marshall Steam Station<br>Terrell, North Carolina<br>1264-02-578 | BORING LOG B-3 |
|---|---|

| | | |
|---|---|---|
| DATE DRILLED: 12/11/02 | ELEVATION: 856.2 | NOTES: Bag samples collected every 10 ft to 70 ft bls. Cut to ~ 826.5' |
| LLING METHOD: 4¼" H.S.A. | BORING DEPTH: 90.0 | |
| GED BY: Julie Petersen | WATER LEVEL: 49 ft bls at 24 hrs. | |
| DRILLER: Jay Little, NC Cert No. 2717 | DRILL RIG: Diedrich D-50 | |

| DEPTH (feet) | MATERIAL DESCRIPTION | ELEVATION (feet-MSL) | SAMPLE NO/TYPE | N VALUE |
|---|---|---|---|---|
| | Rootmat/Topsoil | | | |
| | RESIDUUM: Red Slightly Sandy Clayey **SILT** | | | |
| 5 | RESIDUUM: Tannish Red Silty Fine **SAND** | 851.2 | 1 | 17 |
| 10 | SAPROLITE: Tannish Brown Fine Sandy **SILT** | 846.2 | 2 | 13 |
| 15 | SAPROLITE: Reddish Tan Medium to Fine Sandy Clayey **SILT** with Some Manganese Staining **(MH)** | 841.2 | 3 | 15 |
| 20 | | 836.2 | 4 | 14 |
| 25 | | 831.2 | 5 | 13 |
| 30 | SAPROLITE: Pinkish White Slightly Micaceous Silty Medium to Fine **SAND** with Quartz Fragments and Some Manganese Staining | 826.2 | 6 | 23 |
| 35 | | 821.2 | REC 100% 7 | 42 |
| 40 | SAPROLITE: Olive Green and Reddish Tan Micaceous Fine Sandy **SILT** with Manganese Staining | 816.2 | 8 | 17 |
| 45 | SAPROLITE: Olive Green and Reddish Tan Micaceous Silty Fine **SAND** with Quartz Fragments and Manganese Staining | 811.2 | 9 | 17 |
| 50 | SAPROLITE: Tannish Brown Micaceous Slightly Clayey Silty Medium to Fine **SAND** with Manganese Staining. White Medium to Fine Silty Sand Seam with Quartz Fragments at 65 ft bls. **(SM)** | 806.2 | 10 | 23 |
| 55 | | 801.2 | REC 100% 11 | 38 |
| 60 | | 796.2 | 12 | 36 |
| 65 | | 791.2 | 13 | 58 |
| 70 | PARTIALLY WEATHERED ROCK: When Sampled Becomes Tannish Brown Micaceous Slightly Clayey Silty Medium to Fine **SAND** with Manganese Staining and Rock Fragments **(SM)** | 786.2 | 14 | 50/4 |
| 75 | | 781.2 | 15 | 50/4 |
| 80 | PARTIALLY WEATHERED ROCK: When Sampled Becomes Olive Green and Brown Very Micaceous Fine Sandy **SILT** | 776.2 | 16 | 50/5 |
| 85 | PARTIALLY WEATHERED ROCK: When Sampled Becomes Brown and White Very Micaceous Silty Fine **SAND** | 771.2 | 17 | 50/4 |
| 90 | | 766.2 | 18 | 50/3 |

WATER LEVEL: ▼ ▽

STANDARD PENETRATION TEST DATA (blows/ft)




BORING LOG WELLLOGS.GPJ S&ME.GDT 3/5/03

1. *BORING AND SAMPLING IS IN ACCORDANCE WITH ASTM D-1586.*
2. *PENETRATION (N-VALUE) IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I.D. SAMPLER 1 FT.*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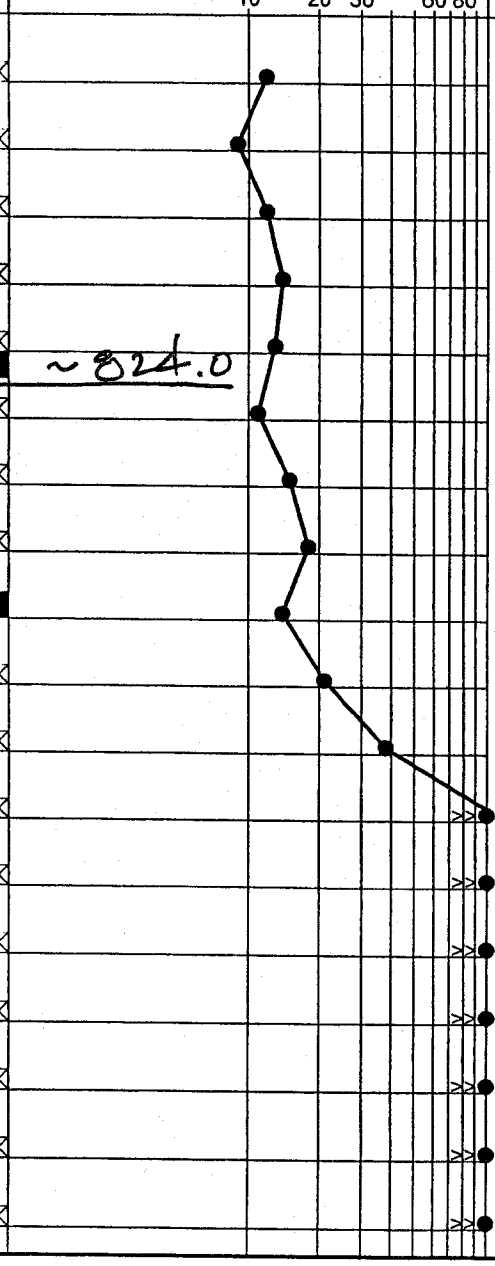
| PROJECT: Marshall Steam Station<br>Terrell, North Carolina<br>1264-02-578 | BORING LOG B-6 |
|---|---|

| | | |
|---|---|---|
| DATE DRILLED: 12/9/02 | ELEVATION: 851.6 | NOTES: Bag Samples collected every 10 ft to 70 ft bls. |
| DRILLING METHOD: 4¼" H.S.A. | BORING DEPTH: 90.0 | ~824.0' |
| LOGGED BY: Julie Petersen | WATER LEVEL: 42.62 ft bls at 24 hrs. | |
| DRILLER: Jay Little, NC Cert No. 2717 | DRILL RIG: Diedrich D-50 | |

| DEPTH (feet) | MATERIAL DESCRIPTION | ELEVATION (feet-MSL) | SAMPLE NO/TYPE | N VALUE |
|---|---|---|---|---|
| | Grass/Rootmat/Topsoil | | | |
| 5 | RESIDUUM: Reddish Brown Micaceous Clayey Silty Medium to Fine **SAND (SM)** | 846.6 | 1 | 12 |
| 10 | SAPROLITE: Pinkish Brown to Pinkish Grey Slightly Micaceous Fine Sandy **SILT** | 841.6 | 2 | 9 |
| 15 | | 836.6 | 3 | 12 |
| 20 | | 831.6 | 4 | 14 |
| 25 | | 826.6 | 5 ~824.0 | 13 |
| 30 | SAPROLITE: Pinkish Tan and White Silty Fine **SAND** | 821.6 | REC 100% 6 | 11 |
| 35 | SAPROLITE: Pinkish Tan and Grey Slightly Micaceous Fine Sandy **SILT** | 816.6 | 7 | 15 |
| 40 | SAPROLITE: Tannish Grey and White Micaceous Silty Fine **SAND** | 811.6 | 8 | 18 |
| 45 | SAPROLITE: Pinkish Brown Micaceous Slightly Clayey Silty Medium to Fine **SAND** with Quartz Fragments **(SM)** | 806.6 | REC 100% 9 | 14 |
| 50 | | 801.6 | 10 | 21 |
| 55 | SAPROLITE: White, Pink and Grey Micaceous Silty Medium to Fine **SAND** | 796.6 | 11 | 38 |
| 60 | PARTIALLY WEATHERED ROCK: When Sampled Becomes White, Pink and Grey Micaceous Slightly Clayey Silty Medium to Fine **SAND** with Rock Fragments **(SM)** | 791.6 | 12 | 50/5 |
| 65 | PARTIALLY WEATHERED ROCK: When Sampled Becomes Pinkish Brown Silty Medium to Fine **SAND** | 786.6 | 13 | 50/5 |
| 70 | | 781.6 | 14 | 50/5 |
| 75 | | 776.6 | 15 | 50/5 |
| 80 | PARTIALLY WEATHERED ROCK: When Sampled Becomes White, Pink and Grey Micaceous Silty Medium to Fine **SAND** with Rock Fragments | 771.6 | 16 | 50/5 |
| 85 | | 766.6 | 17 | 50/3 |
| 90 | | 761.6 | 18 | 50/4 |

WATER LEVEL

STANDARD PENETRATION TEST DATA (blows/ft)




BORING LOG WELLLOGS.GPJ S&ME.GDT 3/5/03

1. *BORING AND SAMPLING IS IN ACCORDANCE WITH ASTM D-1586.*
2. *PENETRATION (N-VALUE) IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I.D. SAMPLER 1 FT.*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| PROJECT: | Marshall Steam Station<br>Terrell, North Carolina<br>1264-02-578 | BORING LOG MS-7 |
|---|---|---|
| DATE DRILLED: 1/3/03 | ELEVATION: 833.1 | NOTES: Bag Samples Collected at 20 and 30 ft bls.<br>Auger Refusal Encountered at 36.8 ft bls.<br>~827.5' |
| DRILLING METHOD: 4¼" H.S.A. | BORING DEPTH: 46.8 | |
| LOGGED BY: Julie Petersen | WATER LEVEL: 19.5 ft bls at 24 hrs. | |
| DRILLER: Jay Little, NC Cert No. 2717 | DRILL RIG: Mobile B-57 | |

| DEPTH (feet) | GRAPHIC LOG | MATERIAL DESCRIPTION | WATER LEVEL | ELEVATION (feet-MSL) | SAMPLE NO/TYPE | STANDARD PENETRATION TEST DATA (blows/ft) | N VALUE |
|---|---|---|---|---|---|---|---|
| | | RESIDUUM: Reddish Tan Slightly Sandy Clayey **SILT** with Grey Clay Nodules | | | | | |
| 5 | | | | 828.1 | 1 | ~827.5' | 46 |
| | | RESIDUUM: Grey and Reddish Tan Mottled Medium to Fine Sandy Silty **CLAY** (high plasticity) **(CH)** | | | | | |
| 10 | | | | 823.1 | 2 | | 16 |
| | | SAPROLITE: Yellowish Red to Brown Very Micaceous Slightly Clayey Silty Medium to Fine **SAND** with Manganese Staining **(SM)** | | 818.1 | 3 | | 9 |
| 15 | | | | | REC 100% | | |
| 20 | | | ▼ | 813.1 | 4 | | 12 |
| 25 | | | | 808.1 | 5 | | 9 |
| 30 | | | | 803.1 | 6 | | 12 |
| 35 | | PARTIALLY WEATHERED ROCK: When Sampled Becomes Brown Very Micaceous Fine Sandy **SILT** | | 798.1 | 7 | >> | 50/1 |
| | | **Auger Refusal at 36.8 ft bls** | | | | | |
| | | BEDROCK: Sampled as Pink Granite (Very Weathered) | | | | | |
| 40 | | BEDROCK: Sampled as Gneiss (Grades from Coarse to Fine Grain) | | 793.1 | REC 93% RQD 54% | | |
| 45 | | | | 788.1 | | | |

STANDARD PENETRATION TEST DATA (blows/ft): 10, 20, 30, 60, 80

BORING LOG WELLLOGS.GPJ S&ME.GDT 3/13/03

1. *BORING AND SAMPLING IS IN ACCORDANCE WITH ASTM D-1586.*
2. *PENETRATION (N-VALUE) IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I.D. SAMPLER 1 FT.*

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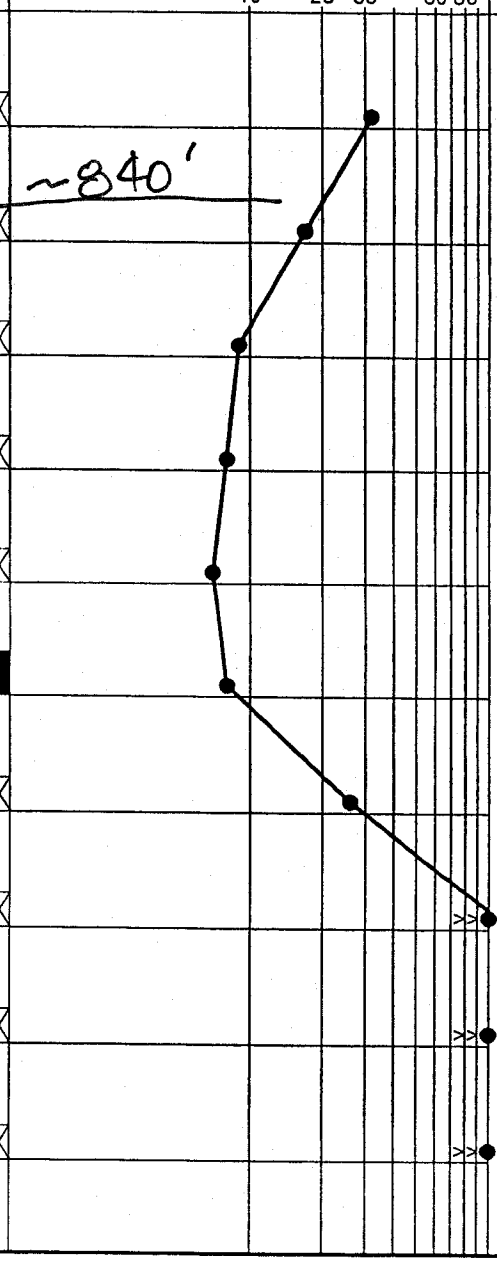
| PROJECT: Marshall Steam Station<br>Terrell, North Carolina<br>1264-02-578 | BORING LOG MS-8 |
|---|---|

| | | |
|---|---|---|
| DATE DRILLED: 1/14/03 | ELEVATION: 848.3 | NOTES: Bag Samples Collected at 30 and 40 ft bls.<br>~~Auger Refusal Encountered~~ at 52 ft bls.<br>Cut to ~840' |
| DRILLING METHOD: 4¼" H.S.A. | BORING DEPTH: 52.0 | |
| LOGGED BY: Julie Petersen | WATER LEVEL: 27.2 ft bls at 24 hrs. | |
| DRILLER: Brian Wilson, NC Cert No. 2718 | DRILL RIG: Mobile B-57 | |

| DEPTH (feet) | MATERIAL DESCRIPTION | ELEVATION (feet-MSL) | SAMPLE NO/TYPE | N VALUE |
|---|---|---|---|---|
| | RESIDUUM: Yellowish Red Fine Sandy Silty **CLAY** | | | |
| 5 | | 843.3 | 1 | 32 |
| | RESIDUUM: Yellow and Red Mottled Micaceous Fine Sandy Silty **CLAY** | | ~840' | |
| 10 | | 838.3 | 2 | 17 |
| | SAPROLITE: Greyish Tan and Pinkish White Very Micaceous Slightly Clayey Silty Coarse to Fine **SAND (SM)** | | | |
| 15 | | 833.3 | 3 | 9 |
| 20 | | 828.3 | 4 | 8 |
| 25 | | 823.3 | 5 | 7 |
| | SAPROLITE: Greyish Tan and Pinkish White Very Micaceous Silty Medium to Fine **SAND** with Some Manganese Staining | | | |
| 30 | | 818.3 | REC 100% 6 | 8 |
| 35 | | 813.3 | 7 | 26 |
| 40 | PARTIALLY WEATHERED ROCK: When Sampled Becomes Greyish Tan and Pinkish White Very Micaceous Silty Medium to Fine **SAND** with Some Manganese Staining | 808.3 | 8 | >> 50/5 |
| | PARTIALLY WEATHERED ROCK: When Sampled Becomes Red and White Silty Medium to Fine **SAND** | | | |
| 45 | **Auger Refusal at 52 ft bls** | 803.3 | 9 | >> 50/2 |
| 50 | | 798.3 | 10 | >> 50/2 |

STANDARD PENETRATION TEST DATA (blows/ft)




WATER LEVEL

BORING LOG WELLLOGS.GPJ S&ME.GDT 3/5/03

1. *BORING AND SAMPLING IS IN ACCORDANCE WITH ASTM D-1586.*
2. *PENETRATION (N-VALUE) IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I.D. SAMPLER 1 FT.*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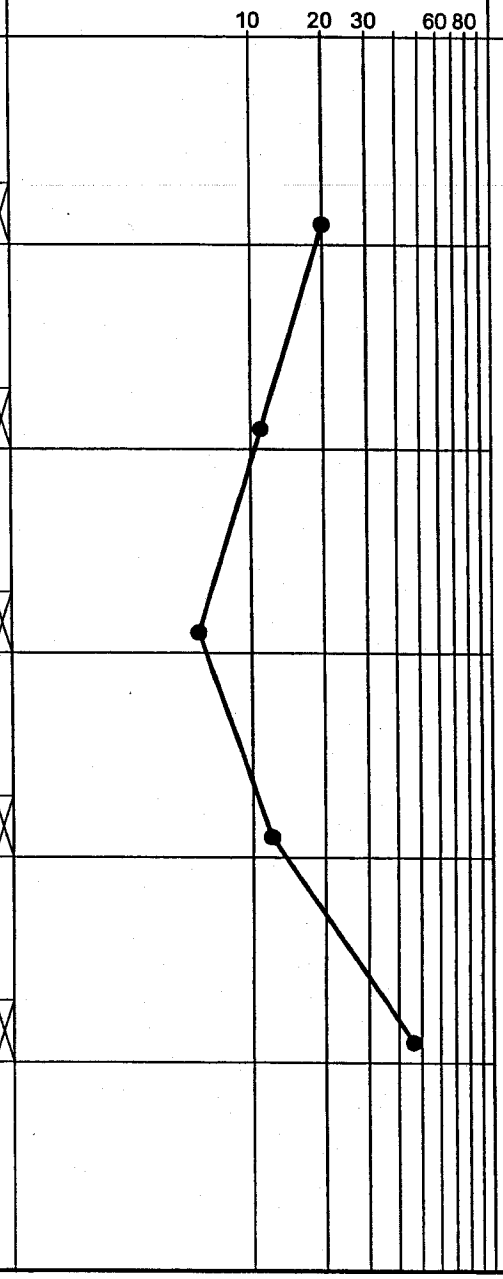
| PROJECT: | Marshall Steam Station<br>Terrell, North Carolina<br>1264-02-578 | BORING LOG OW-1 |
|---|---|---|
| DATE DRILLED: 2/12/03 | ELEVATION: | NOTES: No Cut |
| ILLING METHOD: 4¼" H.S.A. | BORING DEPTH: 26.0 | |
| GGED BY: Julie Petersen | WATER LEVEL: 20.01 ft bls at 24 hrs. | |
| DRILLER: Brian Wilson, NC Cert No. 2718 | DRILL RIG: Mobile B-57 | |

| DEPTH (feet) | GRAPHIC LOG | MATERIAL DESCRIPTION | WATER LEVEL | ELEVATION (feet-MSL) | SAMPLE NO/TYPE | STANDARD PENETRATION TEST DATA (blows/ft) | N VALUE |
|---|---|---|---|---|---|---|---|
| | | RESIDUUM: Red Slightly Sandy Silty **CLAY** | | | | | |
| | | RESIDUUM: Red and Grey Fine Sandy Silty **CLAY** (high plasiticy) | | | 1 | | 20 |
| 5 | | | | | | | |
| | | | | | 2 | | 11 |
| 10 | | | | | | | |
| | | SAPROLITE: Grey and White Very Micaceous Silty Coarse to Fine **SAND** | | | 3 | | 6 |
| 15 | | | | | | | |
| | | | ▼ | | 4 | | 12 |
| 20 | | | | | | | |
| | | SAPROLITE: Grey, Tan and White Very Micaceous Silty Coarse to Fine **SAND** | | | 5 | | 46 |
| 25 | | | | | | | |




BORING LOG WELLLOGS.GPJ S&ME.GDT 3/4/03

1. *BORING AND SAMPLING IS IN ACCORDANCE WITH ASTM D-1586.*
2. *PENETRATION (N-VALUE) IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I.D. SAMPLER 1 FT.*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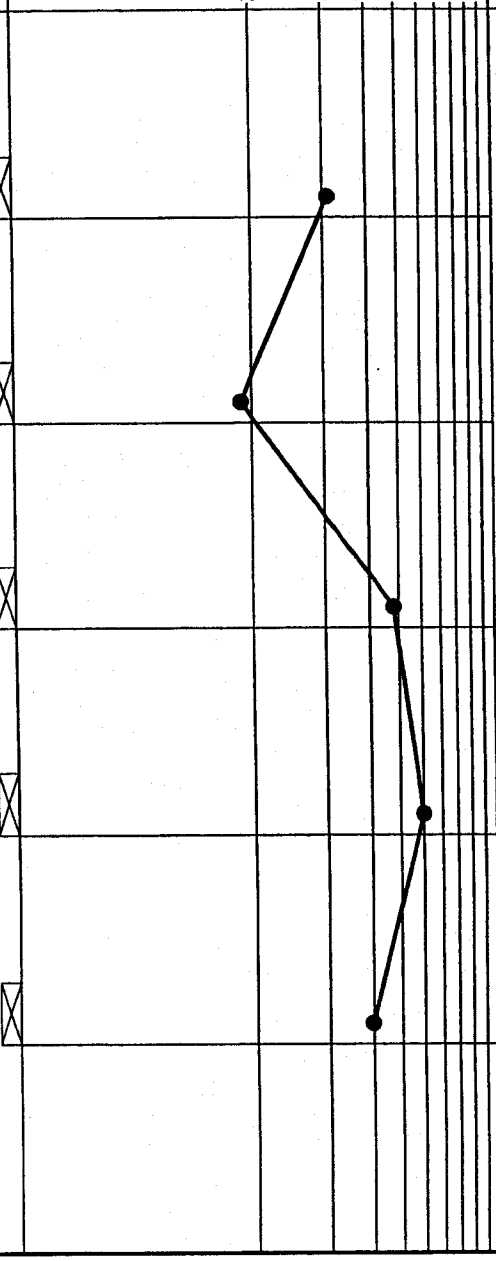
| PROJECT: | Marshall Steam Station<br>Terrell, North Carolina<br>1264-02-578 | BORING LOG OW-2 |
|---|---|---|
| DATE DRILLED: 2/12/03 | ELEVATION: | NOTES: Ground @ 833.38'<br>~ 830.6' |
| DRILLING METHOD: 4¼" H.S.A. | BORING DEPTH: 26.0 | |
| LOGGED BY: Julie Petersen | WATER LEVEL: 17.56 ft bls at 24 hrs. | |
| DRILLER: Brian Wilson, NC Cert No. 2718 | DRILL RIG: Mobile B-57 | |

| DEPTH (feet) | GRAPHIC LOG | MATERIAL DESCRIPTION | WATER LEVEL | ELEVATION (feet-MSL) | SAMPLE NO/TYPE | STANDARD PENETRATION TEST DATA (blows/ft) | N VALUE |
|---|---|---|---|---|---|---|---|
| | | RESIDUUM: Red Slightly Sandy Silty **CLAY** | | | | | |
| 5 | | RESIDUUM: Tan to Reddish Tan Slightly Sandy Silty **CLAY** with Grey High Plasticity Clay Mottles | | | 1 | | 21 |
| 10 | | | | | 2 | | 9 |
| | | RESIDUUM: Grey Fine Sandy **CLAY** (high plasticity) | | | | | |
| 15 | | SAPROLITE: Red and Grey to White and Grey Very Micaceous Silty Fine **SAND** | | | 3 | | 38 |
| 20 | | | | | 4 | | 50 |
| | | SAPROLITE: Grey and White Very Micaceous Coarse to Fine **SAND** | | | | | |
| 25 | | | | | 5 | | 30 |




BORING LOG WELLLOGS.GPJ S&ME.GDT 3/13/03

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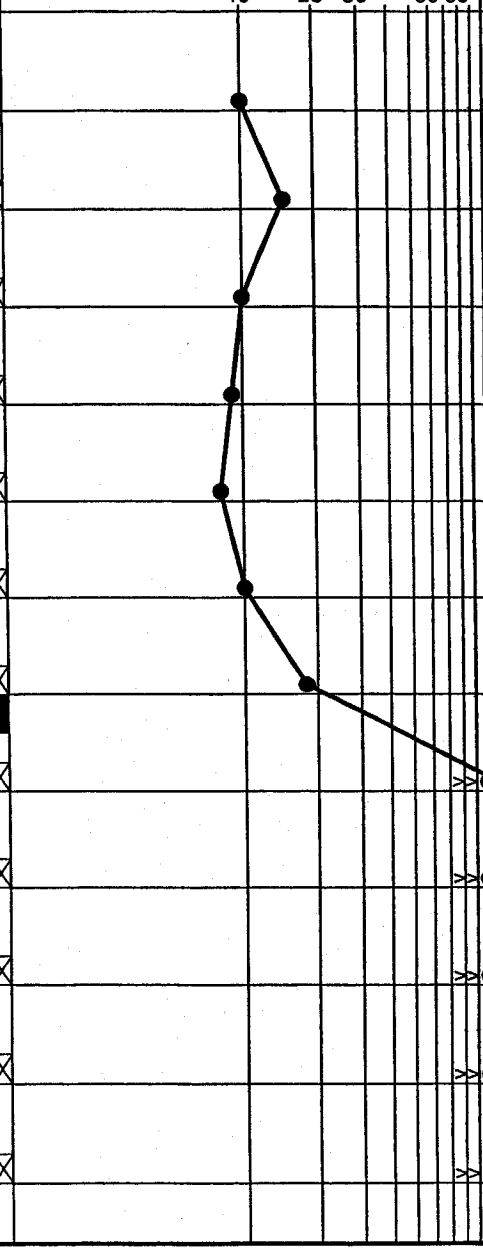
9751 Southern Pine Blvd.
Charlotte, NC 28273

| PROJECT: | Marshall Steam Station<br>Terrell, North Carolina<br>1264-02-578 | BORING LOG MS-6 |
|---|---|---|
| DATE DRILLED: 12/17/02 | ELEVATION: 839.2 | NOTES: Bag Samples Collected Every 10 ft from 30 to 60 ft bls.<br>Auger Refusal Encountered at 61 ft bls. |
| ILLING METHOD: 4¼" H.S.A. | BORING DEPTH: 61.0 | |
| GED BY: Julie Petersen | WATER LEVEL: 26.89 ft bls at 24 hrs. | Outside Footprint |
| DRILLER: Jay Little, NC Cert No. 2717 | DRILL RIG: Diedrich D-50 | |

| DEPTH (feet) | MATERIAL DESCRIPTION | ELEVATION (feet-MSL) | SAMPLE NO/TYPE | N VALUE |
|---|---|---|---|---|
| | RESIDUUM: Greyish Tan Miaceous Fine Sandy **SILT** | | | |
| | RESIDUUM: Tannish Brown Miaceous Fine Sandy Silty **CLAY** | | | |
| 5 | | 834.2 | 1 | 10 |
| | | 829.2 | 2 | 15 |
| 10 | RESIDUUM: Reddish Brown and Tan Micaceous Clayey Fine Sandy **SILT** | | | |
| | SAPROLITE: Tannish Grey Miaceous Fine Sandy **SILT** | | | |
| 15 | SAPROLITE: Brownish Tan Micaceous Fine Sandy **SILT** | 824.2 | 3 | 10 |
| | SAPROLITE: Greyish Tan Silty Fine **SAND** | 819.2 | 4 | 9 |
| 20 | | | | |
| 25 | | 814.2 | 5 | 8 |
| | SAPROLITE: Grey and White Micaceous Silty Medium to Fine **SAND** | 809.2 | 6 | 10 |
| 30 | | | | |
| 35 | | 804.2 | 7 | 18 |
| | | | REC 50% | |
| 40 | PARTIALLY WEATHERED ROCK: When Sampled Becomes Grey and White Miaceous Silty Medium to Fine **SAND** | 799.2 | 8 | 50/3 |
| 45 | | 794.2 | 9 | 50/2 |
| 50 | PARTIALLY WEATHERED ROCK: When Sampled Becomes Red, Tan and White Silty Medium to Fine **SAND** | 789.2 | 10 | 50/5 |
| 55 | PARTIALLY WEATHERED ROCK: When Sampled Becomes Red, Grey and White Slightly Micaceous Silty Medium to Fine **SAND**<br>**Auger Refusal at 61 ft bls** | 784.2 | 11 | 50/4 |
| 60 | | 779.2 | 12 | 50/3 |

STANDARD PENETRATION TEST DATA (blows/ft)




BORING LOG WELLLOGS.GPJ S&ME.GDT 3/4/03

1. *BORING AND SAMPLING IS IN ACCORDANCE WITH ASTM D-1586.*
2. *PENETRATION (N-VALUE) IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I.D. SAMPLER 1 FT.*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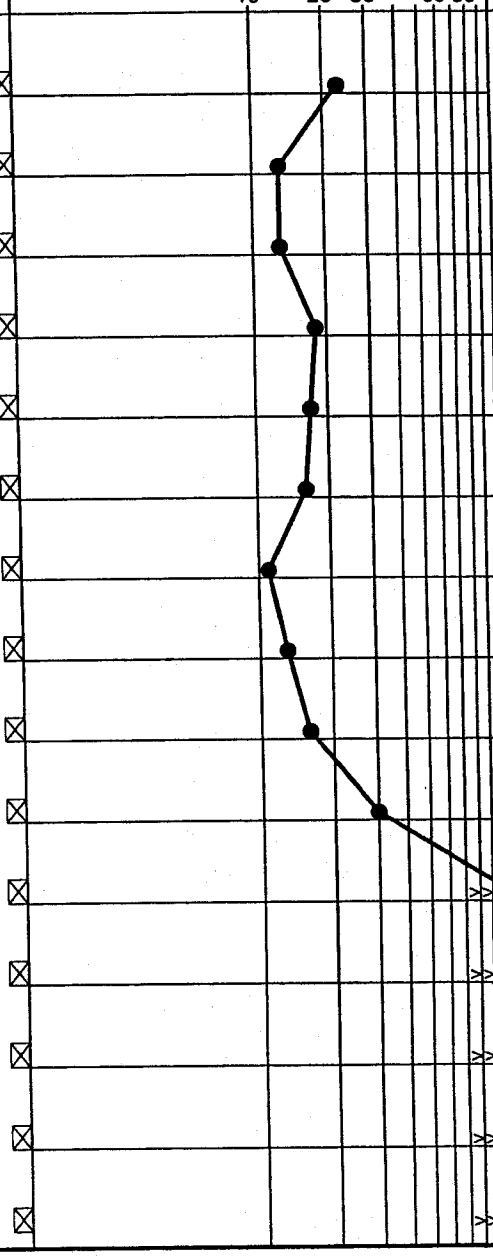
| PROJECT: | Marshall Steam Station<br>Terrell, North Carolina<br>1264-02-578 | **BORING LOG MS-9** |
|---|---|---|
| DATE DRILLED: 1/16/03 | ELEVATION: 865.4 | NOTES: Bag Sample Collected at 60 ft bls.<br>Auger Refusal Encountered at 74 ft bls. |
| LLING METHOD: 4¼" H.S.A. | BORING DEPTH: 74.0 | Outside Footprint |
| GED BY: Julie Petersen | WATER LEVEL: 44.5 ft bls at 24 hrs. | |
| DRILLER: Brian Wilson, NC Cert No. 2718 | DRILL RIG: Mobile B-57 | |

| DEPTH (feet) | MATERIAL DESCRIPTION | ELEVATION (feet-MSL) | SAMPLE NO/TYPE | N VALUE |
|---|---|---|---|---|
| | RESIDUUM: Red Slightly Sandy Silty **CLAY** | | | |
| 5 | RESIDUUM: Yellowish Red Miaceous Fine Sandy **SILT** | 860.4 | 1 | 23 |
| 10 | SAPROLITE: Greyish Brown and Pinkish White Micaceous Silty Very Fine **SAND** with Some Manganese Staining | 855.4 | 2 | 13 |
| 15 | | 850.4 | 3 | 13 |
| 20 | | 845.4 | 4 | 18 |
| 25 | | 840.4 | 5 | 17 |
| 30 | | 835.4 | 6 | 16 |
| 35 | | 830.4 | 7 | 11 |
| 40 | | 825.4 | 8 | 13 |
| 45 | | 820.4 | 9 | 16 |
| 50 | | 815.4 | 10 | 30 |
| 55 | PARTIALLY WEATHERED ROCK: When Sampled Becomes Greyish Brown and Pinkish White Micaceous Silty Fine **SAND** | 810.4 | 11 | 50/4 |
| 60 | PARTIALLY WEATHERED ROCK: When Sampled Becomes Grey, Brown and White Micaceous Silty Medium to Fine **SAND** with Rock Fragments | 805.4 | 12 | 50/3 |
| 65 | **Auger Refusal at 74 ft bls** | 800.4 | 13 | 50/5 |
| 70 | | 795.4 | 14 | 50/1 |
| | | | 15 | 50/2 |

STANDARD PENETRATION TEST DATA (blows/ft)




BORING LOG WELLLOGS.GPJ S&ME.GDT 3/4/03

*1. BORING AND SAMPLING IS IN ACCORDANCE WITH ASTM D-1586.*
*2. PENETRATION (N-VALUE) IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I.D. SAMPLER 1 FT.*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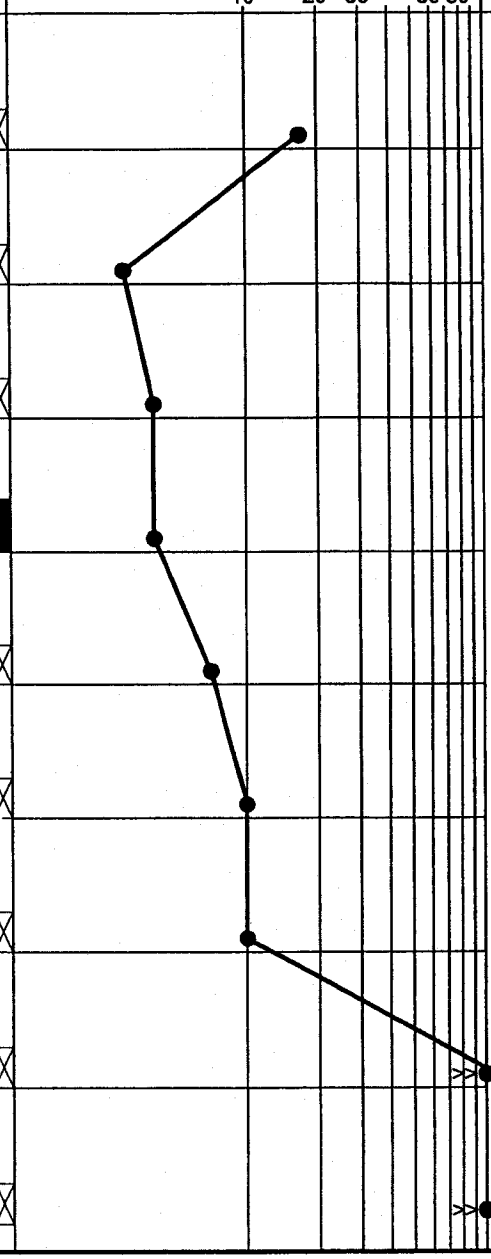
| PROJECT: Marshall Steam Station<br>Terrell, North Carolina<br>1264-02-578 | BORING LOG MS-10 |
|---|---|

| | | |
|---|---|---|
| DATE DRILLED: 7/7/03 | ELEVATION: 848.3 | NOTES: Auger Refusal Encountered at 44 ft bls. |
| DRILLING METHOD: 4¼" H.S.A. | BORING DEPTH: 44.0 | Outside Footprint |
| LOGGED BY: Julie Petersen | WATER LEVEL: 12 ft bls at 24 hrs. | |
| DRILLER: Jay Little, NC Cert No. 2717 | DRILL RIG: Mobile B-57 | |

| DEPTH (feet) | MATERIAL DESCRIPTION | ELEVATION (feet-MSL) | SAMPLE NO/TYPE | N VALUE |
|---|---|---|---|---|
| 5 | RESIDUUM: Tan Silty Fine Sandy **CLAY** | 843.3 | 1 | 17 |
| 10 | RESIDUUM: Grey Very Micaceous Slightly Sandy Silty **CLAY** | 838.3 | 2 | 3 |
| 15 | SAPROLITE: Tan, Brown and Grey Very Micaceous Fine Sandy **SILT** with Manganese Staining | 833.3 | 3 | 4 |
| 20 | | 828.3 | 4 | 4 |
| 25 | | 823.3 | 5 | 7 |
| 30 | SAPROLITE: Reddish Tan, Brown and Grey Very Micaceous Silty Fine **SAND** with Manganese Staining | 818.3 | 6 | 10 |
| 35 | | 813.3 | 7 | 10 |
| 40 | PARTIALLY WEATHERED ROCK: When Sampled Becomes Grey and White Very Micaceous Silty Medium to Fine **SAND**<br>**Auger Refusal at 44 ft bls** | 808.3 | 8 | 50/6 |
| | | | 9 | 50/1 |

WATER LEVEL

STANDARD PENETRATION TEST DATA (blows/ft)




BORING LOG WELLLOGS.GPJ S&ME.GDT 8/1/03

1. *BORING AND SAMPLING IS IN ACCORDANCE WITH ASTM D-1586.*
2. *PENETRATION (N-VALUE) IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I.D. SAMPLER 1 FT.*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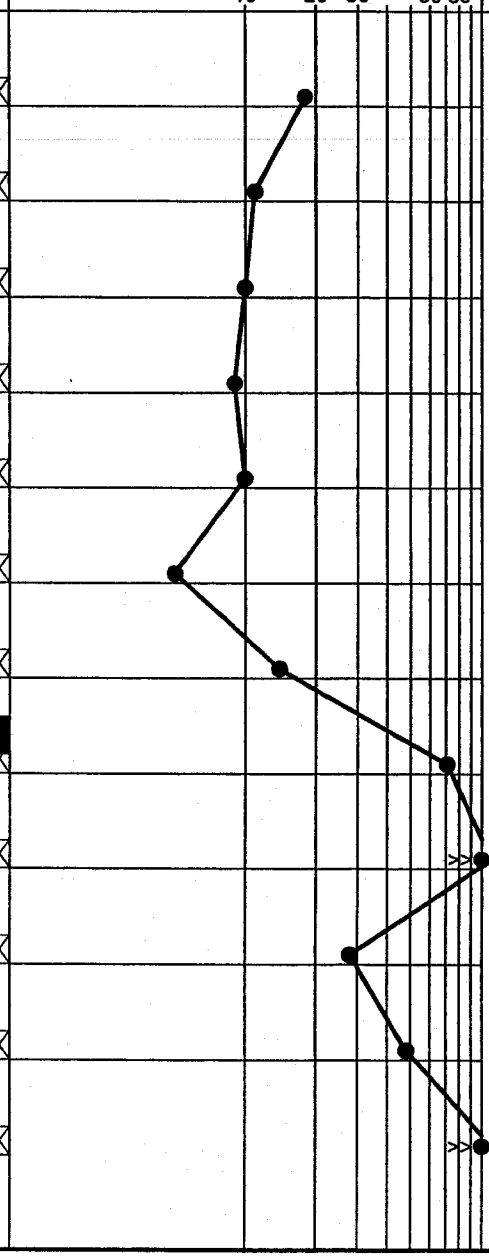
| PROJECT: Marshall Steam Station<br>Terrell, North Carolina<br>1264-02-578 | **BORING LOG MS-11** |
|---|---|

| | | |
|---|---|---|
| DATE DRILLED: **7/8/03** | ELEVATION: **856.9** | NOTES: **Auger Refusal Encountered at 59 ft bls.** |
| DRILLING METHOD: **4¼" H.S.A.** | BORING DEPTH: **59.0** | Outside Footprint |
| LOGGED BY: **Julie Petersen** | WATER LEVEL: **33 ft bls at 24 hrs.** | |
| DRILLER: **Jay Little, NC Cert No. 2717** | DRILL RIG: **Mobile B-57** | |

| DEPTH (feet) | MATERIAL DESCRIPTION | WATER LEVEL | ELEVATION (feet-MSL) | SAMPLE NO/TYPE | N VALUE |
|---|---|---|---|---|---|
| | Rootmat/Topsoil | | | | |
| | RESIDUUM: Red Micaceous Fine Sandy Silty **CLAY** | | | | |
| 5 | | | 851.9 | 1 | 18 |
| | RESIDUUM: Tannish Red Micaceous Fine Sandy **SILT** | | | | |
| 10 | | | 846.9 | 2 | 11 |
| | SAPROLITE: Reddish Tan Micaceous Slightly Clayey Silty Fine **SAND** | | | | |
| 15 | | | 841.9 | 3 | 10 |
| 20 | | | 836.9 | 4 | 9 |
| | SAPROLITE: Reddish Tan Slightly Clayey Silty Coarse to Fine **SAND** | | | | |
| 25 | | | 831.9 | 5 | 10 |
| | SAPROLITE: Reddish Tan Slightly Clayey Fine Sandy **SILT** | | | | |
| 30 | | | 826.9 | 6 | 5 |
| | SAPROLITE: Brownish Tan Silty Medium to Fine **SAND** with Traces of Manganese Staining | ▽ | | | |
| 35 | | | 821.9 | 7 | 14 |
| | SAPROLITE: Tan and White Silty Coarse to Fine **SAND** with Quartz Fragments | | | | |
| 40 | | | 816.9 | 8 | 71 |
| | PARTIALLY WEATHERED ROCK: When Sampled Becomes Tan and White Silty Coarse to Fine **SAND** with Quartz Fragments | | | | |
| 45 | | | 811.9 | 9 | 50/5 |
| | SAPROLITE: Tannish Red, Black and White Stratified Slightly Clayey Fine Sandy **SILT** | | | | |
| 50 | | | 806.9 | 10 | 28 |
| 55 | | | 801.9 | 11 | 48 |
| | PARTIALLY WEATHERED ROCK: When Sampled Becomes Tannish Red, Black and White Stratified Slightly Clayey Medium to Fine Sandy **SILT** | | | 12 | 50/3 |
| | **Auger Refusal at 59 ft bls** | | | | |




BORING LOG WELLLOGS.GPJ S&ME.GDT 8/1/03

1. *BORING AND SAMPLING IS IN ACCORDANCE WITH ASTM D-1586.*
2. *PENETRATION (N-VALUE) IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I.D. SAMPLER 1 FT.*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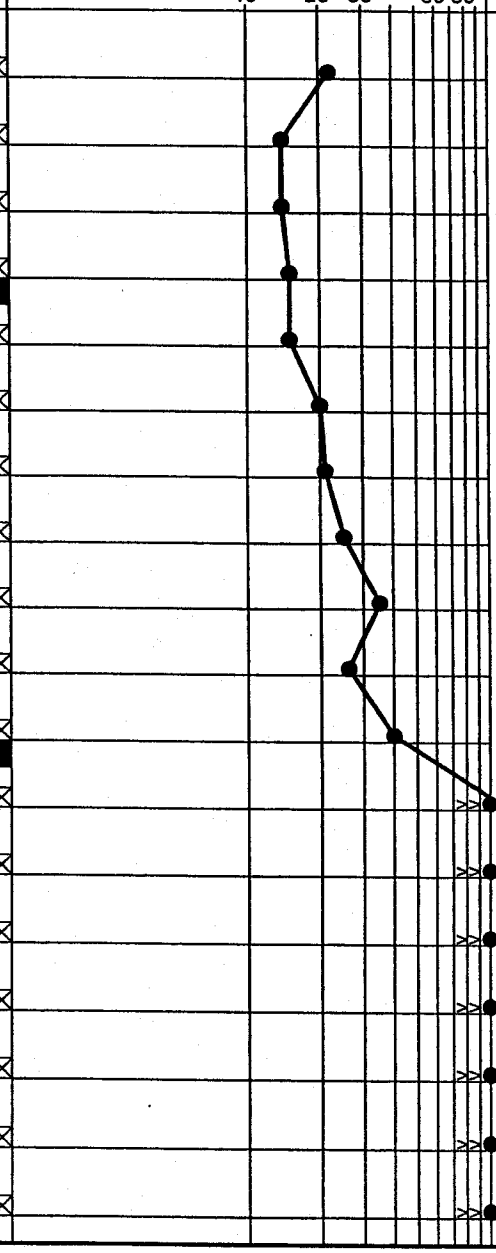
| PROJECT: | Marshall Steam Station<br>Terrell, North Carolina<br>1264-02-578 | BORING LOG B-4 |
|---|---|---|
| DATE DRILLED: 12/3/02 | ELEVATION: 858.4 | NOTES: Bag samples collected every 10 ft to 70 ft bls. |
| DRILLING METHOD: 4¼" H.S.A. | BORING DEPTH: 90.0 | Outside Footprint |
| LOGGED BY: Julie Petersen | WATER LEVEL: 52 ft bls at 24 hrs. | |
| DRILLER: Jay Little, NC Cert No. 2717 | DRILL RIG: Diedrich D-50 | |

| DEPTH (feet) | MATERIAL DESCRIPTION | ELEVATION (feet-MSL) | SAMPLE NO/TYPE | N VALUE |
|---|---|---|---|---|
| 5 | RESIDUUM: Yellowish Red Micaceous Medium to Fine Sandy Clayey **SILT (MH)** | 853.4 | 1 | 22 |
| 10 | SAPROLITE: Yellowish Red Micaceous Slightly Sandy **SILT** with Manganese Staining | 848.4 | 2 | 14 |
| 15 | SAPROLITE: Reddish Grey to Greyish Brown Very Micaceous Slightly Sandy **SILT** | 843.4 | 3 | 14 |
| 20 | | 838.4 | 4 | 15 |
| 25 | | 833.4 | REC 100% 5 | 15 |
| 30 | | 828.4 | 6 | 20 |
| 35 | SAPROLITE: Greyish Red Micaceous Fine Sandy **SILT** with Manganese Staining | 823.4 | 7 | 21 |
| 40 | | 818.4 | 8 | 25 |
| 45 | SAPROLITE: Greyish Brown Micaceous Silty Fine **SAND** with Manganese Staining | 813.4 | 9 | 35 |
| 50 | SAPROLITE: Greyish Brown Micaceous Fine Sandy **SILT** | 808.4 | 10 | 26 |
| 55 | SAPROLITE: Greyish Brown Micaceous Silty Fine **SAND** | 803.4 | 11 | 40 |
| 60 | PARTIALLY WEATHERED ROCK: When Sampled Becomes Greyish Brown Micaceous Silty Fine **SAND** | 798.4 | REC 50% 12 | 50/5 |
| 65 | | 793.4 | 13 | 50/3 |
| 70 | PARTIALLY WEATHERED ROCK: When Sampled Becomes Olive Green and Brown Micaceous Fine Sandy **SILT** | 788.4 | 14 | 50/3 |
| 75 | PARTIALLY WEATHERED ROCK: When Sampled Becomes Olive Green and Brown Micaceous Silty Fine **SAND** | 783.4 | 15 | 50/3 |
| 80 | | 778.4 | 16 | 50/3 |
| 85 | | 773.4 | 17 | 50/4 |
| 90 | | 768.4 | 18 | 50/3 |

WATER LEVEL

STANDARD PENETRATION TEST DATA (blows/ft)




BORING LOG WELLLOGS.GPJ S&ME.GDT 3/4/03

*1. BORING AND SAMPLING IS IN ACCORDANCE WITH ASTM D-1586.*
*2. PENETRATION (N-VALUE) IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I.D. SAMPLER 1 FT.*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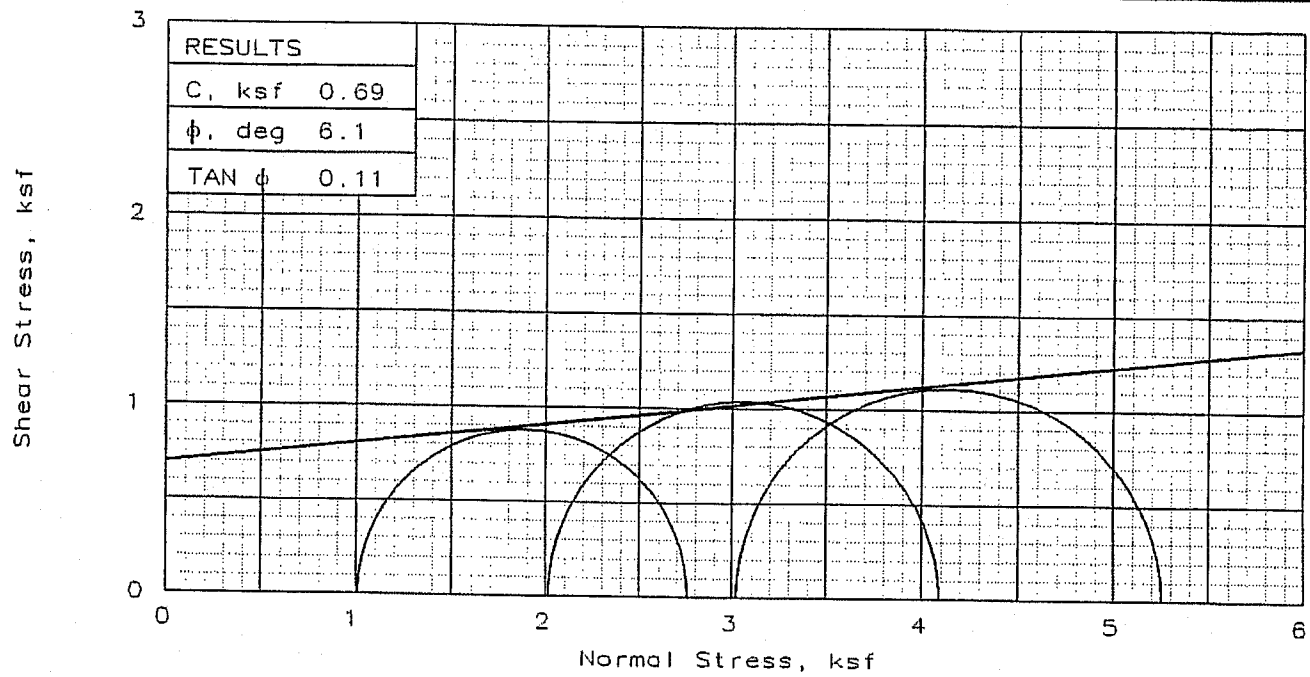
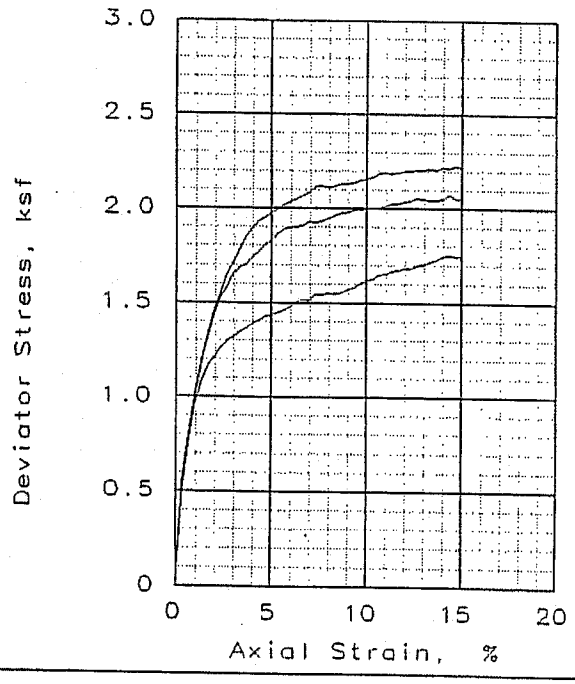
# Marshall FGD Residue Landfill

## Slope Stability Evaluation

## Copies of Triaxial Tests




| SAMPLE NO.: | | 1 | 2 | 3 |
|---|---|---|---|---|
| INITIAL | WATER CONTENT, % | 94.9 | 94.9 | 94.9 |
| | DRY DENSITY, pcf | 41.5 | 40.9 | 40.5 |
| | SATURATION, % | 88.3 | 86.5 | 85.5 |
| | VOID RATIO | 2.493 | 2.545 | 2.576 |
| | DIAMETER, in | 2.86 | 2.85 | 2.85 |
| | HEIGHT, in | 6.00 | 6.00 | 6.00 |
| AT TEST | WATER CONTENT, % | 99.2 | 92.9 | 94.4 |
| | DRY DENSITY, pcf | 41.5 | 40.9 | 40.5 |
| | SATURATION, % | 92.3 | 84.7 | 85.0 |
| | VOID RATIO | 2.493 | 2.545 | 2.576 |
| | DIAMETER, in | 2.86 | 2.85 | 2.85 |
| | HEIGHT, in | 6.00 | 6.00 | 6.00 |
| Strain rate, in/min | | 0.0040 | 0.0040 | 0.0040 |
| BACK PRESSURE, ksf | | 0.00 | 0.00 | 0.00 |
| CELL PRESSURE, ksf | | 1.01 | 2.02 | 3.02 |
| FAIL. STRESS, ksf | | 1.76 | 2.07 | 2.23 |
| STRAIN, % | | 14.3 | 14.3 | 14.7 |
| ULT. STRESS, ksf | | | | |
| STRAIN, % | | | | |
| $\sigma_1$ FAILURE, ksf | | 2.76 | 4.09 | 5.25 |
| $\sigma_3$ FAILURE, ksf | | 1.01 | 2.02 | 3.02 |

TYPE OF TEST:
Unconsolidated Undrained
SAMPLE TYPE: REMOLDED
DESCRIPTION: BIG BEND CLARIFIER SLUDGE

SPECIFIC GRAVITY= 2.32
REMARKS: SAMPLES REMOLDED TO 92 TO 94% MDD, +1.7% OPTIMUM

CLIENT: DUKE ENERGY

PROJECT: FGD CLARIFIER SLUDGE TESTING II

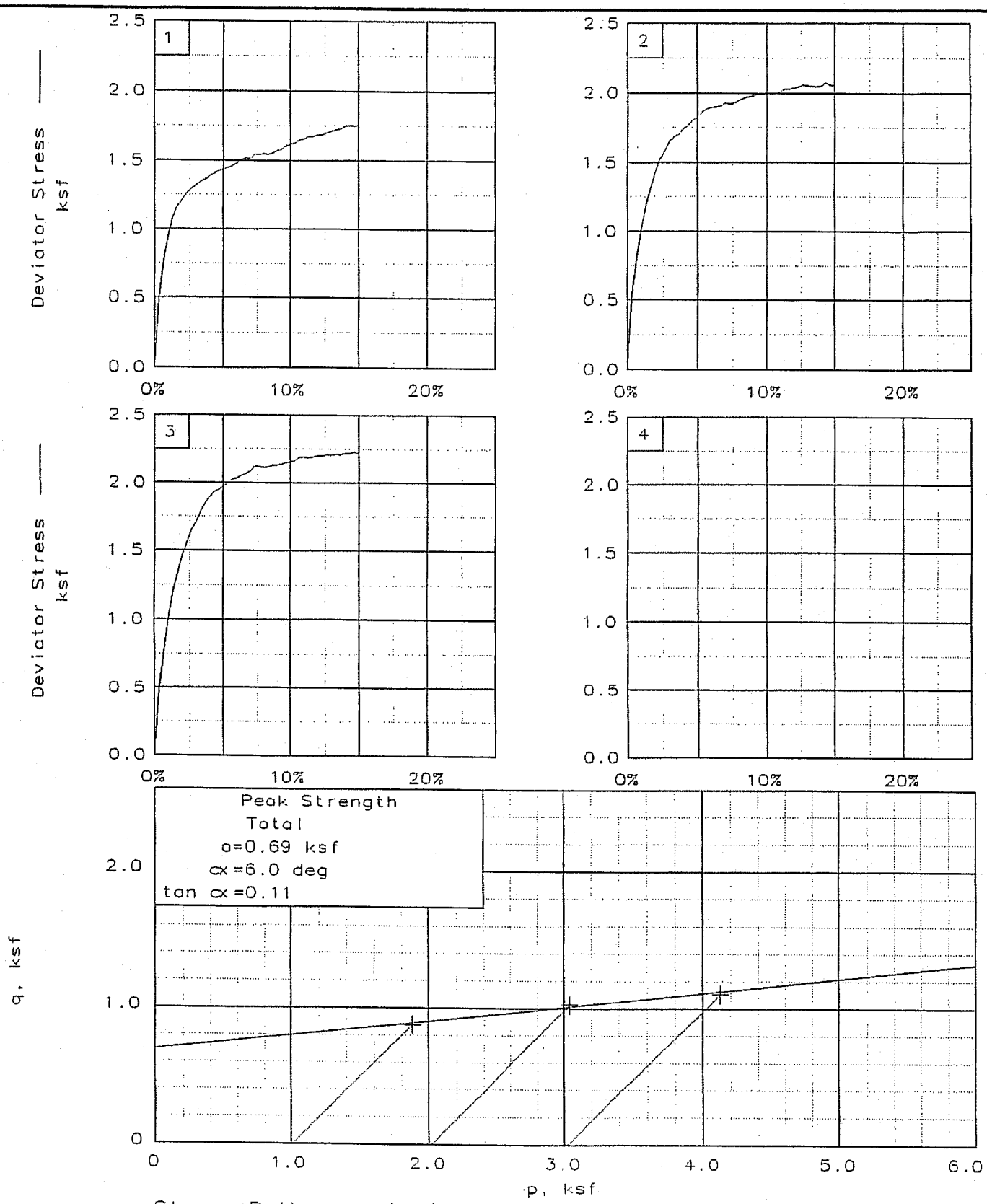
SAMPLE LOCATION: BIG BEND POWER PLANT

PROJ. NO.: 1264-03-057 DATE: 9/14/05

TRIAXIAL SHEAR TEST REPORT

S & ME, INC.

Fig. No.: II

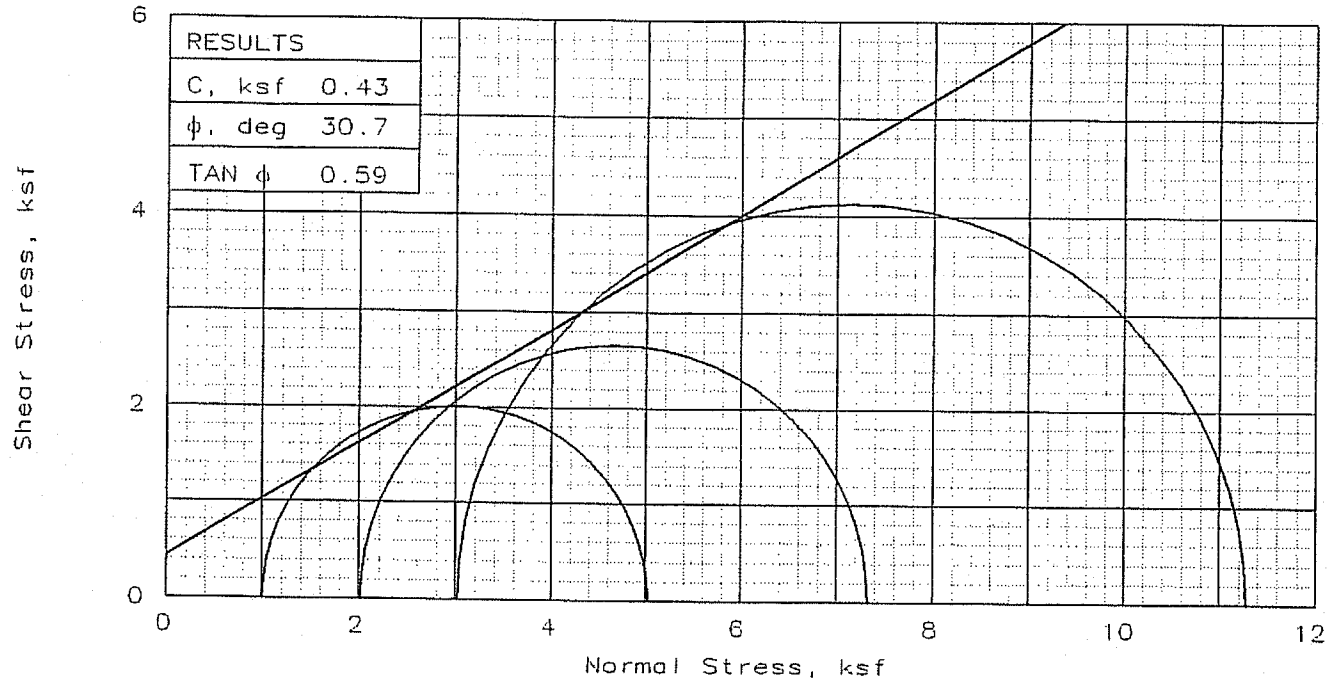


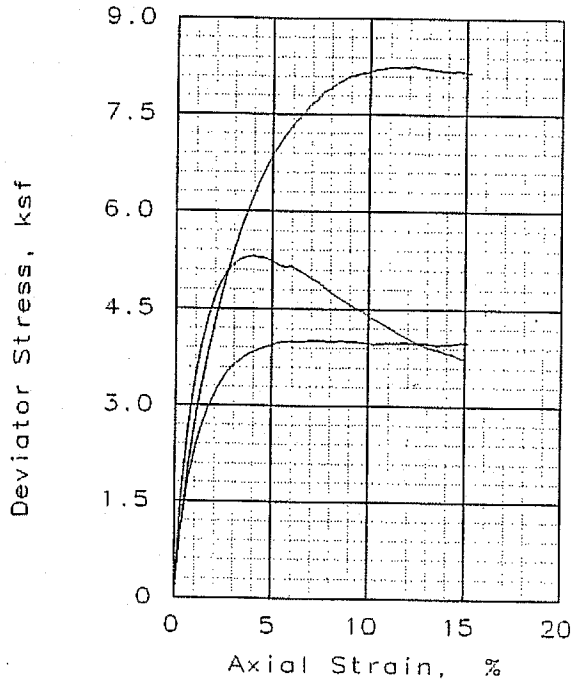

Client: DUKE ENERGY

Project: FGD CLARIFIER SLUDGE TESTING II

Location: BIG BEND POWER PLANT

File: FGDCLAUU Project No.: 1264-03-057 Fig. No.: II




| SAMPLE NO.: | | 1 | 2 | 3 |
|---|---|---|---|---|
| INITIAL | WATER CONTENT, % | 28.1 | 39.0 | 27.0 |
| | DRY DENSITY, pcf | 66.9 | 62.2 | 67.3 |
| | SATURATION, % | 55.8 | 67.9 | 54.2 |
| | VOID RATIO | 1.175 | 1.339 | 1.161 |
| | DIAMETER, in | 2.86 | 2.86 | 2.86 |
| | HEIGHT, in | 6.06 | 6.00 | 6.03 |
| AT TEST | WATER CONTENT, % | 33.3 | 33.3 | 33.3 |
| | DRY DENSITY, pcf | 66.9 | 62.2 | 67.3 |
| | SATURATION, % | 66.1 | 58.0 | 66.9 |
| | VOID RATIO | 1.175 | 1.339 | 1.161 |
| | DIAMETER, in | 2.86 | 2.86 | 2.86 |
| | HEIGHT, in | 6.06 | 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 | 2.0 | 3.0 |
| FAIL. STRESS, ksf | | 4.0 | 5.3 | 8.3 |
| STRAIN, % | | 7.3 | 4.0 | 12.3 |
| ULT. STRESS, ksf | | | | |
| STRAIN, % | | | | |
| $\sigma_1$ FAILURE, ksf | | 5.0 | 7.3 | 11.3 |
| $\sigma_3$ FAILURE, ksf | | 1.0 | 2.0 | 3.0 |

TYPE OF TEST:
Unconsolidated Undrained
SAMPLE TYPE: REMOLDED
DESCRIPTION: ONE PART CLARIFIER /ONE PART GYPSUM

SPECIFIC GRAVITY= 2.33
REMARKS: REMOLDED TO 90% MMD AT OPTIMUM MOISTURE CONTENT

CLIENT: DUKE ENERGY

PROJECT: FGD CLARIFIER SLUDGE TESTING

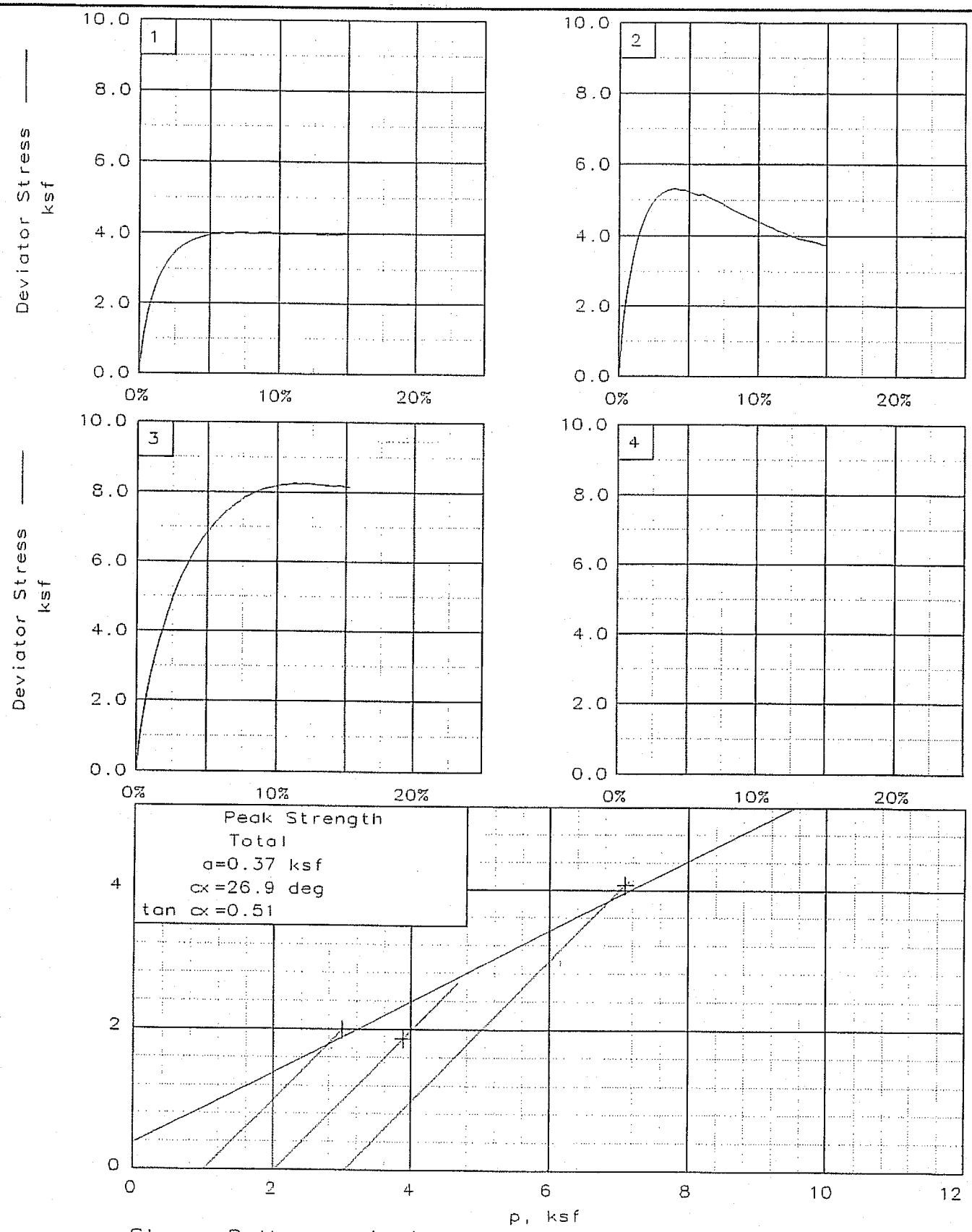
SAMPLE LOCATION: BIG BEND POWER PLANT

PROJ. NO.: 1264-03-057 DATE: 10/20/05

TRIAXIAL SHEAR TEST REPORT

S & ME, INC.

Fig. No.: S-1



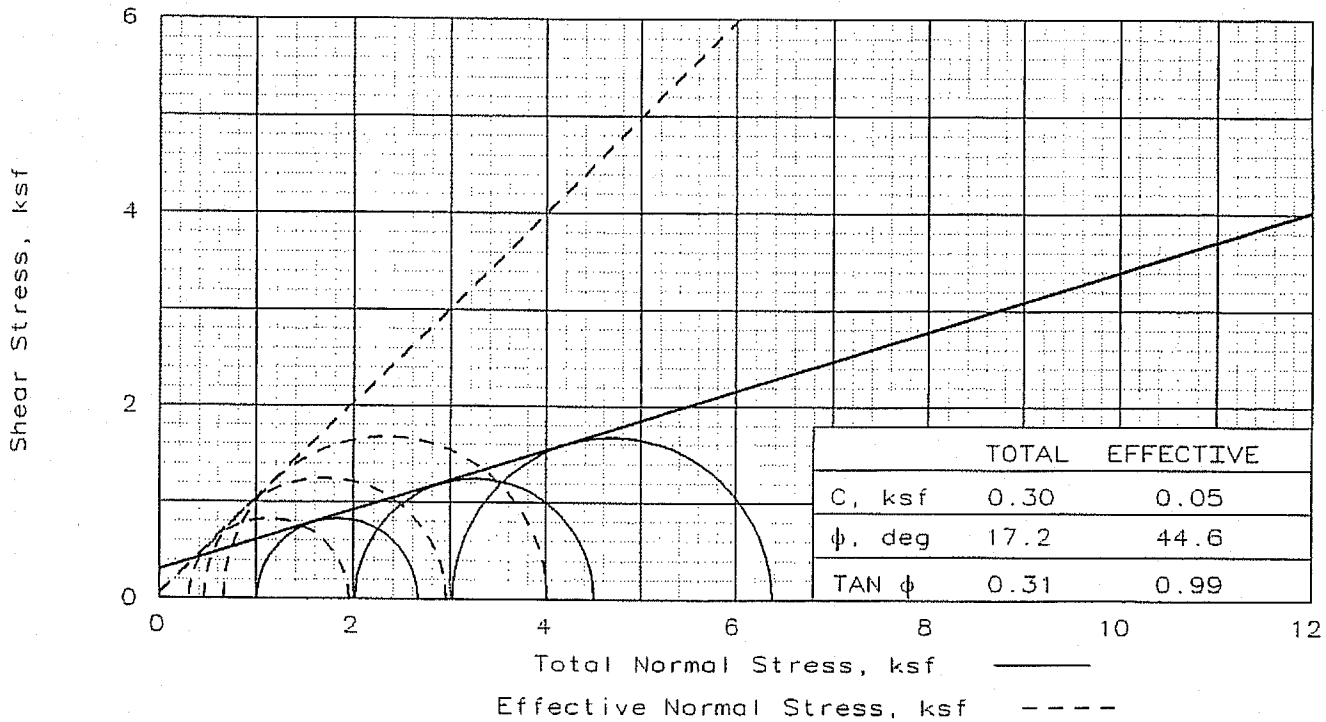

Stress Paths: + indicates end

Client: DUKE ENERGY

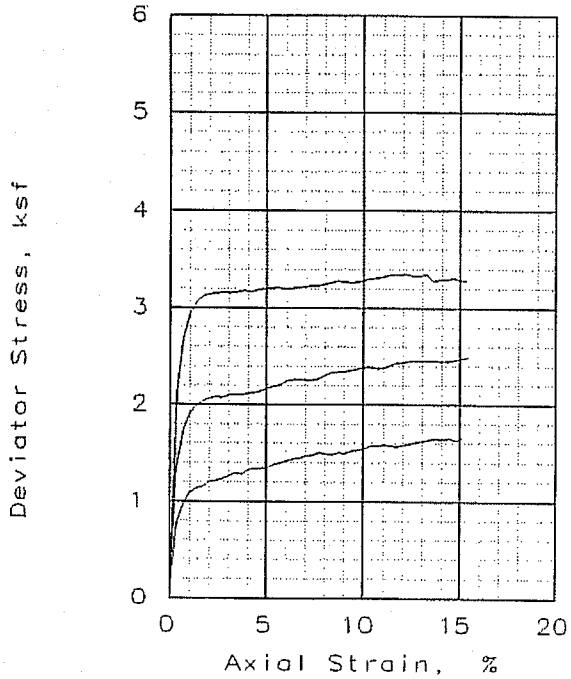
Project: FGD CLARIFIER SLUDGE TESTING

Location: BIG BEND POWER PLANT

File: CLAR1-1 Project No.: 1264-03-057 Fig. No.: S-1




| | TOTAL | EFFECTIVE |
|---|---|---|
| C, ksf | 0.30 | 0.05 |
| $\phi$, deg | 17.2 | 44.6 |
| TAN $\phi$ | 0.31 | 0.99 |




| SAMPLE NO.: | | 1 | 2 | 3 |
|---|---|---|---|---|
| INITIAL | WATER CONTENT, % | 90.9 | 90.9 | 90.9 |
| | DRY DENSITY, pcf | 42.3 | 41.5 | 41.9 |
| | SATURATION, % | 87.0 | 84.6 | 85.8 |
| | VOID RATIO | 2.424 | 2.492 | 2.458 |
| | DIAMETER, in | 2.85 | 2.85 | 2.85 |
| | HEIGHT, in | 6.00 | 5.97 | 5.97 |
| AT TEST | WATER CONTENT, % | 102.0 | 97.9 | 89.5 |
| | DRY DENSITY, pcf | 43.0 | 44.3 | 47.1 |
| | SATURATION, % | 100.0 | 100.0 | 100.0 |
| | VOID RATIO | 2.366 | 2.272 | 2.077 |
| | DIAMETER, in | 2.83 | 2.79 | 2.74 |
| | HEIGHT, in | 5.97 | 5.84 | 5.74 |
| Strain rate, in/min | | 0.0040 | 0.0040 | 0.0040 |
| EFF CELL PRESSURE, ksf | | 1.01 | 2.02 | 3.02 |
| FAIL. STRESS, ksf | | 1.66 | 2.49 | 3.35 |
| TOTAL PORE PR., ksf | | 12.23 | 13.08 | 13.88 |
| STRAIN, % | | 14.4 | 15.4 | 12.2 |
| ULT. STRESS, ksf | | | | |
| TOTAL PORE PR., ksf | | | | |
| STRAIN, % | | | | |
| $\bar{\sigma}_1$ FAILURE, ksf | | 1.96 | 2.95 | 4.02 |
| $\bar{\sigma}_3$ FAILURE, ksf | | 0.30 | 0.46 | 0.66 |

TYPE OF TEST:
CU with Pore Pressures
SAMPLE TYPE: REMOLDED
DESCRIPTION: BIG BEND CLARIFIER SLUDGE

SPECIFIC GRAVITY= 2.32
REMARKS: REMOLDED AT 90% MDD AT OPTIMUM MOISTURE

CLIENT: DUKE ENERGY

PROJECT: FGD CLARIFIER SLUDGE TESTING II

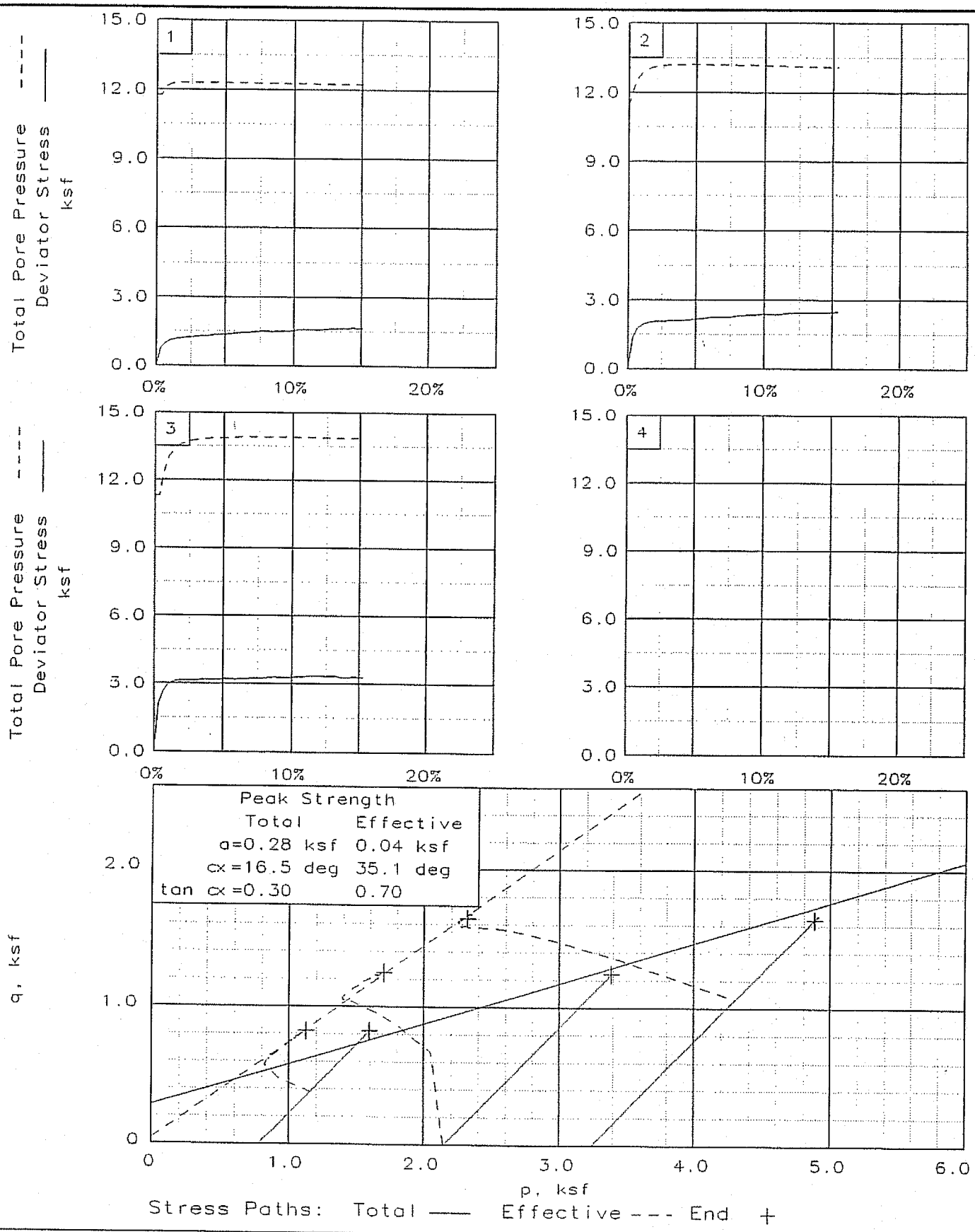
SAMPLE LOCATION: BIG BEND POWER PLANT

PROJ. NO.: 1264-03-057 DATE: 9/19/05

TRIAXIAL SHEAR TEST REPORT

S & ME, INC.

Fig. No.: II




Client: DUKE ENERGY

Project: FGD CLARIFIER SLUDGE TESTING II

Location: BIG BEND POWER PLANT

File: FGDCLACU Project No.: 1264-03-057 Fig. No.: II

## 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

| | | | |
|---|---|---|---|
| Job No. | 046003 | | |
| Comp. By | WMM | Date | 12/12/2005 |
| Chkd. By | DEB | Date | 12/16/05 |
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**Objective:** **Determine the requirements for the geotextile component of the LCRS for the landfill.**

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.

**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) *Hydrologic Study, FGD Scrubber Landfill, Duke Power Marshall Steam Station, Terrell, North Carolina S&ME Project 1264-02-578, August 20, 2003.*

5) *Soil Sampling and Testing, Duke Power - Marshall Steam Station, FGD Landfill, Terrell, North Carolina, S&ME Project No. 1356-05-571, December 5, 2005.*

6) *Clarifier Sludge Test Results I, Duke Power FGD Planning S&ME Project No. 1264-03-057*

| | | | |
|---|---|---|---|
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**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 5)

| | | Soil Particle Sizes in mm | | | | | | |
|---|---|---|---|---|---|---|---|---|
| **Sample #** | **PI** | **d100** | **d85** | **d60** | **d50** | **d30** | **d15** | **d10** |
| 1 | 5 | 9.525 | 0.317 | 0.117 | 0.0791 | 0.0165 | 0.0036 | - |
| 2 | 8 | 9.525 | 0.304 | 0.147 | 0.109 | 0.0212 | 0.00062 | - |
| 3 | 15 | 4.76 | 0.216 | 0.0169 | 0.0076 | - | - | - |
| 4 | 15 | 4.76 | 0.218 | 0.0386 | 0.0272 | 0.0064 | - | - |
| 5 | 13 | 4.76 | 0.238 | 0.0956 | 0.0364 | 0.0105 | - | - |
| 6 | 9 | 4.76 | 0.204 | 0.049 | 0.0355 | 0.0153 | 0.0049 | 0.0017 |

**Particle Size Distribution for Gypsum and Clarifier Sludge**

| | | Soil Particle Sizes in mm | | | | | | |
|---|---|---|---|---|---|---|---|---|
| **Clarifier Sludge:** | **PI** | **d100** | **d85** | **d60** | **d50** | **d30** | **d15** | **d10** |
| **Gypsum (1:1)** | NP | 2.00 | 0.0448 | 0.0289 | 0.0256 | 0.0203 | 0.0164 | 0.0147 |

(Reference 6) **Cc**
1.0

**Use d85 and Cu to Check Ability of Geotextile to Retain Site Soil (Reference 1)**

**Check $O_{95}/d85 < B$**

Where:

B = a function of the coefficient of uniformity, Cu

$Cu = \frac{d60}{d10}$

for soils where > 50% is coarser than #200

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

**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

Since the site soils all have (approximately) > 50% passing the #200 sieve, then use B = 1.8 for all samples.

| Sample # | % Fines (< #200) | d85 (mm) | B | $O_{95}$= d85 * B | $O_{95}/d85<B$ | Check $O_{95}/d85<B$ |
|---|---|---|---|---|---|---|
| 1 | 49.1% | 0.32 | 1.8 | 0.57 | 0.47 | **OK** |
| 2 | 42.9% | 0.30 | 1.8 | 0.55 | 0.49 | **OK** |
| 3 | 70.5% | 0.22 | 1.8 | 0.39 | 0.69 | **OK** |
| 4 | 63.2% | 0.22 | 1.8 | 0.39 | 0.68 | **OK** |
| 5 | 56.3% | 0.24 | 1.8 | 0.43 | 0.63 | **OK** |
| 6 | 62.1% | 0.20 | 1.8 | 0.37 | 0.73 | **OK** |

**All soil samples meet criterion of O95/d85 < B**
**Therefore, a non-woven geotextile with AOS of #100 sieve will retain site soils.**

| | | | |
|---|---|---|---|
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**Check to see if Site Soil is Well Graded and Stable**

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.

Broad graded soils are characterized by Reference 1 as having d60/d10 ratios greater than 20.

For a soil to be well-graded and stable, it must meet the criteria:

$$Cu = \frac{d60}{d10} > 4 \quad \text{and} \quad 1 < Cc = \frac{d^2 30}{d10 * d60} < 3$$

Reference 1 Eqn. 2.1

**Site Soils**
The site soils available for use as the operational cover are fine grained soils. In addition, as shown on the grain size distribution curves, these soils are broad graded.

As discussed in Reference 1, the fine particles in broad graded soils tend to pipe through coarser particles.

The criteria are checked with the proposed requirement that site soils have a d10 of 0.004 or greater.

| Sample # | PI | d60 | d30 | d10 | Cu | Cc | Meets Criteria for Cu>4 | Meets Criteria for Cc? |
|---|---|---|---|---|---|---|---|---|
| 1 | 5 | 0.117 | 0.0165 | 0.004 | 29.3 | 0.6 | Yes | No |
| 2 | 8 | 0.147 | 0.0212 | 0.004 | 36.8 | 0.8 | Yes | No |
| 3 | 15 | 0.0169 | | 0.004 | 4.2 | NA | Yes | No |
| 4 | 15 | 0.0386 | 0.0064 | 0.004 | 9.7 | 0.3 | Yes | No |
| 5 | 13 | 0.0956 | 0.0105 | 0.004 | 23.9 | 0.3 | Yes | No |
| 6 | 9 | 0.049 | 0.0153 | 0.004 | 12.3 | 1.2 | Yes | Yes |

**<u>The site soils do not meet the requirements without further screening.</u>**

**Soil sample 6 meets the criteria for well graded and stable soils.**

**Based on the gradation of Sample 6, develop gradation limits that will meet the criteria for well graded and stable soils.**

| | | | |
|---|---|---|---|
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**Develop Gradation Limits for Operational Cover Soil**

Since the clarifier:gypsum mix will be placed on top over the operational soil cover, design the gradation for the soil to:
1) serve as a soil filter for the clarifier:gypsum mix.
2) meet the above requirements for a well graded, stable soil.

Use filter criteria from Reference 3, Figure 4.

The clarifier:gypsum mix is considered the base material:

The Trial Gradation for the operational cover was designed by using the gradation of Soil Sample 6 as a starting point.

**Operational Cover Soil Gradation**

| | Coarse | |
|---|---|---|
| Sieve | mm | % Finer |
| #20 | 0.841 | 100% |
| #40 | 0.420 | 86% |
| | 0.410 | 85% |
| #60 | 0.250 | 70% |
| #100 | 0.149 | 60% |
| | 0.100 | 50% |
| #200 | 0.075 | 45% |
| | 0.040 | 30% |
| | 0.030 | 25% |
| | 0.020 | 18% |
| | 0.016 | 15% |
| | 0.010 | 10% |

| | Finer | |
|---|---|---|
| Sieve | mm | % Finer |
| #20 | 0.841 | |
| #40 | 0.420 | 100% |
| #60 | 0.250 | 92% |
| | 0.165 | 85% |
| #100 | 0.149 | 84% |
| #200 | 0.075 | 73% |
| | 0.043 | 60% |
| | 0.030 | 55% |
| | 0.024 | 50% |
| | 0.020 | 47% |
| | 0.010 | 30% |
| | 0.004 | 15% |
| | 0.002 | 10% |

**The Trial Gradation is checked below:**

**Step 1**

**Check gradation of clarifier sludge:gypsum mix against the Trial Operational Cover Soil gradation.**

| | Trial Gradation - Filter (mm) | | Clarifier Sludge:Gypsum Mix |
|---|---|---|---|
| | coarse | fine | Base (mm) |
| **d15** | 0.016 | 0.004 | 0.0164 |
| **d50** | 0.100 | 0.024 | 0.0256 |
| **d85** | 0.410 | 0.165 | 0.0448 |

**To avoid movement of particles from base, check the following:** (Reference 3)

$\frac{\text{d15 filter}}{\text{d85 base}} < 5$ $\quad \frac{\text{d15 filter}}{\text{d85 base}} = \frac{0.016}{0.0448} = 0.4$ OK

$\frac{\text{d50 filter}}{\text{d50 base}} < 25$ $\quad \frac{\text{d50 filter}}{\text{d50 base}} = \frac{0.100}{0.0256} = 3.9$ OK

$\frac{\text{d15 filter}}{\text{d15 base}} < 20$ $\quad \frac{\text{d15 filter}}{\text{d15 base}} = \frac{0.016}{0.0164} = 1.0$ OK

**Trial Gradation is acceptable to prevent gypsum:clarifer mix particles from moving into geotextile.**

**Step 2**

Verify that Trial Operational Cover Soil gradation is well graded and stable.

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.

Broad graded soils are characterized by Reference 1 as having d60/d10 ratios greater than 20.

For a soil to be well-graded and stable, it must meet the criteria:

$$Cu = \frac{d60}{d10} > 4 \quad \text{and} \quad 1 < Cc = \frac{d^2 30}{d10 * d60} < 3$$

Reference 1
Eqn. 2.1

| | d60 | d30 | d10 | Cu | Cc | Meets Criteria Meets Criteria for Cu>4 | Meets Criteria Meets Criteria for Cc? |
|---|---|---|---|---|---|---|---|
| **Coarse Limit** | 0.149 | 0.040 | 0.010 | 14.9 | 1.1 | Yes | Yes |
| **Fine Limit** | 0.043 | 0.010 | 0.002 | 21.3 | 1.2 | Yes | Yes |

**Trial Operational Cover Soil above meets criteria for being well graded and stable.**

| | | | |
|---|---|---|---|
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**Step 3**

**Use d85 and Cu to Check Ability of Geotextile to Retain Trial Gradation Soil (Reference 1)**

**Check $O_{95}/d85 < B$**

Where:

B = a function of the coefficient of uniformity, Cu

$Cu = \frac{d60}{d10}$

for soils where > 50% is coarser than #200

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

**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

| Sample # | % Fines (< #200) | d85 (mm) | B | AOS ($O_{95}$) #100 Sieve | $O_{95}/d85$ | Check $O_{95}/d85<B$ |
|---|---|---|---|---|---|---|
| Coarse Limit | 45% | 0.400 | 1.0 | 0.149 | 0.37 | OK |
| Fine Limit | 73% | 0.165 | 1.8 | 0.149 | 0.90 | OK |

**Step 4**

**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

| | d15 | Cu | Ratio $O_{95}/d15$ | Meets $O_{95}$>d15 for Cu > 3? |
|---|---|---|---|---|
| Coarse Limit | 0.016 | 14.9 | 9.2 | Yes |
| Fine Limit | 0.004 | 21.3 | 37.3 | Yes |

**Gradation is acceptable to use over AOS #100 geotextile.**

**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 |

**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.**

**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.

| Sample | K (cm/sec) | 100 * Ksoil (cm/sec | Ratio Kgeotextile / 100 * Ksoil |
|---|---|---|---|
| 2 | 1.66E-04 | 1.66E-02 | 12 |
| 3 | 8.17E-05 | 8.17E-03 | 24 |
| 4 | 2.91E-05 | 2.91E-03 | 69 |
| 6 | 5.52E-05 | 5.52E-03 | 36 |
| Average K = | 8.30E-05 | 100* $K_{soil\ (avg)}$ = | 8.30E-03 cm/sec |
| Minimum K = | 2.91E-05 | 100* $K_{soil\ (min)}$ = | 2.91E-03 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 for Geotextile**

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 for Geotextile**

Minimum UV exposure ==> no special requirements
Chemical compatibility ==> use polyethylene or polypropylene material.

| | | | |
|---|---|---|---|
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**Requirements for Site Soils**

**Site soils do not meet the requirements for use as cover for the geotextile without additional screening.**

**The soils used as Operational Cover Soil should meet the following gradation requirements:**

**Operational Cover Soil Gradation**

| | Coarse | |
|---|---|---|
| Sieve | mm | % Finer |
| #20 | 0.841 | 100% |
| #40 | 0.420 | 86% |
| | 0.410 | 85% |
| #60 | 0.250 | 70% |
| #100 | 0.149 | 60% |
| | 0.100 | 50% |
| #200 | 0.075 | 45% |
| | 0.040 | 30% |
| | 0.030 | 25% |
| | 0.020 | 18% |
| | 0.016 | 15% |
| | 0.010 | 10% |

| | Finer | |
|---|---|---|
| Sieve | mm | % Finer |
| #20 | 0.841 | |
| #40 | 0.420 | 100% |
| #60 | 0.250 | 92% |
| | 0.165 | 85% |
| #100 | 0.149 | 84% |
| #200 | 0.075 | 73% |
| | 0.043 | 60% |
| | 0.030 | 55% |
| | 0.024 | 50% |
| | 0.020 | 47% |
| | 0.010 | 30% |
| | 0.004 | 15% |
| | 0.002 | 10% |

| | | | |
|---|---|---|---|
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**SUMMARY OF REQUIRED PROPERTIES FOR GEOTEXTILE:**

**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 $sec^{-1}$ |

**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

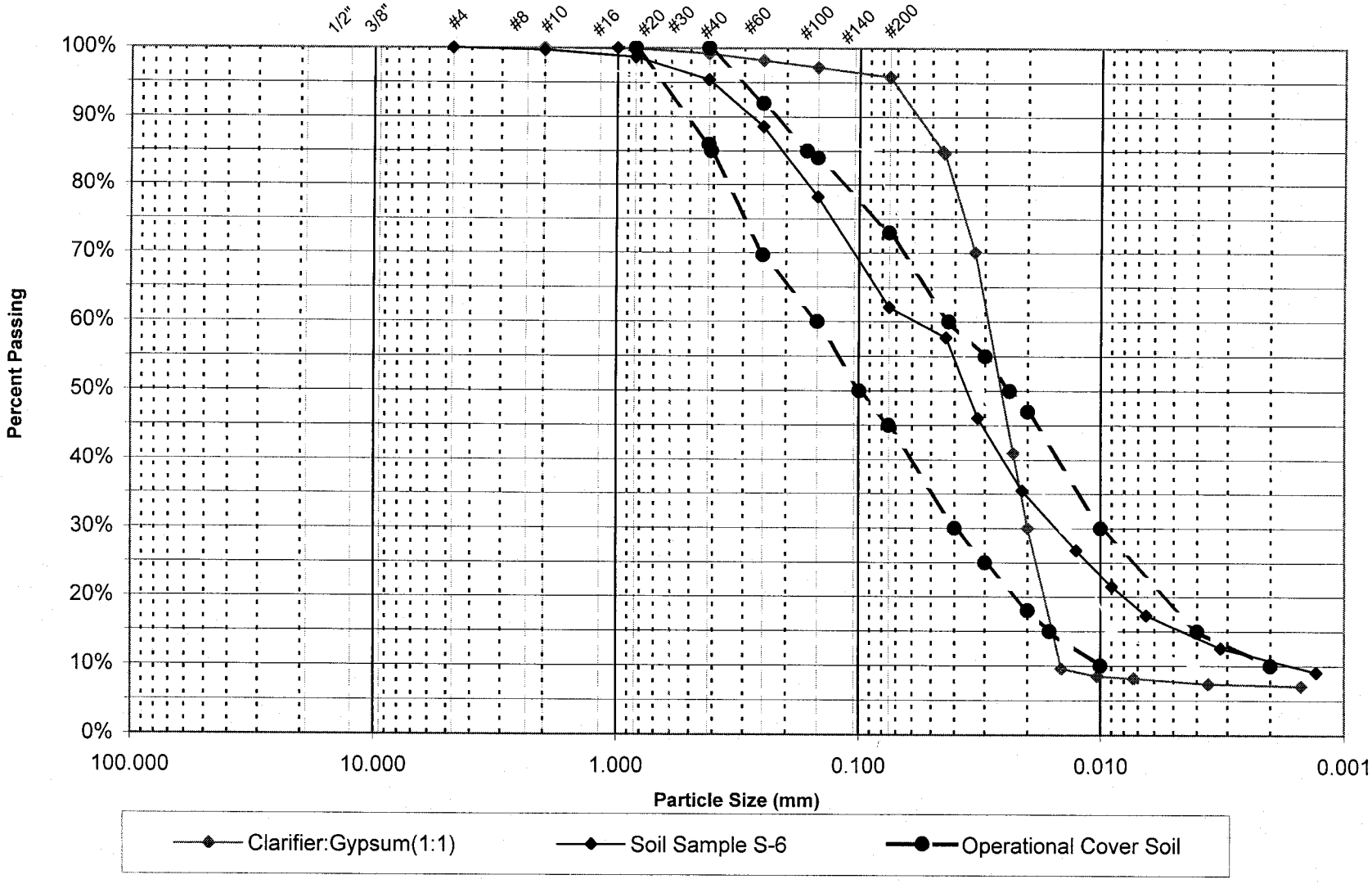
| | | | |
|---|---|---|---|
| $p'$ | = | 67.5 psi | (Cat 623B Scraper) |
| $d_a$ | = | 1.0 " | for #57 Stone (conservative) |
| $S_1$ | = | 1.0 | assume protrusion height =1", so 1"/1" |
| $S_2$ | = | 0.31 | = 0.31/1" |
| $S_3$ | = | 0.6 | =1 - 0.4 |

$F_{reqd}$ = 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.

**Figure 1**
**Marshall FGD Residue Landfill**
**Particle Size Distribution For Clarifier Sludge-Gypsum Mix and Operational Soil**



# Geotextile Design

## References

DRAFT

# DESIGN OF LATERAL DRAINAGE SYSTEMS FOR LANDFILLS

**Gregory N. Richardson, Ph.D., P.E.**
G.N. Richardson & Associates
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**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

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:

$$O_{95} > 3\ D_{15} \qquad \text{for } C_u > 3$$

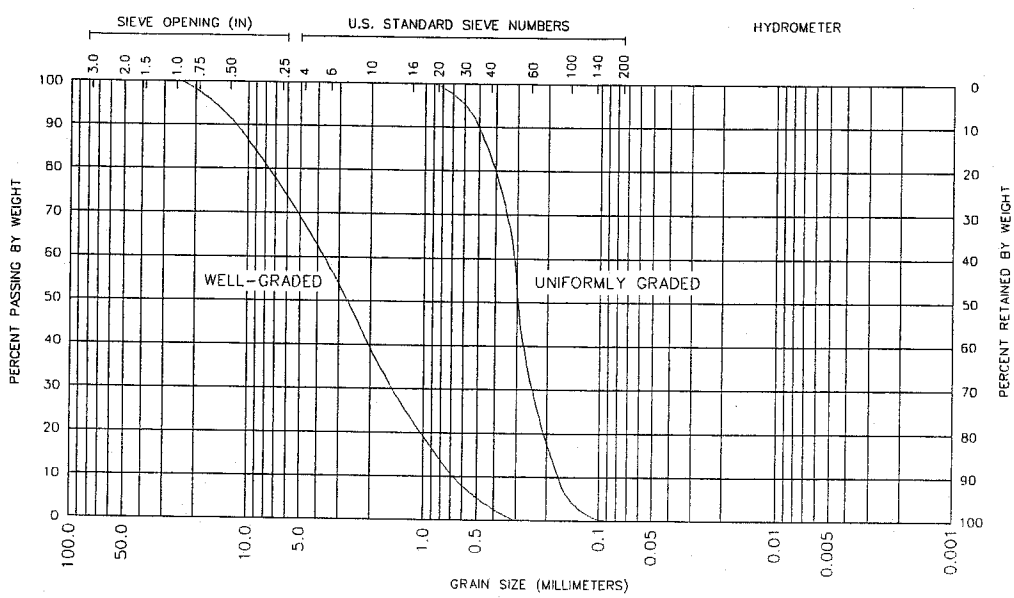



**Figure 2.5 Well-graded and uniformly-graded soils.**

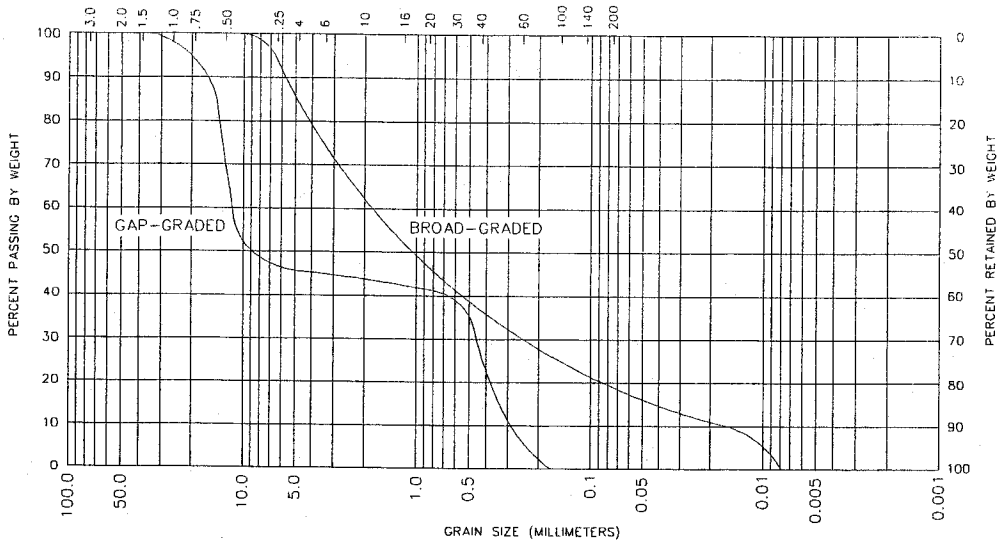



**Figure 2.6 Gap-graded and broad-graded soils**

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):

$K_{geotextile} > 100\ K_{soil}$ 'Active' Liquid

$K_{geotextile} > 10\ K_{soil}$ '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:

$$\begin{aligned} T_{\text{reqd}} &= p'(d_v)^2(0.52) \\ &= p'(0.33\ d_a)^2(0.52) \\ &= 0.057\ p'd_a^2 \\ &= 0.057(100)(2.0)^2 \\ &= 22.6 \text{ lb.} \end{aligned}$$

(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:

$$\begin{aligned} FS &= \frac{T_{\text{allow}}}{T_{\text{reqd}}} \\ &= \frac{125/2.5}{22.6} \\ &= 2.2, \text{ which is acceptable.} \end{aligned}$$

### 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:

$$F_{\text{reqd}} = p'd_a^2 S_1 S_2 S_3 \tag{2.30}$$

where $F_{\text{reqd}}$ = 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

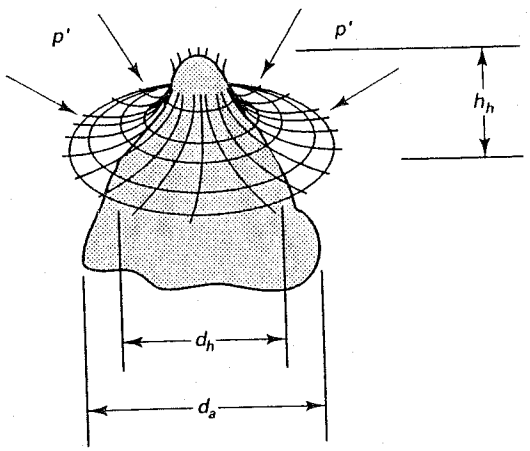



**Figure 2.29** Visualization of a stone puncturing a geotextile as pressure is applied from above.

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,

$$\begin{aligned} F_{\text{reqd}} &= p' d_a^2 S_1 S_2 S_3 \\ &= 80 \times (2.0)^2 (0.33)(0.155)(0.6) \\ &= 9.82 \text{ lb.} \end{aligned}$$

Assuming that the cumulative partial factor of safety is 2.0, the global factor of safety is as follows:

$$\begin{aligned} FS &= \frac{F_{\text{allow}}}{F_{\text{reqd}}} \\ &= \frac{45/2.0}{9.82} \\ &= 2.3, \text{ which is acceptable.} \end{aligned}$$

# Particle Size Analysis for "Cut" Material

## Geonet Design

- Calculation for Geonet for Liner System of Landfill
- Calculation for Geonet for Cover System of Landfill
- Copies of References

| | | | | |
|---|---|---|---|---|
| **Marshall FGD Residue Landfill** | **Job No.** | **046003** | | |
| **Geonet Transmissivity Calculations** | **Comp. By** | **WMM** | **Date** | **12/12/2005** |
| **Bottom of Landfill** | **Chkd. By** | DEB | **Date** | 12/16/05 |
| | **Sheet** | 1 | **of** | 6 |

**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)**

$$\Theta_{min} = \frac{B^2 q}{hmax + B\sin\Phi}$$

$\Theta$ = geonet transmissivity
B = slope length
q = leachate quantity, gal/acre/day (From HELP)
hmax = allowable head on liner, ft
$\Phi$ = slope of geonet, degrees

| | | | | |
|---|---|---|---|---|
| B = | 110 | ft | | |
| q = | 9,158 | gal/ac/day | | |
| hmax = | 1.0 | ft | | |
| $\Phi$ = | 1.5% = | | 0.573 degrees | (minimum slope) |

convert q into $ft^3/ft^2/day$

| | | |
|---|---|---|
| q = | 9,158 | gal/ac/day |
| q = | 0.028 | $ft^3/ft^2/day$ |

| | | |
|---|---|---|
| $\Theta_{min}$ = | 54.9 | $ft^2/day$ |
| $\Theta_{min}$ = | 5.87E-05 | $m^2/sec$ |

**Marshall FGD Residue Landfill**
**Geonet Transmissivity Calculations**
**Bottom of Landfill**

| | | | |
|---|---|---|---|
| **Job No.** | **046003** | | |
| **Comp. By** | **WMM** | **Date** | **12/12/2005** |
| **Chkd. By** | **DEB** | **Date** | 12/16/05 |
| **Sheet** | 2 | **of** | 6 |

**Calculate the Normal Stress**

Operational Cover:

| | | |
|---|---|---|
| Density | 125 lb/ft$^3$ | (assumed) |
| Thickness | 1 ft | |

Waste:

| | | |
|---|---|---|
| Density | 60 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 = 5902.5 lb/ft$^2$

**Calculate Hydraulic Gradient**

$$i = \frac{hmax + B\sin\Phi}{B/\cos\Phi}$$

i = 0.019

**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

**For leachate collection systems**

| | | | |
|---|---|---|---|
| FSin = | 1.5 to 2.0 | use | 1.75 |
| FScr = | 1.4 to 2.0 | use | 1.75 |
| FScc = | 1.5 to 2.0 | use | 1.75 |
| FSbc = | 1.5 to 2.0 | use | 1.75 |

$$q_{allowable} = \frac{1}{(1.75)\ (1.75)\ (1.75)\ (1.75)} \times .38$$ gal/min-ft

$q_{allowable}$ = 0.042 gal/min-ft

**Marshall FGD Residue Landfill**
**Geonet Transmissivity Calculations**
**Bottom of Landfill**

| | | | |
|---|---|---|---|
| **Job No.** | **046003** | | |
| **Comp. By** | **WMM** | **Date** | **12/12/2005** |
| **Chkd. By** | DEB | **Date** | 12/16/05 |
| **Sheet** | 3 | **of** | 6 |

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

$\Theta_{actual} = \frac{0.042}{0.019}$ gal/min-ft

$\Theta_{actual} =$ 2.18 gal/min-ft
$=$ 4.54E-04 $m^2$/sec

**Check $\Theta_{actual} > \Theta_{min}$**

$\Theta_{actual} =$ 4.54E-04 $m^2$/sec

$\Theta_{min} =$ 5.87E-05 $m^2$/sec

**OK**

Factor of Safety

$$FS = \frac{\Theta_{actual}}{\Theta_{min}}$$

$$FS = \frac{4.54E\text{-}04 \ m^2/sec}{5.87E\text{-}05 \ m^2/sec}$$

**FS = 7.7**

| | | | | |
|---|---|---|---|---|
| **Marshall FGD Residue Landfill** | **Job No.** | **046003** | | |
| **Geonet Transmissivity Calculations** | **Comp. By** | **WMM** | **Date** | **12/15/2005** |
| **Cover of Landfill** | **Chkd. By** | DEB | **Date** | 12/16/05 |
| | **Sheet** | 4 | **of** | 6 |

**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)**

$$\Theta_{min} = \frac{B^2 q}{hmax + B\sin\Phi}$$

$\Theta$ = geonet transmissivity
B = slope length
q = leachate quantity, gal/acre/day (From HELP)
hmax = allowable head on liner, ft
$\Phi$ = slope of geonet, degrees

| | | | |
|---|---|---|---|
| B = | 300 | ft | |
| q = | 51,166 | gal/ac/day | |
| hmax = | 1.00 | ft | |
| $\Phi$ = | 7.8% = | 4.5 ° | (minimum slope) |

convert q into $ft^3/ft^2/day$

| | | |
|---|---|---|
| q = | 51,166 | gal/ac/day |
| q = | 0.157 | $ft^3/ft^2/day$ |

| | | |
|---|---|---|
| $\Theta_{min}$ = | 279.0 | $ft^2/day$ |
| $\Theta_{min}$ = | 2.99E-04 | $m^2/sec$ |

**Marshall FGD Residue Landfill**
**Geonet Transmissivity Calculations**
**Cover of Landfill**

| | | | |
|---|---|---|---|
| **Job No.** | **046003** | | |
| **Comp. By** | **WMM** | **Date** | **12/15/2005** |
| **Chkd. By** | DEB | **Date** | 12/16/05 |
| **Sheet** | 5 | **of** | 6 |

**Calculate the Normal Stress**

Final Cover

| | | |
|---|---|---|
| Density | 125 $lb/ft^3$ | (assumed) |
| Thickness | 2 ft | |

Normal Stress = height of material x unit weight of material

Normal Stress = 250 $lb/ft^2$

**Calculate Hydraulic Gradient**

$$i = \frac{hmax + B\sin\Phi}{B/\cos\Phi}$$

i = 0.081 ft/ft

**Estimate Actual Transmissivity**

from Figure 4.8, Reference 2, the flow rate considering the normal stress

is = 0.75 gal/min-ft

$$q_{allowable} = \frac{1}{FSin \times FScr \times FScc\ FSbc} \times q_{ult}$$ Reference 2, Table 4.2

**For surface water drains for landfill caps**

| | | | |
|---|---|---|---|
| FSin = | 1.3 to 1.5 | use | 1.5 |
| FScr = | 1.2 to 1.4 | use | 1.4 |
| FScc = | 1.0 to 1.2 | use | 1.2 |
| FSbc = | 1.2 to 1.5 | use | 1.5 |

$$q_{allowable} = \frac{1}{1.5 \times 1.4 \times 1.2 \times 1.5} \times .75$$ gal/min-ft

$q_{allowable}$ = 0.198 gal/min-ft

**Marshall FGD Residue Landfill**
**Geonet Transmissivity Calculations**
**Cover of Landfill**

| | | | |
|---|---|---|---|
| **Job No.** | **046003** | | |
| **Comp. By** | **WMM** | **Date** | **12/15/2005** |
| **Chkd. By** | DEB | **Date** | 12/16/05 |
| **Sheet** | 6 | **of** | 6 |

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

$$\Theta_{actual} = \frac{0.198 \text{ gal/min-ft}}{0.081 \text{ ft/ft}}$$

$\Theta_{actual} =$ 2.45 gal/min-ft
$=$ 5.10E-04 $m^2$/sec

**Check $\Theta_{actual} > \Theta_{min}$**

$\Theta_{actual} =$ 5.10E-04 $m^2$/sec

$\Theta_{min} =$ 2.99E-04 $m^2$/sec

**OK**

Factor of Safety

$$FS = \frac{\Theta_{actual}}{\Theta_{min}}$$

$$FS = \frac{5.10E\text{-}04 \ m^2/sec}{2.99E\text{-}04 \ m^2/sec}$$

**FS = 1.7**

**Cell Component:** LEACHATE COLLECTION/REMOVAL SYSTEMS

**Consideration:** TRANSMISSIVITY, VERIFY THAT LCR PROVIDES ADEQUATE PLANAR FLOW.

| Required Material Properties | Range | Test | Standard |
|---|---|---|---|
| PLANAR FLOW CAPACITY | $3 \times 10^{-5}$ * ($m^2/s$) | TRANSMISSIVITY | ASTM D4617 |

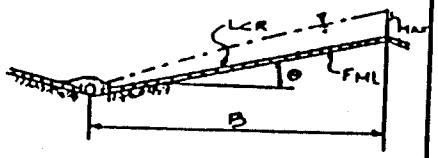
* Draft MTG Minimum

## Analysis Procedure:

(1) DEFINE MINIMUM TRANSMISSIVITY

- STATUTORY = $3 \times 10^{-5}$ $m^2/s$
- 1-FOOT HEAD

$$T = \frac{B^2 q}{h_{max} + B \sin\theta}$$




q = LEACHATE INFLOW RATE

$h_{max}$ = 1 FOOT OF HEAD

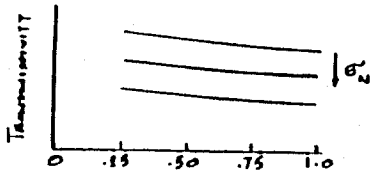
(2) CALCULATE MAXIMUM NORMAL STRESS

$$\sigma_N = D \times \gamma$$

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/ft³
- 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)

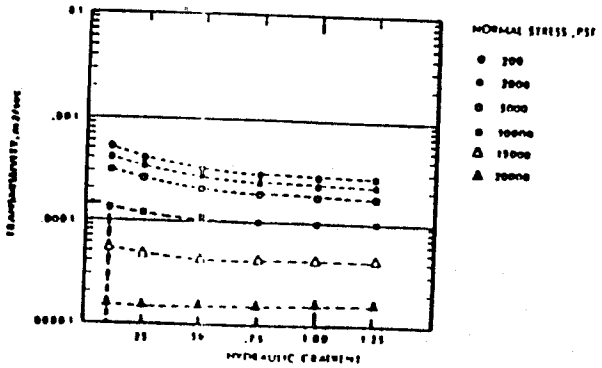
$$T = \frac{60^2 \times 0.01}{1 + 60 \sin 4^\circ} = 6.94 \text{ ft}^2/\text{day} = 7.46 \times 10^{-6} \text{ m}^2/\text{sec}$$

$$\therefore T_{REQ} = 7.46 \times 10^{-6} \text{ m}^2/\text{s} \quad (> 3 \times 10^{-5} \text{ m}^2/\text{s})$$

(2) CALCULATE MAXIMUM NORMAL STRESS, $\sigma_N$

$$\sigma_N = 120 \times 80 = 9600 \text{ PSF}$$

(3) OBTAIN LCR TRANSMISSIVITY FROM LAB. DATA, $T_{LCR}$




GRADIENT, $i = \frac{1 + 60 \sin 4^\circ}{(60/\cos 4^\circ)}$

$i = .086$

$T_{LCR} = .00014$ $m^2/s$

(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



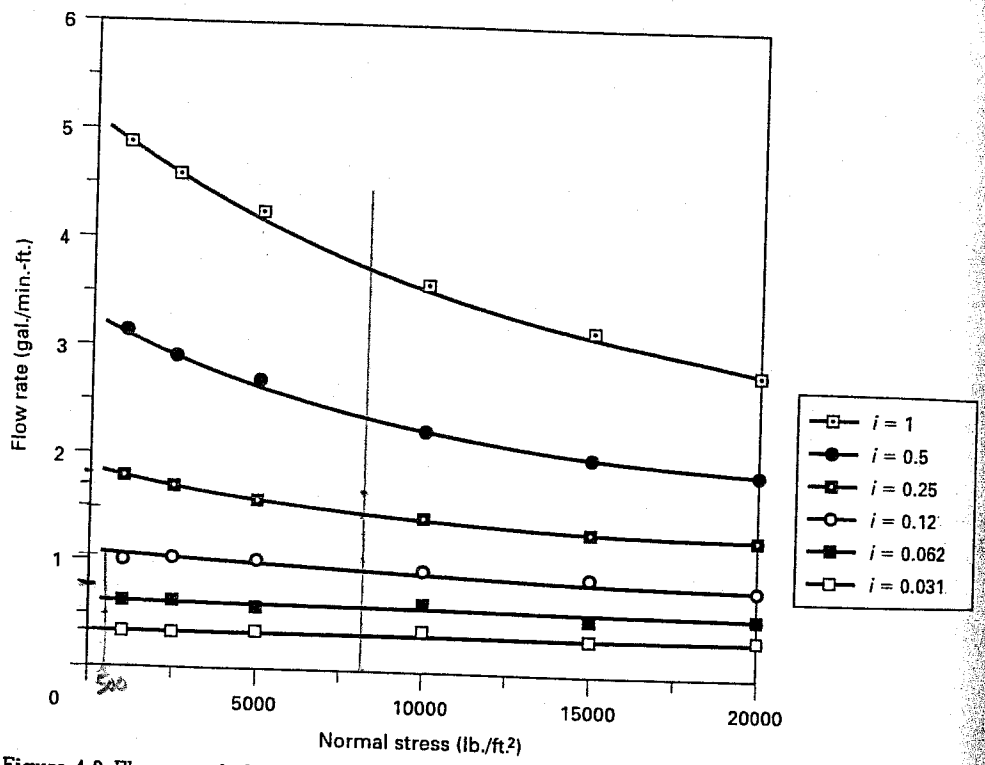



**Figure 4.8** Flow rate behavior of a 0.25-in. (6.3-mm) geonet sandwiched between a 16-oz./yd.² (540-g/m²) nonwoven needle-punched geotextile with clay above and a 60-mil (1.5-mm) HDPE geomembrane below.

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:

$$q = kiA \tag{4.1}$$

$$q = ki(t \times W)$$

$$q/W = i(k \times t)$$

$$q/W = i\theta \tag{4.2}$$

e problem does not
landfill leachate
landfill, tem-
ce) is available, and
available or not only
lata regarding micro-
is currently ongoing,
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and weather, is not
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on black is included
covered as soon as

concept is the estab-
, where flow rate is

(4.3)

n loading conditions

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to obtain the equiv-

(4.4)

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ip in contrast to the
n some adjustments
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Reference 2

generated value is an ultimate value which, using ASTM D4716 for flow rate determination, must be reduced before use in design; that is,

$$q_{\text{allow}} < q_{\text{ult}}$$

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,

$$q_{\text{allow}} = q_{\text{ult}}\left[\frac{1}{FS_{IN} \times FS_{CR} \times FS_{CC} \times FS_{BC}}\right] \tag{4.5}$$

or if all of the partial factors of safety are lumped together,

$$q_{\text{allow}} = q_{\text{ult}}\left[\frac{1}{\Sigma\, FS_p}\right] \tag{4.6}$$

where $q_{\text{ult}}$ = the flow rate determined from ASTM D4716 for short-term tests between solid plates using water as the transported liquid under laboratory test temperatures,

$q_{\text{allow}}$ = 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.

$$q_{allow} = q_{ult}\left[\frac{1}{FS_{IN} \times FS_{CR} \times FS_{CC} \times FS_{BC}}\right] \tag{4.5}$$

$$= 1.2\left[\frac{1}{1.1 \times 1.1 \times 1.1 \times 1.2}\right]$$

$$= 1.2\left[\frac{1}{1.60}\right]$$

$$= 0.75 \text{ gal./min.-ft.}$$

**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.



## 4.2 DESIGNING F(

This section w
transportation-
field.

### 4.2.1 Theoreti

Design by func

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where $q_{allow}$ = t
$q_{reqd}$ = t

As stated prev
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flow rate, name
equations 4.1 a

ermining allowable flow

| in ion 4.5 | |
|---|---|
| $FS_{CC}$ | $FS_{BC}$ |
| 0 to 1.2 | 1.1 to 1.3 |
| 1 to 1.5 | 1.1 to 1.3 |
| 0 to 1.2 | 1.1 to 1.3 |
| 1 to 1.5 | 1.0 to 1.5 |
| 0 to 1.2 | 1.0 to 1.2 |
| 0 to 1.2 | 1.2 to 1.5 |
| .5 to 2.0 | 1.5 to 2.0 |
| .5 to 2.0 | 1.5 to 2.0 |

ormal pressure of 1.5 to 2
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that this testing yielded

values from Table 4.2

$S_B$ (4.5)

e design of a secondary
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en-rigid-plates value of

er, note the large reduc-



Ref. 2

$$q_{allow} = q_{ult}\left[\frac{1}{FS_{IN} \times FS_{CR} \times FS_{CC} \times FS_{BC}}\right] \tag{4.5}$$

$$= 1.2\left[\frac{1}{1.75 \times 1.75 \times 1.75 \times 1.75}\right]$$

$$= 1.2\left[\frac{1}{9.38}\right]$$

$$= 0.13 \text{ gal./min.-ft.}$$

## 4.2 DESIGNING FOR GEONET DRAINAGE

This section will be subdivided into required theory, drainage examples in the transportation-related field, and drainage examples in the environment-related field.

### 4.2.1 Theoretical Concepts

Design by function requires the formulation of a global factor of safety as follows:

$$FS = \frac{\text{allowable (test) value}}{\text{required (design) value}}$$

For geonets serving as a drainage medium, the targeted value is flow rate and the aforementioned concepts become the following equation:

$$FS = \frac{q_{allow}}{q_{reqd}} \tag{4.1}$$

where $q_{allow}$ = the allowable flow rate (as discussed in Section 4.1.6), and
$q_{reqd}$ = the required flow rate (to be discussed here).

As stated previously, calculations can be based on Darcy's formula (assuming saturated conditions and laminar flow) if one desires to arrive at an alternative to flow rate, namely, the transmissivity, θ. This important concept is repeated from equations 4.1 and 4.2:

$$q = kiA \tag{4.1}$$

$$q = ki(W \times t)$$

$$q = (kt)iW$$

$$kt = \theta = \frac{q}{iW} \tag{4.7}$$

Ref. 2



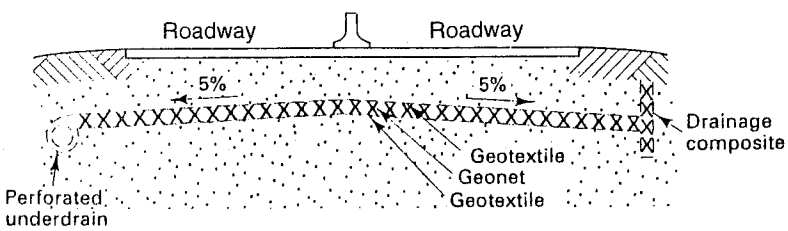
where $q$ = the volumetric flow rate,
$k$ = the coefficient of permeability,
$i$ = the hydraulic gradient,
$A$ = the flow cross-sectional area,
$\theta$ = the transmissivity,
$W$ = the width, and
$t$ = the thickness.

As with Equation 4.2, it is seen that $q/W$ and $\theta$ carry the same units and are directly related to one another by means of the hydraulic gradient $i$. At a hydraulic gradient of 1.0, they are numerically identical. At other values of hydraulic gradient they are not equal. Also note that the system must be saturated and flow must be laminar in order to use transmissivity. When in doubt, it is usually best to use flow rate per unit width.

### 4.2.2 Transportation-Related Examples

**Example:**

An area is to be retrofitted with a new pavement because of past problems with frost heave. The new scheme will have a geonet with geotextiles on both sides of it and will be placed immediately beneath the depth of frost penetration (see sketch). Based on a flow net analysis of the subsoil, the required flow rate to be conveyed will be 0.011 ft.$^3$/min.-ft. A candidate geonet has been selected, and tests performed at a normal pressure two times actual (to accommodate creep) and at a gradient of 0.05 resulted in a flow rate of 0.4 gal./min.-ft. What is the factor of safety?




**Solution:** (a) Assuming that the laboratory tests were short term and between rigid plates, the value of 0.4 gal./min.-ft (= 0.053 ft.$^3$/min.-ft.) must be reduced according to Table 4.2 as per the earlier example problems, thus

$$q_{\text{allow}} = 0.053\left[\frac{1}{1.2 \times 1.1 \times 1.3 \times 1.2}\right]$$

$$q_{\text{allow}} = 0.053\left[\frac{1}{2.06}\right]$$

$$= 0.026 \text{ ft.}^3\text{/min.-ft.}$$

Designing for Ge

(b) Now the ac

**Example:**

Determine the
has for the fol
(ML-SW) with
attempted using
to be 0.078.

**Solution:** (a) Ca
flow net, we hav

(b) Determine th

## Anchor Trench Design

- Calculations for Anchor Trench Design
- Copies of References

| | | | | |
|---|---|---|---|---|
| **Marshall FGD Residue Landfill** | **Job No.** | **046003** | | |
| **Anchor Trench Calculations** | **Comp. By** | **DHB** | **Date** | **12/19/2005** |
| | **Chkd. By** | DEB | **Date** | 12/20/05 |
| | **Sheet** | 1 | **of** | 4 |

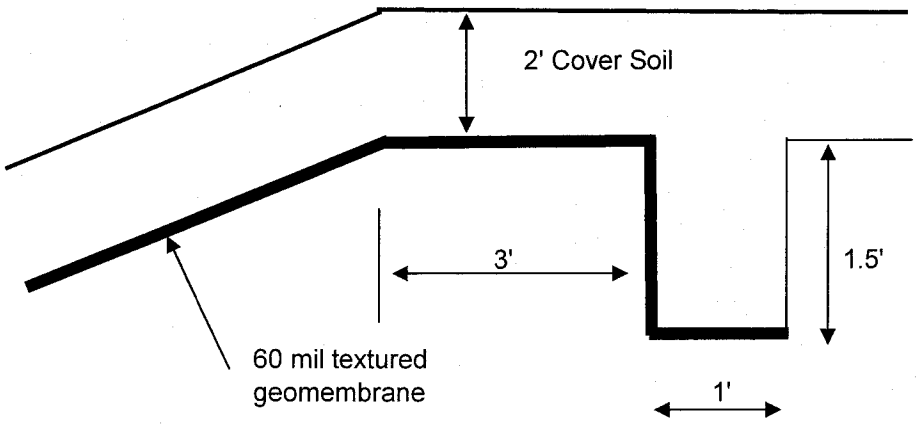
Objective: Determine Anchor Design Requirements

References: 1) *Geosynthetic Design Guidance for Hazardous Waste Landfills and Surface Impoundments*, Richardson, Example 3.17

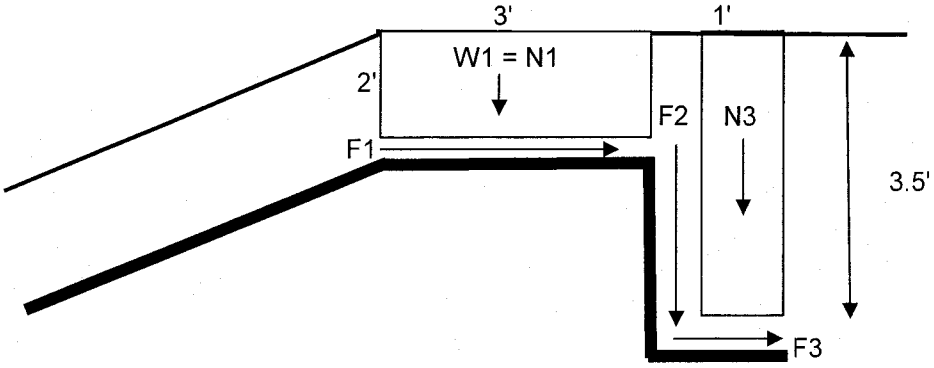
2) *Civil Engineering Reference Manual,* 4th Edition, Lindeburg

| | | | | |
|---|---|---|---|---|
| Given: | unit weight soil cover, $\gamma$ = | 100 | $lb/ft^3$ | (assumed) |
| | soil cover $\Phi$ = | 30 | degrees | |
| | textured geomembrane to GCL friction angle, $\varphi$ = (60 mil textured geomembrane) | 15 | degrees | |

**Detail of Anchor Trench**




**Resisting Forces:**



| | | | | |
|---|---|---|---|---|
| **Marshall FGD Residue Landfill** | **Job No.** | **046003** | | |
| **Anchor Trench Calculations** | **Comp. By** | **DHB** | **Date** | **12/19/2005** |
| | **Chkd. By** | **DEB** | **Date** | **12/20/05** |
| | **Sheet** | **4** | **of** | **4** |

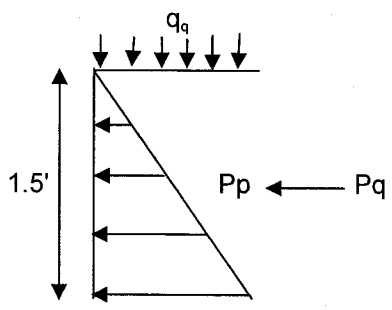
**F1: Normal Force from Soil Block (2' x 3')**

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 = 160.8 lb/ft

**F2: Friction From Normal Force of Surcharge Loaded Retaining Wall (Passive State)**




$q_q$ = 2' x unit weight of soil

$q_q$ = 200 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 = 900.0 lb/ft

$Pp = \frac{\gamma H^2}{2} \tan^2(45 + \theta/2)$

unit weight soil cover, γ = 100 lb/ft$^3$

Pp = 337.5 lb/ft

F2 = (Pq + Pp) tanφ

F2 = 331.6 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 = 94 lb/ft

| | | | | |
|---|---|---|---|---|
| **Marshall FGD Residue Landfill** | **Job No.** | **046003** | | |
| **Anchor Trench Calculations** | **Comp. By** | **DHB** | **Date** | **12/19/2005** |
| | **Chkd. By** | DEB | **Date** | 12/20/05 |
| | **Sheet** | 4 | **of** | 4 |

**Total Force Available = F1 + F2 +F3**

$F_{total}$ = 586.1 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 drawing MM6451.00-0007.001 are acceptable.

**Check Material Failure:**

Geomembrane T= 57lbs/in = 684.0lb/ft > 586.1 lb/ft

From Reference 2, the maximum stress in a 60 mil HDPE in Wide -Width tension tests (8" wide strip of geomembrane) is 2300 psi.

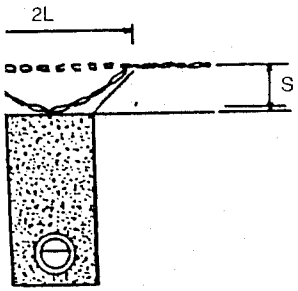
$\sigma_{allowable} = \frac{P_{allowable}}{Area}$

$P_{allowable}$ = Area x $\sigma_{allowable}$
for a 1 foot wide strip
Area = 12" x 0.060"
= 0.72 sq in

$P_{allowable}$ = (12" x 0.060") x 2300 psi
$P_{allowable}$ = 1656 lbs

Geomembrane = 1656lb/ft > 586.1 lb/ft

The force available in the anchor trench is less than the force required to overstress the geomembrane, therefore, the anchor trench would fail before the geomembrane is overstressed.




(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 $\beta$
  - EMBEDMENT LENGTH, L
  - SOIL COVER, $d_{cs}$
  - ANCHOR BURIAL, $d_{AT}$
- MATERIAL
  - SOIL FRICTION ANGLE, $\phi$
  - SOIL/FML FRICTION ANGLE, $\delta$
  - SOIL UNIT WEIGHT, $\gamma_{cs}$

(2) SOLVE FOR ANCHOR CAPACITY

HORIZONTAL ANCHOR

$$T = \frac{qL\tan\delta}{(DR)\cos\beta - \sin\beta\tan\delta}$$

"V" ANCHOR

$$T = \frac{\tan\delta\left[q\left(L - L_V + \frac{L_V}{\cos i}\right) + \frac{d_V L_V \gamma_{cs}}{2\cos i}\right]}{(DR)\cos\beta - \sin\beta\tan\delta}$$

CONCRETE ANCHOR

$$T = \frac{qL\tan\delta + (K_p - K_a)\left[.5\gamma_{cs} d_{AT}^2 + q d_{AT}\right]}{(DR)\cos\beta - \sin\beta\tan\delta}$$

ANCHOR TRENCH

$$T = \frac{qL\tan\delta + (K' + K_a)\tan\delta\left[.5\gamma_{cs} d_{AT}^2 + q d_{AT}\right]}{(DR)\cos\beta - \sin\beta\tan\delta}$$

$K' = K_p$ OR $K_o$

REF. FIGURES 3.9 & 3.10

**Design Ratio:** NOT APPLICABLE

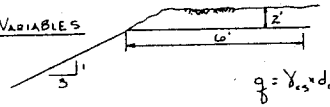
**References:**

**Example:**

GIVEN: GEOMETRY
- $\beta = 18.4°$ (3:1 SLOPE)
- $L = 6$ FT
- $d_{cs} = 2'-0$
- DR = 1.5 MINIMUM

SOIL
- $\phi = 35°$
- $\delta = 15°$
- $\gamma_{cs} = 120$ PCF

(1) DEFINE VARIABLES




$q = \gamma_{cs} \times d_{cs} = 120 \times 2 = 240$ PSF

(2) SOLVE FOR ANCHOR CAPACITIES

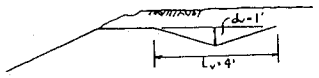
· HORIZONTAL (AS SHOWN)

$$T = \frac{240 \times 6 \times \tan 13°}{1.5\cos 18.4° - \sin 18.4° \tan 13°} = \underline{\underline{248}}\ \text{lb/FT}$$

· "V" TRENCH

- $L_V = 4'$
- $d_V = 1'$

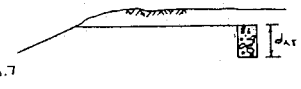
$\Big\}\ i = 26.5°$




$$T = \frac{\tan 13°\left[240\left(6.4' - \frac{4}{\cos 26.5°}\right) - \frac{1 \times 4 \times 120}{2\cos 26.5°}\right]}{1.339} = \underline{\underline{313}}\ \text{lb/FT}$$

· CONCRETE ANCHOR

- $d_{AT} = 2'-0"$
- $\phi = 35° \Rightarrow K_A = .27$ $K_P = 3.7$




$$T = \frac{240 \times 6 \times \tan 13° + (3.7 - .27)\left[0.5 \times 120 \times 2^2 + 240 \times 2\right]}{1.339} = \underline{\underline{2070}}\ \text{lb/FT}$$

· ANCHOR TRENCH

- $d_{AT} = 2'-0"$
- $\phi = 35° \Rightarrow K_o = .426$

$$[T]_{K_P} = \frac{240 \times 6 \times \tan 13° + (3.7 + .27)\tan 13°\left[.5 \times 120 \times 2^2 + 240 \times 2\right]}{1.339} = \underline{\underline{735}}\ \text{lb/FT}$$

$$[T]_{K_o} = \frac{240 \times 6 \times \tan 13° + (.426 + .27)\tan 13°\left[.5 \times 120 \times 2^2 + 240 \times 2\right]}{1.339} = 334\ \text{lb/FT}$$

**Example No.** 3.17

 Reference 2

C. COHESIONLESS SAND ($c = 0$)

The maximum slope angle for cohesionless sand is the angle of internal friction, $\phi$.

## 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.

$p_{\text{vertical}}$ 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.*

$$\boxed{k_A = \tan^2\left(45° - \frac{\phi}{2}\right) = \frac{1 - \sin\phi}{1 + \sin\phi}} \qquad 10.54$$

The general equation for horizontal *passive earth pressure* is:

$$\boxed{p_{\text{horizontal}} = p_{\text{vertical}} \tan^2\left(45° + \frac{\phi}{2}\right) + 2c\tan\left(45° + \frac{\phi}{2}\right)} \qquad 10.55$$

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

# Marshall 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 a GCL. 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

The landfill was modeled with HELP to simulate the 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:clarifier sudge placement.
- Closed Period – models the infiltration that occurs after placement of the engineered cover.

The HELP cases modeled are described below:

**Table 2-1**

| Operational Cells | Description of Conditions | Other |
|---|---|---|
| MSSFGD LF – Cell 1 - 10 feet of Waste – 60 mil HDPE+GCL | Cell 1 Open - 10 feet FGD waste placed in cell | CN = 82<br>LAI = 0<br>Model period 100 years |
| MSSFGD LF – Cell 1 - 32 feet of Waste – 60 mil HDPE+GCL | Cell 1 Open - 32 feet FGD waste placed in cell | CN = 82<br>LAI = 0<br>Model period 10 years |
| MSSFGD LF – Cell 1 - 64 feet of Waste – 60 mil HDPE+GCL | Cell 1 Open - 64 feet FGD waste placed in cell | CN = 61[2]<br>LAI = 0<br>Model period 10 years |
| MSSFGD LF – Cell 2 – 10 feet of Waste – 60 mil HDPE+GCL | Cell 2 Open - 10 feet FGD waste placed in cell | CN = 82<br>LAI = 0<br>Model period 10 years |
| MSSFGD LF – Cell 2 - 42 feet of Waste – 60 mil HDPE+GCL | Cell 1 Open - 42 feet FGD waste placed in cell | CN = 82<br>LAI = 0<br>Model period 10 years |
| MSSFGD LF – Cell 2 – 84 feet of Waste – 60 mil HDPE+GCL | Cell 2 Open – 84 feet FGD waste placed in cell | CN = 82<br>LAI = 0<br>Model period 10 years |
| MSSFGD LF – Closed W/GCL – Cell 1 and Cell 2 | Cell 1 and Cell 2 Closed with final cover in place | CN = 61[3]<br>LAI = 2<br>Model period 100 years<br>8 holes/acre in geomembranes |

[2] CN of 61 is used to simulate vegetative cover on portions of the landfill.
[3] CN of 61 is used to simulate vegetative cover on the side slopes of the landfill.

# 3.0 HELP Model Inputs

## 3.1 - Soil Properties

The soil properties that are used in the HELP analyses were developed from tests performed on soil samples collected from the footprint after excavation of the landfill and from soil stockpiled adjacent to the landfill (Reference 3). This stockpile will be the source for the operational cover soil and for the final soil cover. Additional information for soil properties came from the original site hydrogeologic report (Reference 4). Calculations showing these properties are found in Attachment 1.

## 3.2 Properties for Gypsum:Clarifier Material

Samples of the gypsum:clarifier material were sent to the geotechnical firm S&ME Inc. for testing. The results of the lab analyses on these samples are found in References 5 and 6. Copies of the relevant portions of these reports are included. The clarifier sludge and gypsum used for these tests was obtained from a power plant using similar equipment and processes.

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 Attachment 2.

## 3.3 Properties for Liner System

The landfill design calls for a liner system composed of a leachate collection and removal system, a 60 mil HDPE membrane liner, underlain by a geosynthetic clay liner. Default properties from HELP were used for these layers. A total of 8 holes per acre in the HDPE liner were used in the HELP analysis.

The hydraulic conductivity of 1.0E-6 cm/sec was used for the GCL layer. Manufacturer's literature typically reports a hydraulic conductivity of 1 E-9 cm/sec. However, the divalent ions, primarily calcium, present in the leachate can alter the performance of the clay platelets (Reference 7). Since the material in the landfill will have high concentrations of calcium, the hydraulic conductivity of the GCL was increased to account for this factor.

The spacing of the leachate collection system drain piping used in the analysis is 110 feet. This is the maximum distance between LCRS pipes. A minimum slope of 1.5% was used in these analyses.

## 3.4 Properties for Cover System

The final closure cover system 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.

As described above, the soil properties were obtained from geotechnical tests on stockpiled soil. Default properties from HELP were used for the geocomposite drainage layer and the geomembrane layers. A total of 8 holes per acre in the geomembrane were used in the HELP analysis.

## 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 0.00001" per year, occurring during the operational period for Cell1 and for Cell 2. For Cell 1, this equates to a volume of 0.528 cubic feet of annual leakage for the entire 14.8 acre footprint. For Cell 2, this equates to a volume of 0.610 cubic feet of annual leakage for the entire 17.1 acre footprint.

The average annual HELP results for leakage through the liner system, after placement of the engineered cover, are 0.00000".

Copies of the model output are provided in Attachment 4.

### 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 were determined by running each Cell for a 100 year period. The maximum daily precipitation generated by HELP for a 100 year period was 5.58 inches. The 25 year 24 hour precipitation event for the Marshall area is between 5.0 inches and 5.5 inches (Figure 8.03k, Reference 8). The maximum head on the liner from this event is 0.086 feet (for Cell1 and Cell 2).

### 4.1.3 Volume of Leachate and Precipitation

The HELP model results for the volume of leachate and precipitation collected by the LCRS is during operations of the landfill is shown in Table 4-2. The maximum peak daily flow into the leachate collection and removal system during this period is 9,158 gal/ac/day. The average annual flow into the LCRS during the operational period is 1040 gal/ac/day.

The flows into the LCRS during the closure period are presented in Table 4-3. As this table shows the flow into the LCRS peaks in year 2 after closure at 1049 gal/ac/day and decreases over the next Four years to zero flow.

## Table 4-1

### Percolation/Leakage Values From HELP Analyses

| HELP Run Title | Case Description | Number of Years for Analysis | Average Annual Percolation/Leakage Through Liner System (inches) |
|---|---|---|---|
| MSSFGD LF – Cell 1 -10 feet of Waste – 60 mil HDPE+GCL | Cell 1 Open - 10 feet FGD waste placed in cell | 100 | 0.00001 |
| MSSFGD LF – Cell 1 -32 feet of Waste – 60 mil HDPE+GCL | Cell 1 Open - 32 feet FGD waste placed in cell | 10 | 0.00001 |
| MSSFGD LF – Cell 1 - 64 feet of Waste – 60 mil HDPE+GCL | Cell 1 Open - 64 feet FGD waste placed in cell | 10 | 0.00001 |
| MSSFGD LF – Cell 2 – 10 feet of Waste – 60 mil HDPE+GCL | Cell 2 Open - 10 feet FGD waste placed in cell | 100 | 0.00001 |
| MSSFGD LF – Cell 2 - 42 feet of Waste – 60 mil HDPE+GCL | Cell 2 Open - 42 feet FGD waste placed in cell | 10 | 0.00001 |
| MSSFGD LF – Cell 2 – 84 feet of Waste – 60 mil HDPE+GCL | Cell 2 Open – 84 feet FGD waste placed in cell | 10 | 0.00001 |
| MSSFGD LF – Closed W/GCL – Cell 1 and Cell 2 | Cell 1 and Cell 2 Closed with final cover in place | 100 | 0.00000 |

## Table 4-2

### HELP Analyses - Summary of Runoff and Leachate Collection Operational Period

| HELP Run Title | Case Description | Number of Years for Analyses | CN | Peak Daily Runoff (cubic feet) | Peak Daily Drainage into LCS (cubic feet) | Peak Daily Flow into LCS (gal/ac/day) | | Average Annual Runoff (cubic feet) | Average Annual Drainage into LCS (cubic feet) | Annual Average Flow into LCS (gal/ac/day) |
|---|---|---|---|---|---|---|---|---|---|---|
| MSSFGD LF – Cell 1 -10 feet of Waste – 60 mil HDPE+GCL | Cell 1 Open - 10 feet FGD waste placed in cell | 100 | 82 | 124,534 | 18,121 | 9,158 | | 124,258 | 751,359 | 1,040 |
| MSSFGD LF – Cell 1 -32 feet of Waste – 60 mil HDPE+GCL | Cell 1 Open - 32 feet FGD waste placed in cell | 10 | 82 | 58,196 | 5,060 | 2,557 | | 87,086 | 682,064 | 944 |
| MSSFGD LF – Cell 1 - 64 feet of Waste – 60 mil HDPE+GCL | Cell 1 Open - 64 feet FGD waste placed in cell | 10 | 61 | 19,626 | 5,554 | 2,807 | | 3,279 | 686,792 | 951 |
| MSSFGD LF – Cell 2 – 10 feet of Waste – 60 mil HDPE+GCL | Cell 2 Open - 10 feet FGD waste placed in cell | 100 | 82 | 143,887 | 20,937 | 9,158 | | 143,568 | 868,124 | 1,040 |
| MSSFGD LF – Cell 2 - 42 feet of Waste – 60 mil HDPE+GCL | Cell 2 Open - 42 feet FGD waste placed in cell | 10 | 82 | 67,240 | 5,348 | 2,339 | | 100,620 | 745,753 | 894 |
| MSSFGD LF – Cell 2 – 84 feet of Waste – 60 mil HDPE+GCL | Cell 2 Open – 84 feet FGD waste placed in cell | 10 | 61 | 67,240 | 5,398 | 2,361 | | 100,620 | 610,145 | 731 |

## Table 4-3

### HELP Analyses - Summary of Leachate Collection Closed Period

| HELP Run Title | Case Description | Number of Years for Analyses | Peak Daily Runoff (cubic feet) | Average Annual Runoff (cubic feet) | Average Annual Drainage into LCS (cubic feet) | Average Annual Drainage into LCS (gal/ac/day) |
|---|---|---|---|---|---|---|
| MSSFGD LF – Closed W/GCL – Cell 1 and Cell 2 | Cell 1 and Cell 2 Closed with final cover in place | 100 | - | - | - | - |
| | Synthetic Liner (beneath waste) | Year 1 | N/A | N/A | 1,143,973 | 735 |
| | | Year 2 | N/A | N/A | 1,633,365 | 1,049 |
| | | Year 3 | N/A | N/A | 1,115,678 | 717 |
| | | Year 4 | N/A | N/A | 1,253,229 | 805 |
| | | Year 5 | N/A | N/A | 1,8069 | 12 |
| | | Years 6 - 100 | N/A | N/A | 0 | 0 |

## Table 4-4

### HELP Analyses - Summary of Stormwater Collection For Final Cover System - Closed Period

| HELP Run Title | Case Description | Number of Years for Analyses | Peak Daily Drainage Collected By Geocomposite (cubic feet) | Peak Daily Drainage Collected By Geocomposite (gal/ac/day) | Average Annual Drainage Collected By Geocomposite (cubic feet) | Average Annual Drainage into LCS (gal/ac/day) |
|---|---|---|---|---|---|---|
| MSSFGD LF – Closed W/GCL – Cell 1 and Cell 2 | Cell 1 and Cell 2 Closed with final cover in place | 100 | 218,210 | 51,166 | 1,834,819 | 1,179 |

## 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 *Soil Sampling and Testing, Duke Power - Marshall Steam Station, FGD Landfill, Terrell, North Carolina, S&ME Project No. 1356-05-571, December 5, 2005.*

4 *Hydrologic Study, FGD Scrubber Landfill, Duke Power Marshall Steam Station, Terrell, North Carolina S&ME Project 1264-02-578, August 20, 2003*

5 *Clarifier Sludge Test Results I, Duke Power FGD Planning, S&ME Project 1264-03-057, August 2005.*

6 *Clarifier Sludge/Gypsum Mix Test Results, Duke Power FGD Planning, S&ME Project 1264-03-057, November 2005.*

7 *GCL Performance &Design Reference, Design Considerations for GCL Cover Systems, CETCO Lining Technologies, TR-33, 7/03.*

8 *North Carolina Erosion and Sediment Control Planning and Design Manual.*

# Attachment 1

# Marshall FGD Residue Landfill

# HELP Analysis

# Soil Properties

Marshall FGD Landfill
HELP Inputs
Soil Properties

| | | | |
|---|---|---|---|
| Job No. | 46003 | | |
| Comp. By | WMM | Date | 12/12/2005 |
| Chkd. By | DEB | Date | 12/17/05 |
| Sheet | 1 | of | |

**Find Average Porosity Values and K's**

**Reference:** *Soil Sampling and Testing, Duke Power - Marshall Steam Station, FGD Landfill, Terrell, North Carolina, S&ME Project No. 1356-05-571, December 5, 2005.*

**Operational Cover Soil**

These results are from remolded samples of material stockpiled at the Marshall landfill.
The samples were remolded at 90% of the maximum density at optimum moisture content.

| Sample | Porosity (%) | K (cm/sec) |
|---|---|---|
| S-1 | - | - |
| S-2 | 50% | 1.66E-04 |
| S-3 | 49% | 8.17E-05 |
| S-4 | 48% | 2.91E-05 |
| S-5 | - | - |
| S-6 | 50% | 5.52E-05 |

| | Porosity (%) | K (cm/sec) |
|---|---|---|
| **Average Values** | 49% | 8.30E-05 |

**Note:**
**Since Operational Cover soil will be graded similar to Sample 6, the values for Sample 6 are used for the Operational Cover properties in HELP.**

**Soil Below Liner**

These results are from undisturbed samples collected at depths of 6" to 18" below the existing excavation in Cell 1.

| Sample | Porosity (%) | K (cm/sec) | |
|---|---|---|---|
| B-1 | 50% | 3.57E-04 | 50 |
| B-2 | 55% | 3.01E-04 | 55 |
| B-3 | 56% | 4.56E-04 | 56 |
| B-4 | 52% | 3.74E-04 | 52 |

| | Porosity (%) | K (cm/sec) |
|---|---|---|
| **Average Values** | 53% | 3.72E-04 |

**Marshall Steam Station, Catawba County**
**FGD Landfill - Soil Properties for HELP Model**
**Soil Properties**
**Soil Beneath Liner**

| | | | |
|---|---|---|---|
| **Job No.** | **46003** | | |
| **Comp. By** | **WMM** | **Date** | **12/12/2005** |
| **Chkd. By** | DEB | **Date** | 12/17/05 |
| **Sheet** | 2 | **of** | |

Reference: *Soil Sampling and Testing, Duke Power - Marshall Steam Station, FGD Landfill, Terrell, North Carolina, S&ME Project No. 1356-05-571, December 5, 2005.*

**Find % of Sand , Silt, and Clay (USDA) for Calculation of Wilting Point and Field Capacity For Soil Beneath the Liner**

| | Sample ID | B-1C | | | B-1C | | | BC-1C | | | B-1D |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | Depth | 0"-6" | | | 6"-12" | | | 12"-18" | | | 18"-24" |
| Screen | Dia. (mm) | % Finer | Screen | Dia. (mm) | % Finer | Screen | Dia. (mm) | % Finer | Screen | Dia. (mm) | % Finer |
| #10 | 2.00 | 99.9 | #10 | 2.00 | 99.9 | #10 | 2.00 | 99.5 | #10 | 2.00 | 95.6 |
| #200 | 0.075 | 62.1 | #200 | 0.075 | 72.5 | #200 | 0.075 | 60.8 | #200 | 0.075 | 41.4 |
| | 0.05 | 58.6 | | 0.05 | 72.0 | | 0.05 | 59.7 | | 0.05 | 54.4 |
| | 0.0437 | 57.7 | | 0.0401 | 71.8 | | 0.0426 | 59.4 | | 0.0437 | 57.7 |
| | 0.0029 | 38.7 | | 0.0029 | 43 | | 0.003 | 32.8 | | 0.003 | 25.8 |
| | 0.002 | 36.1 | | 0.002 | 37.5 | | 0.002 | 29.4 | | 0.002 | 23.6 |
| | 0.0012 | 33.7 | | 0.0012 | 32.6 | | 0.0012 | 26.7 | | 0.0013 | 22.1 |
| % Sand (USDA) | 0.05 - 2.0 mm | 41.3 | % Sand (USDA) | | 27.9 | % Sand (USDA) | | 39.8 | % Sand (USDA) | | 41.2 |
| % Clay (USDA) | < 0.002 mm | 36.1 | % Clay (USDA) | | 37.5 | % Clay (USDA) | | 29.4 | % Clay (USDA) | | 23.6 |

**Marshall Steam Station, Catawba County**
**FGD Landfill - Soil Properties for HELP Model**
**Soil Properties**
**Soil Beneath Liner**

| | | | | | |
|---|---|---|---|---|---|
| **Job No.** | **46003** | **Date** | **12/12/2005** |
| **Comp. By** | **WMM** | **Date** | 12/12/05 |
| **Chkd. By** | DCR | | |
| **Sheet** | 3 | **of** | |

| | Sample ID Depth | B-2C 0"-6" | | Dia. (mm) | B-2C 6"-12" | | | B-2C 12"-18" | | | B-2C 18"-24" |
|---|---|---|---|---|---|---|---|---|---|---|---|
| Screen | Dia. (mm) | % Finer | Screen | | % Finer | Screen | Dia. (mm) | % Finer | Screen | Dia. (mm) | % Finer |
| #10 | 2.00 | 99.7 | #10 | 2.00 | 100 | #10 | 2.00 | 99.8 | #10 | 2.00 | 100 |
| #200 | 0.075 | 47.6 | #200 | 0.075 | 46.8 | #200 | 0.075 | 31.5 | #200 | 0.075 | 38.5 |
| | 0.05 | 46.6 | | 0.05 | 46.0 | | 0.05 | 30.2 | | 0.05 | 37.4 |
| | 0.0444 | 46.4 | | 0.0441 | 45.8 | | 0.047 | 30.0 | | 0.046 | 37.2 |
| | 0.0031 | 10.7 | | 0.0031 | 12.7 | | 0.0032 | 6.4 | | 0.0032 | 6.4 |
| | 0.002 | 9.3 | | 0.002 | 11.0 | | 0.002 | 5.8 | | 0.002 | 5.1 |
| | 0.0013 | 8.4 | | 0.0013 | 9.9 | | 0.0013 | 5.4 | | 0.0013 | 4.4 |
| % Sand (USDA) | 0.05 - 2.0 mm | 53.1 | % Sand (USDA) | | 54.0 | % Sand (USDA) | | 69.6 | % Sand (USDA) | | 62.6 |
| % Clay (USDA) | < 0.002 mm | 9.3 | % Clay (USDA) | | 11.0 | % Clay (USDA) | | 5.8 | % Clay (USDA) | | 5.1 |

**Marshall Steam Station, Catawba County**
**FGD Landfill - Soil Properties for HELP Model**
**Soil Properties**
**Soil Beneath Liner**

| | | | | | |
|---|---|---|---|---|---|
| **Job No.** | **46003** | **Date** | **12/12/2005** |
| **Comp. By** | **WMM** | **Date** | 12/17/05 |
| **Chkd. By** | DEB | **of** | |
| **Sheet** | 4 | | |

| | Sample ID Depth | B-3A 0"-6" | | Dia. (mm) | B-3A 6"-12" | | | B-3A 12"-18" | | | B-3A 18"-24" |
|---|---|---|---|---|---|---|---|---|---|---|---|
| Screen | Dia. (mm) | % Finer | Screen | | % Finer | Screen | Dia. (mm) | % Finer | Screen | Dia. (mm) | % Finer |
| #10 | 2.00 | 98 | #10 | 2.00 | 99.2 | #10 | 2.00 | 98 | #10 | 2.00 | 99.9 |
| #200 | 0.075 | 56.4 | #200 | 0.075 | 49.2 | #200 | 0.075 | 46.4 | #200 | 0.075 | 59.5 |
| | 0.05 | 55.4 | | 0.05 | 49.1 | | 0.05 | 45.2 | | 0.05 | 58.9 |
| | 0.0425 | 55.1 | | 0.0442 | 49.1 | | 0.0449 | 44.9 | | 0.0428 | 58.7 |
| | 0.0031 | 15.5 | | 0.0032 | 8.0 | | 0.0032 | 8.6 | | 0.0032 | 8.0 |
| | 0.002 | 12.6 | | 0.002 | 5.8 | | 0.002 | 6.6 | | 0.002 | 6.5 |
| | 0.0013 | 10.7 | | 0.0013 | 4.5 | | 0.0013 | 5.5 | | 0.0013 | 5.6 |
| % Sand (USDA) | 0.05 - 2.0 mm | 42.6 | % Sand (USDA) | | 50.1 | % Sand (USDA) | | 52.8 | % Sand (USDA) | | 41.0 |
| % Clay (USDA) | < 0.002 mm | 12.6 | % Clay (USDA) | | 5.8 | % Clay (USDA) | | 6.6 | % Clay (USDA) | | 6.5 |

**Marshall[illegible]am Station, Catawba County**
**FGD Landfill - Soil Properties for HELP Model**
**Soil Properties**
**Soil Beneath Liner**

| | | | | | |
|---|---|---|---|---|---|
| **Job No.** | **46003** | | **Date** | **12/12/2005** | |
| **Comp. By** | **WMM** | | **Date** | 12/17/05 | |
| **Chkd. By** | DCB | | **Date** | | |
| **Sheet** | 5 | | **of** | | |

| | Sample ID Depth | B-4A 0"-6" | | | B-4A 6"-12" | | | B-4A 12"-18" | | | B-4A 18"-24" |
|---|---|---|---|---|---|---|---|---|---|---|---|
| Screen | Dia. (mm) | % Finer | Screen | Dia. (mm) | % Finer | Screen | Dia. (mm) | % Finer | Screen | Dia. (mm) | % Finer |
| #10 | 2.00 | 94.5 | #10 | 2.00 | 94.6 | #10 | 2.00 | 98.7 | #10 | 2.00 | 94.6 |
| #200 | 0.075 | 42.5 | #200 | 0.075 | 42.8 | #200 | 0.075 | 40.7 | #200 | 0.075 | 52.8 |
| | 0.05 | 41.9 | | 0.05 | 41.5 | | 0.05 | 40.4 | | 0.05 | 51.5 |
| | 0.0447 | 41.8 | | 0.0452 | 41.3 | | 0.0449 | 40.3 | | 0.0436 | 51.2 |
| | 0.0032 | 5.3 | | 0.0032 | 6.3 | | 0.0032 | 5.6 | | 0.0032 | 5.6 |
| | 0.002 | 4.0 | | 0.002 | 3.8 | | 0.002 | 4.5 | | 0.002 | 3.5 |
| | 0.0013 | 3.2 | | 0.0013 | 2.3 | | 0.0013 | 3.8 | | 0.0013 | 2.3 |
| % Sand (USDA) | 0.05 - 2.0 mm | 52.6 | % Sand (USDA) | | 53.1 | % Sand (USDA) | | 58.3 | % Sand (USDA) | | 43.1 |
| % Clay (USDA) | < 0.002 mm | 4.0 | % Clay (USDA) | | 3.8 | % Clay (USDA) | | 4.5 | % Clay (USDA) | | 3.5 |

Marshall Steam Station, Catawba County
FGD Landfill - Soil Properties for HELP Model
Soil Properties
Soil Beneath Liner

| | | | |
|---|---|---|---|
| Job No. | 46003 | | |
| Comp. By | WMM | Date | 12/12/2005 |
| Chkd. By | DCB | Date | 12/12/05 |
| Sheet | 6 | of | 14 |

Reference: *Soil Sampling and Testing, Duke Power - Marshall Steam Station, FGD Landfill, Terrell, North Carolina, S&ME Project No. 1356-05-571, December 5, 2005.*

**Summary of Values for % Sand and % Clay (USDA)**
**To be Used in Wilting Point and Field Capacity Calculations**

| | | B-1C 0"-6" | B-1C 6"-12" | BC-1C 12"-18" | B-1D 18"-24" | B-2C 6"-12" | B-2C 6"-12" | B-2C 12"-18" | B-2C 18"-24" |
|---|---|---|---|---|---|---|---|---|---|
| % Sand (USDA) | 0.05 - 2.0 mm | 41.3 | 27.9 | 39.8 | 41.2 | 53.1 | 54.0 | 69.6 | 62.6 |
| % Clay (USDA) | < 0.002 mm | 36.1 | 37.5 | 29.4 | 23.6 | 9.3 | 11.0 | 5.8 | 5.1 |

| | | B-3A 0"-6" | B-3A 6"-12" | B-3A 12"-18" | B-3A 18"-24" | B-4A 0"-6" | B-4A 6"-12" | B-4A 12"-18" | B-4A 18"-24" |
|---|---|---|---|---|---|---|---|---|---|
| % Sand (USDA) | 0.05 - 2.0 mm | 42.6 | 50.1 | 52.8 | 41.0 | 52.6 | 53.1 | 52.8 | 41.0 |
| % Clay (USDA) | < 0.002 mm | 12.6 | 5.8 | 6.6 | 6.5 | 4.0 | 3.8 | 6.6 | 6.5 |

| | | |
|---|---|---|
| Average Values All Samples | % Sand (USDA) | 48.5% |
| | % Clay (USDA) | 13.1% |

Marshall FGD Landfill
HELP Inputs
Soil Properties

| | | | |
|---|---|---|---|
| Job No. | 46003 | | |
| Comp. By | WMM | Date | 12/12/2005 |
| Chkd. By | DEB | Date | 12/17/05 |
| Sheet | 7 | of | 14 |

**Find % of Sand , Silt, and Clay (USDA) for Calculation of Wilting Point and Field Capacity**
**Operational Cover Material**

| Sample ID | % Passing 2.0 mm | 0.05 mm | 0.002 mm | USDA Classification % Sand | %Silt | % Clay |
|---|---|---|---|---|---|---|
| S-1 | 99.3% | 44.6% | 13.6% | 55% | 31% | 14% |
| S-2 | 99.5% | 42.6% | 4.0% | 57% | 39% | 4% |
| S-3 | 99.7% | 70.0% | 41.8% | 30% | 28% | 42% |
| S-4 | 99.6% | 63.0% | 24.0% | 37% | 39% | 24% |
| S-5 | 99.7% | 55.7% | 18.7% | 44% | 37% | 19% |
| S-6 | 99.7% | 58.6% | 10.4% | 41% | 48% | 10% |

| | % Sand | %Silt | % Clay |
|---|---|---|---|
| Average | 43.8% | 37% | 18.8% |

**Note:**
**Since Operational Cover soil will be graded similar to Sample 6, the values for Sample 6 are used to calculate the properties used in HELP for the Operational Cover properties.**

**Modified Saturated Hydraulic Conductivity in Upper Half of Evaporative Zone**

Since the soil properties are not default HELP properties,
the saturated 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 (Average of soil samples from Stockpile)

(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 = 8.3E-05 cm/sec

**Ksv = 2.5E-04 cm/sec**

Marshall FGD Landfill
HELP Inputs
Soil Properties

Calculate Field Capacity and Wilting Point

| | | | |
|---|---|---|---|
| Job No. | 46003 | | |
| Comp. By | WMM | Date | 12/12/2005 |
| Chkd. By | DEB | Date | 12/17/05 |
| Sheet | 9 | of | 14 |

The equations used for these calculations are found in the HELP Users Manual, Appendix A.

**Operational Cover Soil (See Note 1)**

USDA Classification

| | % Sand | % Clay | Porosity% |
|---|---|---|---|
| Average | 41.1% | 10.4% | 0.50 |

| Field Capacity (HELP Eqn A-3) | Wilting Point (HELP Eqn A-4) |
|---|---|
| 0.272 | 0.090 |

**Soil Above Synthetic Cover (Note 2)**

USDA Classification

| | %Sand | %Clay | Porosity% |
|---|---|---|---|
| Average | 43.8% | 18.8% | 0.49 |

| Field Capacity (HELP Eqn A-3) | Wilting Point (HELP Eqn A-4) |
|---|---|
| 0.289 | 0.126 |

**Soil Below Liner**

USDA Classification

| | % Sand | % Clay | Porosity% |
|---|---|---|---|
| Average | 48.5% | 13.1% | 0.53 |

| Field Capacity (HELP Eqn A-3) | Wilting Point (HELP Eqn A-4) |
|---|---|
| 0.221 | 0.101 |

Note 1:
Since Operational Cover soil will be graded similar to Sample 6, the values for Sample 6 are used for the Operational Cover properties.

Note 2:
The average values for the soil samples collected form the stockpile are used to calculate properties for the soil above the synthetic cover.

Marshall FGD Landfill
HELP Inputs
Soil Properties

| | | | |
|---|---|---|---|
| Job No. | 46003 | | |
| Comp. By | WMM | Date | 12/12/2005 |
| Chkd. By | DEB | Date | 12/17/05 |
| Sheet | 10 | of | 14 |

**Calculate Moisture Content for Soil Properties**

**Samples from Stockpile**
These results are from remolded samples of material stockpiled at the Marshall landfill.
Moisture contents are OMC values.

| Sample | Maximum Dry Density (lb/ft^3) | Moisture Content (%) | Porosity (%) | SG |
|---|---|---|---|---|
| S-1 | 94.7 | 24.5% | - | - |
| S-2 | 93.4 | 24.4% | 0.5 | 2.65 |
| S-3 | 93.8 | 25.8% | 0.49 | 2.65 |
| S-4 | 95.8 | 22.6% | 0.48 | 2.65 |
| S-5 | 93.6 | 24.0% | - | - |
| S-6 | 91.0 | 25.2% | 0.5 | 2.65 |
| **Average Values** | 93.7 | 24.4% | 0.5 | 2.65 |

**Marshall FGD Landfill**
**HELP Inputs**
**Soil Properties**

| | | | |
|---|---|---|---|
| **Job No.** | **46003** | | |
| **Comp. By** | **WMM** | **Date** | **12/12/2005** |
| **Chkd. By** | DEB | **Date** | 12/17/05 |
| **Sheet** | 11 | **of** | 14 |

**Calculate Moisture Content for HELP Soil Properties**

**HELP uses moisture content as an input paramter for the soil layers.**
**This term is defined as:** $\frac{\text{volume of water}}{\text{(volume of water, soil, and voids)}}$

**This definition differs from the method normally used in express moisture content in soil samples where the moisture content , w = weight of water/weight of soil**

**References:**

1) *Duke Power Marshall Steam Station, Catawba County, NC, Flue Gas Disposal(FGD) Residue landfill Permit Application Suitability Information, Voume 2, Hydrogeologic Study S&ME Project No. 1264-02-578.*

2) *Soil Sampling and Testing, Duke Power - Marshall Steam Station, FGD Landfill, Terrell, North Carolina, S&ME Project No. 1356-05-571, December 5, 2005.*

**Calculate the moisture content in terms of volume:**

**Material: Soil Below Liner** (Known Values are estimated using Tables 4 and 6 in Reference 1)

**Knowns:**

| | | | |
|---|---|---|---|
| Unit Weight = | 95.0 | lb/ft^3 | (estimate) |
| moisture content = (wt water/wt soil) | 30.0% | | (estimate) |
| SG = | 2.7 | | (estimate) |
| n = | 0.50 | | (estimate) |

**Find volume of Water in Sample:**

Volume of Solids = V(1 - n)
Volume of Solids = 0.5 ft^3

Weight of Solids (Ws) = $\frac{Wt}{(1 + w)}$
Weight of Solids = 73.1 lb/ft^3

Weight of Water = wWs
Weight of Water = 21.9 lb/ft^3

Volume of Water = $\frac{Ww}{\gamma_w}$

**Volume of Water = 0.35 ft^3**

**Marshall FGD Landfill**
**HELP Inputs**
**Soil Properties**

| | | | |
|---|---|---|---|
| **Job No.** | **46003** | | |
| **Comp. By** | **WMM** | **Date** | **12/12/2005** |
| **Chkd. By** | DEB | **Date** | 12/17/05 |
| **Sheet** | 12 | **of** | 19 |

**Calculate Moisture Content for HELP Soil Properties**

**Material: Operational Cover (Sample 6)** Reference 2

**Knowns:**

Unit Weight = 91.0 lb/ft^3

moisture content = 25.2%
(wt water/wt soil)

SG = 2.65

n = 0.50

**Find volume of Water in Sample:**

Volume of Solids = V(1 - n)
Volume of Solids = 0.5 ft^3

Weight of Solids (Ws) = $\frac{Wt}{(1 + w)}$
Weight of Solids = 72.7 lb/ft^3

Weight of Water = wWs
Weight of Water = 18.3 lb/ft^3

Volume of Water = $\frac{Ww}{\gamma_w}$
**Volume of Water = 0.29 ft^3**

**Marshall FGD Landfill**
**HELP Inputs**
**Soil Properties**

| | | | |
|---|---|---|---|
| **Job No.** | **46003** | | |
| **Comp. By** | **WMM** | **Date** | **12/12/2005** |
| **Chkd. By** | DEB | **Date** | 12/17/05 |
| **Sheet** | 13 | **of** | 14 |

**Calculate Moisture Content for HELP Soil Properties**

**Material: Soil Above Synthetic Cover**

**Knowns:**

Unit Weight = 93.7 lb/ft^3

moisture content = 24.4%
(wt water/wt soil)

SG = 2.65

n = 0.49

Reference 2
Knowns are average values from Stockpile samples

**Find volume of Water in Sample:**

Volume of Solids = V(1 - n)
Volume of Solids = 0.5075 ft^3

Weight of Solids (Ws) = $\frac{Wt}{(1 + w)}$
Weight of Solids = 75.3 lb/ft^3

Weight of Water = wWs
Weight of Water = 18.4 lb/ft^3

Volume of Water = $\frac{Ww}{\gamma_w}$
**Volume of Water = 0.29 ft^3**

Mars[illegible] FGD Landfill
HELP Inputs
Soil Properties

Input Values For HELP

| | | | | |
|---|---|---|---|---|
| Job No. | 46003 | | | |
| Comp. By | WMM | Date | 12/12/2005 | |
| Chkd. By | PEB | Date | 12/12/05 | |
| Sheet | 14 | of | 14 | |

| Operational Soil Properties | Porosity % | Field Capacity (HELP Eqn A-3) | Wilting Point (HELP Eqn A-4) | Moisture Content (vol/vol) | Hydraulic Conductivity, k (cm/sec) |
|---|---|---|---|---|---|
| Values Used in HELP | 0.50 | 0.272 | 0.090 | 0.29 | 5.52E-05 |

| Soil Below Liner | Porosity % | Field Capacity (HELP Eqn A-3) | Wilting Point (HELP Eqn A-4) | Moisture Content (vol/vol) | Hydraulic Conductivity, k (cm/sec) |
|---|---|---|---|---|---|
| Values Used in HELP | 0.53 | 0.221 | 0.101 | 0.35 | 3.72E-04 |

| Soil Above Synthetic Cover: Lower Half of Evaporative Zone (Note 1) | Porosity % | Field Capacity (HELP Eqn A-3) | Wilting Point (HELP Eqn A-4) | Moisture Content (vol/vol) | Hydraulic Conductivity, k (cm/sec) |
|---|---|---|---|---|---|
| Values Used in HELP | 0.49 | 0.289 | 0.126 | 0.29 | 8.30E-05 |

| Soil Above Synthetic Cover: Upper Half of Evaporative Zone | Porosity % | Field Capacity (HELP Eqn A-3) | Wilting Point (HELP Eqn A-4) | Moisture Content (vol/vol) | Hydraulic Conductivity, k (cm/sec) |
|---|---|---|---|---|---|
| Values Used in HELP | 0.49 | 0.289 | 0.126 | 0.29 | 2.49E-04 |

Note 1: These properties are also used for the 18" of soil located beneath the synthetic cover and on top of the waste.

# Attachment 2

## Marshall FGD Residue Landfill

## HELP Analysis

## Gypsum:Clarifier Properties

Marshall FGD Landfill
HELP Inputs
FGD Residue Properties for HELP Model

| | | | |
|---|---|---|---|
| Job No. | 46003 | | |
| Comp. By | WMM | Date | 12/12/2005 |
| Chkd. By | DEB | Date | 12/17/05 |
| Sheet | 1 | of | 3 |

**Calculate Moisture Content for HELP Properties**

**HELP uses moisture content as an input paramter for the soil layers.**
**This term is defined as:** $\frac{\text{volume of water}}{\text{(volume of water, soil, and voids)}}$

**This definition differs from the method normally used in express moisture content in soil samples where the moisture content , w = weight of water/weight of soil**

**References:** 1) *Clarifier Sludge Test Results I, Duke Power FGD Planning, S&ME Project 1264-03-057, August 2005*

2) *Clarifier Sludge/Gypsum Mix Test Results, Duke Power FGD Planning, S&ME Project 1264-03-057, November 2005*

**Calculate the moisture content in terms of volume:**

**Material:** **1:1 Mix of Clarifier Sludge and Gypsum**

**Knowns:**

| | | | | |
|---|---|---|---|---|
| Unit Weight = | 64.3 | lb/ft^3 | (estimate) | MDD from proctor is 71.4 lb/ft^3 |
| moisture content = (wt water/wt soil) | 31.9% | | (estimate) | The unit wt. used is 90% of MDD. |
| SG = | 2.33 | | (estimate) | |
| n = | 0.50 | | (estimate) | |

**Find volume of Water in Sample:**

Volume of Solids = V(1 - n)
Volume of Solids = 0.5 ft^3

Weight of Solids (Ws) = $\frac{Wt}{(1 + w)}$
Weight of Solids = 48.7 lb/ft^3

Weight of Water = wWs
Weight of Water = 15.5 lb/ft^3

Volume of Water = $\frac{Ww}{\gamma_w}$

**Volume of Water = 0.25 ft^3**

**Marshall Steam Station, Catatwba County**
**FGD Landfill**
**FGD Residue Properties for HELP Model**

| | | | |
|---|---|---|---|
| **Job No.** | **046119** | | |
| **Comp. By** | **WMM** | **Date** | **12/12/2005** |
| **Chkd. By** | DEB | **Date** | 12/17/05 |
| **Sheet** | 2 | **of** | 3 |

**This page presents the information from lab analyses on the FGD samples and calculates values used in the HELP analyses.**

**References:**

1) *Clarifier Sludge Test Results I, Duke Power FGD Planning, S&ME Project 1264-03-057, August 2005*

2) *Clarifier Sludge/Gypsum Mix Test Results, Duke Power FGD Planning, S&ME Project 1264-03-057, November 2005*

**FGD Clarifier Sludge Laboratory Test Results** **1:1 Mix of Clarifier Sludge and Gypsum**

**From Reference 1:**

| Screen | Dia. (mm) | % Finer | | |
|---|---|---|---|---|
| #10 | 2.00 | 100 | | |
| #200 | 0.075 | 95.8 | | |
| | 0.05 | 86.7 | | calculated |
| | 0.0445 | 84.7 | | |
| | 0.0036 | 7.4 | | |
| | 0.002 | 7.2 | | calculated |
| | 0.0015 | 7.1 | | |
| Per Cent Sand (0.05 - 2.0 mm) (USDA) | | 13.3 | % | |
| Per Cent Clay (<0.002 mm) (USDA) | | 7.2 | % | |

Marshall Steam Station, Catatwba County
FGD Landfill
FGD Residue Properties for HELP Model

| | | | |
|---|---|---|---|
| Job No. | 046119 | | |
| Comp. By | WMM | Date | 12/12/2005 |
| Chkd. By | DEB | Date | 12/17/05 |
| Sheet | 3 of 3 | | |

From Reference 2:

| Remolded Sample Conditions | Porosity | Hydraulic Conductivity, k (cm/sec) |
|---|---|---|
| MDD at OMC | 0.56 | 1.76E-04 |

| | Porosity | Field Capacity (HELP Eqn A-3) | Wilting Point (HELP Eqn A-4) | Moisture Content (vol/vol) | Hydraulic Conductivity, k (cm/sec) |
|---|---|---|---|---|---|
| **Values Used in HELP** | **0.56** | **0.266** | **0.090** | **0.25** | **1.76E-04** |

# Attachment 3

## Marshall FGD Residue Landfill

## HELP Analysis

## Selected References

# Soil Sampling and Testing
# Duke Power – Marshall Steam Station
# FGD Landfill

Terrell, North Carolina
S&ME Project No. 1356-05-571

Reference 3

Prepared for:

Duke Power®
A Duke Energy Company

526 South Church Street
Charlotte, North Carolina 28202

Prepared by:

S&ME, Inc.
9751 Southern Pine Boulevard
Charlotte, North Carolina 28273
(704) 523-4726

December 5, 2005

S&ME

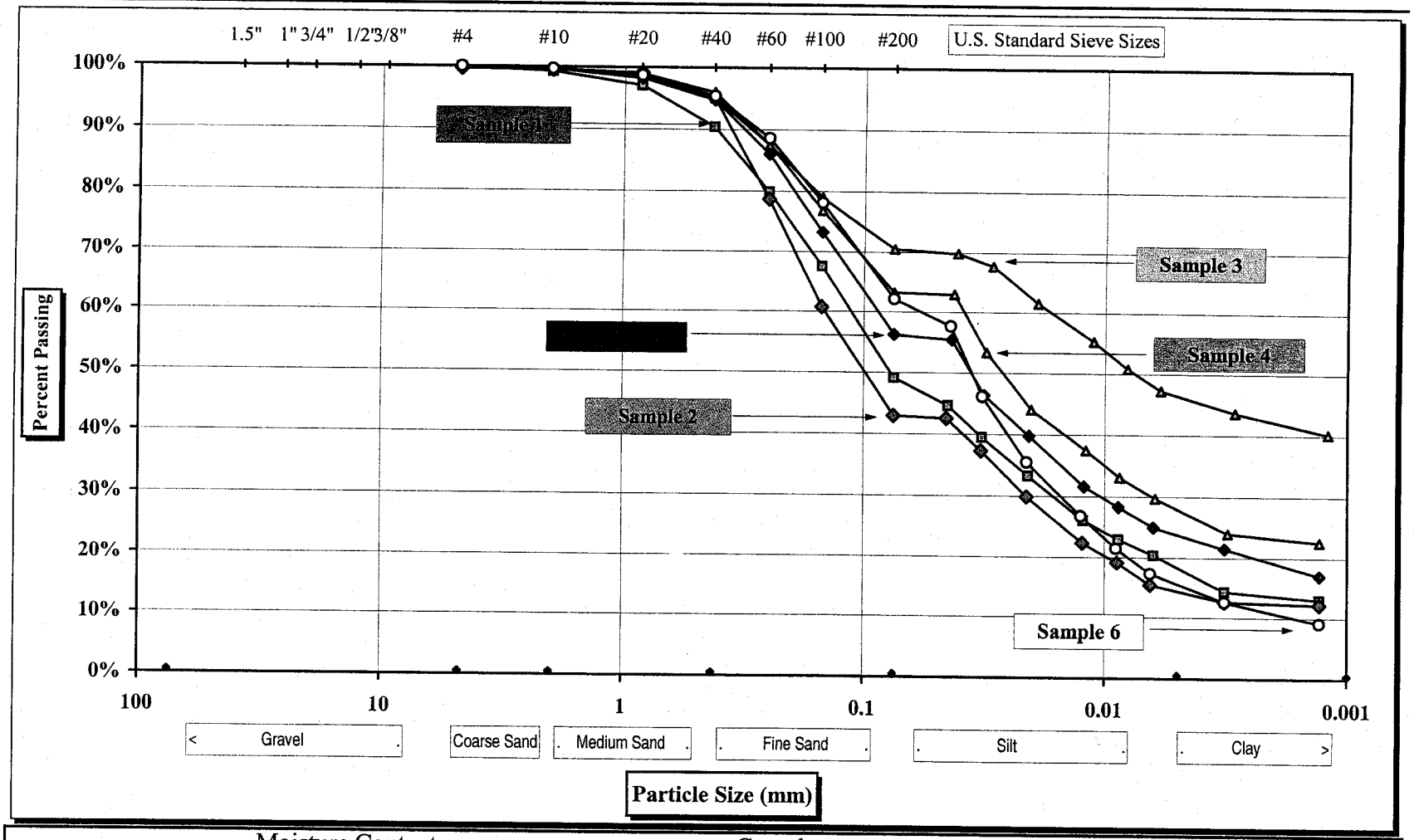
# Plate 1 - Grain Size Distribution Comparison

ASTM D 422

S&ME

| | | | |
|---|---|---|---|
| S&ME Project #: | **1356-05-571** | Report Date: | Varies |
| Project Name: | **Duke Energy Marshall Steam Station Stockpile Sampling** | Test Date(s): | Varies |

| | | | | | |
|---|---|---|---|---|---|
| Boring #: | | Sample #: | | Sample Date: | |
| Location: | As Shown | Offset: | | Elevation: | |
| Sample Description: | Soil Stockpile Material | | | | |




| | | | | | |
|---|---|---|---|---|---|
| Moisture Content: | | Gravel: | | Liquid Limit: | |
| Silt & Clay (% Passing #200): | | Sand: | | Plastic Limit: | |
| Maximum Particle Size: | | Silt: | | Plastic Index: | |
| Specific Gravity: | | Clay & Colloids: | | | |

Ref. 3

**Table 1**

**Summary of Laboratory Test Data**

**Duke Power - MSS Undisturbed Samples**

**S&ME Project No. 1356-05-571**

| Boring No. | Sample Depth (Ft) | Sample Type * | USCS Class. | Soil Description | % Finer No. 200 | Atterberg Limits | | Proctor Data | | Unit Weight (pcf) | Remolding Conditions | | Hydraulic Conductivity k (cm/sec) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | | LL | PI | Max Dry Density (pcf) | Optimum Moisture Content (%) | | Dry Density (pcf) | Moisture Content (%) | |
| B-1 | 0-6" | UD | MH | C/F Sandy Clayey Silt | 70.1 | 53 | 18 | | | | | | |
| | 6-12" | UD | MH | C/F Sandy Clayey Silt | 72.5 | 53 | 22 | | | 104.6 | | | 3.57E-04 |
| | 12-18" | UD | ML | Clayey C/F Sandy Silt | 60.8 | 47 | 19 | | | | | | |
| | 18-24" | UD | SM | Clayey Silty C/F Sand | 41.4 | 39 | 13 | | | | | | |
| B-2 | 0-6" | UD | SM | Sl. Clayey Silty C/F Sand | 47.6 | NP | NP | | | 90.5 | | | 3.01E-04 |
| | 6-12" | UD | SM | Clayey Silty Fine Sand | 46.8 | NP | NP | | | | | | |
| | 12-18" | UD | SM | Sl. Clayey Silty C/F Sand | 31.5 | NP | NP | | | | | | |
| | 18-24" | UD | SM | Sl. Clayey Silty M/F Sand | 38.5 | NP | NP | | | | | | |
| B-3 | 0-6" | UD | ML | Clayey C/F Sandy Silt | 56.4 | 46 | 8 | | | | | | |
| | 6-12" | UD | ML | Sl. Clayey C/F Sandy Silt | 49.3 | 49 | 5 | | | | | | |
| | 12-18" | UD | SM | Clayey Silty C/F Sand | 46.4 | 50 | 5 | | | | | | |
| | 18-24" | UD | MH | C/F Sandy Clayey Silt | 49.3 | 54 | 5 | | | 90.5 | | | 4.56E-04 |
| B-4 | 0-6" | UD | SM | Sl. Clayey Silty C/F Sand | 42.5 | 49 | 3 | | | | | | |
| | 6-12" | UD | SM | Sl. Clayey Silty C/F Sand | 42.8 | 49 | 0 | | | | | | |
| | 12-18" | UD | SM | Clayey Silty C/F Sand | 40.7 | NP | NP | | | 99.7 | | | 3.74E-04 |
| | 18-24" | UD | ML | Sl. Clayey C/F Sandy Silt | 52.8 | 47 | 2 | | | | | | |
| Bulk | - - - | Bag | SM | Clayey Silty M/F Sand | 48.3 | 54 | 9 | 92.0 | 24.5 | | 90.2 | 24.5 | 2.39E-05 |
| | | | | | | | | | | | 91.4 | 24.8 | 2.30E-05 |

Mean of Samples in Footprint (B-1 through B-4) is $3.7 \times 10^{-4}$ cm/sec

**Notes:**
SS = Split Spoon Sample (ASTM D-1586)
UD = Undisturbed Sample (ASTM D-1587)
BAG = Bulk Sample
NP = Non Plastic

Rev 3

**CLARIFIER SLUDGE TEST RESULTS I**
**DUKE POWER FGD PLANNING**
**S&ME Project 1264-03-057**

Prepared For:

Duke Power
*A Duke Energy Company*

**Charlotte, North Carolina**

Prepared By:

S&ME
ENVIRONMENTAL SERVICES
ENGINEERING • TESTING

**Spartanburg, South Carolina**

August 2005

Reference 5

# SUMMARY OF LABORATORY TEST DATA

| Boring No. | Sample Depth (ft) | Sample Number * | USCS Classification | Natural Moisture Content (%) | % Finer No. 200 | Atterberg Limits | | | Proctor Data | | Specific Gravity | pH | Molding Conditions | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | | LL | PL | PI | Max. Dry Density (pcf) | Moisture Content (%) | | | Dry Density (pcf) | Moisture Content (%) |
| Big Bend Clarifier Sludge | Bulk | | | 116.7 | 97.3 | NP | NP | NP | | | 2.32 | | | |
| Big Bend Clarifier/Marshall Soil 1:1 | Bulk | | | | 45.7 | 35 | 29 | 6 | | | 2.50 | | | |
| Big Bend Clarifier/Marshall Soil 1:2 | Bulk | | | | 41.1 | 32 | 31 | 1 | | | 2.66 | | | |
| Big BendClarifier/Marshall Soil 1:3 | Bulk | | | | 35.2 | NP | NP | NP | | | 2.67 | | | |
| Big Bend Gypsum | Bulk | | | 12.8 | 98.2 | NP | NP | NP | | | 2.37 | | | |
| Big Bend Clarifier/Big Bend Gypsum 1:1 | Bulk | | | | 95.8 | NP | NP | NP | | | 2.33 | | | |
| Big Bend Clarifier/Big Bend Gypsum 1:2 | Bulk | | | | 93.9 | NP | NP | NP | | | 2.33 | | | |
| Big Bend Clarifier/Marshall Fly Ash 1:1 | Bulk | | | | 85.8 | NP | NP | NP | | | 2.08 | | | |

Note:

Ref. 5

* SS = Split Spoon Sample (ASTM D-1586)
UD= Undisturbed Sample (ASTM D-1587)
BAG= Bulk Sample

Job Name: FGD Clarifier Sludge Testing II
Job Location:
Job Number: 1264-03-057

## GRAIN SIZE DISTRIBUTION TEST DATA

**Client:** DUKE ENERGY
**Project:** FGD CLARIFIER SLUDGE TESTING II
**Project Number:** 1264-03-057

### Sample Data

**Source:** Big Bend Power Plant Clarifier Sludge; Marshall Steam Station Soil
**Sample No.:**
**Elev. or Depth:** **Sample Length (in./cm.):**
**Location:**
**Description:** BIG BEND CLARIFIER/MARSHALL SOIL 1:1
**Liquid Limit:** 35 **Plastic Limit:** 29
**USCS Classification:** **AASHTO Classification:**
**Testing Remarks:** TESTED BY KMW

### Mechanical Analysis Data

| | Initial |
|---|---|
| Dry sample and tare= | 73.79 |
| Tare = | 0.00 |
| Dry sample weight = | 73.79 |

Sample split on number 10 sieve
Split sample data:
Sample and tare = 50.00 Tare = .00 Sample weight = 50.00
Cumulative weight retained tare= .00
Tare for cumulative weight retained= .00

| Sieve | Cumul. Wt. retained | Percent finer |
|---|---|---|
| .375 inch | 0.00 | 100.0 |
| # 4 | 0.12 | 99.8 |
| # 10 | 1.43 | 98.1 |
| # 20 | 2.26 | 93.7 |
| # 40 | 6.38 | 85.6 |
| # 60 | 12.49 | 73.6 |
| # 100 | 18.95 | 60.9 |
| # 200 | 26.72 | 45.7 |

### Hydrometer Analysis Data

Separation sieve is #10
Percent -#10 based upon complete sample= 98.1
Weight of hydrometer sample: 50.00
Hygroscopic moisture correction:
Moist weight & tare = 24.76
Dry weight & tare = 24.41
Tare = 14.20
Hygroscopic moisture= 3.4 %
Calculated biased weight= 49.28
Automatic temperature correction
Composite correction at 20 deg C = -4.5

Meniscus correction only= 1
Specific gravity of solids= 2.50
Specific gravity correction factor= 1.038
Hydrometer type: 152H
Effective depth L= 16.294964 - 0.164 x Rm

Ref 5

| Elapsed time, min | Temp, deg C | Actual reading | Corrected reading | K | Rm | Eff. depth | Diameter mm | Percent finer |
|---|---|---|---|---|---|---|---|---|
| 1.00 | 23.5 | 25.0 | 21.3 | 0.0137 | 26.0 | 12.0 | 0.0476 | 44.9 |
| 2.00 | 23.5 | 24.5 | 20.8 | 0.0137 | 25.5 | 12.1 | 0.0337 | 43.8 |
| 5.00 | 23.5 | 21.0 | 17.3 | 0.0137 | 22.0 | 12.7 | 0.0218 | 36.4 |
| 15.00 | 23.5 | 10.0 | 6.3 | 0.0137 | 11.0 | 14.5 | 0.0135 | 13.3 |
| 30.00 | 23.0 | 8.5 | 4.7 | 0.0138 | 9.5 | 14.7 | 0.0097 | 9.8 |
| 60.00 | 23.0 | 8.5 | 4.7 | 0.0138 | 9.5 | 14.7 | 0.0068 | 9.8 |
| 250.00 | 23.5 | 8.0 | 4.3 | 0.0137 | 9.0 | 14.8 | 0.0033 | 9.1 |
| 1440.00 | 23.0 | 7.0 | 3.2 | 0.0138 | 8.0 | 15.0 | 0.0014 | 6.7 |

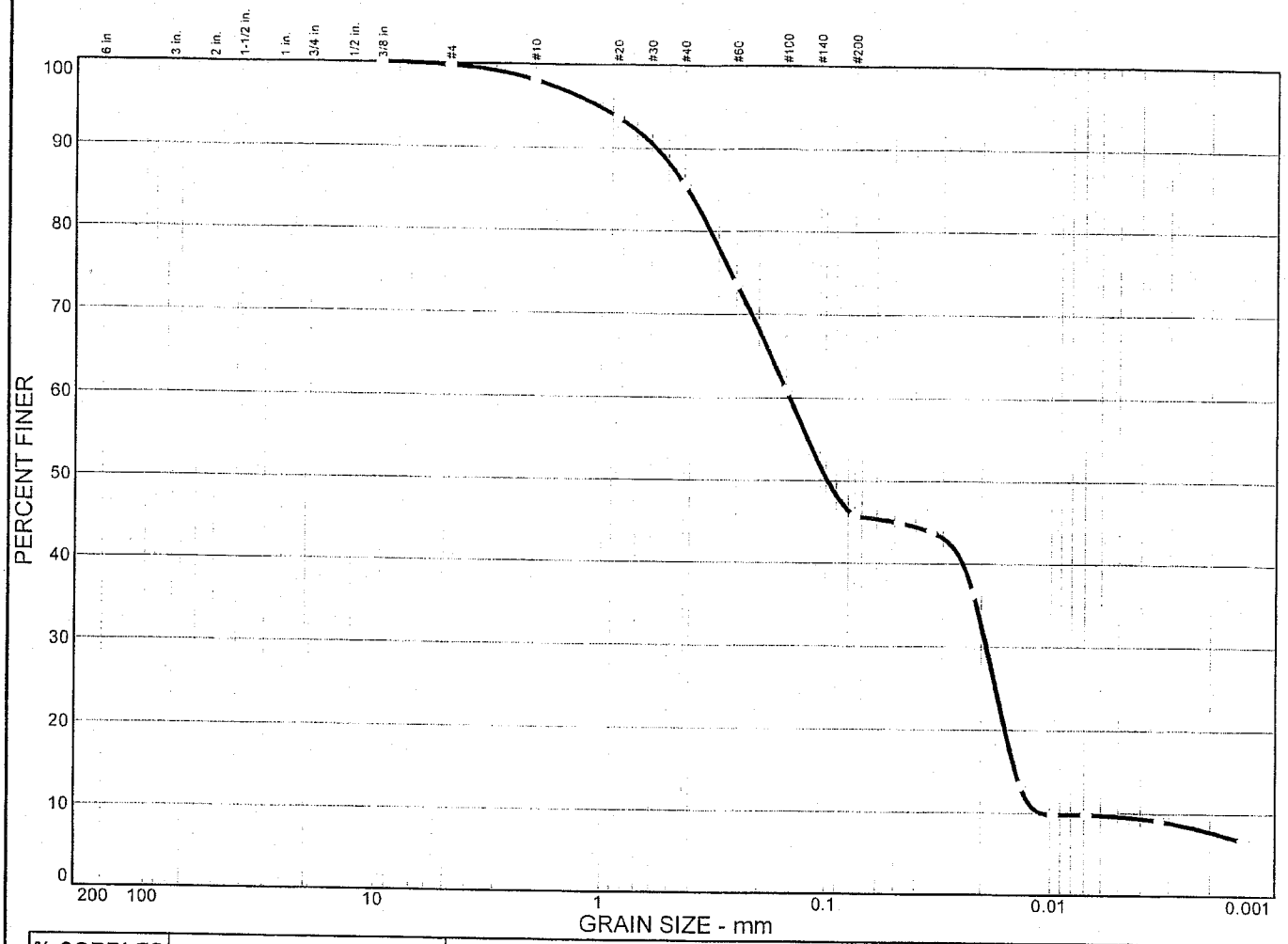
## Fractional Components

Gravel/Sand based on #4
Sand/Fines based on #200
% COBBLES = % GRAVEL = 0.2 % SAND = 54.1
% SILT = 36.1 % CLAY = 9.6

$D_{85}$= 0.41 $D_{60}$= 0.15 $D_{50}$= 0.10
$D_{30}$= 0.02 $D_{15}$= 0.01 $D_{10}$= 0.01
$C_c$= 0.2298 $C_u$= 13.3152

Ref. 5.

# Particle Size Distribution Report




| % COBBLES | % GRAVEL | % SAND | % SILT | % CLAY |
|---|---|---|---|---|
| 0.0 | 0.2 | 54.1 | 36.1 | 9.6 |

| LL | PL | $D_{85}$ | $D_{60}$ | $D_{50}$ | $D_{30}$ | $D_{15}$ | $D_{10}$ | $C_c$ | $C_u$ |
|---|---|---|---|---|---|---|---|---|---|
| 35 | 29 | 0.412 | 0.145 | 0.0980 | 0.0190 | 0.0142 | 0.0109 | 0.23 | 13.32 |

| MATERIAL DESCRIPTION | USCS | AASHTO |
|---|---|---|
| BIG BEND CLARIFIER/MARSHALL SOIL 1:1 | | |

**Project No.** 1264-03-057 **Client:** DUKE ENERGY

**Project:** FGD CLARIFIER SLUDGE TESTING II

**Source:** Big Bend Power Plant Clarifier Sludge; Marshall Steam Station Soil

**Remarks:**

TESTED BY KMW

Particle Size Distribution Report

**S & ME, INC.**

SAMPLE

Ref. 5

# CLARIFIER SLUDGE/GYPSUM MIX TEST RESULTS DUKE POWER FGD PLANNING S&ME Project 1264-03-057

Prepared For:

Duke Power SM

*A Duke Energy Company*

**Charlotte, North Carolina**

Prepared By:

S&ME

ENVIRONMENTAL SERVICES
ENGINEERING• TESTING

**Spartanburg, South Carolina**

Reference 6

November 2005

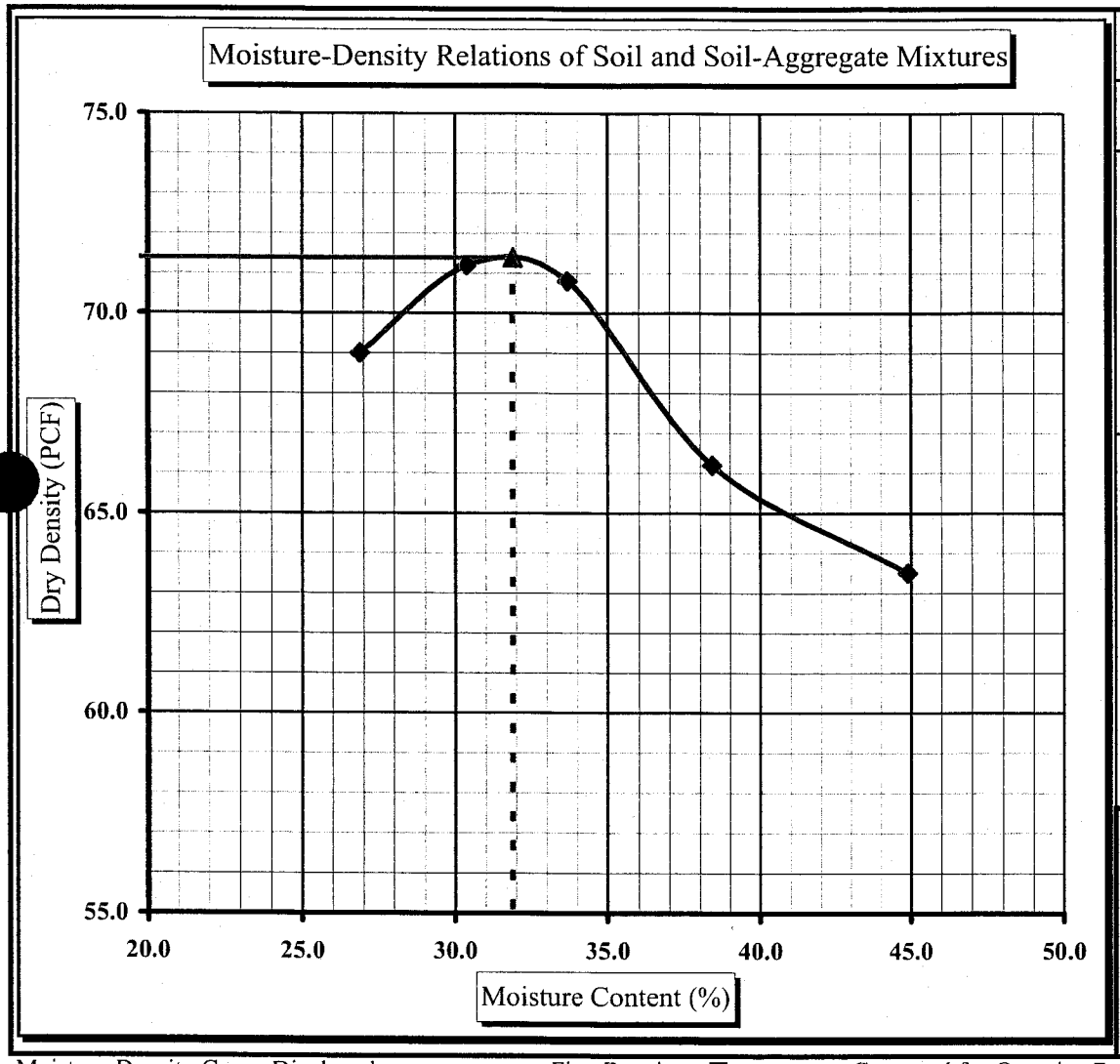
# Moisture - Density Report

S&ME

**S&ME Project #:** **1264-03-057** Report Date: October 20, 2005
**Project Name:** **FGD Clarifier Sludge Gypsum Mix Testing** Test Date(s): 10/13-19/05
Client Name: Duke Energy
Client Address:

Boring #: 0 Sample #: S-1 Sample Date:
Location: Big Bend Power Plant Offset: Depth: Bulk
Sample Description: ONE PART CLARIFIER/ONE PART GYPSUM (1:1)

**Maximum Dry Density 71.4 PCF.** **Optimum Moisture Content 31.9 %**




**Soil Properties**

Natural Moisture Content:

Liquid Limit:

Plastic Limit:

Plastic Index:

Specific Gravity: 2.330

% Passing

3/4"
1/2"
3/8"
#4

**Oversize Fraction**

Bulk Sp. Gravity
% Moisture
Oversize Fraction
**MDD**
**Opt. MC**

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:**

Ref 6

Technical Responsibility: Larry Armstrong

Signature

# CONSTANT HEAD PERMEABILITY

ASTM D5084

S&ME Project Number: 1264-03-057 Project Name: FGD SCRUBBER WASTE II Date: 11/7/2005

Sample Name: ONE PART CLARIFIER/ONE PART GYPSUM (1:1)

Sample Description: BIG BEND POWER PLANT CLARIFIER SLUDGE

Type of Sample: REMOLDED (89% MDD AT +2.4% OPTIMUM MOISTURE)

| Sample Data | | Molding Moisture Content | | Target Molding Conditon | | | Additonal Data | |
|---|---|---|---|---|---|---|---|---|
| D (cm) | 7.262 | Tare No. | G-13 | $\gamma_{wet}$ (pcf) | 84.8 | | Panel No | 8 |
| L (cm) | 7.62 | WWS+ Tare (gm) | 254.59 | $\gamma_{dry}$ (pcf) | 64.3 | | Cell P (psi) | 83.5 |
| A ($cm^2$) | 41.42 | WDS+Tare (gm) | 210.27 | w (%) | 31.9 | | Soil P (psi) | 81.0 |
| V ($cm^3$) | 315.61 | WT of Water (gm) | 44.32 | Actual Molding Condition | | | Back P (psi) | 80.0 |
| WWS (gm) | 431.1 | WT of TARE (gm) | 81.21 | $\gamma_{wet}$ (pcf) | 85.3 | | Gs | 2.33 |
| | | WT Dry Soil (gm) | 129.06 | $\gamma_{dry}$ (pcf) | 63.5 | | n | 0.56 |
| | | w (%) | 34.3 | w (%) | 34.3 | | | |

K = (Q x L) / (t x H x A)

| Run | Temp. (°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 | | 12 | 20 | | 9.8 | 0.0 | 24.8 | | | | |
| | 23 | 12 | 26 | 360 | 9.8 | 22.9 | 2.0 | 22.9 | 70.4 | 0.9310 | 1.54E-04 |
| 2 | | 12 | 27 | | 9.8 | 0.2 | 24.7 | | | | |
| | 23 | 12 | 33 | 360 | 9.8 | 24.4 | 0.4 | 24.3 | 70.4 | 0.9310 | 1.64E-04 |
| 3 | | 12 | 34 | | 9.8 | 0.3 | 24.7 | | | | |
| | 23 | 12 | 40 | 360 | 9.8 | 24.8 | 0.1 | 24.6 | 70.4 | 0.9310 | 1.66E-04 |
| 4 | | 12 | 57 | | 9.8 | 0.0 | 24.8 | | | | |
| | 23 | 13 | 2 | 300 | 9.8 | 22.6 | 2.0 | 22.7 | 70.4 | 0.9310 | 1.84E-04 |
| 5 | | 13 | 3 | | 9.8 | 0.0 | 24.9 | | | | |
| | 23 | 13 | 8 | 300 | 9.8 | 23.2 | 1.5 | 23.3 | 70.4 | 0.9310 | 1.89E-04 |
| | | | | | | | | | | AVG. K | 1.76E-04 |
| | | | | | | | | | | (Runs 2 thru 5) | |

Ref 6

# DESIGN CONSIDERATIONS FOR GCL COVER SYSTEMS

## INTRODUCTION

Compacted clay liners (CCLs) in cap applications are difficult to construct and are subject to deterioration from differential settlement and freeze-thaw. In an EPA field study Rogowski (1990) found that CCL field permeabilities were up to several orders of magnitude higher than expected by laboratory permeability tests. LaPlante and Zimmie (1992) found CCL hydraulic conductivity increased two orders of magnitude from freeze-thaw cycles. LaGatta et al. (1997) state that CCLs fail at only 0.1-4% tensile strain whereas GCLs can withstand up to 12% strain. Geosynthetic clay liners (GCLs) have been increasingly utilized in lieu of CCLs to overcome these problems, but certain design considerations made in GCL cap applications. The sodium bentonite component of GCLs is affected by confining pressure, ion exchange and desiccation.

## FACTORS AFFECTING GCL PERFORMANCE

*Confining Pressure*

The permeability of sodium bentonite is inversely proportional to confining pressure. Petrov and Rowe (1997) determined that there is an increase in hydraulic conductivity of GCLs subjected to low confining pressure compared to the standard 5 psi confining pressure used in ASTM D5887. From their equation, there is an estimated 85% increase in hydraulic conductivity at 1.7 psi confining pressure (equivalent to 2 feet of 120 pcf soil) and an estimated 50% increase in hydraulic conductivity at 2.5 psi confining pressure (equivalent to 3 feet of 120 pcf soil).

*Ion Exchange*

Recent laboratory and field research indicated that the sodium bentonite component of GCLs is subject to changes from ion exchange and desiccation. Ion exchange occurs when divalent ions, primarily calcium, replace the sodium ions attracted to the surface of the clay platelets. Ion exchange is a concern because naturally occurring calcium bentonite has a hydraulic conductivity of ~$10^{-5}$ cm/s.

Jo et al. (2001) reported laboratory studies that indicated that the effect of inorganic chemicals on nonprehydrated sodium bentonite is related to both valence and concentration. They found that a significant increase in hydraulic conductivity occurred between 0.01M and 0.025M $CaCl_2$ solution. The hydraulic conductivity rose from $1.2 \times 10^{-9}$ to $2.6 \times 10^{-7}$ cm/s.



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Ref 7

Benson & Shackelford (2000) reported laboratory studies indicating that the effect of inorganic chemicals on sodium bentonite is also related to whether the bentonite is prehydrated. A GCL was permeated with a saturated 6.7M $CaCl_2$ solution at 2.5 psi confining pressure. After 7 pore volumes the hydraulic conductivity was ~3 x $10^{-6}$ cm/s. The $Ca^{+2}$ ions were apparently exchanging for the sodium ions on the bentonite platelets. However, they noted that if a sodium bentonite is prehydrated with deionized water and then permeated with the saturated $CaCl_2$ solution, the bentonite hydraulic conductivity only rose to ~1 x $10^{-8}$ cm/s.

Egloffstein (2000) presents a theory on prehydration of sodium bentonite. Sodium bentonite undergoes an initial osmotic swelling which results in colloidal dispersion. Naturally occurring calcium bentonite does not undergo dispersion due to its higher valence and electrostatic attraction (see Figure 1). This dispersed platelet structure of sodium bentonite in part contributes to its low hydraulic conductivity by creating a tortuous pathway for free water molecules to flow.

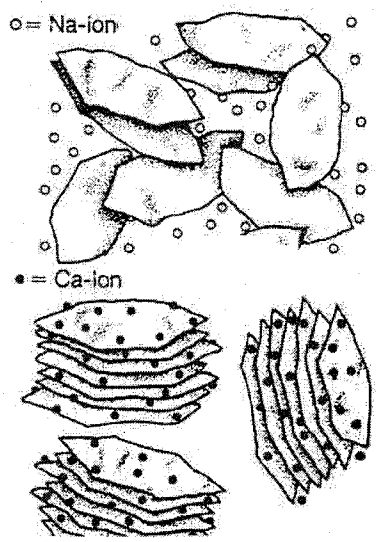


Figure 1. Dispersion of sodium montmorillonite platelets compared to naturally occurring calcium montmorillonite (Jasmund & Lagaly 1993)

This dispersed orientation can be maintained after swelling even if the pore water divalent concentration increases. An ion exchange of the sodium with divalent cations can take place over one to several years. The exchange continues until there is an equilibrium balance between the ion distribution on the surface of the bentonite and the ionic concentration in the pore water. Although the divalent cations cause a certain reduction in volume, some of the dispersed micro-structure is maintained as long as there is sufficient confining pressure. Thus, the resulting hydraulic conductivity is lower than that of naturally occurring calcium bentonite. Egloffstein estimated an increase in permeability of 3-5 times after ion exchange. Figure 2 compares the microstructure of virgin sodium bentonite, ion exchanged sodium bentonite, and calcium bentonite. The photo in Figure 2a is natural sodium bentonite with its dispersed honey-combed structure. The photo in Figure 2b is the sodium bentonite after permeation with a 0.3M $CaCl_2$ solution. The honeycombed structure remains, however, somewhat thicker.



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Figure 2c is bentonite from an excavated GCL sample. It is quite similar to Figure 2b. Figure 2d is a naturally occurring calcium bentonite with a thick flat aggregate structure.

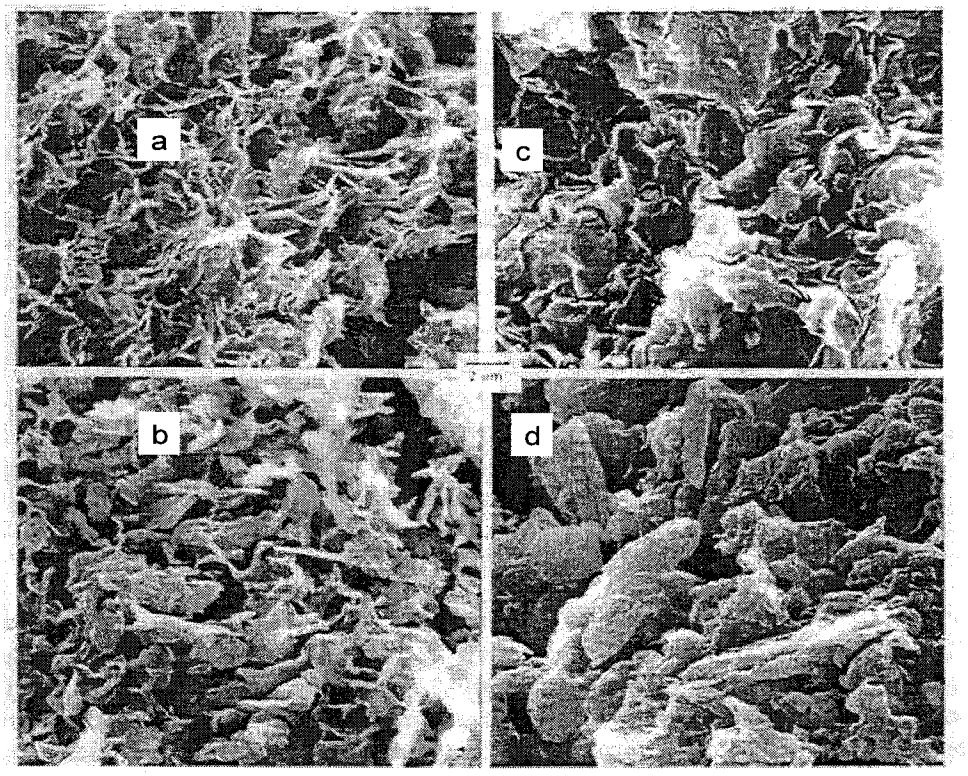


Figure 2. Comparison of montmorillonite micro structures using electron scan microscope pictures (Egloffstein 2000)

However, Lin and Benson (2000) performed laboratory work that suggests that desiccation can negate the beneficial effects of prehydration. Tests alternated permeation under 2.5 psi confining pressure and air drying under zero confining pressure. When a GCL was initially hydrated with deionized water, dried and then permeated with the saturated 6.7M $CaCl_2$ solution, the permeability rose to $3 \times 10^{-6}$ cm/s. This is 300 times higher than without drying. The problem with the test procedure is that unconfined drying would not occur in the field.

Field study data provide some good examples of what can be expected with varying confining pressure. Melchior (1997) tested a sodium bentonite GCL under 1 foot of cover at a landfill site in Germany. Shortly after one year, the GCL produced high leakage rates up to $1 \times 10^{-7}$ $m^3/m^2/s$, an order of magnitude greater than typical GCL certified index flux at 4.6 foot head pressure. Desiccation cracking was observed in exhumed samples.

GCL samples were exhumed from two landfill cover systems in the Southeast U.S. (Mackey and Olsta, 2003). Original laboratory tests for the lots of GCL material installed indicated hydraulic conductivities of 1.2 to $2.3 \times 10^{-9}$ cm/s at 2 psi confining pressure.



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Chemistry tests of the exhumed samples revealed that they had undergone significant ion exchange. Permeameter tests conducted at 2 psi confining pressure indicated a decreasing hydraulic conductivity with increasing depth (see Figure 3), but the hydraulic conductivity at 34 inches cover was still 2 to 4 times the initial hydraulic conductivity.

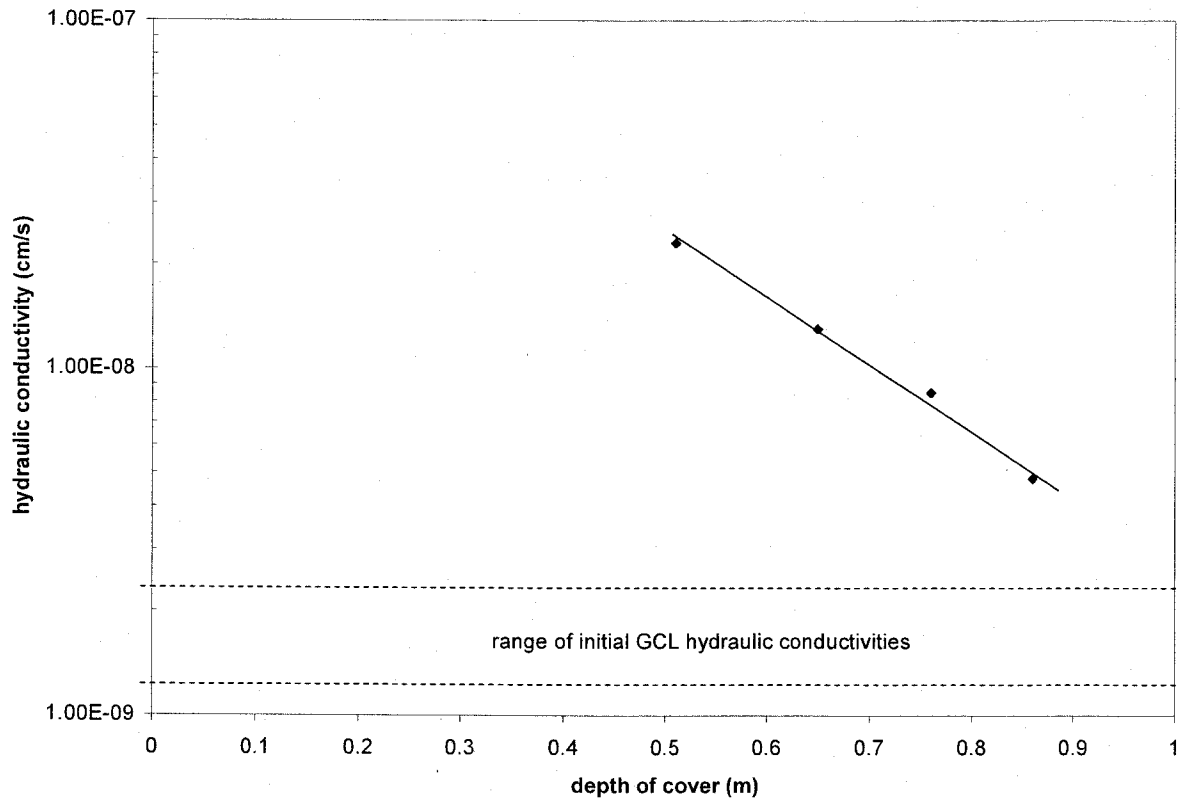


Figure 3. Hydraulic conductivity of exhumed GCL samples versus depth of cover soil.

## GUIDELINES

So what permeability can be expected in a typical cover application? It will depend on pore water soil chemistry and ability of the cover soil to provide confining pressure and protect the GCL from desiccation. The aqueous solubility of calcite (limestone) is quite low, 0.0006M $Ca^{+2}$, while gypsum has aqueous solubility of 0.015 M $Ca^{+2}$. However, the solubility increases dramatically with lower pH. At pH 5, typical of acid rain, the solubility of calcite is 0.2M $Ca^{+2}$. Thus, if the GCL is in contact with high dissolved cation concentration soil, such as gypsum, then ion exchange of the bentonite can be expected to occur during hydration. This would result in a significant increase in hydraulic conductivity.

CETCO suggests the following guidelines when designing a GCL cap:

- Avoid gypsum or other high dissolved divalent cation pore water soils in direct contact with the GCL. This will allow for proper prehydration of the GCL bentonite that creates the dispersed platelet structure needed for low permeability.



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- For geotextile-based GCL-only caps, use a minimum of 36" of cover to minimize potential for desiccation and achieve sufficient confining pressure to maintain the dispersed orientation of the bentonite platelets even during ion exchange. However, even with 36" of cover, the GCL hydraulic conductivity will increase by a factor of approximately 3.
- Assume a higher hydraulic conductivity than the ASTM D5887 certified hydraulic conductivity due to the lower confining pressure. For example, at 2.5 psi confining pressure, assume a 50% increase in hydraulic conductivity.
- Where lower hydraulic conductivity is required, consider a composite liner. Membrane-laminated GCLs, such as Bentomat CL, Bentomat CLT and Claymax 600CL can be economical alternatives for use in cap applications.

Using the previous referenced literature and Darcy's Law, Q/A = k * i, it is possible to estimate the following flux comparisons for CCL and GCL covers with 1 foot of head:

*Compacted Clay*

| | |
|---|---|
| Initial hydraulic conductivity | $1 \times 10^{-7}$ cm/s |
| Increase in hydraulic conductivity | 100 |
| Long-term hydraulic conductivity | $1 \times 10^{-5}$ cm/s |
| Thickness per Subtitle D | 1.5 feet |
| Head pressure | 1 foot |
| **Flux through liner** | **$1.7 \times 10^{-7}$ $m^3/m^2/s$** |

*Geosynthetic Clay Liner w/ 36" of cover*

| | |
|---|---|
| Initial hydraulic conductivity | $5 \times 10^{-9}$ cm/s |
| Increase in permeability due to confining pressure | 1.5 |
| Increase in permeability due to ion exchange | 3 |
| Long-term hydraulic conductivity | $2.2 \times 10^{-8}$ cm/s |
| Thickness | 0.7 cm |
| Head pressure | 1 foot |
| **Flux through liner** | **$1.0 \times 10^{-8}$ $m^3/m^2/s$** |

Thus, the GCL cover would be approximately 17 times more effective than a compacted clay cover.

## CONCLUSIONS

Compacted clay liners have been shown to be difficult to construct and can undergo significant (several orders of magnitude) increases in permeability due to desiccation, freeze-thaw and differential settlement. For these reasons, GCLs have become increasing used in cap design. Nevertheless, designers need to be aware of the effect of low confining pressure, ion exchange and desiccation on the long-term performance of GCLs. However, a well designed GCL cap is still expected to outperform a CCL cap.



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## REFERENCES


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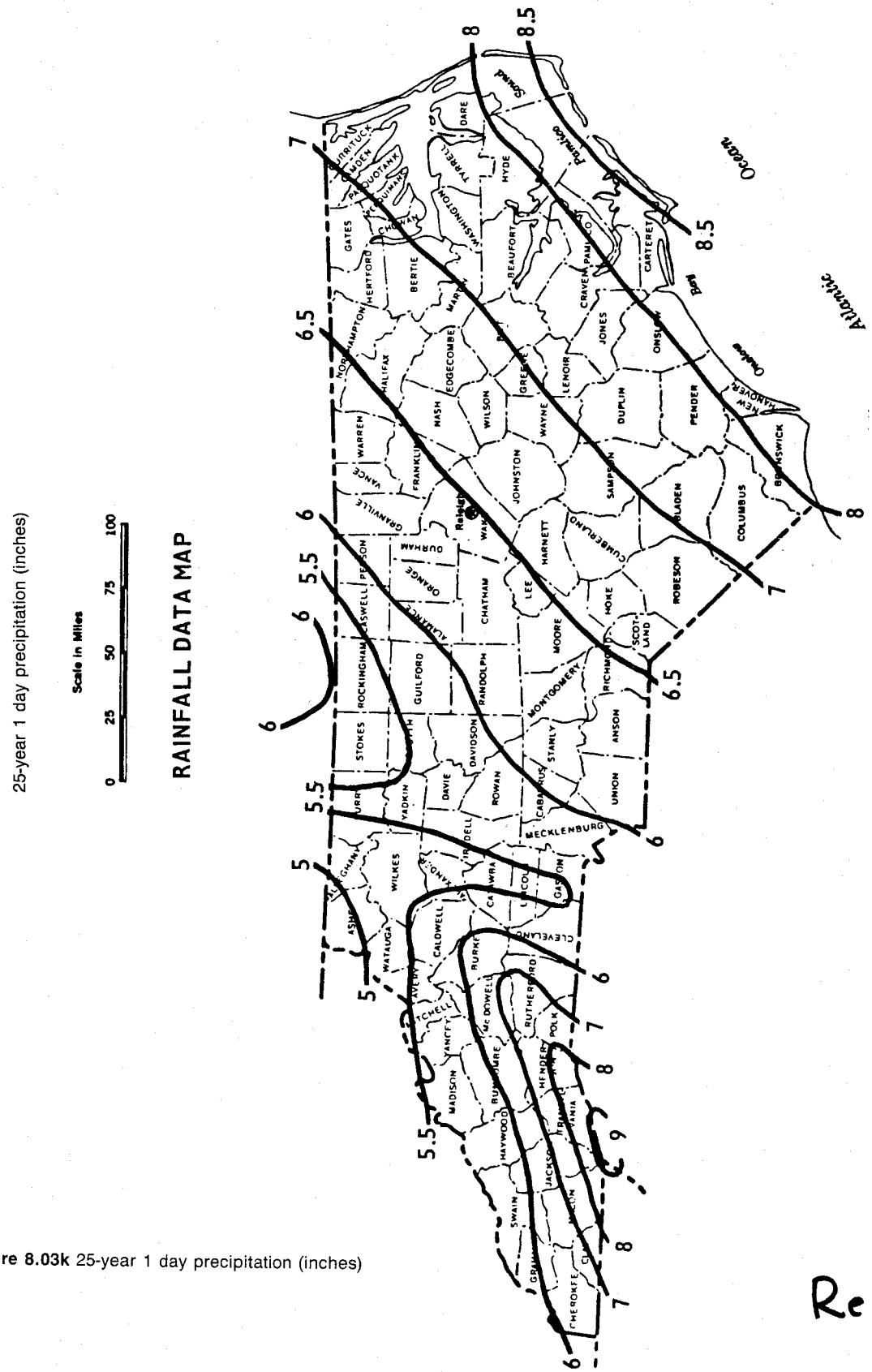


**Figure 8.03k** 25-year 1 day precipitation (inches)

Ref 8

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Consulting Engineers, Surveyors & Photogrammetrists

Comp. By ..... Date ..... Project ..... Sheet No. ..... of .....
Chkd. By ..... Date ..... Subject ..... Job No. .....

Objective: Calculate thickness of GCL to be used in HELP.

Reference: CETCO Claymax 600CL Certified Properties Sheet TR-401-CMCL

GCL

SG bentonite = 2.4

Bentonite Mass / Area = $0.75 \frac{lb}{ft^2}$ [Ref above]

Density = $2.4 \left(62.4 \frac{lb}{ft^3}\right)$

$= 149.7 \frac{lb}{ft^3}$

Thickness = $0.75 \frac{lb}{ft^2} \cdot \frac{ft^3}{149.7 lb}$

$= 0.005 ft$

Thickness = 0.06"

CETCO LINING TECHNOLOGIES

Certified Properties

# CLAYMAX® 600CL CERTIFIED PROPERTIES

| MATERIAL PROPERTY | TEST METHOD | TEST FREQUENCY $ft^2(m^2)$ | REQUIRED VALUES |
|---|---|---|---|
| Bentonite Swell Index[1] | ASTM D 5890 | 1 per 50 tonnes | 24 mL/2g min. |
| Bentonite Fluid Loss[1] | ASTM D 5891 | 1 per 50 tonnes | 18 mL max. |
| Bentonite Mass/Area[2] | ASTM D 5993 | 40,000 $ft^2$ (4,000 $m^2$) | 0.75 $lb/ft^2$ (3.6 $kg/m^2$) min |
| GCL Grab Strength[3] | ASTM D 4632<br>ASTM D 6768 | 200,000 $ft^2$ (20,000 $m^2$) | 75 lbs (330 N) MARV<br>18.75 lbs/in (33 N/cm) MARV |
| GCL Peel Strength[3] | ASTM D 4632<br>ASTM D 6496 | N/A | N/A |
| GCL Index Flux[4] | ASTM D 5887 | Periodic | $1 \times 10^{-9}$ $m^3/m^2/sec$ max |
| GCL Hydraulic Conductivity[4] | ASTM D 5887 | Periodic | $5 \times 10^{-10}$ cm/sec max |
| GCL Hydrated Internal Shear Strength[5] | ASTM D 5321<br>ASTM D 6243 | Periodic | 50 psf (2.4 kPa) typical |

***Claymax 600CL is an unreinforced GCL consisting of a layer of sodium bentonite between a geotextile, and a laminate comprised of a geotextile and a polyethylene membrane which are continuously adhered together.***

## Notes

[1] Bentonite property tests performed at a bentonite processing facility before shipment to CETCO's GCL production facilities.

[2] Bentonite mass/area reported at 0 percent moisture content.

[3] All tensile strength testing is performed in the machine direction using 4 inch grips per modified ASTM D 4632. Results are reported as minimum average roll values unless otherwise indicated. Upon request, tensile strength can be reported per ASTM D 6768.

[4] ASTM D5887 Index flux and hydraulic conductivity testing with deaired distilled/deionized water at 80 psi (551 kPa) cell pressure, 77 psi (531 kPa) headwater pressure and 75 psi (517 kPa) tailwater pressure. Reported value is equivalent to 92 gal/acre/day. This flux value is equivalent to a permeability of $5 \times 10^{-10}$ cm/sec for typical GCL thickness. ASTM D 5887 testing is performed only on a periodic basis because the membrane is essentially impermeable.

[5] Peak value measured at 200 psf (10 kPa) normal stress for a specimen hydrated for 48 hours. Site-specific materials, GCL products, and test conditions must be used to verify internal and interface strength of the proposed design.

CETCO® LINING TECHNOLOGIES

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Revised 09/04
TR 401-CMCL

TABLE 2. MODERATE AND HIGH DENSITY DEFAULT SOILS

| Soil Texture Class | | | Total Porosity vol/vol | Field Capacity vol/vol | Wilting Point vol/vol | Saturated Hydraulic Conductivity cm/sec |
|---|---|---|---|---|---|---|
| HELP | USDA | USCS | | | | |
| 22 | L (Moderate) | ML | 0.419 | 0.307 | 0.180 | $1.9 \times 10^{-5}$ |
| 23 | SiL (Moderate) | ML | 0.461 | 0.360 | 0.203 | $9.0 \times 10^{-6}$ |
| 24 | SCL (Moderate) | SC | 0.365 | 0.305 | 0.202 | $2.7 \times 10^{-6}$ |
| 25 | CL (Moderate) | CL | 0.437 | 0.373 | 0.266 | $3.6 \times 10^{-6}$ |
| 26 | SiCL (Moderate) | CL | 0.445 | 0.393 | 0.277 | $1.9 \times 10^{-6}$ |
| 27 | SC (Moderate) | SC | 0.400 | 0.366 | 0.288 | $7.8 \times 10^{-7}$ |
| 28 | SiC (Moderate) | CH | 0.452 | 0.411 | 0.311 | $1.2 \times 10^{-6}$ |
| 29 | C (Moderate) | CH | 0.451 | 0.419 | 0.332 | $6.8 \times 10^{-7}$ |
| 16 | Liner Soil (High) | | 0.427 | 0.418 | 0.367 | $1.0 \times 10^{-7}$ |
| 17 | Bentonite (High) | | 0.750 | 0.747 | 0.400 | $3.0 \times 10^{-9}$ |

### 3.5.2 Default Waste Characteristics

Table 4 provides a summary of default moisture retention values for various waste layers. Municipal waste properties provided in Tchobanoglous et al. (1977) and Equations 6 and 7 were used to determine the total porosity, field capacity, and wilting point of a well compacted municipal waste. The field capacity and wilting point were calculated using Tchobanoglous et al.'s high and low water content values, respectively. Oweis et al. (1990) provided information on the in-situ saturated hydraulic conductivity of municipal waste. Zeiss and Major (1993) described the moisture flow through

# Attachment 4

## Marshall FGD Residue Landfill

## HELP Model Results

```
******************************************************************************
******************************************************************************
**                                                                          **
**                                                                          **
**              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\mssfgd\MSSRAIN.D4
TEMPERATURE DATA FILE:        C:\WMM\HELP3-7\mssfgd\MSSTEMP.D7
SOLAR RADIATION DATA FILE:    C:\wmm\help3-7\mssfgd\MSSRAD.D13
EVAPOTRANSPIRATION DATA:      C:\wmm\help3-7\mssfgd\MSSOPEN.D11
SOIL AND DESIGN DATA FILE:    C:\wmm\help3-7\mssfgd\C1-10.D10
OUTPUT DATA FILE:             C:\WMM\HELP3-7\mssfgd\c1-10-25.OUT


TIME:   8:39     DATE:  12/16/2005


******************************************************************************

     TITLE:  MSSFGD LF - Cell 1 - 10 feet of Waste - 60 mil HDPE + 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   0
          THICKNESS                    =     20.00   INCHES
          POROSITY                     =      0.5600 VOL/VOL
          FIELD CAPACITY               =      0.2660 VOL/VOL
          WILTING POINT                =      0.0900 VOL/VOL
          INITIAL SOIL WATER CONTENT   =      0.2860 VOL/VOL
          EFFECTIVE SAT. HYD. COND.    =  0.176000001000E-03 CM/SEC
```

LAYER 2
---

TYPE 1 - VERTICAL PERCOLATION LAYER
MATERIAL TEXTURE NUMBER 0

| | | | |
|---|---|---|---|
| THICKNESS | = | 20.00 | INCHES |
| POROSITY | = | 0.5600 | VOL/VOL |
| FIELD CAPACITY | = | 0.2660 | VOL/VOL |
| WILTING POINT | = | 0.0900 | VOL/VOL |
| INITIAL SOIL WATER CONTENT | = | 0.3659 | VOL/VOL |
| EFFECTIVE SAT. HYD. COND. | = | 0.176000001000E-03 | CM/SEC |

LAYER 3
---

TYPE 1 - VERTICAL PERCOLATION LAYER
MATERIAL TEXTURE NUMBER 0

| | | | |
|---|---|---|---|
| THICKNESS | = | 20.00 | INCHES |
| POROSITY | = | 0.5600 | VOL/VOL |
| FIELD CAPACITY | = | 0.2660 | VOL/VOL |
| WILTING POINT | = | 0.0900 | VOL/VOL |
| INITIAL SOIL WATER CONTENT | = | 0.3043 | VOL/VOL |
| EFFECTIVE SAT. HYD. COND. | = | 0.176000001000E-03 | CM/SEC |

LAYER 4
---

TYPE 1 - VERTICAL PERCOLATION LAYER
MATERIAL TEXTURE NUMBER 0

| | | | |
|---|---|---|---|
| THICKNESS | = | 20.00 | INCHES |
| POROSITY | = | 0.5600 | VOL/VOL |
| FIELD CAPACITY | = | 0.2660 | VOL/VOL |
| WILTING POINT | = | 0.0900 | VOL/VOL |
| INITIAL SOIL WATER CONTENT | = | 0.3093 | VOL/VOL |
| EFFECTIVE SAT. HYD. COND. | = | 0.176000001000E-03 | CM/SEC |

LAYER 5
---

TYPE 1 - VERTICAL PERCOLATION LAYER
MATERIAL TEXTURE NUMBER 0

| | | | |
|---|---|---|---|
| THICKNESS | = | 20.00 | INCHES |
| POROSITY | = | 0.5600 | VOL/VOL |
| FIELD CAPACITY | = | 0.2660 | VOL/VOL |
| WILTING POINT | = | 0.0900 | VOL/VOL |
| INITIAL SOIL WATER CONTENT | = | 0.2893 | VOL/VOL |

```
EFFECTIVE SAT. HYD. COND.    =  0.176000001000E-03 CM/SEC


                        LAYER  6
                        --------

          TYPE 1 - VERTICAL PERCOLATION LAYER
              MATERIAL TEXTURE NUMBER   0
THICKNESS                    =     20.00   INCHES
POROSITY                     =      0.5600 VOL/VOL
FIELD CAPACITY               =      0.2660 VOL/VOL
WILTING POINT                =      0.0900 VOL/VOL
INITIAL SOIL WATER CONTENT   =      0.2928 VOL/VOL
EFFECTIVE SAT. HYD. COND.    =  0.176000001000E-03 CM/SEC


                        LAYER  7
                        --------

          TYPE 1 - VERTICAL PERCOLATION LAYER
              MATERIAL TEXTURE NUMBER   0
THICKNESS                    =     12.00   INCHES
POROSITY                     =      0.5000 VOL/VOL
FIELD CAPACITY               =      0.2720 VOL/VOL
WILTING POINT                =      0.0900 VOL/VOL
INITIAL SOIL WATER CONTENT   =      0.2720 VOL/VOL
EFFECTIVE SAT. HYD. COND.    =  0.551999983000E-04 CM/SEC


                        LAYER  8
                        --------

            TYPE 2 - LATERAL DRAINAGE LAYER
              MATERIAL TEXTURE NUMBER  20
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   =      0.0129 VOL/VOL
EFFECTIVE SAT. HYD. COND.    =   10.0000000000     CM/SEC
SLOPE                        =      1.50   PERCENT
DRAINAGE LENGTH              =    110.0    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        =  2 - EXCELLENT
```

LAYER 10
---

TYPE 1 - VERTICAL PERCOLATION LAYER
MATERIAL TEXTURE NUMBER 0

```
THICKNESS                    =      0.06    INCHES
POROSITY                     =      0.7500 VOL/VOL
FIELD CAPACITY               =      0.7470 VOL/VOL
WILTING POINT                =      0.4000 VOL/VOL
INITIAL SOIL WATER CONTENT   =      0.6086 VOL/VOL
EFFECTIVE SAT. HYD. COND.    =  0.999999997000E-06 CM/SEC
```

LAYER 11
---

TYPE 1 - VERTICAL PERCOLATION LAYER
MATERIAL TEXTURE NUMBER 0

```
THICKNESS                    =     48.00    INCHES
POROSITY                     =      0.5300 VOL/VOL
FIELD CAPACITY               =      0.2210 VOL/VOL
WILTING POINT                =      0.1010 VOL/VOL
INITIAL SOIL WATER CONTENT   =      0.2210 VOL/VOL
EFFECTIVE SAT. HYD. COND.    =  0.372000010000E-03 CM/SEC
```

GENERAL DESIGN AND EVAPORATIVE ZONE DATA
---

NOTE: SCS RUNOFF CURVE NUMBER WAS USER-SPECIFIED.

```
SCS RUNOFF CURVE NUMBER                =     82.00
FRACTION OF AREA ALLOWING RUNOFF       =    100.0     PERCENT
AREA PROJECTED ON HORIZONTAL PLANE     =     14.800   ACRES
EVAPORATIVE ZONE DEPTH                 =     10.0     INCHES
INITIAL WATER IN EVAPORATIVE ZONE      =      2.351   INCHES
UPPER LIMIT OF EVAPORATIVE STORAGE     =      5.600   INCHES
LOWER LIMIT OF EVAPORATIVE STORAGE     =      0.900   INCHES
INITIAL SNOW WATER                     =      0.000   INCHES
```

| | | | |
|---|---|---|---|
| INITIAL WATER IN LAYER MATERIALS | = | 50.863 | INCHES |
| TOTAL INITIAL WATER | = | 50.863 | INCHES |
| TOTAL SUBSURFACE INFLOW | = | 0.00 | INCHES/YEAR |

## EVAPOTRANSPIRATION AND WEATHER DATA

NOTE: EVAPOTRANSPIRATION DATA WAS OBTAINED FROM
CHARLOTTE NORTH CAROLINA

| | | | |
|---|---|---|---|
| STATION LATITUDE | = | 35.60 | DEGREES |
| MAXIMUM LEAF AREA INDEX | = | 0.00 | |
| START OF GROWING SEASON (JULIAN DATE) | = | 83 | |
| END OF GROWING SEASON (JULIAN DATE) | = | 312 | |
| EVAPORATIVE ZONE DEPTH | = | 10.0 | INCHES |
| AVERAGE ANNUAL WIND SPEED | = | 7.50 | MPH |
| AVERAGE 1ST QUARTER RELATIVE HUMIDITY | = | 64.00 | % |
| AVERAGE 2ND QUARTER RELATIVE HUMIDITY | = | 67.00 | % |
| AVERAGE 3RD QUARTER RELATIVE HUMIDITY | = | 74.00 | % |
| AVERAGE 4TH QUARTER RELATIVE HUMIDITY | = | 70.00 | % |

NOTE: PRECIPITATION DATA WAS SYNTHETICALLY GENERATED USING
COEFFICIENTS FOR CHARLOTTE NORTH CAROLINA

NORMAL MEAN MONTHLY PRECIPITATION (INCHES)

| JAN/JUL | FEB/AUG | MAR/SEP | APR/OCT | MAY/NOV | JUN/DEC |
|---|---|---|---|---|---|
| 4.10 | 4.10 | 4.70 | 3.80 | 3.60 | 3.90 |
| 5.00 | 5.60 | 3.80 | 3.40 | 3.10 | 4.10 |

NOTE: TEMPERATURE DATA WAS SYNTHETICALLY GENERATED USING
COEFFICIENTS FOR CHARLOTTE NORTH CAROLINA

NORMAL MEAN MONTHLY TEMPERATURE (DEGREES FAHRENHEIT)

| JAN/JUL | FEB/AUG | MAR/SEP | APR/OCT | MAY/NOV | JUN/DEC |
|---|---|---|---|---|---|
| 41.00 | 42.00 | 49.00 | 59.00 | 67.00 | 75.00 |
| 77.00 | 76.00 | 70.50 | 60.50 | 49.00 | 41.50 |

NOTE: SOLAR RADIATION DATA WAS SYNTHETICALLY GENERATED USING
COEFFICIENTS FOR CHARLOTTE NORTH CAROLINA
AND STATION LATITUDE = 35.20 DEGREES

**********

AVERAGE MONTHLY VALUES IN INCHES FOR YEARS 1 THROUGH 100

| | JAN/JUL | FEB/AUG | MAR/SEP | APR/OCT | MAY/NOV | JUN/DEC |
|---|---|---|---|---|---|---|
| **PRECIPITATION** | | | | | | |
| TOTALS | 4.00 | 4.06 | 4.67 | 4.27 | 3.64 | 4.31 |
| | 5.01 | 5.15 | 3.58 | 3.23 | 3.08 | 4.25 |
| STD. DEVIATIONS | 1.99 | 1.84 | 2.23 | 1.97 | 1.79 | 2.10 |
| | 2.36 | 3.06 | 2.62 | 2.06 | 1.67 | 1.89 |
| **RUNOFF** | | | | | | |
| TOTALS | 0.198 | 0.196 | 0.184 | 0.147 | 0.091 | 0.150 |
| | 0.183 | 0.396 | 0.282 | 0.191 | 0.081 | 0.214 |
| STD. DEVIATIONS | 0.345 | 0.464 | 0.352 | 0.257 | 0.232 | 0.220 |
| | 0.380 | 0.572 | 0.585 | 0.325 | 0.166 | 0.317 |
| **EVAPOTRANSPIRATION** | | | | | | |
| TOTALS | 1.784 | 2.033 | 3.188 | 3.521 | 3.384 | 3.532 |
| | 4.059 | 3.490 | 2.684 | 2.166 | 1.642 | 1.448 |
| STD. DEVIATIONS | 0.262 | 0.349 | 0.451 | 0.984 | 1.265 | 1.471 |
| | 1.393 | 1.270 | 1.158 | 0.794 | 0.523 | 0.231 |
| **LATERAL DRAINAGE COLLECTED FROM LAYER 8** | | | | | | |
| TOTALS | 0.9477 | 1.2384 | 1.7420 | 1.8241 | 1.6844 | 1.3529 |
| | 1.1508 | 0.9703 | 0.7550 | 0.7426 | 0.7529 | 0.8245 |
| STD. DEVIATIONS | 0.8872 | 1.0887 | 1.2738 | 1.0930 | 0.7796 | 0.4583 |
| | 0.3902 | 0.2899 | 0.3717 | 0.6177 | 0.7254 | 0.6298 |
| **PERCOLATION/LEAKAGE THROUGH LAYER 9** | | | | | | |
| TOTALS | 0.0000 | 0.0000 | 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 |
| **PERCOLATION/LEAKAGE THROUGH LAYER 11** | | | | | | |
| TOTALS | 0.0000 | 0.0000 | 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 |

AVERAGES OF MONTHLY AVERAGED DAILY HEADS (INCHES)

DAILY AVERAGE HEAD ON TOP OF LAYER 9

| | | | | | | |
|---|---|---|---|---|---|---|
| AVERAGES | 0.0040 | 0.0057 | 0.0073 | 0.0079 | 0.0070 | 0.0058 |
| | 0.0048 | 0.0040 | 0.0033 | 0.0031 | 0.0032 | 0.0034 |
| STD. DEVIATIONS | 0.0037 | 0.0050 | 0.0053 | 0.0047 | 0.0033 | 0.0020 |
| | 0.0016 | 0.0012 | 0.0016 | 0.0026 | 0.0031 | 0.0026 |

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AVERAGE ANNUAL TOTALS & (STD. DEVIATIONS) FOR YEARS 1 THROUGH 100

| | INCHES | | CU. FEET | PERCENT |
|---|---|---|---|---|
| PRECIPITATION | 49.24 | ( 7.134) | 2645300.2 | 100.00 |
| RUNOFF | 2.313 | ( 1.3402) | 124258.06 | 4.697 |
| EVAPOTRANSPIRATION | 32.931 | ( 3.5716) | 1769179.87 | 66.880 |
| LATERAL DRAINAGE COLLECTED FROM LAYER 8 | 13.98553 | ( 4.28253) | 751358.687 | 28.40353 |
| PERCOLATION/LEAKAGE THROUGH LAYER 9 | 0.00001 | ( 0.00000) | 0.528 | 0.00002 |
| AVERAGE HEAD ON TOP OF LAYER 9 | 0.005 | ( 0.002) | | |
| PERCOLATION/LEAKAGE THROUGH LAYER 11 | 0.00001 | ( 0.00005) | 0.475 | 0.00002 |
| CHANGE IN WATER STORAGE | 0.009 | ( 4.0086) | 502.37 | 0.019 |

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PEAK DAILY VALUES FOR YEARS 1 THROUGH 100

| | (INCHES) | (CU. FT.) |
|---|---|---|
| PRECIPITATION | 5.58 | 299779.906 |
| RUNOFF | 2.318 | 124533.6870 |
| DRAINAGE COLLECTED FROM LAYER 8 | 0.33729 | 18120.80660 |
| PERCOLATION/LEAKAGE THROUGH LAYER 9 | 0.000000 | 0.01112 |
| AVERAGE HEAD ON TOP OF LAYER 9 | 0.044 | |
| MAXIMUM HEAD ON TOP OF LAYER 9 | 0.086 | |
| LOCATION OF MAXIMUM HEAD IN LAYER 8 (DISTANCE FROM DRAIN) | 2.1 FEET | |
| PERCOLATION/LEAKAGE THROUGH LAYER 11 | 0.000295 | 15.82347 |
| SNOW WATER | 3.87 | 207962.6410 |

| | |
|---|---|
| MAXIMUM VEG. SOIL WATER (VOL/VOL) | 0.5237 |
| 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 | 5.6531 | 0.2827 |
| 2 | 6.5462 | 0.3273 |
| 3 | 5.9146 | 0.2957 |
| 4 | 6.2078 | 0.3104 |
| 5 | 6.5859 | 0.3293 |
| 6 | 6.6485 | 0.3324 |
| 7 | 3.5965 | 0.2997 |
| 8 | 0.0017 | 0.0100 |
| 9 | 0.0000 | 0.0000 |
| 10 | 0.0365 | 0.6086 |
| 11 | 10.6078 | 0.2210 |
| 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\mssfgd\MSSRAIN.D4
TEMPERATURE DATA FILE:        C:\WMM\HELP3-7\mssfgd\MSSTEMP.D7
SOLAR RADIATION DATA FILE:    C:\wmm\help3-7\mssfgd\MSSRAD.D13
EVAPOTRANSPIRATION DATA:      C:\wmm\help3-7\mssfgd\MSSOPEN.D11
SOIL AND DESIGN DATA FILE:    C:\wmm\help3-7\mssfgd\C1-32.D10
OUTPUT DATA FILE:             C:\WMM\HELP3-7\mssfgd\C1-32.OUT


TIME:   8:43     DATE:  12/16/2005


******************************************************************************

     TITLE:  MSSFGD LF - Cell 1 - 32 feet of Waste - 60 mil HDPE + 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   0
          THICKNESS                    =     64.00   INCHES
          POROSITY                     =      0.5600 VOL/VOL
          FIELD CAPACITY               =      0.2660 VOL/VOL
          WILTING POINT                =      0.0900 VOL/VOL
          INITIAL SOIL WATER CONTENT   =      0.3175 VOL/VOL
          EFFECTIVE SAT. HYD. COND.    =  0.176000001000E-03 CM/SEC
```

LAYER 2
---

TYPE 1 - VERTICAL PERCOLATION LAYER
MATERIAL TEXTURE NUMBER 0

| | | | |
|---|---|---|---|
| THICKNESS | = | 64.00 | INCHES |
| POROSITY | = | 0.5600 | VOL/VOL |
| FIELD CAPACITY | = | 0.2660 | VOL/VOL |
| WILTING POINT | = | 0.0900 | VOL/VOL |
| INITIAL SOIL WATER CONTENT | = | 0.2887 | VOL/VOL |
| EFFECTIVE SAT. HYD. COND. | = | 0.176000001000E-03 | CM/SEC |

LAYER 3
---

TYPE 1 - VERTICAL PERCOLATION LAYER
MATERIAL TEXTURE NUMBER 0

| | | | |
|---|---|---|---|
| THICKNESS | = | 64.00 | INCHES |
| POROSITY | = | 0.5600 | VOL/VOL |
| FIELD CAPACITY | = | 0.2660 | VOL/VOL |
| WILTING POINT | = | 0.0900 | VOL/VOL |
| INITIAL SOIL WATER CONTENT | = | 0.2774 | VOL/VOL |
| EFFECTIVE SAT. HYD. COND. | = | 0.176000001000E-03 | CM/SEC |

LAYER 4
---

TYPE 1 - VERTICAL PERCOLATION LAYER
MATERIAL TEXTURE NUMBER 0

| | | | |
|---|---|---|---|
| THICKNESS | = | 64.00 | INCHES |
| POROSITY | = | 0.5600 | VOL/VOL |
| FIELD CAPACITY | = | 0.2660 | VOL/VOL |
| WILTING POINT | = | 0.0900 | VOL/VOL |
| INITIAL SOIL WATER CONTENT | = | 0.3033 | VOL/VOL |
| EFFECTIVE SAT. HYD. COND. | = | 0.176000001000E-03 | CM/SEC |

LAYER 5
---

TYPE 1 - VERTICAL PERCOLATION LAYER
MATERIAL TEXTURE NUMBER 0

| | | | |
|---|---|---|---|
| THICKNESS | = | 64.00 | INCHES |
| POROSITY | = | 0.5600 | VOL/VOL |
| FIELD CAPACITY | = | 0.2660 | VOL/VOL |
| WILTING POINT | = | 0.0900 | VOL/VOL |
| INITIAL SOIL WATER CONTENT | = | 0.2747 | VOL/VOL |

```
EFFECTIVE SAT. HYD. COND.    =  0.176000001000E-03 CM/SEC


                         LAYER  6
                         --------

          TYPE 1 - VERTICAL PERCOLATION LAYER
              MATERIAL TEXTURE NUMBER   0
THICKNESS                    =     64.00   INCHES
POROSITY                     =      0.5600 VOL/VOL
FIELD CAPACITY               =      0.2660 VOL/VOL
WILTING POINT                =      0.0900 VOL/VOL
INITIAL SOIL WATER CONTENT   =      0.2660 VOL/VOL
EFFECTIVE SAT. HYD. COND.    =  0.176000001000E-03 CM/SEC


                         LAYER  7
                         --------

          TYPE 1 - VERTICAL PERCOLATION LAYER
              MATERIAL TEXTURE NUMBER   0
THICKNESS                    =     12.00   INCHES
POROSITY                     =      0.4900 VOL/VOL
FIELD CAPACITY               =      0.2720 VOL/VOL
WILTING POINT                =      0.0900 VOL/VOL
INITIAL SOIL WATER CONTENT   =      0.2720 VOL/VOL
EFFECTIVE SAT. HYD. COND.    =  0.551999983000E-04 CM/SEC


                         LAYER  8
                         --------

            TYPE 2 - LATERAL DRAINAGE LAYER
              MATERIAL TEXTURE NUMBER  20
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   =      0.0101 VOL/VOL
EFFECTIVE SAT. HYD. COND.    =   10.0000000000     CM/SEC
SLOPE                        =      1.50   PERCENT
DRAINAGE LENGTH              =    110.0    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 | = | 2 - EXCELLENT | |

LAYER 10
---

TYPE 1 - VERTICAL PERCOLATION LAYER
MATERIAL TEXTURE NUMBER 0

| | | | |
|---|---|---|---|
| THICKNESS | = | 0.06 | INCHES |
| POROSITY | = | 0.7500 | VOL/VOL |
| FIELD CAPACITY | = | 0.7470 | VOL/VOL |
| WILTING POINT | = | 0.4000 | VOL/VOL |
| INITIAL SOIL WATER CONTENT | = | 0.6086 | VOL/VOL |
| EFFECTIVE SAT. HYD. COND. | = | 0.999999997000E-06 | CM/SEC |

LAYER 11
---

TYPE 1 - VERTICAL PERCOLATION LAYER
MATERIAL TEXTURE NUMBER 0

| | | | |
|---|---|---|---|
| THICKNESS | = | 48.00 | INCHES |
| POROSITY | = | 0.5300 | VOL/VOL |
| FIELD CAPACITY | = | 0.2210 | VOL/VOL |
| WILTING POINT | = | 0.1010 | VOL/VOL |
| INITIAL SOIL WATER CONTENT | = | 0.2210 | VOL/VOL |
| EFFECTIVE SAT. HYD. COND. | = | 0.372000010000E-03 | CM/SEC |

GENERAL DESIGN AND EVAPORATIVE ZONE DATA
---

NOTE: SCS RUNOFF CURVE NUMBER WAS USER-SPECIFIED.

| | | | |
|---|---|---|---|
| SCS RUNOFF CURVE NUMBER | = | 82.00 | |
| FRACTION OF AREA ALLOWING RUNOFF | = | 100.0 | PERCENT |
| AREA PROJECTED ON HORIZONTAL PLANE | = | 14.800 | ACRES |
| EVAPORATIVE ZONE DEPTH | = | 10.0 | INCHES |
| INITIAL WATER IN EVAPORATIVE ZONE | = | 2.351 | INCHES |
| UPPER LIMIT OF EVAPORATIVE STORAGE | = | 5.600 | INCHES |
| LOWER LIMIT OF EVAPORATIVE STORAGE | = | 0.900 | INCHES |
| INITIAL SNOW WATER | = | 0.000 | INCHES |

| | | | |
|---|---|---|---|
| INITIAL WATER IN LAYER MATERIALS | = | 124.476 | INCHES |
| TOTAL INITIAL WATER | = | 124.476 | INCHES |
| TOTAL SUBSURFACE INFLOW | = | 0.00 | INCHES/YEAR |

## EVAPOTRANSPIRATION AND WEATHER DATA

NOTE: EVAPOTRANSPIRATION DATA WAS OBTAINED FROM
CHARLOTTE NORTH CAROLINA

| | | | |
|---|---|---|---|
| STATION LATITUDE | = | 35.60 | DEGREES |
| MAXIMUM LEAF AREA INDEX | = | 0.00 | |
| START OF GROWING SEASON (JULIAN DATE) | = | 83 | |
| END OF GROWING SEASON (JULIAN DATE) | = | 312 | |
| EVAPORATIVE ZONE DEPTH | = | 10.0 | INCHES |
| AVERAGE ANNUAL WIND SPEED | = | 7.50 | MPH |
| AVERAGE 1ST QUARTER RELATIVE HUMIDITY | = | 64.00 | % |
| AVERAGE 2ND QUARTER RELATIVE HUMIDITY | = | 67.00 | % |
| AVERAGE 3RD QUARTER RELATIVE HUMIDITY | = | 74.00 | % |
| AVERAGE 4TH QUARTER RELATIVE HUMIDITY | = | 70.00 | % |

NOTE: PRECIPITATION DATA WAS SYNTHETICALLY GENERATED USING
COEFFICIENTS FOR CHARLOTTE NORTH CAROLINA

NORMAL MEAN MONTHLY PRECIPITATION (INCHES)

| JAN/JUL | FEB/AUG | MAR/SEP | APR/OCT | MAY/NOV | JUN/DEC |
|---|---|---|---|---|---|
| 4.10 | 4.10 | 4.70 | 3.80 | 3.60 | 3.90 |
| 5.00 | 5.60 | 3.80 | 3.40 | 3.10 | 4.10 |

NOTE: TEMPERATURE DATA WAS SYNTHETICALLY GENERATED USING
COEFFICIENTS FOR CHARLOTTE NORTH CAROLINA

NORMAL MEAN MONTHLY TEMPERATURE (DEGREES FAHRENHEIT)

| JAN/JUL | FEB/AUG | MAR/SEP | APR/OCT | MAY/NOV | JUN/DEC |
|---|---|---|---|---|---|
| 41.00 | 42.00 | 49.00 | 59.00 | 67.00 | 75.00 |
| 77.00 | 76.00 | 70.50 | 60.50 | 49.00 | 41.50 |

NOTE: SOLAR RADIATION DATA WAS SYNTHETICALLY GENERATED USING
COEFFICIENTS FOR CHARLOTTE NORTH CAROLINA
AND STATION LATITUDE = 35.20 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 | 3.68 | 3.72 | 5.27 | 3.14 | 3.45 | 4.68 |
| | 4.56 | 5.51 | 3.73 | 3.69 | 2.78 | 4.00 |
| STD. DEVIATIONS | 1.86 | 1.79 | 1.54 | 2.32 | 2.04 | 1.24 |
| | 1.67 | 3.79 | 1.87 | 2.26 | 1.48 | 2.09 |
| **RUNOFF** | | | | | | |
| TOTALS | 0.180 | 0.047 | 0.179 | 0.034 | 0.109 | 0.091 |
| | 0.141 | 0.288 | 0.121 | 0.164 | 0.070 | 0.194 |
| STD. DEVIATIONS | 0.230 | 0.059 | 0.320 | 0.064 | 0.210 | 0.132 |
| | 0.329 | 0.425 | 0.131 | 0.306 | 0.128 | 0.344 |
| **EVAPOTRANSPIRATION** | | | | | | |
| TOTALS | 1.805 | 2.170 | 3.341 | 3.138 | 3.019 | 3.693 |
| | 4.103 | 3.579 | 2.987 | 2.323 | 1.946 | 1.358 |
| STD. DEVIATIONS | 0.244 | 0.171 | 0.238 | 1.367 | 1.817 | 1.425 |
| | 1.348 | 1.419 | 1.236 | 0.780 | 0.292 | 0.133 |
| **LATERAL DRAINAGE COLLECTED FROM LAYER 8** | | | | | | |
| TOTALS | 0.9378 | 0.8834 | 0.9422 | 0.9734 | 1.0286 | 0.9876 |
| | 1.1162 | 1.2089 | 1.1764 | 1.2161 | 1.0779 | 1.1469 |
| STD. DEVIATIONS | 0.5512 | 0.5097 | 0.5020 | 0.4529 | 0.3970 | 0.4335 |
| | 0.5777 | 0.6907 | 0.6672 | 0.4626 | 0.4237 | 0.5388 |
| **PERCOLATION/LEAKAGE THROUGH LAYER 9** | | | | | | |
| TOTALS | 0.0000 | 0.0000 | 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 |
| **PERCOLATION/LEAKAGE THROUGH LAYER 11** | | | | | | |
| TOTALS | 0.0000 | 0.0000 | 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 |

AVERAGES OF MONTHLY AVERAGED DAILY HEADS (INCHES)

DAILY AVERAGE HEAD ON TOP OF LAYER 9

| | | | | | | |
|---|---|---|---|---|---|---|
| AVERAGES | 0.0039 | 0.0040 | 0.0039 | 0.0042 | 0.0043 | 0.0043 |
| | 0.0047 | 0.0050 | 0.0051 | 0.0051 | 0.0046 | 0.0048 |
| STD. DEVIATIONS | 0.0023 | 0.0023 | 0.0021 | 0.0020 | 0.0017 | 0.0019 |
| | 0.0024 | 0.0029 | 0.0029 | 0.0019 | 0.0018 | 0.0022 |

AVERAGE ANNUAL TOTALS & (STD. DEVIATIONS) FOR YEARS 1 THROUGH 10

| | INCHES | | CU. FEET | PERCENT |
|---|---|---|---|---|
| PRECIPITATION | 48.22 | ( 6.597) | 2590839.7 | 100.00 |
| RUNOFF | 1.621 | ( 1.1149) | 87086.38 | 3.361 |
| EVAPOTRANSPIRATION | 33.461 | ( 2.0580) | 1797641.12 | 69.384 |
| LATERAL DRAINAGE COLLECTED FROM LAYER 8 | 12.69570 | ( 4.09591) | 682063.875 | 26.32598 |
| PERCOLATION/LEAKAGE THROUGH LAYER 9 | 0.00001 | ( 0.00000) | 0.492 | 0.00002 |
| AVERAGE HEAD ON TOP OF LAYER 9 | 0.004 | ( 0.001) | | |
| PERCOLATION/LEAKAGE THROUGH LAYER 11 | 0.00000 | ( 0.00000) | 0.000 | 0.00000 |
| CHANGE IN WATER STORAGE | 0.448 | ( 5.3294) | 24048.57 | 0.928 |

**********

PEAK DAILY VALUES FOR YEARS 1 THROUGH 10

| | (INCHES) | (CU. FT.) |
|---|---|---|
| PRECIPITATION | 3.08 | 165469.922 |
| RUNOFF | 1.083 | 58196.0977 |
| DRAINAGE COLLECTED FROM LAYER 8 | 0.09418 | 5059.86230 |
| PERCOLATION/LEAKAGE THROUGH LAYER 9 | 0.000000 | 0.00312 |
| AVERAGE HEAD ON TOP OF LAYER 9 | 0.012 | |
| MAXIMUM HEAD ON TOP OF LAYER 9 | 0.024 | |
| LOCATION OF MAXIMUM HEAD IN LAYER 8 (DISTANCE FROM DRAIN) | 0.9 FEET | |
| PERCOLATION/LEAKAGE THROUGH LAYER 11 | 0.000000 | 0.00000 |
| SNOW WATER | 2.86 | 153667.5620 |

| | |
|---|---|
| MAXIMUM VEG. SOIL WATER (VOL/VOL) | 0.4432 |
| 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 10

| LAYER | (INCHES) | (VOL/VOL) |
|---|---|---|
| 1 | 18.9949 | 0.2968 |
| 2 | 17.9631 | 0.2807 |
| 3 | 18.2887 | 0.2858 |
| 4 | 19.1809 | 0.2997 |
| 5 | 20.2272 | 0.3160 |
| 6 | 19.8867 | 0.3107 |
| 7 | 3.7608 | 0.3134 |
| 8 | 0.0058 | 0.0343 |
| 9 | 0.0000 | 0.0000 |
| 10 | 0.0365 | 0.6086 |
| 11 | 10.6078 | 0.2210 |
| SNOW WATER | 0.000 | |

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```
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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\mssfgd\MSSRAIN.D4
TEMPERATURE DATA FILE:        C:\WMM\HELP3-7\mssfgd\MSSTEMP.D7
SOLAR RADIATION DATA FILE:    C:\wmm\help3-7\mssfgd\MSSRAD.D13
EVAPOTRANSPIRATION DATA:      C:\wmm\help3-7\mssfgd\MSSOPEN.D11
SOIL AND DESIGN DATA FILE:    C:\wmm\help3-7\mssfgd\C1-64.D10
OUTPUT DATA FILE:             C:\WMM\HELP3-7\mssfgd\c1-64.OUT


TIME:   8:47     DATE:  12/16/2005


******************************************************************************

     TITLE:  MSSFGD LF - Cell 1 - 64 feet of Waste - 60 mil HDPE + 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   0
          THICKNESS                    =     18.00   INCHES
          POROSITY                     =      0.4900 VOL/VOL
          FIELD CAPACITY               =      0.2890 VOL/VOL
          WILTING POINT                =      0.1260 VOL/VOL
          INITIAL SOIL WATER CONTENT   =      0.2602 VOL/VOL
          EFFECTIVE SAT. HYD. COND.    =  0.249000004000E-03 CM/SEC
```

LAYER 2
---

TYPE 1 - VERTICAL PERCOLATION LAYER
MATERIAL TEXTURE NUMBER 0

| | | | |
|---|---|---|---|
| THICKNESS | = | 76.80 | INCHES |
| POROSITY | = | 0.5600 | VOL/VOL |
| FIELD CAPACITY | = | 0.2660 | VOL/VOL |
| WILTING POINT | = | 0.0900 | VOL/VOL |
| INITIAL SOIL WATER CONTENT | = | 0.3288 | VOL/VOL |
| EFFECTIVE SAT. HYD. COND. | = | 0.176000001000E-03 | CM/SEC |

LAYER 3
---

TYPE 1 - VERTICAL PERCOLATION LAYER
MATERIAL TEXTURE NUMBER 0

| | | | |
|---|---|---|---|
| THICKNESS | = | 76.80 | INCHES |
| POROSITY | = | 0.5600 | VOL/VOL |
| FIELD CAPACITY | = | 0.2660 | VOL/VOL |
| WILTING POINT | = | 0.0900 | VOL/VOL |
| INITIAL SOIL WATER CONTENT | = | 0.2927 | VOL/VOL |
| EFFECTIVE SAT. HYD. COND. | = | 0.176000001000E-03 | CM/SEC |

LAYER 4
---

TYPE 1 - VERTICAL PERCOLATION LAYER
MATERIAL TEXTURE NUMBER 0

| | | | |
|---|---|---|---|
| THICKNESS | = | 76.80 | INCHES |
| POROSITY | = | 0.5600 | VOL/VOL |
| FIELD CAPACITY | = | 0.2660 | VOL/VOL |
| WILTING POINT | = | 0.0900 | VOL/VOL |
| INITIAL SOIL WATER CONTENT | = | 0.2918 | VOL/VOL |
| EFFECTIVE SAT. HYD. COND. | = | 0.176000001000E-03 | CM/SEC |

LAYER 5
---

TYPE 1 - VERTICAL PERCOLATION LAYER
MATERIAL TEXTURE NUMBER 0

| | | | |
|---|---|---|---|
| THICKNESS | = | 76.80 | INCHES |
| POROSITY | = | 0.5600 | VOL/VOL |
| FIELD CAPACITY | = | 0.2660 | VOL/VOL |
| WILTING POINT | = | 0.0900 | VOL/VOL |
| INITIAL SOIL WATER CONTENT | = | 0.2843 | VOL/VOL |

```
EFFECTIVE SAT. HYD. COND.    =  0.176000001000E-03 CM/SEC


                        LAYER  6
                        --------

          TYPE 1 - VERTICAL PERCOLATION LAYER
              MATERIAL TEXTURE NUMBER   0
THICKNESS                    =     76.80   INCHES
POROSITY                     =      0.5600 VOL/VOL
FIELD CAPACITY               =      0.2660 VOL/VOL
WILTING POINT                =      0.0900 VOL/VOL
INITIAL SOIL WATER CONTENT   =      0.2660 VOL/VOL
EFFECTIVE SAT. HYD. COND.    =  0.176000001000E-03 CM/SEC


                        LAYER  7
                        --------

          TYPE 1 - VERTICAL PERCOLATION LAYER
              MATERIAL TEXTURE NUMBER   0
THICKNESS                    =     76.80   INCHES
POROSITY                     =      0.5600 VOL/VOL
FIELD CAPACITY               =      0.2660 VOL/VOL
WILTING POINT                =      0.0900 VOL/VOL
INITIAL SOIL WATER CONTENT   =      0.2660 VOL/VOL
EFFECTIVE SAT. HYD. COND.    =  0.176000001000E-03 CM/SEC


                        LAYER  8
                        --------

          TYPE 1 - VERTICAL PERCOLATION LAYER
              MATERIAL TEXTURE NUMBER   0
THICKNESS                    =     76.80   INCHES
POROSITY                     =      0.5600 VOL/VOL
FIELD CAPACITY               =      0.2660 VOL/VOL
WILTING POINT                =      0.0900 VOL/VOL
INITIAL SOIL WATER CONTENT   =      0.2660 VOL/VOL
EFFECTIVE SAT. HYD. COND.    =  0.176000001000E-03 CM/SEC


                        LAYER  9
                        --------

          TYPE 1 - VERTICAL PERCOLATION LAYER
              MATERIAL TEXTURE NUMBER   0
THICKNESS                    =     76.80   INCHES
POROSITY                     =      0.5600 VOL/VOL
```

```
FIELD CAPACITY                =      0.2660 VOL/VOL
WILTING POINT                 =      0.0900 VOL/VOL
INITIAL SOIL WATER CONTENT    =      0.2663 VOL/VOL
EFFECTIVE SAT. HYD. COND.     =  0.176000001000E-03 CM/SEC
```

LAYER 10
---

```
          TYPE 1 - VERTICAL PERCOLATION LAYER
              MATERIAL TEXTURE NUMBER   0
THICKNESS                     =     76.80   INCHES
POROSITY                      =      0.5600 VOL/VOL
FIELD CAPACITY                =      0.2660 VOL/VOL
WILTING POINT                 =      0.0900 VOL/VOL
INITIAL SOIL WATER CONTENT    =      0.2661 VOL/VOL
EFFECTIVE SAT. HYD. COND.     =  0.176000001000E-03 CM/SEC
```

LAYER 11
---

```
          TYPE 1 - VERTICAL PERCOLATION LAYER
              MATERIAL TEXTURE NUMBER   0
THICKNESS                     =     76.80   INCHES
POROSITY                      =      0.5600 VOL/VOL
FIELD CAPACITY                =      0.2660 VOL/VOL
WILTING POINT                 =      0.0900 VOL/VOL
INITIAL SOIL WATER CONTENT    =      0.2660 VOL/VOL
EFFECTIVE SAT. HYD. COND.     =  0.176000001000E-03 CM/SEC
```

LAYER 12
---

```
          TYPE 1 - VERTICAL PERCOLATION LAYER
              MATERIAL TEXTURE NUMBER   0
THICKNESS                     =     12.00   INCHES
POROSITY                      =      0.5000 VOL/VOL
FIELD CAPACITY                =      0.2720 VOL/VOL
WILTING POINT                 =      0.0900 VOL/VOL
INITIAL SOIL WATER CONTENT    =      0.2730 VOL/VOL
EFFECTIVE SAT. HYD. COND.     =  0.551999983000E-04 CM/SEC
```

LAYER 13
---

TYPE 2 - LATERAL DRAINAGE LAYER
MATERIAL TEXTURE NUMBER 20

| | | | |
|---|---|---|---|
| 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 | = | 0.0189 | VOL/VOL |
| EFFECTIVE SAT. HYD. COND. | = | 10.0000000000 | CM/SEC |
| SLOPE | = | 1.50 | PERCENT |
| DRAINAGE LENGTH | = | 110.0 | FEET |

LAYER 14
---

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 | = | 2 - EXCELLENT | |

LAYER 15
---

TYPE 1 - VERTICAL PERCOLATION LAYER
MATERIAL TEXTURE NUMBER 0

| | | | |
|---|---|---|---|
| THICKNESS | = | 0.06 | INCHES |
| POROSITY | = | 0.7500 | VOL/VOL |
| FIELD CAPACITY | = | 0.7470 | VOL/VOL |
| WILTING POINT | = | 0.4000 | VOL/VOL |
| INITIAL SOIL WATER CONTENT | = | 0.6086 | VOL/VOL |
| EFFECTIVE SAT. HYD. COND. | = | 0.999999997000E-06 | CM/SEC |

LAYER 16
---

TYPE 1 - VERTICAL PERCOLATION LAYER
MATERIAL TEXTURE NUMBER 0

| | | | |
|---|---|---|---|
| THICKNESS | = | 48.00 | INCHES |
| POROSITY | = | 0.5300 | VOL/VOL |
| FIELD CAPACITY | = | 0.2210 | VOL/VOL |
| WILTING POINT | = | 0.1010 | VOL/VOL |
| INITIAL SOIL WATER CONTENT | = | 0.2210 | VOL/VOL |

EFFECTIVE SAT. HYD. COND. = 0.372000010000E-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 | = | 14.800 | ACRES |
| EVAPORATIVE ZONE DEPTH | = | 10.0 | INCHES |
| INITIAL WATER IN EVAPORATIVE ZONE | = | 2.372 | INCHES |
| UPPER LIMIT OF EVAPORATIVE STORAGE | = | 4.900 | INCHES |
| LOWER LIMIT OF EVAPORATIVE STORAGE | = | 1.260 | INCHES |
| INITIAL SNOW WATER | = | 0.000 | INCHES |
| INITIAL WATER IN LAYER MATERIALS | = | 233.185 | INCHES |
| TOTAL INITIAL WATER | = | 233.185 | INCHES |
| TOTAL SUBSURFACE INFLOW | = | 0.00 | INCHES/YEAR |

## EVAPOTRANSPIRATION AND WEATHER DATA

NOTE: EVAPOTRANSPIRATION DATA WAS OBTAINED FROM
CHARLOTTE NORTH CAROLINA

| | | | |
|---|---|---|---|
| STATION LATITUDE | = | 35.60 | DEGREES |
| MAXIMUM LEAF AREA INDEX | = | 0.00 | |
| START OF GROWING SEASON (JULIAN DATE) | = | 83 | |
| END OF GROWING SEASON (JULIAN DATE) | = | 312 | |
| EVAPORATIVE ZONE DEPTH | = | 10.0 | INCHES |
| AVERAGE ANNUAL WIND SPEED | = | 7.50 | MPH |
| AVERAGE 1ST QUARTER RELATIVE HUMIDITY | = | 64.00 | % |
| AVERAGE 2ND QUARTER RELATIVE HUMIDITY | = | 67.00 | % |
| AVERAGE 3RD QUARTER RELATIVE HUMIDITY | = | 74.00 | % |
| AVERAGE 4TH QUARTER RELATIVE HUMIDITY | = | 70.00 | % |

NOTE: PRECIPITATION DATA WAS SYNTHETICALLY GENERATED USING
COEFFICIENTS FOR CHARLOTTE NORTH CAROLINA

NORMAL MEAN MONTHLY PRECIPITATION (INCHES)

| JAN/JUL | FEB/AUG | MAR/SEP | APR/OCT | MAY/NOV | JUN/DEC |
|---|---|---|---|---|---|
| 4.10 | 4.10 | 4.70 | 3.80 | 3.60 | 3.90 |
| 5.00 | 5.60 | 3.80 | 3.40 | 3.10 | 4.10 |

NOTE: TEMPERATURE DATA WAS SYNTHETICALLY GENERATED USING
COEFFICIENTS FOR CHARLOTTE NORTH CAROLINA

NORMAL MEAN MONTHLY TEMPERATURE (DEGREES FAHRENHEIT)

| JAN/JUL | FEB/AUG | MAR/SEP | APR/OCT | MAY/NOV | JUN/DEC |
|---|---|---|---|---|---|
| 41.00 | 42.00 | 49.00 | 59.00 | 67.00 | 75.00 |
| 77.00 | 76.00 | 70.50 | 60.50 | 49.00 | 41.50 |

NOTE: SOLAR RADIATION DATA WAS SYNTHETICALLY GENERATED USING
COEFFICIENTS FOR CHARLOTTE NORTH CAROLINA
AND STATION LATITUDE = 35.20 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 | 3.68 | 3.72 | 5.27 | 3.14 | 3.45 | 4.68 |
| | 4.56 | 5.51 | 3.73 | 3.69 | 2.78 | 4.00 |
| STD. DEVIATIONS | 1.86 | 1.79 | 1.54 | 2.32 | 2.04 | 1.24 |
| | 1.67 | 3.79 | 1.87 | 2.26 | 1.48 | 2.09 |
| **RUNOFF** | | | | | | |
| TOTALS | 0.005 | 0.000 | 0.001 | 0.000 | 0.000 | 0.000 |
| | 0.037 | 0.007 | 0.000 | 0.000 | 0.000 | 0.011 |
| STD. DEVIATIONS | 0.015 | 0.000 | 0.004 | 0.000 | 0.000 | 0.000 |
| | 0.116 | 0.021 | 0.000 | 0.000 | 0.000 | 0.036 |
| **EVAPOTRANSPIRATION** | | | | | | |
| TOTALS | 1.800 | 2.168 | 3.335 | 3.037 | 2.904 | 3.636 |
| | 3.984 | 3.461 | 2.962 | 2.233 | 1.897 | 1.356 |
| STD. DEVIATIONS | 0.243 | 0.173 | 0.235 | 1.444 | 1.818 | 1.436 |
| | 1.373 | 1.511 | 1.230 | 0.834 | 0.381 | 0.143 |
| **LATERAL DRAINAGE COLLECTED FROM LAYER 13** | | | | | | |
| TOTALS | 0.9167 | 1.0450 | 1.2095 | 1.0826 | 1.1437 | 0.9271 |
| | 1.1911 | 1.1351 | 1.0808 | 1.2049 | 0.7904 | 1.0567 |

| | | | | | | |
|---|---|---|---|---|---|---|
| STD. DEVIATIONS | 0.6553 | 0.7831 | 0.9061 | 0.7729 | 0.8089 | 0.5485 |
| | 0.6926 | 0.6340 | 0.6548 | 0.7145 | 0.3880 | 0.5655 |

PERCOLATION/LEAKAGE THROUGH LAYER 14

| | | | | | | |
|---|---|---|---|---|---|---|
| TOTALS | 0.0000 | 0.0000 | 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 |

PERCOLATION/LEAKAGE THROUGH LAYER 16

| | | | | | | |
|---|---|---|---|---|---|---|
| TOTALS | 0.0000 | 0.0000 | 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 |

---

AVERAGES OF MONTHLY AVERAGED DAILY HEADS (INCHES)

---

DAILY AVERAGE HEAD ON TOP OF LAYER 14

| | | | | | | |
|---|---|---|---|---|---|---|
| AVERAGES | 0.0038 | 0.0048 | 0.0050 | 0.0047 | 0.0048 | 0.0040 |
| | 0.0050 | 0.0047 | 0.0047 | 0.0050 | 0.0034 | 0.0044 |
| STD. DEVIATIONS | 0.0027 | 0.0035 | 0.0038 | 0.0033 | 0.0034 | 0.0024 |
| | 0.0029 | 0.0026 | 0.0028 | 0.0030 | 0.0017 | 0.0024 |

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AVERAGE ANNUAL TOTALS & (STD. DEVIATIONS) FOR YEARS 1 THROUGH 10

| | INCHES | | CU. FEET | PERCENT |
|---|---|---|---|---|
| PRECIPITATION | 48.22 | ( 6.597) | 2590839.7 | 100.00 |
| RUNOFF | 0.061 | ( 0.1704) | 3278.97 | 0.127 |
| EVAPOTRANSPIRATION | 32.775 | ( 2.0214) | 1760782.25 | 67.962 |
| LATERAL DRAINAGE COLLECTED FROM LAYER 13 | 12.78370 | ( 6.71859) | 686791.562 | 26.50846 |
| PERCOLATION/LEAKAGE THROUGH LAYER 14 | 0.00001 | ( 0.00000) | 0.494 | 0.00002 |
| AVERAGE HEAD ON TOP OF LAYER 14 | 0.005 | ( 0.002) | | |
| PERCOLATION/LEAKAGE THROUGH LAYER 16 | 0.00000 | ( 0.00000) | 0.000 | 0.00000 |
| CHANGE IN WATER STORAGE | 2.606 | ( 9.4685) | 139987.44 | 5.403 |

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PEAK DAILY VALUES FOR YEARS 1 THROUGH 10

| | (INCHES) | (CU. FT.) |
|---|---|---|
| PRECIPITATION | 3.08 | 165469.922 |
| RUNOFF | 0.365 | 19626.3301 |
| DRAINAGE COLLECTED FROM LAYER 13 | 0.10340 | 5554.82666 |
| PERCOLATION/LEAKAGE THROUGH LAYER 14 | 0.000000 | 0.00340 |
| AVERAGE HEAD ON TOP OF LAYER 14 | 0.013 | |
| MAXIMUM HEAD ON TOP OF LAYER 14 | 0.027 | |
| LOCATION OF MAXIMUM HEAD IN LAYER 13 (DISTANCE FROM DRAIN) | 0.6 FEET | |
| PERCOLATION/LEAKAGE THROUGH LAYER 16 | 0.000000 | 0.00000 |
| SNOW WATER | 2.86 | 153667.5620 |

| | |
|---|---|
| MAXIMUM VEG. SOIL WATER (VOL/VOL) | 0.4392 |
| MINIMUM VEG. SOIL WATER (VOL/VOL) | 0.1260 |

*** 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 | 4.6958 | 0.2609 |
| 2 | 23.5605 | 0.3068 |
| 3 | 22.3933 | 0.2916 |
| 4 | 23.1624 | 0.3016 |
| 5 | 23.4390 | 0.3052 |
| 6 | 24.7318 | 0.3220 |
| 7 | 24.5904 | 0.3202 |
| 8 | 25.1089 | 0.3269 |
| 9 | 25.2259 | 0.3285 |
| 10 | 23.6133 | 0.3075 |
| 11 | 24.8100 | 0.3230 |
| 12 | 3.2640 | 0.2720 |
| 13 | 0.0024 | 0.0139 |
| 14 | 0.0000 | 0.0000 |
| 15 | 0.0365 | 0.6086 |
| 16 | 10.6078 | 0.2210 |
| 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\mssfgd\MSSRAIN.D4
TEMPERATURE DATA FILE:        C:\WMM\HELP3-7\mssfgd\MSSTEMP.D7
SOLAR RADIATION DATA FILE:    C:\wmm\help3-7\mssfgd\MSSRAD.D13
EVAPOTRANSPIRATION DATA:      C:\wmm\help3-7\mssfgd\MSSOPEN.D11
SOIL AND DESIGN DATA FILE:    C:\wmm\help3-7\mssfgd\C2-10.D10
OUTPUT DATA FILE:             C:\WMM\HELP3-7\mssfgd\C2-10.OUT


TIME:   9:16     DATE:  12/16/2005


******************************************************************************

     TITLE:  MSSFGD LF - Cell 2 - 10 feet of Waste - 60 mil HDPE + 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   0
          THICKNESS                    =     20.00   INCHES
          POROSITY                     =      0.5600 VOL/VOL
          FIELD CAPACITY               =      0.2660 VOL/VOL
          WILTING POINT                =      0.0900 VOL/VOL
          INITIAL SOIL WATER CONTENT   =      0.2860 VOL/VOL
          EFFECTIVE SAT. HYD. COND.    =  0.176000001000E-03 CM/SEC
```

LAYER 2
---

TYPE 1 - VERTICAL PERCOLATION LAYER
MATERIAL TEXTURE NUMBER 0

| | | | |
|---|---|---|---|
| THICKNESS | = | 20.00 | INCHES |
| POROSITY | = | 0.5600 | VOL/VOL |
| FIELD CAPACITY | = | 0.2660 | VOL/VOL |
| WILTING POINT | = | 0.0900 | VOL/VOL |
| INITIAL SOIL WATER CONTENT | = | 0.3659 | VOL/VOL |
| EFFECTIVE SAT. HYD. COND. | = | 0.176000001000E-03 | CM/SEC |

LAYER 3
---

TYPE 1 - VERTICAL PERCOLATION LAYER
MATERIAL TEXTURE NUMBER 0

| | | | |
|---|---|---|---|
| THICKNESS | = | 20.00 | INCHES |
| POROSITY | = | 0.5600 | VOL/VOL |
| FIELD CAPACITY | = | 0.2660 | VOL/VOL |
| WILTING POINT | = | 0.0900 | VOL/VOL |
| INITIAL SOIL WATER CONTENT | = | 0.3043 | VOL/VOL |
| EFFECTIVE SAT. HYD. COND. | = | 0.176000001000E-03 | CM/SEC |

LAYER 4
---

TYPE 1 - VERTICAL PERCOLATION LAYER
MATERIAL TEXTURE NUMBER 0

| | | | |
|---|---|---|---|
| THICKNESS | = | 20.00 | INCHES |
| POROSITY | = | 0.5600 | VOL/VOL |
| FIELD CAPACITY | = | 0.2660 | VOL/VOL |
| WILTING POINT | = | 0.0900 | VOL/VOL |
| INITIAL SOIL WATER CONTENT | = | 0.3093 | VOL/VOL |
| EFFECTIVE SAT. HYD. COND. | = | 0.176000001000E-03 | CM/SEC |

LAYER 5
---

TYPE 1 - VERTICAL PERCOLATION LAYER
MATERIAL TEXTURE NUMBER 0

| | | | |
|---|---|---|---|
| THICKNESS | = | 20.00 | INCHES |
| POROSITY | = | 0.5600 | VOL/VOL |
| FIELD CAPACITY | = | 0.2660 | VOL/VOL |
| WILTING POINT | = | 0.0900 | VOL/VOL |
| INITIAL SOIL WATER CONTENT | = | 0.2893 | VOL/VOL |

```
EFFECTIVE SAT. HYD. COND.    =  0.176000001000E-03 CM/SEC


                        LAYER  6
                        --------

          TYPE 1 - VERTICAL PERCOLATION LAYER
              MATERIAL TEXTURE NUMBER   0
THICKNESS                    =     20.00   INCHES
POROSITY                     =      0.5600 VOL/VOL
FIELD CAPACITY               =      0.2660 VOL/VOL
WILTING POINT                =      0.0900 VOL/VOL
INITIAL SOIL WATER CONTENT   =      0.2928 VOL/VOL
EFFECTIVE SAT. HYD. COND.    =  0.176000001000E-03 CM/SEC


                        LAYER  7
                        --------

          TYPE 1 - VERTICAL PERCOLATION LAYER
              MATERIAL TEXTURE NUMBER   0
THICKNESS                    =     12.00   INCHES
POROSITY                     =      0.5000 VOL/VOL
FIELD CAPACITY               =      0.2720 VOL/VOL
WILTING POINT                =      0.0900 VOL/VOL
INITIAL SOIL WATER CONTENT   =      0.2720 VOL/VOL
EFFECTIVE SAT. HYD. COND.    =  0.551999983000E-04 CM/SEC


                        LAYER  8
                        --------

            TYPE 2 - LATERAL DRAINAGE LAYER
              MATERIAL TEXTURE NUMBER  20
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   =      0.0129 VOL/VOL
EFFECTIVE SAT. HYD. COND.    =   10.0000000000     CM/SEC
SLOPE                        =      1.50   PERCENT
DRAINAGE LENGTH              =    110.0    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 | = | 2 - EXCELLENT | |

LAYER 10
---

TYPE 1 - VERTICAL PERCOLATION LAYER
MATERIAL TEXTURE NUMBER 0

| | | | |
|---|---|---|---|
| THICKNESS | = | 0.06 | INCHES |
| POROSITY | = | 0.7500 | VOL/VOL |
| FIELD CAPACITY | = | 0.7470 | VOL/VOL |
| WILTING POINT | = | 0.4000 | VOL/VOL |
| INITIAL SOIL WATER CONTENT | = | 0.6086 | VOL/VOL |
| EFFECTIVE SAT. HYD. COND. | = | 0.999999997000E-06 | CM/SEC |

LAYER 11
---

TYPE 1 - VERTICAL PERCOLATION LAYER
MATERIAL TEXTURE NUMBER 0

| | | | |
|---|---|---|---|
| THICKNESS | = | 48.00 | INCHES |
| POROSITY | = | 0.5300 | VOL/VOL |
| FIELD CAPACITY | = | 0.2210 | VOL/VOL |
| WILTING POINT | = | 0.1010 | VOL/VOL |
| INITIAL SOIL WATER CONTENT | = | 0.2210 | VOL/VOL |
| EFFECTIVE SAT. HYD. COND. | = | 0.372000010000E-03 | CM/SEC |

GENERAL DESIGN AND EVAPORATIVE ZONE DATA
---

NOTE: SCS RUNOFF CURVE NUMBER WAS USER-SPECIFIED.

| | | | |
|---|---|---|---|
| SCS RUNOFF CURVE NUMBER | = | 82.00 | |
| FRACTION OF AREA ALLOWING RUNOFF | = | 100.0 | PERCENT |
| AREA PROJECTED ON HORIZONTAL PLANE | = | 17.100 | ACRES |
| EVAPORATIVE ZONE DEPTH | = | 10.0 | INCHES |
| INITIAL WATER IN EVAPORATIVE ZONE | = | 2.351 | INCHES |
| UPPER LIMIT OF EVAPORATIVE STORAGE | = | 5.600 | INCHES |
| LOWER LIMIT OF EVAPORATIVE STORAGE | = | 0.900 | INCHES |
| INITIAL SNOW WATER | = | 0.000 | INCHES |

| | | | |
|---|---|---|---|
| INITIAL WATER IN LAYER MATERIALS | = | 50.863 | INCHES |
| TOTAL INITIAL WATER | = | 50.863 | INCHES |
| TOTAL SUBSURFACE INFLOW | = | 0.00 | INCHES/YEAR |

## EVAPOTRANSPIRATION AND WEATHER DATA

NOTE: EVAPOTRANSPIRATION DATA WAS OBTAINED FROM
CHARLOTTE NORTH CAROLINA

| | | | |
|---|---|---|---|
| STATION LATITUDE | = | 35.60 | DEGREES |
| MAXIMUM LEAF AREA INDEX | = | 0.00 | |
| START OF GROWING SEASON (JULIAN DATE) | = | 83 | |
| END OF GROWING SEASON (JULIAN DATE) | = | 312 | |
| EVAPORATIVE ZONE DEPTH | = | 10.0 | INCHES |
| AVERAGE ANNUAL WIND SPEED | = | 7.50 | MPH |
| AVERAGE 1ST QUARTER RELATIVE HUMIDITY | = | 64.00 | % |
| AVERAGE 2ND QUARTER RELATIVE HUMIDITY | = | 67.00 | % |
| AVERAGE 3RD QUARTER RELATIVE HUMIDITY | = | 74.00 | % |
| AVERAGE 4TH QUARTER RELATIVE HUMIDITY | = | 70.00 | % |

NOTE: PRECIPITATION DATA WAS SYNTHETICALLY GENERATED USING
COEFFICIENTS FOR CHARLOTTE NORTH CAROLINA

NORMAL MEAN MONTHLY PRECIPITATION (INCHES)

| JAN/JUL | FEB/AUG | MAR/SEP | APR/OCT | MAY/NOV | JUN/DEC |
|---|---|---|---|---|---|
| 4.10 | 4.10 | 4.70 | 3.80 | 3.60 | 3.90 |
| 5.00 | 5.60 | 3.80 | 3.40 | 3.10 | 4.10 |

NOTE: TEMPERATURE DATA WAS SYNTHETICALLY GENERATED USING
COEFFICIENTS FOR CHARLOTTE NORTH CAROLINA

NORMAL MEAN MONTHLY TEMPERATURE (DEGREES FAHRENHEIT)

| JAN/JUL | FEB/AUG | MAR/SEP | APR/OCT | MAY/NOV | JUN/DEC |
|---|---|---|---|---|---|
| 41.00 | 42.00 | 49.00 | 59.00 | 67.00 | 75.00 |
| 77.00 | 76.00 | 70.50 | 60.50 | 49.00 | 41.50 |

NOTE: SOLAR RADIATION DATA WAS SYNTHETICALLY GENERATED USING
COEFFICIENTS FOR CHARLOTTE NORTH CAROLINA
AND STATION LATITUDE = 35.20 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 | 4.00 | 4.06 | 4.67 | 4.27 | 3.64 | 4.31 |
| | 5.01 | 5.15 | 3.58 | 3.23 | 3.08 | 4.25 |
| STD. DEVIATIONS | 1.99 | 1.84 | 2.23 | 1.97 | 1.79 | 2.10 |
| | 2.36 | 3.06 | 2.62 | 2.06 | 1.67 | 1.89 |
| **RUNOFF** | | | | | | |
| TOTALS | 0.198 | 0.196 | 0.184 | 0.147 | 0.091 | 0.150 |
| | 0.183 | 0.396 | 0.282 | 0.191 | 0.081 | 0.214 |
| STD. DEVIATIONS | 0.345 | 0.464 | 0.352 | 0.257 | 0.232 | 0.220 |
| | 0.380 | 0.572 | 0.585 | 0.325 | 0.166 | 0.317 |
| **EVAPOTRANSPIRATION** | | | | | | |
| TOTALS | 1.784 | 2.033 | 3.188 | 3.521 | 3.384 | 3.532 |
| | 4.059 | 3.490 | 2.684 | 2.166 | 1.642 | 1.448 |
| STD. DEVIATIONS | 0.262 | 0.349 | 0.451 | 0.984 | 1.265 | 1.471 |
| | 1.393 | 1.270 | 1.158 | 0.794 | 0.523 | 0.231 |
| **LATERAL DRAINAGE COLLECTED FROM LAYER 8** | | | | | | |
| TOTALS | 0.9477 | 1.2384 | 1.7420 | 1.8241 | 1.6844 | 1.3529 |
| | 1.1508 | 0.9703 | 0.7550 | 0.7426 | 0.7529 | 0.8245 |
| STD. DEVIATIONS | 0.8872 | 1.0887 | 1.2738 | 1.0930 | 0.7796 | 0.4583 |
| | 0.3902 | 0.2899 | 0.3717 | 0.6177 | 0.7254 | 0.6298 |
| **PERCOLATION/LEAKAGE THROUGH LAYER 9** | | | | | | |
| TOTALS | 0.0000 | 0.0000 | 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 |
| **PERCOLATION/LEAKAGE THROUGH LAYER 11** | | | | | | |
| TOTALS | 0.0000 | 0.0000 | 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 |

AVERAGES OF MONTHLY AVERAGED DAILY HEADS (INCHES)

DAILY AVERAGE HEAD ON TOP OF LAYER 9

| | | | | | | |
|---|---|---|---|---|---|---|
| AVERAGES | 0.0040 | 0.0057 | 0.0073 | 0.0079 | 0.0070 | 0.0058 |
| | 0.0048 | 0.0040 | 0.0033 | 0.0031 | 0.0032 | 0.0034 |
| STD. DEVIATIONS | 0.0037 | 0.0050 | 0.0053 | 0.0047 | 0.0033 | 0.0020 |
| | 0.0016 | 0.0012 | 0.0016 | 0.0026 | 0.0031 | 0.0026 |

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AVERAGE ANNUAL TOTALS & (STD. DEVIATIONS) FOR YEARS 1 THROUGH 100

| | INCHES | | CU. FEET | PERCENT |
|---|---|---|---|---|
| PRECIPITATION | 49.24 | ( 7.134) | 3056394.2 | 100.00 |
| RUNOFF | 2.313 | ( 1.3402) | 143568.44 | 4.697 |
| EVAPOTRANSPIRATION | 32.931 | ( 3.5716) | 2044120.12 | 66.880 |
| LATERAL DRAINAGE COLLECTED FROM LAYER 8 | 13.98553 | ( 4.28253) | 868123.875 | 28.40353 |
| PERCOLATION/LEAKAGE THROUGH LAYER 9 | 0.00001 | ( 0.00000) | 0.610 | 0.00002 |
| AVERAGE HEAD ON TOP OF LAYER 9 | 0.005 | ( 0.002) | | |
| PERCOLATION/LEAKAGE THROUGH LAYER 11 | 0.00001 | ( 0.00005) | 0.549 | 0.00002 |
| CHANGE IN WATER STORAGE | 0.009 | ( 4.0086) | 580.44 | 0.019 |

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PEAK DAILY VALUES FOR YEARS 1 THROUGH 100

| | (INCHES) | (CU. FT.) |
|---|---|---|
| PRECIPITATION | 5.58 | 346367.344 |
| RUNOFF | 2.318 | 143886.8910 |
| DRAINAGE COLLECTED FROM LAYER 8 | 0.33729 | 20936.87700 |
| PERCOLATION/LEAKAGE THROUGH LAYER 9 | 0.000000 | 0.01285 |
| AVERAGE HEAD ON TOP OF LAYER 9 | 0.044 | |
| MAXIMUM HEAD ON TOP OF LAYER 9 | 0.086 | |
| LOCATION OF MAXIMUM HEAD IN LAYER 8 (DISTANCE FROM DRAIN) | 2.1 FEET | |
| PERCOLATION/LEAKAGE THROUGH LAYER 11 | 0.000295 | 18.28252 |
| SNOW WATER | 3.87 | 240281.1720 |

MAXIMUM VEG. SOIL WATER (VOL/VOL) 0.5237

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 | 5.6531 | 0.2827 |
| 2 | 6.5462 | 0.3273 |
| 3 | 5.9146 | 0.2957 |
| 4 | 6.2078 | 0.3104 |
| 5 | 6.5859 | 0.3293 |
| 6 | 6.6485 | 0.3324 |
| 7 | 3.5965 | 0.2997 |
| 8 | 0.0017 | 0.0100 |
| 9 | 0.0000 | 0.0000 |
| 10 | 0.0365 | 0.6086 |
| 11 | 10.6078 | 0.2210 |
| 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\mssfgd\MSSRAIN.D4
TEMPERATURE DATA FILE:        C:\WMM\HELP3-7\mssfgd\MSSTEMP.D7
SOLAR RADIATION DATA FILE:    C:\wmm\help3-7\mssfgd\MSSRAD.D13
EVAPOTRANSPIRATION DATA:      C:\wmm\help3-7\mssfgd\MSSOPEN.D11
SOIL AND DESIGN DATA FILE:    C:\wmm\help3-7\mssfgd\C2-42.D10
OUTPUT DATA FILE:             C:\WMM\HELP3-7\mssfgd\c2-42.OUT


TIME:   8:58     DATE:  12/16/2005


******************************************************************************

     TITLE:  MSSFGD LF - Cell 2 - 42 feet of Waste - 60 mil HDPE + 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   0
          THICKNESS                  =     50.40   INCHES
          POROSITY                   =      0.5600 VOL/VOL
          FIELD CAPACITY             =      0.2660 VOL/VOL
          WILTING POINT              =      0.0900 VOL/VOL
          INITIAL SOIL WATER CONTENT =      0.3261 VOL/VOL
          EFFECTIVE SAT. HYD. COND.  =  0.176000001000E-03 CM/SEC
```

```
                    LAYER  2
                    --------

          TYPE 1 - VERTICAL PERCOLATION LAYER
              MATERIAL TEXTURE NUMBER   0
THICKNESS                     =     50.40   INCHES
POROSITY                      =      0.5600 VOL/VOL
FIELD CAPACITY                =      0.2660 VOL/VOL
WILTING POINT                 =      0.0900 VOL/VOL
INITIAL SOIL WATER CONTENT    =      0.2981 VOL/VOL
EFFECTIVE SAT. HYD. COND.     =  0.176000001000E-03 CM/SEC


                    LAYER  3
                    --------

          TYPE 1 - VERTICAL PERCOLATION LAYER
              MATERIAL TEXTURE NUMBER   0
THICKNESS                     =     50.40   INCHES
POROSITY                      =      0.5600 VOL/VOL
FIELD CAPACITY                =      0.2660 VOL/VOL
WILTING POINT                 =      0.0900 VOL/VOL
INITIAL SOIL WATER CONTENT    =      0.2740 VOL/VOL
EFFECTIVE SAT. HYD. COND.     =  0.176000001000E-03 CM/SEC


                    LAYER  4
                    --------

          TYPE 1 - VERTICAL PERCOLATION LAYER
              MATERIAL TEXTURE NUMBER   0
THICKNESS                     =     50.40   INCHES
POROSITY                      =      0.5600 VOL/VOL
FIELD CAPACITY                =      0.2660 VOL/VOL
WILTING POINT                 =      0.0900 VOL/VOL
INITIAL SOIL WATER CONTENT    =      0.2759 VOL/VOL
EFFECTIVE SAT. HYD. COND.     =  0.176000001000E-03 CM/SEC


                    LAYER  5
                    --------

          TYPE 1 - VERTICAL PERCOLATION LAYER
              MATERIAL TEXTURE NUMBER   0
THICKNESS                     =     50.40   INCHES
POROSITY                      =      0.5600 VOL/VOL
FIELD CAPACITY                =      0.2660 VOL/VOL
WILTING POINT                 =      0.0900 VOL/VOL
INITIAL SOIL WATER CONTENT    =      0.3099 VOL/VOL
```

```
EFFECTIVE SAT. HYD. COND.    =  0.176000001000E-03 CM/SEC


                        LAYER  6
                        --------

          TYPE 1 - VERTICAL PERCOLATION LAYER
              MATERIAL TEXTURE NUMBER   0
THICKNESS                    =     50.40   INCHES
POROSITY                     =      0.5600 VOL/VOL
FIELD CAPACITY               =      0.2660 VOL/VOL
WILTING POINT                =      0.0900 VOL/VOL
INITIAL SOIL WATER CONTENT   =      0.2819 VOL/VOL
EFFECTIVE SAT. HYD. COND.    =  0.176000001000E-03 CM/SEC


                        LAYER  7
                        --------

          TYPE 1 - VERTICAL PERCOLATION LAYER
              MATERIAL TEXTURE NUMBER   0
THICKNESS                    =     50.40   INCHES
POROSITY                     =      0.5600 VOL/VOL
FIELD CAPACITY               =      0.2660 VOL/VOL
WILTING POINT                =      0.0900 VOL/VOL
INITIAL SOIL WATER CONTENT   =      0.2660 VOL/VOL
EFFECTIVE SAT. HYD. COND.    =  0.176000001000E-03 CM/SEC


                        LAYER  8
                        --------

          TYPE 1 - VERTICAL PERCOLATION LAYER
              MATERIAL TEXTURE NUMBER   0
THICKNESS                    =     50.40   INCHES
POROSITY                     =      0.5600 VOL/VOL
FIELD CAPACITY               =      0.2660 VOL/VOL
WILTING POINT                =      0.0900 VOL/VOL
INITIAL SOIL WATER CONTENT   =      0.2660 VOL/VOL
EFFECTIVE SAT. HYD. COND.    =  0.176000001000E-03 CM/SEC


                        LAYER  9
                        --------

          TYPE 1 - VERTICAL PERCOLATION LAYER
              MATERIAL TEXTURE NUMBER   0
THICKNESS                    =     50.40   INCHES
POROSITY                     =      0.5600 VOL/VOL
```

```
FIELD CAPACITY                =      0.2660 VOL/VOL
WILTING POINT                 =      0.0900 VOL/VOL
INITIAL SOIL WATER CONTENT    =      0.2660 VOL/VOL
EFFECTIVE SAT. HYD. COND.     =  0.176000001000E-03 CM/SEC
```

LAYER 10
---

TYPE 1 - VERTICAL PERCOLATION LAYER
MATERIAL TEXTURE NUMBER 0

```
THICKNESS                     =     50.40   INCHES
POROSITY                      =      0.5600 VOL/VOL
FIELD CAPACITY                =      0.2660 VOL/VOL
WILTING POINT                 =      0.0900 VOL/VOL
INITIAL SOIL WATER CONTENT    =      0.2660 VOL/VOL
EFFECTIVE SAT. HYD. COND.     =  0.176000001000E-03 CM/SEC
```

LAYER 11
---

TYPE 1 - VERTICAL PERCOLATION LAYER
MATERIAL TEXTURE NUMBER 0

```
THICKNESS                     =     12.00   INCHES
POROSITY                      =      0.4900 VOL/VOL
FIELD CAPACITY                =      0.2720 VOL/VOL
WILTING POINT                 =      0.0900 VOL/VOL
INITIAL SOIL WATER CONTENT    =      0.2720 VOL/VOL
EFFECTIVE SAT. HYD. COND.     =  0.551999983000E-04 CM/SEC
```

LAYER 12
---

TYPE 2 - LATERAL DRAINAGE LAYER
MATERIAL TEXTURE NUMBER 20

```
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    =      0.0103 VOL/VOL
EFFECTIVE SAT. HYD. COND.     =   10.0000000000     CM/SEC
SLOPE                         =      1.50   PERCENT
DRAINAGE LENGTH               =    110.0    FEET
```

LAYER 13
---

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 | = | 2 - EXCELLENT | |

LAYER 14
---

TYPE 1 - VERTICAL PERCOLATION LAYER
MATERIAL TEXTURE NUMBER 0

| | | | |
|---|---|---|---|
| THICKNESS | = | 0.06 | INCHES |
| POROSITY | = | 0.7500 | VOL/VOL |
| FIELD CAPACITY | = | 0.7470 | VOL/VOL |
| WILTING POINT | = | 0.4000 | VOL/VOL |
| INITIAL SOIL WATER CONTENT | = | 0.6086 | VOL/VOL |
| EFFECTIVE SAT. HYD. COND. | = | 0.999999997000E-06 | CM/SEC |

LAYER 15
---

TYPE 1 - VERTICAL PERCOLATION LAYER
MATERIAL TEXTURE NUMBER 0

| | | | |
|---|---|---|---|
| THICKNESS | = | 48.00 | INCHES |
| POROSITY | = | 0.5300 | VOL/VOL |
| FIELD CAPACITY | = | 0.2210 | VOL/VOL |
| WILTING POINT | = | 0.1010 | VOL/VOL |
| INITIAL SOIL WATER CONTENT | = | 0.2210 | VOL/VOL |
| EFFECTIVE SAT. HYD. COND. | = | 0.372000010000E-03 | CM/SEC |

## GENERAL DESIGN AND EVAPORATIVE ZONE DATA

NOTE: SCS RUNOFF CURVE NUMBER WAS USER-SPECIFIED.

| | | | |
|---|---|---|---|
| SCS RUNOFF CURVE NUMBER | = | 82.00 | |
| FRACTION OF AREA ALLOWING RUNOFF | = | 100.0 | PERCENT |
| AREA PROJECTED ON HORIZONTAL PLANE | = | 17.100 | ACRES |
| EVAPORATIVE ZONE DEPTH | = | 10.0 | INCHES |
| INITIAL WATER IN EVAPORATIVE ZONE | = | 2.351 | INCHES |
| UPPER LIMIT OF EVAPORATIVE STORAGE | = | 5.600 | INCHES |
| LOWER LIMIT OF EVAPORATIVE STORAGE | = | 0.900 | INCHES |
| INITIAL SNOW WATER | = | 0.000 | INCHES |
| INITIAL WATER IN LAYER MATERIALS | = | 156.536 | INCHES |
| TOTAL INITIAL WATER | = | 156.536 | INCHES |
| TOTAL SUBSURFACE INFLOW | = | 0.00 | INCHES/YEAR |

## EVAPOTRANSPIRATION AND WEATHER DATA

NOTE: EVAPOTRANSPIRATION DATA WAS OBTAINED FROM
CHARLOTTE NORTH CAROLINA

| | | | |
|---|---|---|---|
| STATION LATITUDE | = | 35.60 | DEGREES |
| MAXIMUM LEAF AREA INDEX | = | 0.00 | |
| START OF GROWING SEASON (JULIAN DATE) | = | 83 | |
| END OF GROWING SEASON (JULIAN DATE) | = | 312 | |
| EVAPORATIVE ZONE DEPTH | = | 10.0 | INCHES |
| AVERAGE ANNUAL WIND SPEED | = | 7.50 | MPH |
| AVERAGE 1ST QUARTER RELATIVE HUMIDITY | = | 64.00 | % |
| AVERAGE 2ND QUARTER RELATIVE HUMIDITY | = | 67.00 | % |
| AVERAGE 3RD QUARTER RELATIVE HUMIDITY | = | 74.00 | % |
| AVERAGE 4TH QUARTER RELATIVE HUMIDITY | = | 70.00 | % |

NOTE: PRECIPITATION DATA WAS SYNTHETICALLY GENERATED USING
COEFFICIENTS FOR CHARLOTTE NORTH CAROLINA

NORMAL MEAN MONTHLY PRECIPITATION (INCHES)

| JAN/JUL | FEB/AUG | MAR/SEP | APR/OCT | MAY/NOV | JUN/DEC |
|---|---|---|---|---|---|
| 4.10 | 4.10 | 4.70 | 3.80 | 3.60 | 3.90 |
| 5.00 | 5.60 | 3.80 | 3.40 | 3.10 | 4.10 |

NOTE: TEMPERATURE DATA WAS SYNTHETICALLY GENERATED USING
COEFFICIENTS FOR CHARLOTTE NORTH CAROLINA

NORMAL MEAN MONTHLY TEMPERATURE (DEGREES FAHRENHEIT)

| JAN/JUL | FEB/AUG | MAR/SEP | APR/OCT | MAY/NOV | JUN/DEC |
|---|---|---|---|---|---|
| 41.00 | 42.00 | 49.00 | 59.00 | 67.00 | 75.00 |
| 77.00 | 76.00 | 70.50 | 60.50 | 49.00 | 41.50 |

NOTE: SOLAR RADIATION DATA WAS SYNTHETICALLY GENERATED USING
COEFFICIENTS FOR CHARLOTTE NORTH CAROLINA
AND STATION LATITUDE = 35.20 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 | 3.68 | 3.72 | 5.27 | 3.14 | 3.45 | 4.68 |
| | 4.56 | 5.51 | 3.73 | 3.69 | 2.78 | 4.00 |
| STD. DEVIATIONS | 1.86 | 1.79 | 1.54 | 2.32 | 2.04 | 1.24 |
| | 1.67 | 3.79 | 1.87 | 2.26 | 1.48 | 2.09 |
| **RUNOFF** | | | | | | |
| TOTALS | 0.180 | 0.047 | 0.179 | 0.034 | 0.109 | 0.091 |
| | 0.141 | 0.288 | 0.121 | 0.164 | 0.070 | 0.194 |
| STD. DEVIATIONS | 0.230 | 0.059 | 0.320 | 0.064 | 0.210 | 0.132 |
| | 0.329 | 0.425 | 0.131 | 0.306 | 0.128 | 0.344 |
| **EVAPOTRANSPIRATION** | | | | | | |
| TOTALS | 1.805 | 2.170 | 3.341 | 3.138 | 3.019 | 3.693 |
| | 4.103 | 3.579 | 2.987 | 2.323 | 1.946 | 1.358 |
| STD. DEVIATIONS | 0.244 | 0.171 | 0.238 | 1.367 | 1.817 | 1.425 |
| | 1.348 | 1.419 | 1.236 | 0.780 | 0.292 | 0.133 |
| **LATERAL DRAINAGE COLLECTED FROM LAYER 12** | | | | | | |
| TOTALS | 1.0049 | 0.9301 | 0.9788 | 1.0379 | 0.9348 | 0.9241 |

| | | | | | | |
|---|---|---|---|---|---|---|
| | 0.9147 | 0.8976 | 1.0495 | 1.0786 | 1.1527 | 1.1104 |
| STD. DEVIATIONS | 0.5619 | 0.4914 | 0.5201 | 0.5133 | 0.4657 | 0.4704 |
| | 0.4917 | 0.4424 | 0.4296 | 0.6349 | 0.5995 | 0.6240 |

PERCOLATION/LEAKAGE THROUGH LAYER 13

| | | | | | | |
|---|---|---|---|---|---|---|
| TOTALS | 0.0000 | 0.0000 | 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 |

PERCOLATION/LEAKAGE THROUGH LAYER 15

| | | | | | | |
|---|---|---|---|---|---|---|
| TOTALS | 0.0000 | 0.0000 | 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 |

---

AVERAGES OF MONTHLY AVERAGED DAILY HEADS (INCHES)

---

DAILY AVERAGE HEAD ON TOP OF LAYER 13

| | | | | | | |
|---|---|---|---|---|---|---|
| AVERAGES | 0.0042 | 0.0043 | 0.0041 | 0.0045 | 0.0039 | 0.0040 |
| | 0.0038 | 0.0037 | 0.0045 | 0.0045 | 0.0050 | 0.0046 |
| STD. DEVIATIONS | 0.0023 | 0.0022 | 0.0022 | 0.0022 | 0.0019 | 0.0020 |
| | 0.0021 | 0.0018 | 0.0019 | 0.0026 | 0.0026 | 0.0026 |

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AVERAGE ANNUAL TOTALS & (STD. DEVIATIONS) FOR YEARS 1 THROUGH 10

| | INCHES | | CU. FEET | PERCENT |
|---|---|---|---|---|
| PRECIPITATION | 48.22 | ( 6.597) | 2993470.2 | 100.00 |
| RUNOFF | 1.621 | ( 1.1149) | 100620.08 | 3.361 |
| EVAPOTRANSPIRATION | 33.461 | ( 2.0580) | 2077004.25 | 69.384 |
| LATERAL DRAINAGE COLLECTED FROM LAYER 12 | 12.01412 | ( 4.83103) | 745752.562 | 24.91264 |
| PERCOLATION/LEAKAGE THROUGH LAYER 13 | 0.00001 | ( 0.00000) | 0.554 | 0.00002 |
| AVERAGE HEAD ON TOP OF LAYER 13 | 0.004 | ( 0.002) | | |
| PERCOLATION/LEAKAGE THROUGH LAYER 15 | 0.00000 | ( 0.00000) | 0.000 | 0.00000 |
| CHANGE IN WATER STORAGE | 1.129 | ( 6.1632) | 70093.73 | 2.342 |

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PEAK DAILY VALUES FOR YEARS 1 THROUGH 10

| | (INCHES) | (CU. FT.) |
|---|---|---|
| PRECIPITATION | 3.08 | 191184.844 |
| RUNOFF | 1.083 | 67240.0859 |
| DRAINAGE COLLECTED FROM LAYER 12 | 0.08648 | 5367.86865 |
| PERCOLATION/LEAKAGE THROUGH LAYER 13 | 0.000001 | 0.05892 |
| AVERAGE HEAD ON TOP OF LAYER 13 | 0.011 | |
| MAXIMUM HEAD ON TOP OF LAYER 13 | 0.022 | |
| LOCATION OF MAXIMUM HEAD IN LAYER 12 (DISTANCE FROM DRAIN) | 0.6 FEET | |
| PERCOLATION/LEAKAGE THROUGH LAYER 15 | 0.000000 | 0.00000 |
| SNOW WATER | 2.86 | 177548.3440 |

| | |
|---|---|
| MAXIMUM VEG. SOIL WATER (VOL/VOL) | 0.4432 |
| 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 10

| LAYER | (INCHES) | (VOL/VOL) |
|---|---|---|
| 1 | 15.0261 | 0.2981 |
| 2 | 14.6485 | 0.2906 |
| 3 | 13.5157 | 0.2682 |
| 4 | 14.6485 | 0.2906 |
| 5 | 15.6188 | 0.3099 |
| 6 | 15.5574 | 0.3087 |
| 7 | 15.4511 | 0.3066 |
| 8 | 16.1088 | 0.3196 |
| 9 | 16.2832 | 0.3231 |
| 10 | 16.4696 | 0.3268 |
| 11 | 3.8495 | 0.3208 |
| 12 | 0.0063 | 0.0368 |
| 13 | 0.0000 | 0.0000 |
| 14 | 0.0365 | 0.6086 |
| 15 | 10.6078 | 0.2210 |
| 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\mssfgd\MSSRAIN.D4
TEMPERATURE DATA FILE:        C:\WMM\HELP3-7\mssfgd\MSSTEMP.D7
SOLAR RADIATION DATA FILE:    C:\wmm\help3-7\mssfgd\MSSRAD.D13
EVAPOTRANSPIRATION DATA:      C:\wmm\help3-7\mssfgd\MSSOPEN.D11
SOIL AND DESIGN DATA FILE:    C:\wmm\help3-7\mssfgd\C2-84.D10
OUTPUT DATA FILE:             C:\WMM\HELP3-7\mssfgd\c2-84.OUT


TIME:   9: 1     DATE:  12/16/2005


******************************************************************************

     TITLE:  MSSFGD LF - Cell 2 - 84 feet of Waste - 60 mil HDPE + 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   0
          THICKNESS                    =    100.80   INCHES
          POROSITY                     =      0.5600 VOL/VOL
          FIELD CAPACITY               =      0.2660 VOL/VOL
          WILTING POINT                =      0.0900 VOL/VOL
          INITIAL SOIL WATER CONTENT   =      0.3040 VOL/VOL
          EFFECTIVE SAT. HYD. COND.    =  0.176000001000E-03 CM/SEC
```

LAYER 2
---

TYPE 1 - VERTICAL PERCOLATION LAYER
MATERIAL TEXTURE NUMBER 0

| | | | |
|---|---|---|---|
| THICKNESS | = | 100.80 | INCHES |
| POROSITY | = | 0.5600 | VOL/VOL |
| FIELD CAPACITY | = | 0.2660 | VOL/VOL |
| WILTING POINT | = | 0.0900 | VOL/VOL |
| INITIAL SOIL WATER CONTENT | = | 0.2846 | VOL/VOL |
| EFFECTIVE SAT. HYD. COND. | = | 0.176000001000E-03 | CM/SEC |

LAYER 3
---

TYPE 1 - VERTICAL PERCOLATION LAYER
MATERIAL TEXTURE NUMBER 0

| | | | |
|---|---|---|---|
| THICKNESS | = | 100.80 | INCHES |
| POROSITY | = | 0.5600 | VOL/VOL |
| FIELD CAPACITY | = | 0.2660 | VOL/VOL |
| WILTING POINT | = | 0.0900 | VOL/VOL |
| INITIAL SOIL WATER CONTENT | = | 0.2899 | VOL/VOL |
| EFFECTIVE SAT. HYD. COND. | = | 0.176000001000E-03 | CM/SEC |

LAYER 4
---

TYPE 1 - VERTICAL PERCOLATION LAYER
MATERIAL TEXTURE NUMBER 0

| | | | |
|---|---|---|---|
| THICKNESS | = | 100.80 | INCHES |
| POROSITY | = | 0.5600 | VOL/VOL |
| FIELD CAPACITY | = | 0.2660 | VOL/VOL |
| WILTING POINT | = | 0.0900 | VOL/VOL |
| INITIAL SOIL WATER CONTENT | = | 0.2693 | VOL/VOL |
| EFFECTIVE SAT. HYD. COND. | = | 0.176000001000E-03 | CM/SEC |

LAYER 5
---

TYPE 1 - VERTICAL PERCOLATION LAYER
MATERIAL TEXTURE NUMBER 0

| | | | |
|---|---|---|---|
| THICKNESS | = | 100.80 | INCHES |
| POROSITY | = | 0.5600 | VOL/VOL |
| FIELD CAPACITY | = | 0.2660 | VOL/VOL |
| WILTING POINT | = | 0.0900 | VOL/VOL |
| INITIAL SOIL WATER CONTENT | = | 0.2660 | VOL/VOL |

```
EFFECTIVE SAT. HYD. COND.    =  0.176000001000E-03 CM/SEC


                        LAYER  6
                        --------

          TYPE 1 - VERTICAL PERCOLATION LAYER
              MATERIAL TEXTURE NUMBER   0
THICKNESS                    =    100.80   INCHES
POROSITY                     =      0.5600 VOL/VOL
FIELD CAPACITY               =      0.2660 VOL/VOL
WILTING POINT                =      0.0900 VOL/VOL
INITIAL SOIL WATER CONTENT   =      0.2660 VOL/VOL
EFFECTIVE SAT. HYD. COND.    =  0.176000001000E-03 CM/SEC


                        LAYER  7
                        --------

          TYPE 1 - VERTICAL PERCOLATION LAYER
              MATERIAL TEXTURE NUMBER   0
THICKNESS                    =    100.80   INCHES
POROSITY                     =      0.5600 VOL/VOL
FIELD CAPACITY               =      0.2660 VOL/VOL
WILTING POINT                =      0.0900 VOL/VOL
INITIAL SOIL WATER CONTENT   =      0.2660 VOL/VOL
EFFECTIVE SAT. HYD. COND.    =  0.176000001000E-03 CM/SEC


                        LAYER  8
                        --------

          TYPE 1 - VERTICAL PERCOLATION LAYER
              MATERIAL TEXTURE NUMBER   0
THICKNESS                    =    100.80   INCHES
POROSITY                     =      0.5600 VOL/VOL
FIELD CAPACITY               =      0.2660 VOL/VOL
WILTING POINT                =      0.0900 VOL/VOL
INITIAL SOIL WATER CONTENT   =      0.2660 VOL/VOL
EFFECTIVE SAT. HYD. COND.    =  0.176000001000E-03 CM/SEC


                        LAYER  9
                        --------

          TYPE 1 - VERTICAL PERCOLATION LAYER
              MATERIAL TEXTURE NUMBER   0
THICKNESS                    =    100.80   INCHES
POROSITY                     =      0.5600 VOL/VOL
```

```
FIELD CAPACITY                =      0.2660 VOL/VOL
WILTING POINT                 =      0.0900 VOL/VOL
INITIAL SOIL WATER CONTENT    =      0.2660 VOL/VOL
EFFECTIVE SAT. HYD. COND.     =  0.176000001000E-03 CM/SEC
```

LAYER 10
---

```
          TYPE 1 - VERTICAL PERCOLATION LAYER
              MATERIAL TEXTURE NUMBER   0
THICKNESS                     =    100.80   INCHES
POROSITY                      =      0.5600 VOL/VOL
FIELD CAPACITY                =      0.2660 VOL/VOL
WILTING POINT                 =      0.0900 VOL/VOL
INITIAL SOIL WATER CONTENT    =      0.2660 VOL/VOL
EFFECTIVE SAT. HYD. COND.     =  0.176000001000E-03 CM/SEC
```

LAYER 11
---

```
          TYPE 1 - VERTICAL PERCOLATION LAYER
              MATERIAL TEXTURE NUMBER   0
THICKNESS                     =     12.00   INCHES
POROSITY                      =      0.5000 VOL/VOL
FIELD CAPACITY                =      0.2720 VOL/VOL
WILTING POINT                 =      0.0900 VOL/VOL
INITIAL SOIL WATER CONTENT    =      0.2720 VOL/VOL
EFFECTIVE SAT. HYD. COND.     =  0.551999983000E-04 CM/SEC
```

LAYER 12
---

```
            TYPE 2 - LATERAL DRAINAGE LAYER
              MATERIAL TEXTURE NUMBER  20
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.     =   10.0000000000     CM/SEC
SLOPE                         =      1.50   PERCENT
DRAINAGE LENGTH               =    110.0    FEET
```

LAYER 13

---

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 | = | 2 - EXCELLENT | |

LAYER 14
---

TYPE 1 - VERTICAL PERCOLATION LAYER
MATERIAL TEXTURE NUMBER 0

| | | | |
|---|---|---|---|
| THICKNESS | = | 0.06 | INCHES |
| POROSITY | = | 0.7500 | VOL/VOL |
| FIELD CAPACITY | = | 0.7470 | VOL/VOL |
| WILTING POINT | = | 0.4000 | VOL/VOL |
| INITIAL SOIL WATER CONTENT | = | 0.6086 | VOL/VOL |
| EFFECTIVE SAT. HYD. COND. | = | 0.999999997000E-06 | CM/SEC |

LAYER 15
---

TYPE 1 - VERTICAL PERCOLATION LAYER
MATERIAL TEXTURE NUMBER 0

| | | | |
|---|---|---|---|
| THICKNESS | = | 48.00 | INCHES |
| POROSITY | = | 0.5300 | VOL/VOL |
| FIELD CAPACITY | = | 0.2210 | VOL/VOL |
| WILTING POINT | = | 0.1010 | VOL/VOL |
| INITIAL SOIL WATER CONTENT | = | 0.2210 | VOL/VOL |
| EFFECTIVE SAT. HYD. COND. | = | 0.372000010000E-03 | CM/SEC |

GENERAL DESIGN AND EVAPORATIVE ZONE DATA
---

NOTE: SCS RUNOFF CURVE NUMBER WAS USER-SPECIFIED.

| | | | |
|---|---|---|---|
| SCS RUNOFF CURVE NUMBER | = | 82.00 | |
| FRACTION OF AREA ALLOWING RUNOFF | = | 100.0 | PERCENT |
| AREA PROJECTED ON HORIZONTAL PLANE | = | 17.100 | ACRES |
| EVAPORATIVE ZONE DEPTH | = | 10.0 | INCHES |

| | | | |
|---|---|---|---|
| INITIAL WATER IN EVAPORATIVE ZONE | = | 2.351 | INCHES |
| UPPER LIMIT OF EVAPORATIVE STORAGE | = | 5.600 | INCHES |
| LOWER LIMIT OF EVAPORATIVE STORAGE | = | 0.900 | INCHES |
| INITIAL SNOW WATER | = | 0.000 | INCHES |
| INITIAL WATER IN LAYER MATERIALS | = | 290.478 | INCHES |
| TOTAL INITIAL WATER | = | 290.478 | INCHES |
| TOTAL SUBSURFACE INFLOW | = | 0.00 | INCHES/YEAR |

## EVAPOTRANSPIRATION AND WEATHER DATA

NOTE: EVAPOTRANSPIRATION DATA WAS OBTAINED FROM
CHARLOTTE NORTH CAROLINA

| | | | |
|---|---|---|---|
| STATION LATITUDE | = | 35.60 | DEGREES |
| MAXIMUM LEAF AREA INDEX | = | 0.00 | |
| START OF GROWING SEASON (JULIAN DATE) | = | 83 | |
| END OF GROWING SEASON (JULIAN DATE) | = | 312 | |
| EVAPORATIVE ZONE DEPTH | = | 10.0 | INCHES |
| AVERAGE ANNUAL WIND SPEED | = | 7.50 | MPH |
| AVERAGE 1ST QUARTER RELATIVE HUMIDITY | = | 64.00 | % |
| AVERAGE 2ND QUARTER RELATIVE HUMIDITY | = | 67.00 | % |
| AVERAGE 3RD QUARTER RELATIVE HUMIDITY | = | 74.00 | % |
| AVERAGE 4TH QUARTER RELATIVE HUMIDITY | = | 70.00 | % |

NOTE: PRECIPITATION DATA WAS SYNTHETICALLY GENERATED USING
COEFFICIENTS FOR CHARLOTTE NORTH CAROLINA

NORMAL MEAN MONTHLY PRECIPITATION (INCHES)

| JAN/JUL | FEB/AUG | MAR/SEP | APR/OCT | MAY/NOV | JUN/DEC |
|---|---|---|---|---|---|
| 4.10 | 4.10 | 4.70 | 3.80 | 3.60 | 3.90 |
| 5.00 | 5.60 | 3.80 | 3.40 | 3.10 | 4.10 |

NOTE: TEMPERATURE DATA WAS SYNTHETICALLY GENERATED USING
COEFFICIENTS FOR CHARLOTTE NORTH CAROLINA

NORMAL MEAN MONTHLY TEMPERATURE (DEGREES FAHRENHEIT)

| JAN/JUL | FEB/AUG | MAR/SEP | APR/OCT | MAY/NOV | JUN/DEC |
|---|---|---|---|---|---|
| 41.00 | 42.00 | 49.00 | 59.00 | 67.00 | 75.00 |
| 77.00 | 76.00 | 70.50 | 60.50 | 49.00 | 41.50 |

NOTE: SOLAR RADIATION DATA WAS SYNTHETICALLY GENERATED USING
COEFFICIENTS FOR CHARLOTTE NORTH CAROLINA
AND STATION LATITUDE = 35.20 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 | 3.68 | 3.72 | 5.27 | 3.14 | 3.45 | 4.68 |
| | 4.56 | 5.51 | 3.73 | 3.69 | 2.78 | 4.00 |
| STD. DEVIATIONS | 1.86 | 1.79 | 1.54 | 2.32 | 2.04 | 1.24 |
| | 1.67 | 3.79 | 1.87 | 2.26 | 1.48 | 2.09 |
| **RUNOFF** | | | | | | |
| TOTALS | 0.180 | 0.047 | 0.179 | 0.034 | 0.109 | 0.091 |
| | 0.141 | 0.288 | 0.121 | 0.164 | 0.070 | 0.194 |
| STD. DEVIATIONS | 0.230 | 0.059 | 0.320 | 0.064 | 0.210 | 0.132 |
| | 0.329 | 0.425 | 0.131 | 0.306 | 0.128 | 0.344 |
| **EVAPOTRANSPIRATION** | | | | | | |
| TOTALS | 1.805 | 2.170 | 3.341 | 3.138 | 3.019 | 3.693 |
| | 4.103 | 3.579 | 2.987 | 2.323 | 1.946 | 1.358 |
| STD. DEVIATIONS | 0.244 | 0.171 | 0.238 | 1.367 | 1.817 | 1.425 |
| | 1.348 | 1.419 | 1.236 | 0.780 | 0.292 | 0.133 |
| **LATERAL DRAINAGE COLLECTED FROM LAYER 12** | | | | | | |
| TOTALS | 0.8085 | 0.7901 | 0.8841 | 0.7207 | 0.7861 | 0.6918 |
| | 0.7502 | 0.7691 | 0.8421 | 0.8415 | 0.9812 | 0.9641 |
| STD. DEVIATIONS | 0.7090 | 0.7255 | 0.6880 | 0.6231 | 0.4907 | 0.4640 |
| | 0.5522 | 0.6271 | 0.6382 | 0.5601 | 0.6204 | 0.6502 |
| **PERCOLATION/LEAKAGE THROUGH LAYER 13** | | | | | | |
| TOTALS | 0.0000 | 0.0000 | 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 |

PERCOLATION/LEAKAGE THROUGH LAYER 15

| | | | | | | |
|---|---|---|---|---|---|---|
| TOTALS | 0.0000 | 0.0000 | 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 |

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AVERAGES OF MONTHLY AVERAGED DAILY HEADS (INCHES)

---

DAILY AVERAGE HEAD ON TOP OF LAYER 13

| | | | | | | |
|---|---|---|---|---|---|---|
| AVERAGES | 0.0034 | 0.0036 | 0.0037 | 0.0031 | 0.0033 | 0.0030 |
| | 0.0031 | 0.0032 | 0.0036 | 0.0035 | 0.0042 | 0.0040 |
| STD. DEVIATIONS | 0.0030 | 0.0033 | 0.0029 | 0.0027 | 0.0020 | 0.0020 |
| | 0.0023 | 0.0026 | 0.0028 | 0.0023 | 0.0027 | 0.0027 |

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AVERAGE ANNUAL TOTALS & (STD. DEVIATIONS) FOR YEARS 1 THROUGH 10

| | INCHES | | CU. FEET | PERCENT |
|---|---|---|---|---|
| PRECIPITATION | 48.22 | ( 6.597) | 2993470.2 | 100.00 |
| RUNOFF | 1.621 | ( 1.1149) | 100620.08 | 3.361 |
| EVAPOTRANSPIRATION | 33.461 | ( 2.0580) | 2077004.25 | 69.384 |
| LATERAL DRAINAGE COLLECTED FROM LAYER 12 | 9.82947 | ( 6.25876) | 610144.687 | 20.38252 |
| PERCOLATION/LEAKAGE THROUGH LAYER 13 | 0.00001 | ( 0.00000) | 0.475 | 0.00002 |
| AVERAGE HEAD ON TOP OF LAYER 13 | 0.003 | ( 0.002) | | |
| PERCOLATION/LEAKAGE THROUGH LAYER 15 | 0.00000 | ( 0.00000) | 0.000 | 0.00000 |
| CHANGE IN WATER STORAGE | 3.314 | ( 7.7371) | 205701.28 | 6.872 |

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PEAK DAILY VALUES FOR YEARS 1 THROUGH 10

| | (INCHES) | (CU. FT.) |
|---|---|---|
| PRECIPITATION | 3.08 | 191184.844 |
| RUNOFF | 1.083 | 67240.0859 |
| DRAINAGE COLLECTED FROM LAYER 12 | 0.08697 | 5398.40527 |
| PERCOLATION/LEAKAGE THROUGH LAYER 13 | 0.000001 | 0.06239 |
| AVERAGE HEAD ON TOP OF LAYER 13 | 0.011 | |
| MAXIMUM HEAD ON TOP OF LAYER 13 | 0.022 | |
| LOCATION OF MAXIMUM HEAD IN LAYER 12 (DISTANCE FROM DRAIN) | 0.9 FEET | |
| PERCOLATION/LEAKAGE THROUGH LAYER 15 | 0.000000 | 0.00000 |
| SNOW WATER | 2.86 | 177548.3440 |

| | |
|---|---|
| MAXIMUM VEG. SOIL WATER (VOL/VOL) | 0.4432 |
| 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 10

| LAYER | (INCHES) | (VOL/VOL) |
|---|---|---|
| 1 | 29.2064 | 0.2897 |
| 2 | 28.9695 | 0.2874 |
| 3 | 30.9182 | 0.3067 |
| 4 | 31.6746 | 0.3142 |
| 5 | 32.7744 | 0.3251 |
| 6 | 31.4265 | 0.3118 |
| 7 | 30.4656 | 0.3022 |
| 8 | 30.0702 | 0.2983 |
| 9 | 31.8321 | 0.3158 |
| 10 | 31.8231 | 0.3157 |
| 11 | 3.8052 | 0.3171 |
| 12 | 0.0061 | 0.0358 |
| 13 | 0.0000 | 0.0000 |
| 14 | 0.0365 | 0.6086 |
| 15 | 10.6078 | 0.2210 |
| 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\mssfgd\MSSRAIN.D4
TEMPERATURE DATA FILE:        C:\WMM\HELP3-7\mssfgd\MSSTEMP.D7
SOLAR RADIATION DATA FILE:    C:\wmm\help3-7\mssfgd\MSSRAD.D13
EVAPOTRANSPIRATION DATA:      C:\wmm\help3-7\mssfgd\MSSCLOSE.D11
SOIL AND DESIGN DATA FILE:    C:\wmm\help3-7\mssfgd\MSSCLOSE.D10
OUTPUT DATA FILE:             C:\WMM\HELP3-7\mssfgd\MSSCLOSE.OUT


TIME:  16:22     DATE:  12/15/2005


******************************************************************************

     TITLE:  MSSFGD LF - Closed W/GCL - Cell 1 and Cell 2

******************************************************************************


     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   0
          THICKNESS                    =      9.00   INCHES
          POROSITY                     =      0.4900 VOL/VOL
          FIELD CAPACITY               =      0.2890 VOL/VOL
          WILTING POINT                =      0.1260 VOL/VOL
          INITIAL SOIL WATER CONTENT   =      0.2900 VOL/VOL
          EFFECTIVE SAT. HYD. COND.    =  0.249000004000E-03 CM/SEC
```

LAYER 2
---

TYPE 1 - VERTICAL PERCOLATION LAYER
MATERIAL TEXTURE NUMBER 0

| | | | |
|---|---|---|---|
| THICKNESS | = | 15.00 | INCHES |
| POROSITY | = | 0.4900 | VOL/VOL |
| FIELD CAPACITY | = | 0.2890 | VOL/VOL |
| WILTING POINT | = | 0.1260 | VOL/VOL |
| INITIAL SOIL WATER CONTENT | = | 0.2900 | VOL/VOL |
| EFFECTIVE SAT. HYD. COND. | = | 0.829999990000E-04 | CM/SEC |

LAYER 3
---

TYPE 2 - LATERAL DRAINAGE LAYER
MATERIAL TEXTURE NUMBER 20

| | | | |
|---|---|---|---|
| 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. | = | 10.0000000000 | CM/SEC |
| SLOPE | = | 12.50 | PERCENT |
| DRAINAGE LENGTH | = | 300.0 | 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   0
THICKNESS                    =     18.00   INCHES
POROSITY                     =      0.4900 VOL/VOL
FIELD CAPACITY               =      0.2890 VOL/VOL
WILTING POINT                =      0.1260 VOL/VOL
INITIAL SOIL WATER CONTENT   =      0.2750 VOL/VOL
EFFECTIVE SAT. HYD. COND.    =  0.829999990000E-04 CM/SEC


                    LAYER  6
                    --------

          TYPE 1 - VERTICAL PERCOLATION LAYER
              MATERIAL TEXTURE NUMBER   0
THICKNESS                    =    126.75   INCHES
POROSITY                     =      0.5600 VOL/VOL
FIELD CAPACITY               =      0.2660 VOL/VOL
WILTING POINT                =      0.0900 VOL/VOL
INITIAL SOIL WATER CONTENT   =      0.2970 VOL/VOL
EFFECTIVE SAT. HYD. COND.    =  0.176000001000E-03 CM/SEC


                    LAYER  7
                    --------

          TYPE 1 - VERTICAL PERCOLATION LAYER
              MATERIAL TEXTURE NUMBER   0
THICKNESS                    =    126.75   INCHES
POROSITY                     =      0.5600 VOL/VOL
FIELD CAPACITY               =      0.2660 VOL/VOL
WILTING POINT                =      0.0900 VOL/VOL
INITIAL SOIL WATER CONTENT   =      0.2970 VOL/VOL
EFFECTIVE SAT. HYD. COND.    =  0.176000001000E-03 CM/SEC


                    LAYER  8
                    --------

          TYPE 1 - VERTICAL PERCOLATION LAYER
              MATERIAL TEXTURE NUMBER   0
THICKNESS                    =    126.75   INCHES
POROSITY                     =      0.5600 VOL/VOL
FIELD CAPACITY               =      0.2660 VOL/VOL
WILTING POINT                =      0.0900 VOL/VOL
INITIAL SOIL WATER CONTENT   =      0.3150 VOL/VOL
EFFECTIVE SAT. HYD. COND.    =  0.176000001000E-03 CM/SEC
```

LAYER 9
---

TYPE 1 - VERTICAL PERCOLATION LAYER
MATERIAL TEXTURE NUMBER 0

| | | | |
|---|---|---|---|
| THICKNESS | = | 126.75 | INCHES |
| POROSITY | = | 0.5600 | VOL/VOL |
| FIELD CAPACITY | = | 0.2660 | VOL/VOL |
| WILTING POINT | = | 0.0900 | VOL/VOL |
| INITIAL SOIL WATER CONTENT | = | 0.3210 | VOL/VOL |
| EFFECTIVE SAT. HYD. COND. | = | 0.176000001000E-03 | CM/SEC |

LAYER 10
---

TYPE 1 - VERTICAL PERCOLATION LAYER
MATERIAL TEXTURE NUMBER 0

| | | | |
|---|---|---|---|
| THICKNESS | = | 126.75 | INCHES |
| POROSITY | = | 0.5600 | VOL/VOL |
| FIELD CAPACITY | = | 0.2660 | VOL/VOL |
| WILTING POINT | = | 0.0900 | VOL/VOL |
| INITIAL SOIL WATER CONTENT | = | 0.3250 | VOL/VOL |
| EFFECTIVE SAT. HYD. COND. | = | 0.176000001000E-03 | CM/SEC |

LAYER 11
---

TYPE 1 - VERTICAL PERCOLATION LAYER
MATERIAL TEXTURE NUMBER 0

| | | | |
|---|---|---|---|
| THICKNESS | = | 126.75 | INCHES |
| POROSITY | = | 0.5600 | VOL/VOL |
| FIELD CAPACITY | = | 0.2660 | VOL/VOL |
| WILTING POINT | = | 0.0900 | VOL/VOL |
| INITIAL SOIL WATER CONTENT | = | 0.3040 | VOL/VOL |
| EFFECTIVE SAT. HYD. COND. | = | 0.176000001000E-03 | CM/SEC |

LAYER 12
---

TYPE 1 - VERTICAL PERCOLATION LAYER
MATERIAL TEXTURE NUMBER 0

| | | | |
|---|---|---|---|
| THICKNESS | = | 126.75 | INCHES |
| POROSITY | = | 0.5600 | VOL/VOL |
| FIELD CAPACITY | = | 0.2660 | VOL/VOL |
| WILTING POINT | = | 0.0900 | VOL/VOL |
| INITIAL SOIL WATER CONTENT | = | 0.3090 | VOL/VOL |
| EFFECTIVE SAT. HYD. COND. | = | 0.176000001000E-03 | CM/SEC |

LAYER 13
---

TYPE 1 - VERTICAL PERCOLATION LAYER
MATERIAL TEXTURE NUMBER 0

| | | | |
|---|---|---|---|
| THICKNESS | = | 126.75 | INCHES |
| POROSITY | = | 0.5600 | VOL/VOL |
| FIELD CAPACITY | = | 0.2660 | VOL/VOL |
| WILTING POINT | = | 0.0900 | VOL/VOL |
| INITIAL SOIL WATER CONTENT | = | 0.3070 | VOL/VOL |
| EFFECTIVE SAT. HYD. COND. | = | 0.176000001000E-03 | CM/SEC |

LAYER 14
---

TYPE 1 - VERTICAL PERCOLATION LAYER
MATERIAL TEXTURE NUMBER 0

| | | | |
|---|---|---|---|
| THICKNESS | = | 12.00 | INCHES |
| POROSITY | = | 0.5000 | VOL/VOL |
| FIELD CAPACITY | = | 0.2720 | VOL/VOL |
| WILTING POINT | = | 0.0900 | VOL/VOL |
| INITIAL SOIL WATER CONTENT | = | 0.3230 | VOL/VOL |
| EFFECTIVE SAT. HYD. COND. | = | 0.551999983000E-04 | CM/SEC |

LAYER 15
---

TYPE 2 - LATERAL DRAINAGE LAYER
MATERIAL TEXTURE NUMBER 20

| | | | |
|---|---|---|---|
| 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 | = | 0.0320 | VOL/VOL |
| EFFECTIVE SAT. HYD. COND. | = | 10.0000000000 | CM/SEC |
| SLOPE | = | 1.00 | PERCENT |
| DRAINAGE LENGTH | = | 65.0 | FEET |

LAYER 16
---

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 | = | 1 - PERFECT | |

LAYER 17
---

TYPE 1 - VERTICAL PERCOLATION LAYER
MATERIAL TEXTURE NUMBER 0

| | | | |
|---|---|---|---|
| THICKNESS | = | 0.06 | INCHES |
| POROSITY | = | 0.7500 | VOL/VOL |
| FIELD CAPACITY | = | 0.7400 | VOL/VOL |
| WILTING POINT | = | 0.4000 | VOL/VOL |
| INITIAL SOIL WATER CONTENT | = | 0.6100 | VOL/VOL |
| EFFECTIVE SAT. HYD. COND. | = | 0.999999997000E-06 | CM/SEC |

## LAYER 18

TYPE 1 - VERTICAL PERCOLATION LAYER
MATERIAL TEXTURE NUMBER 0

| | | | |
|---|---|---|---|
| THICKNESS | = | 48.00 | INCHES |
| POROSITY | = | 0.5300 | VOL/VOL |
| FIELD CAPACITY | = | 0.2210 | VOL/VOL |
| WILTING POINT | = | 0.1010 | VOL/VOL |
| INITIAL SOIL WATER CONTENT | = | 0.2210 | VOL/VOL |
| EFFECTIVE SAT. HYD. COND. | = | 0.372000010000E-03 | CM/SEC |

### GENERAL DESIGN AND EVAPORATIVE ZONE DATA

NOTE: SCS RUNOFF CURVE NUMBER WAS COMPUTED FROM A USER-SPECIFIED CURVE NUMBER OF 61.0, A SURFACE SLOPE OF 12.% AND A SLOPE LENGTH OF 300. FEET.

| | | | |
|---|---|---|---|
| SCS RUNOFF CURVE NUMBER | = | 63.70 | |
| FRACTION OF AREA ALLOWING RUNOFF | = | 100.0 | PERCENT |
| AREA PROJECTED ON HORIZONTAL PLANE | = | 31.900 | ACRES |
| EVAPORATIVE ZONE DEPTH | = | 13.0 | INCHES |
| INITIAL WATER IN EVAPORATIVE ZONE | = | 3.770 | INCHES |
| UPPER LIMIT OF EVAPORATIVE STORAGE | = | 6.370 | INCHES |
| LOWER LIMIT OF EVAPORATIVE STORAGE | = | 1.638 | INCHES |
| INITIAL SNOW WATER | = | 0.000 | INCHES |
| INITIAL WATER IN LAYER MATERIALS | = | 340.144 | INCHES |
| TOTAL INITIAL WATER | = | 340.144 | INCHES |
| TOTAL SUBSURFACE INFLOW | = | 0.00 | INCHES/YEAR |

### EVAPOTRANSPIRATION AND WEATHER DATA

NOTE: EVAPOTRANSPIRATION DATA WAS OBTAINED FROM
CHARLOTTE NORTH CAROLINA

| | | | |
|---|---|---|---|
| STATION LATITUDE | = | 35.60 | DEGREES |
| MAXIMUM LEAF AREA INDEX | = | 0.00 | |
| START OF GROWING SEASON (JULIAN DATE) | = | 83 | |
| END OF GROWING SEASON (JULIAN DATE) | = | 312 | |
| EVAPORATIVE ZONE DEPTH | = | 13.0 | INCHES |
| AVERAGE ANNUAL WIND SPEED | = | 7.50 | MPH |
| AVERAGE 1ST QUARTER RELATIVE HUMIDITY | = | 64.00 | % |
| AVERAGE 2ND QUARTER RELATIVE HUMIDITY | = | 67.00 | % |
| AVERAGE 3RD QUARTER RELATIVE HUMIDITY | = | 74.00 | % |
| AVERAGE 4TH QUARTER RELATIVE HUMIDITY | = | 70.00 | % |

NOTE: PRECIPITATION DATA WAS SYNTHETICALLY GENERATED USING
COEFFICIENTS FOR CHARLOTTE NORTH CAROLINA

NORMAL MEAN MONTHLY PRECIPITATION (INCHES)

| JAN/JUL | FEB/AUG | MAR/SEP | APR/OCT | MAY/NOV | JUN/DEC |
|---|---|---|---|---|---|
| 4.10 | 4.10 | 4.70 | 3.80 | 3.60 | 3.90 |
| 5.00 | 5.60 | 3.80 | 3.40 | 3.10 | 4.10 |

NOTE: TEMPERATURE DATA WAS SYNTHETICALLY GENERATED USING
COEFFICIENTS FOR CHARLOTTE NORTH CAROLINA

NORMAL MEAN MONTHLY TEMPERATURE (DEGREES FAHRENHEIT)

| JAN/JUL | FEB/AUG | MAR/SEP | APR/OCT | MAY/NOV | JUN/DEC |
|---|---|---|---|---|---|
| 41.00 | 42.00 | 49.00 | 59.00 | 67.00 | 75.00 |
| 77.00 | 76.00 | 70.50 | 60.50 | 49.00 | 41.50 |

NOTE: SOLAR RADIATION DATA WAS SYNTHETICALLY GENERATED USING
COEFFICIENTS FOR CHARLOTTE NORTH CAROLINA
AND STATION LATITUDE = 35.20 DEGREES

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ANNUAL TOTALS FOR YEAR 1

| | INCHES | CU. FEET | PERCENT |
|---|---|---|---|
| PRECIPITATION | 45.22 | 5236340.500 | 100.00 |
| RUNOFF | 0.010 | 1168.259 | 0.02 |
| EVAPOTRANSPIRATION | 35.134 | 4068429.250 | 77.70 |
| DRAINAGE COLLECTED FROM LAYER 3 | 10.0289 | 1161321.120 | 22.18 |
| PERC./LEAKAGE THROUGH LAYER 4 | 0.000394 | 45.642 | 0.00 |
| AVG. HEAD ON TOP OF LAYER 4 | 0.0012 | | |
| DRAINAGE COLLECTED FROM LAYER 15 | 9.8791 | 1143972.750 | 21.85 |
| PERC./LEAKAGE THROUGH LAYER 16 | 0.000001 | 0.115 | 0.00 |
| AVG. HEAD ON TOP OF LAYER 16 | 0.0040 | | |
| PERC./LEAKAGE THROUGH LAYER 18 | 0.000000 | 0.000 | 0.00 |
| CHANGE IN WATER STORAGE | -9.832 | -1138548.000 | -21.74 |
| SOIL WATER AT START OF YEAR | 345.130 | 39965076.000 | |
| SOIL WATER AT END OF YEAR | 335.298 | 38826528.000 | |
| SNOW WATER AT START OF YEAR | 0.000 | 0.000 | 0.00 |
| SNOW WATER AT END OF YEAR | 0.000 | 0.000 | 0.00 |
| ANNUAL WATER BUDGET BALANCE | 0.0000 | -2.973 | 0.00 |

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ANNUAL TOTALS FOR YEAR 2

| | INCHES | CU. FEET | PERCENT |
|---|---|---|---|
| PRECIPITATION | 50.92 | 5896383.500 | 100.00 |
| RUNOFF | 0.038 | 4391.346 | 0.07 |
| EVAPOTRANSPIRATION | 33.530 | 3882713.500 | 65.85 |
| DRAINAGE COLLECTED FROM LAYER 3 | 18.1754 | 2104653.500 | 35.69 |
| PERC./LEAKAGE THROUGH LAYER 4 | 0.000684 | 79.166 | 0.00 |
| AVG. HEAD ON TOP OF LAYER 4 | 0.0021 | | |
| DRAINAGE COLLECTED FROM LAYER 15 | 14.1054 | 1633364.500 | 27.70 |
| PERC./LEAKAGE THROUGH LAYER 16 | 0.000001 | 0.117 | 0.00 |
| AVG. HEAD ON TOP OF LAYER 16 | 0.0058 | | |
| PERC./LEAKAGE THROUGH LAYER 18 | 0.000000 | 0.000 | 0.00 |
| CHANGE IN WATER STORAGE | -14.929 | -1728738.870 | -29.32 |
| SOIL WATER AT START OF YEAR | 335.298 | 38826528.000 | |
| SOIL WATER AT END OF YEAR | 320.369 | 37097788.000 | |
| SNOW WATER AT START OF YEAR | 0.000 | 0.000 | 0.00 |
| SNOW WATER AT END OF YEAR | 0.000 | 0.000 | 0.00 |
| ANNUAL WATER BUDGET BALANCE | 0.0000 | -0.545 | 0.00 |

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ANNUAL TOTALS FOR YEAR 3

| | INCHES | CU. FEET | PERCENT |
|---|---|---|---|
| PRECIPITATION | 57.05 | 6606218.500 | 100.00 |
| RUNOFF | 0.144 | 16646.311 | 0.25 |
| EVAPOTRANSPIRATION | 35.775 | 4142614.000 | 62.71 |
| DRAINAGE COLLECTED FROM LAYER 3 | 20.3652 | 2358233.500 | 35.70 |
| PERC./LEAKAGE THROUGH LAYER 4 | 0.000764 | 88.484 | 0.00 |
| AVG. HEAD ON TOP OF LAYER 4 | 0.0024 | | |
| DRAINAGE COLLECTED FROM LAYER 15 | 9.6348 | 1115677.870 | 16.89 |
| PERC./LEAKAGE THROUGH LAYER 16 | 0.000001 | 0.115 | 0.00 |
| AVG. HEAD ON TOP OF LAYER 16 | 0.0039 | | |
| PERC./LEAKAGE THROUGH LAYER 18 | 0.000000 | 0.000 | 0.00 |
| CHANGE IN WATER STORAGE | -8.869 | -1026950.620 | -15.55 |
| SOIL WATER AT START OF YEAR | 320.369 | 37097788.000 | |
| SOIL WATER AT END OF YEAR | 311.147 | 36029924.000 | |
| SNOW WATER AT START OF YEAR | 0.000 | 0.000 | 0.00 |
| SNOW WATER AT END OF YEAR | 0.353 | 40913.371 | 0.62 |
| ANNUAL WATER BUDGET BALANCE | 0.0000 | -2.309 | 0.00 |

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ANNUAL TOTALS FOR YEAR 4

| | INCHES | CU. FEET | PERCENT |
|---|---|---|---|
| PRECIPITATION | 40.75 | 4718727.500 | 100.00 |
| RUNOFF | 0.000 | 0.000 | 0.00 |
| EVAPOTRANSPIRATION | 32.854 | 3804413.750 | 80.62 |
| DRAINAGE COLLECTED FROM LAYER 3 | 7.5287 | 871806.437 | 18.48 |
| PERC./LEAKAGE THROUGH LAYER 4 | 0.000313 | 36.214 | 0.00 |
| AVG. HEAD ON TOP OF LAYER 4 | 0.0009 | | |
| DRAINAGE COLLECTED FROM LAYER 15 | 10.8226 | 1253229.120 | 26.56 |
| PERC./LEAKAGE THROUGH LAYER 16 | 0.000001 | 0.116 | 0.00 |
| AVG. HEAD ON TOP OF LAYER 16 | 0.0044 | | |
| PERC./LEAKAGE THROUGH LAYER 18 | 0.000000 | 0.000 | 0.00 |
| CHANGE IN WATER STORAGE | -10.456 | -1210721.750 | -25.66 |
| SOIL WATER AT START OF YEAR | 311.147 | 36029924.000 | |
| SOIL WATER AT END OF YEAR | 301.045 | 34860116.000 | |
| SNOW WATER AT START OF YEAR | 0.353 | 40913.371 | 0.87 |
| SNOW WATER AT END OF YEAR | 0.000 | 0.000 | 0.00 |
| ANNUAL WATER BUDGET BALANCE | 0.0000 | 0.238 | 0.00 |

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ANNUAL TOTALS FOR YEAR 5

| | INCHES | CU. FEET | PERCENT |
|---|---|---|---|
| PRECIPITATION | 52.97 | 6133767.000 | 100.00 |
| RUNOFF | 0.002 | 200.717 | 0.00 |
| EVAPOTRANSPIRATION | 33.205 | 3845028.000 | 62.69 |
| DRAINAGE COLLECTED FROM LAYER 3 | 17.9448 | 2077956.620 | 33.88 |
| PERC./LEAKAGE THROUGH LAYER 4 | 0.000686 | 79.435 | 0.00 |
| AVG. HEAD ON TOP OF LAYER 4 | 0.0021 | | |
| DRAINAGE COLLECTED FROM LAYER 15 | 0.1560 | 18069.053 | 0.29 |
| PERC./LEAKAGE THROUGH LAYER 16 | 0.000000 | 0.003 | 0.00 |
| AVG. HEAD ON TOP OF LAYER 16 | 0.0001 | | |
| PERC./LEAKAGE THROUGH LAYER 18 | 0.000000 | 0.000 | 0.00 |
| CHANGE IN WATER STORAGE | 1.663 | 192513.219 | 3.14 |
| SOIL WATER AT START OF YEAR | 301.045 | 34860116.000 | |
| SOIL WATER AT END OF YEAR | 302.708 | 35052628.000 | |
| SNOW WATER AT START OF YEAR | 0.000 | 0.000 | 0.00 |
| SNOW WATER AT END OF YEAR | 0.000 | 0.000 | 0.00 |
| ANNUAL WATER BUDGET BALANCE | 0.0000 | -0.459 | 0.00 |

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ANNUAL TOTALS FOR YEAR 6

| | INCHES | CU. FEET | PERCENT |
|---|---|---|---|
| PRECIPITATION | 54.35 | 6293566.500 | 100.00 |
| RUNOFF | 0.000 | 0.000 | 0.00 |
| EVAPOTRANSPIRATION | 34.861 | 4036744.500 | 64.14 |
| DRAINAGE COLLECTED FROM LAYER 3 | 21.1902 | 2453763.250 | 38.99 |
| PERC./LEAKAGE THROUGH LAYER 4 | 0.000802 | 92.869 | 0.00 |
| AVG. HEAD ON TOP OF LAYER 4 | 0.0025 | | |
| DRAINAGE COLLECTED FROM LAYER 15 | 0.0000 | 0.000 | 0.00 |
| PERC./LEAKAGE THROUGH LAYER 16 | 0.000000 | 0.000 | 0.00 |
| AVG. HEAD ON TOP OF LAYER 16 | 0.0000 | | |
| PERC./LEAKAGE THROUGH LAYER 18 | 0.000000 | 0.000 | 0.00 |
| CHANGE IN WATER STORAGE | -1.701 | -196937.594 | -3.13 |
| SOIL WATER AT START OF YEAR | 302.708 | 35052628.000 | |
| SOIL WATER AT END OF YEAR | 301.007 | 34855692.000 | |
| SNOW WATER AT START OF YEAR | 0.000 | 0.000 | 0.00 |
| SNOW WATER AT END OF YEAR | 0.000 | 0.000 | 0.00 |
| ANNUAL WATER BUDGET BALANCE | 0.0000 | -3.313 | 0.00 |

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ANNUAL TOTALS FOR YEAR 7

| | INCHES | CU. FEET | PERCENT |
|---|---|---|---|
| PRECIPITATION | 37.96 | 4395654.000 | 100.00 |
| RUNOFF | 0.000 | 0.000 | 0.00 |
| EVAPOTRANSPIRATION | 30.260 | 3503998.750 | 79.72 |
| DRAINAGE COLLECTED FROM LAYER 3 | 8.5043 | 984775.312 | 22.40 |
| PERC./LEAKAGE THROUGH LAYER 4 | 0.000340 | 39.368 | 0.00 |
| AVG. HEAD ON TOP OF LAYER 4 | 0.0010 | | |
| DRAINAGE COLLECTED FROM LAYER 15 | 0.0000 | 0.000 | 0.00 |
| PERC./LEAKAGE THROUGH LAYER 16 | 0.000000 | 0.000 | 0.00 |
| AVG. HEAD ON TOP OF LAYER 16 | 0.0000 | | |
| PERC./LEAKAGE THROUGH LAYER 18 | 0.000000 | 0.000 | 0.00 |
| CHANGE IN WATER STORAGE | -0.804 | -93120.320 | -2.12 |
| SOIL WATER AT START OF YEAR | 301.007 | 34855692.000 | |
| SOIL WATER AT END OF YEAR | 300.203 | 34762572.000 | |
| SNOW WATER AT START OF YEAR | 0.000 | 0.000 | 0.00 |
| SNOW WATER AT END OF YEAR | 0.000 | 0.000 | 0.00 |
| ANNUAL WATER BUDGET BALANCE | 0.0000 | 0.331 | 0.00 |

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ANNUAL TOTALS FOR YEAR 8

| | INCHES | CU. FEET | PERCENT |
|---|---|---|---|
| PRECIPITATION | 51.69 | 5985545.000 | 100.00 |
| RUNOFF | 0.618 | 71558.328 | 1.20 |
| EVAPOTRANSPIRATION | 31.347 | 3629852.250 | 60.64 |
| DRAINAGE COLLECTED FROM LAYER 3 | 17.1354 | 1984226.620 | 33.15 |
| PERC./LEAKAGE THROUGH LAYER 4 | 0.000630 | 72.953 | 0.00 |
| AVG. HEAD ON TOP OF LAYER 4 | 0.0020 | | |
| DRAINAGE COLLECTED FROM LAYER 15 | 0.0000 | 0.000 | 0.00 |
| PERC./LEAKAGE THROUGH LAYER 16 | 0.000000 | 0.000 | 0.00 |
| AVG. HEAD ON TOP OF LAYER 16 | 0.0000 | | |
| PERC./LEAKAGE THROUGH LAYER 18 | 0.000000 | 0.000 | 0.00 |
| CHANGE IN WATER STORAGE | 2.590 | 299906.750 | 5.01 |
| SOIL WATER AT START OF YEAR | 300.203 | 34762572.000 | |
| SOIL WATER AT END OF YEAR | 302.793 | 35062480.000 | |
| SNOW WATER AT START OF YEAR | 0.000 | 0.000 | 0.00 |
| SNOW WATER AT END OF YEAR | 0.000 | 0.000 | 0.00 |
| ANNUAL WATER BUDGET BALANCE | 0.0000 | 0.911 | 0.00 |

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ANNUAL TOTALS FOR YEAR 9

| | INCHES | CU. FEET | PERCENT |
|---|---|---|---|
| PRECIPITATION | 50.73 | 5874381.000 | 100.00 |
| RUNOFF | 0.000 | 0.000 | 0.00 |
| EVAPOTRANSPIRATION | 36.651 | 4244094.500 | 72.25 |
| DRAINAGE COLLECTED FROM LAYER 3 | 14.8052 | 1714401.370 | 29.18 |
| PERC./LEAKAGE THROUGH LAYER 4 | 0.000585 | 67.752 | 0.00 |
| AVG. HEAD ON TOP OF LAYER 4 | 0.0017 | | |
| DRAINAGE COLLECTED FROM LAYER 15 | 0.0000 | 0.000 | 0.00 |
| PERC./LEAKAGE THROUGH LAYER 16 | 0.000000 | 0.000 | 0.00 |
| AVG. HEAD ON TOP OF LAYER 16 | 0.0000 | | |
| PERC./LEAKAGE THROUGH LAYER 18 | 0.000000 | 0.000 | 0.00 |
| CHANGE IN WATER STORAGE | -0.726 | -84110.453 | -1.43 |
| SOIL WATER AT START OF YEAR | 302.793 | 35062480.000 | |
| SOIL WATER AT END OF YEAR | 300.974 | 34851908.000 | |
| SNOW WATER AT START OF YEAR | 0.000 | 0.000 | 0.00 |
| SNOW WATER AT END OF YEAR | 1.092 | 126460.703 | 2.15 |
| ANNUAL WATER BUDGET BALANCE | 0.0000 | -4.638 | 0.00 |

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ANNUAL TOTALS FOR YEAR 10

| | INCHES | CU. FEET | PERCENT |
|---|---|---|---|
| PRECIPITATION | 40.61 | 4702515.500 | 100.00 |
| RUNOFF | 0.005 | 596.066 | 0.01 |
| EVAPOTRANSPIRATION | 32.848 | 3803668.000 | 80.89 |
| DRAINAGE COLLECTED FROM LAYER 3 | 9.5292 | 1103449.620 | 23.47 |
| PERC./LEAKAGE THROUGH LAYER 4 | 0.000388 | 44.944 | 0.00 |
| AVG. HEAD ON TOP OF LAYER 4 | 0.0011 | | |
| DRAINAGE COLLECTED FROM LAYER 15 | 0.0000 | 0.000 | 0.00 |
| PERC./LEAKAGE THROUGH LAYER 16 | 0.000000 | 0.000 | 0.00 |
| AVG. HEAD ON TOP OF LAYER 16 | 0.0000 | | |
| PERC./LEAKAGE THROUGH LAYER 18 | 0.000000 | 0.000 | 0.00 |
| CHANGE IN WATER STORAGE | -1.772 | -205194.750 | -4.36 |
| SOIL WATER AT START OF YEAR | 300.974 | 34851908.000 | |
| SOIL WATER AT END OF YEAR | 300.294 | 34773172.000 | |
| SNOW WATER AT START OF YEAR | 1.092 | 126460.703 | 2.69 |
| SNOW WATER AT END OF YEAR | 0.000 | 0.000 | 0.00 |
| ANNUAL WATER BUDGET BALANCE | 0.0000 | -3.595 | 0.00 |

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AVERAGE ANNUAL TOTALS & (STD. DEVIATIONS) FOR YEARS 1 THROUGH 100

| | INCHES | | CU. FEET | PERCENT |
|---|---|---|---|---|
| PRECIPITATION | 49.24 | ( 7.134) | 5701694.0 | 100.00 |
| RUNOFF | 0.255 | ( 0.6190) | 29552.04 | 0.518 |
| EVAPOTRANSPIRATION | 33.138 | ( 3.5047) | 3837277.50 | 67.301 |
| LATERAL DRAINAGE COLLECTED FROM LAYER 3 | 15.84513 | ( 4.84305) | 1,834,818.750 | 32.18024 |
| PERCOLATION/LEAKAGE THROUGH LAYER 4 | 0.00060 | ( 0.00017) | 69.881 | 0.00123 |
| AVERAGE HEAD ON TOP OF LAYER 4 | 0.002 | ( 0.001) | | |
| LATERAL DRAINAGE COLLECTED FROM LAYER 15 | 0.44598 | ( 2.21713) | 51643.133 | 0.90575 |
| PERCOLATION/LEAKAGE THROUGH LAYER 16 | 0.00000 | ( 0.00000) | 0.005 | 0.00000 |
| AVERAGE HEAD ON TOP OF LAYER 16 | 0.000 | ( 0.001) | | |
| PERCOLATION/LEAKAGE THROUGH LAYER 18 | 0.00000 | ( 0.00000) | 0.000 | 0.00000 |
| CHANGE IN WATER STORAGE | -0.446 | ( 2.5149) | -51597.41 | -0.905 |

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PEAK DAILY VALUES FOR YEARS 1 THROUGH 100

| | (INCHES) | (CU. FT.) |
|---|---|---|
| PRECIPITATION | 5.58 | 646147.250 |
| RUNOFF | 1.516 | 175558.0310 |
| DRAINAGE COLLECTED FROM LAYER 3 | 1.88441 | 218209.54700 |
| PERCOLATION/LEAKAGE THROUGH LAYER 4 | 0.000055 | 6.34607 |
| AVERAGE HEAD ON TOP OF LAYER 4 | 0.081 | |
| MAXIMUM HEAD ON TOP OF LAYER 4 | 0.159 | |
| LOCATION OF MAXIMUM HEAD IN LAYER 3 (DISTANCE FROM DRAIN) | 0.6 FEET | |
| DRAINAGE COLLECTED FROM LAYER 15 | 0.05757 | 6666.55811 |
| PERCOLATION/LEAKAGE THROUGH LAYER 16 | 0.000000 | 0.00053 |
| AVERAGE HEAD ON TOP OF LAYER 16 | 0.008 | |
| MAXIMUM HEAD ON TOP OF LAYER 16 | 0.013 | |
| LOCATION OF MAXIMUM HEAD IN LAYER 15 (DISTANCE FROM DRAIN) | 0.7 FEET | |
| PERCOLATION/LEAKAGE THROUGH LAYER 18 | 0.000000 | 0.00000 |
| SNOW WATER | 3.87 | 448243.8120 |

MAXIMUM VEG. SOIL WATER (VOL/VOL) 0.4900

MINIMUM VEG. SOIL WATER (VOL/VOL) 0.1260

*** 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 | 2.1982 | 0.2442 |
| 2 | 4.7386 | 0.3159 |
| 3 | 0.0040 | 0.0234 |
| 4 | 0.0000 | 0.0000 |
| 5 | 5.0103 | 0.2784 |
| 6 | 33.7155 | 0.2660 |
| 7 | 33.7155 | 0.2660 |
| 8 | 33.7155 | 0.2660 |
| 9 | 33.7155 | 0.2660 |
| 10 | 33.7155 | 0.2660 |
| 11 | 33.7155 | 0.2660 |
| 12 | 33.7155 | 0.2660 |
| 13 | 33.7155 | 0.2660 |
| 14 | 3.2640 | 0.2720 |
| 15 | 0.0017 | 0.0100 |
| 16 | 0.0000 | 0.0000 |
| 17 | 0.0366 | 0.6101 |
| 18 | 10.6080 | 0.2210 |
| SNOW WATER | 0.000 | |

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FINAL WATER STORAGE AT END OF YEAR 10

| LAYER | (INCHES) | (VOL/VOL) |
|---|---|---|
| 1 | 4.9540 | 0.2752 |
| 2 | 37.6371 | 0.2969 |
| 3 | 37.6697 | 0.2972 |
| 4 | 39.9804 | 0.3154 |
| 5 | 40.6592 | 0.3208 |
| 6 | 41.1976 | 0.3250 |
| 7 | 38.5121 | 0.3038 |
| 8 | 39.1572 | 0.3089 |
| 9 | 38.8516 | 0.3065 |
| 10 | 3.8745 | 0.3229 |
| 11 | 0.0055 | 0.0322 |
| 12 | 0.0000 | 0.0000 |
| 13 | 0.0366 | 0.6102 |
| 14 | 10.6080 | 0.2210 |
| SNOW WATER | 0.000 | |

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## Leachate Collection and Removal Pipe

- LCR Pipe Flow Calculations
- LCR Pipe Stresses

**Marshall FGD Residue Landfill**
**LCRS Pipe - Flow Calculations**

| | | | |
|---|---|---|---|
| **Job No.** | **046003** | | |
| **Comp. By** | **DHB** | **Date** | **12/16/2005** |
| **Chkd. By** | DEB | **Date** | 12/19/05 |
| **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 | = | 9,158 | gal/ac/day |
| pipe spacing of LCRS piping | = | 75 | ft |
| maximum length of 6" collector pipe | = | 801 | ft |
| area that contributes to pipe | = | 101174 | $ft^2$ |
| | = | 2.32 | ac |

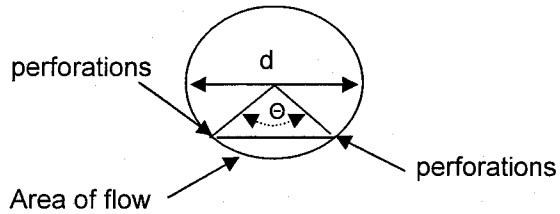
$q_{max}$ = (q from 24 hr, 25 yr storm) x area that contributes to pipe

$q_{max}$ = 0.0329 $ft^3$/sec

compute $Q_{available}$ in 6" dia. pipe (Manning)

$$Q = \frac{1.49}{n} A R^{2/3} S^{1/2}$$

Pipe (typ)




Θ = 120°

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:

$q_{max} = 0.0329$ $ft^3/sec$

$Q_{available} > q_{max}$ **OK**

**The 6" pipe has adequate flow capacity**

**Check flow capacity of 8" dia. pipe**

| | | | | |
|---|---|---|---|---|
| Cell 2 has largest area | = | 12.8 | ac | |
| $q_{max}$, from 24 hr, 25 | = | 9,158 | gal/ac/day | |
| $q_{max}$ | = | 0.18 | $ft^3/sec$ | |
| S, slope of pipe | = | 0.011 | 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 | = | 1.61 | $ft^3/sec$ | |

Check against $q_{max}$ from 24 hour, 25 year storm:

| | | | | |
|---|---|---|---|---|
| available flow capacity, Qa: | = | 1.61 | $ft^3/sec$ | |
| $q_{max}$ | = | 0.18 | $ft^3/sec$ | |
| 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 164 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 = 4.31 $ft^3/sec$

$q_{max}$, from 24 hr, 25 = 0.18 $ft^3/sec$ **OK**

**Check the Hole Spacing and Hole Diameter for All Cells:**

**Marshall FGD Residue Landfill**
**LCRS Pipe - Flow Calculations**

| | | | |
|---|---|---|---|
| **Job No.** | **046003** | | |
| **Comp. By** | **DHB** | **Date** | **12/16/2005** |
| **Chkd. By** | DEB | **Date** | 12/19/05 |
| **Sheet** | 5 | **of** | 5 |

Use the calculation process to evaluate the flow capacity for all Cells.

$q_{max}$, from 24 hr, 25 yr storm = 9,158 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 | 9.8 | 0.14 | 109 | 2.86 | **OK** |
| 2 | 12.75 | 0.18 | 164 | 4.31 | **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.

DriscoPlex and PERFORMANCE PIPE™ are trademarks of Chevron Phillips Chemical Company LP

Reference 2



May 2001 Supercedes all previous publications
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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 | 83 feet of gypsum over Operational Soil Cover | 20,000 lb tire load |
| Operations | 8" | 17 | 80 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.

| | | | | |
|---|---|---|---|---|
| **Marshall FGD Residue Landfill** | **Job No.** | **46003** | **Date** | **12/19/2005** |
| **Pipe Stress Calculations** | **Comp. By** | **DHB** | **Date** | **12/19/2005** |
| | **Chkd. By** | DEB | | |
| | **Sheet** | 1 | | |

**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 | 100 lb/ft$^3$ | (assumed) |
| Thickness | 1 ft | (measured from bottom of pipe) |

Waste:

| | | |
|---|---|---|
| Density | 60 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 = 100 psf
= 0.7 psi

Static Load:

Ps = Operational Cover = depth of cover to top of pipe * density of cover

| | | | |
|---|---|---|---|
| Ps | = | 100 | psf |
| Ps | = | 0.7 | 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 = 91.21 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 = | | 99.5 psi | |

**Wall Crushing Check for Operational Loads**

SA = Compressive Strength

$$SA = \frac{SDR - 1}{2} * Pt$$

$$SF = \frac{1500}{SA}$$

Pt = 99.5 psi

SDR = 17

$$SA = \frac{SDR - 1}{2} * Pt$$

SA = 796 psi

Factor of safety, FS:

$$SF = \frac{1500}{SA}$$

= 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,
$S_A$ = 796 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 = 1.4 **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'

$$SF = 2$$

$$E'\ min = SF \times E'$$

$$E'\ min = 6000psi$$

**Check Ring Deflection**

$$\%\ Soil\ Strain = \varepsilon_s = \frac{Pt * 100}{E'min}$$

$$\varepsilon_s = \frac{Pt \times 100}{E'min}$$

$$\varepsilon_s = 1.7\%$$

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 | 100 lb/ft$^3$ | (assumed) |
| Thickness | 1.5 ft | (measured from bottom of pipe) |

Waste:

| | | |
|---|---|---|
| Density | 60 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 = 100 psf
= 0.7 psi

Static Load:

Ps = Operational Cover = depth of cover to top of pipe * density of cover
Ps = 150 psf
Ps = 1.0 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 = 33.63 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**

SA = Compressive Strength

$$SA = \frac{SDR - 1}{2} * Pt$$

$$SF = \frac{1500}{SA}$$

Pt = 37 psi

SDR = 17

$$SA = \frac{SDR - 1}{2} * Pt$$

SA = 294 psi

Factor of safety, FS:

$$SF = \frac{1500}{SA}$$

= 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,
$S_A$ = 294 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

$$E' = 3000 \text{ psi}$$

Apply Safety Factor (SF) to E'

$$SF = 2$$

$$E' \text{ min} = SF \times E'$$

$$E' \text{ min} = 6000 \text{psi}$$

**Check Ring Deflection**

$$\% \text{ Soil Strain} = \varepsilon_s = \frac{Pt * 100}{E'_{min}}$$

$$\varepsilon_s = \frac{Pt \times 100}{E'min}$$

$$\varepsilon_s = 0.6\%$$

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 | 100 | $lb/ft^3$ | (assumed) |
| Thickness | 1 | ft | (measured from bottom of pipe) |

Waste:

| | | | |
|---|---|---|---|
| Density | 60 | $lb/ft^3$ | |
| 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 = 160 psf
= 1.1 psi

Static Load:

Ps = Operational Cover = depth of cover to top of pipe * density of cover
Ps = 160 psf
Ps = 1.1 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 = 45.41 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 | = | 49.5 psi | |

**Wall Crushing Check for Operational Loads**

SA = Compressive Strength

$$SA = \frac{SDR - 1}{2} * Pt$$

$$SF = \frac{1500}{SA}$$

Pt = 49.5 psi

SDR = 17

$$SA = \frac{SDR - 1}{2} * Pt$$

SA = 396 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,
$S_A$ = 396 psi
to find :
E = Modulus of Elasticity

E = 32,000 psi
SDR = 17

$$Pc = \frac{2.32 * E}{(SDR)^3}$$

Pc = 15.1 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 = 170 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**

$$\% \text{ Soil Strain} = \varepsilon_s = \frac{Pt * 100}{E'min}$$

$$\varepsilon_s = \frac{Pt \times 100}{E'min}$$

$\varepsilon_s$ = 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 | 100 lb/ft$^3$ | (assumed) |
| Thickness | 1 ft | (measured from bottom of pipe) |

Waste:

| | | |
|---|---|---|
| Density | 60 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 = 160 psf
= 1.1 psi

Static Load:

Ps = Operational Cover = depth of cover to top of pipe * density of cover
Ps = 160 psf
Ps = 1.1 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 = 57.13 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 | = | 62.3 psi | |

**Wall Crushing Check for Operational Loads**

SA = Compressive Strength

$$SA = \frac{SDR - 1}{2} * Pt$$

$$SF = \frac{1500}{SA}$$

Pt = 62.3 psi

SDR = 17

$$SA = \frac{SDR - 1}{2} * Pt$$

SA = 499 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,
$S_A$ = 499 psi
to find :
E = Modulus of Elasticity

E = 29,000 psi
SDR = 17

$$Pc = \frac{2.32 * E}{(SDR)^3}$$

Pc = 13.7 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 = 162 psi

Calculate the Factor of Safety, SF
SF = Pcb ÷ Pt
SF = 3 **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 = 83 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 | 100 | lb/ft$^3$ | (assumed) |
| Thickness | 1 | ft | (measured from bottom of pipe) |

Waste:

| | | | |
|---|---|---|---|
| Density | 60 | lb/ft$^3$ | |
| Thickness | 83 | 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 = 5080 psf
= 35.3 psi

Static Load:

Ps = Operational Cover = depth of cover to top of pipe * density of cover
Ps = 5080 psf
Ps = 35.3 psi

**6" Diameter Pipe - Operational Loads**
**Calculate Stresses in 6" Dia. Pipe From Operational Loads**
**Depth of Gypsum over Operational Cover = 83 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 = 83.5 ft
x = 0.0 ft
R = 83.5 ft

Load Factor = 1.5

Plive = 2 psf
= 0.01 psi

This is the load from 1 track. Add load for other track.

H = 83.5 ft
x = 5.0 ft
R = 83.6 ft

Plive = 2.0 psf
0.0 psi

Ps + Plive = 35.31 psi

**6" Diameter Pipe - Operational Loads**
**Calculate Stresses in 6" Dia. Pipe From Operational Loads**
**Depth of Gypsum over Operational Cover = 83 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 | = | 38.5 psi | |

**Wall Crushing Check for Operational Loads**

SA = Compressive Strength

$$SA = \frac{SDR - 1}{2} * Pt$$

$$SF = \frac{1500}{SA}$$

Pt = 38.5 psi

SDR = 17

$$SA = \frac{SDR - 1}{2} * Pt$$

SA = 308 psi

Factor of safety, FS:

$$SF = \frac{1500}{SA}$$

= 4.9 **OK**

**6" Diameter Pipe - Operational Loads**
**Calculate Stresses in 6" Dia. Pipe From Operational Loads**
**Depth of Gypsum over Operational Cover = 83 feet**

**Critical Collapse**

Calculate the critical-collapse differential pressure, Pc

$$Pc = \frac{2.32 * E}{(SDR)^3}$$

From Chart 25, Reference 1,
$S_A$ = 308 psi
to find :
E = Modulus of Elasticity

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**

**6" Diameter Pipe - Operational Loads**
**Calculate Stresses in 6" Dia. Pipe From Operational Loads**
**Depth of Gypsum over Operational Cover = 83 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**

$$\% \text{ Soil Strain} = \varepsilon_s = \frac{Pt * 100}{E'min}$$

$$\varepsilon_s = \frac{Pt \times 100}{E'min}$$

$\varepsilon_s$ = 0.6%

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 83 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 = 80 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 | 100 | lb/ft$^3$ | (assumed) |
| Thickness | 1 | ft | (measured from bottom of pipe) |

Waste:

| | | | |
|---|---|---|---|
| Density | 60 | lb/ft$^3$ | |
| Thickness | 80 | 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 = 4900 psf
= 34.0 psi

Static Load:

Ps = Operational Cover = depth of cover to top of pipe * density of cover
Ps = 4900 psf
Ps = 34.0 psi

**8" Diameter Pipe - Operational Loads**
**Calculate Stresses in 8" Dia. Pipe From Operational Loads**
**Depth of Gypsum over Operational Cover = 80 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 = 80.3333 ft
x = 0.0 ft
R = 80.3 ft

Load Factor = 1.5

Plive = 2 psf
= 0.02 psi

This is the load from 1 track. Add load for other track.

H = 80.3333 ft
x = 5.0 ft
R = 80.5 ft

Plive = 2.2 psf
0.0 psi

Ps + Plive = 34.06 psi

**8" Diameter Pipe - Operational Loads**
**Calculate Stresses in 8" Dia. Pipe From Operational Loads**
**Depth of Gypsum over Operational Cover = 80 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 | = | 37.2 psi | |

**Wall Crushing Check for Operational Loads**

SA = Compressive Strength

$$SA = \frac{SDR - 1}{2} * Pt$$

$$SF = \frac{1500}{SA}$$

Pt = 37.2 psi

SDR = 17

$$SA = \frac{SDR - 1}{2} * Pt$$

SA = 297 psi

Factor of safety, FS:

$$SF = \frac{1500}{SA}$$

= 5.0 **OK**

**8" Diameter Pipe - Operational Loads**
**Calculate Stresses in 8" Dia. Pipe From Operational Loads**
**Depth of Gypsum over Operational Cover = 80 feet**

**Critical Collapse**

Calculate the critical-collapse differential pressure, Pc

$$Pc = \frac{2.32 * E}{(SDR)^3}$$

From Chart 25, Reference 1,
SA = 297 psi
to find :
E = Modulus of Elasticity

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 - Operational Loads**
**Calculate Stresses in 8" Dia. Pipe From Operational Loads**
**Depth of Gypsum over Operational Cover = 80 feet**

**Soil Modulus Required to Resist Buckling**

$$E' = \frac{Pcb^2}{0.64 * Pc}$$

Assume Pcb = Pt

$$E' = 3000 \text{ psi}$$

Apply Safety Factor (SF) to E'

$$SF = 2$$

$$E' \min = SF \times E'$$

$$E' \min = 6000 \text{psi}$$

**Check Ring Deflection**

$$\% \text{ Soil Strain} = \varepsilon_s = \frac{Pt * 100}{E'_{min}}$$

$$\varepsilon_s = \frac{Pt \times 100}{E'min}$$

$$\varepsilon_s = 0.6\%$$

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 operational loads with 80 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. | | | Maximum External Pressure psi | | | Maximum Deflection, % after installation | | |
|---|---|---|---|---|---|---|---|---|---|
| | Soil Modulus, psi* | | | Soil Modulus, psi* | | | 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
$\mu$ = Poission's Ratio
$\mu$ = .45 for Driscopipe
E = stress and time dependent tensile modulus of elasticity, psi

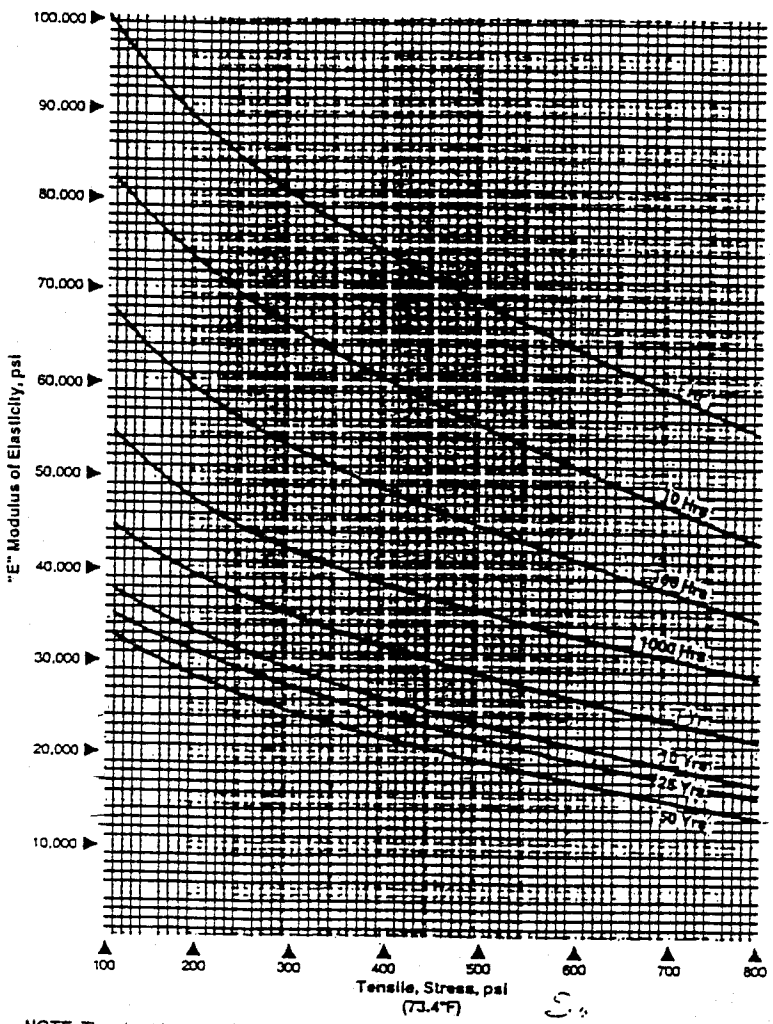
In a direct burial pressurized pipeline, the internal pressure is usually great enough to exceed the external critical-buckling soil pressure. When a pressurized line is to be shut down for a period, wall buckling should be examined.

*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, $P_C$:

$$P_C = \frac{2.32\,(E)}{(SDR)^3}$$

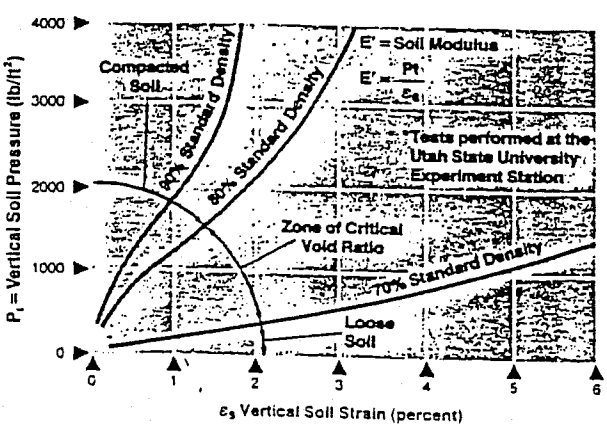
5. Calculate the soil modulus, E', by plotting the total external soil pressure, $P_t$, against a specified soil density to derive the soil strain as shown in the example problem on Chart 26.
6. Calculate the critical buckling pressure at the top of the pipe by the formula:

$$P_{CB} = 0.8\sqrt{E' \times P_c}$$

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

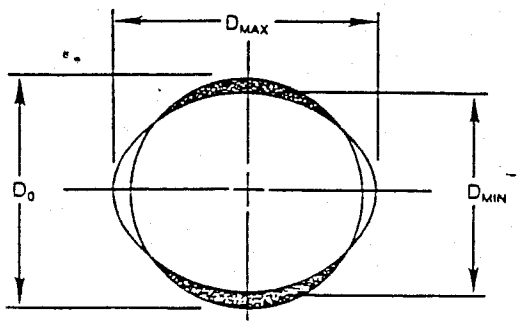
Note: The curves shown on this chart are sample curves for a granular soil. If other types of soil are used for backfill, such as clay or clay loam, curves should be developed from laboratory test data for the material used. Soil pressures greater than 4000 psf may be examined by extrapolating the slope 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 SDRs 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%.

$P_i$: Apparent External Pressure Due To Vacuum:

3 psi × 144 = 432 psf

TOTAL EXTERNAL PRESSURE: $P_t = P_S + P_L + P_i$

$P_t$ = 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:

   $$S_A = \frac{(SDR - 1)\,P_t}{2}$$

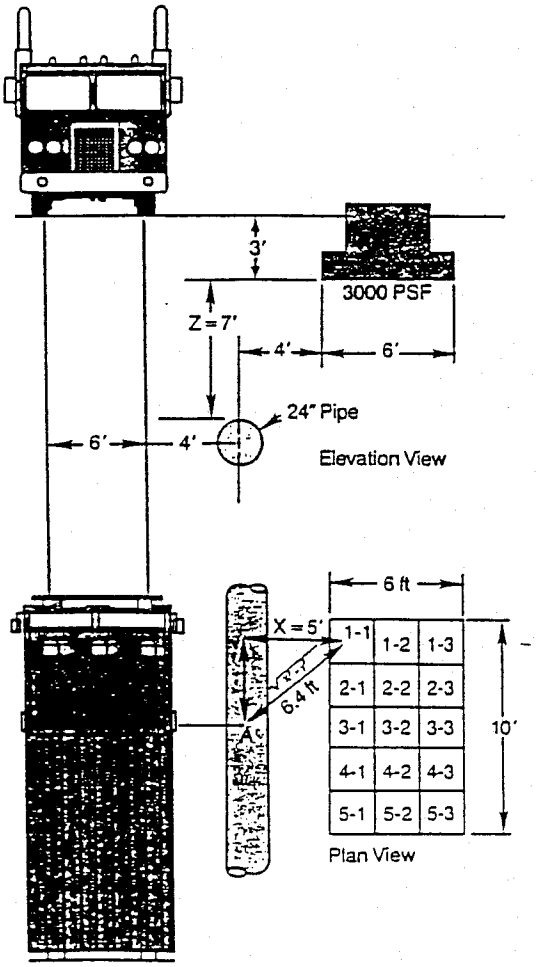
   (a) If $S_A$<1500 psi proceed to #3
   (b) If $S_A$>1500 psi consider a heavier pipe wall
3. Calculate the critical-collapse pressure, $P_C$, from this formula using the time dependent modulus of elasticity, E, rated at the stress level calculated above in #2 (see Chart 25).

   $$P_C = \frac{2.32\,E}{(SDR)^3}$$
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, $\xi_S$, by using soil stress/strain curves developed from field specimens or from Chart 26 on page 38.



**Stormwater Management**

- Cell 1 Stormwater Pond
- Cell 2 Stormwater Pond

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

Comp. By DHB Date 12-7-05
Chkd. By DEB Date 12-10-05

Project MARSHALL STEAM STA.
Subject CELL 1 STORMWATER POND.

Sheet No. 1 of 1
Job No. 0460003.001

D.A. = 19.5 Ac
C = .60
$I_{10}$ = 6.8 in/hr
$I_{25}$ = 7.0 in/hr.
$I_2$ = 4.8 in/hr

$Q_{10}$ = 0.6(6.8)(19.5) = 79.56 cfs
$Q_{25}$ = 0.6(7.0)(19.5) = 81.9 cfs
$Q_2$ = 0.6(4.8)(19.5) = 56.16 cfs.

SA/DA @ H = 4 ft, I = 10% = 0.49 ∴ SA MIN. = 4,162 ft
EXIST. BARREL ø = 42" REQ. 54" ø RISER
n FOR SMOOTH-WALLED HDPE = .009
FOR $Q_{25}$ = 81.9 cfs IN 42" ø BARREL: H = 4 ft w/ $Q_{CAP}$ = 107.52
PER TABLE 8.07b FOR CONCRETE w/ n = .013
INV. 42" = 823.0, ℄ = 824.75 ∴ ELEV. @ H = 828.75
REQ'D HEAD ABOVE CREST OF RISER = 1.5 ft.
CREST OF RISER ELEV. = 827.25
S.A. PROVIDED @ CREST OF RISER = 30,681 ft
VOL. BELOW CREST OF RISER = 30,681 (1.25) +
[(23717 + 8887)/2 (1.0)] = 54,653 ft³
VOL. BELOW CREST OF RISER DRAINED FROM TOP BY FAIRCLOTH SKIMMER IN 2 TO 3 DAYS.

5" SKIMMER WILL DRAIN 65,664 ft³ IN 2 DAYS.

$$\text{ORIFICE} = \frac{54{,}653\ ft^3 \cdot in^2}{3{,}283\ ft^3} = 16.65\ in^2$$

$\pi r^2 = 16.65\ in^2$ r = 2.30 in ⟹ USE 2.5" ORIFICE.

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

Comp. By DHB Date 12-7-05 Project MARSHALL STEAM STA. Sheet No. 1 of 1
Chkd. By DED Date 12/10/05 Subject CELL 2 STORMWATER POND Job No. 046003.001

D.A. = 30.2 Ac.
C = 0.6
$I_{25}$ = 7.0 in/hr
$Q_{25}$ = 0.6 (7.0)(30.2) = 126.84 cfs
SA/DA = 0.35 @ H = 6 FT, I = 10% ⇒ SA MIN = 4,605 ft.
EXIST. BARREL ø = 42" REQ. 54" ø RISER.

FOR $Q_{25}$ = 126.84 cfs IN 42" ø BARREL H = 6 FT. w/ $Q_{CAP}$ = 131.66
PER TABLE 8.07b FOR CONCRETE w/ n = .013

INV. 42" = 802.4, ℄ = 804.15 ∴ ELEV. @ H = 810.15
REQ'D HEAD ABOVE CREST OF RISER = 2.05 ft.
CREST OF RISER ELEV. = 808.10
S.A. PROVIDED @ CREST OF RISER = 27,335 ft
VOL. BELOW CREST OF RISER = $\left(\frac{27,335 + 20881}{2}\right)(3.9)$ = 94,021 $ft^3$

VOL. BELOW CREST OF RISER DRAINED FROM TOP BY FAIRCLOTH SKIMMER IN 2 TO 3 DAYS.

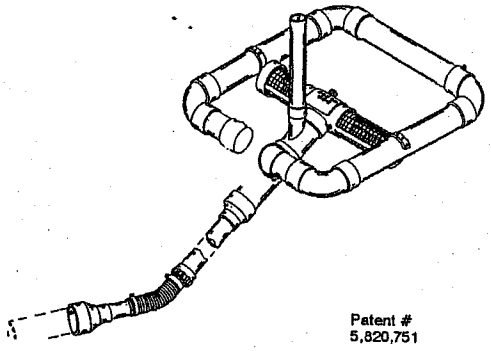
6" SKIMMER WILL DRAIN 103,680 $ft^3$ IN 2 DAYS.

ORIFICE = $\frac{94,021\ ft^3}{3,628\ ft^3} \cdot in^2$ = 25.92 $in^2$

$\pi r^2$ = 25.92 $in^2$ r = 2.87 in ⇒ USE 3" ORIFICE.

# Determining Orifice Size for the Faircloth Skimmers

March 2005




Determining the orifice required to drain the sediment basin in the required time involves: **#1**, determining the size skimmer required based on the volume of the sediment basin to be drained and the maximum capacity of the particular skimmer with the orifice wide open; and **#2**, then determining the radius of the orifice by dividing the volume of the basin to be drained by a factor for the number of days to drain the basin, which gives the required area of the orifice, then calculating the orifice radius using Area $= \pi r^2$ and solving for r. The cutter can be adjusted to that radius and the orifice cut in the plastic plug that fits into the inlet.

1. Approximate **skimmer maximum capacities** based on typical draw down requirements, which can vary between States and jurisdictions and watersheds. If one skimmer does not provide enough capacity, multiple skimmers can be used to drain the basin.

| | |
|---|---|
| 2" skimmer<br>with a 2" head | 3,283 cubic feet in **24 hours**<br>6,566 cubic feet in **2 days**<br>22,982 cubic feet in **7 days** |
| 2 1/2" skimmer:<br>with a 2" head | 5,500 cubic feet in **24 hours**<br>11,000 cubic feet in **2 days**<br>38,500 cubic feet in **7 days** |
| 3" skimmer:<br>with a 3" head<br>Revised 3-05 | 9,774 cubic feet in **24 hours**<br>19,547 cubic feet in **2 days**<br>68,415 cubic feet in **7 days** |
| 4" skimmer:<br>with a 3.3" head | 18,267 cubic feet in **24 hours**<br>36,534 cubic feet in **2 days**<br>127,869 cubic feet in **7 days** |
| 5" skimmer<br>with a 4" head | 32,832 cubic feet in **24 hours**<br>65,664 cubic feet in **2 days**<br>229,824 cubic feet in **7 days** |
| 6" skimmer<br>with a 5" Head | 51,840 cubic feet in **24 hours**<br>103,680 cubic feet in **2 days**<br>362,880 cubic feet in **7 days** |

2. **Factors** (in cubic feet of flow per square inch of opening through a **round** orifice for the draw down times shown) to use in determining the orifice radius for a particular basin volume to be drained. This quick method works because the orifice is centered and has a constant head.

An alternative method is to use the orifice equation with the head for the particular skimmer shown on the previous page and determine the required orifice to give the required flow for the volume and draw down time.

| | |
|---|---|
| 2" skimmer | 1,123 to drain the basin in **24 hours**<br>2,246 to drain the basin in **2 days**<br>7,861 to drain the basin in **7 days** |
| 2 1/2" skimmer: | 1,144 to drain the basin in **24 hours**<br>2,304 to drain the basin in **2 days**<br>8,064 to drain the basin in **7 days** |
| 3" skimmer:<br>Revised 3-05 | 1,382 to drain the basin in **24 hours**<br>2,765 to drain the basin in **2 days**<br>9,677 to drain the basin in **7 days** |
| 4" skimmer: | 1,454 to drain the basin in **24 hours**<br>2,909 to drain the basin in **2 days**<br>10,178 to drain the basin in **7 days** |
| 5" skimmer | 1,642 to drain the basin in **24 hours**<br>3,283 to drain the basin in **2 days**<br>11,491 to drain the basin in **7 days** |
| 6" skimmer | 1,814 to drain the basin in **24 hours**<br>3,628 to drain the basin in **2 days**<br>12,701 to drain the basin in **7 days** |

CFS @ S1

$\pi r^2$

The size skimmer necessary for the sediment basin and the required orifice radius for the skimmer should be shown on the sediment and erosion control plan for each basin. During the skimmer installation the required orifice can be cut in the plastic plug using the supplied cutter and installed in the skimmer using the instructions with the skimmer.

The plan review and enforcement authority may require calculations showing that the skimmer used can drain the basin in the required time.

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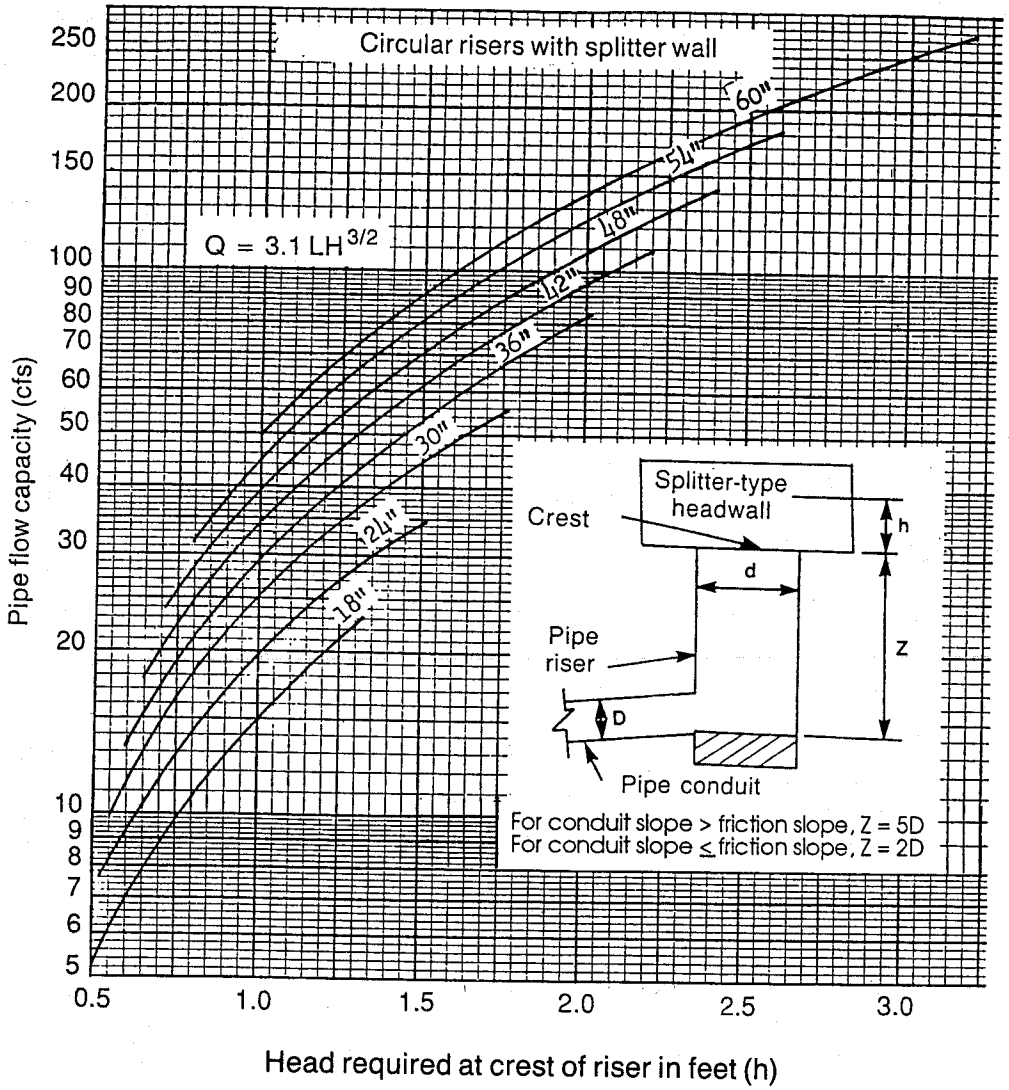
Revised 2-2-01; 3-3-05

## Table 8.07b
## Pipe Flow Chart for Design of Concrete Outlet Conduit
## (Q in cubic ft/sec)

For R/C Drop Inlet, $K_e + K_b = 0.65$ with 70 feet of R/C conduit, n = .013. Note correction factors for other pipe lengths.

| Dia. / H | 12" | 15" | 18" | 21" | 24" | 30" | 36" | 42" |
|---|---|---|---|---|---|---|---|---|
| 2 | 4.54 | 8.01 | 11.74 | 16.60 | 22.44 | 36.74 | 54.65 | 76.02 |
| 3 | 5.56 | 9.81 | 14.39 | 20.33 | 27.49 | 45.00 | 66.94 | 93.11 |
| 4 | 6.42 | 11.33 | 16.61 | 23.48 | 31.74 | 51.96 | 77.30 | 107.52 |
| 5 | 7.18 | 12.66 | 18.57 | 26.25 | 35.49 | 58.09 | 86.42 | 120.21 |
| 6 | 7.87 | 13.86 | 20.34 | 28.75 | 38.87 | 63.63 | 94.65 | 131.66 |
| 7 | 8.50 | 14.98 | 21.98 | 31.06 | 41.99 | 68.74 | 102.27 | 142.25 |
| 8 | 9.08 | 16.01 | 23.49 | 33.20 | 44.88 | 73.47 | 109.30 | 152.03 |
| 9 | 9.64 | 17.00 | 24.92 | 35.22 | 47.61 | 77.94 | 115.95 | 161.28 |
| 10 | 10.16 | 17.91 | 26.26 | 37.12 | 50.18 | 82.15 | 122.21 | 169.99 |
| 11 | 10.65 | 18.78 | 27.55 | 38.94 | 52.64 | 86.18 | 128.20 | 178.32 |
| 12 | 11.13 | 19.62 | 28.77 | 40.67 | 54.97 | 89.99 | 133.88 | 186.22 |
| 13 | 11.58 | 20.42 | 29.95 | 42.33 | 57.23 | 93.68 | 139.37 | 193.86 |
| 14 | 12.01 | 21.18 | 31.07 | 43.93 | 59.37 | 97.19 | 144.59 | 201.12 |
| 15 | 12.44 | 21.93 | 32.17 | 45.47 | 61.46 | 100.62 | 149.69 | 208.21 |
| 16 | 12.85 | 22.65 | 33.22 | 46.96 | 63.48 | 103.92 | 154.60 | 215.04 |
| 17 | 13.24 | 23.35 | 34.24 | 48.40 | 65.43 | 107.12 | 159.35 | 221.65 |
| 18 | 13.63 | 24.03 | 35.24 | 49.81 | 67.34 | 110.23 | 163.99 | 228.10 |
| 19 | 14.00 | 24.68 | 36.21 | 51.17 | 69.18 | 113.25 | 168.48 | 234.34 |
| 20 | 14.36 | 25.32 | 37.14 | 52.50 | 70.97 | 116.18 | 172.84 | 240.41 |
| 21 | 14.72 | 25.95 | 38.07 | 53.80 | 72.73 | 119.07 | 177.13 | 246.38 |
| 22 | 15.06 | 26.56 | 38.96 | 55.06 | 74.43 | 121.85 | 181.27 | 252.13 |
| 23 | 15.40 | 27.16 | 39.84 | 56.31 | 76.11 | 124.60 | 185.36 | 257.83 |
| 24 | 15.73 | 27.74 | 40.69 | 57.51 | 77.75 | 127.28 | 189.35 | 263.37 |
| 25 | 16.06 | 28.32 | 41.53 | 58.70 | 79.35 | 129.90 | 193.25 | 268.80 |
| L | Correction Factors For Other Pipe Lengths | | | | | | | |
| 40 | 1.15 | 1.13 | 1.11 | 1.09 | 1.08 | 1.06 | 1.06 | 1.05 |
| 50 | 1.09 | 1.08 | 1.07 | 1.06 | 1.05 | 1.04 | 1.04 | 1.03 |
| 60 | 1.04 | 1.04 | 1.04 | 1.03 | 1.03 | 1.02 | 1.02 | 1.02 |
| 70 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 |
| 80 | 0.96 | 0.96 | 0.97 | 0.97 | 0.98 | 0.98 | 0.98 | 0.99 |
| 90 | 0.93 | 0.94 | 0.94 | 0.95 | 0.95 | 0.96 | 0.97 | 0.97 |
| 100 | 0.90 | 0.91 | 0.92 | 0.93 | 0.93 | 0.95 | 0.95 | 0.96 |




| Inlet Proportions | |
|---|---|
| Pipe Conduit (D) - in | Pipe Riser (d) - in |
| 8-12 | 18 |
| 15 | 21 |
| 18 | 24 |
| 21 | 30 |
| 24 | 30 |
| 30 | 36 |
| 36 | 48 |
| 42 | 54 |
| 48 | 60 |

Pipe drop inlet spillway design:

For a given Q and H, refer to Table 8.07a or 8.07b for conduit size. Then determine the riser diameter (d) from the Inlet Proportions Table on this figure. Next, refer to the above curves, using the conduit capacity and riser diameter, and find the head (h) required above the crest of the riser. The height of the riser should not be less than 5D - h, except as noted in the above sketch.

Example - Given: CMP; Q = 20 cfs; H = 14 ft, h max. 1.0 ft; L = 70 ft. From Table 8.07a find conduit size (D) = 18 inches. From Inlet Proportions Table, riser size = 24 inches. Head (h) required for Q = 20 and d = 24 is 1.0 ft.

**Figure 8.07b** Design chart for riser outlet.