Carmen Johnson

Fac/Perm/Co ID#	Date	Doc ID#
85-05	5/11/12	DIN

APPROVED
DIVISION OF WASTE MANAGEMENT
SOLID WASTE SECTION
DATE 6/29/2006 BY TAO
PERMIT # 85-05
PTC ATTACH 1, DOC #12

CONSTRUCTION PLAN APPLICATION
FGD RESIDUE LANDFILL

RALEIGH
CENTRAL FILE COPY

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

Sells Project Number: 046119

Revision 1

April 19, 2006

Received
APR 2006
Solid Waste Section

Prepared By:

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

Document BCM6451.00-0000.001
ERN No. FHS00AO8

Table of Contents

CONSTRUCTION PLAN APPLICATION

Revision 1

TABLE OF CONTENTS

I. CONSTRUCTION PLAN APPLICATION REPORT

II. TECHNICAL SPECIFICATIONS

III. CONSTRUCTION QUALITY ASSURANCE PROGRAM

IV. OPERATIONS PLAN

APPENDICES

A CALCULATIONS

B DRAWINGS

Belews Creek FGD Landfill
Construction Plan Application

List of Drawings – Revision 1

Description of Drawing	Drawing Number	Drawing Revision	Duke Document Number
Existing Conditions Plan	CP-1	1	BCM 6451.00-0007.001
Top of Liner Grading Plan	CP-2	1	BCM 6451.00-0008.001
Leachate Collection System	CP-3	1	BCM 6451.00-0009.001
Cell Operations - Leachate Collection and Removal System	CP-3.1	0	BCM 6451.00-0009.002
Operational Cover Grading Plan	CP-4	1	BCM 6451.00-0010.001
Cell Details	CP-5	1	BCM 6451.00-0011.001
Stormwater Basin Plan and Details	CP-6	1	BCM 6451.00-0012.001
Stormwater Basin Leak Detection System	CP-6.1	0	BCM 6451.00-0012.002
Pump Station Details	CP-7	1	BCM 6451.00-0013.001
Final Closure Plan and Details	CP-8	1	BCM 6451.00-0014.001
Final Closure Gas Venting System Plan	CP-9	1	BCM 6451.00-0015.001
FGD Erosion and Sediment Control Plan FGD Residue Landfill	CP-10	1	BCM 6451.00-0016.001
FGD Project Erosion and Sediment Control Plan Stockpile Areas	CP-11	1	BCM 6451.00-0017.001
FGD Project Erosion and Sediment Control Plan Details	CP-12	1	BCM 6451.00-0018.001
Profile A-A' & B-B'	CP-13	1	BCM 6451.00-0019.001
Profile C-C' & D-D'	CP-14	1	BCM 6451.00-0020.001

1

CONSTRUCTION PLAN APPLICATION
FGD RESIDUE LANDFILL

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

Revision 1

Prepared by:

CHAS. H. SELLS, INC.

Consulting Engineers, Surveyors & Photogrammetrists

128 Overhill Drive, Suite 105, Mooresville, NC 28117
Phone: (704) 662-0100
Fax: (704) 662-0101

April 19, 2006

Document BCM6451.00-0000.001
ERN No. FHS00AO8

NORTH CAROLINA PROFESSIONAL SEAL 17562 ENGINEER DAN E. BREWER

4/19/06

Table of Contents

1.0 INTRODUCTION 1
1.1 Project Description 1
1.2 Site Conditions 1

2.0 LANDFILL DESIGN 2
2.1 Site Development 2
2.2 Landfill Operation 3
2.3 Closure Design 3
2.4 Groundwater Considerations 3
2.5 Engineering Considerations of Landfill Components 4

3.0 LEACHATE MANAGEMENT 10
3.1 Leachate Generation 10
3.2 Leachate Storage 10

4.0 ENVIRONMENTAL CONTROL AND MANAGEMENT 11
4.1 Gas Migration 11
4.2 Nuisance and Vectors 11
4.3 Wetlands 11

5.0 WASTE STREAM 12
5.1 Types of Waste 12
5.2 Disposal Rates 12
5.3 Service Area 12
5.4 Equipment Requirements 12

6.0 LANDFILL CAPACITY AND BORROW SOIL 13
6.1 Design Parameters 13
6.2 Operating Capacity 13
6.3 Borrow Soil Quantities 14

Revision 1

Description of Revisions to the Construction Plan Application from Revision 1:

Addition of GCL in Landfill Footprint
After submittal of the initial Construction Plan Application, Duke decided to add a geosynthetic clay liner (GCL) underneath the 60 mil HDPE geomembrane that covers the bottom of the landfill.

Shift in Landfill Footprint
The footprint of the landfill was modified by shifting the northern side southward approximately 40 feet. This move provides the clearance required between the adjacent transmission lines and the landfill. This change required a change in the upper portion of the final cap to regain volume (increase in elevation).

Stormwater Basin
Based on discussions between DENR and Duke, the design of the Stormwater Basin was modified to incorporate a secondary liner system and a liquid collection sump, equipped with a pump. To accommodate the required 4 foot separation between the primary liner for the basin and estimated seasonal high groundwater, the low end of the pond was relocated.

Changes to the previous document are noted by a revision line in the right margin.

Revisions/changes to formatting in parts of the document were also incorporated into this revision. These changes are not marked.

Marked revisions to the original document are listed below.

Section	Revision
1.1	Revised area of landfill footprint from 24 ac. to 22.6 ac. Revised volume from 1.6 million cy to 1.5 million cy.
1.2	Revised area of landfill footprint from 24 ac. to 22.6 ac.
2.1	Added wording on GCL.
2.5.2	Added section describing GCL.
2.5.4	Revised wording to describing ability of active leachate sumps to accommodate 25 year, 24 hour storm.
3.1	Added description of secondary liner and leak detection system for Stormwater Basin. Added footnote concerning change in area for footprint.
6.2	Revised volume from 1.6 million cy to 1.5 million cy. Revised operating life discussion.

1.0 INTRODUCTION

1.1 Project Description

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

The FGD gypsum landfill will be located on Duke Power property, south of the Belews Creek plant. The Phase 1 landfill footprint contains approximately 22.6 acres. The FGD residue will be conveyed to the landfill site by conveyor and truck, where the material will be spread and compacted. The landfill will begin receiving FGD residue in early 2007. The volumetric capacity of the landfill is 1.5 million cubic yards.

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

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

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

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

1.2 Site Conditions

The Belews Creek Steam Station is located in the Piedmont physiographic region. The footprint for the landfill contains an area of approximately 22.6 acres.

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

footprint. Surface drainage to the east of the topographic divide also drains to Belews Lake.

2.0 LANDFILL DESIGN

2.1 Site Development

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

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

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

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

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

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

Existing access roads will be utilized for access to the landfill. Temporary access roads will be constructed throughout the operation of the landfill. The roads are a

minimum of 15 feet wide with side ditches to prevent erosion of the roadway surface.

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

2.2 Landfill Operation

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

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

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

2.3 Closure Design

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

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

2.4 Groundwater Considerations

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

2.5 Engineering Considerations of Landfill Components

2.5.1 Landfill Subgrade

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

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

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

2.5.2 Geosynthetic Clay Liner

A geosynthetic clay liner (GCL) will be installed 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.

2.5.3 Geomembrane

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

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

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

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

2.5.4 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 will consist of a geonet/geotextile with perforated collector/header pipe that drain to the sump area. The slope on the geomembrane will be a minimum of 1.2 percent.

The geonet is a polyethelene synthetic mesh with a transmissivity equal to or greater than 1 X 10-4 m2/sec. The geonet will transmit leachate to the leachate collection pipe.

The leachate collection/header pipe in each cell meets the performance criteria of providing a maximum 1 foot of head of leachate on the liner system under normal operating conditions. The pipe spacing is based on results from the Mound model that incorporates the leachate infiltration rates as well as the maximum allowable head. A maximum pipe spacing of 75 feet meets the performance criteria. The

Mound model does not consider the effects of the underlying geocomposite which adds to the reduction of head build up on the geomembrane.

The active leachate sump risers and culvert barrels are designed to accommodate the peak flows from the stormwater generated from a 25-year, 24 hour storm event, while containing the stormwater within the active cell. The elevation of the riser crest provides 1 foot of freeboard. The perforations in the riser pipe allow the stormwater accumulating in the cell below the crest of the riser to drain in less than 24 hours.

The landfill perimeter berm has been designed to contain run-off from the 25 year-24 hour storm event. As shown in the calculations, Appendix A, each cell divider berm has also been designed to contain the 25 year-24 hour storm event. The sump is located at the downgradient end of each cell. This sump will be a 1 inch thick pre-fabricated HDPE sump that is to be extrusion welded to the 60 mil HDPE geomembrane. The leachate will gravity flow from the sump location through a solid HDPE pipe to the stormwater basin.

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

2.5.6 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.

2.5.7 Temporary Side Slope Liner and Ballast System

Due to the steepness of the excavated side slopes and potential long-term exposure of the side slope liner system, a temporary side slope liner with ballast

system is required. The temporary side slope liner and ballast system will protect the geocomposite and geomembrane against potential damage and ultraviolet degradation. The temporary side slope liner will be a minimum 20-mil thick LLDPE geomembrane. The temporary ballast system will consist of 40 pound sand bags spaced at 10 feet on center. The ballast system is designed to offset minor wind uplift pressures.

2.5.8 Surface Water Collection and Removal (SWCR)

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

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

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

2.5.9 Geomembrane Cap

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

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

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

2.5.10 Gas Venting System

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

mixed with the gypsum. Therefore, the gypsum will not have sufficient organic material for an anaerobic bacteria to produce a surmountable quantity of H_2S gas.

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

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

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

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

2.5.11 Final Cover Soil

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

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

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

soil stability. The edge drains will be routed to the final closure stormwater management system.

2.5.12 Vegetative Cover

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

The design objectives in evaluating the vegetative cover include:

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

3.0 LEACHATE MANAGEMENT

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

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

3.1 Leachate Generation

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

3.2 Leachate Storage

The stormwater basin is designed to contain stormwater runoff from two active landfill cells. The stormwater basin pump to the wastewater treatment system is sized for 500 gallons per minute. A 25-year storm event will drain the stormwater basin within approximately 9 days.

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

The stormwater basin is designed with a primary liner and a secondary liner. Both of these liners are 40 mil textured HDPE geomembrane. To detect a potential leak in the primary liner system, the secondary liner is constructed with a sump.

---

[1] The 24 acre footprint for Phase 1 was the area prior to Revision 1. Revision 1 reduced the size of Cell1 to provide adequate clearance between the transmission lines and the landfill. The acreage for the revised footprint is 22.6 acres.

Inside this sump is a 6 gallon per minute pump and level sensor. The discharge from this pump is routed to the stormwater basin pump sump. The control panel for the sump pump is equipped with a high level alarm (flashing red light), and a run-time pump meter. The discharge line is equipped with a magnetic flow measurement device whose readings are fed into a flow totalizer located on the control panel.

4.0 ENVIRONMENTAL CONTROL AND MANAGEMENT

4.1 Gas Migration

Methane or H2S gas is not anticipated to be generated in the landfill. During landfill operations, gas testing from the leachate header cleanout piping and sump discharge pipe inverts in the stormwater basin will be performed in order to identify if gas generation is a problem and if further testing or monitoring is deemed necessary. Furthermore, a passive gas collection system is included for closure to reduce the likelihood of gas migration.

4.2 Nuisance and Vectors

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

4.3 Wetlands

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

5.0 WASTE STREAM

5.1 Types of Waste

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

Gypsum	93% to 95%
Sulfite	0.35%
CO3	1.3%
CaF2	0.2%
Inerts	2.5% to 3.5%
Fly Ash Content	0.5% to 0.8%
pH	6.0 to 8.3
Unit Weight	76 lb/ft3 to 97 lb/ft3
Specific Gravity	2.35
Moisture	10% to 12%

5.2 Disposal Rates

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

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

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

5.3 Service Area

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

5.4 Equipment Requirements

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

6.0 LANDFILL CAPACITY AND BORROW SOIL

6.1 Design Parameters

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

6.2 Operating Capacity

The gross capacity for Phase 1 of the proposed landfill will be approximately 1.5 million cubic yards. This is the volume of the space above the upper surface of the operational cover (1' of soil above the geomembrane) to the uppermost surface of the waste.

Duke assumes an annual production rate of 350,000 tons of gypsum and 50,000 tons of clarifier sludge, creating a total annual production of 375,500 cy of material that would be placed in the landfill. This is calculated using an in-place density for gypsum of 89 pcf and an in-place density of 44 pcf for clarifier sludge.

<u>Annual Gypsum Production</u>:
(350,000 tons/yr) x (2000lb/ton) x (ft^3/89 lb) x (yd^3/27 ft^3) = 291,300 yd^3

<u>Annual Clarifier Sludge Production:</u>
(50,000 tons/yr) x (2000lb/ton) x (ft^3/44 lb) x (yd^3/27 ft^3) = 84,200 yd^3

<u>Total Annual Volume Needed:</u>
291,000 yd^3 + 84,200 yd^3 = 375,500 yd^3

$$\text{Operating Life} = \frac{\text{Landfill Volume}}{\text{Annual Volume Used}} = \frac{1{,}500{,}000\ yd^3}{375{,}500\ yd^3/yr}$$

Operating Life = 4 years

Based on this annual production rate, the 1.5 million cy of capacity will provide an operating life of approximately 4 years for Phase 1. Reduction in the volume of gypsum placed in the landfill through beneficial uses would increase the life.

6.3 Borrow Soil Quantities

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

2

TECHNICAL SPECIFICATIONS

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

Revision 1

Prepared by:

CHAS. H. SELLS, INC.

Consulting Engineers, Surveyors & Photogrammetrists

128 Overhill Drive, Suite 105, Mooresville, NC 28117
Phone: (704) 662-0100
Fax: (704) 662-0101

Revised April 19, 2006

Document BCM6451.00-0000.001
ERN No. FHS00AO8

NORTH CAROLINA PROFESSIONAL SEAL 17582 ENGINEER DAN E. BREWER

4/19/06

1.0 INTRODUCTION

1.1 Scope of Work

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

1.2 Definitions

The following list of definitions is provided for reference.

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

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

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

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

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

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

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

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

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

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

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

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

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

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

2.0 EARTHWORK

2.1 Site Preparation

2.1.1 Scope of Work

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

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

2.1.2 Specification

2.1.2.1 Stripping

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

2.1.2.2 Undercutting

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

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

2.1.2.3 Stormwater Management Structures and Erosion/Sediment Control

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

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

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

Culverts shall be reinforced concrete[1] 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.

---

[1] Requirements for culverts identified on the contract drawings as HDPE are found in Section 7.0.

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

2.1.3 Measurement and Payment

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

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

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

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

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

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

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

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

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

2.2 Structural Fill

2.2.1 Scope of Work

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

2.2.2 Materials

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

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

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

2.2.3 Construction

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

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

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

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

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

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

2.2.4 Measurement and Payment

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

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

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

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

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

2.3 Compacted Soil Layer

2.3.1 Scope of Work

Phase 1 shall include a smooth soil layer for placement of the geosynthetic clay liner (GCL). 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 GCL placement, the prepared subgrade shall be proofrolled with a loaded dump truck (minimum 20 tons) or similar rubber tired pneumatic vehicle, a minimum of two passes in each direction. Proofrolling shall be conducted at the discretion of the ENGINEER or his representative. Any soft, saturated or yielding areas exhibited by pumping and/or rutting will require removal and replacement with the appropriate compacted soil as specified in the Site Preparation and Earthwork sections of these specifications.

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

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

2.3.4.4 Compaction of lifts shall be as follows:

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

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

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

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

Dozer equipment shall not be used for primary compaction efforts.

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

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

Compacted moisture content shall be between 2 and 5 percent wet of optimum moisture content.

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 Belews Creek 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 under the GCL shall be performed in accordance with Section 2.3.

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 GCL shall be installed 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 Leachate Sumps have been achieved. The GCL is to cover the bottom of the cells 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 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.

3.3.4 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.

3.3.5 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.

3.3.6 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)

3.3.7 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.

3.3.8 Once the daily panels are placed and smooth, commence placement of the geomembrane in accordance with Section 4.0. If exposed GCL cannot be covered by geomembrane

before the end of the working day, the panels or sections of panels shall be temporarily covered with plastic and ballasted until geomembrane placement can commence.

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.

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 Belews Creek FGD Gypsum Landfill, Phase 1 including, but not limited to, anchor trench excavation and backfill, liner layout, seaming, patching, and all necessary and incidental items required to complete the work, in accordance with the Contract Drawings and these Specifications.

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 1. Textured 60 mil geomembrane liner shall be used on side slopes and smooth 60 mil geomembrane liner shall be used on 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 and installation of the GCL. 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 4.3.10).

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.

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

4.3.4 Field seaming shall be in accordance with EPA Technical Guidance document: "The Fabrication of Polyethylene FML Field Seams" EPA/530/SW-89/069 or as follows. Where

conflicts exist between the two guidance specifications, the most stringent specification prevails unless otherwise directed by the ENGINEER.

All foreign matter (dirt, water, oil, etc.) shall be removed from the edges to be bonded. For extrusion welds, the bonding surfaces must be thoroughly cleaned by mechanical abrasion or alternate methods approved by the ENGINEER to remove surface cure and prepare the surfaces for bonding. All abrasive buffing shall be performed using No. 80 grit or finer sandpaper. The grinding shall be performed so that any and all grind marks are perpendicular to the edge of sheet. No grinding greater than 1/4 inch outside the welds is permitted or the ENGINEER can require patching. No solvents shall be used to clean the geomembrane liner.

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

The completed liner shall not exhibit <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.

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

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

4.3.5 Field seaming may be fusion or extrusion welding or a combination of these methods. Solvent welding is not acceptable. The ENGINEER reserves the right

to reject any proposed seaming method it believes unacceptable. Double hot wedge fusion welding shall be the predominant seaming method. Additional concepts and requirements of proper field seaming include the following:

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

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

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

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

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

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

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

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

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

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

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

4.3.6 Quality Control

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

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

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

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

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

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

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

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

4.3.6.4 Nondestructive Testing

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

Double Weld Seams - the GEOSYNTHETIC CONTRACTOR shall maintain and use equipment and personnel to perform air pressure testing under the observation of the ENGINEER of all double weld seams greater than 20 ft. (double weld seams less than 20 ft. may be vacuum box tested as specified above). The system shall be capable of applying a pressure of at least 30 psi for not less than 5 minutes. After the 5 minute testing period, the end of the seam shall be cut open and pressure loss monitored to verify the entire length of the seam channel is open. If no pressure loss is realized, the location of the blocked channel must be found and the remainder of the seam tested separately. The air pressure test results shall be documented on the Seam Inspection Quality Control Form enclosed in the "Supplement to the Technical Specification". Pressure loss tests shall be conducted in accordance with the procedures outlined in "Pressurized Air Channel Test for Dual Seamed Geomembranes," Geosynthetic Research Institute Test Method GM-6. As outlined by the test method, following a 2 minute pressurized stabilization period, pressure losses over a measurement period of 5 minutes shall not exceed 3 psi.

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

4.3.6.5 Destructive Testing

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

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

laboratory testing at the option of the ENGINEER OR GEOSYNTHETIC CONTRACTOR.

Before the sample is sent to the laboratory, two specimens, one from each end of the specimen will be tested in peel in a calibrated field tensiometer, supplied by the GEOSYNTHETIC CONTRACTOR. Both specimens must meet the qualitative and quantitative criteria listed below. Testing requirements are as follows: Each sample shall be large enough to test five specimens in peel and five specimens in shear. The average values of each set of five specimens must meet the specification, and four of the five specimen tests must meet the specifications for the seam to be considered a passing seam. If the average of the five specimens is adequate, but one of the specimens is failing, values for the failing specimen must be at least 80 percent of the values required for the seam for the sample to pass. All samples must fail in film tear bond (FTB). Tests shall be conducted using a calibrated tensiometer and must meet the criteria outlined below. Samples which do not pass the shear and peel tests will be re-sampled from locations at least 10 feet on each side of the original location. These two re-test samples must pass both shear and peel testing. If these two samples do not pass, then additional samples will continue to be obtained until the questionable seam area is defined. If the length of the questionable seam area is defined to be excessive by the ENGINEER, a cap pad may be required over the entire seam with nondestructive testing prior to acceptance of the seam.

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

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

Each sample area will be clearly marked both on the liner itself (LDT or FDT) using the procedures outlined in the marking section and on the "as- built" record drawing prepared by the GEOSYNTHETIC CONTRACTOR. Destructive Test Forms will be retested and completed by the ENGINEER.

All areas cut out for testing should be immediately patched by the GEOSYNTHETIC CONTRACTOR and the patches should be tested and

approved by the ENGINEER. Patches shall extend a minimum of 6 inches beyond the cut area.

A passing double welded seam will be achieved in peel (ASTM D413 modified) when (1) failure is by Film Tear Bond (FTB), National Sanitation Foundation (NSF), Standard 54, definition 2.16, and (2) yield strength for the seam is not less than 62 percent of the minimum tensile strength at yield as specified in Table 1, and (3) no greater than 10 percent of the seam width peels (separates) at any point and (4) the failed sheet exhibits elongation, prior to failure. Both sides of the double welded seam must be tested and must meet all of the criteria listed above. A passing double welded seam will be achieved in shear (ASTM D3083 modified) when (1) failure is by FTB, and (2) yield strength for the seam is not less than 95 percent of the minimum tensile strength at yield specified in Table 1, and (3) yield strain for the seam is at least 10 percent and (4) break strain for the seam is at least 50 percent.

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

4.3.7 Repair of Damaged and Sampled Areas

Damaged and sample coupon areas of geomembrane shall be repaired by the GEOSYNTHETIC CONTRACTOR by construction of an extrusion welded patch. Patches shall extend a minimum of 6 inches beyond the damaged or cut area. No repairs shall be made to seams by application of an extrusion bead to a seam edge previously welded by fusion or extrusion methods. Repaired areas will be tested for seam integrity as outlined in Section 4.3.6.4. 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.

4.3.8 Seams at Panel Corners

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

4.3.9 Potentially Damaging Activities

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

4.3.10 Anchor Trench Backfilling

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

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

4.3.11 Protection of Leading Edges

Between construction of partial sections of the geomembrane liner, leading edges of the geomembrane may be exposed or buried for extended periods of time prior to their joining to adjacent, subsequent membrane sections. The combined action of abrasive soil and equipment impact stresses may "etch" unprotected membrane surfaces sufficiently to affect seam strengths. Therefore, it is necessary to protect leading edges in high activity areas with sacrificial layers of geotextile and HDPE sheet until they are ready for final seaming. As a minimum, each leading edge to be seamed that must be buried or which must be exposed for periods of one month or longer shall be continuously covered by a layer of HDPE sheet. The geotextile

shall be nonwoven and have a minimum weight of 8 oz per square yard. The sacrificial HDPE sheet shall have a minimum thickness equal to that of the geomembrane liner to be protected. Both protective layers shall have a minimum width of 2 feet. The protective cover sheets shall be either covered with soil or weighted with sand bags to prevent displacement by wind. The edge of the sheet to be protected shall be approximately centered beneath the overlying protective layers prior to burial or weighing with sandbags. Leading edges located in areas expected to receive direct traffic from construction equipment shall be buried under a minimum thickness of one foot of buffer soil.

4.3.12 Warranty

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

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

If it is determined by the procedures described herein that the damage is the fault of the GEOSYNTHETIC CONTRACTOR, the GEOSYNTHETIC CONTRACTOR will, at no cost to the OWNER, expose the damaged area as needed to replace and/or repair the damaged material at the discretion of the

ENGINEER, perform the necessary quality control testing and rebackfill or restore the area as required.

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

4.3.13 Marking on Liner by the Geosynthetic Contractor

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

Panel Identification

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

Seam Identification

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

Quality Control Marking

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

Similarly, after each quality control test, the CQAO or a representative will record the following immediately adjacent to the area tested: initials of QC TECHNICIAN performing the test, date of the test, type of test (i.e. VB, SP, AP for vacuum box, spark test and air pressure test respectively) and the words "pass" or "fail". For the air pressure test, the QC TECHNICIAN must also define the limits or zone of the test as well as the amount of pressure loss observed. Again, a permanent white or red marking pen is required. If the test fails and the necessary repair is made, the technician will cross out the previous markings and mark appropriately for the new test results.

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

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

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

4.3.14 Documentation

Forms by the GEOSYNTHETIC CONTRACTOR:

- Subgrade Certification (signed daily by the GEOSYNTHETIC CONTRACTOR'S Superintendent and submitted to the CQAO)

- Seam Test Weld Form (completed after each seam test by the Welding Technician for approval by the CQAO.)

- Pre-construction shop drawings and post construction as built record drawings.

Forms by the CQAO:

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

4.3.15 Daily Meetings

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

4.3.16 GEOSYNTHETIC CONTRACTOR Equipment and Personnel

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

During the installation of the 60 mil geomembrane liner for the landfill, the GEOSYNTHETIC CONTRACTOR shall supply and maintain at least three extrusion welders and three double hot wedge fusion welders, at least one of

which must be available at the working space at all times. At least one extra generator shall be supplied and maintained by the GEOSYNTHETIC CONTRACTOR to be used as a spare.

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

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 1

REQUIRED PHYSICAL PROPERTIES OF MEMBRANE LINER (HDPE) SHEET

Property	Test Method	Required Values 60 Mil HDPE
Thickness	ASTM D-751	57/60 Mil*
Sheet Density	ASTM D-1505	0.940 g/cm^3 min.
% Elongation at Yield	ASTM D-638	12
% Elongation at Break	ASTM D-638	560
Tensile Stress at Yield	Test Specimen Type IV	130 lb/in. min.
Tensile Strength at Break	ASTM D-638	228 lb/in. min.
Carbon Black Content	ASTM D-1603	2% min-3% max
Carbon Black Dispersion	ASTM D-3015	A1, A2, or B1
Environmental Stress-Crack Resistance	ASTM D-1693 (As Modified in NSF54 Appendix A)	1500 hours min.
Low Temperature Impact	ASTM D-746	-60°F or lower
Dimensional Stability	ASTM D-1204 (As Modified in NSF54 Appendix A)	± 2.0% max.
Puncture Resistance	FTMS 101B Method 2065	70 lb. min.
Tear Resistance	ASTM D 1004	42 lb. min.
Water Vapor Transmission Rate	ASTM E96 DIE C	0.03 gm/m^2-day
Seam Strengths		
1. Shear Strength	ASTM D-3083 as modified in NSF 54 Appendix A	FTB, 95% of Parent material (min.)
2. Shear Strain @ Yield	ASTM D-3083 as modified in NSF 54 Appendix A	10% (min.)
3. Peel Strength	ASTM D-413 as modified in NSF 54 Appendix A	FTB, 62% of Parent Material (min.)
Non-Destructive Testing		
1. Single Weld	Continuous Vacuum Box;	Maintain Vacuum of 3 psi, hold vacuum for 15 seconds
2. Double Weld	Air Testing	Maintain 30 psi for not less than 5 min.; pressure loss not greater than 3 psi for last 3 minutes

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

TABLE 2

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

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

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

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, temporary cover liner with ballast system, pipe and sump, aggregate with geotextile, and all necessary and incidental items required to complete the work, all in accordance with the Contract Drawings and these Specifications.

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

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

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

The chemical resistance of the PE pipe, manholes, sumps and all fittings shall be in keeping with typical properties of high quality polyethylene products currently available through

commercial sources and equal to or greater than that of the 60 mil HDPE geomembrane specified. All mechanical fasteners or fittings shall be stainless steel.

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

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

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

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

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

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

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

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

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

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

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

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 5.2.3. No bonding is required between the nonwoven geotextile and the geonet.

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

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

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

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

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

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

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

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

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 Temporary Cover Liner and Ballast System

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

5.2.7 Grout for Sump

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

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.

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

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

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 Temporary Cover Liner and Ballast System

Installation shall be performed in accordance with the geomembrane installation procedures in Section 4.0. The leading edge of the temporary cover liner shall be continuously ballasted with sandbags. The sand bags shall be tied and spaced at 10-foot on center and anchored at the top of slope as shown on the Contract Drawings.

5.3.7 Grout for Sump

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

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

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 4.0 of these specifications at no additional costs to the OWNER.

6.0 OPERATIONAL COVER AND INTERIOR BERMS

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 and interior berms for the Belews Creek FGD Gypsum Landfill, Phase 1 including but not limited to excavation, hauling, scraping, stockpiling, placement, maintenance, and removing accumulated rain water during construction. The operational cover and interior berms shall be constructed to the lines and grades shown on the Contract Drawings.

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

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

6.3 Construction

6.3.1 The operational cover and interior berms 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 the geotextile and geomembrane to minimize wrinkles in the underlying liner is to enclose a small area using a trackhoe. With the trackhoe operating from a staging area, the operational cover shall be placed in piles to

enclose a small area away from the working face. The operational cover may then be pushed outward using a small dozer into the enclosed area. Other methods that are demonstrated to be acceptable by the ENGINEER may be used.

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

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

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

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

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

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 Belews Creek FGD Gypsum Landfill, Phase 1 including but not limited to supply and installation. The SWCR system shall be constructed to the lines and grades shown on the Contract Drawings.

7.2 Materials

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

HDPE pipe shall be manufactured with first quality virgin polyethylene as referenced in Section 5.0.

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

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 FINAL CAP (NOT IN CONTRACT)

9.1 Scope of Work

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

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

9.2 Materials

9.2.1 Compacted Soil Layer

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

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

9.2.3 The geocomposite shall consist of a geotextile/geonet/geotextile laminate. The geotextile and geonet shall meet the minimum criteria specified in Section 5.2.

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

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

9.3 Construction

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

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

Compaction of lifts shall be as follows:

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

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

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

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

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

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

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

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

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

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

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

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

Installation of the geomembrane shall be as follows:

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

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

Once panels are in place and smooth, commence field seaming operations. The geomembrane placement, seaming, and testing shall be in accordance with Section 4.3 of these specifications.

9.3.4 The geocomposite drainage layer and toe drains should be constructed concurrently with the geomembrane placement at specified locations. The geocomposite shall be constructed in accordance with Section 5.3 of these Specifications.

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

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

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

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

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

10.0 STORMWATER BASIN

10.1 Scope of Work

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

10.2 Materials

10.2.1 Earthwork shall be in accordance with Section 2.2.

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

The bentonite shall be high quality sodium bentonite comprised of 100 percent montmorillonite. The bentonite shall be continuously adhered to the underlying and overlying geotextile with a water soluble adhesive.

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

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

10.2.3 The stormwater basin secondary geomembrane shall be 40 mil textured HDPE in accordance with Section 4.0. The stormwater basin primary geomembrane shall be 40 mil textured HDPE in accordance with Section 4.0.

10.2.4 The geotextile above the secondary geomembrane on the basin side slopes shall be 12 oz/sy geotextile in accordance with Section 5.0.

10.2.5 The geonet above the secondary geomembrane on the basin floor shall meet the requirements of Section 5.2.4. The geonet shall have an 8 oz. geotextile above the geonet.

The 8 oz. nonwoven geotextile shall conform to Section 5.2.3. No bonding is required between the nonwoven geotextile and the geonet.

10.2.6 The geotextile above the primary geomembrane on the basin floor shall be 12 oz/sy geotextile in accordance with Section 5.0.

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

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

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

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

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

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

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

10.2.10 The HDPE riser structure shall be SDR 21 HDPE pipe and meet the requirements of Section 5.0. The riser structure shall meet the dimensions as specified on the Contract Drawings. Shop drawings for the riser shall be submitted to the ENGINEER for review and approval.

10.2.11 The HDPE discharge piping to the pump station shall meet the requirements of Section 5.0.

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

10.3 Construction

10.3.1 Earthwork for the stormwater basin shall be performed in accordance to Section 2.0 and to the lines and grades shown on the Contract Drawings.

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

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

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

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

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

After panels are initially in place, remove wrinkles as directed by the ENGINEER. Unroll several panels and allow the GCL to relax before beginning adjacent panels. The panels shall be placed on the side slopes first and extend a minimum of 3 feet beyond the toe of

slope. No horizontal seams of GCL will be permitted on the side slopes. The panels in the basin floor shall be placed from the upgradient end of the basin and progress downgradient in order to keep stormwater runoff from accumulating under the GCL.

Once the daily panels are placed and smooth, commence placement of the geomembrane in accordance with Section 4.0. If exposed GCL cannot be covered by geomembrane before the end of the working day, the panels or sections of panels shall be temporarily covered with plastic and ballasted until geomembrane placement can commence.

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

10.3.3 The geomembrane shall be installed as specified in Section 4.0.

10.3.4 The geotextile shall be installed as specified in Section 5.0.

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

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

10.3.5 The HDPE piping shall be installed as specified in Section 5.0.

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

10.4 Measurement and Payment

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

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

10.4.3 Measurement and payment of the geomembrane shall be in accordance with Section 4.0.

10.4.4 Measurement and payment of the geonet and the geotextile shall be in accordance with Section 5.0.

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

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

10.4.7 Measurement and payment for the HDPE piping shall be in accordance with Section 5.0.

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

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

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

11.0 LIFT STATION, VALVE VAULT AND HDPE FORCEMAIN

11.1 Scope of Work

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

11.2 Materials

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

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

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

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

11.2.3.1 Pipe and Fittings

The pipe shall be produced to the nominal dimensions and tolerances specified by ASTM F-714. The pipe shall contain no recycled compound except that generated in the manufacturer's plant from resin of the same specification from the same raw material. The pipe shall be free of visible cracks, holes, voids, or other deleterious defects and shall be homogeneous in color, density, melt index, and other physical properties throughout.

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

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

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

11.2.4 Valves

The CONTRACTOR shall provide and install two 6-inch diameter ductile iron ball check valves and two 6-inch diameter ductile iron plug valves within the valve vault as shown

on the contract drawings. Shop drawings for the proposed valves shall be submitted to the ENGINEER for review and approval. 300 series stainless steel backing rings and bolts shall be installed on the HDPE flange adaptors at each valve connection. Each flanged connection shall have a full face EPDM gasket.

11.2.5 Pumps and Controls

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

11.3 Construction

11.3.1 Concrete Lift Station and Valve Vault Installation

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

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

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

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

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

11.3.2 Forcemain Piping Installation

11.3.2.1 Trenching and Backfilling

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

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

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

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

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

11.3.2.2 Joining

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

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

11.3.2.3 Pressure Testing

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

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

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

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

11.3.2.4 Thrust Blocking

Gradual field sweeps (any alignments less than a 33 foot radius described in Section 11.3.2.1) should not require thrust blocks. Thrust Blocks shall be installed along the forcemain at all abrupt changes in direction (tees, elbows, etc.) both vertical and horizontal, changes in size (reducers, etc.), dead ends (plugs, caps, hydrants, etc.), valves, and slopes or as directed by the ENGINEER. Thrust blocking shall be constructed against the undisturbed trench wall with 3000 psi concrete. Reinforcement rods shall be installed as required. Rods shall be ASTM A-242 or A-36 steel, No. 4 rods. The ENGINEER shall approve all locations and reinforcement details for thrust blocks.

11.3.3 Lift Station Piping

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

11.3.3.2 All lift station HDPE piping shall be joined by the butt fusion welding method as described in Section 11.3.2.2 of these specifications. The 6-inch diameter discharge pipes shall be connected to the pump discharge fittings by means of a flanged connector utilizing a 300 series stainless steel backing ring arrangement with stainless steel bolts to provide a minimum 150 psi connection. Each flange connection shall have a full face EPDM gasket.

11.3.4 Pump and Control Panel Installation

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

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

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

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

11.4 Measurement and Payment

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

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

11.4.3 Payment for HDPE piping shall be based on the unit price bid per linear foot installed, including trenching, pipe installation, pressure testing, trench bedding and backfilling, and all connections within the valve vault and manhole. Pay quantities will not include wastage or extension of material beyond required limits.

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

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

3

CONSTRUCTION QUALITY ASSURANCE PROGRAM

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

Revision 1

Prepared By:

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

April 19, 2006

Document Number BCM6451.00-0000.001
ERN FHS00AO8

NORTH CAROLINA PROFESSIONAL SEAL 17582 ENGINEER DAN E. BREWER

4/19/06

CONSTRUCTION QUALITY ASSURANCE PROGRAM

TABLE OF CONTENTS

1.0 PROJECT TEAM AND RESPONSIBILITIES 1

2.0 STRUCTURAL FILL 3

2.1 Material 3

2.2 Construction 3

3.0 COMPACTED SOIL LAYER 5

3.1 Material 5

3.2 Construction 5

4.0 GEOSYNTHETIC CLAY LINER 7

4.1 Material 7

4.2 Geosynthetic Clay Liner Manufacturer and Contractor 7

4.3 GCL Installation 8

5.0 GEOMEMBRANE 10

5.1 Material 10

5.2 Geosynthetic Manufacturer and Contractor 10

5.3 Geomembrane Installation 10

5.4 Test Seams 12

5.5 Field Destructive Samples 13

5.6 Non-Destructive Testing 13

5.7 Laboratory Destructive Testing 13

6.0 LEACHATE COLLECTION AND REMOVAL SYSTEM (LCRS) 15

6.1 Geocomposite 15

6.2 Leachate Collection Pipe and Sump 17

6.3 Operational Cover 17

7.0 FINAL COVER SYSTEM 18

7.1 Compacted Soil Layer 18

7.2 Geomembrane Cap 18

7.3 Final Cover Soil 18

7.4 Drainage System 18

7.5 Gas Venting System 19

8.0 AS-BUILT CONDITIONS 20

9.0 CQA SUBMITTAL 21

10.0 REFERENCED LIST OF STANDARDS 22

Revision 1

This document was revised to reflect the addition of the geosynthetic clay liner (GCL) as a component of the landfill design. The CQA requirements for the GCL are found in Section 4.0.

There were no revisions to this document other than the addition of the section describing the GCL.

Text revised by this revision is indicated by a vertical bar in the right margin.

CONSTRUCTION QUALITY ASSURANCE (CQA) PROGRAM

1.0 PROJECT TEAM AND RESPONSIBILITIES

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

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

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

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

- Monitor the quality assurance activities of the field testing and assure conformance with test procedures and the technical specifications.
- Inform the Project Engineer of non-conformance to the approved CQA program.
- Logging in of geosynthetic samples and establishing laboratory conformance testing lots.
- Insure that sampling is conducted in a manner consistent with ASTM methods.

- Insure that sample handling procedures are in accordance with the appropriate guidelines for the testing to be conducted.

- Approve all field and laboratory test data before the data are reported or entered into the data base for analysis.

- Assign and direct field technicians.

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

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

2.0 STRUCTURAL FILL

2.1 Material

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

2.1.2 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 GCL 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 Material

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

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

5.2 Geosynthetic Manufacturer and Contractor

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

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

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

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

- Qualifications and resumes of the GEOSYNTHETIC CONTRACTOR Superintendent and Foreman

- Four sets of proposed geosynthetic field panel layout drawings

5.3 Geomembrane Installation

5.3.1 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.

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

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

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

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

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

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

5.3.7 Overheating of the geomembrane will be monitored by the ENGINEER or his representative. Coupons will be cut from the end of the extrusion seams and the bottom side of the seam will be observed for visible warping or deformation.

5.3.8 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.

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

5.3.10 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.

5.3.11 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.

5.4 Test Seams

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

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

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

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

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

5.5 Field Destructive Samples

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

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

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

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 Laboratory Destructive Testing

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

5.7.2 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.

5.7.3 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).

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

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

5.7.6 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.

5.7.7 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.

5.7.8 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.

6.0 LEACHATE COLLECTION AND REMOVAL SYSTEM (LCRS)

6.1 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. Within the operational cover 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 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:

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

6.1.4 The transmissivity test must confirm that the Geonet exhibits a hydraulic transmissivity of at least 1×10^{-4} m^2/sec.

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

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

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

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

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

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

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.

6.3 Operational Cover

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

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 3.0) 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. Geomembrane Liner
3. Leachate Collection and Removal System
4. Operational Cover and Intermediate Berms
5. Final Cover System (ie: compacted soil layer, geomembrane cap, and final soil cover after closure)*

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

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

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

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

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	ndex Puncture Resistance of Geotextiles, Geomembranes, and Related Products
ASTM D5084	Standard Test Method for Measurement of Hydraulic Conductivity of Saturated Porous Materials Using a Flexible Wall Permeameter.
EPA/530/SW-89/069	The Fabrication of Polyethylene FML Field Seams

4

OPERATIONS PLAN

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

Revision 1

Prepared by:

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

April 19, 2006

Document BCM6451.00-0000.001
ERN No. FHS00AO8

NORTH CAROLINA PROFESSIONAL SEAL 17582 ENGINEER DAN E. BREWER

9/19/06

TABLE OF CONTENTS

1.0 LANDFILL OPERATIONS AND MAINTENANCE 1
1.1 Landfilling Sequence 1
1.2 Stormwater Collection and Removal System 1
1.3 Leachate Collection and Removal System 2
1.4 Stormwater Basin – Leak Detection System 3
1.5 Stormwater Basin – Leak Detection System – Action Leakage Rate Response Plan 3
1.6 Placement of Waste and Cover Material 4
1.7 Waste Acceptance and Disposal Requirements 5
1.8 Hours of Operation 5
1.9 Training of Facility Personnel 5
1.10 Operations Record Keeping 6

2.0 SITE OPERATIONS AND MAINTENANCE 6
2.1 Access and Security Requirements 6
2.2 Erosion/Sediment Control Maintenance 6
2.3 Stormwater Structures 7
2.4 Dust, Litter, Odors and Vectors 7
2.5 Fire Control 7
2.6 Groundwater Monitoring Wells 7
2.7 Gas Monitoring 8

Revision 1

Description of Revision 1:

Revision 1 to the Operations Plan was performed to reflect changes in the operations of the landfill. In general, these changes pertain to changes in the operations of the stormwater piping and leachate collection system piping as the landfill operations is completed in Cells 1, 2, and 3. The operation of the leak detection system for the Stormwater Basin is also presented.

Changes to the previous document are noted by a revision line in the right margin.

Revisions/changes to formatting in parts of the document were also incorporated into this revision. These changes are not marked.

Marked revisions to the original document are listed below.

Section	Revision
1.0	Revised area of landfill footprint from 24 ac. to 22.6 ac.
1.2	Revised to describe process for abandonment of stormwater piping during operations.
1.3	Revised to describe operations of Leachate Collection and Removal System cell flange assembly. Revised wording describing conversion of leachate sump.
1.4	Added section describing Leak Detection System for Stormwater Basin.
1.5	Added section describing Leak Detection System Action Leakage Rate Response Plan.

1.0 LANDFILL OPERATIONS AND MAINTENANCE

The Belews Creek Steam Station Flue Gas Desulfurization (FGD) system will create a residue generally comprising of gypsum. Residue will be landfilled in the proposed FGD Gypsum Landfill located just south of the Belews Creek Steam Station Plant. The landfill footprint encompasses approximately 22.6 acres. Phase 1 will be developed in four cells with a capacity of three to five years (depending upon the amount of gypsum reused). The gypsum landfill will be owned and operated by Duke Energy Corp.

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

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

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

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

1.1 Landfilling Sequence

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

1.2 Stormwater Collection and Removal System

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

Surface water falling on an inactive cell will be routed through the cell divider berms and a series of pipes into Sediment Basin SB-5, located to the south-west of the landfill.

The surface water falling on an active cell will either:

1) by overland flow, collect at the lower end of the cell and discharge through the Active Leachate Sump into the lined Stormwater Basin. The stormwater/leachate is then pumped to the plant wastewater treatment system.
2) infiltrate through the operational cover and flow into the leachate collection and removal system. This drains into the Active Leachate Sump, located at the low point of the cell, which in turn drains to the lined Stormwater Basin.

Prior to placement of waste within a cell, the abandonment of the stormwater piping and the activation of the leachate collection and removal system piping described on sheets OP-1, OP-2, OP-3, OP-4, and OP-5 need to be performed.

Landfilling will progress from upgradient to downgradient; therefore, surface water will drain downgradient of the active face and flow into the leachate collection system. Landfilled areas upgradient of the active face will be graded so that surface water drains around the active face.

1.3 Leachate Collection and Removal System

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

The general operation required to activate a cell for waste placement will include the activation of the leachate collection and removal system within the active cell. This is performed as directed on sheets OP-1, OP-2, OP-3, OP-4, and OP-5. In general, this requires the blind flange located in the cell flange assembly to be replaced with a fabricated flange, allowing leachate flow from the sump into the header piping in the active cell.

The temporary cover liner on the side slopes shall be removed within the cell to be activated. The exposed edge of the temporary cover liner shall have continuous sandbags along the edge to prevent wind from blowing beneath the temporary liner.

Prior to landfilling over the sump area, the riser pipe at the sump shall be modified to the configuration shown for Detail 8A-5 INACTIVE LEACHATE SUMP, as shown on the Cell Details, Sheet CP-5.

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

1.4 Stormwater Basin – Leak Detection System

During the operational phase of the landfill, the Stormwater Basin, located on the west side of the landfill, will collect stormwater that has contacted waste and leachate from cells that contain waste. After operation ceases within a cell, only leachate will drain to the lined stormwater basin. The liquid collected in this lined basin gravity drains to a pump station, where it is pumped to the station wastewater treatment system.

In addition to the primary geomembrane basin liner, the lined stormwater basin is equipped with a Leak Detection System, consisting of a secondary geomembrane liner and drainage system that drains to a sump, located on the south end of the basin. The purpose of this system is to detect and to collect liquids that may leak through the primary liner and to pump the liquid to the pump station.

Located inside the 14 inch diameter side slope riser pipe is an electric powered pump and level sensor. The discharge from the pump is piped to the pump station sump.

The control panel for the pump and level sensor is equipped with a dome mounted flashing alarm light that will activate on a high level setting of 2 feet of head. The control panel is equipped with an hour meter, indicating run time for the pump, and a flow totalizer, indicating the quantity of liquid that has been pumped. The readings for the hour meter and for the flow totalizer are to be recorded weekly.

1.5 Stormwater Basin – Leak Detection System – Action Leakage Rate Response Plan

The Action Leakage Rate is set at a rate of 50 gallons-per-day collected over a 30 day period. The Action Leakage Rate (ALR) Response Plan for the Stormwater Basin Leak Detection System is as follows:

- The NCDENR Solid Waste Regional Office in Winston-Salem is notified.
- Samples of the collected liquid are analyzed for specific conductivity, pH, sulfates, chlorides, and boron. Analytical methods are to be the same as described in the Groundwater Sampling and Analysis Plan.
- Samples of the liquid are collected from inside the Stormwater Basin. These samples are subjected to the same analyses.
- The results of these analyses are compared to determine if the source of the collected liquid is the Stormwater Basin.

- The results of the analyses and a summary of the comparison will be sent to the NCDENR Solid Waste Regional Office in Winston-Salem.
- Further actions will be determined based on the results of the comparison of analytical results.

1.6 Placement of Waste and Cover Material

The gypsum residue will be stockpiled adjacent to the landfill by a conveyor system. The waste will then be loaded onto dump trucks and hauled to the landfill active face. As waste is dumped from trucks, the waste shall be spread with a dozer in lifts no greater than 12 inches. Landfilling shall proceed as shown on the Phasing Diagrams, Sheets OP-1 through OP-4.

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

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

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

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

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

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

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

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

1.7 Waste Acceptance and Disposal Requirements

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

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

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

1.8 Hours of Operation

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

1.9 Training of Facility Personnel

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

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

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

1.10 Operations Record Keeping

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

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

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

2.0 SITE OPERATIONS AND MAINTENANCE

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

2.1 Access and Security Requirements

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

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

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

2.2 Erosion/Sediment Control Maintenance

The site is designed with erosion and sedimentation control in accordance with the requirements of the Sedimentation Pollution Control Law (15A NCAC4). Erosion/sedimentation control structures include sediment basins, outlet protection aprons and diversion ditches. Sediment basins shall be checked after periods of significant runoff and as specified in the Erosion and Sediment Control

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

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

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

2.3 Stormwater Structures

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

2.4 Dust, Litter, Odors and Vectors

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

2.5 Fire Control

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

2.6 Groundwater Monitoring Wells

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

2.7 Gas Monitoring

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

PHASING DIAGRAMS

Under Seperate Cover

Appendices

A

CALCULATIONS – REVSION 1

Foundation Evaluation

- Subgrade Bearing Capacity
- Subgrade Settlement Evaluation

Geosynthetics

- Geonet Design – Thickness of Liquid Layer in Final Cover Geonet

Stability

- Liner Stability – Revised for GCL

Leachate Collection and Removal Pipe

- LCR Pipe Stresses

Stormwater Basin – Leak Detection System (LDS)

- Estimated Leakage Rate Through Primary Liner
- Head Loss Calculation for LDS Pump
- Determine Sump Size and Cycle Time for Pump

Stormwater Management

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

Foundation Evaluation

- Subgrade Bearing Capacity
- Subgrade Settlement Evaluation

Discussion of Foundation Evaluation and Conclusions – Revision 1
Revision 0 of the CPA evaluated the stresses imposed on the landfill subgrade and evaluated the settlement of the subgrade due to the imposed loads. The maximum elevation for the final cover in the Revision 0 design was El. 858'. The maximum elevation for the final cover in Revision 1 is El. 870'. The increase in elevation will result in only slight increases in the stresses imposed by the waste. There were also minor changes in the excavated grade in Cell 2 during Revision 1. The excavated grades were raised 1' to 2' in the lower portions of the cell to provide additional distance to the estimated seasonal high groundwater contours.

Figure 1, copied from the Revision 0 CPA, shows the previous final cover design and elevations. Figure 2, shows the Revision 1 final cover design. Figure 3 shows the Revision 1 excavated landfill grades.

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

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

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

The landfill footprint will be primarily located in excavated material. The exception to this is an area in the northwest corner of the landfill, in the eastern end of Cell 1. The maximum width of this fill area is about 160' (measured from the western edge of the footprint) along the El. 776 finished grade contour. The area extends from approximately the El. 770' finished grade contour to the east end of the cell. The reduction in fill area is a result of moving the outside of Cell 1 to the south in Revision 1. The approximate extents of this area is noted on Figure 3.

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

Evaluation of the Bearing Capacity of the Landfill Subgrade
An evaluation of the bearing capacity of the landfill subgrade material was performed and is presented on the attached pages. This evaluation considered the ultimate bearing capacity of the landfill subgrade, determined from SPT blow count, and the applied loads from the waste placement. The maximum ratio of applied load to the bearing capacity was 0.28, with the average ratio equal to 0.17. The bearing capacity of the landfill subgrade is adequate for the applied load.

placement. The maximum ratio of applied load to the bearing capacity was 0.28, with the average ratio equal to 0.17. The bearing capacity of the landfill subgrade is adequate for the applied load.

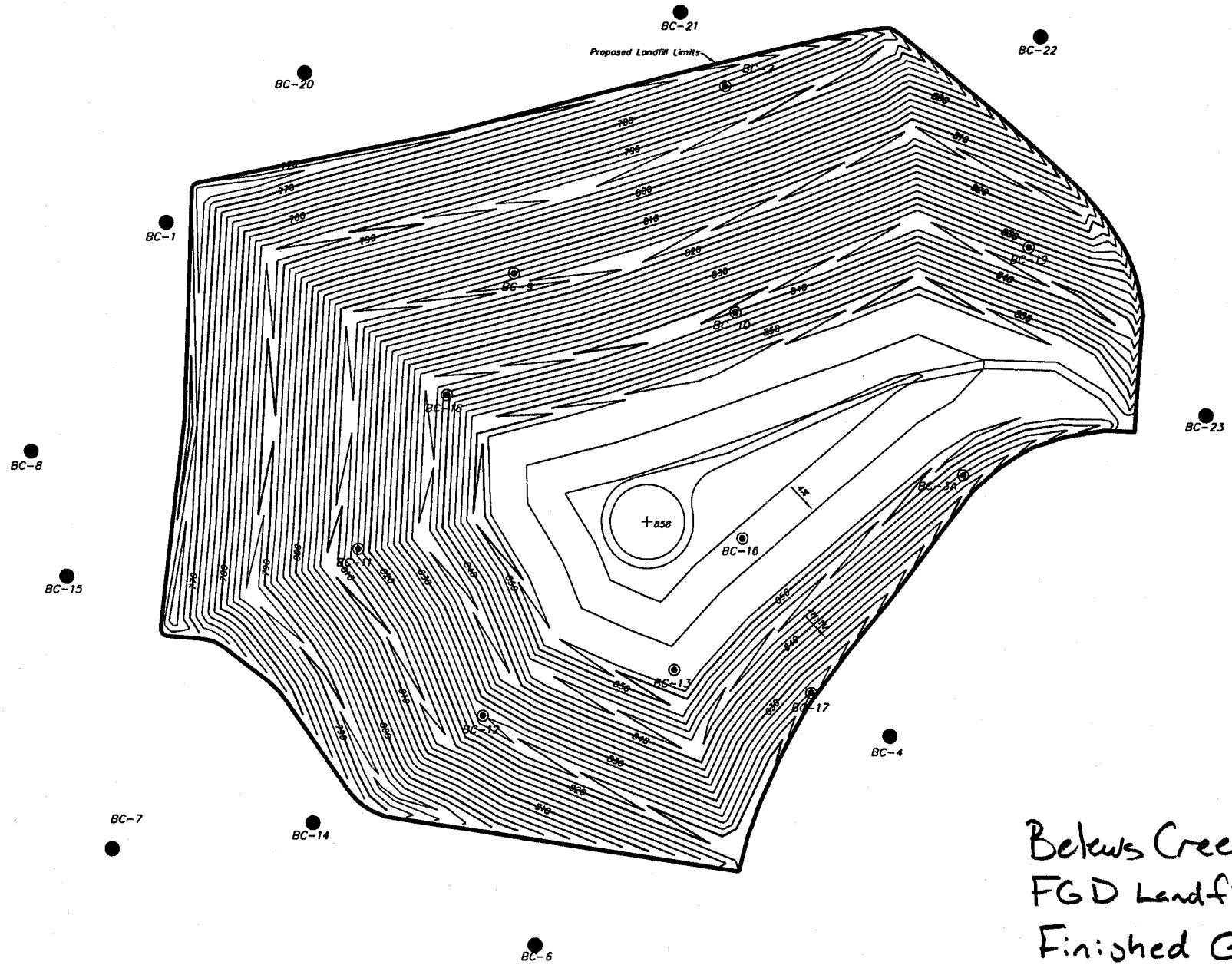
Settlement Analysis

The settlement analysis of the landfill subgrade was performed in Revision 0. The review was based on the boring logs and on the loading applied from the waste, the maximum settlement was found to occur at (or near) BC-10. The settlement at this location is calculated to be 0.11'. The height of fill did not increase at this area; therefore the Revision 0 settlement values remain valid.

Revision 0 also evaluated the settlement near BC-2, since this area will constructed of fill material. The Revision 0 evaluation used 8.3 feet of fill at BC-2 and found the calculated settlement at BC-2 to be 0.18'. The Revision 1 design will have approximately 15' feet of fill at BC-2. This will result in a negligible increase in stresses (and the associated settlement) at this location and the previous settlement values are considered acceptable.

Conclusion

A review was performed of the geotechnical information for the landfill soils and subgrade in Revision 0. Revision 1 increased the height of fill over a portion of the landfill. Evaluations were performed of the bearing capacity of the landfill subgrade and the anticipated settlements of the landfill subgrade. Based on these evaluations, the foundation of the landfill was found to be acceptable.



Scale
1" = 200'

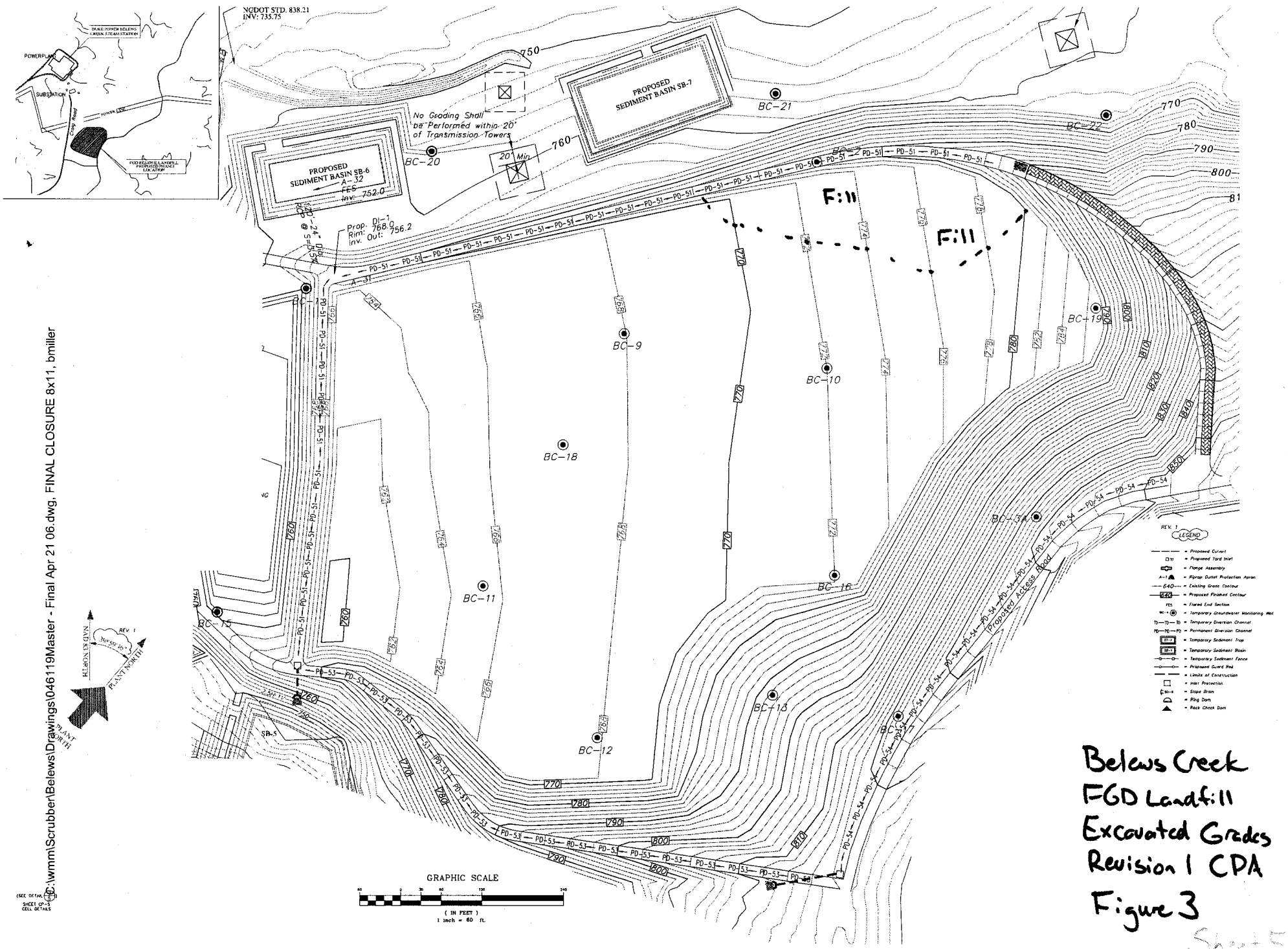
Belews Creek
FGD Landfill
Finished Grades
Rev 0 CPA

Figure 1

Sheet 3

C:\wmm\Scrubber\Belews\Drawings\0461119\Master - Final Apr 21 06.dwg, FINAL CLOSURE 8x11, bmiller

Belews Creek
FGD Landfill
Finished Grades
Revision 1 CPA
Figure 2



Belews Creek
FGD Landfill
Excavated Grades
Revision 1 CDA
Figure 3

Sheet 5

G:\wmm\Scrubber\Belews\Drawings\046119Master - Final Apr 21 06.dwg, FINAL CLOSURE 8x11, bmiller

Objective: Evaluate Foundation Bearing Capacity

References: 1) *Geologic and Hydrologic Siting Report*
FGD Scrubber Residue Disposal Site
Belews Creek Steam Station
3195 Pine Hall Road
Belews Creek, North Carolina
S&ME Project 1054-04-955
February 2005

2) *FGD Scrubber Sludge Testing (Revision 1),*
Power Coal Fired Steam Stations in North Carolina,
Duke S&ME Project No. 1264-03-57, February 2004.

3) *Physical and Geotechnical Properties of Soil, Jospeh E. Bowles, 2nd Edition,*

Belews Creek FGD Residue Landfill
Foundation Evaluation
Bearing Capacity
Revision 1

Job No.	**046119**		
Comp. By	**WMM**	**Date**	**4/22/2006**
Chkd. By	—	**Date**	—
Sheet	7	**of**	—

Foundation Evaluation - Information From Boring Logs

	Boring	Existing Ground Elev. (ft)	Elevation Subgrade (ft)	Depth to Auger Refusal (AR) (bgs) (ft)	Elevation of AR (ft)	Thickness of Material Below Grade & Above AR	Thickness of Material (ft) Below Grade & Above N ≥50	Minimum SPT Blow Count Below Grade	Comments (on material below grade and above auger refusal)
BC -	2	764.74	763.74	43.5	721.2	42.5	22	17	mainly SM and ML (12' of fill to grade)
BC -	3	842.34	834	38.0	804.3	29.7	9	25	mainly MH/SM
BC -	9	773.54	767	13.8	759.7	7.3	0	50/5"	mainly SP/SM
BC -	10	781.84	772	38.5	743.3	28.7	8	18	mainly sand, layer where N=18 is silt
BC -	11	785.15	765	67.9	717.3	47.8	0	21	N = 50+ below layer where N=31
BC -	12	802.91	768	62.8	740.1	27.9	3	31	sand and silt
BC -	13	812.06	792	25.0	787.1	4.9	0	50/2"	4' until coring began
BC -	16	833.77	772	68.3	765.5	6.5	0	N > 50	N = 50+ below grade
BC -	18	780.97	766.6	38.3	742.7	23.9	0	50/3"	N = 50+ below grade
BC -	19	813.09	787	57.2	755.9	31.1	7	20	Thin layer (approx. 10') of silty clayey sand saprolite. Below this layer, N>50.

Belews Creek FGD Residue Landfill
Foundation Evaluation
Bearing Capacity
Revision 1

Job No.	046119		
Comp. By	WMM	**Date**	4/22/2006
Chkd. By	–	**Date**	–
Sheet	8	**of**	

Landfill Loads

	Boring	Existing Ground Elev. (ft)	Proposed Subgrade Elev. (ft) (approx.)	Proposed Cut (ft)	Proposed Final Elev. (ft) (approx.)	Height of Material Over Subgrade ΔH (ft)	Load From Landfill (ksf)
BC -	2	764.74	763.7	(Fill 8.3')	779	15	2.5
BC -	3	842.34	834	8.34	846	12	1.2
BC -	9	773.54	767	6.54	814	47	4.3
BC -	10	781.84	772	9.84	840	68	6.2
BC -	11	785.15	765	20.15	816	51	4.6
BC -	12	802.91	768	34.91	826	58	5.3
BC -	13	812.06	792	20.06	860	68	6.2
BC -	16	833.77	772	61.77	870	98	8.8
BC -	18	780.97	766.6	14.37	839	72.4	6.6
BC -	19	813.09	787	26.09	832	45	4.1

Unit weight of :

site soil (assumed)	125	lb/ft^3
gypsum	89	lb/ft^3

Thickness of cover soil

over liner	1	ft
over cover	2	ft

Belews Creek FGD Residue Landfill
Foundation Evaluation
Bearing Capacity
Revision 1

Job No.	**046119**		
Comp. By	**WMM**	**Date**	**4/22/2006**
Chkd. By	–	**Date**	–
Sheet	9	**of**	–

Bowles (Reference 3) gives the following equation for the ultimate bearing pressure on cohesionless soils:

q_{ult} = 30 N (kPa) (Eqn 15-16)

where N = SPT for layer

using the following conversion: kPa = 20.8 lb/ft²
then kPa = 0.0208 ksf

	Boring	Proposed Final Elev. (ft) (approx.)	Height of Material Over Subgrade ΔH (ft)	Load From Landfill (ksf)	SPT Blow Count Below Grade	Ultimate Bearing Capacity (ksf)	Ratio: Applied Load to Ultimate Bearing Capacity
BC -	2	779	15.26	2.5	17	10.6	0.24
BC -	3	846	12	1.2	25	15.6	0.08
BC -	9	814	47	4.3	50	31.2	0.14
BC -	10	840	68	6.2	70	43.6	0.14
BC -	11	816	51	4.6	85	53.1	0.09
BC -	12	826	58	5.3	31	19.3	0.27
BC -	13	860	68	6.2	50	31.2	0.20
BC -	16	870	98	8.8	50	31.2	0.28
BC -	18	839	72.4	6.6	50	31.2	0.21
BC -	19	832	45	4.1	70	43.5	0.09

Average	0.17
Maximum	0.28

For mat foundations which have a very large B,

$$q_a = \frac{N}{F_2} K_d \tag{15-15}$$

where terms are as defined in Eq. (15-14). For settlements other than 25 mm, assume a linear variation in q_a for both Eqs. (15-14) and (15-15).

Parry (1977) suggested that the ultimate bearing pressure on cohesionless soils using the SPT number N could be computed as

$$q_{ult} = 30N \qquad \text{(kPa)} \tag{15-16}$$

The cone penetration test (CPT) can be used in cohesionless soils according to Meyerhof (1956) to obtain

$$\begin{aligned} q_a &= \frac{q_c}{30} && B \le F_4 \\ q_a &= \frac{q_c}{50}\left(\frac{B + F_3}{B}\right)^2 && B > F_4 \end{aligned} \tag{15-17}$$

where q_c = cone resistance, kPa or k/ft^2, and other terms as defined in Eq. (15-14).

In cohesive soils, the undrained shear strength s_u can be estimated according to Begemann (1974) as

$$s_u = \frac{q_c - p_o}{N'_c} \tag{15-18}$$

where p_o = effective overburden pressure at CPT depth
N'_c = constant ranging from 5 to 70 depending on deposit and OCR (Values ranging from 9 to 15 are most common.)

Since $s_u = c$, we can use Eqs. (15-10) to obtain q_{ult}. This rough approximation would not justify the use of "refined" bearing capacity equations such as Eq. (15-11). The cone resistance q_c should be an average obtained in a zone similar to N of about 0 to 0.75B below the estimated footing depth.

Example 15-6 The average SPT blow count N in the effective zone beneath a footing is 15. Estimate the allowable bearing capacity q_a, assuming that the footing depth D is 3.3 m and the soil in the zone of influence is a medium dense, coarse sand.

SOLUTION Since q_a depends on B we will make a graph as in Fig. E15-6 of q_a versus B. We should also be aware that while the value of q_a should limit sand settlement to 25 mm, if there is a soft clay layer in close proximity which may consolidate, the total settlement will be more than 25 mm unless q_a is further reduced.

Reference 3

Geosynthetics

- Geonet Design – Thickness of Liquid Layer in Final Cover Geonet

Belews Creek FGD Residue Landfill	**Job No.**	**046119**	
Check Thickness of Liquid in Geonet Used	**Comp. By**	**WMM**	**Date** 4/12/2006
For Final Cover System	**Chkd. By**	DEB	**Date** 4/17/06
	Sheet	1	**of** 2

Objective: **Check Thickness of Liquid in Geonet Used for Final Cover For Rainfall from 25 Year - 24 Hour Storm**

References:

(1) Hydraulic Design of Geosynthetic and Granular Liquid Collection Layers, Giroud, Zornberg, Zhao, Geosynthetic International, 2000 Vol.7.

(2) CETCO Texdrain Geocomposites Product Sheet

Reference 1 presents a method for calcualting the thickness of a layer of liquid collected in a drainage layer.

This calculated thickness can be compared to the actual thickness of the drainage layer used.

From Reference 1, Eqn. 16:

$$t\ max = \left[\frac{\sqrt{\tan^2 B + 4qh/k} - \tan B}{2 \cos B} \right] * L$$

B = slope angle, degrees
qh = liquid inflow, m/s
k = perm of geonet, m/s
L = length of slope, m

geonet tranmissivity, $m^2/s, \Theta$ =	1.50E-03	
geonet thickness, mil =	200	
=	0.005 m	
length of slope, ft =	102	
slope (ft/ft) =	0.25	
rainfall, in =	5.6	Assume entire quantity of rainfall from 25 year, 24 hour storm

B =	14.04	degrees
B =	0.24	radians
qh =	1.65E-06	m
k =	2.95E-01	m/s
L =	31.10	m

Belews Creek FGD Residue Landfill
Check Thickness of Liquid in Geonet Used
For Final Cover System

Job No.	**046119**		
Comp. By	**WMM**	**Date**	**4/12/2006**
Chkd. By	OGB	**Date**	4/17/06
Sheet	2	**of**	2

Re-write equation for solution:

$$t\ max = \left[\frac{a - b}{c}\right] * L$$

a =	0.25004
b =	0.25
c =	1.94

tmax =	0.0007 m
tmax =	**0.72 mm**

Thickness of geonet = 5 mm

Maximum thickness from rainfall is less than thickness of geonet.

ratio of maximum thickness of liquid to geonet thickness = 0.14

Technical Paper by J.P. Giroud, J.G. Zornberg, and A. Zhao

Hydraulic Design of Geosynthetic and Granular Liquid Collection Layers

ABSTRACT: The present paper provides equations for the hydraulic design of liquid collection layers. A first series of equations gives the maximum thickness of the liquid collected in a liquid collection layer. These equations are used in design to check that the maximum liquid thickness is less than an allowable thickness. Some of the equations make it possible to rigorously calculate the liquid thickness, whereas other equations, which are simpler, give an approximate value of the liquid thickness. A second series of equations makes it possible to calculate the required hydraulic conductivity of the liquid collection layer material and the required hydraulic transmissivity of the liquid collection layer. These equations are useful for selecting the material used to construct the liquid collection layer. The equations provided in the present paper include reduction factors to quantify the decrease in flow capacity of liquid collection layers due to thickness reduction (caused by the applied stresses) and hydraulic conductivity reduction (caused by clogging). Practical recommendations and design examples are presented for both geosynthetic and granular liquid collection layers.

KEYWORDS: Liquid collection layer, Leachate collection layer, Leakage detection and collection layer, Drainage layer, Thickness, Granular, Geosynthetic.

AUTHORS: J.P. Giroud, Chairman Emeritus, GeoSyntec Consultants, 621 N.W. 53rd Street, Suite 650, Boca Raton, Florida 33487, USA, Telephone: 1/561-995-0900, Telefax: 1/561-995-0925, E-mail: jpgiroud@geosyntec.com; J.G. Zornberg, Assistant Professor, Department of Civil, Environmental and Architectural Engineering, University of Colorado at Boulder, Campus Box 428, Boulder, Colorado 80309-0428, USA, Telephone: 1/303-492-4699, Telefax:1/303-492-7317, E-mail: zornberg@colorado.edu; and A. Zhao, Vice President of Engineering, Tenax Corp., 4800 E. Monument Street, Baltimore, Maryland 21205, USA, Telephone: 1/410-522-7000, Telefax: 1/410-522-7015, E-mail: azhao@us.tenax.com.

PUBLICATION: *Geosynthetics International* is published by the Industrial Fabrics Association International, 1801 County Road B West, Roseville, Minnesota 55113-4061, USA, Telephone: 1/651-222-2508, Telefax: 1/651-631-9334. *Geosynthetics International* is registered under ISSN 1072-6349.

DATES: Original manuscript submitted 12 July 2000, revised version received 4 October 2000, and accepted 7 October 2000. Discussion open until 1 June 2001.

REFERENCE: Giroud, J.P., Zornberg, J.G., and Zhao, A., 2000, "Hydraulic Design of Geosynthetic and Granular Liquid Collection Layers", *Geosynthetics International*, Special Issue on Liquid Collection Systems, Vol. 7, Nos. 4-6, pp. 285-380.

Reference 1

of the slope on which the liquid collection layer is installed are the slope angle and the slope length.

If design calculations show that the considered liquid collection layer material does not provide adequate flow capacity, the design engineer has the following options: (i) a liquid collection layer with a greater thickness (but this option is inappropriate if, in the original design, the liquid thickness was equal to a regulatory maximum liquid thickness); (ii) a different drainage material with a greater hydraulic conductivity (or greater hydraulic transmissivity); and (iii) a liquid collection layer with a shorter length and/or a steeper slope. The last option is the only one available if the liquid collection layer and its material may not be changed. However, slope steepness may be limited by stability and/or by waste storage capacity considerations.

2 AVAILABLE EQUATIONS AND DISCUSSIONS

2.1 Overview

As indicated in Section 1.5, the liquid flowing in a liquid collection layer has a maximum thickness at a certain location along the slope on which the liquid collection layer is constructed. Equations given in Section 2 provide the maximum liquid thickness as a function of the following parameters: the hydraulic conductivity of the liquid collection layer material; the length and slope of the liquid collection layer; and the rate of liquid supply. Some solutions provide an accurate value of the maximum liquid thickness, some provide an approximate value. The validity of the equations that provide approximate values is assessed. Section 2 only presents the equations and discusses their accuracy. The use of the equations will be presented in Sections 3 and 4.

2.2 Solution Based on Simplified Assumptions

As indicated in Section 1.5, the hydraulic gradient increases from the top to the toe of the slope on which the liquid collection layer is constructed. Giroud (1985) used an average value of the hydraulic gradient and developed the following equation:

$$t_{max} = \frac{\sqrt{\tan^2\beta + 4\,q_h/k} - \tan\beta}{2\cos\beta}\,L \qquad (16)$$

The demonstration of Equation 16 can be found in Appendix A.

Equation 16 can be written as follows:

$$t_{max} = \frac{\sqrt{1+4\lambda} - 1}{2\cos\beta/\tan\beta}\,L \qquad (17)$$

where λ is the characteristic parameter (dimensionless) defined by Equation 9.

It is important to note that, as shown in Appendix A, Equations 16 and 17 tend toward well known equations for the two limit cases where λ tends toward zero and toward infinity. This confirms the validity of Equations 16 and 17. Equation 16 (or 17, which is equivalent) has been used as "Giroud's equation" for the design of numerous leachate

Reference 1

collection layers and leakage collection layers in many landfills in the United States. Giroud's equation has progressively replaced the use, in the United States, of "Moore's equations", which had been presented in documents published by the US Environmental Protection Agency (Moore 1980, 1983; USEPA 1989), but for which no derivation was ever published or otherwise made available, to the best of the authors' knowledge. In the early 1980s, the senior author suspected the validity of Moore's equations because they did not tend toward the well-known limits when λ tends toward zero or toward infinity. This prompted the development of equations by the senior author. In the remainder of the present paper, Equation 16 (or 17) will be referred to as the "original Giroud's equation".

2.3 Governing Differential Equation

2.3.1 *Establishment of the Governing Differential Equation*

The differential equation that governs the flow of liquid in a liquid collection layer receiving a uniform liquid supply can be established as follows. Based on the principle of mass conservation, the flow rate at abscissa x is:

$$Q = q_h B x \tag{18}$$

where: Q = flow rate; B = width of the liquid collection layer in the direction perpendicular to the direction of the flow; and x = distance, measured horizontally, between the top of the liquid collection layer slope and the point where the liquid thickness is evaluated (Figure 5).

Darcy's equation can be written as follows:

$$Q = k B t i \tag{19}$$

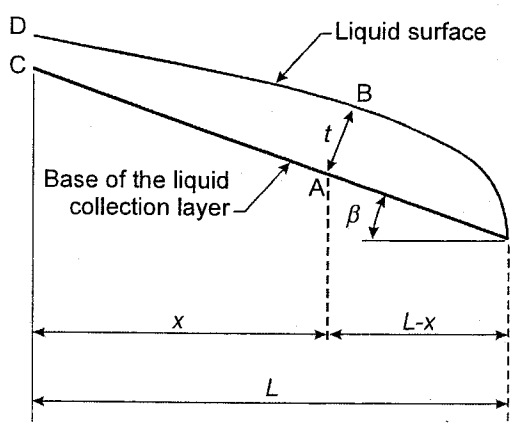


Figure 5. Definition of parameters used in the establishment of the governing differential equation.

Reference 1

- The original Giroud's equation (i.e. Equation 16 or 17) can be solved for all parameters: k, q_h , or even β.

Because it is simple and accurate, the modified Giroud's equation (i.e. Equation 40 or 41) will be used extensively in the remainder of the present paper. Also, because it is accurate and easily programmable, the modified Giroud's equation is the recommended equation for parametric studies where precision is required. Finally, the original Giroud's equation is recommended for usual design calculations because it is very simple, while being sufficiently accurate for all values of the parameters.

The purpose of Example 1, which follows, is to illustrate the comparison between various equations. For the sake of simplicity, and to focus the attention on the comparisons between equations, the reduction factors presented in Section 1.7 are not used in Example 1. Therefore, Example 1 should not be considered as a "design example". Design examples illustrating the use of reduction factors and factor of safety are presented in Sections 3 and 4.

Example 1. A liquid collection layer used in a landfill cover has a length (measured horizontally) of 30 m and a 2% slope. The rate of liquid supply is 100 mm per day. A drainage geocomposite having a core thickness of 9 mm is used. Its hydraulic transmissivity, measured under hydraulic gradients consistent with a slope of 2%, is 3.6×10^{-3} m^2/s. Calculate the maximum liquid thickness.

The liquid supply rate is calculated using Equation 8 as follows:

$$q_h = \frac{0.1}{86{,}400} = 1.157\times10^{-6}\ \text{m/s}$$

The hydraulic conductivity of the geosynthetic is derived from its hydraulic transmissivity and its thickness using Equation 2 as follows:

$$k = \frac{3.6\times10^{-3}}{9\times10^{-3}} = 0.4\ \text{m/s}$$

An approximate value of the maximum liquid thickness can be obtained by using the original Giroud's equation (i.e. Equation 16) as follows:

$$t_{max} = \left[\frac{\sqrt{(0.02)^2 + (4)\left(1.157\times10^{-6}\right)/0.4} - 0.02}{2\cos\left(\tan^{-1}0.02\right)}\right](30) = 4.31\times10^{-3}\,\text{m} = 4.31\,\text{mm}$$

An accurate value of the maximum liquid thickness can be obtained by using the modified Giroud's equation (i.e. Equation 36), which is equivalent to multiplying the value calculated above using Equation 16 by the dimensionless modifying factor, j, calculated using Equation 38 as follows:

$$j = 1 - 0.12\exp\left\{-\left[\log\left[\frac{(8)\left(1.157\times10^{-6}\right)/(0.4)}{(5)(0.02)^2}\right]^{5/8}\right]^2\right\} = 0.972$$

Reference 1

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/ft^2	.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	m^2/s	1 x 10-3

* tested between two metal plates at 15,000 lb/ft^2 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/ft^2	.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	m^2/sec	1.5 x 10-3

* tested between two metal plates at 15,000 lb/ft^2 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 (m^2/sec)
TexDrain 200 SS 6	0.97	2×10^{-4}
TexDrain 200 SS 8	0.48	1×10^{-4}
TexDrain 200 DS 6	0.48	1×10^{-4}
TexDrain 200 DS 8	0.19	4×10^{-5}
TexDrain 250 SS 6	2.17	4.5×10^{-4}
TexDrain 250 SS 8	1.2	2.5×10^{-4}
TexDrain 250 DS 6	1.44	3×10^{-4}
TexDrain 250 DS 8	0.84	1.75×10^{-4}

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

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

Reference 2

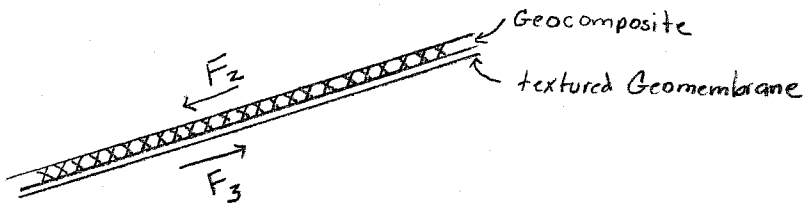
Stability

Liner Stability – Revised for GCL

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

Comp. By PMB Date 9-30-05 Project Belows Creek FGD Landfill Sheet No. 3 of ~~3~~ 4 } - △1
Chkd. By WMM Date 9/30/05 Subject Linear Stability Job No. 046119

Geocomposite / Textured Geomembrane



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

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

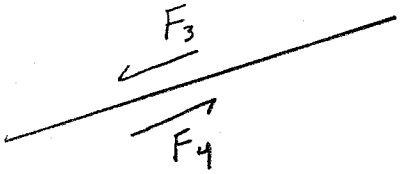
$FS = \frac{3650}{946} = \underline{\underline{3.9}}$ ok

△1

Textured Geomembrane / ~~soil~~ GCL

△1

Coefficients of Friction:
Textured Geomembrane / GCL ⇒ $\delta_3 = 15°$
GCL / Soil $= \delta_4 = 23°$



$F_3 = F_2 = \underline{946\ ^{lbs}/_{ft}}$ △1

$F_{4\,Avail} = N \tan \delta_3$
$= 7{,}485\ ^{lbs}/_{ft} (\cancel{\tan 22°}\ \tan 15°)$
$= \cancel{3{,}024}\ 2006\ ^{lbs}/_{ft}$

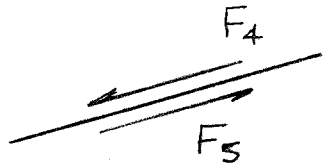
$FS = \frac{\cancel{3024}\ 2006}{946} = \cancel{3.2}\ 2.1$ ok ok

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

Revision △1
by: WMM 1/19/2006

Reference: CETCO Labratory Report - Bentomat Direct Shear Testing Summary TR-114bm

GCL / Soil



$F_4 = F_5 = 946$ lb/ft

$F_{4\,avail} = N \tan \delta_4$

$= 7{,}485$ lb/ft $(\tan 23°)$

$F_{4\,avail} = 3{,}177$ lb/ft

$FS = \frac{3{,}177 \text{ lb/ft}}{946 \text{ lb/ft}} = 3.3$ OK

∴ Liner system is stable with 5' of gypsum

BENTOMAT® DIRECT SHEAR TESTING SUMMARY

The following table summarizes the direct shear testing on Bentomat that has been performed by CETCO and other laboratories on a project-specific basis for the past several years. This data will give the designer some general information about the shear strength of commonly used GCL interfaces and should be the first step in evaluating a proposed liner system where slope stability is a concern.

The variables in any direct shear test are numerous, including specimen preparation; hydration pressures, liquids, and sequencing, and rate of shear, and others. Test results will vary accordingly, which is partially accountable for the wide range of data reported even for similar interfaces.

This data is for informational purposes only and is not intended to replace project-specific interface testing, which CETCO emphatically recommends. CETCO makes no warranty as to the usefulness of the data. Individual test reports for most of the summarized data can be provided upon request.

1500 W. Shure Drive • Arlington Heights, IL 60004 • USA • (847) 392-5800 • FAX (847) 577-5571 /www.CETCO.com
A wholly owned subsidiary of AMCOL International

The information and data contained herein are believed to be accurate and reliable. CETCO makes no warranty of any kind and accepts no responsibility for the results obtained through application of this information.

SUMMARY OF BENTOMAT DIRECT SHEAR TEST DATA
INTERFACE W/ GEOMEMBRANE (Continued)

Lab[1]	Report Date	Interface Tested[2]	Normal Stresses (psi)	Bentomat Moisture[3]	Shear Rate (in/min)	Peak Friction Angle (deg)	Residual Friction Angle (deg)[4]	Apparent Peak Cohesion (psf)	Comments
GSC	03-12-96	W/ 30 mil PVC	2 - 4 - 6	Hydrated	0.04	17	17	24	
GSC	05-29-96	NW/80mil Text. HDPE	140	Hydrated	0.04	19	5	475	h
AGP	11-08-96	NW/60mil Text. HDPE	1.4 - 3.5 - 7.0	Hydrated	0.04	34.8	22.7	149	
		W/60 mil Text. HDPE	1.4 - 3.5 - 7.0	Hydrated	0.04	28.8	20.4	83	
GSC	01-08-97	NW/60mil Text. HDPE	14 - 40 - 70	Hydrated	0.04	17	9	255	
TRI	4-15-97	NW/60mil Text. HDPE	14 – 28 – 56	Hydrated	0.04	21.9	10.8	722	
Emcon	6-16-97	NW/40mil Text. LLPE	1.0 – 2.0 – 3.5	Hydrated	0.04	32.0	18.5	111	
		NW/40mil Text. LLPE	1.0 – 2.0 – 3.5	Hydrated	0.04	37.5	27.1	118	
TRI	10-15-97	NW/ 60mil Text.HDPE	3.5 – 7 – 14	Hydrated	0.04	20.3	18.6	278	

1500 W. Shure Drive • Arlington Heights, IL 60004 • USA • (847) 392-5800 • FAX (847) 577-5571 /www.CETCO.com
A wholly owned subsidiary of AMCOL International

The information and data contained herein are believed to be accurate and reliable. CETCO makes no warranty of any kind and accepts no responsibility for the results obtained through application of this information.

SUMMARY OF BENTOMAT DIRECT SHEAR TEST DATA
INTERFACE W/ GEOMEMBRANE (Continued)

Lab[1]	Report Date	Interface Tested[2]	Normal Stresses (psi)	Bentomat Moisture[3]	Shear Rate (in/min)	Peak Friction Angle (deg)	Residual Friction Angle (deg)[4]	Apparent Peak Cohesion (psf)	Comments
		NW/ 60mil Text.HDPE	.35 - .7 – 1.4	Hydrated	0.04	36.6	25.8	2	
TRI	12-01-97	NW/60mil Text. HDPE	0.35 – 0.7 – 1.4	Hydrated	0.001	25.6	23.3	54	Residual @ 4"
		NW/60mil Text. HDPE	3.5 – 7 – 14	Hydrated	0.001	23.2	17.8	85	"
Emcon	04-06-98	NW/60mil Text. HDPE	14 – 28 – 70	Hydrated	0.04	24.1	12.2	290	
Cetco	06-08-98	NW/ 60mil Text. HDPE	0.5 – 1.4 - 7	Hydrated	0.04	25.7	21.9	85	
		NW/ 60mil Text. HDPE	28 – 50 – 70	Hydrated	0.04	16.1	14.4	482	
GSC	07-18-98	W/ 60mil text. HDPE	25 – 50 – 100	Hydrated	0.04	15	15	295	

1500 W. Shure Drive • Arlington Heights, IL 60004 • USA • (847) 392-5800 • FAX (847) 577-5571 /www.CETCO.com
A wholly owned subsidiary of AMCOL International

The information and data contained herein are believed to be accurate and reliable. CETCO makes no warranty of any kind and accepts no responsibility for the results obtained through application of this information.

SUMMARY OF BENTOMAT DIRECT SHEAR TEST DATA
INTERFACE W/ GEOMEMBRANE (Continued)

Lab[1]	Report Date	Interface Tested[2]	Normal Stresses (psi)	Bentomat Moisture[3]	Shear Rate (in/min)	Peak Friction Angle (deg)	Residual Friction Angle (deg)[4]	Apparent Peak Cohesion (psf)	Comments
	07-25-98	W/ 80mil text. HDPE	25 – 50 – 100	Hydrated	0.04	18	18	85	
	08-13-98	W/ 80mil text. HDPE	25 – 50 – 100	Hydrated	0.04	18	18	205	
VE	08-14-98	W/ 80mil Text. HDPE	5 – 20 – 45	Hydrated	0.004	26	13	50	
		W/ 80mil Text. HDPE	5 – 20 – 45	Hydrated	0.004	22	13	40	
		W/ 80mil Text. HDPE	5 – 20 – 45	Hydrated	0.004	19	11	180	
		W/ 80mil Text. HDPE	5 – 20 – 45	Hydrated	0.004	24	13	30	
		W/ 80mil Text. HDPE	5 – 20 – 45	Hydrated	0.004	28	13	60	
		W/ 80mil Text. HDPE	5 – 20 – 45	Hydrated	0.004	18	13	250	
		W/ 80mil Text. HDPE	5 – 20 – 45	Hydrated	0.004	21	14	240	

1500 W. Shure Drive • Arlington Heights, IL 60004 • USA • (847) 392-5800 • FAX (847) 577-5571 /www.CETCO.com
A wholly owned subsidiary of AMCOL International

The information and data contained herein are believed to be accurate and reliable. CETCO makes no warranty of any kind and accepts no responsibility for the results obtained through application of this information.

SUMMARY OF BENTOMAT DIRECT SHEAR TEST DATA
INTERFACE W/ GEOMEMBRANE (Continued)

Lab[1]	Report Date	Interface Tested[2]	Normal Stresses (psi)	Bentomat Moisture[3]	Shear Rate (in/min)	Peak Friction Angle (deg)	Residual Friction Angle (deg)[4]	Apparent Peak Cohesion (psf)	Comments
GSC	08-28-98	W/60mil text. HDPE	25 – 50 – 100	Hydrated	0.04	20	18	360	
	09-12-98	W/ 60mil text HDPE	25 – 50 – 100	Hydrated	0.04	23	11	135	
TRI	09-22-98	NW/ 60mil Text. HDPE	14 – 28 – 63	Hydrated	0.04	17.9	7.8	344	Residual @ 3"
GSC	09-24-98	W/ 80mil text. HDPE	25 – 50 – 100	Hydrated	0.04	18	15	40	
	10-05-98	W/ 60mil text HDPE	25 – 50 – 100	Hydrated	0.04	22	10	195	
		W/ 80mil text. HDPE	25 – 50 – 100	Hydrated	0.04	19	14	135	
	10-09-98	W/ 60mil text HDPE	25 – 50 – 100	Hydrated	0.04	20	11	345	
	10-21-98	W/ 80mil text. HDPE	25 – 50 – 100	Hydrated	0.04	18	16	235	
	10-22-98	W/ 80mil text. HDPE	25 – 50 – 100	Hydrated	0.04	19	15	120	
	10-24-98	W/ 60mil text HDPE	25 – 50 – 100	Hydrated	0.04	21	11	0	

1500 W. Shure Drive • Arlington Heights, IL 60004 • USA • (847) 392-5800 • FAX (847) 577-5571 /www.CETCO.com
A wholly owned subsidiary of AMCOL International

The information and data contained herein are believed to be accurate and reliable. CETCO makes no warranty of any kind and accepts no responsibility for the results obtained through application of this information.

SUMMARY OF BENTOMAT DIRECT SHEAR TEST DATA
INTERFACE W/ GEOMEMBRANE (Continued)

Lab[1]	Report Date	Interface Tested[2]	Normal Stresses (psi)	Bentomat Moisture[3]	Shear Rate (in/min)	Peak Friction Angle (deg)	Residual Friction Angle (deg)[4]	Apparent Peak Cohesion (psf)	Comments
	01-19-99	W/ 80mil text. HDPE	25 – 50 – 100	Hydrated	0.04	19	8	150	
	01-22-99	W/ 80mil text. HDPE	25 – 50 – 100	Hydrated	0.04	18	9	425	
	01-27-99	W/ 80mil text. HDPE	25 – 50 – 100	Hydrated	0.04	19	7	330	
PL	03-08-99	NW/ 60mil text. HDPE	7 – 28 - 56	Hydrated	0.04	25	12.5	182	Residual @ 3"
		NW/ 60mil text. HDPE	7 – 28 - 56	Hydrated	0.04	27	13.6	70	"
QORE	03-23-99	W/ 40mil PVC	27.1 – 54.3 – 81.4	Hydrated	0.0039	9.5	9.2	677	
VE	06-16-99	W/ 80mil Text. HDPE	5.0 – 20.0 – 45.0	Hydrated	0.004	22	11	100	Residual @ 3"
		W/ 80mil Text. HDPE	5.0 – 20.0 – 45.0	Hydrated	0.004	18	12	190	"
		W/ 80mil Text. HDPE	5.0 – 20.0 – 45.0	Hydrated	0.004	19	10	20	"

1500 W. Shure Drive • Arlington Heights, IL 60004 • USA • (847) 392-5800 • FAX (847) 577-5571 /www.CETCO.com
A wholly owned subsidiary of AMCOL International

The information and data contained herein are believed to be accurate and reliable. CETCO makes no warranty of any kind and accepts no responsibility for the results obtained through application of this information.

SUMMARY OF BENTOMAT DIRECT SHEAR TEST DATA
INTERFACE W/ GEOMEMBRANE (Continued)

Lab[1]	Report Date	Interface Tested[2]	Normal Stresses (psi)	Bentomat Moisture[3]	Shear Rate (in/min)	Peak Friction Angle (deg)	Residual Friction Angle (deg)[4]	Apparent Peak Cohesion (psf)	Comments
GSC	06-21-99	NW/ 40mil text. HDPE	14 – 28 – 56 – 112	Hydrated	0.04	18	9	375	
	06-25-99	NW/ 40mil text. HDPE	14 – 28 – 56 – 112	Hydrated	0.04	20	4	590	sheared w/in GCL @ 56, 112 psi
GSC	08-06-99	NW/ 60mil text. HDPE	10 – 25 – 50 – 75	Hydrated	0.04	21	16	150	
STS	10-28-99	NW/ 40mil text LLDPE	0.5 – 1 – 2 – 3	Hydrated	0.04	28.8	21.8	72	
GSC	02-01-00	NW/ 60mil text HDPE	7 – 14 – 35	Hydrated	0.0016	29	18	370	
		NW/ 60mil text. HDPE	35 – 100 - 150	Hydrated	0.0016	21	9	130	
PL	2-5-00	ST/ 40mil text LLDPE	0.35-0.69-1.39	Hydrated	0.04	29	16	196	residual @ 3.2"
PL	6-4-01	NW/60mil text HDPE	1.3-2.6-6.3	Hydrated	0.001	24.8	18.9	230	Hydrated under 72 psf
		NW/60mil text HDPE	1.3-2.6-6.3	Hydrated	0.001	21.4	18.5	225	Hydrated under 216 psf

1500 W. Shure Drive • Arlington Heights, IL 60004 • USA • (847) 392-5800 • FAX (847) 577-5571 /www.CETCO.com
A wholly owned subsidiary of AMCOL International

The information and data contained herein are believed to be accurate and reliable. CETCO makes no warranty of any kind and accepts no responsibility for the results obtained through application of this information.

SUMMARY OF BENTOMAT DIRECT SHEAR TEST DATA
INTERFACE W/ SOIL

Lab[1]	Report Date	Interface Tested[2]	Normal Stresses (psi)	Bentomat Moisture[3]	Shear Rate (in/min)	Peak Friction Angle (deg)	Residual Friction Angle (deg)[4]	Apparent Peak Cohesion (psf)	Comments
GSC	03-09-95	NW/ Soaked GC Soil	7.5 - 15 - 30	Hydrated	0.04	33	31	385	
Cetco	08-29-95	W/Silty Sand	1 - 2 - 3	Hydrated	0.04	32.3	ND	127	
GSC	03-12-96	NW/ Soaked Soil	2 - 4 - 6	Hydrated	0.04	29	29	70	
STS	05-30-96	NW/Silty Sandy Gravel	0.7 – 2 – 3.5	Hydrated	0.04	28	ND	222	
Cetco	07-08-96	NW/ Sandy Silty Gravel	0.7 - 2.0 - 3.5	Hydrated	0.04	40.4	ND	238	
	07-31-96	W/ Sandy Silty Gravel	0.7 - 2.0 - 3.5	Hydrated	0.04	37.8	ND	194	
AGP	11-08-96	NW/ SC Soil	1.4 - 3.5 - 7.0	Hydrated	0.04	27.8	ND	186	
		W/ SC Soil	1.4 - 3.5 - 7.0	Hydrated	0.04	23.5	22.1	187	

* All $\phi \geq 23°$

TR-114bm
Revised 8/02

1500 W. Shure Drive • Arlington Heights, IL 60004 • USA • (847) 392-5800 • FAX (847) 577-5571 /www.CETCO.com
A wholly owned subsidiary of AMCOL International

The information and data contained herein are believed to be accurate and reliable. CETCO makes no warranty of any kind and accepts no responsibility for the results obtained through application of this information.

SUMMARY OF BENTOMAT DIRECT SHEAR TEST DATA
INTERFACE W/ SOIL

Lab[1]	Report Date	Interface Tested[2]	Normal Stresses (psi)	Bentomat Moisture[3]	Shear Rate (in/min)	Peak Friction Angle (deg)	Residual Friction Angle (deg)[4]	Apparent Peak Cohesion (psf)	Comments
GSC	01-08-97	NW/ Soaked Soils	14 - 40 - 70	Hydrated	0.04	18	18	155	
		← Soaked soils							
TRI	09-23-97	NW/ Subbase Soil	3.5 – 7 – 14	Hydrated	0.001	24.0	20.7	4	
		NW/ Subbase Soil	.35 - .7 – 1.4	Hydrated	0.001	35.5	37.2	88	
GSC	01-22-98	CLS/ SM soil	5 - 25 - 45	Hydrated	0.04	17	13	70	
		← different material see Notes							
Cetco	02-10-98	CLS/ SC Soil	0.5 – 1 – 1.5	Hydrated	0.04	9.5	ND	46	
			← low confining stress						
VE	03-03-98	W/ Cover soil	1 - 2.1 – 4.9	Hydrated	0.04	37	33	50	
Cetco	08-11-98	W/ SC soil	2.1 – 4.2 – 6.3	Hydrated	0.04	33.6	35.2	183	
		NW/ Non-Pag Quartz Monzonite Waste	2.1 – 4.2 – 6.3	Hydrated	0.04	39.8	37.5	190	
		← Non-woven							
TRI	09-22-98	NW/ soil	16 – 32 – 73	Hydrated	0.04	17.5	16.8	505	Residual @ 3"
Cetco	11-3-98	CLS/SM soil	0.5 – 1 – 1.5	Hydrated	0.04	27	27	80	
		CLS/SP soil	0.5 – 1 – 1.5	Hydrated	0.04	15.2	15.2	55	
		↑ different material See Notes							

TR-114bm
Revised 8/02

1500 W. Shure Drive • Arlington Heights, IL 60004 • USA • (847) 392-5800 • FAX (847) 577-5571 /www.CETCO.com
A wholly owned subsidiary of AMCOL International

The information and data contained herein are believed to be accurate and reliable. CETCO makes no warranty of any kind and accepts no responsibility for the results obtained through application of this information.

SUMMARY OF BENTOMAT DIRECT SHEAR TEST DATA
INTERFACE W/ SOIL (Continued)

Lab[1]	Report Date	Interface Tested[2]	Normal Stresses (psi)	Bentomat Moisture[3]	Shear Rate (in/min)	Peak Friction Angle (deg)	Residual Friction Angle (deg)[4]	Apparent Peak Cohesion (psf)	Comments
GSC	12-18-98	CLS/MH soil	0.5 – 1 – 1.5	Hydrated	0.04	16	15	14	
		← different material See Notes				←			
ATT	03-18-99	W/ SC soil	1.4 – 2.8 – 5.6	Hydrated	0.04	32.6	31.9	19.5	
STS	05-13-99	NW/ sand	1.0 – 2.0 – 4.0	Hydrated	0.04	38.4	ND	142	
GSC	08-06-99	NW/ subgrade soil	10 – 25 – 50 – 75	Hydrated	0.04	33	32	575	
GSC	12-07-99	NW/ site soil	0.5 – 1.0 – 1.5	Hydrated	0.04	35	35	29	
STS	01-17-00	NW/ tailings	1.0 – 2 – 4	Hydrated	0.04	40.8	36.7	171	
		W/ SC soil	1.0 – 2 – 4	Hydrated	0.04	28.6	28.0	292.5	
		W/ tailings	1.0 – 2 – 4	Hydrated	0.04	41.2	36.9	125	
Cetco	01-31-00	CLT/sand	0.35 - 0.7 - 1.4	Hydrated	0.04	44.6	ND	155	
		CLS/sand	0.35 - 0.7 - 1.4	Hydrated	0.04	38.7	ND	250	

1500 W. Shure Drive • Arlington Heights, IL 60004 • USA • (847) 392-5800 • FAX (847) 577-5571 /www.CETCO.com
A wholly owned subsidiary of AMCOL International

The information and data contained herein are believed to be accurate and reliable. CETCO makes no warranty of any kind and accepts no responsibility for the results obtained through application of this information.

SUMMARY OF BENTOMAT DIRECT SHEAR TEST DATA
INTERFACE W/ SOIL (Continued)

Lab[1]	Report Date	Interface Tested[2]	Normal Stresses (psi)	Bentomat Moisture[3]	Shear Rate (in/min)	Peak Friction Angle (deg)	Residual Friction Angle (deg)[4]	Apparent Peak Cohesion (psf)	Comments
Cetco	02-24-00	CLT/ SP soil	0.7 – 1.4 – 2.1	Hydrated	0.04	41.7	ND	108	
	03-01-00	W/ SP soil	0.7 – 1.4 – 2.1	Hydrated	0.04	24.9	ND	278	
	03-09-00	NW/ SP soil	0.7 – 1.4 – 2.1	Hydrated	0.04	25.2	ND	315	
GSC	6-24-00	CLS/drainage soil	139	Hydrated	0.04	24	0	13	Angle w/ origin
GSC	03-12-01	CLT/subgrade soil	56	Hydrated	0.04	24	21	0	
GSC	03-27-01	NW/subgrade soil	56	Hydrated	0.04	26	23	0	
GSC	04-17-01	NW/onsite soil	1-5-10	Hydrated	0.04	36	35	35	
PL	6-4-01	NW/soil	1.3-2.6-6.3	Hydrated	0.001	21.2	21.6	207	Hydrated under 72psf
		NW/soil	1.3-2.6-6.3	Hydrated	0.001	23.2	20.8	206	Hydrated under 218 psf
ARD	08-08-01	W/SP soil	2.28-2.99-3.75	Hydrated	0.007	38.8	38.8	0	
		NW/SP soil	2.29-3.02-3.90	Hydrated	0.007	36.5	36.5	0	
		W/SP soil	2.18-3.02-3.89	Hydrated	0.007	38.1	38.1	0	
		NW/SP soil	2.21-3.05-3.72	Hydrated	0.007	36.7	36.7	0	
SGI	02-26-02	CLS/SM soil	0.7-1.4-2.8	Hydrated	0.04	18	16	40	
		CLS/CL soil	0.7-1.4-2.8	Hydrated	0.04	19	18	70	
		CLS/GW-GM soil	0.7-1.4-2.8	Hydrated	0.04	20	20	50	Gravel ≤ 1"

Different material See Notes

TR-114bm
Revised 8/02

1500 W. Shure Drive • Arlington Heights, IL 60004 • USA • (847) 392-5800 • FAX (847) 577-5571 /www.CETCO.com
A wholly owned subsidiary of AMCOL International

The information and data contained herein are believed to be accurate and reliable. CETCO makes no warranty of any kind and accepts no responsibility for the results obtained through application of this information.

Notes

[1] AGP = AGP Laboratories Inc., (now Geotesting Express) Arlington, TX (12-inch direct shear box)
ARD = Ardaman and Associates, Orlando FL. (12-inch direct shear box)
ATE = Atlanta Testing & Engrg. (now QORE), Atlanta, GA (12-inch direct shear box)
ATT = Advanced Terra Testing, inc. Lakewood, CO (12-inch direct shear box)
CETCO = CETCO, Arlington Heights, IL (12-inch direct shear box)
CGC = Cumberland Geotechnical Consultants, Carlisle, PA (12-inch direct shear box)
Emcon = Emcon Assoc. (now IT Corp.), Mahwah, NJ (12-inch direct shear box)
GA = Golder Associates, Denver, Colorado (12-inch direct shear box)
GRI = Geosynthetic Research Institute, Drexel University, Philadelphia, PA (Wykeham Farrance device)
GSC = GeoSyntec Consultants, Atlanta, Georgia (12-inch direct shear box and 6-inch direct shear box)
GT = Geotechnics, East Pittsburgh, PA (12-inch direct shear box)
GTX = Geotesting Express, Boxborough, MA (12-inch direct shear box)
PL= Precision Laboratory, Orange, CA (12-inch direct shear box)
PUWL = Purdue University, W. Lafayette, IN (16" x 42" direct shear box)
SGI= SGI Testing Services LLC, Atlanta, GA (12-inch direct shear box and 6-inch direct shear box)
SL = Singleton Labs, Inc.(12-inch shear box)
STS = STS Consultants, Ltd., Vernon Hills, IL (12-inch direct shear box)
TRI = TRI Laboratory, Austin, TX (12-inch direct shear box)
UTA = University of Texas at Austin, Civil Engineering Laboratory (2.4-inch direct shear box)
VE = Vector Engineering, Grass Valley, CA (12-inch direct shear box)

[2] Internal = Failure forced within the Bentomat (between the geotextiles).
NW = Non-woven geotextile of Bentomat.
W = Woven geotextile of Bentomat.
CLS = Smooth flexible membrane liner laminated onto Bentomat CL and Claymax 600CL
CLT = Textured flexible membrane liner laminated onto Bentomat CLT

[3] "Dry" = specimen was tested in the as-received moisture state which is typically 30 percent.
"Hydrated" = specimen was hydrated prior to testing, *although the actual hydration methods and durations vary.*
All hydrated apecimens were hydrated with distilled or tap water, unless otherwise noted.

[4] Residual at 2" displacement unless noted
ND = Not Determined

1500 W. Shure Drive • Arlington Heights, IL 60004 • USA • (847) 392-5800 • FAX (847) 577-5571 /www.CETCO.com
A wholly owned subsidiary of AMCOL International

The information and data contained herein are believed to be accurate and reliable. CETCO makes no warranty of any kind and accepts no responsibility for the results obtained through application of this information.

Leachate Collection and Removal Pipe

- LCR Pipe Stresses

Belews Creek FGD Residue Landfill	**Job No.**	**046119**	**Date**	**4/19/2006**
Pipe Stress Calculations	**Comp. By**	**WMM**	**Date**	4/19/06
Revision 1	**Chkd. By**	DED	**of**	
	Sheet	1		

6" Diameter Pipe - Operational Loads
Calculate Stresses in 6" Dia. Pipe From Operational Loads
Depth of Gypsum over Operational Cover = 98 feet

Objective: **To determine the pipe stresses for the perforated leachate collection pipes during operation.**

References: 1) Driscopipe Material (copies attached)

Design Data: Operational Cover:

Density	125	lb/ft^3	(assumed)
Thickness	1	ft	(measured from bottom of pipe)

Waste:

Density	89	lb/ft^3	
Thickness	98	ft	measured from top of operational cover

Final Cover

Density	125	lb/ft^3	(assumed)
Thickness	3.5	ft	(includes 18" on top of waste and 2' vegetative 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 = 9285 psf
= 64.5 psi

Static Load:

Ps = Operational Cover = depth of cover to top of pipe * density of cover
Ps = 8847 psf
Ps = 61.4 psi

6" Diameter Pipe - Operational Loads
Calculate Stresses in 6" Dia. Pipe From Operational Loads
Depth of Gypsum over Operational Cover = 98 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 = 98.5 ft
x = 0.0 ft
R = 98.5 ft

Load Factor = 1.5

Plive = 1 psf
= 0.01 psi

This is the load from 1 track. Add load for other track.

H = 98.5 ft
x = 5.0 ft
R = 98.6 ft

Plive = 1.5 psf
0.0 psi

Ps + Plive = 61.46 psi

6" Diameter Pipe - Operational Loads
Calculate Stresses in 6" Dia. Pipe From Operational Loads
Depth of Gypsum over Operational Cover = 98 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	=	67.0 psi	

Wall Crushing Check for Operational Loads

SA = Compressive Strength

$$SA = \frac{SDR - 1}{2} * Pt$$

$$SF = \frac{1500}{SA}$$

Pt = 67.0 psi

SDR = 17

$$SA = \frac{SDR - 1}{2} * Pt$$

SA = 536 psi

Factor of safety, FS:

$$SF = \frac{1500}{SA}$$

= 2.8 **OK**

6" Diameter Pipe - Operational Loads
Calculate Stresses in 6" Dia. Pipe From Operational Loads
Depth of Gypsum over Operational Cover = 98 feet

Critical Collapse

Calculate the critical-collapse differential pressure, Pc

$$Pc = \frac{2.32 * E}{(SDR)^3}$$

From Chart 25, Reference 1,
S_A = 536 psi
to find :
E = Modulus of Elasticity

E = 19,000 psi
SDR = 17

$$Pc = \frac{2.32 * E}{(SDR)^3}$$

Pc = 9.0 psi

Calculate the critical buckling pressure at the top of the pipe, Pcb

$$Pcb = 0.8 (E' \times Pc)^{1/2}$$

from Reference 1, for angular, granular materials with moderate compaction

E' = 3000 psi

Pcb = 131 psi

Calculate the Factor of Safety, SF
SF = Pcb ÷ Pt
SF = 2 **OK**

6" Diameter Pipe - Operational Loads
Calculate Stresses in 6" Dia. Pipe From Operational Loads
Depth of Gypsum over Operational Cover = 98 feet

Soil Modulus Required to Resist Buckling

$$E' = \frac{Pcb^2}{0.64 * Pc}$$

Assume Pcb = Pt

E' = 3000 psi

Apply Safety Factor (SF) to E'

SF = 2

E' min = SF x E'
E' min = 6000psi

Check Ring Deflection

$$\% \text{ Soil Strain} = \varepsilon_s = \frac{Pt * 100}{E'min}$$

$$\varepsilon_s = \frac{Pt \times 100}{E'min}$$

ε_s = 1.1%

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 98 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 = 98 feet

Objective: **To determine the pipe stresses for the perforated leachate collection pipes during operation.**

References: 1) Driscopipe Material (copies attached)

Design Data: Operational Cover:

Density	125	lb/ft^3	(assumed)
Thickness	1	ft	(measured from bottom of pipe)

Waste:

Density	89	lb/ft^3	
Thickness	98	ft	measured from top of operational cover

Final Cover

Density	125	lb/ft^3	(assumed)
Thickness	3.5	ft	(includes 18" on top of waste and 2' vegetative 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 = 9285 psf
= 64.5 psi

Static Load:

Ps = Operational Cover = depth of cover to top of pipe * density of cover
Ps = 8847 psf
Ps = 61.4 psi

Load from Equipment: (live load increased 50% for dynamic load)

8" Diameter Pipe - Operational Loads
Calculate Stresses in 8" Dia. Pipe From Operational Loads
Depth of Gypsum over Operational Cover = 98 feet

$$Plive = \frac{3 \times W \times H^3}{2 \times \pi \times R^5} \times 1.5$$

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 = 98.3333 ft
x = 0.0 ft
R = 98.3 ft

Load Factor = 1.5

Plive = 1 psf
= 0.01 psi

This is the load from 1 track. Add load for other track.

H = 98.3333 ft
x = 5.0 ft
R = 98.5 ft

Plive = 1.5 psf
0.0 psi

Ps + Plive = 61.46 psi

8" Diameter Pipe - Operational Loads
Calculate Stresses in 8" Dia. Pipe From Operational Loads
Depth of Gypsum over Operational Cover = 98 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 =		67.0 psi	

Wall Crushing Check for Operational Loads

SA = Compressive Strength

$$SA = \frac{SDR - 1}{2} * Pt$$

$$SF = \frac{1500}{SA}$$

Pt = 67.0 psi

SDR = 17

$$SA = \frac{SDR - 1}{2} * Pt$$

SA = 536 psi

Factor of safety, FS:

$$SF = \frac{1500}{SA}$$

= 2.8 **OK**

8" Diameter Pipe - Operational Loads
Calculate Stresses in 8" Dia. Pipe From Operational Loads
Depth of Gypsum over Operational Cover = 98 feet

Critical Collapse

Calculate the critical-collapse differential pressure, Pc

$$Pc = \frac{2.32 * E}{(SDR)^3}$$

From Chart 25, Reference 1,
S_A = 536 psi
to find :
E = Modulus of Elasticity

E = 19,000 psi
SDR = 17

$$Pc = \frac{2.32 * E}{(SDR)^3}$$

Pc = 9.0 psi

Calculate the critical buckling pressure at the top of the pipe, Pcb

$$Pcb = 0.8 (E' \times Pc)^{1/2}$$

from Reference 1, for angular, granular materials with moderate compaction

E' = 3000 psi

Pcb = 131 psi

Calculate the Factor of Safety, SF
SF = Pcb ÷ Pt
SF = 2 **OK**

8" Diameter Pipe - Operational Loads
Calculate Stresses in 8" Dia. Pipe From Operational Loads
Depth of Gypsum over Operational Cover = 98 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'

$$SF = 2$$

$$E' \text{ min} = SF \times E'$$

$$E' \text{ min} = 6000 \text{psi}$$

Check Ring Deflection

$$\% \text{ Soil Strain} = \varepsilon_s = \frac{Pt * 100}{E'_{min}}$$

$$\varepsilon_s = \frac{Pt \times 100}{E'min}$$

$$\varepsilon_s = 1.1\%$$

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 98 ft of gypsum over operational cover.

Stormwater Basin – Leak Detection System (LDS)

- Estimated Leakage Rate Through Primary Liner
- Head Loss Calculation for LDS Pump
- Determine Sump Size and Cycle Time for Pump

Belews Creek FGD Residue Landfill	**Job No.**	**046119**		
Design Sump for Leak Detection System	**Comp. By**	**WMM**	**Date**	**4/21/2006**
For Stormwater Basin	**Chkd. By**	DEB	**Date**	
	Sheet	1	**of**	—

Objective: **Calculate Estimated Leakage Rate Through Holes in Primary Liner**

References: **1) Geotecnical Aspects of Landfill Design and Construction, Qian, Koerner, Gray, 2002.**
2) *The Hydrologic Evaluation of Landfill Performance (HELP) Model User's Guide for Version 3, USEPA, September 1994.*

Calculate estimated leakage rate through holes in Primary Liner by using orifice equation.

Orifice Equation

Q = Ca(2gh)^0.5

Q = leakage, cm^3/sec
C = 0.6 , orifice coefficient
a = 0.1 cm^2, area of hole — Reference 1, Section 4.8.2
g = 981 cm/sec^2

Assume h:

h = 5.75 ft, head above liner — (assumed)

h = 146.05 cm, head above liner

Area of Pond = 1.3 acres

Calculate Leakage Rate per Hole

Q = 32.1 cm^3/sec

Q = 733.1 gal/day/hole

Calculate Estimated Leakage Rate for Entire Basin, Q_{total}

Assumed Number of holes/acre — (Reference 2)
4 holes/acre

For basin, use 5.2 holes

Q_{total} = 3812 gal/day

Q_{total} = 2932 gal/ac/day

Q_{total} = 2.6 gpm

Belews Creek FGD Residue Landfill	**Job No.**	**046119**		
Design Sump for Leak Detection System	**Comp. By**	**WMM**	**Date**	**3/21/2006**
For Stormwater Basin	**Chkd. By**	DEB	**Date**	4/19/06
	Sheet	1	**of**	—

Objective: **Determine Head Loss for Pump Used With Leak Detection System**

Reference **(1)** ***Geotechnical Aspects of Landfill Design and Construction*, Qian, Koerner, Gray, 2002.**

Calculate the Total Discharge Head for the pump:

TDH = Hs + Hf + Hm

Hs = static head, ft
Hf = friction head loss, ft
Hm = minor head loss ,t

Hs, static head = **15 ft** assume v = 1.41 ft/sec

763 ft
- 748 ft
15 ft

Hf, friction head loss:

$$Hf = \frac{3.022\, v^{1.85}\, L}{C^{1.85}\, D_i^{4.87}}$$

v = velocity, ft/sec
L = flow length, ft
C = Hazen Willaims coefficient
Di = inside diameter of pipe, ft

Length of pipe

side slope	30 ft	
pipe to sump	130 ft	(Maximum)

$$\mathbf{Hf = \frac{3.022\, v^{1.85}\, L}{C^{1.85}\, D_i^{1.165}}}$$

v =	1.41 ft/sec	Q = VA	
L =	160 ft		
C =	130	V =	Q/A
Di =	1.075 in	Q =	4 gpm
=	0.09 ft		0.01 cfs
		A =	0.01 ft^2
Hf =	**1.9 ft**	V =	1.41 ft/sec

Belews Creek FGD Residue Landfill	**Job No.**	**046119**		
Design Sump for Leak Detection System	**Comp. By**	**WMM**	**Date**	**3/21/2006**
For Stormwater Basin	**Chkd. By**	DEB	**Date**	4/19/06
	Sheet	2	**of**	—

Hm, minor losses = $\sum \frac{Ki\, v^2}{2g}$

		K, loss coefficient
90 deg	3	1.00
45 deg	2	0.70
Misc fittings	6	0.03
Outlet	1	1.00
	Sum	2.73

Hm = 0.08

Total Head Loss:

TDH = Hs + Hf + Hm
TDH = 17.0 ft

Belews Creek FGD Residue Landfill
Design Sump for Leak Detection System
For Stormwater Basin

Job No.	**046119**		
Comp. By	**WMM**	**Date**	**4/21/2006**
Chkd. By	DEJ	**Date**	4/19/06
Sheet	1	**of**	—

Objective: **Determine Size of Sump Required for Leak Detection System**

References: *1) ISCO pipe Handbook*

2) Leachator Pump Performance Data - Model LPS005MSTGXX005-05

Calculate Volume of Sump and Cycle Time for Filling and Discharge

The volume of the sump that is available for liquid is the total volume of the sump, minus the volume of the side slope riser pipe, the pump, and the gravel.

The sump volume is calculated by determining the gross volume of the sump, and subtracting the area of the pipe wall, the volume of the pump, and the space occupied by the gravel, and the multiplying the corrected area by the length of the sump.

Since the DENR regulations require the head on a liner to be limited to 12", this value will be used as the depth that the pump activates.

The side slope riser pipe containing the pump will have a 14" diameter. The volume loss due to the pipe will use entire cross section area of the 14" pipe, even though 12" is used as the activation depth.

Try the following dimensions for the sump:

Depth for activation, h =	1	ft
length of sump =	4	ft
width at base=	1.5	ft
Side Slope (H:1V) =	1	(1:1)

Assume:
sump is filled with #57 stone

Calculate Gross Volume of Sump (using 12" of liquid as depth)

V = 10 ft^3

Calculate Volume Due to Side Slope Riser Pipe Wall

Use 14.0 in SDR17 (Reference 1)

Calculate Area of Pipe (outside dia.)

OD pipe = 14 in
Aod = 1.07 ft^2

Calculate Area of Pipe (inside dia.)

ID pipe = 12.286 in
A id = 0.82 ft^2

Area lost due to pipe wall thickness:

Aod - Aid = Area lost
Area lost = 0.25 ft^2

Length of Side Side Slope Riser Pipe: (approximate, only use horizontal portion)

L pipe = 2.5 ft

Volume loss due to Side Slope Riser Pipe:

Volume loss pipe = 0.61 ft^3

Belews Creek FGD Residue Landfill
Design Sump for Leak Detection System
For Stormwater Basin

Job No.	**046119**		
Comp. By	**WMM**	**Date**	**4/21/2006**
Chkd. By	DEB	**Date**	4/12/06
Sheet	2	**of**	–

Calculate Volume Loss Due to Pump Volume

Length of pump =	19.8	in	(Reference 2)
diameter of pump =	3.8	in	
Volume loss pump =	0.13	ft^3	

Calculate Volume Loss Due to Gravel

Calculate volume loss to due gravel:

Volume occupied by gravel = gross volume - volume of side slope riser pipe

Volume occupied by gravel = 9.4 ft^3
(includes pore space)

The porosity, n, of this stone is = 0.4

The volume of solids (gravel) = (Volume occupied by gravel including pore space) * (1 - n)
= 5.6 ft^2

Calculate Volume Available for Liquid:
Volume Available = Gross Volume of Sump - volume of pipe- volume of pump - volume of gravel

Gross Volume of Sump =	10	ft^3
Volume of pipe =	0.25	ft^3
Volume of pump =	0.13	ft^3
Volume of gravel =	5.6	ft^3
Volume availble for liquid =	4.12	ft^2

Assume flow rate from leakage calculations:
Q in = 3.0 gpm

Calculate time to fill sump volume:
V = 4.1 ft^3
V = 30.8 gal

$$\text{Fill Time} = \frac{\text{volume}}{\text{Q in}}$$

$$\text{Fill Time} = \frac{30.8 \text{ gal}}{3.0 \text{ gpm}}$$

Fill Time = 10.3 minutes

Belews Creek FGD Residue Landfill
Design Sump for Leak Detection System
For Stormwater Basin

Job No.	**046119**		
Comp. By	**WMM**	**Date**	**4/21/2006**
Chkd. By	DEB	**Date**	4/19/06
Sheet	3	**of**	–

Calculate Time to Empty Sump with Pump:

Use pump flow rate of:

Q pump = 5.5 gpm (Reference 2)

Assume TDH is approx. 50 feet.

$$\text{Empty Time} = \frac{\text{volume}}{(\text{Qpump} - \text{Q in})}$$

$$\text{Empty Time} = \frac{30.8 \text{ gal}}{2.5 \text{ gpm}}$$

Empty Time = 12.3 minutes

Calculate Cycle Time

Fill Time	10.3	minutes
+ Empty Time	12.3	minutes
Cycle Time	22.6	minutes

This will provide appoximately 3 cycles per hour at the 3 gpm inflow rate.

HDPE Pipe

Typical IPS HDPE Pipe Sizes

Pipe Size Nominal	Actual OD	Dimension Ratio	Pressure rating, psi @73°F	Minimum Wall	Average ID	Weight Lbs./ft
10"	10.750"	DR 7	267	1.536"	7.555"	19.314
10"	10.750"	DR 9	200	1.194"	8.266"	15.618
10"	**10.750"**	**DR 11**	**160**	**0.977"**	**8.718"**	**13.089**
10"	10.750"	DR 13.5	128	0.796"	9.094"	10.875
10"	10.750"	DR 15.5	110	0.694"	9.306"	9.576
10"	**10.750"**	**DR 17**	**100**	**0.632"**	**9.435"**	**8.788**
10"	10.750"	DR 19	89	0.566"	9.570"	7.918
10"	10.750"	DR 21	80	0.512"	9.685"	7.204
10"	10.750"	DR 26	64	0.413"	9.891"	5.878
10"	10.750"	DR 32.5	51	0.331"	10.062"	4.742
12"	12.750"	DR 7	267	1.821"	8.962"	27.17
12"	12.750"	DR 9	200	1.417"	9.803"	21.97
12"	**12.750"**	**DR 11**	**160**	**1.150"**	**10.339"**	**18.412**
12"	12.750"	DR 13.5	128	0.944"	10.786"	15.298
12"	12.750"	DR 15.5	110	0.823"	11.038"	13.471
12"	**12.750"**	**DR 17**	**100**	**0.750"**	**11.190"**	**12.362**
12"	12.750"	DR 19	89	0.671"	11.350"	11.138
12"	12.750"	DR 21	80	0.607"	11.487"	10.134
12"	12.750"	DR 26	64	0.490"	11.731"	8.269
12"	12.750"	DR 32.5	51	0.392"	11.935"	6.671
14"	14.000"	DR 7	267	2.000"	9.840"	32.758
14"	14.000"	DR 9	200	1.556"	10.764"	26.489
14"	**14.000"**	**DR 11**	**160**	**1.273"**	**11.352"**	**22.199**
14"	14.000"	DR 13.5	128	1.037"	11.843"	18.445
14"	14.000"	DR 15.5	110	0.903"	12.122"	16.242
14"	**14.000"**	**DR 17**	**100**	**0.824"**	**12.286"**	**14.905**
14"	14.000"	DR 19	89	0.737"	12.470"	13.429
14"	14.000"	DR 21	80	0.667"	12.613"	12.218
14"	14.000"	DR 26	64	0.538"	12.881"	9.97
14"	**14.000"**	**DR 32.5**	**51**	**0.431"**	**13.104"**	**8.044**
16"	16.000"	DR 7	267	2.286"	11.245"	42.786
16"	16.000"	DR 9	200	1.778"	12.302"	34.598
16"	**16.000"**	**DR 11**	**160**	**1.455"**	**12.974"**	**28.994**
16"	16.000"	DR 13.5	128	1.185"	13.535"	24.092
16"	16.000"	DR 15.5	110	1.032"	13.853"	21.214
16"	**16.000"**	**DR 17**	**100**	**0.941"**	**14.043"**	**19.467**
16"	16.000"	DR 19	89	0.842"	14.250"	17.540
16"	16.000"	DR 21	80	0.762"	14.415"	15.959
16"	16.000"	DR 26	64	0.615"	14.721"	13.022
16"	**16.000"**	**DR 32.5**	**51**	**0.492"**	**14.977"**	**10.506**

***Blue Bold indicates standard/more readily available items.**

1-800-345-ISCO

www.isco-pipe.com

© Copyright 2005 ISCO Industries, LLC. All Rights Reserved.

ISCO

Reference 1

Bill Miller

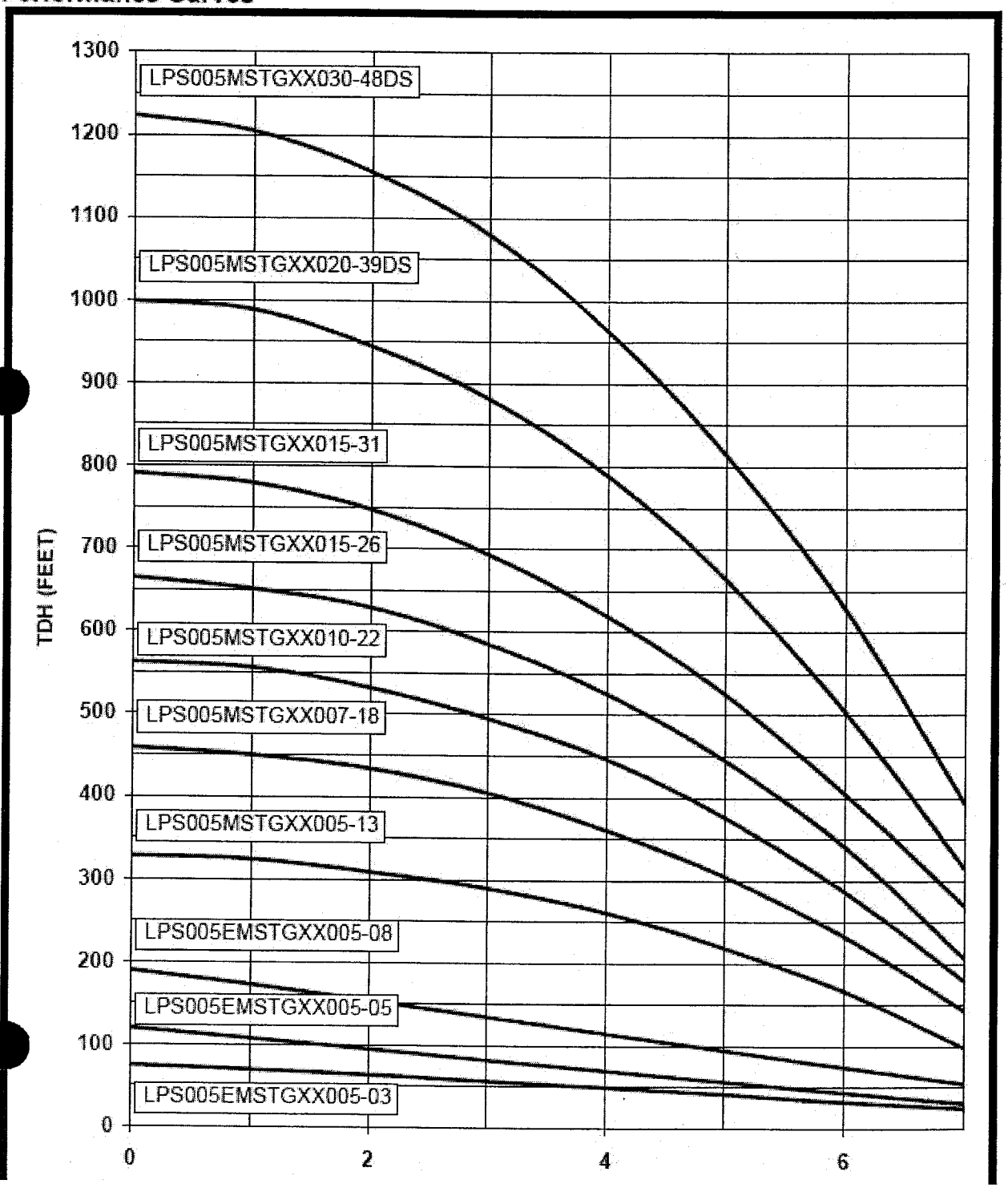
From: Fred Tolar [fptolar@mindspring.com]
Sent: Thursday, March 02, 2006 4:50 PM
To: 'Bill Miller'

LEACHATOR™ **LPSMSTG - SUBMERSIBLE PUMPS**

MODEL LPS005MSTG

PUMP PERFORMANCE DATA

Performance Curves



Reference 2

LEACHATOR™ LPSMSTG - SUBMERSIBLE PUMPS

Models

Model Number	Motor Size	HP	RPM	Voltage
LPS005EMSTG1X005-03	4"	1/2	3450	115-230V/1Ph/60Hx
LPS005EMSTG3X005-03	4"	1/2	3450	230-460V/3Ph/60Hz
LPS005EMSTG1X005-05	4"	1/2	3450	115-230V/1Ph/60Hz
LPS005EMSTG3X005-05	4"	1/2	3450	230-460V/3Ph/60Hz
LPS005EMSTG1X005-08	4"	1/2	3450	115-230V/1Ph/60Hz
LPS005EMSTG3X005-08	4"	1/2	3450	230-460V/3Ph/60Hz
LPS005MSTG1X005-13	4"	1/2	3450	115-230V/1Ph/60Hz
LPS005MSTG3X005-13	4"	1/2	3450	230-460V/3Ph/60Hz
LPS005MSTG12007-18	4"	3/4	3450	230V/1Ph/60Hz
LPS005MSTG3X007-18	4"	3/4	3450	230-460V/3Ph/60Hz
LPS005MSTG12010-22	4"	1	3450	230V/1Ph/60Hz
LPS005MSTG3X010-22	4"	1	3450	230-460V/3Ph/60Hz
LPS005MSTG12010-26	4"	1	3450	230V/1Ph/60Hz
LPS005MSTG3X010-26	4"	1	3450	230-460V/3Ph/60Hz
LPS005MSTG12015-31	4"	1-1/2	3450	230V/1Ph/60Hz
LPS005MSTG3X015-31	4"	1-1/2	3450	230-460V/3Ph/60Hz
LPS005MSTG12020-39DS	4"	2	3450	230V/1Ph/60Hz
LPS005MSTG3X020-39DS	4"	2	3450	230-460V/3Ph/60Hz
LPS005MSTG12030-48DS	4"	3	3450	230V/1Ph/60Hz
LPS005MSTG3X030-48DS	4"	3	3450	230-460V/3Ph/60Hz

Please insert a "1" (115V), "2" (230V) or "4" (460V) to complete the model number.

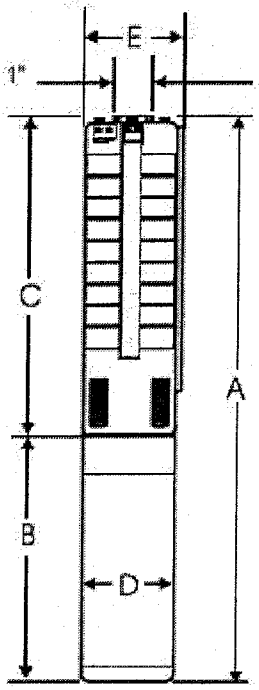
Dimensions And Weights

Model Number	Disch Size	Dimensions In Inches					Approx. Wt
		A	B	C	D	E	
LPS005EMSTGXX005-03	1" NPT	18.0	9.5	8.5	3.8	3.9	27
LPS005EMSTGXX005-05	1" NPT	19.8	9.5	10.3	3.8	3.9	27
LPS005EMSTGXX005-08	1" NPT	22.3	9.5	12.8	3.8	3.9	27
LPS005MSTGXX005-13	1" NPT	26.4	9.5	16.9	3.8	3.9	31
LPS005MSTGXX007-18	1" NPT	31.7	10.7	21.0	3.8	3.9	34
LPS005MSTGXX010-22	1" NPT	36.1	11.8	24.3	3.8	3.9	42
LPS005MSTGXX015-26	1" NPT	41.2	13.6	27.6	3.8	3.9	46
LPS005MSTGXX015-31	1" NPT	47.1	13.6	33.5	3.8	3.9	58
LPS005MSTGXX020-39DS	1" NPT	55.2	15.1	40.1	3.8	3.9	65
LPS005MSTGXX030-48DS	1" NPT	70.0	20.8	45.8	3.8	3.9	90

Note(s): *Dimensions and weight are approximate (subject to change) and does not include the carriage.*

Materials Of Construction

Impeller	304 Stainless Steel
Impeller Seal Ring	Teflon
Motor Adapter	304 Stainless Steel
Inlet Screen	304 Stainless Steel
Pump Shaft	431 Stainless Steel
Coupling	329/416 Stainless Steel
Check Valve Housing	304 Stainless Steel
Check Valve	304 Stainless Steel
Check Valve Seat	Teflon/316 Stainless Steel
Diffuser Chamber	304 Stainless Steel
Fasteners	304 Stainless Steel
Bearing	Teflon/316 Stainless Steel
Suspension Cables	304 Stainless Steel



Reference 2

Belews Creek FGD Residue Landfill
Design Sump for Leak Detection System
For Stormwater Basin

Job No.	**046119**		
Comp. By	**WMM**	**Date**	**4/5/2006**
Chkd. By	DEB	**Date**	4/21/06
Sheet	1	**of**	2

Objective: **Determine Number of Holes in Leachate Sump Riser To Drain 25 Year - 24 Hour Storm Determine Barrel and Riser Size, Set Crest if Riser**

Reference **(1) NC Sedimentation and Erosion Control Manual**

Calculate area of holes to dewater Cell in 24 hours:

$$Ao = \frac{As\ (2h)^{0.5}}{T\ Cd\ 20{,}428}$$

Reference 1, Page 8.07.8

Ao = area of holes, ft^2
As = surface area of basin,ft^2
Cd = orifice coeffcient (0.6)
h = head over hole, ft
T = time to dewater basin, hrs (24 hours)

	Cell 1	Cell 2	Cell 3	Cell 4
Peak Flow's (cfs)	**23.9**	**36.2**	**57.9**	**62.9**
Inlet Invert (Elev)	762.40	761.22	759.49	758.62
Elevation Operational Cover	763.40	762.22	760.49	759.62
Riser Diameter (in)	30	36	48	54
Crest of Riser	766.1	765.5	765.4	764.6
Elevation top of barrel	764.4	763.7	762.5	761.6
Height of Riser above Invert (ft)	3.7	4.3	5.9	6.0
Height of Riser above Top of Barre	1.7	1.8	2.9	3.0
As, Area in Cell at Crest of Riser (f	74,927	58,237	66,691	55,430
h, use 1/2 distance between Operational Cover and Crest of Riser	1.4	1.6	2.5	2.5
Ao, area of holes required, ft^2 =	0.42	0.36	0.50	0.42
hole diameter, in	0.75	0.75	0.75	0.75
area of hole , ft^2	0.0031	0.0031	0.0031	0.0031
number of holes	136.4	116.9	163.7	137.3
Riser Circumference, in	94	113	151	170

Belews Creek FGD Residue Landfill
Design Sump for Leak Detection System
For Stormwater Basin

Job No.	**046119**		
Comp. By	**WMM**	**Date**	**4/5/2006**
Chkd. By	DEB	**Date**	4/21/06
Sheet	2	**of**	2

Objective:

Determine Number of Holes in Leachate Sump Riser
To Drain 25 Year - 24 Hour Storm
Determine Barrel and Riser Size, Set Crest if Riser

Number of holes per foot, 4" on center	4	4	4	4
Number of holes per row	31.4	37.7	50.3	56.5
Number of rows required	**4.3**	**3.1**	**3.3**	**2.4**

Drawing CP-5 shows 6 rows of holes at 4" center-to-center.
This spacing is adequate to drain cell.

Stormwater Management

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

STORMWATER BASIN DETENTION CALCULATIONS

FOR

BELEWS CREEK GYPSUM LANDFILL

Project Number: 046121

Prepared For:

DUKE ENERGY

CHAS. H. SELLS, INC.

Consulting Engineers, Surveyors & Photogrammetrists

128 Overhill Drive, Suite 105, Mooresville, NC 28117

Phone: (704) 662-0100

Fax: (704) 662-0101

NORTH CAROLINA PROFESSIONAL ENGINEER SEAL 17582 DAN E. BREWER

4/19/06

STORMWATER PEAK RUNOFF SUMMARY
STORMWATER BASIN TO SERVE
BELEW'S CREEK FGD GYPSUM LANDFILL

PROJECT NO. 046119

Design Storm	Runoff Into Pond Cells 1&2 (cfs)	Outflow From Pond Riser Structure (cfs)	Outflow From Pond Riser Structure (GPM)
2 year - 24 hour	58.7	0.9	391
25 year - 24 hour	121.7	1.1	503
100 year - 24 hour	150.0	2.6	1158

SET PUMP CAPACITY = 500 GPM TO MEET 25 YEAR STORM EVENT

Table of Contents

********** MASTER SUMMARY **********

Watershed..... Master Network Summary 1.01

********** NETWORK SUMMARIES (DETAILED) **********

Watershed..... 25
Executive Summary (Nodes) 2.01

Watershed..... 100
Executive Summary (Nodes) 2.02
Network Calcs Sequence 2.03

Watershed..... 2YR
Executive Summary (Nodes) 2.05

********** DESIGN STORMS SUMMARY **********

CMLDS-24HR..... Design Storms 3.01

CMLDS-24HR..... 2YR
Design Storms 3.02

********** TC CALCULATIONS **********

SUBAREA 10..... Tc Calcs 4.01

********** CN CALCULATIONS **********

SUBAREA 10..... Runoff CN-Area 5.01

Table of Contents (continued)

********** RUNOFF HYDROGRAPHS **********

Unit Hyd. Equations 6.01

SUBAREA 10..... 25
Unit Hyd. Summary 6.03

SUBAREA 10..... 100
Unit Hyd. Summary 6.04

SUBAREA 10..... 2YR
Unit Hyd. Summary 6.05

********** POND VOLUMES **********

POND 10..... Vol: Elev-Area 7.01

********** OUTLET STRUCTURES **********

Outlet 1..... Outlet Input Data 8.01
Individual Outlet Curves 8.04
Composite Rating Curve 8.22

********** POND ROUTING **********

POND 10..... Pond E-V-Q Table 9.01

POND 10 IN 25
Node: Pond Inflow Summary 9.07

POND 10 IN 100
Node: Pond Inflow Summary 9.11

Table of Contents (continued)

POND 10 IN 2YR
Node: Pond Inflow Summary 9.15

POND 10 OUT 25
Pond Routing Summary 9.19

POND 10 OUT 100
Pond Routing Summary 9.20

POND 10 OUT 2YR
Pond Routing Summary 9.21

MASTER DESIGN STORM SUMMARY

Network Storm Collection: CMLDS-24HR

Return Event	Total Depth in	Rainfall Type	RNF ID
2YR	3.1200	Synthetic Curve	TypeII 24hr
25	5.7600	Synthetic Curve	TypeII 24hr
100	6.9600	Synthetic Curve	TypeII 24hr

MASTER NETWORK SUMMARY
SCS Unit Hydrograph Method

(*Node=Outfall; +Node=Diversion;)
(Trun= HYG Truncation: Blank=None; L=Left; R=Rt; LR=Left&Rt)

Node ID		Type	Return Event	HYG Vol ac-ft	Trun	Qpeak hrs	Qpeak cfs	Max WSEL ft	Max Pond Storage ac-ft
*OUT 10		JCT	2	3.384		17.2500	.87		
*OUT 10		JCT	25	7.060		23.0000	1.12		
*OUT 10		JCT	100	8.362		17.2000	2.58		
POND 10	IN	POND	2	3.384		11.9500	58.73		
POND 10	IN	POND	25	7.320		11.9500	121.73		
POND 10	IN	POND	100	9.144		11.9500	150.01		
POND 10	OUT	POND	2	3.384		17.2500	.87	754.84	2.455
POND 10	OUT	POND	25	7.060		23.0000	1.12	757.74	5.930
POND 10	OUT	POND	100	8.362		17.2000	2.58	758.58	7.081
SUBAREA 10		AREA	2	3.384		11.9500	58.73		
SUBAREA 10		AREA	25	7.320		11.9500	121.73		
SUBAREA 10		AREA	100	9.144		11.9500	150.01		

NETWORK SUMMARY -- NODES
(Trun.= HYG Truncation: Blank=None; L=Left; R=Rt; LR=Left & Rt)

DEFAULT Design Storm File,ID = CMLDS-24HR

Storm Tag Name = 25

Data Type, File, ID = Synthetic Storm TypeII 24hr
Storm Frequency = 25 yr
Total Rainfall Depth= 5.7600 in
Duration Multiplier = 1
Resulting Duration = 24.0000 hrs
Resulting Start Time= .0000 hrs Step= .1000 hrs End= 24.0000 hrs

	Node ID		Type	HYG Vol ac-ft	Trun.	Qpeak hrs	Qpeak cfs	Max WSEL ft
Outfall	OUT 10		JCT	7.060		23.0000	1.12	
	POND 10	IN	POND	7.320		11.9500	121.73	
	POND 10	OUT	POND	7.060		23.0000	1.12	757.74
	SUBAREA 10		AREA	7.320		11.9500	121.73	

NETWORK SUMMARY -- NODES
(Trun.= HYG Truncation: Blank=None; L=Left; R=Rt; LR=Left & Rt)

DEFAULT Design Storm File,ID = CMLDS-24HR

Storm Tag Name = 100

Data Type, File, ID = Synthetic Storm TypeII 24hr
Storm Frequency = 100 yr
Total Rainfall Depth= 6.9600 in
Duration Multiplier = 1
Resulting Duration = 24.0000 hrs
Resulting Start Time= .0000 hrs Step= .1000 hrs End= 24.0000 hrs

	Node ID		Type	HYG Vol ac-ft	Trun.	Qpeak hrs	Qpeak cfs	Max WSEL ft
Outfall	OUT 10		JCT	8.362		17.2000	2.58	
	POND 10	IN	POND	9.144		11.9500	150.01	
	POND 10	OUT	POND	8.362		17.2000	2.58	758.58
	SUBAREA 10		AREA	9.144		11.9500	150.01	

NETWORK RUNOFF NODE SEQUENCE

Runoff Data	Apply to Node	Receiving Link
SCS UH SUBAREA 10	Subarea SUBAREA 10	Add Hyd SUBAREA 10

NETWORK ROUTING SEQUENCE

Link Operation	UPstream Node	DNstream Node
Add Hyd ADDLINK 10	Subarea SUBAREA 10	Pond POND 10 IN
POND ROUTE TOTAL OUTFLOW...		
Total Pond Outflow	Pond POND 10 IN	Outflow POND 10 OUT
SET POND ROUTING LINK TO TOTAL POND OUTFLOW...		
Outlet ROUTE 10	Outflow POND 10 OUT	Jct OUT 10

NETWORK SUMMARY -- NODES
(Trun.= HYG Truncation: Blank=None; L=Left; R=Rt; LR=Left & Rt)

DEFAULT Design Storm File,ID = CMLDS-24HR

Storm Tag Name = 2YR

Data Type, File, ID = Synthetic Storm TypeII 24hr
Storm Frequency = 2 yr
Total Rainfall Depth= 3.1200 in
Duration Multiplier = 1
Resulting Duration = 24.0000 hrs
Resulting Start Time= .0000 hrs Step= .1000 hrs End= 24.0000 hrs

	Node ID		Type	HYG Vol ac-ft	Trun.	Qpeak hrs	Qpeak cfs	Max WSEL ft
Outfall	OUT 10		JCT	3.384		17.2500	.87	
	POND 10	IN	POND	3.384		11.9500	58.73	
	POND 10	OUT	POND	3.384		17.2500	.87	754.84
	SUBAREA 10		AREA	3.384		11.9500	58.73	

File.... J:\PROJECT\2004\046119BelewsCreek\Pond Files\
Title... Project Date: 8/4/2004
Project Engineer: Chas H. Sells
Project Title: Belew's Creek Landfill
Project Comments:

DESIGN STORMS SUMMARY

Design Storm File,ID = CMLDS-24HR

Storm Tag Name = 2YR

Data Type, File, ID = Synthetic Storm TypeII 24hr
Storm Frequency = 2 yr
Total Rainfall Depth= 3.1200 in
Duration Multiplier = 1
Resulting Duration = 24.0000 hrs
Resulting Start Time= .0000 hrs Step= .1000 hrs End= 24.0000 hrs

Storm Tag Name = 25

Data Type, File, ID = Synthetic Storm TypeII 24hr
Storm Frequency = 25 yr
Total Rainfall Depth= 5.7600 in
Duration Multiplier = 1
Resulting Duration = 24.0000 hrs
Resulting Start Time= .0000 hrs Step= .1000 hrs End= 24.0000 hrs

Storm Tag Name = 100

Data Type, File, ID = Synthetic Storm TypeII 24hr
Storm Frequency = 100 yr
Total Rainfall Depth= 6.9600 in
Duration Multiplier = 1
Resulting Duration = 24.0000 hrs
Resulting Start Time= .0000 hrs Step= .1000 hrs End= 24.0000 hrs

DESIGN STORMS SUMMARY

Design Storm File,ID = CMLDS-24HR

Storm Tag Name = 2YR

Data Type, File, ID = Synthetic Storm TypeII 24hr
Storm Frequency = 2 yr
Total Rainfall Depth= 3.1200 in
Duration Multiplier = 1
Resulting Duration = 24.0000 hrs
Resulting Start Time= .0000 hrs Step= .1000 hrs End= 24.0000 hrs

Storm Tag Name = 25

Data Type, File, ID = Synthetic Storm TypeII 24hr
Storm Frequency = 25 yr
Total Rainfall Depth= 5.7600 in
Duration Multiplier = 1
Resulting Duration = 24.0000 hrs
Resulting Start Time= .0000 hrs Step= .1000 hrs End= 24.0000 hrs

Storm Tag Name = 100

Data Type, File, ID = Synthetic Storm TypeII 24hr
Storm Frequency = 100 yr
Total Rainfall Depth= 6.9600 in
Duration Multiplier = 1
Resulting Duration = 24.0000 hrs
Resulting Start Time= .0000 hrs Step= .1000 hrs End= 24.0000 hrs

File.... J:\PROJECT\2004\046119BelewsCreek\Pond Files\POND46119-4-06.PPW

::
TIME OF CONCENTRATION CALCULATOR
::

Segment #1: Tc: TR-55 Sheet

Mannings n	.0110	
Hydraulic Length	110.00	ft
2yr, 24hr P	3.1200	in
Slope	.330000	ft/ft
Avg.Velocity	4.25	ft/sec

Segment #1 Time: .0072 hrs

Segment #2: Tc: Length & Vel.

Hydraulic Length	1551.00	ft
Avg.Velocity	4.00	ft/sec

Segment #2 Time: .1077 hrs

========================
Total Tc: .1149 hrs
========================

File.... J:\PROJECT\2004\046119BelewsCreek\Pond Files\POND46119-4-06.PPW

Tc Equations used...

==== SCS TR-55 Sheet Flow ==========

```
Tc = (.007 * ((n * Lf)**0.8)) / ((P**.5) * (Sf**.4))

Where:  Tc = Time of concentration, hrs
        n  = Mannings n
        Lf = Flow length, ft
        P  = 2yr, 24hr Rain depth, inches
        Sf = Slope, %
```

==== User Defined Length & Velocity ==========

```
Tc = (Lf / V) / (3600sec/hr)

Where:  Tc = Time of concentration, hrs
        Lf = Flow length, ft
        V  = Velocity, ft/sec
```

Name.... SUBAREA 10

File.... J:\PROJECT\2004\046119BelewsCreek\Pond Files\POND46119-4-06.PPW

RUNOFF CURVE NUMBER DATA

Soil/Surface Description	CN	Area acres	Impervious Adjustment %C	Impervious Adjustment %UC	Adjusted CN
Newly graded area (pervious only)	91	18.600			91.00
COMPOSITE AREA & WEIGHTED CN --->		18.600			91.00 (91)

SCS UNIT HYDROGRAPH METHOD
(Computational Notes)

DEFINITION OF TERMS: ----------

```
 At    = Total area (acres):  At = Ai+Ap
 Ai    = Impervious area (acres)
 Ap    = Pervious area (acres)
 CNi   = Runoff curve number for impervious area
 CNp   = Runoff curve number for pervious area
 fLoss = f loss constant infiltration (depth/time)
 gKs   = Saturated Hydraulic Conductivity (depth/time)
 Md    = Volumetric Moisture Deficit
 Psi   = Capillary Suction (length)
 hK    = Horton Infiltration Decay Rate (time^-1)
 fo    = Initial Infiltration Rate (depth/time)
 fc    = Ultimate(capacity)Infiltration Rate (depth/time)
 Ia    = Initial Abstraction (length)
 dt    = Computational increment (duration of unit excess rainfall)
         Default dt is smallest value of 0.1333Tc, rtm, and th
         (Smallest dt is then adjusted to match up with Tp)
UDdt   = User specified override computational main time increment
         (only used if UDdt is => .1333Tc)
 D(t)  = Point on distribution curve (fraction of P) for time step t

 K     = 2 / (1 + (Tr/Tp)):  default K = 0.75: (for Tr/Tp = 1.67)
 Ks    = Hydrograph shape factor
       = Unit Conversions * K:
       = ((1hr/3600sec) * (1ft/12in) * ((5280ft)**2/sq.mi)) * K
         Default Ks = 645.333 * 0.75 = 484

 Lag   = Lag time from center of excess runoff (dt) to Tp:  Lag = 0.6Tc
 P     = Total precipitation depth, inches
 Pa(t) = Accumulated rainfall at time step t
 Pi(t) = Incremental rainfall at time step t
 qp    = Peak discharge (cfs) for 1in. runoff, for 1hr, for 1 sq.mi.
       = (Ks * A * Q) / Tp   (where Q = 1in. runoff, A=sq.mi.)
 Qu(t) = Unit hydrograph ordinate (cfs) at time step t
 Q(t)  = Final hydrograph ordinate (cfs) at time step t
 Rai(t)= Accumulated runoff (inches) at time step t for impervious area
 Rap(t)= Accumulated runoff (inches) at time step t for pervious area
 Rii(t)= Incremental runoff (inches) at time step t for impervious area
 Rip(t)= Incremental runoff (inches) at time step t for pervious area
 R(t)  = Incremental weighted total runoff (inches)
 Rtm   = Time increment for rainfall table
 Si    = S for impervious area:  Si = (1000/CNi) - 10
 Sp    = S for pervious area:    Sp = (1000/CNp) - 10
 t     = Time step (row) number
 Tc    = Time of concentration
 Tb    = Time (hrs) of entire unit hydrograph:  Tb = Tp + Tr
 Tp    = Time (hrs) to peak of a unit hydrograph:  Tp = (dt/2) + Lag
 Tr    = Time (hrs) of receding limb of unit hydrograph:  Tr = ratio of Tp
```

SCS UNIT HYDROGRAPH METHOD
(Computational Notes)

```
PRECIPITATION: ------------------------------------------------------------
Column (1):  Time for time step t
Column (2):  D(t)  = Point on distribution curve for time step t
Column (3):  Pi(t) = Pa(t) - Pa(t-1): Col.(4) - Preceding Col.(4)
Column (4):  Pa(t) = D(t) x P:        Col.(2) x P


PERVIOUS AREA RUNOFF (using SCS Runoff CN Method) -----------------------
Column (5):  Rap(t) = Accumulated pervious runoff for time step t
             If (Pa(t) is <= 0.2Sp) then use: Rap(t) = 0.0
             If (Pa(t) is >  0.2Sp) then use:

             Rap(t) = (Col.(4)-0.2Sp)**2 / (Col.(4)+0.8Sp)

Column (6):  Rip(t) = Incremental pervious runoff for time step t
             Rip(t) =         Rap(t)          -         Rap(t-1)
             Rip(t) = Col.(5) for current row - Col.(5) for preceding row.

IMPERVIOUS AREA RUNOFF ---------------------------------------------------
Column (7 & 8)... Did not specify to use impervious areas.


INCREMENTAL WEIGHTED RUNOFF: ---------------------------------------------
Column (9):  R(t)  = (Ap/At) x Rip(t)   +  (Ai/At) x Rii(t)
             R(t)  = (Ap/At) x Col.(6)  +  (Ai/At) x Col.(8)


SCS UNIT HYDROGRAPH METHOD: ----------------------------------------------
Column (10): Q(t)   is computed with the SCS unit hydrograph method
                    using R() and Qu().
```

```
SCS UNIT HYDROGRAPH METHOD

STORM EVENT: 25 year storm
Duration      = 24.0000 hrs     Rain Depth = 5.7600 in
Rain Dir      = J:\PROJECT\2004\046119BelewsCreek\Pond Files\
Rain File -ID =  - TypeII  24hr
Unit Hyd Type = Default Curvilinear
HYG Dir       = J:\PROJECT\2004\046119BelewsCreek\Pond Files\
HYG File - ID =  - SUBAREA 10 25
Tc            = .1149 hrs
Drainage Area = 18.600 acres  Runoff CN= 91

=========================================
Computational Time Increment   =   .01532 hrs
Computed Peak Time             =  11.9343 hrs
Computed Peak Flow             =   122.98 cfs

Time Increment for HYG File    =    .0500 hrs
Peak Time, Interpolated Output =  11.9500 hrs
Peak Flow, Interpolated Output =   121.73 cfs
=========================================

                 DRAINAGE AREA
              -------------------
              ID:SUBAREA 10
              CN    =   91
              Area =      18.600 acres
              S     =    .9890 in
              0.2S =     .1978 in

               Cumulative Runoff
              -------------------
                      4.7225 in
                       7.320 ac-ft

HYG Volume...          7.320 ac-ft (area under HYG curve)

***** SCS UNIT HYDROGRAPH PARAMETERS *****

Time Concentration, Tc =   .11490 hrs (ID: SUBAREA 10)
Computational Incr, Tm =   .01532 hrs = 0.20000 Tp

Unit Hyd. Shape Factor =  483.432 (37.46% under rising limb)
K =  483.43/645.333,  K =    .7491 (also, K = 2/(1+(Tr/Tp))
Receding/Rising, Tr/Tp =   1.6698 (solved from K =   .7491)

Unit peak,          qp =   183.42 cfs
Unit peak time      Tp =   .07660 hrs
Unit receding limb, Tr =   .30640 hrs
Total unit time,    Tb =   .38300 hrs
```

```
SCS UNIT HYDROGRAPH METHOD

STORM EVENT: 100 year storm
Duration      = 24.0000 hrs     Rain Depth = 6.9600 in
Rain Dir      = J:\PROJECT\2004\046119BelewsCreek\Pond Files\
Rain File -ID =  - TypeII  24hr
Unit Hyd Type = Default Curvilinear
HYG Dir       = J:\PROJECT\2004\046119BelewsCreek\Pond Files\
HYG File - ID =  - SUBAREA 10 100
Tc            = .1149 hrs
Drainage Area = 18.600 acres  Runoff CN= 91

============================================
Computational Time Increment   =   .01532 hrs
Computed Peak Time             =  11.9343 hrs
Computed Peak Flow             =   151.68 cfs

Time Increment for HYG File    =    .0500 hrs
Peak Time, Interpolated Output =  11.9500 hrs
Peak Flow, Interpolated Output =   150.01 cfs
============================================

                 DRAINAGE AREA
              -------------------
              ID:SUBAREA 10
              CN    =   91
              Area =      18.600 acres
              S     =    .9890 in
              0.2S =    .1978 in

               Cumulative Runoff
              -------------------
                      5.8994 in
                       9.144 ac-ft

HYG Volume...          9.144 ac-ft (area under HYG curve)

***** SCS UNIT HYDROGRAPH PARAMETERS *****

Time Concentration, Tc =   .11490 hrs (ID: SUBAREA 10)
Computational Incr, Tm =   .01532 hrs = 0.20000 Tp

Unit Hyd. Shape Factor =  483.432 (37.46% under rising limb)
K =  483.43/645.333,  K =    .7491 (also, K = 2/(1+(Tr/Tp))
Receding/Rising, Tr/Tp =   1.6698 (solved from K =    .7491)

Unit peak,          qp =   183.42 cfs
Unit peak time      Tp =   .07660 hrs
Unit receding limb, Tr =   .30640 hrs
Total unit time,    Tb =   .38300 hrs
```

```
SCS UNIT HYDROGRAPH METHOD

STORM EVENT: 2 year storm
Duration      = 24.0000 hrs     Rain Depth = 3.1200 in
Rain Dir      = J:\PROJECT\2004\046119BelewsCreek\Pond Files\
Rain File -ID =  - TypeII  24hr
Unit Hyd Type = Default Curvilinear
HYG Dir       = J:\PROJECT\2004\046119BelewsCreek\Pond Files\
HYG File - ID =  - SUBAREA 10 2YR
Tc            = .1149 hrs
Drainage Area = 18.600 acres  Runoff CN= 91

==========================================
Computational Time Increment   =   .01532 hrs
Computed Peak Time             =  11.9343 hrs
Computed Peak Flow             =    59.03 cfs

Time Increment for HYG File    =    .0500 hrs
Peak Time, Interpolated Output =  11.9500 hrs
Peak Flow, Interpolated Output =    58.73 cfs
==========================================

                 DRAINAGE AREA
              -------------------
              ID:SUBAREA 10
              CN   =   91
              Area =      18.600 acres
              S    =    .9890 in
              0.2S =    .1978 in

               Cumulative Runoff
              -------------------
                      2.1833 in
                       3.384 ac-ft

HYG Volume...          3.384 ac-ft (area under HYG curve)

***** SCS UNIT HYDROGRAPH PARAMETERS *****

Time Concentration, Tc =   .11490 hrs (ID: SUBAREA 10)
Computational Incr, Tm =   .01532 hrs = 0.20000 Tp

Unit Hyd. Shape Factor =  483.432 (37.46% under rising limb)
K =  483.43/645.333,  K =    .7491 (also, K = 2/(1+(Tr/Tp))
Receding/Rising, Tr/Tp =   1.6698 (solved from K =   .7491)

Unit peak,          qp =   183.42 cfs
Unit peak time      Tp =   .07660 hrs
Unit receding limb, Tr =   .30640 hrs
Total unit time,    Tb =   .38300 hrs
```

File.... J:\PROJECT\2004\046119BelewsCreek\Pond Files\POND46119-4-06.PPW

Elevation (ft)	Planimeter (sq.in)	Area (sq.ft)	A1+A2+sqr(A1*A2) (sq.ft)	Volume (ac-ft)	Volume Sum (ac-ft)
750.00	-----	765	0	.000	.000
752.00	-----	18247	22748	.348	.348
754.00	-----	39631	84769	1.297	1.646
756.00	-----	51719	136623	2.091	3.736
758.00	-----	59298	166396	2.547	6.283
760.00	-----	67202	189626	2.902	9.185

POND VOLUME EQUATIONS

* Incremental volume computed by the Conic Method for Reservoir Volumes.

Volume = (1/3) * (EL2-EL1) * (Area1 + Area2 + sq.rt.(Area1*Area2))

where: EL1, EL2 = Lower and upper elevations of the increment
Area1,Area2 = Areas computed for EL1, EL2, respectively
Volume = Incremental volume between EL1 and EL2

File.... J:\PROJECT\2004\046119BelewsCreek\Pond Files\POND46119-4-06.PPW

REQUESTED POND WS ELEVATIONS:

Min. Elev.= 750.00 ft
Increment = .10 ft
Max. Elev.= 760.00 ft

****
OUTLET CONNECTIVITY
****

---> Forward Flow Only (UpStream to DnStream)
<--- Reverse Flow Only (DnStream to UpStream)
<---> Forward and Reverse Both Allowed

Structure	No.		Outfall	E1, ft	E2, ft
Stand Pipe	R1	--->	CV	758.400	760.000
Orifice-Circular	O1	--->	CV	750.000	760.000
Culvert-Circular	CV	--->	TW	750.000	760.000
TW SETUP, DS Channel					

File.... J:\PROJECT\2004\046119BelewsCreek\Pond Files\POND46119-4-06.PPW

OUTLET STRUCTURE INPUT DATA

```
Structure ID       = R1
Structure Type     = Stand Pipe
-----------------------------------
# of Openings      =         1
Invert Elev.       =    758.40 ft
Diameter           =    2.0000 ft
Orifice Area       =    3.1416 sq.ft
Orifice Coeff.     =      .600
Weir Length        =      6.28 ft
Weir Coeff.        =     3.000
K, Submerged       =      .000
K, Reverse         =     1.000
Kb,Barrel          =   .000000  (per ft of full flow)
Barrel Length      =       .00 ft
Mannings n         =     .0000
```

```
Structure ID       = O1
Structure Type     = Orifice-Circular
-----------------------------------
# of Openings      =         1
Invert Elev.       =    750.00 ft
Diameter           =     .3333 ft
Orifice Coeff.     =      .600
```

File.... J:\PROJECT\2004\046119BelewsCreek\Pond Files\POND46119-4-06.PPW

OUTLET STRUCTURE INPUT DATA

```
Structure ID       = CV
Structure Type     = Culvert-Circular
-----------------------------------
No. Barrels        =         1
Barrel Diameter    =    1.5000 ft
Upstream Invert    =    750.00 ft
Dnstream Invert    =    749.18 ft
Horiz. Length      =     82.00 ft
Barrel Length      =     82.00 ft
Barrel Slope       =    .01000 ft/ft

OUTLET CONTROL DATA...
Mannings n         =     .0240
Ke                 =     .2000  (forward entrance loss)
Kb                 =   .062076  (per ft of full flow)
Kr                 =     .2000  (reverse entrance loss)
HW Convergence     =      .001  +/- ft

INLET CONTROL DATA...
Equation form      =         1
Inlet Control K    =     .0078
Inlet Control M    =    2.0000
Inlet Control c    =    .03790
Inlet Control Y    =     .6900
T1 ratio (HW/D)    =     1.131
T2 ratio (HW/D)    =     1.291
Slope Factor       =     -.500
```

Use unsubmerged inlet control Form 1 equ. below T1 elev.
Use submerged inlet control Form 1 equ. above T2 elev.

In transition zone between unsubmerged and submerged inlet control,
interpolate between flows at T1 & T2...
At T1 Elev = 751.70 ft ---> Flow = 7.58 cfs
At T2 Elev = 751.94 ft ---> Flow = 8.66 cfs

```
Structure ID       = TW
Structure Type     = TW SETUP, DS Channel
-----------------------------------
FREE OUTFALL CONDITIONS SPECIFIED

CONVERGENCE TOLERANCES...
Maximum Iterations=    30
Min. TW tolerance =    .01 ft
Max. TW tolerance =    .01 ft
Min. HW tolerance =    .01 ft
Max. HW tolerance =    .01 ft
Min.  Q tolerance =    .10 cfs
Max.  Q tolerance =    .10 cfs
```

RATING TABLE FOR ONE OUTLET TYPE

Structure ID = R1 (Stand Pipe)

Upstream ID = (Pond Water Surface)
DNstream ID = CV (Culvert-Circular)

Pond WS. Elev. ft	Device Q cfs	(into) HW HGL ft	Converge DS HGL ft	Next DS HGL ft	DS HGL Error +/-ft	Q SUM Error +/-cfs	DS Chan. TW ft	TW Error +/-ft
750.00	.00	...	...	...	...	...	Free Outfall	
		WS below an invert; no flow.						
750.10	.00	...	...	...	...	...	Free Outfall	
		WS below an invert; no flow.						
750.20	.00	...	...	...	...	...	Free Outfall	
		WS below an invert; no flow.						
750.30	.00	...	...	...	...	...	Free Outfall	
		WS below an invert; no flow.						
750.40	.00	...	...	...	...	...	Free Outfall	
		WS below an invert; no flow.						
750.50	.00	...	...	...	...	...	Free Outfall	
		WS below an invert; no flow.						
750.60	.00	...	...	...	...	...	Free Outfall	
		WS below an invert; no flow.						
750.70	.00	...	...	...	...	...	Free Outfall	
		WS below an invert; no flow.						
750.80	.00	...	...	...	...	...	Free Outfall	
		WS below an invert; no flow.						
750.90	.00	...	...	...	...	...	Free Outfall	
		WS below an invert; no flow.						
751.00	.00	...	...	...	...	...	Free Outfall	
		WS below an invert; no flow.						
751.10	.00	...	...	...	...	...	Free Outfall	
		WS below an invert; no flow.						
751.20	.00	...	...	...	...	...	Free Outfall	
		WS below an invert; no flow.						
751.30	.00	...	...	...	...	...	Free Outfall	
		WS below an invert; no flow.						
751.40	.00	...	...	...	...	...	Free Outfall	
		WS below an invert; no flow.						
751.50	.00	...	...	...	...	...	Free Outfall	
		WS below an invert; no flow.						
751.60	.00	...	...	...	...	...	Free Outfall	
		WS below an invert; no flow.						
751.70	.00	...	...	...	...	...	Free Outfall	
		WS below an invert; no flow.						

RATING TABLE FOR ONE OUTLET TYPE

Structure ID = R1 (Stand Pipe)

Upstream ID = (Pond Water Surface)
DNstream ID = CV (Culvert-Circular)

Pond WS. Elev. ft	Device Q cfs	(into) HW HGL ft	Converge DS HGL ft	Next DS HGL ft	DS HGL Error +/-ft	Q SUM Error +/-cfs	DS Chan. TW ft	TW Error +/-ft
751.80	.00	...	...	...	...	...	Free Outfall	
		WS below an invert; no flow.						
751.90	.00	...	...	...	...	...	Free Outfall	
		WS below an invert; no flow.						
752.00	.00	...	...	...	...	...	Free Outfall	
		WS below an invert; no flow.						
752.10	.00	...	...	...	...	...	Free Outfall	
		WS below an invert; no flow.						
752.20	.00	...	...	...	...	...	Free Outfall	
		WS below an invert; no flow.						
752.30	.00	...	...	...	...	...	Free Outfall	
		WS below an invert; no flow.						
752.40	.00	...	...	...	...	...	Free Outfall	
		WS below an invert; no flow.						
752.50	.00	...	...	...	...	...	Free Outfall	
		WS below an invert; no flow.						
752.60	.00	...	...	...	...	...	Free Outfall	
		WS below an invert; no flow.						
752.70	.00	...	...	...	...	...	Free Outfall	
		WS below an invert; no flow.						
752.80	.00	...	...	...	...	...	Free Outfall	
		WS below an invert; no flow.						
752.90	.00	...	...	...	...	...	Free Outfall	
		WS below an invert; no flow.						
753.00	.00	...	...	...	...	...	Free Outfall	
		WS below an invert; no flow.						
753.10	.00	...	...	...	...	...	Free Outfall	
		WS below an invert; no flow.						
753.20	.00	...	...	...	...	...	Free Outfall	
		WS below an invert; no flow.						
753.30	.00	...	...	...	...	...	Free Outfall	
		WS below an invert; no flow.						
753.40	.00	...	...	...	...	...	Free Outfall	
		WS below an invert; no flow.						
753.50	.00	...	...	...	...	...	Free Outfall	
		WS below an invert; no flow.						

RATING TABLE FOR ONE OUTLET TYPE

Structure ID = R1 (Stand Pipe)

Upstream ID = (Pond Water Surface)
DNstream ID = CV (Culvert-Circular)

Pond WS. Elev. ft	Device Q cfs	(into) HW HGL ft	Converge DS HGL ft	Next DS HGL ft	DS HGL Error +/-ft	Q SUM Error +/-cfs	DS Chan. TW ft	TW Error +/-ft
753.60	.00	...	...	...	...	...	Free Outfall	
		WS below an invert; no flow.						
753.70	.00	...	...	...	...	...	Free Outfall	
		WS below an invert; no flow.						
753.80	.00	...	...	...	...	...	Free Outfall	
		WS below an invert; no flow.						
753.90	.00	...	...	...	...	...	Free Outfall	
		WS below an invert; no flow.						
754.00	.00	...	...	...	...	...	Free Outfall	
		WS below an invert; no flow.						
754.10	.00	...	...	...	...	...	Free Outfall	
		WS below an invert; no flow.						
754.20	.00	...	...	...	...	...	Free Outfall	
		WS below an invert; no flow.						
754.30	.00	...	...	...	...	...	Free Outfall	
		WS below an invert; no flow.						
754.40	.00	...	...	...	...	...	Free Outfall	
		WS below an invert; no flow.						
754.50	.00	...	...	...	...	...	Free Outfall	
		WS below an invert; no flow.						
754.60	.00	...	...	...	...	...	Free Outfall	
		WS below an invert; no flow.						
754.70	.00	...	...	...	...	...	Free Outfall	
		WS below an invert; no flow.						
754.80	.00	...	...	...	...	...	Free Outfall	
		WS below an invert; no flow.						
754.90	.00	...	...	...	...	...	Free Outfall	
		WS below an invert; no flow.						
755.00	.00	...	...	...	...	...	Free Outfall	
		WS below an invert; no flow.						
755.10	.00	...	...	...	...	...	Free Outfall	
		WS below an invert; no flow.						
755.20	.00	...	...	...	...	...	Free Outfall	
		WS below an invert; no flow.						
755.30	.00	...	...	...	...	...	Free Outfall	
		WS below an invert; no flow.						

File.... J:\PROJECT\2004\046119BelewsCreek\Pond Files\POND46119-4-06.PPW

RATING TABLE FOR ONE OUTLET TYPE

Structure ID = R1 (Stand Pipe)

Upstream ID = (Pond Water Surface)
DNstream ID = CV (Culvert-Circular)

Pond WS. Elev. ft	Device Q cfs	(into) HW HGL ft	Converge DS HGL ft	Next DS HGL ft	DS HGL Error +/-ft	Q SUM Error +/-cfs	DS Chan. TW ft	TW Error +/-ft
755.40	.00	...	...	...	...	...	Free Outfall	
		WS below an invert; no flow.						
755.50	.00	...	...	...	...	...	Free Outfall	
		WS below an invert; no flow.						
755.60	.00	...	...	...	...	...	Free Outfall	
		WS below an invert; no flow.						
755.70	.00	...	...	...	...	...	Free Outfall	
		WS below an invert; no flow.						
755.80	.00	...	...	...	...	...	Free Outfall	
		WS below an invert; no flow.						
755.90	.00	...	...	...	...	...	Free Outfall	
		WS below an invert; no flow.						
756.00	.00	...	...	...	...	...	Free Outfall	
		WS below an invert; no flow.						
756.10	.00	...	...	...	...	...	Free Outfall	
		WS below an invert; no flow.						
756.20	.00	...	...	...	...	...	Free Outfall	
		WS below an invert; no flow.						
756.30	.00	...	...	...	...	...	Free Outfall	
		WS below an invert; no flow.						
756.40	.00	...	...	...	...	...	Free Outfall	
		WS below an invert; no flow.						
756.50	.00	...	...	...	...	...	Free Outfall	
		WS below an invert; no flow.						
756.60	.00	...	...	...	...	...	Free Outfall	
		WS below an invert; no flow.						
756.70	.00	...	...	...	...	...	Free Outfall	
		WS below an invert; no flow.						
756.80	.00	...	...	...	...	...	Free Outfall	
		WS below an invert; no flow.						
756.90	.00	...	...	...	...	...	Free Outfall	
		WS below an invert; no flow.						
757.00	.00	...	...	...	...	...	Free Outfall	
		WS below an invert; no flow.						
757.10	.00	...	...	...	...	...	Free Outfall	
		WS below an invert; no flow.						

File.... J:\PROJECT\2004\046119BelewsCreek\Pond Files\POND46119-4-06.PPW

RATING TABLE FOR ONE OUTLET TYPE

Structure ID = R1 (Stand Pipe)

Upstream ID = (Pond Water Surface)
DNstream ID = CV (Culvert-Circular)

Pond WS. Elev. ft	Device Q cfs	(into) HW HGL ft	Converge DS HGL ft	Next DS HGL ft	DS HGL Error +/-ft	Q SUM Error +/-cfs	DS Chan. TW ft	TW Error +/-ft
757.20	.00	...	...	...	...	...	Free Outfall	
		WS below an invert; no flow.						
757.30	.00	...	...	...	...	...	Free Outfall	
		WS below an invert; no flow.						
757.40	.00	...	...	...	...	...	Free Outfall	
		WS below an invert; no flow.						
757.50	.00	...	...	...	...	...	Free Outfall	
		WS below an invert; no flow.						
757.60	.00	...	...	...	...	...	Free Outfall	
		WS below an invert; no flow.						
757.70	.00	...	...	...	...	...	Free Outfall	
		WS below an invert; no flow.						
757.80	.00	...	...	...	...	...	Free Outfall	
		WS below an invert; no flow.						
757.90	.00	...	...	...	...	...	Free Outfall	
		WS below an invert; no flow.						
758.00	.00	...	...	...	...	...	Free Outfall	
		WS below an invert; no flow.						
758.10	.00	...	...	...	...	...	Free Outfall	
		WS below an invert; no flow.						
758.20	.00	...	...	...	...	...	Free Outfall	
		WS below an invert; no flow.						
758.30	.00	...	...	...	...	...	Free Outfall	
		WS below an invert; no flow.						
758.40	.00	...	...	...	...	...	Free Outfall	
		WS below an invert; no flow.						
758.50	.60	758.50	Free	750.72	.000	.000	Free Outfall	
		Weir: H =.10						
758.60	1.69	758.60	Free	750.94	.000	.000	Free Outfall	
		Weir: H =.20						
758.70	3.10	758.70	Free	751.20	.000	.000	Free Outfall	
		Weir: H =.30						
758.80	4.77	758.80	Free	751.52	.000	.000	Free Outfall	
		Weir: H =.40						
758.90	6.66	758.90	Free	752.32	.000	.000	Free Outfall	
		Weir: H =.50						

RATING TABLE FOR ONE OUTLET TYPE

Structure ID = R1 (Stand Pipe)

Upstream ID = (Pond Water Surface)
DNstream ID = CV (Culvert-Circular)

Pond WS. Elev. ft	Device Q cfs	(into) HW HGL ft	Converge DS HGL ft	Next DS HGL ft	DS HGL Error +/-ft	Q SUM Error +/-cfs	DS Chan. TW ft	TW Error +/-ft
759.00	8.76	759.00	Free	753.50	.000	.000	Free Outfall	
		Weir: H =.60						
759.10	11.04	759.10	Free	755.02	.000	.000	Free Outfall	
		Weir: H =.70						
759.20	13.49	759.20	Free	756.88	.000	.000	Free Outfall	
		Weir: H =.80						
759.30	14.34	759.30	Free	757.59	.000	.000	Free Outfall	
		Orifice: H =.90						
759.40	15.12	759.40	Free	758.24	.000	.000	Free Outfall	
		Orifice: H =1.00						
759.50	16.19	759.50	759.50	759.50	.000	.000	Free Outfall	
		DS HGL+Loss > crest: Flow set to Downstream outlet.						
759.60	16.90	759.60	759.60	759.60	.000	.000	Free Outfall	
		DS HGL+Loss > crest: Flow set to Downstream outlet.						
759.70	16.99	759.70	759.70	759.70	.000	.000	Free Outfall	
		DS HGL+Loss > crest: Flow set to Downstream outlet.						
759.80	17.09	759.80	759.80	759.80	.000	.000	Free Outfall	
		DS HGL+Loss > crest: Flow set to Downstream outlet.						
759.90	17.18	759.90	759.90	759.90	.000	.000	Free Outfall	
		DS HGL+Loss > crest: Flow set to Downstream outlet.						
760.00	17.27	760.00	760.00	760.00	.000	.000	Free Outfall	
		DS HGL+Loss > crest: Flow set to Downstream outlet.						

File.... J:\PROJECT\2004\046119BelewsCreek\Pond Files\POND46119-4-06.PPW

RATING TABLE FOR ONE OUTLET TYPE

Structure ID = O1 (Orifice-Circular)

Upstream ID = (Pond Water Surface)
DNstream ID = CV (Culvert-Circular)

Pond WS. Elev. ft	Device Q cfs	(into) HW HGL ft	Converge DS HGL ft	Next DS HGL ft	DS HGL Error +/-ft	Q SUM Error +/-cfs	DS Chan. TW ft	TW Error +/-ft
750.00	.00	...	...	...	...	...	Free Outfall	
		WS below an invert; no flow.						
750.10	.02	750.10	750.07	750.07	.000	.000	Free Outfall	
		BACKWATER CONTROL.. Vh= .027ft hwDi= .073ft Lbw= .0ft						
750.20	.07	750.20	750.14	750.14	.000	.000	Free Outfall	
		CRIT.DEPTH CONTROL Vh= .054ft Dcr= .146ft CRIT.DEPTH						
750.30	.14	750.30	750.20	750.20	.000	.000	Free Outfall	
		CRIT.DEPTH CONTROL Vh= .092ft Dcr= .208ft CRIT.DEPTH						
750.40	.18	750.40	750.22	750.22	.006	.000	Free Outfall	
		H =.18						
750.50	.21	750.50	750.24	750.24	.006	.000	Free Outfall	
		H =.26						
750.60	.25	750.60	750.26	750.26	.004	.000	Free Outfall	
		H =.34						
750.70	.27	750.70	750.27	750.28	.002	.000	Free Outfall	
		H =.43						
750.80	.30	750.80	750.29	750.29	.003	.000	Free Outfall	
		H =.51						
750.90	.33	750.90	750.30	750.30	.002	.000	Free Outfall	
		H =.60						
751.00	.35	751.00	750.31	750.31	.001	.000	Free Outfall	
		H =.69						
751.10	.37	751.10	750.32	750.32	.001	.000	Free Outfall	
		H =.78						
751.20	.39	751.20	750.33	750.33	.002	.000	Free Outfall	
		H =.87						
751.30	.41	751.30	750.34	750.34	.002	.000	Free Outfall	
		H =.96						
751.40	.43	751.40	750.35	750.35	.001	.000	Free Outfall	
		H =1.05						
751.50	.45	751.50	750.35	750.35	.001	.000	Free Outfall	
		H =1.15						
751.60	.47	751.60	750.36	750.36	.001	.000	Free Outfall	
		H =1.24						
751.70	.48	751.70	750.37	750.37	.001	.000	Free Outfall	
		H =1.33						

File.... J:\PROJECT\2004\046119BelewsCreek\Pond Files\POND46119-4-06.PPW

RATING TABLE FOR ONE OUTLET TYPE

Structure ID = O1 (Orifice-Circular)

Upstream ID = (Pond Water Surface)
DNstream ID = CV (Culvert-Circular)

Pond WS. Elev. ft	Device Q cfs	(into) HW HGL ft	Converge DS HGL ft	Next DS HGL ft	DS HGL Error +/-ft	Q SUM Error +/-cfs	DS Chan. TW ft	TW Error +/-ft
751.80	.50	751.80	750.37	750.37	.002	.000	Free Outfall	
		H =1.43						
751.90	.52	751.90	750.38	750.38	.002	.000	Free Outfall	
		H =1.52						
752.00	.53	752.00	750.39	750.39	.002	.000	Free Outfall	
		H =1.61						
752.10	.55	752.10	750.39	750.39	.002	.000	Free Outfall	
		H =1.71						
752.20	.56	752.20	750.40	750.40	.002	.000	Free Outfall	
		H =1.80						
752.30	.58	752.30	750.40	750.40	.002	.000	Free Outfall	
		H =1.90						
752.40	.59	752.40	750.41	750.41	.000	.000	Free Outfall	
		H =1.99						
752.50	.61	752.50	750.41	750.41	.000	.000	Free Outfall	
		H =2.09						
752.60	.62	752.60	750.42	750.42	.000	.000	Free Outfall	
		H =2.18						
752.70	.63	752.70	750.42	750.42	.001	.000	Free Outfall	
		H =2.28						
752.80	.65	752.80	750.43	750.43	.000	.000	Free Outfall	
		H =2.37						
752.90	.66	752.90	750.43	750.43	.000	.000	Free Outfall	
		H =2.47						
753.00	.67	753.00	750.43	750.44	.002	.000	Free Outfall	
		H =2.57						
753.10	.69	753.10	750.44	750.44	.000	.000	Free Outfall	
		H =2.66						
753.20	.70	753.20	750.44	750.44	.001	.000	Free Outfall	
		H =2.76						
753.30	.71	753.30	750.45	750.45	.000	.000	Free Outfall	
		H =2.85						
753.40	.72	753.40	750.45	750.45	.000	.000	Free Outfall	
		H =2.95						
753.50	.73	753.50	750.45	750.45	.000	.000	Free Outfall	
		H =3.05						

File.... J:\PROJECT\2004\046119BelewsCreek\Pond Files\POND46119-4-06.PPW

RATING TABLE FOR ONE OUTLET TYPE

Structure ID = O1 (Orifice-Circular)

Upstream ID = (Pond Water Surface)
DNstream ID = CV (Culvert-Circular)

Pond WS. Elev. ft	Device Q cfs	(into) HW HGL ft	Converge DS HGL ft	Next DS HGL ft	DS HGL Error +/-ft	Q SUM Error +/-cfs	DS Chan. TW ft	TW Error +/-ft
753.60	.74	753.60	750.46	750.46	.001	.000	Free Outfall	
		H =3.14						
753.70	.76	753.70	750.46	750.46	.000	.000	Free Outfall	
		H =3.24						
753.80	.77	753.80	750.47	750.47	.001	.000	Free Outfall	
		H =3.33						
753.90	.78	753.90	750.47	750.47	.002	.000	Free Outfall	
		H =3.43						
754.00	.79	754.00	750.47	750.47	.001	.000	Free Outfall	
		H =3.53						
754.10	.80	754.10	750.48	750.48	.000	.000	Free Outfall	
		H =3.62						
754.20	.81	754.20	750.48	750.48	.001	.000	Free Outfall	
		H =3.72						
754.30	.82	754.30	750.48	750.48	.000	.000	Free Outfall	
		H =3.82						
754.40	.83	754.40	750.49	750.49	.001	.000	Free Outfall	
		H =3.91						
754.50	.84	754.50	750.49	750.49	.000	.000	Free Outfall	
		H =4.01						
754.60	.85	754.60	750.49	750.49	.000	.000	Free Outfall	
		H =4.11						
754.70	.86	754.70	750.49	750.50	.001	.000	Free Outfall	
		H =4.21						
754.80	.87	754.80	750.51	750.50	.010	.000	Free Outfall	
		H =4.29						
754.90	.88	754.90	750.50	750.50	.000	.000	Free Outfall	
		H =4.40						
755.00	.89	755.00	750.51	750.50	.009	.000	Free Outfall	
		H =4.49						
755.10	.90	755.10	750.52	750.51	.010	.000	Free Outfall	
		H =4.58						
755.20	.91	755.20	750.52	750.51	.010	.000	Free Outfall	
		H =4.68						
755.30	.92	755.30	750.52	750.51	.009	.000	Free Outfall	
		H =4.78						

File.... J:\PROJECT\2004\046119BelewsCreek\Pond Files\POND46119-4-06.PPW

RATING TABLE FOR ONE OUTLET TYPE

Structure ID = O1 (Orifice-Circular)

Upstream ID = (Pond Water Surface)
DNstream ID = CV (Culvert-Circular)

Pond WS. Elev. ft	Device Q cfs	(into) HW HGL ft	Converge DS HGL ft	Next DS HGL ft	DS HGL Error +/-ft	Q SUM Error +/-cfs	DS Chan. TW ft	TW Error +/-ft
755.40	.93	755.40	750.51	750.51	.000	.000	Free Outfall	
		H =4.89						
755.50	.94	755.50	750.53	750.52	.010	.000	Free Outfall	
		H =4.97						
755.60	.95	755.60	750.53	750.52	.009	.000	Free Outfall	
		H =5.07						
755.70	.96	755.70	750.52	750.52	.000	.000	Free Outfall	
		H =5.18						
755.80	.96	755.80	750.53	750.53	.009	.000	Free Outfall	
		H =5.27						
755.90	.97	755.90	750.54	750.53	.009	.000	Free Outfall	
		H =5.36						
756.00	.98	756.00	750.54	750.53	.008	.000	Free Outfall	
		H =5.46						
756.10	.99	756.10	750.54	750.53	.009	.000	Free Outfall	
		H =5.56						
756.20	1.00	756.20	750.54	750.54	.008	.000	Free Outfall	
		H =5.66						
756.30	1.01	756.30	750.55	750.54	.009	.000	Free Outfall	
		H =5.75						
756.40	1.02	756.40	750.55	750.54	.009	.000	Free Outfall	
		H =5.85						
756.50	1.02	756.50	750.55	750.54	.010	.000	Free Outfall	
		H =5.95						
756.60	1.03	756.60	750.55	750.54	.008	.000	Free Outfall	
		H =6.05						
756.70	1.04	756.70	750.56	750.55	.008	.000	Free Outfall	
		H =6.14						
756.80	1.05	756.80	750.56	750.55	.009	.000	Free Outfall	
		H =6.24						
756.90	1.06	756.90	750.56	750.55	.008	.000	Free Outfall	
		H =6.34						
757.00	1.07	757.00	750.56	750.55	.008	.000	Free Outfall	
		H =6.44						
757.10	1.07	757.10	750.56	750.56	.008	.000	Free Outfall	
		H =6.54						

File.... J:\PROJECT\2004\046119BelewsCreek\Pond Files\POND46119-4-06.PPW

RATING TABLE FOR ONE OUTLET TYPE

Structure ID = O1 (Orifice-Circular)

Upstream ID = (Pond Water Surface)
DNstream ID = CV (Culvert-Circular)

Pond WS. Elev. ft	Device Q cfs	(into) HW HGL ft	Converge DS HGL ft	Next DS HGL ft	DS HGL Error +/-ft	Q SUM Error +/-cfs	DS Chan. TW ft	TW Error +/-ft
757.20	1.08	757.20	750.57	750.56	.009	.000	Free Outfall	
		H =6.63						
757.30	1.09	757.30	750.57	750.56	.010	.000	Free Outfall	
		H =6.73						
757.40	1.10	757.40	750.57	750.56	.009	.000	Free Outfall	
		H =6.83						
757.50	1.11	757.50	750.57	750.56	.008	.000	Free Outfall	
		H =6.93						
757.60	1.11	757.60	750.57	750.57	.008	.000	Free Outfall	
		H =7.03						
757.70	1.12	757.70	750.58	750.57	.008	.000	Free Outfall	
		H =7.12						
757.80	1.13	757.80	750.58	750.57	.009	.000	Free Outfall	
		H =7.22						
757.90	1.14	757.90	750.58	750.57	.008	.000	Free Outfall	
		H =7.32						
758.00	1.14	758.00	750.58	750.57	.009	.000	Free Outfall	
		H =7.42						
758.10	1.15	758.10	750.58	750.58	.008	.000	Free Outfall	
		H =7.52						
758.20	1.16	758.20	750.59	750.58	.009	.000	Free Outfall	
		H =7.61						
758.30	1.17	758.30	750.59	750.58	.007	.000	Free Outfall	
		H =7.71						
758.40	1.17	758.40	750.59	750.58	.008	.000	Free Outfall	
		H =7.81						
758.50	1.17	758.50	750.73	750.72	.009	.000	Free Outfall	
		H =7.77						
758.60	1.16	758.60	750.94	750.94	.000	.000	Free Outfall	
		H =7.66						
758.70	1.15	758.70	751.20	751.20	.000	.000	Free Outfall	
		H =7.50						
758.80	1.13	758.80	751.52	751.52	.000	.000	Free Outfall	
		H =7.28						
758.90	1.08	758.90	752.31	752.32	.005	.000	Free Outfall	
		H =6.59						

File.... J:\PROJECT\2004\046119BelewsCreek\Pond Files\POND46119-4-06.PPW

RATING TABLE FOR ONE OUTLET TYPE

Structure ID = O1 (Orifice-Circular)

Upstream ID = (Pond Water Surface)
DNstream ID = CV (Culvert-Circular)

Pond WS. Elev. ft	Device Q cfs	(into) HW HGL ft	Converge DS HGL ft	Next DS HGL ft	DS HGL Error +/-ft	Q SUM Error +/-cfs	DS Chan. TW ft	TW Error +/-ft
759.00	.98	759.00	753.50	753.50	.001	.000	Free Outfall	
		H =5.50						
759.10	.85	759.10	755.02	755.02	.002	.000	Free Outfall	
		H =4.08						
759.20	.64	759.20	756.88	756.88	.001	.000	Free Outfall	
		H =2.32						
759.30	.55	759.30	757.58	757.59	.003	.000	Free Outfall	
		H =1.72						
759.40	.45	759.40	758.23	758.24	.008	.000	Free Outfall	
		H =1.17						
759.50	.00	759.50	759.50	759.50	.000	.000	Free Outfall	
				Full riser flow. Q=0 this opening.				
759.60	.00	759.60	759.60	759.60	.000	.000	Free Outfall	
				Full riser flow. Q=0 this opening.				
759.70	.00	759.70	759.70	759.70	.000	.000	Free Outfall	
				Full riser flow. Q=0 this opening.				
759.80	.00	759.80	759.80	759.80	.000	.000	Free Outfall	
				Full riser flow. Q=0 this opening.				
759.90	.00	759.90	759.90	759.90	.000	.000	Free Outfall	
				Full riser flow. Q=0 this opening.				
760.00	.00	760.00	760.00	760.00	.000	.000	Free Outfall	
				Full riser flow. Q=0 this opening.				

File.... J:\PROJECT\2004\046119BelewsCreek\Pond Files\POND46119-4-06.PPW

RATING TABLE FOR ONE OUTLET TYPE

Structure ID = CV (Culvert-Circular)

Mannings open channel maximum capacity: 6.12 cfs
UPstream ID's= R1, O1
DNstream ID = TW (Pond Outfall)

Pond WS. Elev. ft	Device Q cfs	(into) HW HGL ft	Converge DS HGL ft	Next DS HGL ft	DS HGL Error +/-ft	Q SUM Error +/-cfs	DS Chan. TW ft	TW Error +/-ft
750.00	.00	...	...	...	...	...	Free Outfall	
		WS below an invert; no flow.						
750.10	.02	750.07	Free	Free	.000	.000	Free Outfall	
		BACKWATER CONTROL..		Vh= .009ft	hwDi= .063ft		Lbw= 82.0ft	
750.20	.07	750.14	Free	Free	.000	.000	Free Outfall	
		BACKWATER CONTROL..		Vh= .018ft	hwDi= .115ft		Lbw= 82.0ft	
750.30	.14	750.20	Free	Free	.000	.000	Free Outfall	
		BACKWATER CONTROL..		Vh= .029ft	hwDi= .163ft		Lbw= 82.0ft	
750.40	.18	750.22	Free	Free	.000	.000	Free Outfall	
		BACKWATER CONTROL..		Vh= .033ft	hwDi= .183ft		Lbw= 82.0ft	
750.50	.21	750.24	Free	Free	.000	.000	Free Outfall	
		BACKWATER CONTROL..		Vh= .037ft	hwDi= .199ft		Lbw= 82.0ft	
750.60	.25	750.26	Free	Free	.000	.000	Free Outfall	
		BACKWATER CONTROL..		Vh= .040ft	hwDi= .212ft		Lbw= 82.0ft	
750.70	.27	750.28	Free	Free	.000	.000	Free Outfall	
		BACKWATER CONTROL..		Vh= .042ft	hwDi= .224ft		Lbw= 82.0ft	
750.80	.30	750.29	Free	Free	.000	.000	Free Outfall	
		BACKWATER CONTROL..		Vh= .045ft	hwDi= .235ft		Lbw= 82.0ft	
750.90	.33	750.30	Free	Free	.000	.000	Free Outfall	
		BACKWATER CONTROL..		Vh= .047ft	hwDi= .244ft		Lbw= 82.0ft	
751.00	.35	750.31	Free	Free	.000	.000	Free Outfall	
		BACKWATER CONTROL..		Vh= .049ft	hwDi= .251ft		Lbw= 82.0ft	
751.10	.37	750.32	Free	Free	.000	.000	Free Outfall	
		BACKWATER CONTROL..		Vh= .051ft	hwDi= .260ft		Lbw= 82.0ft	
751.20	.39	750.33	Free	Free	.000	.000	Free Outfall	
		BACKWATER CONTROL..		Vh= .053ft	hwDi= .267ft		Lbw= 82.0ft	
751.30	.41	750.34	Free	Free	.000	.000	Free Outfall	
		BACKWATER CONTROL..		Vh= .055ft	hwDi= .273ft		Lbw= 82.0ft	
751.40	.43	750.35	Free	Free	.000	.000	Free Outfall	
		BACKWATER CONTROL..		Vh= .056ft	hwDi= .280ft		Lbw= 82.0ft	
751.50	.45	750.35	Free	Free	.000	.000	Free Outfall	
		BACKWATER CONTROL..		Vh= .057ft	hwDi= .285ft		Lbw= 82.0ft	
751.60	.47	750.36	Free	Free	.000	.000	Free Outfall	
		BACKWATER CONTROL..		Vh= .058ft	hwDi= .291ft		Lbw= 82.0ft	
751.70	.48	750.37	Free	Free	.000	.000	Free Outfall	
		BACKWATER CONTROL..		Vh= .060ft	hwDi= .295ft		Lbw= 82.0ft	

File.... J:\PROJECT\2004\046119BelewsCreek\Pond Files\POND46119-4-06.PPW

RATING TABLE FOR ONE OUTLET TYPE

Structure ID = CV (Culvert-Circular)

Mannings open channel maximum capacity: 6.12 cfs
UPstream ID's= R1, O1
DNstream ID = TW (Pond Outfall)

Pond WS. Elev. ft	Device Q cfs	(into) HW HGL ft	Converge DS HGL ft	Next DS HGL ft	DS HGL Error +/-ft	Q SUM Error +/-cfs	DS Chan. TW ft	TW Error +/-ft
751.80	.50	750.37	Free	Free	.000	.000	Free Outfall	
		BACKWATER	CONTROL..	Vh= .061ft	hwDi=	.301ft	Lbw= 82.0ft	
751.90	.52	750.38	Free	Free	.000	.000	Free Outfall	
		BACKWATER	CONTROL..	Vh= .063ft	hwDi=	.306ft	Lbw= 82.0ft	
752.00	.53	750.39	Free	Free	.000	.000	Free Outfall	
		BACKWATER	CONTROL..	Vh= .064ft	hwDi=	.310ft	Lbw= 82.0ft	
752.10	.55	750.39	Free	Free	.000	.000	Free Outfall	
		BACKWATER	CONTROL..	Vh= .065ft	hwDi=	.314ft	Lbw= 82.0ft	
752.20	.56	750.40	Free	Free	.000	.000	Free Outfall	
		BACKWATER	CONTROL..	Vh= .065ft	hwDi=	.320ft	Lbw= 82.0ft	
752.30	.58	750.40	Free	Free	.000	.000	Free Outfall	
		BACKWATER	CONTROL..	Vh= .067ft	hwDi=	.324ft	Lbw= 82.0ft	
752.40	.59	750.41	Free	Free	.000	.000	Free Outfall	
		BACKWATER	CONTROL..	Vh= .067ft	hwDi=	.327ft	Lbw= 82.0ft	
752.50	.61	750.41	Free	Free	.000	.000	Free Outfall	
		BACKWATER	CONTROL..	Vh= .068ft	hwDi=	.330ft	Lbw= 82.0ft	
752.60	.62	750.42	Free	Free	.000	.000	Free Outfall	
		BACKWATER	CONTROL..	Vh= .070ft	hwDi=	.333ft	Lbw= 82.0ft	
752.70	.63	750.42	Free	Free	.000	.000	Free Outfall	
		BACKWATER	CONTROL..	Vh= .070ft	hwDi=	.338ft	Lbw= 82.0ft	
752.80	.65	750.43	Free	Free	.000	.000	Free Outfall	
		BACKWATER	CONTROL..	Vh= .071ft	hwDi=	.342ft	Lbw= 82.0ft	
752.90	.66	750.43	Free	Free	.000	.000	Free Outfall	
		BACKWATER	CONTROL..	Vh= .071ft	hwDi=	.346ft	Lbw= 82.0ft	
753.00	.67	750.44	Free	Free	.000	.000	Free Outfall	
		BACKWATER	CONTROL..	Vh= .072ft	hwDi=	.349ft	Lbw= 82.0ft	
753.10	.69	750.44	Free	Free	.000	.000	Free Outfall	
		BACKWATER	CONTROL..	Vh= .073ft	hwDi=	.351ft	Lbw= 82.0ft	
753.20	.70	750.44	Free	Free	.000	.000	Free Outfall	
		BACKWATER	CONTROL..	Vh= .074ft	hwDi=	.355ft	Lbw= 82.0ft	
753.30	.71	750.45	Free	Free	.000	.000	Free Outfall	
		BACKWATER	CONTROL..	Vh= .074ft	hwDi=	.358ft	Lbw= 82.0ft	
753.40	.72	750.45	Free	Free	.000	.000	Free Outfall	
		BACKWATER	CONTROL..	Vh= .076ft	hwDi=	.359ft	Lbw= 82.0ft	
753.50	.73	750.45	Free	Free	.000	.000	Free Outfall	
		BACKWATER	CONTROL..	Vh= .077ft	hwDi=	.363ft	Lbw= 82.0ft	

File.... J:\PROJECT\2004\046119BelewsCreek\Pond Files\POND46119-4-06.PPW

RATING TABLE FOR ONE OUTLET TYPE

Structure ID = CV (Culvert-Circular)

Mannings open channel maximum capacity: 6.12 cfs
UPstream ID's= R1, O1
DNstream ID = TW (Pond Outfall)

Pond WS. Elev. ft	Device Q cfs	(into) HW HGL ft	Converge DS HGL ft	Next DS HGL ft	DS HGL Error +/-ft	Q SUM Error +/-cfs	DS Chan. TW ft	TW Error +/-ft
753.60	.74	750.46	Free	Free	.000	.000	Free Outfall	
		BACKWATER	CONTROL..	Vh= .077ft	hwDi=	.366ft	Lbw= 82.0ft	
753.70	.76	750.46	Free	Free	.000	.000	Free Outfall	
		BACKWATER	CONTROL..	Vh= .078ft	hwDi=	.368ft	Lbw= 82.0ft	
753.80	.77	750.47	Free	Free	.000	.000	Free Outfall	
		BACKWATER	CONTROL..	Vh= .079ft	hwDi=	.371ft	Lbw= 82.0ft	
753.90	.78	750.47	Free	Free	.000	.000	Free Outfall	
		BACKWATER	CONTROL..	Vh= .080ft	hwDi=	.375ft	Lbw= 82.0ft	
754.00	.79	750.47	Free	Free	.000	.000	Free Outfall	
		BACKWATER	CONTROL..	Vh= .079ft	hwDi=	.378ft	Lbw= 82.0ft	
754.10	.80	750.48	Free	Free	.000	.000	Free Outfall	
		BACKWATER	CONTROL..	Vh= .080ft	hwDi=	.380ft	Lbw= 82.0ft	
754.20	.81	750.48	Free	Free	.000	.000	Free Outfall	
		BACKWATER	CONTROL..	Vh= .080ft	hwDi=	.383ft	Lbw= 82.0ft	
754.30	.82	750.48	Free	Free	.000	.000	Free Outfall	
		BACKWATER	CONTROL..	Vh= .080ft	hwDi=	.386ft	Lbw= 82.0ft	
754.40	.83	750.49	Free	Free	.000	.000	Free Outfall	
		BACKWATER	CONTROL..	Vh= .082ft	hwDi=	.388ft	Lbw= 82.0ft	
754.50	.84	750.49	Free	Free	.000	.000	Free Outfall	
		BACKWATER	CONTROL..	Vh= .083ft	hwDi=	.389ft	Lbw= 82.0ft	
754.60	.85	750.49	Free	Free	.000	.000	Free Outfall	
		BACKWATER	CONTROL..	Vh= .084ft	hwDi=	.391ft	Lbw= 82.0ft	
754.70	.86	750.50	Free	Free	.000	.000	Free Outfall	
		BACKWATER	CONTROL..	Vh= .085ft	hwDi=	.394ft	Lbw= 82.0ft	
754.80	.87	750.50	Free	Free	.000	.000	Free Outfall	
		BACKWATER	CONTROL..	Vh= .084ft	hwDi=	.397ft	Lbw= 82.0ft	
754.90	.88	750.50	Free	Free	.000	.000	Free Outfall	
		BACKWATER	CONTROL..	Vh= .085ft	hwDi=	.399ft	Lbw= 82.0ft	
755.00	.89	750.50	Free	Free	.000	.000	Free Outfall	
		BACKWATER	CONTROL..	Vh= .085ft	hwDi=	.401ft	Lbw= 82.0ft	
755.10	.90	750.51	Free	Free	.000	.000	Free Outfall	
		BACKWATER	CONTROL..	Vh= .086ft	hwDi=	.403ft	Lbw= 82.0ft	
755.20	.91	750.51	Free	Free	.000	.000	Free Outfall	
		BACKWATER	CONTROL..	Vh= .087ft	hwDi=	.405ft	Lbw= 82.0ft	
755.30	.92	750.51	Free	Free	.000	.000	Free Outfall	
		BACKWATER	CONTROL..	Vh= .088ft	hwDi=	.407ft	Lbw= 82.0ft	

File.... J:\PROJECT\2004\046119BelewsCreek\Pond Files\POND46119-4-06.PPW

RATING TABLE FOR ONE OUTLET TYPE

Structure ID = CV (Culvert-Circular)

Mannings open channel maximum capacity: 6.12 cfs
UPstream ID's= R1, O1
DNstream ID = TW (Pond Outfall)

Pond WS. Elev. ft	Device Q cfs	(into) HW HGL ft	Converge DS HGL ft	Next DS HGL ft	DS HGL Error +/-ft	Q SUM Error +/-cfs	DS Chan. TW ft	TW Error +/-ft
755.40	.93	750.51	Free	Free	.000	.000	Free Outfall	
		BACKWATER	CONTROL..	Vh= .087ft	hwDi=	.410ft	Lbw= 82.0ft	
755.50	.94	750.52	Free	Free	.000	.000	Free Outfall	
		BACKWATER	CONTROL..	Vh= .087ft	hwDi=	.413ft	Lbw= 82.0ft	
755.60	.95	750.52	Free	Free	.000	.000	Free Outfall	
		BACKWATER	CONTROL..	Vh= .089ft	hwDi=	.414ft	Lbw= 82.0ft	
755.70	.96	750.52	Free	Free	.000	.000	Free Outfall	
		BACKWATER	CONTROL..	Vh= .089ft	hwDi=	.415ft	Lbw= 82.0ft	
755.80	.96	750.53	Free	Free	.000	.000	Free Outfall	
		BACKWATER	CONTROL..	Vh= .090ft	hwDi=	.417ft	Lbw= 82.0ft	
755.90	.97	750.53	Free	Free	.000	.000	Free Outfall	
		BACKWATER	CONTROL..	Vh= .089ft	hwDi=	.420ft	Lbw= 82.0ft	
756.00	.98	750.53	Free	Free	.000	.000	Free Outfall	
		BACKWATER	CONTROL..	Vh= .090ft	hwDi=	.423ft	Lbw= 82.0ft	
756.10	.99	750.53	Free	Free	.000	.000	Free Outfall	
		BACKWATER	CONTROL..	Vh= .090ft	hwDi=	.423ft	Lbw= 82.0ft	
756.20	1.00	750.54	Free	Free	.000	.000	Free Outfall	
		BACKWATER	CONTROL..	Vh= .091ft	hwDi=	.426ft	Lbw= 82.0ft	
756.30	1.01	750.54	Free	Free	.000	.000	Free Outfall	
		BACKWATER	CONTROL..	Vh= .091ft	hwDi=	.428ft	Lbw= 82.0ft	
756.40	1.02	750.54	Free	Free	.000	.000	Free Outfall	
		BACKWATER	CONTROL..	Vh= .092ft	hwDi=	.429ft	Lbw= 82.0ft	
756.50	1.02	750.54	Free	Free	.000	.000	Free Outfall	
		BACKWATER	CONTROL..	Vh= .093ft	hwDi=	.430ft	Lbw= 82.0ft	
756.60	1.03	750.54	Free	Free	.000	.000	Free Outfall	
		BACKWATER	CONTROL..	Vh= .092ft	hwDi=	.433ft	Lbw= 82.0ft	
756.70	1.04	750.55	Free	Free	.000	.000	Free Outfall	
		BACKWATER	CONTROL..	Vh= .094ft	hwDi=	.434ft	Lbw= 82.0ft	
756.80	1.05	750.55	Free	Free	.000	.000	Free Outfall	
		BACKWATER	CONTROL..	Vh= .093ft	hwDi=	.437ft	Lbw= 82.0ft	
756.90	1.06	750.55	Free	Free	.000	.000	Free Outfall	
		BACKWATER	CONTROL..	Vh= .095ft	hwDi=	.437ft	Lbw= 82.0ft	
757.00	1.07	750.55	Free	Free	.000	.000	Free Outfall	
		BACKWATER	CONTROL..	Vh= .095ft	hwDi=	.439ft	Lbw= 82.0ft	
757.10	1.07	750.56	Free	Free	.000	.000	Free Outfall	
		BACKWATER	CONTROL..	Vh= .095ft	hwDi=	.441ft	Lbw= 82.0ft	

File.... J:\PROJECT\2004\046119BelewsCreek\Pond Files\POND46119-4-06.PPW

RATING TABLE FOR ONE OUTLET TYPE

Structure ID = CV (Culvert-Circular)

Mannings open channel maximum capacity: 6.12 cfs
UPstream ID's= R1, O1
DNstream ID = TW (Pond Outfall)

Pond WS. Elev. ft	Device Q cfs	(into) HW HGL ft	Converge DS HGL ft	Next DS HGL ft	DS HGL Error +/-ft	Q SUM Error +/-cfs	DS Chan. TW ft	TW Error +/-ft
757.20	1.08	750.56	Free	Free	.000	.000	Free Outfall	
		BACKWATER	CONTROL..	Vh= .096ft	hwDi=	.443ft	Lbw= 82.0ft	
757.30	1.09	750.56	Free	Free	.000	.000	Free Outfall	
		BACKWATER	CONTROL..	Vh= .095ft	hwDi=	.445ft	Lbw= 82.0ft	
757.40	1.10	750.56	Free	Free	.000	.000	Free Outfall	
		BACKWATER	CONTROL..	Vh= .097ft	hwDi=	.445ft	Lbw= 82.0ft	
757.50	1.11	750.56	Free	Free	.000	.000	Free Outfall	
		BACKWATER	CONTROL..	Vh= .096ft	hwDi=	.449ft	Lbw= 82.0ft	
757.60	1.11	750.57	Free	Free	.000	.000	Free Outfall	
		BACKWATER	CONTROL..	Vh= .097ft	hwDi=	.450ft	Lbw= 82.0ft	
757.70	1.12	750.57	Free	Free	.000	.000	Free Outfall	
		BACKWATER	CONTROL..	Vh= .097ft	hwDi=	.451ft	Lbw= 82.0ft	
757.80	1.13	750.57	Free	Free	.000	.000	Free Outfall	
		BACKWATER	CONTROL..	Vh= .098ft	hwDi=	.452ft	Lbw= 82.0ft	
757.90	1.14	750.57	Free	Free	.000	.000	Free Outfall	
		BACKWATER	CONTROL..	Vh= .098ft	hwDi=	.455ft	Lbw= 82.0ft	
758.00	1.14	750.57	Free	Free	.000	.000	Free Outfall	
		BACKWATER	CONTROL..	Vh= .099ft	hwDi=	.456ft	Lbw= 82.0ft	
758.10	1.15	750.58	Free	Free	.000	.000	Free Outfall	
		BACKWATER	CONTROL..	Vh= .098ft	hwDi=	.458ft	Lbw= 82.0ft	
758.20	1.16	750.58	Free	Free	.000	.000	Free Outfall	
		BACKWATER	CONTROL..	Vh= .100ft	hwDi=	.459ft	Lbw= 82.0ft	
758.30	1.17	750.58	Free	Free	.000	.000	Free Outfall	
		BACKWATER	CONTROL..	Vh= .100ft	hwDi=	.461ft	Lbw= 82.0ft	
758.40	1.17	750.58	Free	Free	.000	.000	Free Outfall	
		BACKWATER	CONTROL..	Vh= .100ft	hwDi=	.462ft	Lbw= 82.0ft	
758.50	1.77	750.72	Free	Free	.000	.000	Free Outfall	
		BACKWATER	CONTROL..	Vh= .125ft	hwDi=	.574ft	Lbw= 82.0ft	
758.60	2.85	750.94	Free	Free	.000	.000	Free Outfall	
		BACKWATER	CONTROL..	Vh= .160ft	hwDi=	.751ft	Lbw= 82.0ft	
758.70	4.25	751.20	Free	Free	.000	.000	Free Outfall	
		BACKWATER	CONTROL..	Vh= .194ft	hwDi=	.965ft	Lbw= 82.0ft	
758.80	5.90	751.52	Free	Free	.000	.000	Free Outfall	
		BACKWATER	CONTROL..	Vh= .217ft	hwDi=	1.255ft	Lbw= 82.0ft	
758.90	7.74	752.32	Free	Free	.000	.000	Free Outfall	
		FULL FLOW...Lfull=53.77ft		Vh=.298ft		HL=1.354ft		

File.... J:\PROJECT\2004\046119BelewsCreek\Pond Files\POND46119-4-06.PPW

RATING TABLE FOR ONE OUTLET TYPE

Structure ID = CV (Culvert-Circular)

Mannings open channel maximum capacity: 6.12 cfs
UPstream ID's= R1, O1
DNstream ID = TW (Pond Outfall)

Pond WS. Elev. ft	Device Q cfs	(into) HW HGL ft	Converge DS HGL ft	Next DS HGL ft	DS HGL Error +/-ft	Q SUM Error +/-cfs	DS Chan. TW ft	TW Error +/-ft
759.00	9.74	753.50	Free	Free	.000	.000	Free	Outfall
		FULL FLOW...	Lfull=74.03ft		Vh=.473ft	HL=2.739ft		
759.10	11.89	755.02	Free	Free	.000	.000	Free	Outfall
		FULL FLOW...	Lfull=79.61ft		Vh=.703ft	HL=4.318ft		
759.20	14.13	756.88	Free	Free	.000	.000	Free	Outfall
		FULL FLOW...	Lfull=81.12ft		Vh=.993ft	HL=6.193ft		
759.30	14.89	757.59	Free	Free	.000	.000	Free	Outfall
		FULL FLOW...	Lfull=81.37ft		Vh=1.104ft	HL=6.901ft		
759.40	15.57	758.24	Free	Free	.000	.000	Free	Outfall
		FULL FLOW...	Lfull=81.53ft		Vh=1.207ft	HL=7.557ft		
759.50	16.19	759.50	Free	Free	.000	.000	Free	Outfall
		FULL FLOW...	Lfull=81.76ft		Vh=1.405ft	HL=8.817ft		
759.60	16.90	759.60	Free	Free	.000	.000	Free	Outfall
		FULL FLOW...	Lfull=81.77ft		Vh=1.421ft	HL=8.918ft		
759.70	16.99	759.70	Free	Free	.000	.000	Free	Outfall
		FULL FLOW...	Lfull=81.77ft		Vh=1.437ft	HL=9.018ft		
759.80	17.09	759.80	Free	Free	.000	.000	Free	Outfall
		FULL FLOW...	Lfull=81.78ft		Vh=1.453ft	HL=9.117ft		
759.90	17.18	759.90	Free	Free	.000	.000	Free	Outfall
		FULL FLOW...	Lfull=81.78ft		Vh=1.469ft	HL=9.217ft		
760.00	17.27	760.00	Free	Free	.000	.000	Free	Outfall
		FULL FLOW...	Lfull=81.78ft		Vh=1.485ft	HL=9.318ft		

File.... J:\PROJECT\2004\046119BelewsCreek\Pond Files\POND46119-4-06.PPW

***** COMPOSITE OUTFLOW SUMMARY ****

WS Elev, Total Q		Notes		
		Converge		
Elev. ft	Q cfs	TW Elev ft	Error +/-ft	Contributing Structures
750.00	.00	Free Outfall		(no Q: R1,O1,CV)
750.10	.02	Free Outfall		O1,CV (no Q: R1)
750.20	.07	Free Outfall		O1,CV (no Q: R1)
750.30	.14	Free Outfall		O1,CV (no Q: R1)
750.40	.18	Free Outfall		O1,CV (no Q: R1)
750.50	.21	Free Outfall		O1,CV (no Q: R1)
750.60	.25	Free Outfall		O1,CV (no Q: R1)
750.70	.27	Free Outfall		O1,CV (no Q: R1)
750.80	.30	Free Outfall		O1,CV (no Q: R1)
750.90	.33	Free Outfall		O1,CV (no Q: R1)
751.00	.35	Free Outfall		O1,CV (no Q: R1)
751.10	.37	Free Outfall		O1,CV (no Q: R1)
751.20	.39	Free Outfall		O1,CV (no Q: R1)
751.30	.41	Free Outfall		O1,CV (no Q: R1)
751.40	.43	Free Outfall		O1,CV (no Q: R1)
751.50	.45	Free Outfall		O1,CV (no Q: R1)
751.60	.47	Free Outfall		O1,CV (no Q: R1)
751.70	.48	Free Outfall		O1,CV (no Q: R1)
751.80	.50	Free Outfall		O1,CV (no Q: R1)
751.90	.52	Free Outfall		O1,CV (no Q: R1)
752.00	.53	Free Outfall		O1,CV (no Q: R1)
752.10	.55	Free Outfall		O1,CV (no Q: R1)
752.20	.56	Free Outfall		O1,CV (no Q: R1)
752.30	.58	Free Outfall		O1,CV (no Q: R1)
752.40	.59	Free Outfall		O1,CV (no Q: R1)
752.50	.61	Free Outfall		O1,CV (no Q: R1)
752.60	.62	Free Outfall		O1,CV (no Q: R1)
752.70	.63	Free Outfall		O1,CV (no Q: R1)
752.80	.65	Free Outfall		O1,CV (no Q: R1)
752.90	.66	Free Outfall		O1,CV (no Q: R1)
753.00	.67	Free Outfall		O1,CV (no Q: R1)
753.10	.69	Free Outfall		O1,CV (no Q: R1)
753.20	.70	Free Outfall		O1,CV (no Q: R1)
753.30	.71	Free Outfall		O1,CV (no Q: R1)
753.40	.72	Free Outfall		O1,CV (no Q: R1)
753.50	.73	Free Outfall		O1,CV (no Q: R1)
753.60	.74	Free Outfall		O1,CV (no Q: R1)
753.70	.76	Free Outfall		O1,CV (no Q: R1)

File.... J:\PROJECT\2004\046119BelewsCreek\Pond Files\POND46119-4-06.PPW

***** COMPOSITE OUTFLOW SUMMARY ****

WS Elev, Total Q		Notes		
		Converge		
Elev. ft	Q cfs	TW Elev ft	Error +/-ft	Contributing Structures
753.80	.77	Free Outfall		O1,CV (no Q: R1)
753.90	.78	Free Outfall		O1,CV (no Q: R1)
754.00	.79	Free Outfall		O1,CV (no Q: R1)
754.10	.80	Free Outfall		O1,CV (no Q: R1)
754.20	.81	Free Outfall		O1,CV (no Q: R1)
754.30	.82	Free Outfall		O1,CV (no Q: R1)
754.40	.83	Free Outfall		O1,CV (no Q: R1)
754.50	.84	Free Outfall		O1,CV (no Q: R1)
754.60	.85	Free Outfall		O1,CV (no Q: R1)
754.70	.86	Free Outfall		O1,CV (no Q: R1)
754.80	.87	Free Outfall		O1,CV (no Q: R1)
754.90	.88	Free Outfall		O1,CV (no Q: R1)
755.00	.89	Free Outfall		O1,CV (no Q: R1)
755.10	.90	Free Outfall		O1,CV (no Q: R1)
755.20	.91	Free Outfall		O1,CV (no Q: R1)
755.30	.92	Free Outfall		O1,CV (no Q: R1)
755.40	.93	Free Outfall		O1,CV (no Q: R1)
755.50	.94	Free Outfall		O1,CV (no Q: R1)
755.60	.95	Free Outfall		O1,CV (no Q: R1)
755.70	.96	Free Outfall		O1,CV (no Q: R1)
755.80	.96	Free Outfall		O1,CV (no Q: R1)
755.90	.97	Free Outfall		O1,CV (no Q: R1)
756.00	.98	Free Outfall		O1,CV (no Q: R1)
756.10	.99	Free Outfall		O1,CV (no Q: R1)
756.20	1.00	Free Outfall		O1,CV (no Q: R1)
756.30	1.01	Free Outfall		O1,CV (no Q: R1)
756.40	1.02	Free Outfall		O1,CV (no Q: R1)
756.50	1.02	Free Outfall		O1,CV (no Q: R1)
756.60	1.03	Free Outfall		O1,CV (no Q: R1)
756.70	1.04	Free Outfall		O1,CV (no Q: R1)
756.80	1.05	Free Outfall		O1,CV (no Q: R1)
756.90	1.06	Free Outfall		O1,CV (no Q: R1)
757.00	1.07	Free Outfall		O1,CV (no Q: R1)
757.10	1.07	Free Outfall		O1,CV (no Q: R1)
757.20	1.08	Free Outfall		O1,CV (no Q: R1)
757.30	1.09	Free Outfall		O1,CV (no Q: R1)
757.40	1.10	Free Outfall		O1,CV (no Q: R1)
757.50	1.11	Free Outfall		O1,CV (no Q: R1)

File.... J:\PROJECT\2004\046119BelewsCreek\Pond Files\POND46119-4-06.PPW

***** COMPOSITE OUTFLOW SUMMARY ****

WS Elev, Total Q		Notes		
		Converge		
Elev. ft	Q cfs	TW Elev ft	Error +/-ft	Contributing Structures
757.60	1.11	Free Outfall		O1,CV (no Q: R1)
757.70	1.12	Free Outfall		O1,CV (no Q: R1)
757.80	1.13	Free Outfall		O1,CV (no Q: R1)
757.90	1.14	Free Outfall		O1,CV (no Q: R1)
758.00	1.14	Free Outfall		O1,CV (no Q: R1)
758.10	1.15	Free Outfall		O1,CV (no Q: R1)
758.20	1.16	Free Outfall		O1,CV (no Q: R1)
758.30	1.17	Free Outfall		O1,CV (no Q: R1)
758.40	1.17	Free Outfall		O1,CV (no Q: R1)
758.50	1.77	Free Outfall		R1,O1,CV
758.60	2.85	Free Outfall		R1,O1,CV
758.70	4.25	Free Outfall		R1,O1,CV
758.80	5.90	Free Outfall		R1,O1,CV
758.90	7.74	Free Outfall		R1,O1,CV
759.00	9.74	Free Outfall		R1,O1,CV
759.10	11.89	Free Outfall		R1,O1,CV
759.20	14.13	Free Outfall		R1,O1,CV
759.30	14.89	Free Outfall		R1,O1,CV
759.40	15.57	Free Outfall		R1,O1,CV
759.50	16.19	Free Outfall		R1,CV (no Q: O1)
759.60	16.90	Free Outfall		R1,CV (no Q: O1)
759.70	16.99	Free Outfall		R1,CV (no Q: O1)
759.80	17.09	Free Outfall		R1,CV (no Q: O1)
759.90	17.18	Free Outfall		R1,CV (no Q: O1)
760.00	17.27	Free Outfall		R1,CV (no Q: O1)

LEVEL POOL ROUTING DATA

HYG Dir = J:\PROJECT\2004\046119BelewsCreek\Pond Files\
Inflow HYG file = NONE STORED - POND 10 IN 2YR
Outflow HYG file = NONE STORED - POND 10 OUT 2YR

Pond Node Data = POND 10
Pond Volume Data = POND 10
Pond Outlet Data = Outlet 1

No Infiltration

INITIAL CONDITIONS

Starting WS Elev = 750.00 ft
Starting Volume = .000 ac-ft
Starting Outflow = .00 cfs
Starting Infiltr. = .00 cfs
Starting Total Qout= .00 cfs
Time Increment = .0500 hrs

Elevation ft	Outflow cfs	Storage ac-ft	Area sq.ft	Infilt. cfs	Q Total cfs	2S/t + O cfs
750.00	.00	.000	765	.00	.00	.00
750.10	.02	.002	1091	.00	.02	1.04
750.20	.07	.005	1475	.00	.07	2.51
750.30	.14	.009	1916	.00	.14	4.46
750.40	.18	.014	2415	.00	.18	6.91
750.50	.21	.020	2972	.00	.21	9.93
750.60	.25	.028	3586	.00	.25	13.60
750.70	.27	.037	4259	.00	.27	17.98
750.80	.30	.047	4988	.00	.30	23.14
750.90	.33	.060	5776	.00	.33	29.14
751.00	.35	.074	6621	.00	.35	36.04
751.10	.37	.090	7524	.00	.37	43.91
751.20	.39	.108	8485	.00	.39	52.83
751.30	.41	.129	9503	.00	.41	62.83
751.40	.43	.152	10579	.00	.43	74.01
751.50	.45	.178	11713	.00	.45	86.40
751.60	.47	.206	12904	.00	.47	100.09
751.70	.48	.237	14154	.00	.48	115.14
751.80	.50	.271	15460	.00	.50	131.59
751.90	.52	.308	16825	.00	.52	149.55

LEVEL POOL ROUTING DATA

HYG Dir = J:\PROJECT\2004\046119BelewsCreek\Pond Files\
Inflow HYG file = NONE STORED - POND 10 IN 2YR
Outflow HYG file = NONE STORED - POND 10 OUT 2YR

Pond Node Data = POND 10
Pond Volume Data = POND 10
Pond Outlet Data = Outlet 1

No Infiltration

INITIAL CONDITIONS

Starting WS Elev = 750.00 ft
Starting Volume = .000 ac-ft
Starting Outflow = .00 cfs
Starting Infiltr. = .00 cfs
Starting Total Qout= .00 cfs
Time Increment = .0500 hrs

Elevation ft	Outflow cfs	Storage ac-ft	Area sq.ft	Infilt. cfs	Q Total cfs	2S/t + O cfs
752.00	.53	.348	18247	.00	.53	169.04
752.10	.55	.391	19121	.00	.55	189.81
752.20	.56	.436	20017	.00	.56	211.57
752.30	.58	.483	20932	.00	.58	234.33
752.40	.59	.532	21869	.00	.59	258.13
752.50	.61	.583	22825	.00	.61	282.96
752.60	.62	.637	23802	.00	.62	308.87
752.70	.63	.693	24800	.00	.63	335.90
752.80	.65	.751	25818	.00	.65	364.02
752.90	.66	.811	26856	.00	.66	393.31
753.00	.67	.874	27915	.00	.67	423.74
753.10	.69	.939	28994	.00	.69	455.36
753.20	.70	1.007	30095	.00	.70	488.21
753.30	.71	1.078	31215	.00	.71	522.27
753.40	.72	1.151	32356	.00	.72	557.61
753.50	.73	1.226	33517	.00	.73	594.21
753.60	.74	1.304	34699	.00	.74	632.10
753.70	.76	1.386	35901	.00	.76	671.35
753.80	.77	1.469	37124	.00	.77	711.92
753.90	.78	1.556	38368	.00	.78	753.88

LEVEL POOL ROUTING DATA

HYG Dir = J:\PROJECT\2004\046119BelewsCreek\Pond Files\
Inflow HYG file = NONE STORED - POND 10 IN 2YR
Outflow HYG file = NONE STORED - POND 10 OUT 2YR

Pond Node Data = POND 10
Pond Volume Data = POND 10
Pond Outlet Data = Outlet 1

No Infiltration

INITIAL CONDITIONS

Starting WS Elev = 750.00 ft
Starting Volume = .000 ac-ft
Starting Outflow = .00 cfs
Starting Infiltr. = .00 cfs
Starting Total Qout= .00 cfs
Time Increment = .0500 hrs

Elevation ft	Outflow cfs	Storage ac-ft	Area sq.ft	Infilt. cfs	Q Total cfs	2S/t + O cfs
754.00	.79	1.646	39631	.00	.79	797.22
754.10	.80	1.737	40197	.00	.80	841.56
754.20	.81	1.830	40768	.00	.81	886.57
754.30	.82	1.924	41342	.00	.82	932.19
754.40	.83	2.020	41920	.00	.83	978.47
754.50	.84	2.117	42502	.00	.84	1025.37
754.60	.85	2.215	43089	.00	.85	1072.92
754.70	.86	2.315	43679	.00	.86	1121.15
754.80	.87	2.416	44273	.00	.87	1170.01
754.90	.88	2.518	44872	.00	.88	1219.56
755.00	.89	2.622	45474	.00	.89	1269.75
755.10	.90	2.727	46080	.00	.90	1320.61
755.20	.91	2.833	46691	.00	.91	1372.18
755.30	.92	2.941	47305	.00	.92	1424.40
755.40	.93	3.050	47924	.00	.93	1477.33
755.50	.94	3.161	48546	.00	.94	1530.92
755.60	.95	3.273	49173	.00	.95	1585.20
755.70	.96	3.387	49803	.00	.96	1640.22
755.80	.96	3.502	50438	.00	.96	1695.90
755.90	.97	3.619	51077	.00	.97	1752.33

LEVEL POOL ROUTING DATA

HYG Dir = J:\PROJECT\2004\046119BelewsCreek\Pond Files\
Inflow HYG file = NONE STORED - POND 10 IN 2YR
Outflow HYG file = NONE STORED - POND 10 OUT 2YR

Pond Node Data = POND 10
Pond Volume Data = POND 10
Pond Outlet Data = Outlet 1

No Infiltration

INITIAL CONDITIONS

Starting WS Elev = 750.00 ft
Starting Volume = .000 ac-ft
Starting Outflow = .00 cfs
Starting Infiltr. = .00 cfs
Starting Total Qout= .00 cfs
Time Increment = .0500 hrs

Elevation ft	Outflow cfs	Storage ac-ft	Area sq.ft	Infilt. cfs	Q Total cfs	2S/t + O cfs
756.00	.98	3.736	51719	.00	.98	1809.43
756.10	.99	3.856	52086	.00	.99	1867.10
756.20	1.00	3.976	52454	.00	1.00	1925.20
756.30	1.01	4.096	52823	.00	1.01	1983.68
756.40	1.02	4.218	53193	.00	1.02	2042.61
756.50	1.02	4.341	53565	.00	1.02	2101.92
756.60	1.03	4.464	53938	.00	1.03	2161.63
756.70	1.04	4.588	54313	.00	1.04	2221.80
756.80	1.05	4.713	54688	.00	1.05	2282.35
756.90	1.06	4.839	55066	.00	1.06	2343.36
757.00	1.07	4.966	55444	.00	1.07	2404.75
757.10	1.07	5.094	55823	.00	1.07	2466.55
757.20	1.08	5.223	56204	.00	1.08	2528.82
757.30	1.09	5.352	56586	.00	1.09	2591.48
757.40	1.10	5.482	56970	.00	1.10	2654.59
757.50	1.11	5.614	57355	.00	1.11	2718.10
757.60	1.11	5.746	57741	.00	1.11	2782.03
757.70	1.12	5.879	58128	.00	1.12	2846.44
757.80	1.13	6.013	58517	.00	1.13	2911.23
757.90	1.14	6.147	58907	.00	1.14	2976.50

LEVEL POOL ROUTING DATA

HYG Dir = J:\PROJECT\2004\046119BelewsCreek\Pond Files\
Inflow HYG file = NONE STORED - POND 10 IN 2YR
Outflow HYG file = NONE STORED - POND 10 OUT 2YR

Pond Node Data = POND 10
Pond Volume Data = POND 10
Pond Outlet Data = Outlet 1

No Infiltration

INITIAL CONDITIONS

Starting WS Elev = 750.00 ft
Starting Volume = .000 ac-ft
Starting Outflow = .00 cfs
Starting Infiltr. = .00 cfs
Starting Total Qout= .00 cfs
Time Increment = .0500 hrs

Elevation ft	Outflow cfs	Storage ac-ft	Area sq.ft	Infilt. cfs	Q Total cfs	2S/t + O cfs
758.00	1.14	6.283	59298	.00	1.14	3042.16
758.10	1.15	6.420	59681	.00	1.15	3108.25
758.20	1.16	6.557	60066	.00	1.16	3174.81
758.30	1.17	6.695	60452	.00	1.17	3241.75
758.40	1.17	6.835	60839	.00	1.17	3309.17
758.50	1.77	6.975	61228	.00	1.77	3377.56
758.60	2.85	7.116	61617	.00	2.85	3446.87
758.70	4.25	7.258	62008	.00	4.25	3516.98
758.80	5.90	7.400	62400	.00	5.90	3587.73
758.90	7.74	7.544	62794	.00	7.74	3659.15
759.00	9.74	7.689	63188	.00	9.74	3731.12
759.10	11.89	7.834	63584	.00	11.89	3803.68
759.20	14.13	7.981	63981	.00	14.13	3876.81
759.30	14.89	8.128	64379	.00	14.89	3948.87
759.40	15.57	8.276	64779	.00	15.57	4021.33
759.50	16.19	8.425	65180	.00	16.19	4094.13
759.60	16.90	8.576	65582	.00	16.90	4167.47
759.70	16.99	8.727	65985	.00	16.99	4240.68
759.80	17.09	8.879	66389	.00	17.09	4314.30
759.90	17.18	9.031	66795	.00	17.18	4388.41

LEVEL POOL ROUTING DATA

```
HYG Dir          = J:\PROJECT\2004\046119BelewsCreek\Pond Files\
Inflow  HYG file = NONE STORED - POND 10      IN  2YR
Outflow HYG file = NONE STORED - POND 10      OUT 2YR

Pond Node   Data = POND 10
Pond Volume Data = POND 10
Pond Outlet Data = Outlet 1

No Infiltration

INITIAL CONDITIONS
----------------------------------
Starting WS Elev   =   750.00 ft
Starting Volume    =     .000 ac-ft
Starting Outflow   =      .00 cfs
Starting Infiltr.  =      .00 cfs
Starting Total Qout=      .00 cfs
Time Increment     =    .0500 hrs
```

Elevation ft	Outflow cfs	Storage ac-ft	Area sq.ft	Infilt. cfs	Q Total cfs	2S/t + O cfs
760.00	17.27	9.185	67202	.00	17.27	4462.93

SUMMARY FOR HYDROGRAPH ADDITION
at Node: POND 10 IN

HYG Directory: J:\PROJECT\2004\046119BelewsCreek\Pond Files\

Upstream Link ID	Upstream Node ID	HYG file	HYG ID	HYG tag
ADDLINK 10	SUBAREA 10		SUBAREA 10	25

INFLOWS TO: POND 10 IN

HYG file	HYG ID	HYG tag	Volume ac-ft	Peak Time hrs	Peak Flow cfs
	SUBAREA 10	25	7.320	11.9500	121.73

TOTAL FLOW INTO: POND 10 IN

HYG file	HYG ID	HYG tag	Volume ac-ft	Peak Time hrs	Peak Flow cfs
	POND 10 IN	25	7.320	11.9500	121.73

TOTAL NODE INFLOW...
HYG file =
HYG ID = POND 10 IN
HYG Tag = 25

Peak Discharge = 121.73 cfs
Time to Peak = 11.9500 hrs
HYG Volume = 7.320 ac-ft

HYDROGRAPH ORDINATES (cfs)
Output Time increment = .0500 hrs
Time on left represents time for first value in each row.

Time hrs					
3.0000	.00	.00	.01	.02	.03
3.2500	.04	.05	.06	.07	.08
3.5000	.09	.10	.11	.12	.13
3.7500	.14	.15	.16	.17	.19
4.0000	.20	.21	.22	.23	.24
4.2500	.25	.26	.27	.29	.30
4.5000	.31	.32	.34	.35	.36
4.7500	.37	.38	.40	.41	.42
5.0000	.44	.45	.46	.47	.49
5.2500	.50	.51	.53	.54	.55
5.5000	.57	.58	.59	.61	.62
5.7500	.64	.65	.66	.68	.69
6.0000	.71	.72	.73	.75	.76
6.2500	.78	.79	.80	.82	.83
6.5000	.85	.86	.88	.89	.91
6.7500	.92	.94	.95	.96	.98
7.0000	.99	1.01	1.02	1.04	1.05
7.2500	1.07	1.08	1.10	1.11	1.13
7.5000	1.14	1.16	1.17	1.19	1.20
7.7500	1.22	1.23	1.25	1.26	1.28
8.0000	1.29	1.31	1.34	1.37	1.41
8.2500	1.44	1.49	1.53	1.57	1.61
8.5000	1.66	1.70	1.74	1.78	1.83
8.7500	1.87	1.92	1.97	2.02	2.06
9.0000	2.11	2.15	2.18	2.21	2.23
9.2500	2.24	2.25	2.27	2.28	2.29
9.5000	2.30	2.33	2.37	2.42	2.49
9.7500	2.55	2.63	2.70	2.78	2.85
10.0000	2.93	3.00	3.10	3.19	3.31
10.2500	3.41	3.53	3.63	3.76	3.86
10.5000	3.99	4.11	4.26	4.42	4.61

HYDROGRAPH ORDINATES (cfs)
Output Time increment = .0500 hrs
Time on left represents time for first value in each row.

Time hrs					
10.7500	4.78	4.99	5.16	5.37	5.56
11.0000	5.77	6.00	6.33	6.67	7.13
11.2500	7.54	8.03	8.47	8.97	9.41
11.5000	9.96	12.43	17.67	24.83	36.00
11.7500	47.34	62.61	82.70	113.42	121.73
12.0000	108.63	85.71	48.26	28.87	21.35
12.2500	18.10	16.11	14.79	13.48	12.40
12.5000	11.14	10.24	9.38	8.86	8.49
12.7500	8.21	7.91	7.66	7.37	7.13
13.0000	6.83	6.61	6.38	6.21	6.04
13.2500	5.90	5.75	5.61	5.46	5.32
13.5000	5.17	5.04	4.90	4.80	4.68
13.7500	4.58	4.47	4.37	4.26	4.17
14.0000	4.05	3.97	3.89	3.84	3.79
14.2500	3.75	3.72	3.68	3.64	3.61
14.5000	3.57	3.54	3.50	3.46	3.43
14.7500	3.39	3.35	3.32	3.28	3.25
15.0000	3.21	3.17	3.14	3.10	3.06
15.2500	3.03	2.99	2.96	2.92	2.88
15.5000	2.85	2.81	2.77	2.74	2.70
15.7500	2.67	2.62	2.59	2.55	2.52
16.0000	2.48	2.45	2.42	2.40	2.39
16.2500	2.37	2.36	2.35	2.33	2.32
16.5000	2.31	2.30	2.28	2.27	2.26
16.7500	2.24	2.23	2.22	2.20	2.19
17.0000	2.18	2.16	2.15	2.14	2.12
17.2500	2.11	2.10	2.09	2.07	2.06
17.5000	2.05	2.04	2.02	2.01	2.00
17.7500	1.98	1.97	1.96	1.94	1.93
18.0000	1.92	1.90	1.89	1.88	1.86
18.2500	1.85	1.84	1.83	1.81	1.80
18.5000	1.79	1.77	1.76	1.75	1.73
18.7500	1.72	1.71	1.69	1.68	1.67
19.0000	1.65	1.64	1.63	1.62	1.60
19.2500	1.59	1.58	1.57	1.55	1.54
19.5000	1.52	1.51	1.50	1.48	1.47
19.7500	1.46	1.44	1.43	1.42	1.41
20.0000	1.39	1.38	1.37	1.37	1.36
20.2500	1.36	1.36	1.36	1.35	1.35
20.5000	1.35	1.35	1.34	1.34	1.34
20.7500	1.34	1.33	1.33	1.33	1.33
21.0000	1.32	1.32	1.32	1.32	1.31
21.2500	1.31	1.31	1.31	1.30	1.30
21.5000	1.30	1.29	1.29	1.29	1.29
21.7500	1.28	1.28	1.28	1.28	1.27

HYDROGRAPH ORDINATES (cfs)
Output Time increment = .0500 hrs
Time on left represents time for first value in each row.

Time hrs					
22.0000	1.27	1.27	1.27	1.26	1.26
22.2500	1.26	1.26	1.25	1.25	1.25
22.5000	1.25	1.24	1.24	1.24	1.23
22.7500	1.23	1.23	1.23	1.22	1.22
23.0000	1.22	1.21	1.21	1.21	1.21
23.2500	1.20	1.20	1.20	1.20	1.19
23.5000	1.19	1.19	1.19	1.18	1.18
23.7500	1.18	1.18	1.17	1.17	1.17
24.0000	1.16	.93	.41	.14	.05
24.2500	.02	.00	.00		

SUMMARY FOR HYDROGRAPH ADDITION
at Node: POND 10 IN

HYG Directory: J:\PROJECT\2004\046119BelewsCreek\Pond Files\

Upstream Link ID	Upstream Node ID	HYG file	HYG ID	HYG tag
ADDLINK 10	SUBAREA 10		SUBAREA 10	100

INFLOWS TO: POND 10 IN

HYG file	HYG ID	HYG tag	Volume ac-ft	Peak Time hrs	Peak Flow cfs
	SUBAREA 10	100	9.144	11.9500	150.01

TOTAL FLOW INTO: POND 10 IN

HYG file	HYG ID	HYG tag	Volume ac-ft	Peak Time hrs	Peak Flow cfs
	POND 10 IN	100	9.144	11.9500	150.01

TOTAL NODE INFLOW...
HYG file =
HYG ID = POND 10 IN
HYG Tag = 100

Peak Discharge = 150.01 cfs
Time to Peak = 11.9500 hrs
HYG Volume = 9.144 ac-ft

HYDROGRAPH ORDINATES (cfs)
Output Time increment = .0500 hrs
Time on left represents time for first value in each row.

Time hrs					
2.5500	.00	.00	.01	.03	.04
2.8000	.05	.07	.08	.10	.11
3.0500	.12	.14	.15	.17	.18
3.3000	.20	.21	.22	.24	.25
3.5500	.27	.28	.29	.31	.32
3.8000	.34	.35	.36	.38	.39
4.0500	.40	.42	.43	.45	.46
4.3000	.48	.50	.51	.53	.54
4.5500	.56	.58	.59	.61	.62
4.8000	.64	.66	.67	.69	.71
5.0500	.72	.74	.76	.77	.79
5.3000	.81	.83	.84	.86	.88
5.5500	.89	.91	.93	.95	.96
5.8000	.98	1.00	1.02	1.03	1.05
6.0500	1.07	1.09	1.11	1.12	1.14
6.3000	1.16	1.18	1.20	1.21	1.23
6.5500	1.25	1.27	1.28	1.30	1.32
6.8000	1.34	1.36	1.38	1.39	1.41
7.0500	1.43	1.45	1.47	1.49	1.50
7.3000	1.52	1.54	1.56	1.58	1.59
7.5500	1.61	1.63	1.65	1.67	1.69
7.8000	1.70	1.72	1.74	1.76	1.78
8.0500	1.80	1.84	1.87	1.93	1.97
8.3000	2.03	2.08	2.14	2.19	2.25
8.5500	2.30	2.36	2.41	2.47	2.53
8.8000	2.59	2.65	2.71	2.76	2.83
9.0500	2.88	2.92	2.95	2.97	2.98
9.3000	3.00	3.01	3.02	3.04	3.05
9.5500	3.08	3.13	3.19	3.28	3.36
9.8000	3.46	3.55	3.65	3.73	3.84
10.0500	3.93	4.05	4.17	4.32	4.44

HYDROGRAPH ORDINATES (cfs)
Output Time increment = .0500 hrs
Time on left represents time for first value in each row.

Time hrs					
10.3000	4.59	4.73	4.88	5.01	5.17
10.5500	5.32	5.52	5.71	5.95	6.17
10.8000	6.42	6.64	6.91	7.13	7.39
11.0500	7.68	8.09	8.52	9.10	9.61
11.3000	10.23	10.77	11.39	11.94	12.63
11.5500	15.73	22.32	31.27	45.20	59.25
11.8000	78.06	102.67	140.22	150.01	133.52
12.0500	105.16	59.16	35.36	26.14	22.14
12.3000	19.70	18.08	16.48	15.16	13.61
12.5500	12.51	11.46	10.83	10.37	10.03
12.8000	9.66	9.36	9.00	8.70	8.34
13.0500	8.07	7.79	7.58	7.37	7.21
13.3000	7.01	6.85	6.66	6.49	6.30
13.5500	6.15	5.98	5.85	5.71	5.59
13.8000	5.45	5.34	5.20	5.08	4.94
14.0500	4.85	4.75	4.68	4.62	4.58
14.3000	4.53	4.49	4.44	4.40	4.36
14.5500	4.32	4.26	4.22	4.18	4.14
14.8000	4.09	4.05	4.00	3.96	3.91
15.0500	3.87	3.82	3.78	3.73	3.69
15.3000	3.65	3.61	3.55	3.51	3.47
15.5500	3.43	3.37	3.33	3.29	3.25
15.8000	3.20	3.16	3.11	3.07	3.02
16.0500	2.98	2.95	2.93	2.91	2.89
16.3000	2.88	2.86	2.84	2.83	2.81
16.5500	2.80	2.78	2.76	2.75	2.73
16.8000	2.71	2.70	2.68	2.67	2.65
17.0500	2.64	2.62	2.61	2.59	2.57
17.3000	2.56	2.54	2.52	2.51	2.49
17.5500	2.48	2.46	2.45	2.43	2.42
17.8000	2.40	2.38	2.37	2.35	2.33
18.0500	2.32	2.30	2.29	2.27	2.25
18.3000	2.24	2.22	2.21	2.19	2.18
18.5500	2.16	2.14	2.13	2.11	2.10
18.8000	2.08	2.06	2.05	2.03	2.01
19.0500	2.00	1.98	1.97	1.95	1.94
19.3000	1.92	1.91	1.89	1.87	1.86
19.5500	1.84	1.82	1.81	1.79	1.78
19.8000	1.76	1.74	1.73	1.71	1.69
20.0500	1.68	1.67	1.67	1.66	1.66
20.3000	1.65	1.65	1.65	1.64	1.64
20.5500	1.64	1.63	1.63	1.63	1.63
20.8000	1.62	1.62	1.62	1.61	1.61
21.0500	1.60	1.60	1.60	1.60	1.59
21.3000	1.59	1.59	1.58	1.58	1.58

HYDROGRAPH ORDINATES (cfs)
Output Time increment = .0500 hrs
Time on left represents time for first value in each row.

Time hrs					
21.5500	1.58	1.57	1.57	1.57	1.56
21.8000	1.56	1.55	1.55	1.55	1.54
22.0500	1.54	1.54	1.54	1.53	1.53
22.3000	1.53	1.52	1.52	1.52	1.51
22.5500	1.51	1.51	1.50	1.50	1.50
22.8000	1.49	1.49	1.49	1.49	1.48
23.0500	1.48	1.48	1.47	1.47	1.47
23.3000	1.46	1.46	1.46	1.45	1.45
23.5500	1.45	1.44	1.44	1.44	1.44
23.8000	1.43	1.43	1.43	1.42	1.41
24.0500	1.13	.50	.17	.06	.02
24.3000	.01	.00			

SUMMARY FOR HYDROGRAPH ADDITION
at Node: POND 10 IN

HYG Directory: J:\PROJECT\2004\046119BelewsCreek\Pond Files\

Upstream Link ID	Upstream Node ID	HYG file	HYG ID	HYG tag
ADDLINK 10	SUBAREA 10		SUBAREA 10	2YR

INFLOWS TO: POND 10 IN

HYG file	HYG ID	HYG tag	Volume ac-ft	Peak Time hrs	Peak Flow cfs
	SUBAREA 10	2YR	3.384	11.9500	58.73

TOTAL FLOW INTO: POND 10 IN

HYG file	HYG ID	HYG tag	Volume ac-ft	Peak Time hrs	Peak Flow cfs
	POND 10 IN	2YR	3.384	11.9500	58.73

TOTAL NODE INFLOW...
HYG file =
HYG ID = POND 10 IN
HYG Tag = 2YR

Peak Discharge = 58.73 cfs
Time to Peak = 11.9500 hrs
HYG Volume = 3.384 ac-ft

HYDROGRAPH ORDINATES (cfs)
Output Time increment = .0500 hrs
Time on left represents time for first value in each row.

Time hrs					
5.0500	.00	.00	.00	.01	.01
5.3000	.02	.02	.03	.03	.04
5.5500	.04	.05	.05	.06	.06
5.8000	.07	.08	.08	.09	.09
6.0500	.10	.10	.11	.11	.12
6.3000	.13	.13	.14	.14	.15
6.5500	.16	.16	.17	.17	.18
6.8000	.19	.19	.20	.21	.21
7.0500	.22	.23	.23	.24	.25
7.3000	.25	.26	.27	.27	.28
7.5500	.29	.29	.30	.31	.31
7.8000	.32	.33	.33	.34	.35
8.0500	.36	.37	.38	.39	.41
8.3000	.42	.44	.45	.47	.48
8.5500	.50	.52	.53	.55	.57
8.8000	.59	.61	.63	.64	.66
9.0500	.68	.70	.71	.72	.73
9.3000	.74	.75	.76	.76	.77
9.5500	.79	.80	.82	.85	.88
9.8000	.91	.94	.97	1.00	1.04
10.0500	1.07	1.11	1.15	1.20	1.24
10.3000	1.29	1.34	1.39	1.44	1.49
10.5500	1.54	1.61	1.68	1.76	1.84
10.8000	1.92	2.00	2.10	2.18	2.27
11.0500	2.38	2.52	2.68	2.88	3.06
11.3000	3.28	3.48	3.71	3.92	4.17
11.5500	5.25	7.53	10.71	15.75	21.08
11.8000	28.42	38.36	53.77	58.73	53.13
12.0500	42.33	23.95	14.39	10.69	9.08
12.3000	8.10	7.45	6.79	6.26	5.62
12.5500	5.17	4.74	4.48	4.30	4.16

HYDROGRAPH ORDINATES (cfs)
Output Time increment = .0500 hrs
Time on left represents time for first value in each row.

Time hrs					
12.8000	4.01	3.88	3.74	3.62	3.47
13.0500	3.36	3.24	3.16	3.07	3.00
13.3000	2.92	2.86	2.78	2.71	2.63
13.5500	2.57	2.50	2.44	2.39	2.34
13.8000	2.28	2.23	2.18	2.13	2.07
14.0500	2.03	1.99	1.96	1.94	1.92
14.3000	1.90	1.88	1.86	1.85	1.83
14.5500	1.81	1.79	1.77	1.76	1.74
14.8000	1.72	1.70	1.68	1.67	1.64
15.0500	1.63	1.61	1.59	1.57	1.55
15.3000	1.53	1.52	1.50	1.48	1.46
15.5500	1.44	1.42	1.40	1.39	1.37
15.8000	1.35	1.33	1.31	1.30	1.27
16.0500	1.26	1.24	1.24	1.23	1.22
16.3000	1.21	1.21	1.20	1.19	1.19
16.5500	1.18	1.17	1.17	1.16	1.16
16.8000	1.15	1.14	1.13	1.13	1.12
17.0500	1.11	1.11	1.10	1.09	1.09
17.3000	1.08	1.08	1.07	1.06	1.06
17.5500	1.05	1.04	1.04	1.03	1.02
17.8000	1.01	1.01	1.00	1.00	.99
18.0500	.98	.98	.97	.96	.96
18.3000	.95	.94	.93	.93	.92
18.5500	.92	.91	.90	.90	.89
18.8000	.88	.87	.87	.86	.85
19.0500	.85	.84	.84	.83	.82
19.3000	.81	.81	.80	.79	.79
19.5500	.78	.77	.77	.76	.75
19.8000	.75	.74	.73	.73	.72
20.0500	.71	.71	.71	.70	.70
20.3000	.70	.70	.70	.70	.70
20.5500	.70	.69	.69	.69	.69
20.8000	.69	.69	.69	.69	.68
21.0500	.68	.68	.68	.68	.68
21.3000	.68	.68	.67	.67	.67
21.5500	.67	.67	.67	.67	.67
21.8000	.66	.66	.66	.66	.66
22.0500	.66	.66	.65	.65	.65
22.3000	.65	.65	.65	.65	.65
22.5500	.64	.64	.64	.64	.64
22.8000	.64	.64	.63	.63	.63
23.0500	.63	.63	.63	.63	.62
23.3000	.62	.62	.62	.62	.62
23.5500	.62	.62	.61	.61	.61
23.8000	.61	.61	.61	.61	.60

HYDROGRAPH ORDINATES (cfs)
Output Time increment = .0500 hrs
Time on left represents time for first value in each row.

Time hrs					
24.0500	.48	.21	.07	.02	.01
24.3000	.00	.00			

LEVEL POOL ROUTING SUMMARY

```
HYG Dir          = J:\PROJECT\2004\046119BelewsCreek\Pond Files\
Inflow  HYG file = NONE STORED - POND 10      IN  25
Outflow HYG file = NONE STORED - POND 10      OUT 25

Pond Node   Data = POND 10
Pond Volume Data = POND 10
Pond Outlet Data = Outlet 1

No Infiltration

INITIAL CONDITIONS
----------------------------------
Starting WS Elev   =   750.00 ft
Starting Volume    =     .000 ac-ft
Starting Outflow   =      .00 cfs
Starting Infiltr.  =      .00 cfs
Starting Total Qout=      .00 cfs
Time Increment     =    .0500 hrs


INFLOW/OUTFLOW HYDROGRAPH SUMMARY
=====================================================
Peak Inflow       =    121.73 cfs    at   11.9500 hrs
Peak Outflow      =      1.12 cfs    at   23.0000 hrs
-----------------------------------------------------
Peak Elevation    =    757.74 ft
Peak Storage =          5.930 ac-ft
=====================================================


MASS BALANCE (ac-ft)
--------------------------
+ Initial Vol  =        .000
+ HYG Vol IN   =       7.320
- Infiltration =        .000
- HYG Vol OUT  =       7.060
- Retained Vol =        .260
                  ----------
Unrouted Vol =       -.000 ac-ft  (.005% of Inflow Volume)
```

LEVEL POOL ROUTING SUMMARY

```
HYG Dir          = J:\PROJECT\2004\046119BelewsCreek\Pond Files\
Inflow  HYG file = NONE STORED - POND 10      IN  100
Outflow HYG file = NONE STORED - POND 10      OUT 100

Pond Node   Data = POND 10
Pond Volume Data = POND 10
Pond Outlet Data = Outlet 1

No Infiltration

INITIAL CONDITIONS
----------------------------------
Starting WS Elev   =   750.00 ft
Starting Volume    =     .000 ac-ft
Starting Outflow   =      .00 cfs
Starting Infiltr.  =      .00 cfs
Starting Total Qout=      .00 cfs
Time Increment     =    .0500 hrs


INFLOW/OUTFLOW HYDROGRAPH SUMMARY
=====================================================
Peak Inflow       =    150.01 cfs    at   11.9500 hrs
Peak Outflow      =      2.58 cfs    at   17.2000 hrs
-----------------------------------------------------
Peak Elevation    =    758.58 ft
Peak Storage =          7.081 ac-ft
=====================================================


MASS BALANCE (ac-ft)
---------------------------
+ Initial Vol  =        .000
+ HYG Vol IN   =       9.144
- Infiltration =        .000
- HYG Vol OUT  =       8.362
- Retained Vol =        .782
                 ----------
Unrouted Vol =       -.000 ac-ft  (.003% of Inflow Volume)
```

LEVEL POOL ROUTING SUMMARY

```
HYG Dir          = J:\PROJECT\2004\046119BelewsCreek\Pond Files\
Inflow  HYG file = NONE STORED - POND 10      IN  2YR
Outflow HYG file = NONE STORED - POND 10      OUT 2YR

Pond Node   Data = POND 10
Pond Volume Data = POND 10
Pond Outlet Data = Outlet 1

No Infiltration

INITIAL CONDITIONS
----------------------------------
Starting WS Elev   =   750.00 ft
Starting Volume    =     .000 ac-ft
Starting Outflow   =      .00 cfs
Starting Infiltr.  =      .00 cfs
Starting Total Qout=      .00 cfs
Time Increment     =    .0500 hrs


INFLOW/OUTFLOW HYDROGRAPH SUMMARY
=====================================================
Peak Inflow       =     58.73 cfs    at   11.9500 hrs
Peak Outflow      =       .87 cfs    at   17.2500 hrs
-----------------------------------------------------
Peak Elevation    =    754.84 ft
Peak Storage =          2.455 ac-ft
=====================================================


MASS BALANCE (ac-ft)
--------------------------
+ Initial Vol  =        .000
+ HYG Vol IN   =       3.384
- Infiltration =        .000
- HYG Vol OUT  =       3.384
- Retained Vol =        .000
                 ----------
Unrouted Vol =       -.000 ac-ft  (.001% of Inflow Volume)
```

Index of Starting Page Numbers for ID Names

----- C -----
CMLDS-24HR... 3.01, 3.02

----- O -----
Outlet 1... 8.01, 8.04, 8.22

----- P -----
POND 10... 7.01, 9.01
POND 10 IN 25... 9.07, 9.11, 9.15, 9.19, 9.20, 9.21

----- S -----
SUBAREA 10... 4.01, 5.01, 6.03, 6.04, 6.05

----- W -----
Watershed... 1.01, 2.01, 2.02, 2.03, 2.05

Comp. By DHB Date 4-19-06 Project BELEWS CREEK LANDFILL Sheet No. ..... of .....

Chkd. By DEB Date 4-19-06 Subject STORMWATER POND Job No. 046119

ANTI-FLOTATION BLOCK

$F\uparrow = \frac{\pi}{4}(2)^2(8.4)(62.4) = 1{,}647 \text{ lb}\uparrow$

VOL. CONC. REQ'D $= 1.1\left(\frac{1647}{150}\right) = \cancel{12.1}\ 20.7 \text{ ft}^3$

PROVIDE CONC. BTG. $\cancel{3.5'} \times \cancel{3.5'}\ 5' \times 5' \times 1'$ DEEP $= \cancel{12.3}\ 25 \text{ ft}^3$

$F\downarrow = \cancel{12.3}\ 20.7\left(\cancel{150}\ 87.6\right) = \cancel{1838}\ 1813 \text{ lb}\downarrow$

$F.S. = \frac{\cancel{2400}\ 1813}{1647} = \cancel{1.46}\ 1.1$ ~~OK.~~ OK

Comp. By DHB Date 4-19-06 Project BELEWS CREEK LANDFILL Sheet No. ..... of .....

Chkd. By ..... Date ..... Subject NEW CHANNEL CALCS. Job No 046119

PD-55

D.A. = 4.0 Ac.
C = .60
I_{10} = 6.8 in/hr.
Q_{10} = 0.6(6.8)(4.0) = 16.3 cfs.
S = 1.0 %
Q_2 = 0.6(4.8)(4.0) = 11.5 cfs.

PD-56

D.A. = 0.5 Ac.
C = .60
I_{10} = 6.8 in/hr.
I_2 = 4.8 in/hr.
Q_2 = 0.6(4.8)(0.5) = 1.44 cfs.
Q_{10} = 0.6(6.8)(0.5) = 2.04 cfs.
S = 5%.

Worksheet
Worksheet for Trapezoidal Channel

Project Description	
Worksheet	PD-55
Flow Element	Trapezoidal Cha
Method	Manning's Form
Solve For	Channel Depth

Input Data		
Mannings Coeffic	0.033	
Slope	010000	ft/ft
Left Side Slope	3.00	H : V
Right Side Slope	3.00	H : V
Bottom Width	1.00	ft
Discharge	11.50	cfs

Results		
Depth	0.98	ft
Flow Area	3.9	ft²
Wetted Perim	7.20	ft
Top Width	6.88	ft
Critical Depth	0.83	ft
Critical Slope	0.021413	ft/ft
Velocity	2.97	ft/s
Velocity Head	0.14	ft
Specific Energ	1.12	ft
Froude Numb	0.70	
Flow Type	Subcritical	

STRAW W/ NET TEMP. LINER

$n = .033$

$\tau_d = \gamma ds = 62.4(.98)(.01) = 0.61$ lb/ft²

< 1.45 lb/ft²

OK.

Worksheet
Worksheet for Trapezoidal Channel

Project Description	
Worksheet	PD-55
Flow Element	Trapezoidal Cha
Method	Manning's Form
Solve For	Channel Depth

Input Data		
Mannings Coeffic	0.048	
Slope	010000	ft/ft
Left Side Slope	3.00	H : V
Right Side Slope	3.00	H : V
Bottom Width	1.00	ft
Discharge	16.30	cfs

Results		
Depth	1.33	ft
Flow Area	6.6	ft²
Wetted Perim	9.42	ft
Top Width	8.98	ft
Critical Depth	0.98	ft
Critical Slope	0.043268	ft/ft
Velocity	2.45	ft/s
Velocity Head	0.09	ft
Specific Energ	1.42	ft
Froude Numb	0.50	
Flow Type	Subcritical	

TALL FESCUE GRASS

$n = .048$

$R_h = \frac{6.6}{9.42} = 0.7$ ft.

$V_A * R_h = 1.72$

$n = .048$ VALID.

GEOMETRY: $B = 1$

$D = 2$

$Z = 3$

$T = 13$

Worksheet
Worksheet for Trapezoidal Channel

Project Description	
Worksheet	PD-56
Flow Element	Trapezoidal Cha
Method	Manning's Form
Solve For	Channel Depth

Input Data		
Mannings Coeffic	0.065	
Slope	050000	ft/ft
Left Side Slope	3.00	H : V
Right Side Slope	3.00	H : V
Bottom Width	1.00	ft
Discharge	2.04	cfs

Results		
Depth	0.43	ft
Flow Area	1.0	ft²
Wetted Perim	3.70	ft
Top Width	3.56	ft
Critical Depth	0.36	ft
Critical Slope	0.104619	ft/ft
Velocity	2.10	ft/s
Velocity Head	0.07	ft
Specific Energ	0.49	ft
Froude Numb	0.71	
Flow Type	Subcritical	

STRAW W/ NET TEMP. LINER

$n = .065$

$\tau_d = 62.4(.43)(.05) = 1.34$ lb/ft

< 1.45 lb/ft

OK

Worksheet
Worksheet for Trapezoidal Channel

Project Description	
Worksheet	PD-56
Flow Element	Trapezoidal Cha
Method	Manning's Form
Solve For	Channel Depth

Input Data		
Mannings Coeffic	0.080	
Slope	050000	ft/ft
Left Side Slope	3.00	H : V
Right Side Slope	3.00	H : V
Bottom Width	1.00	ft
Discharge	2.04	cfs

Results		
Depth	0.47	ft
Flow Area	1.1	ft²
Wetted Perim	3.97	ft
Top Width	3.82	ft
Critical Depth	0.36	ft
Critical Slope	0.158472	ft/ft
Velocity	1.80	ft/s
Velocity Head	0.05	ft
Specific Energ	0.52	ft
Froude Numb	0.58	
Flow Type	Subcritical	

TALL FESCUE GRASS

$n = .080$

$R_h = \frac{1.1}{3.97} = 0.28$ ft.

$V_A * R_h = 0.5$

$n = .080$ OK

GEOMETRY: $B = 1$

$D = 1$

$Z = 3$

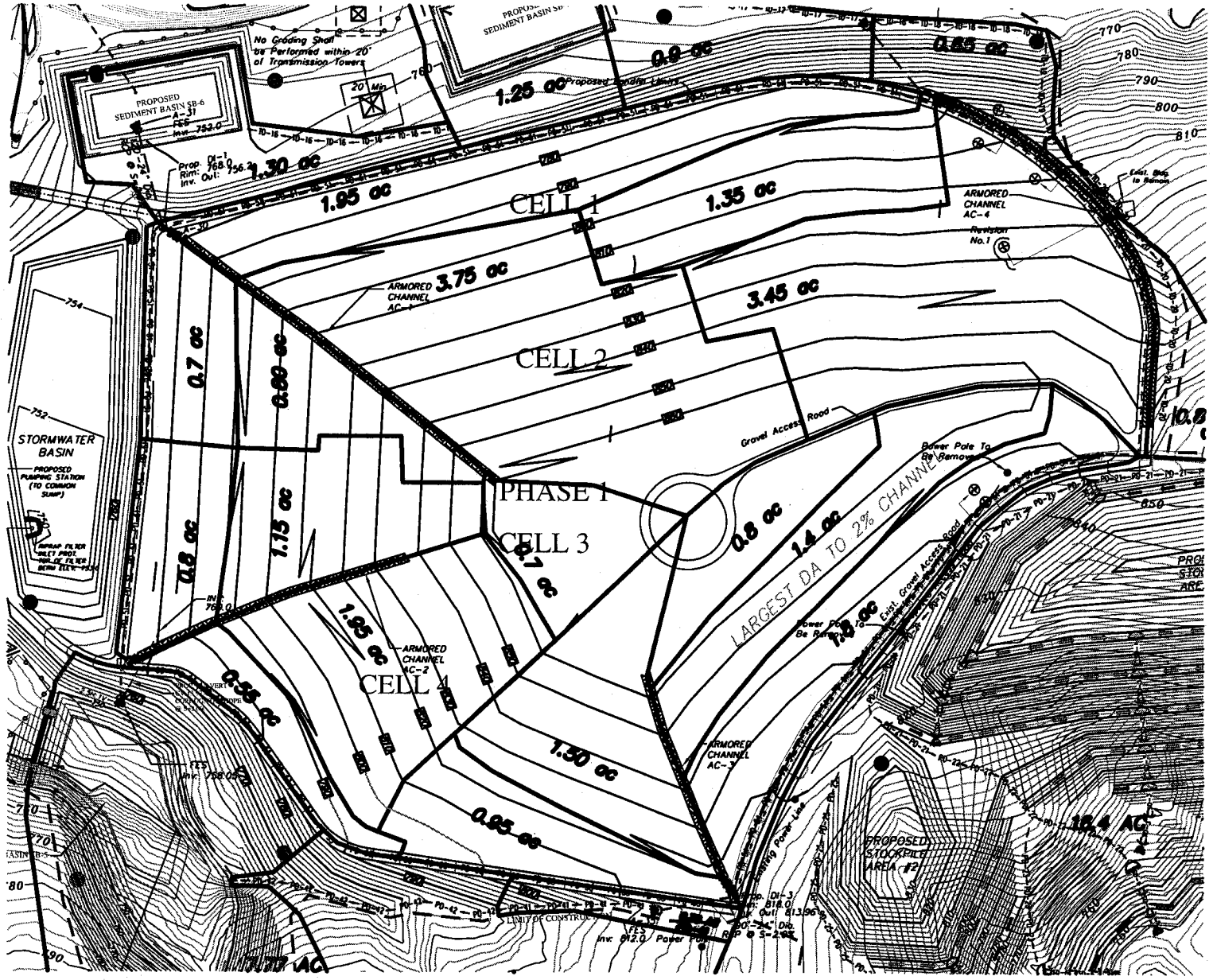
$T = 7$

Ac-1
3.75
0.70
0.80
5.25 Ac.

Ac-2
1.15
1.95
3.10 Ac.

Ac-3
1.50
0.80
1.40
3.70 Ac.

Ac-4
3.45 Ac.



DI-1
5.25
1.95
.70
7.90 Ac.

DI-2
3.10
0.80
0.55
0.95
5.40 Ac.

DI-3
3.70
1.80
5.50 Ac.

DI-4
3.45
1.35
4.80 Ac.

STORM DRAINAGE AREAS
SCALE: 1" = 200'

Comp. By DHB Date 4-19-06 Project BELEWS CREEK LANDFILL Sheet No. ..... of .....

Chkd. By ..... Date ..... Subject ..... Job No D46119

ARMORED CHANNEL CALCS.

ARMORED CHANNEL AC-1

D.A. = 5.25 Ac.
C = .60
I = 6.8 in/hr
Q_{10} = .60 (6.8) (5.25) = 21.4 cfs.
S = 17.3 %

ARMORED CHANNEL AC-2

D.A. = 3.10
C = .60
I = 6.8 in/hr
Q_{10} = 0.6 (6.8) (3.10) = 12.6 cfs.
S = 18.8 %

ARMORED CHANNEL AC-3

D.A. = 3.70
C = .60
I = 6.8 in/hr
Q_{10} = 0.6 (6.8) (3.70) = 15.1 cfs.
S = 15.8 %

ARMORED CHANNEL AC-4

D.A. = 3.45
C = .60
I = 6.8 in/hr
Q_{10} = 0.6 (6.8) (3.45) = 14.1 cfs.
S = 11.75 %

Comp. By DHB Date 4-19-06 Project BELEWS CREEK LANDFILL Sheet No ..... of .....

Chkd. By ..... Date ..... Subject STORM DRAINAGE CALCS. Job No 046119

DI-1 D.A. = 7.90 Ac.
C = .60
I_{25} = 7.2 in/hr.
Q_{10} = 0.6 (6.8)(7.9) = 32.2 cfs.
Q_{25} = 0.6 (7.2)(7.9) = 34.1 cfs.

DI-2 D.A. = 5.40 Ac.
C = .60
I_{25} = 7.2 in/hr.
Q_{25} = 0.6 (7.2)(5.4) = 23.3 cfs.

DI-3 D.A. = 5.50 Ac.
C = .60
I_{25} = 7.2 in/hr.
Q_{25} = 0.6 (7.2)(5.5) = 23.8 cfs.

DI-4 D.A. = 4.80 Ac.
C = .60
I_{25} = 7.2 in/hr.
Q_{25} = 0.6 (7.2)(4.8) = 20.7 cfs.

Worksheet
Worksheet for Circular Channel

Project Description	
Worksheet	CULVERT @ I72 - 1
Flow Element	Circular Chann
Method	Manning's Forr
Solve For	Channel Depth

Input Data		
Mannings Coeffic	0.013	
Slope	035000	ft/ft
Diameter	24	in
Discharge	34.10	cfs

Results		
Depth	1.36	ft
Flow Area	2.3	ft²
Wetted Perime	3.88	ft
Top Width	1.87	ft
Critical Depth	1.91	ft
Percent Full	68.0	%
Critical Slope	0.019744	ft/ft
Velocity	14.99	ft/s
Velocity Head	3.49	ft
Specific Energy	4.85	ft
Froude Numbe	2.39	
Maximum Disc	45.52	cfs
Discharge Full	42.32	cfs
Slope Full	0.022724	ft/ft
Flow Type	Supercritical	

Worksheet
Worksheet for Circular Channel

Project Description	
Worksheet	CULVERT @ DI-2
Flow Element	Circular Chann
Method	Manning's Forr
Solve For	Channel Depth

Input Data		
Mannings Coeffic	0.013	
Slope	040000	ft/ft
Diameter	24	in
Discharge	23.30	cfs

Results		
Depth	1.02	ft
Flow Area	1.6	ft²
Wetted Perime	3.18	ft
Top Width	2.00	ft
Critical Depth	1.72	ft
Percent Full	50.9	%
Critical Slope	0.009863	ft/ft
Velocity	14.51	ft/s
Velocity Head	3.27	ft
Specific Energ	4.29	ft
Froude Numbe	2.85	
Maximum Disc	48.67	cfs
Discharge Full	45.24	cfs
Slope Full	0.010609	ft/ft
Flow Type	Supercritical	

Worksheet
Worksheet for Circular Channel

Project Description	
Worksheet	CULVERT @DI-3
Flow Element	Circular Chann
Method	Manning's Forr
Solve For	Channel Depth

Input Data		
Mannings Coeffic	0.013	
Slope	020000	ft/ft
Diameter	24	in
Discharge	23.80	cfs

Results		
Depth	1.29	ft
Flow Area	2.1	ft²
Wetted Perime	3.72	ft
Top Width	1.92	ft
Critical Depth	1.73	ft
Percent Full	64.3	%
Critical Slope	0.010174	ft/ft
Velocity	11.16	ft/s
Velocity Head	1.93	ft
Specific Energ	3.22	ft
Froude Numbe	1.86	
Maximum Disc	34.41	cfs
Discharge Full	31.99	cfs
Slope Full	0.011069	ft/ft
Flow Type	Supercritical	

Worksheet
Worksheet for Circular Channel

Project Description	
Worksheet	CULVERT @ VI-4
Flow Element	Circular Chann
Method	Manning's Forr
Solve For	Channel Depth

Input Data		
Mannings Coeffic	0.013	
Slope	040000	ft/ft
Diameter	24	in
Discharge	20.70	cfs

Results		
Depth	0.95	ft
Flow Area	1.5	ft²
Wetted Perime	3.04	ft
Top Width	2.00	ft
Critical Depth	1.63	ft
Percent Full	47.5	%
Critical Slope	0.008445	ft/ft
Velocity	14.08	ft/s
Velocity Head	3.08	ft
Specific Energy	4.03	ft
Froude Numbe	2.89	
Maximum Disc	48.67	cfs
Discharge Full	45.24	cfs
Slope Full	0.008374	ft/ft
Flow Type	Supercritical	

Comp. By DHB Date 4-19-06 Project BIELENS CREEK LANDFILL Sheet No. ..... of .....

Chkd. By ..... Date ..... Subject ..... Job No 046119

APRON CALCS.

A-31 @ AC-1

DOWNSTREAM CULVERT DIA. = 24"

Q_{10} = 21.4 cfs

L_A = 14 ft.

W = 16 ft

d_{50} RIPRAP = 6"

d_{max} = 9"

t = 13.5"

H = 16"

A-32 @ 24" ø CULVERT OUTLET D.S. OF DI-1

Q_{10} = 32.2 cfs.

L_A = 20 ft.

W = 22 ft.

d_{50} RIPRAP = 9"

d_{max} = 13.5"

t = 20.25"

H = 16"

A-33 @ AC-2

Q_{10} = 12.6 cfs.

DOWNSTREAM CULVERT ø = 24"

L_A = 12 ft.

W = 14 ft.

d_{50} RIPRAP = 6"

d_{max} = 9"

t = 13.5"

H = 16"

Comp. By DHB Date 4-19-06 Project BELEWS CREEK LANDFILL Sheet No. ..... of .....

Chkd. By ..... Date ..... Subject ..... Job No 046119

APRON CALCS.

A-34 @ 24" ø CULVERT D.S. OF DI-2

$Q_{10} = 0.6(6.8)(5.4) = 22$ cfs.

$L_A = 14$ ft

$W = 16$ ft.

d_{50} RIPRAP = 6"

$d_{max} = 9"$

$t = 13.5"$

$H = 16"$

A-35 @ AC-3

DOWNSTREAM PIPE ø = 24"

$Q_{10} = 15.1$ cfs.

$L_A = 12$ ft

$W = 14$ ft

d_{50} RIPRAP = 6"

$d_{max} = 9"$

$t = 13.5"$

$H = 16"$

A-36 @ 24" ø CULVERT D.S. OF DI-3

$Q_{10} = 0.6(6.8)(5.5) = 22.4$ cfs.

$L_A = 14$ ft

$W = 16$ ft

d_{50} RIPRAP = 6"

$d_{max} = 9"$

$t = 13.5"$

$H = 16"$

Comp. By DHB Date 4-19-06 Project BEEWS CREEK LANDFILL Sheet No. ..... of .....

Chkd. By ..... Date ..... Subject ..... Job No. 046119

APRON CALCS.

A-37 @ AC-4

DOWNSTREAM PIPE ϕ = 24"

Q_{10} = 14.1 cfs.

L_A = 12 ft

W = 14 ft.

d_{50} RIPRAP = 6"

d_{max} = 9"

t = 13.5"

H = 16"

A-38 @ 24" ϕ CULVERT OUTLET D.S. OF DI-4

Q_{10} = 0.6 (6.8)(4.8 Ac). = 19.6 cfs.

L_A = 13 ft.

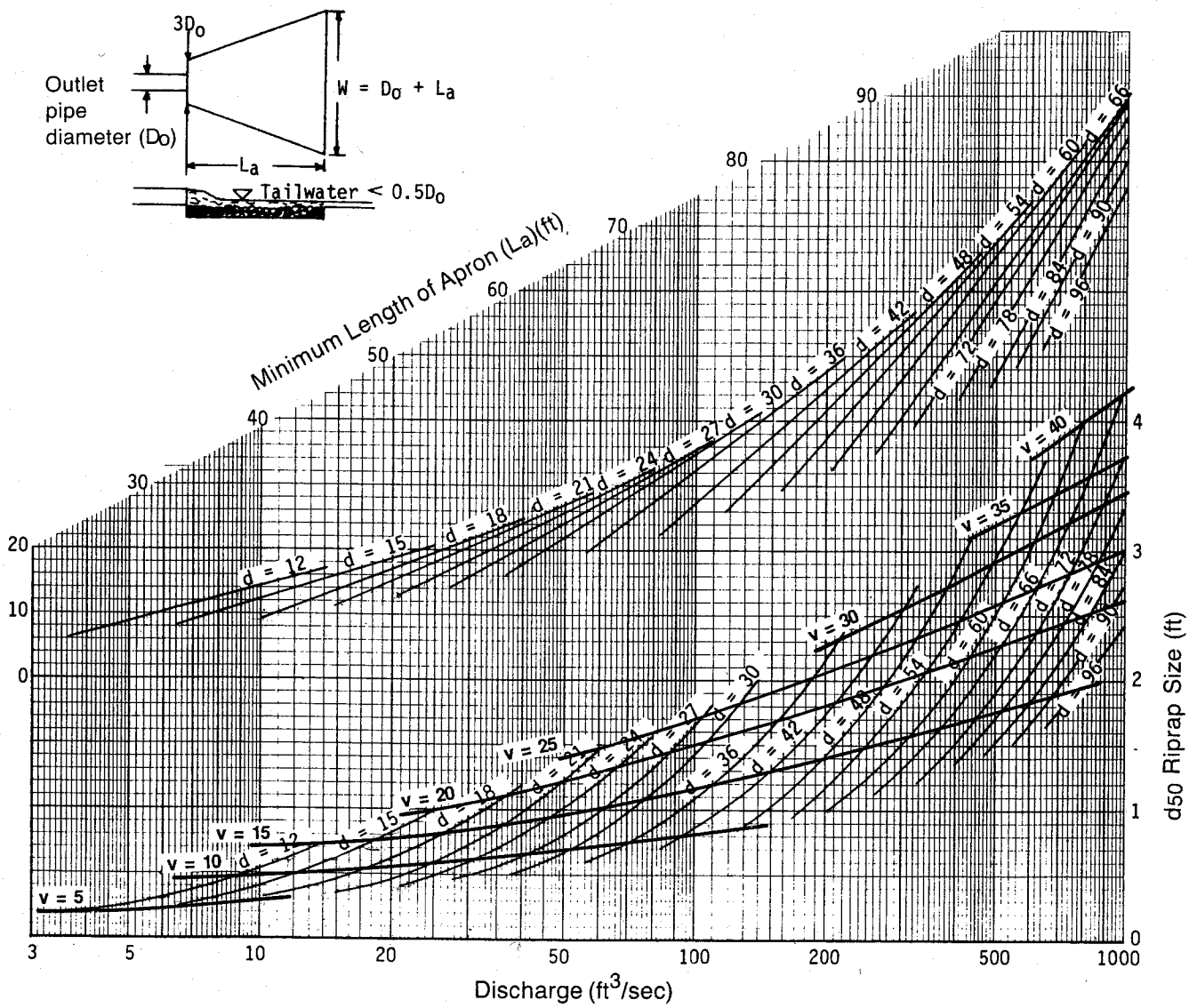
W = 15 ft.

d_{50} RIPRAP = 6"

d_{max} = 9"

t = 13.5"

H = 24"



Curves may not be extrapolated.

Figure 8.06a Design of outlet protection protection from a round pipe flowing full, minimum tailwater condition (T_w < 0.5 diameter).

Belews Creek FGD Residue Landfill	**Job No.**	**046119**		
Design for Cell Stormwater	**Comp. By**	**WMM**	**Date**	**3/10/2006**
	Chkd. By	**DEB**	**Date**	**4/17/2006**
	Sheet	**1**	**of**	

Objective: **Design Riser and Culverts for Landfill Cell**
For 25 Year - 24 Hour Storm
Landfill Cell 1

Reference (1) *NC Sedimentation and Erosion Control Manual*

25 year 24 hour precipitation = 5.6 in Reference Fig 8.03k, Reference 1

Use Rational Method to determine peak runoff in cfs

Q = CiA

Q = peak rate of runoff in cubic feet per second (cfs)
C = runoff coefficient
i = average intensity of rainfall in inches/hour, for a storm duration equal to the time of concentration, Tc
Tc = time of concentration, minutes
A = drainage area in acres

Determine drainage area:

Drainage area for Cell 1 consists of Cell 1 only.

A, area = 5.7 ac

Determine the runoff coefficient, C:

Since portion of Cell 1 with exposed HDPE cover is small, ignore and treat entire Cell as single unit.

Use C for bare, packed, smooth soil. Reference Table 8.03a

C = 0.6

Find Time of Concentration, Tc:

maximum length of travel = 1160 ft
height of most remote point above outlet = 42 ft

Tc = 6.5 min Reference Fig. 8.03a

Belews Creek FGD Residue Landfill	**Job No.**	**046119**		
Design for Cell Stormwater	**Comp. By**	**WMM**	**Date**	**3/10/2006**
	Chkd. By	**DEB**	**Date**	**4/17/2006**
	Sheet	2	**of**	

Determine rainfall intensity factor, i:

Use 25 year storm with Tc from above.

i = 7 in/hour

Calculate Peak Flow, Qp:

Qp = C i A

Qp = 23.9 cfs

Belews Creek FGD Residue Landfill	**Job No.**	**046119**		
Design for Cell Stormwater	**Comp. By**	**WMM**	**Date**	**3/10/2006**
	Chkd. By	**DEB**	**Date**	**4/17/2006**
	Sheet	3	**of**	

Objective: **Design Riser and Culverts for Landfill Cell**
For 25 Year - 24 Hour Storm
Landfill Cell 2

Reference (1) *NC Sedimentation and Erosion Control Manual*

25 year 24 hour precipitation = **5.6 in** Reference Fig 8.03k, Reference 1

Use Rational Method to determine peak runoff in cfs

Q = CiA

Q = peak rate of runoff in cubic feet per second (cfs)
C = runoff coefficient
i = average intensity of rainfall in inches/hour, for a storm duration equal to the time of concentration, Tc
Tc = time of concentration, minutes
A = drainage area in acres

Determine drainage area:

The drainage area for the Cell 2 riser consists of the area in Cell 2 and a portion of the area for Cell 1, since the open portion of Cell 1 drains into Cell 2.

Area Cell 2	=	7.0 ac	Cell 2 only
Total Area	=	9.3 ac	includes a portion of Cell 1

Calculate area weighted C term:

For HDPE area (assumed)
C = 0.99

Use C for bare, packed, smooth soil. Reference Table 8.03a

C = 0.6

	Area		C		
HDPE	1.89	x	0.99	=	1.88
gypsum, soil	7.45	x	0.6	=	4.47
	9.35				6.35

Weighted C = 0.68

Belews Creek FGD Residue Landfill	**Job No.**	**046119**		
Design for Cell Stormwater	**Comp. By**	**WMM**	**Date**	**3/10/2006**
	Chkd. By	**DEB**	**Date**	**4/17/2006**
	Sheet	4	**of**	

Find Time of Concentration, Tc:

maximum length of travel = 1277 ft
height of most remote point above outlet 86 ft

Tc = 5.7 min

Reference Fig. 8.03a

Determine rainfall intensity factor, i:

Use 25 year storm with Tc from above.

i = 7 in/hour

Calculate Peak Flow, Qp:

Qp = C i A

Qp = 36.2 cfs

Belews Creek FGD Residue Landfill	**Job No.**	**046119**		
Design for Cell Stormwater	**Comp. By**	**WMM**	**Date**	**3/10/2006**
	Chkd. By	**DEB**	**Date**	**4/17/2006**
	Sheet	5	**of**	

Objective: **Design Riser and Culverts for Landfill Cell**
For 25 Year - 24 Hour Storm
Landfill Cell 3

Reference (1) *NC Sedimentation and Erosion Control Manual*

25 year 24 hour precipitation = **5.6 in** Reference Fig 8.03k, Reference 1

Use Rational Method to determine peak runoff in cfs

Q = CiA

Q = peak rate of runoff in cubic feet per second (cfs)
C = runoff coefficient
i = average intensity of rainfall in inches/hour, for a storm duration equal to the time of concentration, Tc
Tc = time of concentration, minutes
A = drainage area in acres

Determine drainage area:

The drainage area for the Cell 3 riser consists of the area in Cell 3 and a portion of the area for Cell 2, since the open portion of Cell 2 drains into Cell 3.

Area Cell 3	=	5.6 ac	Cell 3 only
Total Area	=	11.7 ac	includes a portion of Cell 2

Calculate area weighted C term:

For HDPE area (assumed)
C = 0.99

Use C for bare, packed, smooth soil. Reference Table 8.03a

C = 0.6

	Area		C		
HDPE	3.2	x	0.99	=	3.13
gypsum, soil	8.6	x	0.6	=	5.14
	11.73				8.27

Weighted C = 0.71

Belews Creek FGD Residue Landfill	**Job No.**	**046119**		
Design for Cell Stormwater	**Comp. By**	**WMM**	**Date**	**3/10/2006**
	Chkd. By	**DEB**	**Date**	**4/17/2006**
	Sheet	6	**of**	

Find Time of Concentration, Tc:

maximum length of travel = 1317 ft
height of most remote point above outlet = 88 ft

Tc = 5.7 min

Reference Fig. 8.03a

Determine rainfall intensity factor, i:

Use 25 year storm with Tc from above.

i = 7 in/hour

Calculate Peak Flow, Qp:

Qp = C i A

Qp = 57.9 cfs

Belews Creek FGD Residue Landfill	**Job No.**	**046119**		
Design for Cell Stormwater	**Comp. By**	**WMM**	**Date**	**3/10/2006**
	Chkd. By	**DEB**	**Date**	**4/17/2006**
	Sheet	7	**of**	

Objective: **Design Riser and Culverts for Landfill Cell**
For 25 Year - 24 Hour Storm
Landfill Cell 4

Reference (1) *NC Sedimentation and Erosion Control Manual*

25 year 24 hour precipitation = **5.6 in** Reference Fig 8.03k, Reference 1

Use Rational Method to determine peak runoff in cfs

Q = CiA

Q = peak rate of runoff in cubic feet per second (cfs)
C = runoff coefficient
i = average intensity of rainfall in inches/hour, for a storm duration equal to the time of concentration, Tc
Tc = time of concentration, minutes
A = drainage area in acres

Determine drainage area:

The drainage area for the Cell 4 riser consists of the area in Cell 4 and a portion of the area for Cell 3, since the open portion of Cell 3 drains into Cell 4.

Area Cell 4	=	5.7 ac	Cell 4 only
Total Area	=	12.4 ac	includes a portion of Cell 3

Calculate area weighted C term:

For HDPE area (assumed)
C = 0.99

Use C for bare, packed, smooth soil. Reference Table 8.03a

C = 0.6

	Area		C		
HDPE	4.0	x	0.99	=	3.92
gypsum, soil	8.4	x	0.6	=	5.07
	12.41				8.99

Weighted C = 0.72

Belews Creek FGD Residue Landfill
Design for Cell Stormwater

Job No.	**046119**		
Comp. By	**WMM**	**Date**	**3/10/2006**
Chkd. By	**DEB**	**Date**	**4/17/2006**
Sheet	8	**of**	

Find Time of Concentration, Tc:

maximum length of travel = 1187 ft
height of most remote point above outlet = 84 ft

Tc = 5.5 min

Reference Fig. 8.03a

Determine rainfall intensity factor, i:

Use 25 year storm with Tc from above.

i = 7 in/hour

Calculate Peak Flow, Qp:

Qp = C i A

Qp = 62.9 cfs

Belews Creek FGD Residue Landfill
Design for Cell Stormwater

Job No.	**046119**		
Comp. By	**WMM**	**Date**	**4/5/2006**
Chkd. By	**DEB**	**Date**	**4/17/2006**
Sheet	9	**of**	

Objective: **Design Riser and Culverts for Landfill Cell**
For 25 Year - 24 Hour Storm
Determine Barrel and Riser Size, Set Crest if Riser

Reference **(1) NC Sedimentation and Erosion Control Manual**
(2) Applied Hydrology and Sedimentology for Disturbed Areas, Barfield, Warner, and Hann.

	Cell 1	Cell 2	Cell 3	Cell 4	
Peak Flow's from Runoff, Qp (cfs)	**23.9**	**36.2**	**57.9**	**62.9**	
Barrel Data					
Length of Barrel (ft)	58	62	62	90	
Inlet Invert (Elev)	762.40	761.22	759.49	758.62	
Outlet Invert (Elev)	758.0	758.0	758.0	757.72	
Slope (ft/ft)	0.08	0.05	0.02	0.01	
Head (ft) for Barrel Flow to meet Qp (Assume Crest of Riser at 2* Barrel Dia.)	**4.0**	**5.0**	**6.0**	**9.1**	
Nominal Pipe Size (barrel) (in)	24	30	36	36	
Pipe ID	22.80	27.60	27.60	27.60	
Flow Area (ft^2)	2.84	4.15	4.15	4.15	
Manning's n	0.010	0.010	0.010	0.010	
Km	1	1	1	1	
Kp	0.0079	0.0061	0.0061	0.0061	Table 4.1 Ref 2
Qp (cfs) (8.07.5)	**29.0**	**48.3**	**53.0**	**63.0**	

Determine Height of flow over Riser Crest to Supply Sufficient Flow

Riser Diameter (in)	30	36	48	48	
Min. Required Head at Crest of Riser (ft) to meet Qp	0.9	1.45	1.6	1.7	Table 8.07.9

Belews Creek FGD Residue Landfill
Design for Cell Stormwater

Job No.	**046119**		
Comp. By	**WMM**	**Date**	**4/5/2006**
Chkd. By	**DEB**	**Date**	**4/17/2006**
Sheet	16	**of**	

Determine Crest Elev. for Riser				
Crest Elev. for Lowest Adjacent Berm	768	768	768	768
Required Freeboard (ft)	1.0	1.0	1.0	1.0
Elev. 767' - Head for Weir	766.10	765.55	765.40	765.30
Invert + 2 * barrel dia.	766.40	766.22	765.49	764.62
Set Crest at Lower Elevation (round down to 0.1')	766.1	765.5	765.4	764.6
Head Avail for Weir Flow 767' - Crest	0.90	1.50	1.60	2.38
Crest of Riser to Invert (ft)	3.7	4.3	5.9	6.7
Max Water Elev. (Crest El.+ Head for Weir Flow)	767.0	767.0	767.0	766.3
Head at Max Water Elev. (barrel flow)	4.6	5.7	7.5	7.7
Check Barrel Flow at Max Water Elev.				
Qbarrel (cfs) (8.07.5)	**31.1**	**51.8**	**59.3**	**58.0**
Check against Qp:	**23.9**	**36.2**	**57.9**	**62.9**

If Qbarrell > Qp, then no storage in the cell is required

Storage required for Cell 4. Other Cells are OK.

Belews Creek FGD Residue Landfill
Design for Cell Stormwater

Job No.	**046119**		
Comp. By	**WMM**	**Date**	**4/5/2006**
Chkd. By	**DEB**	**Date**	**4/17/2006**
Sheet	**11**	**of**	

The approximate storage required, S = (Qi -Qo) * Tp

Where:

Qi = flow in (Qp), cfs
Qo = flow out (Qbarrel), cfs
Tp = time to peak

$$Tp = \frac{Q^* Aw}{1.39\ Qp}$$

Q* = runoff depth, in
Aw = area of watershed

Calculate Tp

Cell	Area (ac)	Qp (cfs)	Q* (in)	Tp (min)
4	12.4	62.9	5.6	48

Calculate Approximate Storage Required in Cell 4:

S = (Qp -Qo)Tp
Qp = 62.9 cfs
Qo = 58.0 cfs
Tp = 48 min
S = 14,402 ft^3

Cell 4 Area Volume Information:

Cell 4 elevation of Operational Cover adjacent to riser is 759.62 , use 760' as lower elevation.

Elev	Area (ft^2)	Average Area (ft^2)	dH (ft)	Incremental Volume (ft^3)
760	0			
		2831	2	5,661
762	5661			
		9983	2	19,965
764	14304			
			Total Volume	25,626

Available volume below Elev. 764' is greater than the required storage. Therefore, the barrel and riser design for Cell 4 is adequate.

Belews Creek FGD Residue Landfill
Design for Cell Stormwater

Job No.	046119		
Comp. By	WMM	Date	4/5/2006
Chkd. By	DEB	Date	4/17/2006
Sheet	12	of	

Summary of Barrel and Riser Data

	Cell 1	Cell 2	Cell 3	Cell 4
Barrel Diameter, HDPE SDR 26 (in)	24	30	36	36
Riser Diameter, HDPE SDR 26 (in)	30	36	48	48
Barrel Inlet Invert Elev (ft.)	762.40	761.22	759.49	758.62
Barrel Outlet Invert Elev (ft.)	758.0	758.0	758.0	757.7
Riser Crest Elev. (ft)	766.1	765.5	765.4	764.6

Belews Creek FGD Residue Landfill
Design For Cell Stormwater

Job No.	046119		
Comp. By	WMM	Date	4/5/2006
Chkd. By	DEB	Date	4/17/2006
Sheet	13	of	

Objective: **Determine Number of Holes in Leachate Sump Riser To Drain 25 Year - 24 Hour Storm Determine Barrel and Riser Size, Set Crest if Riser**

Reference **(1) NC Sedimentation and Erosion Control Manual**

Calculate area of holes to dewater Cell in 24 hours:

$$Ao = \frac{As\ (2h)^{0.5}}{T\ Cd\ 20{,}428}$$

Reference 1, Page 8.07.8

Ao = area of holes, ft^2
As = surface area of basin,ft^2
Cd = orifice coeffcient (0.6)
h = head over hole, ft
T = time to dewater basin, hrs (24 hours)

	Cell 1	Cell 2	Cell 3	Cell 4
Peak Flow's (cfs)	**23.9**	**36.2**	**57.9**	**62.9**
Inlet Invert (Elev)	762.40	761.22	759.49	758.62
Elevation Operational Cover	763.40	762.22	760.49	759.62
Riser Diameter (in)	30	36	48	54
Crest of Riser	766.1	765.5	765.4	764.6
Elevation top of barrel	764.4	763.7	762.5	761.6
Height of Riser above Invert (ft)	3.7	4.3	5.9	6.0
Height of Riser above Top of Barrel (ft)	1.7	1.8	2.9	3.0
As, Area in Cell at Crest of Riser (ft^2)	74,927	58,237	66,691	55,430
h, use 1/2 distance between Operational Cover and Crest of Riser	1.4	1.6	2.5	2.5
Ao, area of holes required, ft^2 =	0.42	0.36	0.50	0.42
hole diameter, in	0.75	0.75	0.75	0.75
area of hole , ft^2	0.0031	0.0031	0.0031	0.0031
number of holes	136.4	116.9	163.7	137.3
Riser Circumference, in	94	113	151	170

Belews Creek FGD Residue Landfill
Design For Cell Stormwater

Job No.	**046119**		
Comp. By	**WMM**	**Date**	**4/5/2006**
Chkd. By	**DEB**	**Date**	**4/17/2006**
Sheet	**14**	**of**	

Objective:

Determine Number of Holes in Leachate Sump Riser
To Drain 25 Year - 24 Hour Storm
Determine Barrel and Riser Size, Set Crest if Riser

Number of holes per foot, 4" on center	4	4	4	4
Number of holes per row	31.4	37.7	50.3	56.5
Number of rows required	**4.3**	**3.1**	**3.3**	**2.4**

Drawing CP-5 shows 6 rows of holes at 4" center-to-center.
This spacing is adequate to drain cell.

Belews Creek FGD Residue Landfill
Design Cell Stormwater

Job No.	046119		
Comp. By	WMM	Date	4/14/2006
Chkd. By	DEB	Date	4/17/2006
Sheet	15	of	

Objective: Determine Size of Concrete Pad Used to Anchor Leachate Sump Riser

Reference: (1) NC Sedimentation and Erosion Control Manual

Summary of Barrel and Riser Data

	Cell 1	Cell 2	Cell 3	Cell 4
Riser Diameter, HDPE SDR 26 (in)	30	36	48	48
Barrel Inlet Invert Elev (ft.)	762	761	759	759
Riser Crest Elev. (ft)	766	766	765	765
Height of Riser (ft)	3.7	4.3	5.9	6.0

Calculate buoyant force, Fb (lb):

Fb = 62.4 lb/ft^3 * height of riser (ft) * area of riser (ft^2)

	Cell 1	Cell 2	Cell 3	Cell 4
Fb (lb)	1133	1888	4634	4705

Calculate downward force required, Fdr (lb):

Fdr = 1.1 * buoyant force

	Cell 1	Cell 2	Cell 3	Cell 4
Fdr (lb)	1247	2077	5098	5175

Calculate volume of concrete to equal Fdr:

Use buoyant unit wt. concrete

Buoyant wt. concrete = (150 - 62.4)lb/ft^3
Buoyant wt. concrete = 87.6 lb/ft^3

	Cell 1	Cell 2	Cell 3	Cell 4
Volume Concrete Required (ft^3)	14.2	23.7	58.2	59.1

Calculate Length of Side for Square Pad at thickness:

	ft	ft	ft	ft
12 in thick pad	3.8	4.9	7.6	7.7
15 in thick pad	3.4	4.4	6.8	6.9
18 in thick pad	3.1	4.0	6.2	6.3
24 in thick pad	2.7	3.4	5.4	5.4

Belews Creek FGD Residue Landfill
Design Cell Stormwater

Job No.	**046119**		
Comp. By	**WMM**	**Date**	**4/14/2006**
Chkd. By	**DEB**	**Date**	**4/17/2006**
Sheet	**16**	**of**	

Objective: Determine Size of Concrete Pad Used to Anchor Leachate Sump Riser

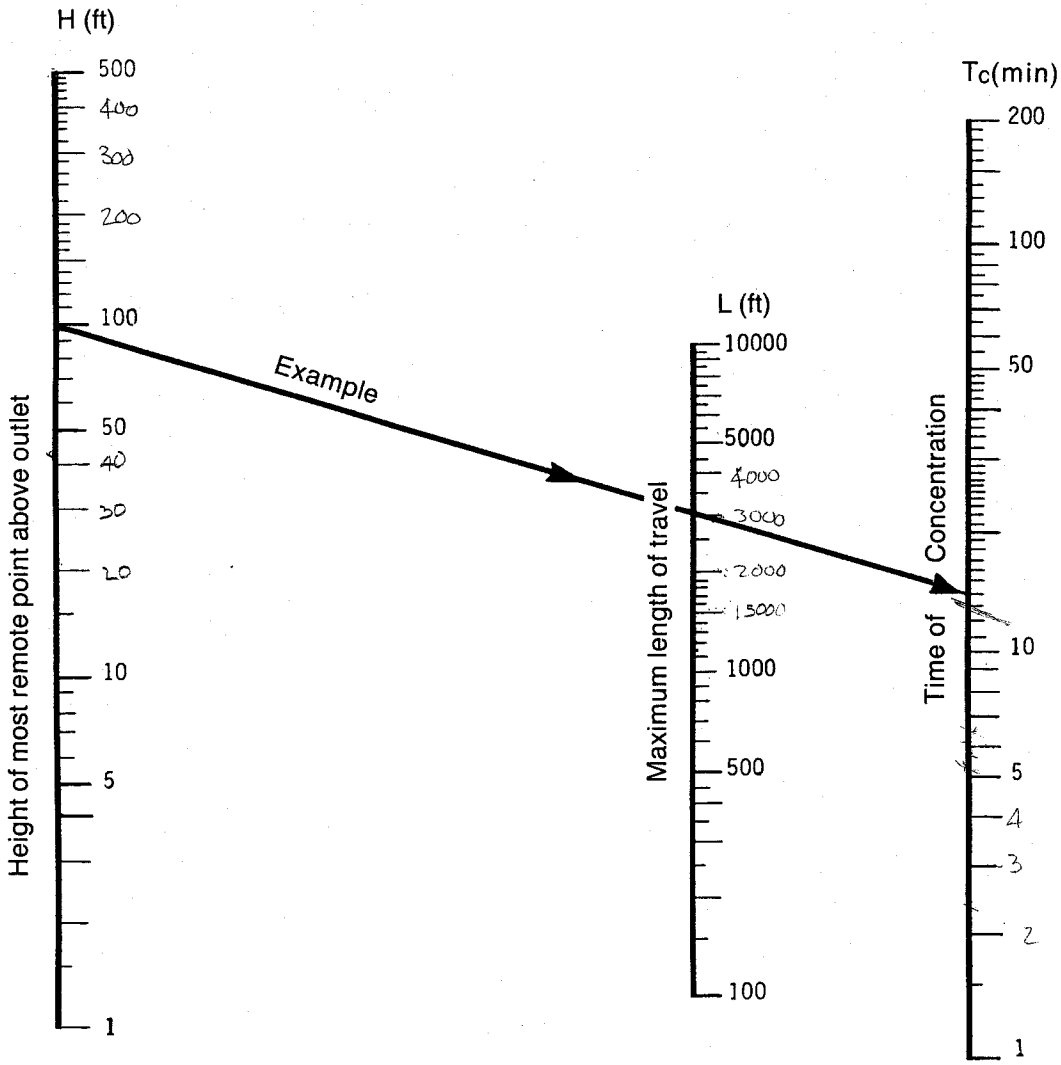
Detail on CP-5 shows 4" long gusset plate used to attached riser to HDPE plate under riser.

Add the following to the riser diameter, for minimum pad length:

8" - for 4" gussett on both sides of riser.
6" - for 3" cover at ends of gussett plate.

Minimum Length of side =	**3.7**	**4.2**	**5.2**	**5.2**

Dimensions for Square Concrete Pad	**Cell 1**	**Cell 2**	**Cell 3**	**Cell 4**
Length (ft)	**3.7**	**4.2**	**5.4**	**5.4**
Thickness (in)	**12**	**18**	**24**	**24**



Note:
Use nomograph Tc for natural basins with well-defined channels, for overland flow on bare earth, and for mowed-grass roadside channels.

For overland flow, grassed surfaces, multiply Tc by 2.

For overland flow, concrete or asphalt surfaces, multiply Tc by 0.4.

For concrete channels, multiply Tc by 0.2.

Reference #1

Figure 8.03a Time of concentration of small drainage basins.

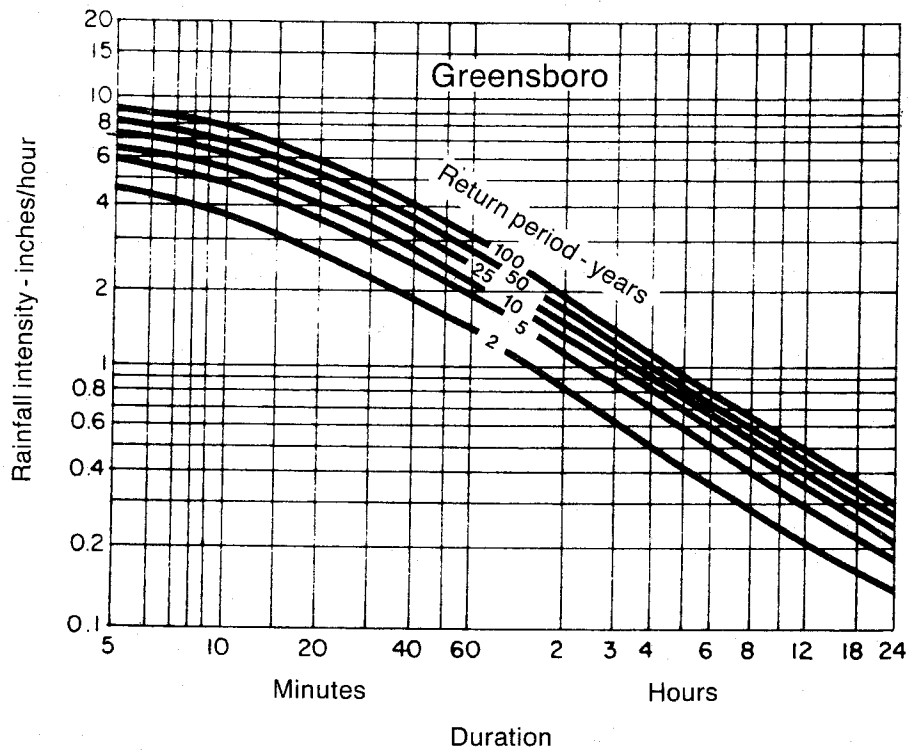


Figure 8.03d Rainfall intensity duration curves—Greensboro.

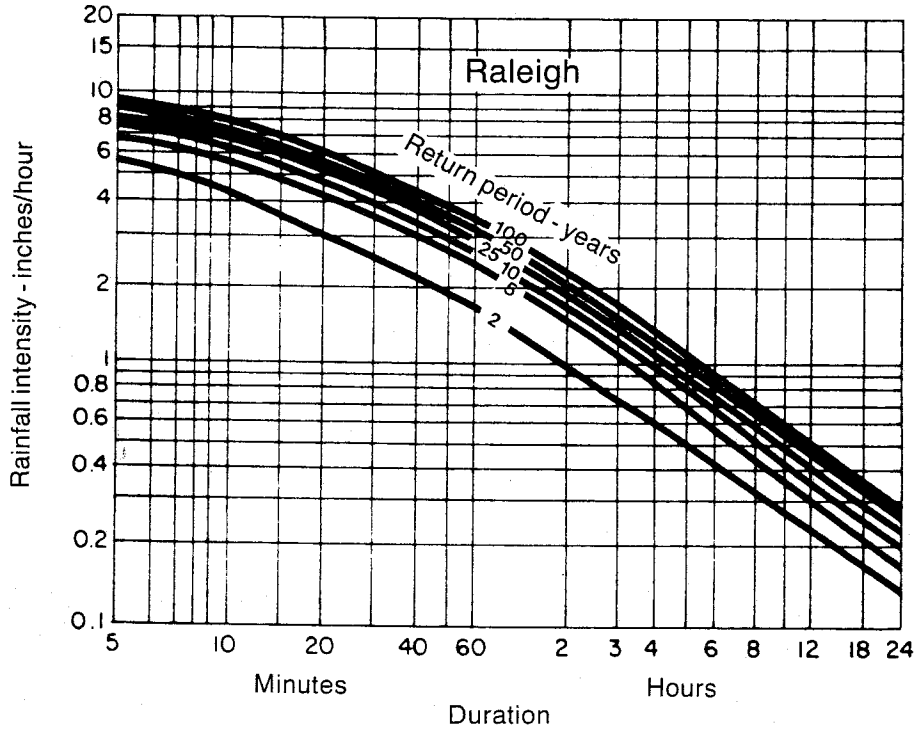


Figure 8.03e Rainfall intensity duration curves—Raleigh.

Reference #1

APPLIED HYDROLOGY AND SEDIMENTOLOGY FOR DISTURBED AREAS

B.J. Barfield
Professor

and

R.C. Warner
Assistant Professor

Department of Agricultural Engineering
University of Kentucky
Lexington, Kentucky

and

C.T. Haan
Professor and Head

Department of Agricultural Engineering
Oklahoma State University
Stillwater, Oklahoma

Reference #2

(4.3)

Figure 4.4, K_e is the
bend head loss coefficient,
ion, L is the length of the
mean velocity in the pipe.
given in Figure 4.5. Since
ocity times area, equation

(4.4)

al area of the pipe. Values
circular and square pipes.
figuration of the entrance
K_b are 1.0 and 0.5, res-
as Hoffman (1974) should

let spillway considering bend

Table 4.1 Head Loss Coefficients for Circular Conduits Flowing Full.

Head Loss Coefficient, K_c, for Circular Pipe Flowing Full $K_c = \frac{5087\, n^2}{d_i^{4/3}}$

Pipe diam. inches	Flow area sq. ft.	Manning's Coefficient of Roughness "n"																			
		0.010	0.011	0.012	0.013	0.014	0.015	0.016	0.017	0.018	0.019	0.020	0.021	0.022	0.023	0.024	0.025
6	0.196	0.0467	0.0565	0.0672	0.0789	0.0914	0.1050	0.1194	0.1348	0.1510	0.1680	0.1870	0.2060	0.2260	0.2470	0.2690	0.2920
8	0.349	.0318	.0385	.0458	.0537	.0623	.0715	.0814	.0919	.1030	.1148	.1272	.1400	.1540	.1680	.1830	.1990
10	0.545	.0236	.0286	.0340	.0399	.0463	.0531	.0604	.0682	.0765	.0852	.0944	.1041	.1143	.1249	.1360	.1480
12	0.785	.0185	.0224	.0267	.0313	.0363	.0417	.0474	.0535	.0600	.0668	.0741	.0817	.0896	.0980	.1067	.1157
14	1.069	.0151	.0182	.0217	.0255	.0295	.0339	.0386	.0436	.0488	.0544	.0603	.0665	.0730	.0798	.0868	.0942
15	1.230	.0138	.0166	.0198	.0232	.0270	.0309	.0352	.0397	.0446	.0496	.0550	.0606	.0666	.0727	.0792	.0859
16	1.400	.0126	.0153	.0182	.0213	.0247	.0284	.0323	.0365	.0409	.0455	.0505	.0556	.0611	.0667	.0727	.0789
18	1.770	.10178	.0130	.0155	.0182	.0211	.0243	.0276	.0312	.0349	.0389	.0431	.0476	.0522	.0570	.0621	.0674
21	2.410	.00878	.01062	.0126	.0148	.0172	.0198	.0225	.0254	.0284	.0317	.0351	.0387	.0425	.0464	.0506	.0549
24	3.140	.00735	.00889	.01058	.0124	.0144	.0165	.0188	.0212	.0238	.0265	.0294	.0324	.0356	.0389	.0423	.0459
27	3.980	.00628	.00760	.00904	.01061	.0123	.0141	.0161	.0181	.0203	.0227	.0251	.0277	.0304	.0332	.0362	.0393
30	4.910	.00546	.00660	.00786	.00922	.01070	.01228	.0140	.0158	.0177	.0197	.0218	.0241	.0264	.0289	.0314	.0341
36	7.070	.00428	.00518	.00616	.00723	.00839	.00963	.01096	.0124	.0139	.0154	.0171	.0189	.0207	.0226	.0246	.0267
42	9.620	.00348	.00422	.00502	.00589	.00683	.00784	.00892	.01007	.01129	.0126	.0139	.0154	.0169	.0184	.0201	.0218
48	12.570	.00292	.00353	.00420	.00493	.00572	.00656	.00747	.00843	.00945	.01053	.01166	.0129	.0141	.0154	.0168	.0182
54	15.900	.00249	.00302	.00359	.00421	.00488	.00561	.00638	.00720	.00808	.00900	.00997	.01099	.0121	.0132	.0144	.0156
60	19.630	.00217	.00262	.00312	.00366	.00424	.00487	.00554	.00622	.00702	.00782	.00866	.00955	.01048	.0115	.0125	.0135

Reference #2

B

Drawings Under Seperate Cover