STEELFIELD CLASS I LANDFILL EXPANSION
BAY COUNTY, FLORIDA

Addendum #2 to RFP No. 19-33

Engineering Report

SOLID WASTE DIVISION
311411 Landfill Road
Panama City Beach, Florida 32413

HDR ENGINEERING, INC.
76 S. Laura Street, Suite 1600
Jacksonville, Florida 32202

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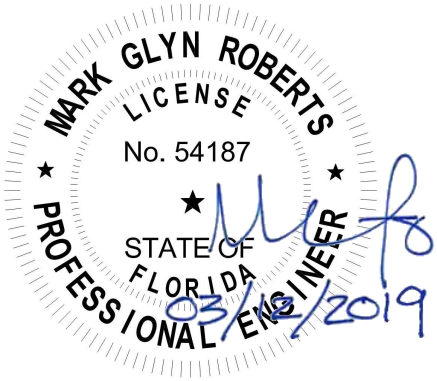

Mark G. Roberts, P.E.
Florida P.E. No. 54187

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PART A

GENERAL INFORMATION

The proposed work is to obtain a Class I Construction Permit from Florida Department of Environmental Protection (FDEP) and subsequent construction of Cells 6 and 7 at Steelfield Road Landfill (SFRL). The engineering report follows the requirements in DEP Form # 62-701.900(1), F.A.C. Relevant general information of the site can be obtained from Cell 5 Permit Application submitted in November 2010 from the following link:

https://depedms.dep.state.fl.us:443/Oculus/servlet/shell?command=getEntity&[guid=8.96619.1]&[profile=Permitting_Authorization]

The proposed liner system will include a double liner system with geosynthetic clay liner (GCL) to compensate for 10^{-5} cm/s low permeability soil subbase. The components of the liner system are illustrated in the preliminary drawings. The leachate generated from the cells shall be stored in above ground leachate tanks for offsite disposal. No leachate recirculation is proposed.

The master plan for SFRL was developed in 1986 within an approximately 289 acre property owned by Bay County at that time. The original design included eight lined cells within 4 separate footprints for Class I waste and ash disposal. There were also separate unlined footprints designated for Class III waste disposal. The stormwater management system for the overall 104 acre landfill footprint (lined and unlined) was designed in 1986 as part of the original master plan (refer to following link).

https://depedms.dep.state.fl.us:443/Oculus/servlet/shell?command=getEntity&[guid=8.48302.1]&[profile=Permitting_Authorization]

Approximately 70 acres of landfill footprint are currently developed which includes 49 acres of Class I footprint and 21 acres of Class III footprint. With the proposed 22.7 acre lateral expansion (i.e. excluding the 11.3 acre area over Cells 1 and 2), total developed landfill footprint will be 92.7 acres. Since stormwater (or ERP) permitting has already been performed for 104 acre landfill footprint, no additional stormwater permitting as part of this expansion is required.

PART B

DISPOSAL FACILITY GENERAL INFORMATION

The SFRL consists of five lined cells (Cells 1, 2, 3, 4 and 5) and closed unlined Class III Cells A and B (Note that Cell B is considered partially lined or constructed over lined cap without a leachate collection mechanism). Bay County plans to continue to excavate, process and relocate Class III waste within Cells A and B as operational cover material. Currently WTE ash, Class I MSW and Class III waste is deposited within the lined Class I footprint. The SFRL is operating under Solid Waste Operations Permit #0007875-009-SO-01 which expires November 20, 2019. The following information describes the existing Class I lined footprint of the SFRL.

Cells 1 and 2 were constructed in July 1987 with a single 60 mil HDPE smooth liner over compacted subgrade overlain by a 2-foot leachate collection layer. Originally, the leachate generated from Cells 1 and 2 drained east and then north to a lined leachate collection surface impoundment. In 1995 the leachate management system of Cells 1 and 2 was modified to allow gravity flow of leachate towards the south into the existing leachate storage tank through a series of manholes using an 8-inch Sch 35 PVC pipe. Cell 1 and part of Cell 2 were capped and closed in 1995/1996 and the remaining portion of Cell 2 was capped and closed in 2000/2001. The closure system for Cells 1 and 2 consists from bottom to top of a 12-inch liner subgrade, 40 mil textured HDPE and 2-foot of cover soil. There has been no reported leachate generation from Cells 1 and 2 since soon after the closure of the two cells was completed.

Cell 3 was constructed in 1998 with a bottom liner system consisting from bottom to top of a 10^{-5} cm/sec subbase, 60 mil HDPE secondary liner, Geonet leachate detection layer, 60 mil HDPE primary liner, geonet leachate collection layer, non-woven geotextile and 2-foot protective cover with a minimum hydraulic conductivity of 10^{-4} cm/s. The cell base was constructed using a smooth liner and sideslopes were constructed using a textured liner. The leachate generated from Cell 3 flows north laterally over the double liner and collects into leachate collection and detection manholes outside the cell footprint on the north side of the cell. The detection manhole discharges leachate into a collection manhole and gravity flows through an 8-inch X 12-inch HDPE dual containment pipe located underneath the Cell 4 liner system draining to the existing leachate storage tank located to south of the landfill footprint.

Cell 4 was constructed in 2002 as an infill with a single liner system over the eastern slopes of closed Cells 1 and 2 (note the cap liner remained in place), and a double liner system within the valley between Cells 1 & 2 and Cell 3. The single liner system from bottom to top consists of compacted subgrade, 60-mil textured HDPE, double sided geocomposite and 2-foot protective cover with a minimum hydraulic conductivity of 10^{-4} cm/s. The double liner system consists from bottom to top of compacted subgrade, GCL, 60 mil textured HDPE secondary liner, geocomposite leachate detection layer, 60 mil textured HDPE primary liner, geocomposite leachate collection layer and 2-foot protective cover with a minimum hydraulic conductivity of 10^{-4} cm/s. The dual contained leachate collection pipe from Cell 3 remained underneath Cell 4 infill area.

Cell 5 was constructed in 2012 with a double liner system which consists from bottom to top of compacted subgrade, GCL, 60 mil textured HDPE secondary liner, geocomposite leachate detection layer, 60 mil textured HDPE primary liner, geocomposite leachate collection layer and 2-foot protective cover with a minimum hydraulic conductivity of 10^{-4} cm/s. The leachate generated from Cell 5 flows north laterally over the double liner system and collects into leachate collection and detection manholes located outside of the cell footprint. The detection manhole discharges leachate into collection manhole and gravity flow to Cell 3 leachate collection manhole that will ultimately convey leachate to the existing above ground leachate storage tank.

RFP No. 19-33 is to design, permit and construct Cells 6 and 7 contiguous to the existing liner footprint. Portions of Cells 6 and 7 are located on the southern side slope and on top of closed Cells 1 and 2. The leachate conveyance pipes from Cells 1 to 5 shall be extended below the proposed Cell 6 liner system to the relocated above ground leachate storage tanks. The existing leachate storage tank, the pump station and associated electrical system shall be abandoned and removed. Landfill infrastructure such as recycling drop off facility shall be relocated.

PART C

PROHIBITIONS

C.1 SITING CRITERIA

The prohibitions of Rule 62-701.300(2), F.A.C. have not and will not be violated with this vertical/lateral expansion project. It should be noted that a portion of the proposed expansion footprint is located over Cells 1 and 2. In accordance with Rule 62-701.300(2), F.A.C., disposal will be prohibited within or in accordance with the following:

a) Geological formations or other subsurface areas that will not provide support:

The requirements of this section were addressed in Section E of 1986 permit application.

https://depedms.dep.state.fl.us:443/Oculus/servlet/shell?command=getEntity&[guid=8.20210.1]&[profile=Permitting_Authorization]

As discussed in Part I of this report, the original investigations and subsequent investigations did not find unsuitable foundation to support the proposed expansion.

In addition, refer to the Section 4 of June 2001 Major Permit Modification Application for Class III Cell B for the most recent hydrogeologic investigation report

b) Within 500 feet of an existing or approved potable water well:

This requires review and confirmation by DBE.

c) In a dewatering pit unless it is lined and has leachate containment:

The landfill is not designed in the groundwater or in a dewatering pit.

d) In any natural or artificial bodies of water:

The proposed expansion area will not be constructed in a natural or artificial body of water. The lined irrigation pond within proposed footprint will be dewatered and liner will be removed and disposed before begin excavation for cell construction.

e) Within 200 feet of a wetland body:

The proposed expansion area is located with the conceptual footprint identified in 1986 permit application drawings. Based on further evaluations by Bay County Engineering in 2018, the footprint area proposed for this expansion is not located in or within 200 feet of wetlands.

f) On the right-of-way of any public road or highway:

The expansion will not be located within or on the right-of-way of any public road or highway.

C.2 EXEMPTIONS

SFRL does not seek exemptions to the prohibitions listed in Rule 62-701.300, F.A.C.

C.3 BURNING RESTRICTION

Burning of solid waste is prohibited at this facility.

C.4 HAZARDOUS WASTE RESTRICTIONS

Hazardous wastes are not disposed of at this facility. No changes are proposed to the requirements of this section as part of proposed expansion application.

C.5 PCB DISPOSAL RESTRICTIONS

PCBs are not disposed of at this facility. No changes are proposed to the requirements of this section as part of proposed expansion application.

C.6 BIOMEDICAL WASTE RESTRICTIONS

Untreated biomedical waste is not knowingly accepted or disposed at the SFRL. Please note that treated biomedical waste may be accepted at SFRL's Class I Landfill provided that the waste containers are marked "Treated Biomedical Waste". No changes are proposed to the requirements of this section as part of this expansion application.

C.7 CLASS I SURFACE WATER RESTRICTIONS

The proposed expansion footprint is not located within 3,000 feet of a Class I surface water.

C.8 SPECIAL WASTE FOR LANDFILLS RESTRICTIONS

Special wastes such as lead-acid batteries, used oil, white goods, and whole tires are not disposed/landfilled at this facility. No changes are proposed to the requirements of this section as part of this expansion application.

C.9 LIQUID RESTRICTIONS

No liquids are disposed of at this facility. No changes are proposed to the requirements of this section as part of this expansion application.

C.10 USED OIL AND OILY WASTE RESTRICTIONS

No used oil is disposed of at this facility. No changes are proposed to the requirements of this section as part of this expansion application.

PART D

GENERAL SOLID WASTE MANAGEMENT FACILITY PERMIT REQUIREMENTS

D.1 PERMIT APPLICATION COPIES

The DBE to submit four (4) copies of the completed construction permit application form and supporting information including information in this Addendum and additional information as required by rule to the FDEP.

D.2 CERTIFICATION

Appropriate professional certifications shall be provided on all applicable submittals herewith by the DBE.

D.3 TRANSMITTAL LETTER

A letter of transmittal to FDEP is submitted by DBE.

D.4 FDEP FORM

A completed, dated, signed and sealed application form shall be submitted by DBE with supporting data.

D.5 PERMIT APPLICATION FEE

A check in the amount of $10,000.00 for the permit application shall be submitted by DBT.

D.6 ENGINEERING REPORT

The Engineering Report as provided in the Addendum HDR is not complete. DBE to review and accept the information contained in this report. The DBE is required to fill in the missing gaps and submit to FDEP for review and approval. If prospective DBEs have doubts or questions regarding acceptance of this report shall be resolved before submitting the priced bids.

D.7 OPERATIONS PLAN AND CLOSURE PLAN

A revised Operations Plan (only main body text revised) has been provided with this Engineering Report in Appendix A. The site plan is updated as shown in the drawings package. The updated Closure Plan is included in Appendix J. Note that the operations plan included in Appendix A is the revised version of the latest submitted operations plan to FDEP in 2018 as part of operations height increase. This plan is currently under review. Proposed changes to the operations plan are indicated as ~~strike through~~ (for deletions) and <u>underline</u> (for additions).

D.8 CONTINGENCY PLAN

The Contingency Plan is submitted as Attachment 2 to the Operations Plan. No changes are proposed to the requirements of this section.

D.9 DRAWINGS FOR THE SOLID WASTE MANAGEMENT FACILITIES

The information required in permit application (a) and (b) of this Section is identified in Cover Sheet, Sheets 00C-01, 00C-02, 00C-03 and 00C-04 of the Preliminary Design Build Drawing Set, submitted under a separate cover.

A boundary survey and legal description of the SFRL property has been provided to the FDEP in the past and will remain unchanged with this permit application.

Other necessary information not included in the preliminary drawing set shall be prepared and submitted by DBE.

D.10 PROOF OF PROPERTY OWNERSHIP

The proof of ownership is submitted as part of the 1986 permit application.

D.11 RECYCLING ACTIVITIES

Bay County has an active solid waste recycling program and promotes solid waste recycling within the County to continually improve the program. The proposed landfill expansion does not influence the achievement of the County's recycling goals.

D.12 HISTORY OF FDEP ENFORCEMENT ACTIVITIES

DBE to submit history of FDEP enforcement actions.

D.13 PROOF OF PUBLICATION OF LANDFILL PERMIT APPLICATIONS

A notice shall be published by the DBE in a local newspaper stating receipt of application for permit by the FDEP. Proof of publication shall be submitted to the FDEP following receipt from the newspaper.

D.14 AIRPORT SAFETY REQUIREMENTS

Based on HDR's evaluation, there are no airports within 10,000 feet of SFRL per 2017 data. However, DBE to confirm and report. Airports located within 5 miles of the SFRL shall be submitted by DBE with Permit Application for FDEP review.

D.15 CERTIFIED OPERATORS AND SPOTTERS

Operator and spotter training/certification requirements have been addressed in the Operations Plan provided in Appendix A of this permit application.

PART E

LANDFILL PERMIT REQUIREMENTS

E.1 REGIONAL MAP

An aerial photograph downloaded from FDOT for year 2016 is included with Drawing Sheet 00C-01. Regarding airport locations within 5-mile radius of SFRL, refer to Section D.14 of this engineering report.

E.2 PLOT PLAN

A plot plan of the SFRL is provided as Drawing Sheet 00C-01 along with existing conditions. The plot plan includes dimensions, monitoring wells, waste disposal areas, zone of discharge and fencing. Cross sections are provided as part of the drawing set for the conceptual Class I landfill final cover. In addition, the select boring locations, logs and interpretation of subsurface resulting from soil borings are presented within Appendix B.

E.3 TOPOGRAPHIC MAP

The topographic map shown in the drawing set is a combined map from LIDAR data obtained from Bay County in 2016 with updates from ground survey performed by Bay County Public Works Department in March 2017 and February 2018. DBT shall perform a topographic survey of the expansion area including the existing landfill footprint and submit with the permit application to FDEP. The AutoCAD drawing of the survey shall be submitted to Bay County. At a minimum, the following shall be included as part of this signed and sealed survey by a Florida Registered Surveyor.

- Topographic survey of SFRL at least 200 feet beyond the existing landfill footprint and/or beyond the proposed limits of disturbance whichever comes is furthest.
- All buildings and appurtenances within the survey limits. These include: culverts (show the inverts, diameter and type of pipe), manholes, electric poles, stormwater discharge structures, lined irrigation pond limits, pumps, groundwater wells, pump stations, manholes, leachate storage tank and slab, household hazardous waste drop-off location with ramps, stormwater channels, access roads (paved and unpaved), stormwater downcommer pipes within Cells 1 and 2 (show the inverts, diameter and type of pipes), paved parking areas, fences, guiderail
- Cleanout pipe locations, diameter and inverts from the existing landfill
- Location of underground septic tank for the administration building
- Tree lines
- Survey bench marks

E.4 LANDFILL REPORT

E.4.a Population

The areas/business/customers served by the SFRL are primarily in Bay County, FL. The population projections for Bay County were taken from the State of Florida Office of Economic and Demographic Research (EDR).
(Source:http://www.edr.state.fl.us/Content/population-demographics/data/CountyPopulation.pdf).

Bay County Population Projections			
Year	2014	2019 (est.)	2024 (est.)
Population	170,781	179,565	187,757

E.4.b Solid Waste

The type of solid waste accepted at the SFRL is described in the Operations Plan provided Appendix A of this permit application. Anticipated accepted waste quantities for landfill disposal are discussed in following section.

E.4.c Facility Life

The following information is provided to estimate facility life by DBE.

- MSW collected in Year 2017 is 226,380 tons
- Assume a density of 1,500 lbs/cy
- Available landfill airspace as of January 01, 2018 is 1,043,000 cy
- Airspace generated from proposed expansion is 4,897,000 cy
- Assume no WTE operation
- Future airspace consumption to be determined by DBE to estimate site life.

E.4.d Cover Material

The material used for initial, intermediate cover, and final cover is described in the Operations Plan provided in Appendix A of this permit application.

E.5 WATER QUALITY MONITORING

SFRL will continue to fulfill water quality monitoring requirements including water quality monitoring conducted by an approved laboratory in accordance with 62-701.330(3)(g), FAC. For further information on water quality monitoring, please refer to Part L of this report.

E.6 FINANCIAL RESPONSIBILITY

Please refer to Part R of this report.

PART F

GENERAL CRITERIA FOR LANDFILLS

F.1 LOCATION

HDR reviewed the Federal Emergency Management Agency (FEMA) Floodplain Map #12005C0160H (https://msc.fema.gov/portal/search) relevant to proposed expansion area. According to this map, the proposed expansion is not located within the 100-year floodplain. It should be noted that the southwest corner of closed Cells 1 and 2 is identified as within 100-year flood zone. HDR believes this is a mistake since this area is at least 10 feet above the surrounding areas identified as non-flood zones.

F.2 HORIZONTAL SEPARATION

The proposed expansion waste limits presented on the site plan represent a minimum of 100-foot separation from the property boundary. The minimum horizontal separation is shown in Sheet 00C-03 of this application package.

PART G

LANDFILL CONSTRUCTION REQUIREMENTS

In general, the following sections are applicable only for the proposed Cells 6 and 7.

G.1 PLANNED CONSTRUCTION AND CLOSURE

The SFRL shall complete landfill construction, including installation of liner, leachate collection and stormwater control systems, in accordance with the FDEP-approved design plans. Detailed design drawings are provided within this report. Detailed final closure of the SFRL will be designed, permitted, and constructed in accordance with Rule 62-701.400, F.A.C. and the closure plans discussed in Parts O and P.

G.2 LINER REQUIREMENTS

Liner System Description

Bottom Liner system will be constructed using a double-liner system which includes a leachate collection layer and leak detection layer within the entire expansion footprint.

a. General Construction Requirements

   A double liner system per Chapter 62-701 is proposed for all Class I expansion areas. The materials composing the proposed liner system are standard in Florida landfill construction as they exhibit the physical, chemical, and mechanical properties to prevent failure due to physical contact with Class I leachate, climatic conditions, stress of installation, and other applied stresses and hydraulic pressures.

   The geological investigations for SFRL by Southern Earth Sciences Inc. in 1986 and 2010 were previously submitted to FDEP. The information collected during these investigations is used to perform Slope Stability, Settlement and Foundation Analyses for the expansion footprint. The relevant calculations are provided in Appendices B and C of this report. These calculations verify that the geologic setting is expected to provide adequate structural support. Note that the bottom liner of the proposed expansion is designed so that it remains above the seasonal high water table; therefore, the liner system is not expected to be affected by groundwater fluctuations, or subjected to hydrostatic uplift. Note that the limits of waste placement will remain within the lined containment berm (see Detail 2 of Drawing 00D-01). Therefore, all liquid that comes into contact with waste will remain inside of the leachate collection and removal system (LCRS).

b. Composite Liners

   Not applicable.

c. Double Liners

The entire expansion footprint will receive a double liner system from top to bottom consists of:

- 24" protective cover layer (minimum k =1 x 10^{-3} cm/s)
- Double-sided geocomposite (referred as Composite Drainage Net or CDN in this application)
- 60-mil HDPE primary liner (textured both sides)
- Double-sided geocomposite
- 60-mil HDPE secondary liner (textured both sides)
- GCL with a maximum hydraulic conductivity of 5 x 10^{-9} cm/s
- Compacted subgrade

The proposed double liner system is consistent with the design described in Rule 62-701.400(3)(c), FAC.

The design of the LCRS is based on HELP Model outputs as attached in Appendix D.

d. Geosynthetic Component Standards

Material specifications for geosynthetics meeting the requirements in this section are provided in Appendix E.

Bay County will submit the test results required by Rule 62.701.400(3)(d)7&8, FAC, and a Certificate of Construction Completion (CCC) to the Department upon completion of each phase of landfill construction.

Bay County will ensure that the first layer of waste placed above the liner and LCRS will be a minimum of four (4) feet in compacted thickness and consist of select waste containing no large rigid objects that may damage the liner or LCRS. Bay County will follow all the necessary precautions to protect the liner and LCRS during its installation and operation. A liner and LCRS Construction Quality Assurance (CQA) Plan, including test methods and frequencies, that meets the requirements of Rule 62-701.400, FAC is provided within Appendix F. Bay County will have a CQA inspector on site during the installation of geosynthetics. By following the CQA plan, the inspector will promote proper construction and protection of the new cell during construction. After the expansion area been certified and the FDEP concurs that it is ready for waste acceptance, procedures identified in the Revised Operations Plan (Appendix A) will be followed by the operations staff to protect the liner and LCRS during waste-filling operations.

e. Geosynthetic Specification Requirements

As noted within the previous section, the geosynthetic specifications, required by Rule 62-701.400(3)(e), FAC, have been provided in Appendix E. DBE to add additional specifications as needed to facilitate construction.

f. Soil Component Standards

Information pertaining to standards for soil components of the liner system is provided in technical specifications within Appendix E.

G.3 LEACHATE COLLECTION AND REMOVAL SYSTEM (LCRS)

Based on the proposed design, the leachate collected from existing Cells 1 and 2 will continue to flow under the proposed liner at a slope of approximately 1.2% into new pump station PS#2. PS #2 will discharge leachate to PS#1 through a flow meter using a portable pump as needed. PS#1 will pump leachate to onsite storage tanks.

The dual contained leachate collection pipes from Cells 3 and 5 will be extended at 1.5% minimum slope under the proposed Cell 6 liner to directly flow into new PS#1 as done in the current system.

The leachate collection pipes from Cell 4 will extend below the expansion liner to manholes MH#1 (detection leachate) and PS#2 (collection leachate). The detection leachate will gravity flow to PS#2 where leachate will be pumped to PS#1 through a flow meter.

A network of perforated piping consisting of 8-inch SDR 11 headers and laterals are provided within the proposed cells to collect and convey leachate to sumps. Based on the settlement analysis provided in Appendix C, it is calculated that the minimum post-settlement slope of the leachate collection pipe trench will remain above 0.36%. A leak detection conveyance system is proposed consisting of stacked geonet layers installed underneath leachate collection pipes within the double liner system.

A flow meter at each of the pump stations will be read daily on working days, to record the leachate amount pumped in gallons, and then converted into units of gallons per day in monthly reports. The separate primary and secondary leachate collection pump stations will allow for separate sampling and flow-recording of leachate collected within the primary and secondary LCRS. Cleanouts will be installed in at least one end of each collection trench lateral/header pipe, such that they can be accessed from the crests of the containment berms as indicated on Drawing 00C-04. The existing cleanouts toward the south side of Cells 3 and 5 will be abandoned in place by fusion welding an end cap. In the future these collection pipes will be accessed only from the north end. Since the pipe length is approximately 1000 feet and diameter is 8-inch HDPE, accessing the full length of the collection pipes from one end should not be an issue.

G.4 LEACHATE RECIRCULATION

Bay County is not planning to recirculate leachate into the proposed expansion cells.

G.5 LEACHATE STORAGE

a. No surface impoundments are proposed at the SFRL at this time; therefore this Rule is not applicable.

b. Bay County currently utilizes one above ground storage tank (AST) with approximately 100,000 gallon capacity, to store leachate produced from the lined Class I cells at the SFRL. The future leachate storage capacity calculations are provided in Appendix H. Based on the calculations, three 150,000 gal storage tanks are required to meet the required storage capacity. DBT shall design and install the leachate storage system including truck loading station based on the concepts provided in the drawing package and meeting the requirements of 62-701, F.A.C.

c. Bay County is not proposing to utilize underground leachate storage tanks at the SFRL; therefore this Rule is not applicable.

d. The existing cells gravity flow leachate to the storage location. The proposed cells are installed with sideslope risers to pump out leachate. For mechanical and electrical components (pumps) of the LCRS, Bay County will follow the manufacturer's recommendations for maintenance. Gravity-drained leachate collection and header pipes will be cleaned using water-jetting, or other effective techniques, upon initial installation and prior to each applicable FDEP permit renewal.

G.6 LINER SYSTEM CQA

The liner CQA plan is provided within Appendix F.

G.7 SOIL LINER CQA

Soil Liner CQA is not applicable to this permit application.

G.8 SURFACE WATER MANAGEMENT SYSTEM

The surface water at the SFRL is currently managed per the original master plan developed in 1986. Per this plan, a stormwater management system is designed for 104 acre landfill footprint. Approximately 70 acres of landfill footprint is currently developed which includes 49 acres of Class I footprint and 21 acres of Class III footprint. With the proposed 22.7 acre lateral expansion (i.e. excluding the 11.3 acre area over Cells 1 and 2), total developed landfill footprint will be 92.7 acres. Since stormwater (or ERP) permitting is already performed for 104 acre landfill footprint, no additional stormwater permitting as part of this expansion is required. As part of the proposed design, the capacity of the perimeter channels and culverts are confirmed as shown in Appendix I.

G.9 GAS CONTROL SYSTEM

As discussed in Section N of this Report an active gas control system is not required at SFRL.

G.10 LANDFILLS DESIGNED IN GROUNDWATER

The SFRL is not situated in groundwater; therefore, Rule 62-701.400(11), FAC, is not applicable.

PART H

HYDROGEOLOGICAL INVESTIGATION REQUIREMENTS

H.1 HYDROGEOLOGICAL INVESTIGATION AND SITE REPORT

Refer to Section 4 of the June 2001 Major Permit Modification Application for Class III Cell B for the most recent hydrogeologic investigation report for the SFRL site. No changes are proposed to the requirements of this section.

DBT to update the potable well inventory and provide a figure showing potable wells within 500 feet of the waste storage and disposal areas.

H.2 CERTIFICATION

No change is proposed to the existing hydrogeological investigation.

PART I

GEOTECHNICAL INVESTIGATION REQUIREMENTS

Refer to 1986, 2001 and 2010 permit applications for previous geotechnical investigation data for the SFRL. Based on the review of this data, the opinion is that no additional investigation is required for the proposed expansion areas and that the previous studies generally represent the expansion footprint. Slope stability, settlement and foundation analysis as provided in Appendices B and C utilize existing geotechnical data for the evaluations. In addition, based on the sinkhole map for FL (Refer to Appendix B), Bay County is in a region that is not favorable for sinkhole development.

PART J

VERTICAL EXPANSION REQUIREMENTS

The proposed expansion involves vertical expansion of the existing Class I landfill.

J.1 IMPACTS TO EXISTING LANDFILL

Leachate generation and collection has been addressed in Section G of this report. The calculations provided in Appendices B and C evaluate slope stability, foundation capacity and settlement and address any concerns regarding adverse impact to the LCRS of the existing landfill. The expansion will follow similar landfill operational procedures as currently exist and will not result in any objectionable odor issues.

J.2 VERTICAL EXPANSION OVER UNLINED AREA

There is no vertical expansion over unlined landfill area. Cells 1 and 2 are lined with a single liner. Currently these cells are closed and no leachate generation is reported. However, the leachate conveyance header is extended below the expansion liner footprint to a pump station outside of the expansion footprint.

J.3 FOUNDATION AND SETTLEMENT ANALYSIS (ERTICAL EXPANSION)

The foundation and settlement analysis report is provided in Appendix C.

J.4 TOTAL SETTLEMENT ANALYSIS

Total settlement calculations are provided in Appendix C.

J.5 STABILITY ANALYSIS

Slope stability calculations are provided in Appendix B.

J.6 STORMWATER SYSTEM

The proposed expansion will not adversely impact the existing stormwater management system. The final cover for the expansion area is designed such that the stormwater runoff flows to the on-site stormwater pond located towards the north of landfill footprint via a series of terraces, downcommers and perimeter channels. The perimeter channel and culvert capacity is confirmed as part of current design as shown in Appendix I.

J.7 GAS MIGRATION CONTROLS

Currently the Class I landfill mostly accepts WTE ash which creates a minimal amount/no landfill gas generation. There are landfill gas probes installed at the property boundary to monitor offside gas migration. Bay County will install gas migration controls if deemed necessary in the future.

PART K

LANDFILL OPERATION REQUIREMENTS

A landfill Operations Plan, which contains a contingency plan and addresses the requirements of Rule 62-701.500, FAC, was submitted previously to FDEP. However minor changes to the Operations Plan were necessary due to the proposed expansion as shown in Appendix A. Proposed changes to the operations plan are indicated as ~~strike through~~ (for deletions) and <u>underline</u> (for additions).

PART L

WATER QUALITY MONITORING REQUIREMENTS

The groundwater and surface water monitoring requirements have been integrated into the current Water Quality Monitoring Plan (WQMP) provided as Section 2.14 of the Operations Permit for the Steelfield Road Landfill (modified on January 28, 2013). A detailed review of the existing WQMP was included in the Water Quality Report, dated September 2014. The report provides recommendations on the adequacy of current WQMP. The proposed expansion does not propose any changes to water quality monitoring requirements.

PART M

SPECIAL WASTE HANDLING REQUIREMENTS

The Operations Plan in Appendix A addresses the management of Special Waste.

PART N

GAS MANAGEMENT SYSTEM REQUIREMENTS

The majority of the waste stream accepted at SFRL Class I Cells was transported from the WTE facility. The SFRL operates under Title V Permit No. 0050090-004-AV. Per the Tier II test results reported on January 06, 2017, the non-methanogenic organic compound (NMOC) emission rate estimated in year 2021 at SFRL is 7.672 Mg/year. No gas management system is required at SFRL since the NMOC emission rate is less than 34 Mg/year. Per the Title V permit requirements, a detailed GCCS design report will be submitted if the facility triggers NMOC threshold limit based on site-specific NMOC concentrations.

PART O

LANDFILL FINAL CLOSURE REQUIREMENTS

A Closure Plan for SFRL was submitted to FDEP as part of Operations Permit Renewal in 2014. This plan is now updated as shown in Appendix J.

PART P

OTHER CLOSURE PROCEDURES

A Closure Plan for SFRL discussing other closure procedures was submitted to FDEP as part of Operations Permit Renewal in 2014. This plan is now updated as shown in Appendix J.

PART Q

LONG TERM CARE REQUIREMENTS

No changes have been proposed to long-term care requirements of this section. These requirements were previously addressed in the Closure Plan submitted with the Operations Permit Renewal in 2014 (refer to Appendix J).

PART R

FINANCIAL ASSURANCE

R.1 ESTIMATED COSTS

Preparation of Financial Assurance Cost Estimates and completion of form 62-701.900(28) F.A.C. to be performed by DBT.

R.2 ANNUAL COST ADJUSTMENTS

Cost estimates for closure and long-term care will be adjusted and provided to the FDEP prior to September 1 of each year. Costs will be listed separately for closure and for long-term care. These estimates will be based on one of the two following methods as appropriate:

a. Recalculation of the total cost in current dollars, prepared and certified by a professional engineer for a third party performing the work, on a per unit basis, with the source of estimates indicated.

b. Inflating the previous year's estimate by the appropriate inflation factor as required by Rule 62-701.630(4) F.A.C.

During the life of those portions of the landfill that have not been finally closed, as well as during the long-term care period, the SFRL will revise the closure cost estimate for inflation and changes in the closing and long-term care plan. Such revisions shall be made annually and used as the basis for comparison against the Guarantee Bond in place for SFRL.

R.3 FUNDING MECHANISMS

Bay County uses an Escrow Account as its financial mechanism to demonstrate financial responsibility for the closure and long-term care of the landfill.

APPENDIX A
REVISED OPERATIONS PLAN

Bay County Steelfield Road Landfill

Operations Plan

Facility ID Number: NWD/03/0000016
Permit No.:
0007875-009-SO-01 & 0127100-005-SF-T3
~~0007875-005-SO & 0007876-006-SF~~

Prepared for:

Bay County General Services Department
11411 Landfill Road
West Bay, FL 32413

Revised by:

HDR, Inc.
76 S. Laura St, Suite 1600~~200 W Forsyth St, Suite 800~~
Jacksonville, Florida 32202
904-598-8900

Revised on: ~~September 2014~~March 2019

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Table 2 Information and Forms Used at the Landfill
Table 3 Landfill Equipment

ATTACHMENTS

Attachment A-1 Mining Plan
Attachment A-2 Contingency Plan
Attachment A-3 Site Plan

Note: *A copy of the current Operations Permit must be kept with this Operations Plan for reference to sampling frequency and procedures for perimeter gas probe monitoring, groundwater monitoring and surface water monitoring.*

OPERATIONS PLAN

The Bay County solid waste management system is an integrated system comprised of two (2) components, The Bay County Steelfield Road Landfill (landfill or facility) and Waste-to-Energy Facility (WTEF). The Landfill began operations in 1987 and is operated by the Bay County ~~General Services Department (BCGSD~~BCSWD~~),~~ Solid Waste Division (BCSWD). The Steelfield Road Landfill consists of Class I and Class III Landfills. Class I Landfill areas consist of closed Cells 1 and 2, active Cells 3, 4 and 5, and future Cells 6 ~~through 12~~and 7. Class III landfill areas consist of partially closed Cell A, inactive Cells Aa and B.

The facility also performs waste mining operations to excavate waste disposed in Class III Cells A, Aa and B. These mining activities are performed in accordance with the mining plan provided in June 2012 Permit Modification Application. Per the mining plan, excavated soil/waste will be partially reused as cover material, and/or disposed in the lined Class I landfill cells. No new waste is placed in, or re-located within, the unlined Class III Cells A, Aa and B. For reference, the mining plan is provided as Attachment A-1 because the mining plan includes certain landfill operational guidelines that can be considered as part of landfill operation plan. For attachments to the mining plan refer to the June 2012 Minor Permit Modification and associated Response to Request for Additional Information.

The Steelfield Road Landfill originally functioned primarily as a repository for ash residue from the WTEF and as a disposal site for Class III materials, Construction and Demolition Debris and other non-burnables that could not be processed at the WTEF. Bay County (County) has continued to grow and the county has had to adapt its facilities to meet this growth. The Steelfield Road Landfill facility now receives most of the Class I Municipal Solid Waste (MSW) generated in the western portion of the county. The WTEF requires a certain amount of tonnage in order to sustain steam flow and electrical generation. At times it is advantageous to transfer a portion of the waste being delivered to the Steelfield Road Landfill to the WTEF to insure sufficient fuel for efficient operation of the facility.

1.0 Designated Staff and Equipment (62-701.500(1), FAC)

Normal working hours at the landfill are Monday through Saturday from 7:00 am until 4:30 pm. Operating hours may be extended as needed depending on situation and workload.

A certified Landfill Operator, having satisfied the requirements of Chapter 62-701.320(15), FAC will be on site at all times that the facility is accepting waste. A certified Transfer Station Operator will be on site at all times that waste is being transferred from the facility. All heavy equipment operators shall be trained as spotters, transfer station operators, and/or landfill operators. Training will be conducted either through the Solid Waste Association of North America or the TREEO Center. A current list of available courses is posted online on the organizations' websites.

A list of landfill staff is provided on the next page in Table 1 with training for each personnel.

Table 1. List of Landfill Staff and Trainings Received*	
Name	**Training Received**
John Beals, Belinda Davis, Rosella Davis, Cristi Karr, David Kelly, Kathleen McDonald, Cynthia Thompson, and Sharon Tinsley	Spotter / Waste Screener
James Dorough and James George	Class I, III Landfill Operator
	Construction and Demolition Debris Landfill Operator
	Material Recovery Facility Operator
	Spotter / Waste Screener
	Transfer Station Operator
Leroy Goodman, Donald Miller and Cory Parsons	Class I, III Landfill Operator
	Construction and Demolition Debris Landfill Operator
	Spotter / Waste Screener
Johnny Gurganus	Class I, III Landfill Operator
	Construction and Demolition Debris Landfill Operator
Mitchell Harris	Material Recovery Facility Operator
	Spotter / Waste Screener
	Transfer Station Operator
Charles Johnson	Class I, III Landfill Operator
	Molo Swana Certication
	Construction and Demolition Debris Landfill Operator
	Material Recovery Facility Operator
	Spotter / Waste Screener
	Transfer Station Operator
Otis Johnson and Barney McCosh	Class I, III Landfill Operator
	Construction and Demolition Debris Landfill Operator
	Material Recovery Facility Operator
	Spotter / Waste Screener
	Transfer Station Operator
Glenn Ogborn	Class I, III Landfill Operator
	Molo Swana Certication
	Construction and Demolition Debris Landfill Operator
James Vinson	Class I, III Landfill Operator
	Spotter / Waste Screener
* ~~Updated per information provided by Bay County in June 2014.~~DBE to updated based on the information collected from the County	

2.0 Operation Plan (62.701-500(2), FAC)

2.1 Designation of Responsible Personnel

Solid Waste ~~Superintendent~~Division Manager– Mr. Glenn Ogborn – is responsible official for the landfill and is responsible for overall planning, day-to-day operations and maintenance of the landfill.

2.2 Emergency Preparedness and Response

The Contingency Plan, as required by 62-701.320(7) (e) (2), FAC, is provided as Attachment A-2 of this Operations Plan. Routine operations may be temporarily modified, curtailed or suspended in the event of a hurricane or other natural disaster. The landfill maintains a Hurricane and Severe Weather Landfill Action Plan (see Contingency Plan) which sets forth specific actions to be taken immediately prior to, during, and following an emergency event to minimize danger to personnel and damage to facility structures and equipment. These procedures provide for post-emergency event damage assessment, repair, and prompt return to service.

The facility is designated for post-emergency staging, temporary storage and processing of storm or other emergency debris. The specific configuration and details of such post-emergency activities will be dependent upon the nature of the emergency event. ~~BCGSD~~BCSWD will coordinate closely with the Florida Department of Environmental Protection (FDEP) prior to implementing the referenced activities at the facility.

2.3 Waste Processing Procedures

Figure 1 identifies the normal waste flow process at the Steelfield Road Landfill.

The landfill has sufficient equipment to ensure proper operation of the landfill and for excavating, spreading, compacting, and covering waste; sufficient reserve equipment or arrangements to obtain additional equipment within 24 hours of equipment breakdown; and equipment for emergency and routine communications. Additionally, signs are posted indicating the name of the operating authority, traffic flow, hours of operation and disposal restrictions.

All industrial waste is required to pass a Toxic Characteristic Leaching Procedure (TCLP) prior to receiving authorization for disposal. TCLPs will be submitted to the Bay County Lab for evaluation. Upon receiving confirmation from the lab manager that the material tested is not hazardous, the Solid Waste Superintendent will issue and authorization number for the disposal of the waste. This authorization will be for a limited period and will have to be renewed upon expiration. The scale operator will not accept Industrial waste without an authorization number.

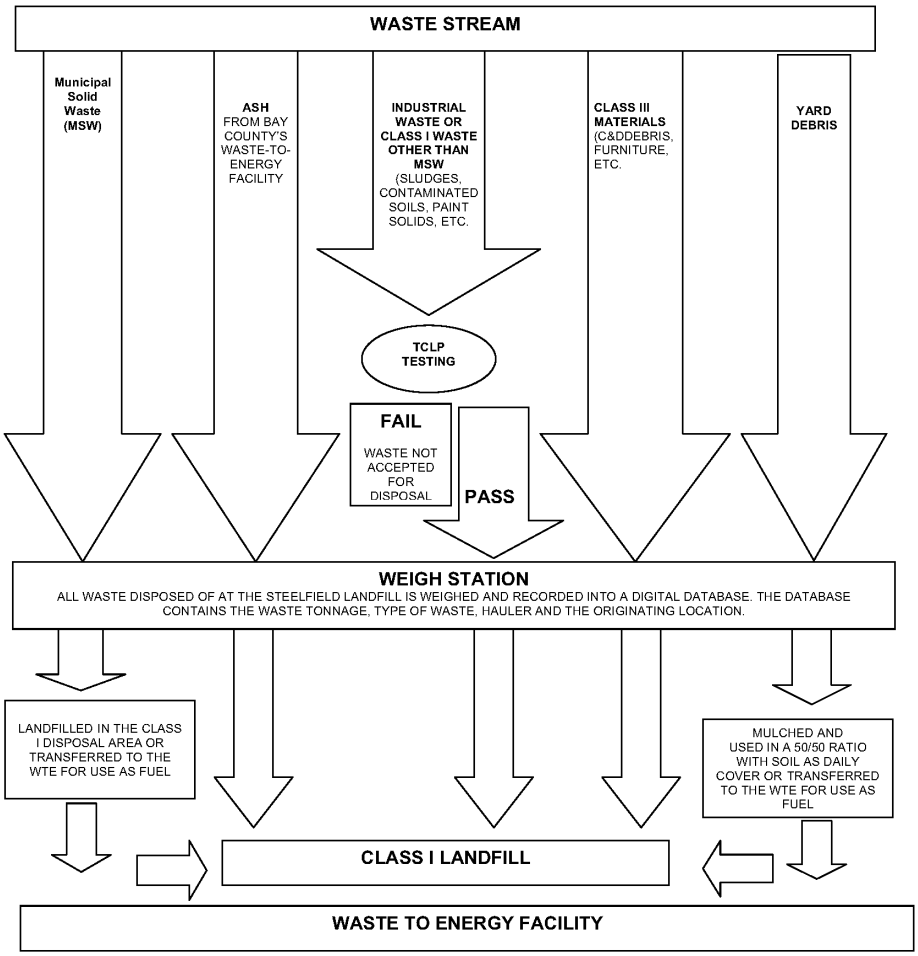

Figure 1. Process Flow Diagram

2.4 Controlling types of waste received at the landfill

The detailed process flow diagram provided as Figure 1 shows incoming wastes received at the facility and management of each of the waste components. In general, the following procedures are in place to control the type of wastes received at the facility.

Signage: The County maintains signage at the entrance to the landfill facility that clearly identifies wastes which may enter the site and those which are prohibited.

Vehicles Processing: Solid waste is delivered to the facility from pickup points throughout the county. Haulers enter the facility through the truck gates and proceed to the scale house. All waste entering the facility is processed through the scale house. The scale house attendant takes the driver's information and directs them to the inbound scale. The truck pulls onto the scale to be weighed in. There is a traffic light at the end of the scales that lets the driver know when they have been weighed. The light is red when the driver pulls onto the scale and turns green once weighing is complete. Once the driver receives the green light, the truck leaves the scale for the tipping face. The light turns red and the scale is ready to perform the next transaction.

Random Inspections: The heavy equipment operator conducts a minimum of three (3) random load checks each week. When the heavy equipment operator selects a vehicle for random screening, the Scale Attendant on duty is notified and provided with the vehicles information. This allows electronic tracking of random load checks. In addition the heavy equipment operator will fill out a Random Load Check form. This form will be retained for review by the FDEP.

Working Face: Spotters and equipment operators spot check waste loads at the working face. If unacceptable wastes are identified and the person disposing of the waste is present, the waste is reloaded into the vehicle and removed from the site. The individual responsible for the incident is advised of the nature of the problem, warned regarding recurrence, and provided information regarding proper management of the rejected wastes.

Any unauthorized or hazardous wastes will be handled as described below:

***2.4.1 Universal Waste*:** Universal wastes are household type hazardous wastes that contain substances hazardous to human and environmental health. Examples of these wastes are batteries, fluorescent tubes, and PCB containing devices. These wastes are prohibited items and must be segregated from the waste stream. All universal waste collected from residential customers will be processed through the on-site Household Hazardous Waste facility (HHW). Universal waste will normally not be accepted from commercial customers. Universal waste pulled from the waste stream at the tipping face will be transferred to the HHW facility for processing.

2.4.2 White Goods: White goods are prohibited from landfill disposal and must be segregated from the normal waste stream. White goods will be processed as follows:

- Refrigerators and freezers that contain food products will not be accepted for disposal. Refrigerated appliances that have the door or opening taped shut or otherwise secured will be assumed to contain food products and will not be accepted.
- White goods collected from residential customers must be placed in the designated white goods staging area until they can be sanitized, and the refrigerant can be removed. This staging area has storage capacity for approximately 3,000 appliances and or small engine containing devices. See site plan provided as Attachment A-3.

- White goods pulled from the waste on the tipping face will be staged away from the active face until they can be transported to the white goods processing area.
- *Refrigerant Removal Process:* All refrigerant extraction will be performed in accordance with the following safe disposal requirements outlined in Section 608 of the Clean Air Act of 1990.
- *Record Keeping*: The technician will keep a record of all appliances from which Freon is extracted. The record shall include the date, name and signature of person performing the extraction, address of person performing the extraction, and the type and amount of refrigerant extracted. See Bay County Solid Waste Extraction Log (sample) provided as Figure 2.

2.4.3 Special Waste Disposal: Special wastes, as specified in Rule 62-701.520(1) to 62-701.520(5), are not accepted for disposal at the facility except for asbestos and treated biomedical waste.

- Asbestos waste is disposed in designated areas in Class I landfill in accordance with the following guidelines:
  - Asbestos-containing waste materials are accepted for disposal in designated Class I active cells in accordance with the Rule 701.520(3)(a);
  - The waste generator will make arrangements with the landfill operator before disposal of such regulated asbestos-containing waste materials, and inform the operator of the quantity of the waste and the scheduled date the shipment will arrive at the landfill.
  - The landfill operator will direct the waste transporter to the designated disposal location. The disposal location will be recorded in accordance with 40 CFR Part 61.154, and a record of the asbestos location will be maintained.
- Treated biomedical waste handling will follow following guidelines per 62-701.520(5)(d):
  - Treated Biomedical waste must be in containers clearly labeled with the phrase "Treated Biomedical Waste." The local governments that are responsible for solid waste collection and disposal shall be notified that

treated biomedical waste will be disposed of in their facility before such disposal. This requirement does not supersede the provisions of Section 381.0098(8), F.S., regarding acute care hospitals. All transport vehicles transporting treated biomedical waste to a solid waste facility for disposal shall be fully enclosed and secured when unattended. This provision shall not be construed as superseding a solid waste management facility operator's authority to set limitations or restrictions on the disposal of treated biomedical waste at that facility.

REFRIGERANT EXTRACTION LOG

BAY COUNTY SOLID WASTE
3410 TRANSMITTER RD.
PANAMA CITY, FLORIDA 32404
OFFICE: 850-784-4028

PERSON PERFORMING REFRIGERANT EXTRACTION		DATE	
SIGNATURE			
ADDRESS			

	TYPE REFRIGERANT AND QUANTITY REMOVED (IN OUNCES)					
STARTING TANK WEIGHT						
Date	R-12	R-22	R-134	R-410A	OTHER	No. of units

Figure 2. Solid Waste Refrigerant Extraction Log

2.4.4 Small Engine Containing Devices/Used Oil Filters: Small engine devices include lawn mowers, weed eaters and similar items. Small engine devices and used oil filters contain oil or oil residue. They must be segregated from the normal waste stream for processing. Small engine devices will be processed in the same manner

as white goods. Used oil filters will be stored in the facilities Household Hazardous Waste Facility until the can be processed/recycled.

- *Small Engine Containing Devices Processing*: The attendant on duty will remove all oil/fuels from all lawn mowers, tools and other equipment that has a small gas powered engine. Once sanitized of petroleum projects the items will be placed in the same pile as the sanitized white goods.

***2.4.5 Yard Debris*:** Yard debris cannot be disposed of in a lined cell. Yard debris collected at the facility will be stockpiled and processed into mulch suitable for use as daily cover or intermediate. When mixed with soil in a 50/50 ratio, road base or transferred to the WTEF for use as fuel.

2.4.6 Yard Debris Processing: Yard debris processing will be carried out on site via a large commercial tub or horizontal grinder. Grinder operations will be conducted either under contract or by the County personnel.

2.4.7 Hazardous Waste Handling: Any hazardous waste found at the facility will be handled in accordance with the guidelines provided in the contingency plan attached to this operations plan.

The landfill facility shall make arrangements or shall have equipment for temporary storage, handling and transport to an authorized disposal or recycling facility for unauthorized waste which is inadvertently accepted by the facility. Putrescible unauthorized waste shall not be stored for longer than 48 hours. Non-putrescible authorized waste shall not be stored for longer than 30 days.

2.5 Scale House Operations

2.5.1 Waste Vehicle Processing: Solid waste vehicle processing will be carried out in accordance with the vehicle processing procedure listed in Section 2.4 of this Operations Plan.

2.5.2 Documentation/Recordkeeping: Records will be kept of all information used to develop or support the permit applications and any supplemental information submitted to comply with this chapter pertaining to construction of the landfill throughout the design period. Records pertaining to the design and operation of the landfill shall be kept for the design period of the landfill. Records will be retained of all monitoring information, including calibration and maintenance records, all original chart recordings for continuous monitoring instrumentation, and copies of all reports required by permit for at least ten years. Background water quality records shall be kept for the design period of the landfill. An annual estimate will be maintained of the remaining life and capacity in cubic yards of the existing, constructed landfill and remaining capacity and site life of other permitted areas not

yet constructed. The annual estimate shall be based on a summary of the heights, lengths, and widths of the solid waste disposal units. The estimate shall be made and reported annually to the Department. Records which are more than five years old and which are required to be retained may be archived, provided that the landfill operator can retrieve them for inspection within seven days. At the scale house a record will be created that documents the following:

- net weight of the material being delivered
- origin of the waste
- type of waste
- tipping fee accessed
- time of transaction
- customer data including:
  - company name (if any)
  - truck number (if any)
  - can or container number (if any)
  - tag number (if non-commercial)

Information on the recordkeeping requirements is further provided in Sections 3 and 13 of this Operations Plan.

2.6 Screening Operations

2.6.1 Scale House Pre-screening Operations: The scale operator on duty will attempt to verify the type of materials being received and whether or not the material contains any prohibited items or items that may require special handling. This will be accomplished by:

- Questioning the customer as to exactly what kind of materials are contained within the load and whether or not the load contains any items that may be prohibited or that may require special handling.
- To as great an extent as possible all untarped or exposed loads will be visually inspected to verify type of material being received and to screen for prohibited waste. This visual inspection may include an examination of the load or screening the load via an overhead mounted camera system.
- Acceptable waste will be directed to the appropriate disposal location. Commercial loads and large non-commercial loads of MSW will be directed to the tipping face area. Small, residential loads may be directed to use containers in the on-site drop-off area.

2.6.2 Tipping Face Screening Operations: The heavy equipment operator will be the primary spotter in charge of screening waste at the tipping (or active/working) face. All heavy equipment operators shall be trained as either spotters or transfer station operators or landfill operators. Screening operations at the tipping face will be conducted as follows:

- MSW and other acceptable solid waste will be deposited on the facility's tipping face. The heavy equipment heavy equipment operator will watch the unloading process to observe any prohibited waste.

- The heavy equipment operator will spread the load as it is pushed across the slope of the working face. This maximizes compaction and enables the heavy equipment operator to visibly inspect the load for prohibited items.
- During transfer operations the heavy equipment operator will inspect the waste on the tipping face as he is removing material from the waste pile and loading it into a transfer vehicle.
- If unacceptable waste is observed, the heavy equipment operator will remove it for proper disposal.
- A litter policing operation shall be employed to keep litter from leaving the working area of the landfill. Litter outside the working area shall be picked up within 24 hours.
- The landfill shall have litter control devices, portable fences, or other suitable devices to manage litter.

2.6.2 Random Load Inspections: The heavy equipment operator will conduct a minimum of three random load inspections each week. When the heavy equipment operator selects a vehicle for random screening, the scale operator on duty will be notified and provided with the vehicles information. This will allow electronic tracking of random load checks. In addition the heavy equipment operator will fill out a Random Load Check form. This form will be retained for review by the FDEP.

2.7 Tipping Face Operations

2.7.1 Traffic Control: The heavy equipment operator(s) working the tipping face will maintain control of customer traffic at all times. Customers will be directed to a tipping area that will be the minimum width needed to safely handle the flow of traffic.

- *Traffic Safety* - The heavy equipment operator and/or spotter directing traffic on the tipping face will wear a Day-Glo orange safety vest at all times as well as any other personnel protective gear that may be required.

2.7.2 Tipping face slope: The slope of the tipping face shall be maintained to maximize the effectiveness of the compactor. The slope should be at least at 3 (H) to 1 (V).

2.7.3 Lift height: Lifts shall be constructed in heights form 6 to 8 feet. The maximum lift height shall not exceed 10 feet.

- *First layer thickness* – The first layer of waste placed above the liner and leachate collection system shall be a minimum of four feet in thickness and compacted thickness and consist of selected wastes containing no large rigid or protruding objects that may damage the liner or leachate collection system.

2.7.4 Waste compaction: Waste shall be spread as thinly as possible along the tipping face to maximize compaction. Once spread, the waste the operator will go

over the waste for a minimum of 3 to 5 passes with the garbage compactor. Large bulky items that cannot be spread or compacted along the tipping face will be placed at the toes of the tipping face and the tipping face shall be constructed around them.

2.7.5 Daily Cover: Daily cover will be applied to the working face at the end of each business day. The daily cover is appropriately selected to control vector breeding/animal attraction, blowing of litter, odors and moisture infiltration. Daily cover shall consist of any of the following:

- Six inches of ash
- Six inches of soil
- Six inches of a 50/50 mix of soil and yard debris mulch
- A covering of latex paint
- A tarp or similar covering
- Separated materials made up of cover sand and composted organics from Class III mining activities

2.7.6 Working Face: The working face shall be wide enough to accommodate vehicles discharging waste, but no more than 100 feet to minimize the exposed area and unnecessary use of cover material.

2.8 Method and sequence of filling waste

After completion of filling actives within existing Class I footprint, the County will develop the next double lined cell located south of existing Class I footprint with an upslope liner partially covering existing Cells 1 and 2.

The County has requested FDEP to temporarily increase the currently permitted maximum landfill elevation from 198 NGVD to 220 NGVD and steepen the southern and western sideslopes to accommodate emergency debris received after hurricane Michael. The west slope of existing Cell 4 will be filled at a 2:1 slope whereas the south slope of Cells 3, 4 and 5 will be filled to a 2.5:1 slope.

As shown in the plans, Cells 6 and 7 will be constructed south and east of existing lined landfill footprint. Cell 6 is divided into Sub-Cells 6A and 6B to facilitate stormwater control during initial stages of filling. The development of the landfill after Cells 6 and 7 are constructed will be finalized at a later date.

~~After the mining activities in Cells A, Aa and B are completed, the area will be lined and used as future Class I Cell 6. Future Class I Cells 7 and 8 are located on west side of the exiting landfill, and futures Class I Cells 9 through 12 are located on South side of the existing landfill. Cells 6 through 12 will be used for waste disposal in the numerical sequence. Moreover Phases I through IV will provide additional space for waste disposal as~~

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~~future Class I landfill areas. The future disposal areas are shown on the Figures 4A and 4B provided in Appendix A of this permit renewal application.~~

2.9 Operations of gas controls

The current landfill gas management system is comprised of passive gas vents and gas probe monitoring locations, the maintenance building, and the scale house. The passive gas vents were installed in the closed landfill areas to alleviate any potential gas pressure. For a safety precaution, flame and detonation arresters have been installed on all passive gas vents. Flame arresters permit normal gas vent breathing while preventing the flashback of an unconfined deflagration (the ignition of the vapor cloud in the open atmosphere) from flashing back past the arrester into the piping of the passive gas vent wells.

The landfill gas probe locations have been shown on the Site Plan provided as Attachment A-3. The gas probe monitoring will be performed as specified in the Operations Permit. A copy of permit will be kept on-site along with this Operations Plan for reference to the gas probe monitoring procedures and sampling frequency.

2.10 Operation of Leachate Control System

Leachate generated within Class I landfill area drains by gravity via a series of header pipes, ~~and~~ collection lines, manholes, forcemains and pump stations to a central lift station. Leachate is then pumped into ~~a~~ three 150,000~~100,000~~ gallon storage tank with a secondary containment wall of ~~120~~180,000 gallons. The existing gravity leachate conveyance system will be extended beneath Cell 6 and the existing leachate storage tank will be removed and replaced with three 150,000 gallon tanks. Leachate from the storage tanks is transported to the Beach Advanced Wastewater Treatment Facility (AWT) by in house personnel or by a commercial contractor. New leachate collection systems shall be water pressure cleaned or inspected by video recording after construction but prior to initial placement of wastes.

2.10.1 Loading: All loading of leachate tankers will take place on the new loading pad constructed next to the proposed storage tank farm. Prior to loading the operator will insure that the leachate diverter valve is open on the drain pad so any leachate that may be spilt during loading operations will drain back into the lift station.

2.10.2 Weighing: The operator will load the tanker to the pre-measured level on the tanker sight tube, if so equipped. Once full the tanker will be weighed on the outbound scale to ensure that the load is within legal transportable limits. If the load exceeds permissible limits then the operator will:

- Go back to the loading drain pad.
- Verify that the leachate diverter valve is open.

- Discharge a quantity of leachate sufficient to meet the maximum transport weight capacity.
- Re-weigh the tanker truck combination to make sure it is under the maximum permissible limit.
- After discharging the leachate at the appropriate treatment facility and returning to the landfill the empty truck/tanker combination will again be weighed so a net weight can be established.
- The landfill operator is responsible for the operation of the leachate collection and removal system and for maintaining the system as designed for the design period. Leachate shall be collected and treated as necessary so that water quality standards and criteria are not violated. If the leachate is classified as a hazardous waste, it shall be managed in accordance with Chapter 62-730, F.A.C.

2.10.3 Contingency Operations: In the event that the Beach AWT cannot accept leachate or if it becomes more economical to do so, leachate will be transported to the County AWT located on Tyndall AFB.

2.10.4 Precipitation and Leachate Comparison: A recording rain gauge is maintained to record precipitation at the landfill. Precipitation records are included with the operating record and are maintained and used by the County to compare with leachate generation rates.

2.10.5 Inspections: Inspection of leachate control system will be performed as follows:

- **Leachate Tank**: The exposed exterior of leachate tank shall be inspected weekly for adequacy of the cathodic protection system, leaks, corrosion, and maintenance deficiencies. Interior inspection of tanks shall be performed whenever the tank is drained or at a minimum of every three years. If the inspection reveals a tank or equipment deficiency, leak, or any other deficiency which could result in failure of the tank to contain the leachate, remedial measures shall be taken immediately to eliminate the leak or correct the deficiency. Inspection reports shall be maintained and made available to the Department upon request for the lifetime of the liquid storage system.
- **Secondary Containment Area**: Any stormwater collected in the secondary containment area will be visually inspected on weekly basis. If collected stormwater, based on its color and odor, appears to be leachate (or diluted leachate), such stormwater must be handed as leachate.

2.11 Operation of Stormwater Control System

Each landfill owner or operator shall have an operational plan that provides written, detailed instructions or the daily operation of the landfill. The operation plan shall be kept at or near the landfill facility and shall be accessible to landfill operators. The operation plan shall be

substantially complied with at all times, and shall be revised if operational procedures change. The stormwater controls and/or erosion control measures shall be employed to correct any erosion which exposes waste or causes malfunction of the storm water management system. Such measures shall be implemented within three days of occurrence. If the erosion cannot be corrected within seven days of occurrence the landfill operator shall notify the Department and propose a correction schedule.

Stormwater management systems shall be operated and maintained as necessary to meet the requirements of subsection 62- 701.400(9), F.A.C. Stormwater or other surface water which comes into contact with the landfilled solid waste or mixes with leachate shall be considered leachate and is subject to the requirements of subsection 62-701.500(8), F.A.C.

2.12 Recycling Operations

2.12.1 Recycling Drop Off Center: The Landfill operates a recycling drop off center. The recycling drop off center consists of driving ramp with four ten cubic yard containers. In addition, eleven recycling containers are located in the recycling area with each container having approximately ten cubic yard capacity. Recycling opportunities are available for paper, cardboard, plastic (PETE 1&2) and mixed metals. See site preparation plan provided ~~as Attachment 3~~in the drawing package for the relocated Recycling Drop Off Center.

2.12.2 Post Burn Metals Extraction: A post burn ferrous metals extraction program was implemented by Bay County in October of 1996. Under this program, ferrous metals were removed from the ash residue at the landfill cell by a private contractor. The ferrous metals removal process was accomplished utilizing a crane-mounted magnet. Bay County terminated its contract with the private contractor and now conducts post burn metal extraction at the WTEF prior to transporting ash to the landfill.

2.12.3 Ash Management: As part of the total ash management program, the County will continue to utilize ash as a beneficial resource as a landfill operations cover material in lined areas of the Class I landfill.

2.13 Transfer Station Operations

2.13.1 Loading: All waste loading, unloading, storage and processing procedures related to the transfer of MSW will take place within the confines of the currently operational Class I disposal area. Mulched yard debris may be loaded from within the yard debris staging area. Transfer station operations will be conducted by the direct loading of open top type transfer trailers via a front end loader or grizzly style crane. The loaded trailer will then be transported to the WTEF for utilization as fuel.

2.13.2 Transfer: Normal transfer operations will be based on the ability to utilize the ash transfer trailers to back haul fuel (waste) from the landfill to the WTEF. The transfer vehicles will discharge the ash residue from the WTEF and pick up a load of MSW or mulched yard debris for transport back to the WTEF for utilization as fuel. During transport to the WTEF, the trailers will be tarped to control the release of litter.

2.14 Water Quality Monitoring

The water quality monitoring plan can be found in the existing Operations Permit for the facility. A copy of operations permit will be kept on-site along with this Operations Plan for reference to the water quality monitoring requirements, including sampling locations and sampling frequencies. The monitoring locations (groundwater and surfacewater) are also indicated on the Site Plan provided as Attachment A-3. No changes to the current water quality monitoring program are proposed at this time.

2.15 Maintaining and Cleaning the Leachate Collection System

In the event a suspected blockage has occurred in the existing leachate collection system, hydro jetting will be used to clean the leachate lines. The lines may also be inspected with a camera to determine where a problem might exist.

New leachate collection systems shall be water pressure cleaned or inspected by video recording after construction but prior to initial placement of wastes. Existing leachate collection systems shall be water pressure cleaned or inspected by camera at least once every five years. Results of the collection system cleanings or inspections shall be kept on-site in accordance with recordkeeping requirements listed in Section 13. The documentation shall include at the minimum following details:

- General details (a signed letter/report with company name that performed the cleaning/video inspection, dates & time for jet ~~clening~~cleaning/video inspection, any historical issues associated with jet cleaning/video inspection, etc.);
- Pipe IDs that were jet cleaned/video inspected; for example, Cleanout 1 was jet cleaned/video inspected;
- Length of each pipe jet cleaned/video inspected; for example, cleanout 1 was jet cleaned/video inspected for 400' feet; and
- Any obstruction or unusual situation occurred during jet cleaning/video inspection. For example, Cleanout 2 was jet cleaned 20' only as pressure hose did not go beyond.

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3.0 Operating Record (62-701.500(3), FAC)

The landfill maintains an Operating Record consisting of all records, reports, analytical results, demonstrations, and notifications required by Chapter 62-701, FAC, all permits and permit modifications, and training records as required by subsection 62-701.320(15), FAC. The Operating Record is maintained within the filing system at the landfill. Records that are more than five years old and are required to be retained will be archived. The landfill operator can retrieve all archived records for inspection within seven days. This practice will continue for the design life of the landfill.

Forms presently used in documenting and reporting these operations of the landfill are listed in **Table 2**. Information and forms will be updated as needed with changes reported to FDEP.

Table 2: Information and Forms Used at the Landfill

Item	Frequency	Distribution
Public & Customer Information		
Information Package	As Needed	Facility User
Hazardous & Special Waste Information Sheet	As Needed	Facility User
Special Waste Companies	As Needed	Special Waste Generators
Separation Information	As Needed	Facility Users
Account Application	As Needed	Commercial Customers Desiring To Open An Account
Disposal and Income Records		
Monthly Status Reports	Monthly	~~BCGSD~~BCSWD, Office
Comparison Current FY to Previous FY	Monthly	~~BCGSD~~BCSWD, Office
Comparison, Current Month to Same Month Previous Year	Monthly	~~BCGSD~~BCSWD, Office
Scale Weights	Monthly	~~BCGSD~~BCSWD, Office
Wood Waste Reports – Incoming Tonnages	Monthly	~~BCGSD~~BCSWD, Office
Annual Category Report	Annual	~~BCGSD~~BCSWD, Office, FDEP
MSW Report: Tons Per Day	Monthly	~~BCGSD~~BCSWD, Office
Cash Account Totals	Monthly/Update Daily	~~BCGSD~~BCSWD, Office, Finance Dept.
Cash Receipts Report	Daily	~~BCGSD~~BCSWD, Office, Finance Dept.
Daily Reconciliation Report	Daily	~~BCGSD~~BCSWD, Office, Finance Dept.

Waste Identification / Quality Control		
Random Load Inspection Report	Min. 3 Loads Per Week	~~BCGSD~~BCSWD, Office
Asbestos Disposal Data	Each Asbestos Waste Delivery	Office
Disposal Violation Warnings	Monthly	Office
Weather / Environmental Monitoring Reports		
Rainfall Data Sheet	As specified in Operations Permit	Office
Monthly Rainfall Totals	As specified in Operations Permit	Office
Well Construction Report	As specified in Operations Permit	Office, FDEP
Water Level Report	As specified in Operations Permit	Office, FDEP
Groundwater Monitoring Reports	As specified in Operations Permit	Office, FDEP
Leachate Hauling & Storage Report	As specified in Operations Permit	Office, FDEP
Quarterly Gas Monitoring Report	As specified in Operations Permit	Office, FDEP
Maintenance / Repair Forms and Reports		
Work Order	Each Ind. Task/Job	Office
Daily & Routine Service – Heavy Equipment Maintenance	Daily	Office
Weekly Storage Tank Inspection Data Sheet (Exterior Inspections)	Weekly	Office
Storage Tank Inspection Data Sheet (Interior Inspections)	At a minimum every Three Years	Office

4.0 Waste Records (62-701.500(4), FAC)

Waste records for the landfill that are kept onsite for operations personnel use and FDEP inspection are included in Table 2. Waste reports are compiled monthly and submitted to the FDEP annually.

5.0 Control of Site Access (62-701.500(5), FAC)

The landfill is surrounded on all sides by natural barriers, fencing, or an equivalent means of controlling vehicle access and preventing illegal disposal. Access is limited by locked gates. Access to the facility shall be permitted only when an attendant is on duty and only during normal operating hours, unless otherwise authorized by the FDEP.

6.0 Monitoring of Waste (62.701.500(6), FAC)

Landfill staff is trained to identify unauthorized wastes and potential sources of hazardous wastes. The guidelines provided in Section 2 of this Operations Plan will be followed for monitoring of waste.

7.0 Waste Handling Requirements (62-701.500(7), FAC)

Waste handing requirements have been addressed in Section 2 of this Operations Plan. The requirements not addressed in aforementioned Section are discussed below.

- ***Time Frames for Applying Final Cover:*** After a Class I landfill disposal unit has been filled to design dimensions, final cover will be installed within 180 days of attaining final elevation.

- ***Salvaging for Recycling:*** The site controls (Section 5) are in place to restrict unauthorized access to the landfill. The gates are locked at the end of each business day. Public access will only be allowed when an attendant is on duty. Uncontrolled and unauthorized scavenging shall not be permitted at the landfill. Controlled salvaging for recycling by landfill personnel may be permitted by the landfill operator.

- ***Description of Litter Policing Methods:*** Litter is picked up and removed from the landfill area daily or as needed. Litter is removed from the perimeter fence on a daily basis. Mobile litter fences are sometimes utilized near the active face to reduce blown litter.

- ***Erosion Control Procedures:*** Erosion is controlled with mulching, seeding, grassing, and terraces. In addition, temporary berms may be used as necessary to contain runoff from the terraces and prevent stormwater flow over side slopes where the intermediate cover soil has not yet stabilized.

- ***Leachate Seep Management:*** Due to the anisotropic nature of waste placed in compacted layers, leachate follows the path of least resistance and may seep through the side slope of the landfill. Leachate seep occurrences are minimized by:

    - Routine monitoring of the slopes as part of the inspection process to check for leachate seeps.
    - If seepage of leachate occurs on the side slope of the landfill, additional cover soil will be applied and recompacted along with the adjacent waste to minimize the path of least resistance.
    - For leachate seeps which reoccur at the same location after placement of cover soil, berms will be constructed to contain the seeps and prevent leachate from

flowing into the stormwater system. Berms may be used to divert stormwater around leachate seeps

- A French drain system (consisting of trenches and vertical excavations filled with porous media such as mulch, gravel, rocks, etc.) may be used to provide a preferential path for the leachate down into the leachate base liner collection system.

8.0 Leachate Management (62-701.500(8), FAC)

The requirements of this Section have been adequately addressed in Section 2.10 of this Operations Plan.

9.0 Gas Monitoring (62.701.500(9), FAC)

The requirements of this Section have been adequately addressed in the previous Sections of this Operations Plan.

10.0 Stormwater System Management (62-701.500(10), FAC)

The requirements of this Section have been adequately addressed in the previous Sections of this Operations Plan.

11.0 Equipment and operation feature requirements (62-701.500(11), FAC)

- ***Equipment:*** The landfill has sufficient equipment to provide flexible landfill operations. **Table 3** lists current heavy equipment for daily landfill operation.

  Shelter and sanitation facilities are provided at the landfill office. First aid kits are provided in the cabs of all heavy equipment vehicles, the scale house and the landfill office.

- ***Reserve Equipment:*** **Table 3** indicates the facility possesses sufficient equipment to operate the landfill. In the event the compactors are out of service less than 24 hours, the dozers will be used to spread refuse over the active face. Bay County has the ability to obtain back-up equipment in the event the equipment is not available within 24 hours from equipment distributors.

Table 3 Landfill Equipment (To be updated by DBE)			
YEAR	**MAKE**	**MODEL**	**YEAR**
ARTICULATED DUMP TRUCK	MOXY	MT30S	1997
ARTICULATED DUMP TRUCK	KOMATSU	350-2 DU TRK	2009
COMPACTOR-LANDFILL	TEREX	TC-400	2009
DOZER, 6-WAY BLADE	CATERPILLAR	DK5XL	2009
DOZER, LGP, 6-WAY BLADE	JOHN DEERE	850J	2007
DOZER, LGP, 6-WAY BLADE	KOMATSU	D61PX-15EO	2012
DOZER, WASTE HANDLER	CATERPILLAR	D6T	2014
EXCAVATOR, MANUAL THUMB	KOMATSU	PC220LC	2005
EXCAVATOR,HYDRAULIC THUMB	JOHN DEERE	350-G LC	2013
FRONT END LOADER	CAT	938K	2013
KNUCKLEBOOM TRUCK	FREIGHTLINER	M2	2013
LEACHATE TANKER	HEIL	N/A	2013
LEACHATE TANKER	HEIL	N/A	2013
LOADER-SKID STEER	BOBCAT	A300	2004
LOWBOY TRAILER	ROLLS RITE	KP25HD	2004
SEMI-TRACTOR	KENWORTH	T800	2014
SEMI-TRACTOR	KENWORTH	T800	2014
TRUCK-DUMP-ROAD-6X6	INTERNATIONAL	5600I	2008

- ***Communication Equipment:*** All landfill equipment operators and spotters are equipped with hand held radios. This radio transmission service links the field personnel to the office and management.

- ***Dust Control:*** Fugitive dust emissions shall be controlled for compliance with state and/or local regulations. The Steelfield Road Landfill incorporates a number of measures to minimize fugitive air emissions. These include:

  - The use of paved access roads to landfill disposal sites. The primary access roads to and from all operational areas are paved.
  - The paved sections of road are cleaned on a semi-annual basis by the use of a conventional street sweeper with a rotary broom attachment.

- The use of site irrigation system for dust control on the unpaved portions of road. All of the unpaved sections of roads at the landfill are constructed out of recycled concrete/graded gravel. During dry periods this material is capable of producing fugitive emissions (dust). To counteract this, the irrigation system that supports vegetative growth on the site is designed so that the sprinkler heads overlap the concrete/gravel roads. This is very effective at minimizing dust. If it is dry enough for the roads to become dusty it is dry enough that the irrigation system is on to supply water to the green areas. It should also be noted that portion of the irrigation system is removed as part of Cells 6 and 7 construction. Tanker trucks will be used for dust control where no irrigation system is available.
- The Steelfield Road Landfill is permitted to re-use the leachate generated from its operation for dust control within the Class I cell. This is accomplished via a pumping system connected to the leachate lift station or by discharging the leachate from tankers.
- The Steelfield Road Landfill is permitted to use latex based paints as daily cover. The latex reduces the loss of airspace that would be taken up by traditional daily cover (dirt) and minimizes the creation of dust and other airborne contaminants. At the present time latex is use infrequently and does not play a major part in the control of fugitive emissions.

- ***Fire Protection:*** Soil is stockpiled adjacent to the active face to be used in fighting fire at the working face. In the event of a hot load, the hauler will be directed to place the load in a containable area for extinguishing the fire. The load will be monitored for several days to ensure the fire is completely out. Upon complete satisfaction the fire is extinguished, the load will be moved to the active face for proper disposal. In the event the landfill personnel cannot extinguish a fire, the Bay County Fire Department is notified for assistance. The heavy equipment is also provided with fire extinguishers.

- ***Litter Control Devices:*** Litter control fences are used near the active face to control litter when practical. Also, chain link fence along Bay Road offers a secondary control of blowing litter.

- ***Signs:*** Signs are used around the site to direct traffic appropriately for waste disposal.

12.0 Roads (62-701.500(12), FAC)

The primary access roads to and from all operational areas are paved. These and other landfill access roads will be properly maintained in good, safe repair during all weather conditions.

13.0 Recordkeeping and Reporting Requirements (62-701.500(13), FAC)

Waste recordkeeping requirements have been addressed in Section 2.5.2 and Section 3 of this Operations Plan. The requirements not addressed in aforementioned Section are discussed below.

- ***Permit Application Development:*** All reports used to develop permit applications and operation records will be maintained for the design period of the permitted landfill. Records such as geotechnical investigations, foundation analyses, demonstration reports, and previous permits and regulations are examples of records to be maintained.

- ***Monitoring Records:*** Records for monitoring required by facility's permit (such as water quality and gas monitoring) will be maintained for the design period as required.

- ***Remaining Site-Life Estimates:*** Maintain an annual estimate of the remaining life and capacity in cubic yards of the existing, constructed landfill and an annual estimate of the life and capacity in cubic yards of other permitted areas not yet constructed. The estimate shall be made and reported annually to the Department.

Attachment A-1
Mining Plan
(Same as Previously Submitted)

Attachment A-2
Contingency Plan
(Same as Previously Submitted)

Attachment A-3
Site Plan
(Updated as Shown in the Drawing Package)

APPENDIX B
SLOPE STABILITY REQUIREMENTS

Slope Stability

Subsurface Investigation 2
Estimation of Groundwater Elevations 2
Potential Fault Areas and Seismic Impact Zones 3
Slope Stability 3
Shear Strength Parameters 3
Results 6
Veneer Stability of Final Cover 7

Attachments

B-1 Selections for Slope Stability Analysis and Boring Logs

B-2 Shear Strength and Interface Friction Angle Test References

B-3 SLIDE Output

B-4 Final Cover Geocomposite Hydraulic Conductivity Determination

B-5 Veneer Stability of Side Slopes

Subsurface Investigation

The geological investigations by Southern Earth Sciences Inc. in 1986 and 2010 are provided with this report. The information collected during these investigations is used to perform Slope Stability for the expansion footprint. Select borings used to characterize subsurface are included with Attachment B-1.

Estimation of Groundwater Elevations

A seasonal High Water Level estimate is based on the site specific water levels from semiannual sampling events, roughly correlated with rainfall data for the National Weather Service – Climate Data – Station: Panama City NW.

Seasonal High Water Table is defined as the shallowest depth to free water (highest water table elevation) that stands in an unlined borehole or where free soil moisture is zero for a significant period (more than a few weeks) – Watts and Hurt, 1991[1].

The Florida Administrative Code defines the SHWT elevation as the highest level of the saturated zone in the soil in a year with normal rainfall.

SHWT estimates for a specific location can be made using:

- Water levels determined from water well or piezometer readings
- Historical rainfall data
- Soil profiles such as color variations and the depth to impermeable/semi-impermeable layers

HDR compared SFRL semiannual groundwater sampling events from July 2010 through August 2018 and monthly rainfall data from 2013 through 2018. The highest and second highest water levels were measured during August 2013 and August 2018 semiannual sampling events respectively. Four upper surficial wells (MW-22, MW-23US, MW-44, and PW-3) were selected as representative of the SHWT around the new cell. The water table contour map shown in drawing package was developed using these wells. National Weather Service rainfall data indicates that from 2013 through 2018 with the exception of 2014 the highest rainfall occurred in the summer months. Therefore, highest water levels would have occurred in the summer and the 2nd semiannual sampling events are representative indicators of the most likely annual high groundwater levels.

The water table contour map was developed using the average second semiannual groundwater elevations collected from MW-22, MW-23US, MW-44, and PW-3 between July 2010 and August 2018. The contoured average groundwater elevations indicate that the SHWT is approximately 58 ft. at the eastern end of the site, 46 ft. at the western end of the site, and less than 42 ft. mid site. Slope stability analysis is based on a groundwater elevation of 44 ft NAVD.

[1] Watts F.C. and G. Wade Hurt. Determining Depths to the Seasonal High Water Table and Hydric Soils in Florida. “Soil Survey Horizons, Vol 32, No. 4, pp. 117-120, Winter 1991.

Potential Fault Areas and Seismic Impact Zones

There are no known potential faults in the vicinity of the proposed footprint. 40 CFR 258.14 of the federal regulations identifies seismic impact zones as areas having a peak bedrock acceleration exceeding 0.1 g based on a 90% probability of non-exceedance over a 250 year time period. Landfill located within seismic impact zones are required to be analyzed to confirm that the landfill can resist the site-specific maximum horizontal acceleration (MHA) in lithified earth material.

The peak ground acceleration (PGA) of the lithified earth material for the site based on 2% probability of exceedance in 50-years as obtained from a U.S. Geological Survey Seismic Hazard Map is 0.02g (Refer to Attachment B-1 in Appendix B). This probability is the same as 90% probability of non-exceedance in 250 years. Accordingly, site-specific seismic stability analysis is not required to confirm that the proposed landfill can withstand the design seismic event.

Slope Stability

Slope stability analyses were performed to evaluate the most critical failure surface and the potential for failure through the proposed bottom liner. Deep rotational failure was modeled using the circular failure mode and the potential for failure along the bottom liner was modeled using the wedgeblock failure mode. As shown on the preliminary drawings, the proposed maximum final cover elevation is approximately 247 feet. The side slopes of the proposed landfill will be maintained at 3H:1V between terraces. Built-in terraces are located at approximately 40 foot vertical intervals in the final cover.

Two critical sections were selected for slope stability analysis as shown in Attachment B-1. The boring logs used to characterize the subsurface are also included with Attachment B-1. The critical sections represented the longest slopes with tallest fill heights.

Shear Strength Parameters

Subsurface Soil Parameters

The soil parameters used in the analysis were based on the results of laboratory tests performed on the soil samples, boring data, empirical correlations and past slope stability reports. We also utilized geotechnical publications such as NAVFAC DM-7 and Allpile manual as an aid in determining the appropriate soil parameters to be used for analysis. For cohesionless soils, due to the permeable nature of these materials, drained strength parameters were assumed under all the loading conditions. For fined grained soils, where excess pore pressures were expected to exist, undrained strength parameters were used for static-end of construction (EOC) conditions. Soils parameters used for the slope stability analyses for both the undrained and drained conditions are summarized in Table 2 and references are provided in Attachment B-2.

Table 2 Summary of Subsurface Soil Parameters

				Effective Stress (Drained)		Total Stress (Undrained)			
Soil Type	Layer	USCS Classification	Overall Ave. SPT N	c[b] (psf)	Ø[c] (deg)	c[b] (psf)	Ø[c] (deg)	Unit Weight (pcf)	Saturated Unit Weight (pcf)
Sand	1	SP, SW, SM	12.9		30	-	-	89	116
Clayey Sand/Clayey Sand with Gravel	2	SC, CL, CH	-	927	19	927	0	92	120
Ash/MSW	3	-	-	0	34	-	-	90	100

Where:

a. Consolidated Undrained Triaxial Test (ASTM D4767)

b. c = cohesion (psf)

c. Ø = friction angle (degrees)

Bottom Liner System and Final Cover System

The shear-strength parameters for the liner system were selected based on the results of interface direct shear tests in the technical literature and information available to HDR. Some of the interface friction angle test results are included with Attachment B-2 for reference. The selected parameters for bottom liner interface are summarized in Table 3.

Cell 3 bottom liner consisted of detection geonet sandwiched between primary and secondary geomembrane liners. Textured geomembrane was used within the sideslopes of Cell 3 and smooth geomembrane was used within the base of Cell 3. Conservatively, an 8 degree friction angle was used for Cell 3 bottom liner for both liner types.

The shear strength parameters for the final cover system were selected recognizing the relatively low normal stress expected to be present in the final cover system (i.e., up to 250 lb/ft^2). Based on the typical interface shear strength information provided in Attachment B-3, a critical interface friction angle (δ) of 23 degrees and adhesion of zero was selected. Veneer type failure analysis was performed separately for the cover soil stability. The selected parameters are summarized in Table 3.

Table 3: Summary of Proposed Landfill Construction Properties

Landfill Component	Cohesion (psf)	Friction Angle	Unit Weight (pcf)
Structural Fill	0	33	120
Proposed Bottom Liner	0	18	120
Cell 3 Geonet/Geomembrane Interface	0	8	120
Critical Interface Shear Strength between Final Cover Layers	0	23	120

Municipal Solid Waste Combined with Incinerator Ash

The proposed landfill area will be filled with municipal solid waste (MSW) combined with incinerator ash. The shear strength properties for waste materials were selected based on a compilation of published information on the shear strength of municipal solid waste (MSW) by Bray, et. al. (2009). The author evaluated MSW shear strength testing from various studies, which included more than 100 large-scale laboratory tests. The author characterized MSW by cohesion (c) = 300 psf and friction angle (Ø) = 36°. In the interest of providing reasonably conservative shear strength parameters for the existing and proposed waste while considering MSW co-disposed with or without incinerator as, HDR assigned MSW a friction angle of 34 degrees with no cohesion. It is expected that MSW co-disposed with incinerator ash is stronger than traditional MSW. The select co-disposal MSW strength parameters are summarized in Table 4.

Table 4: MSW Strength Parameters

	Cohesion (psf)	Friction Angle (deg.)	Unit Weight (pcf)
Effective Stress Analysis	0	34	90

Results

HDR selected two critical sections oriented North-South and East-West to perform the slope stability analysis. The North-South section is located through Cell 3 that has a smooth geomembrane bottom liner as shown in Attachment B-1. The computer program SLIDE 6.0 was used to compute factors of safety for the critical failure surfaces.

The calculations were performed using the Spencer slope stability method for both rotational and translational failure mechanisms. The minimum factors of safety obtained for the worst case analyzed and the targeted FS are tabulated in Table 5. Detailed stability results, including input parameters, profile geometry, and plots are presented in Attachment B-3.

Table 5: Summary of Slope Stability Analysis by SLIDE

Section	Surface Type	Conditions	Calculated FS	Target FS	Comments
East-West	Block- Through Cells 1 & 2 Bottom Liner	ESA-Static	2.06	1.50	OK
	Block - Through Proposed Bottom Liner	ESA-Static	2.01	1.50	OK
	Circular	ESA-Static	2.12	1.50	OK
	Circular	TSA-Static	2.27	1.50	OK
North-South	Block - Through Bottom Liner	ESA-Static	1.91	1.5	OK
	Circular	ESA-Static	2.12	1.5	OK
	Circular	TSA-Static	2.09	1.5	OK

ESA = Effective Stress Analysis
TSA = Total Stress Analysis

Veneer Stability of Final Cover

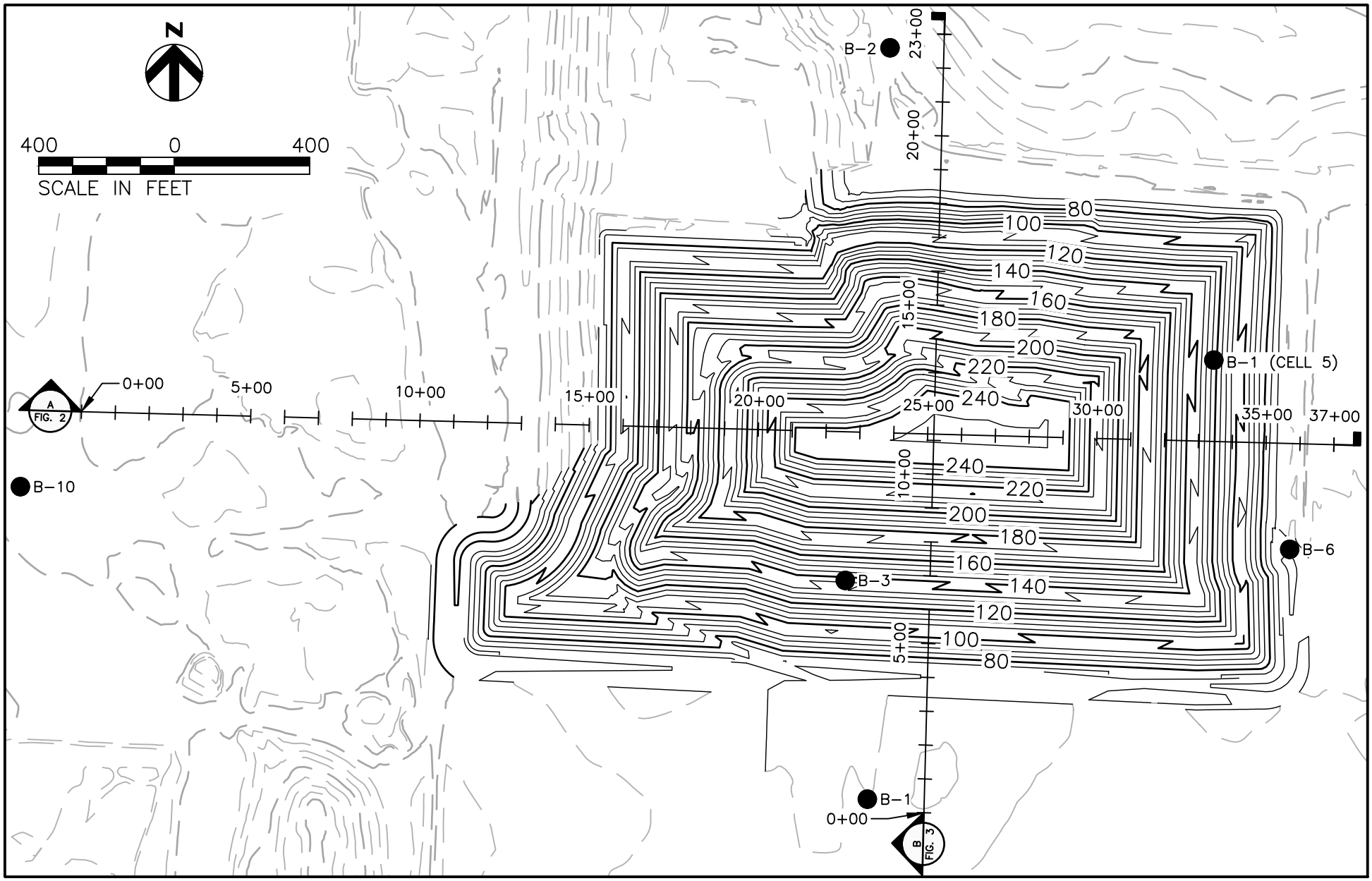
Veneer stability analysis was performed for the proposed final cover. The final cover will consist from top to bottom:

- 6-inch top soil layer
- 18-inches of erosion control layer with $1x10^{-4}$ cm/s (maximum) permeability
- 300 mil geocomposite drainage layer
- 40 mil LLDPE geoemembrane liner (textured on sideslope and smooth on top deck)
- GCL

The density of cover soil is based on typical mixed grain loose sand with dry density of 100 pcf (15.6 kN/m^3), saturated density of 125 pcf (19.64 kN/m^3) and a minimum interface friction angle of 23 degrees The transmissivity of the geocomposite is based on 300 mil GSE geocomposite or similar material. The designed permeability of the geocomposite drainage layer is calculated by applying appropriate reduction factors for chemical clogging, biological clogging and creep (refer to Attachment B-4). The precipitation is obtained from NOAA Atlas as included in Attachment B-4. Accordingly, the 25 year, 5 minute duration storm has a rainfall intensity of 12.1 inches/hour or 307.34 mm/hour. The results of the veneer stability analysis are included in Attachment B-5. The results indicate that the factors of safety for veneer stability is 1.3 based on interface friction angle of 23 degrees.

Attachment B-1

Sections for Slope Stability Analysis and Boring Logs



PROJECT TITLE
STEELFIELD ROAD LANDFILL
PRELIMINARY DESIGN BUILD BID - BAY COUNTY, FLORIDA

SHEET TITLE
SLOPE STABILITY ANALYSIS PLAN

PROJECT NUMBER
10150537

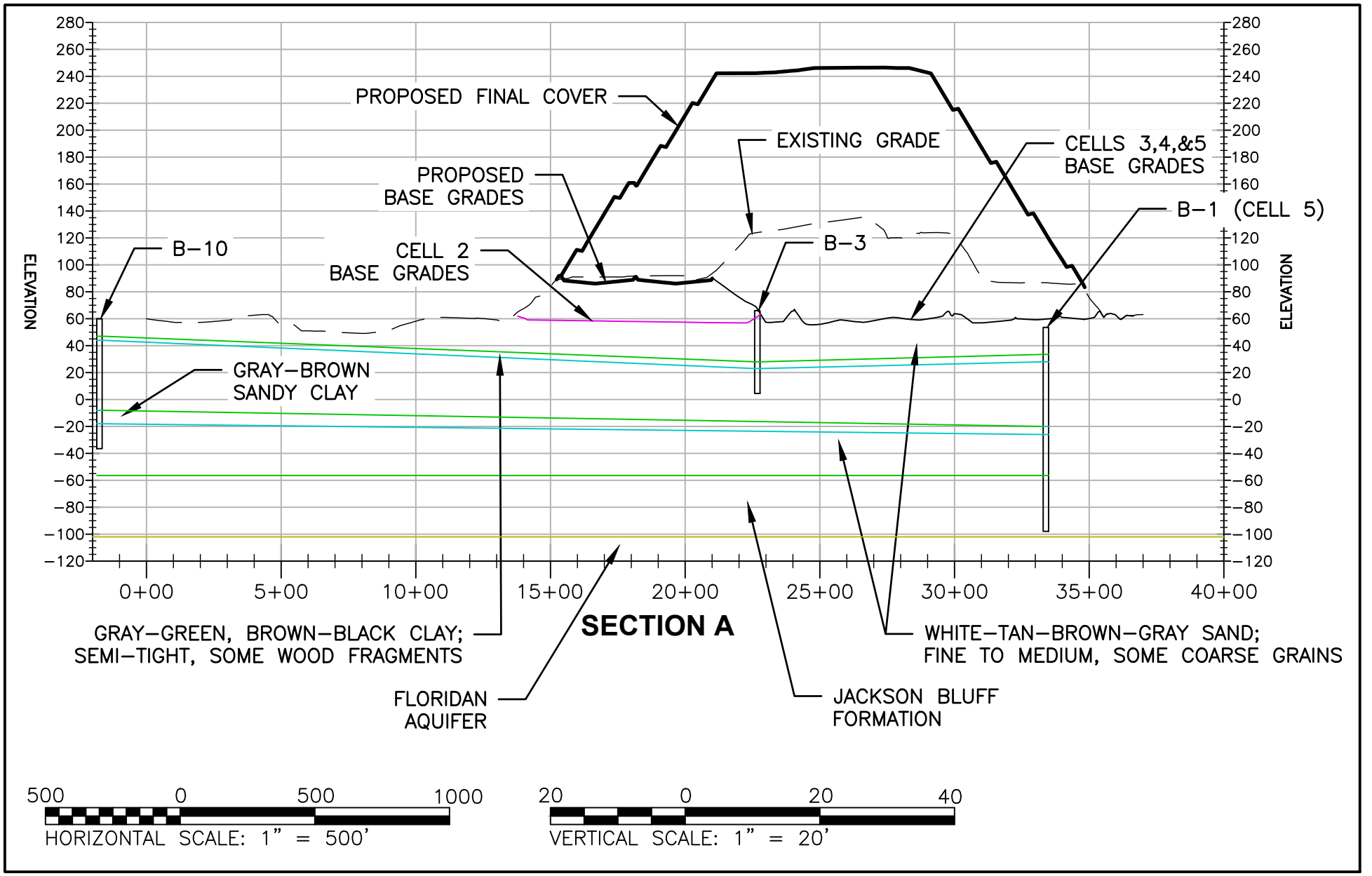
PROJECT MANAGER
M. ROBERTS, P.E.

DATE
FEB. 2019

REFERENCE SHEET

REFERENCE DOCUMENT

EXHIBIT NUMBER
FIGURE 1



PROJECT TITLE
STEELFIELD ROAD LANDFILL
PRELIMINARY DESIGN BUILD BID - BAY COUNTY, FLORIDA

SHEET TITLE
SLOPE STABILITY ANALYSIS
SECTION A - WEST-EAST

PROJECT NUMBER
10150537

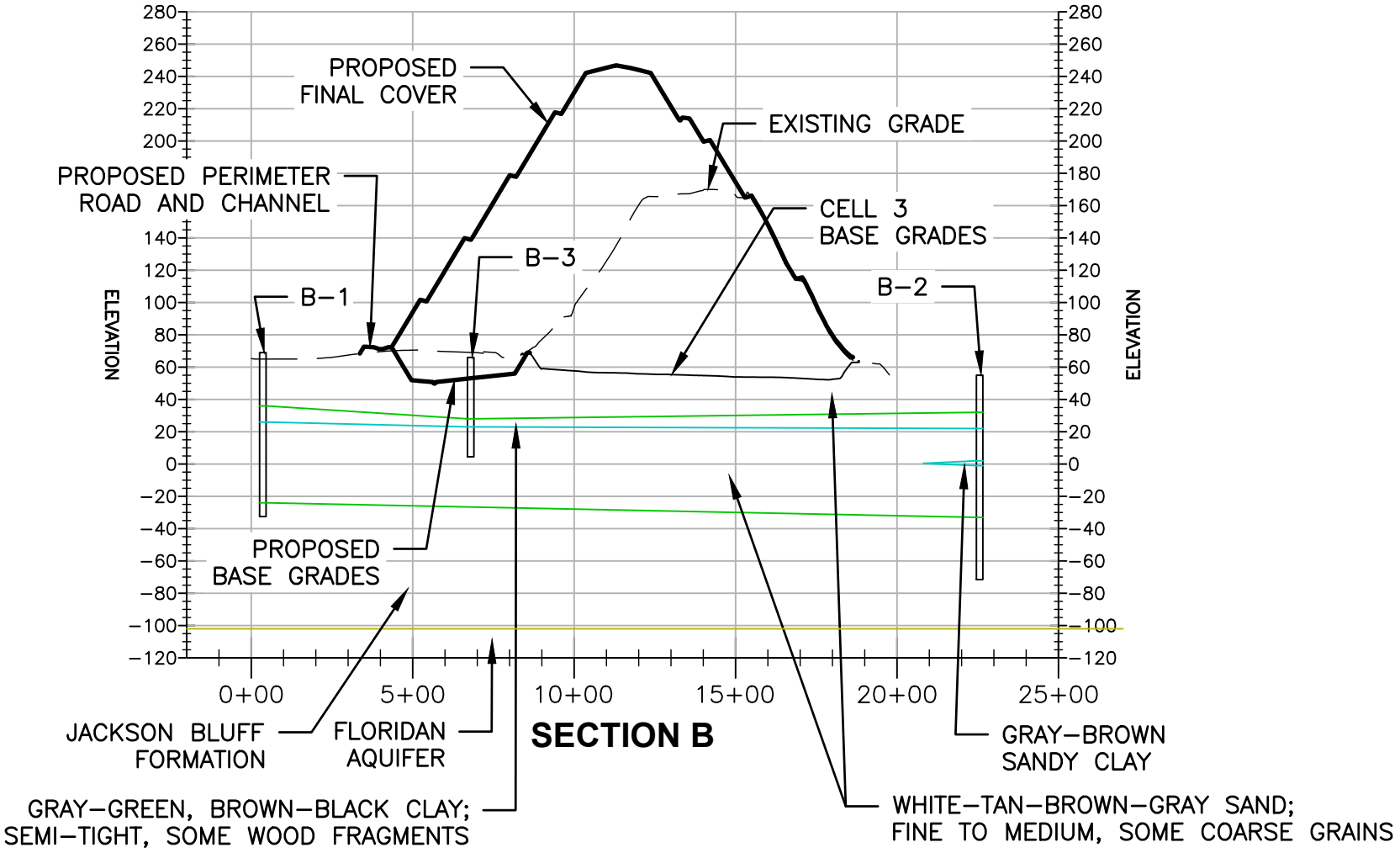
PROJECT MANAGER
M. ROBERTS, P.E.

DATE
FEB. 2019

REFERENCE SHEET

REFERENCE DOCUMENT

EXHIBIT NUMBER
FIGURE 2



PROJECT TITLE
STEELFIELD ROAD LANDFILL
PRELIMINARY DESIGN BUILD BID - BAY COUNTY, FLORIDA

SHEET TITLE
SLOPE STABILITY ANALYSIS
SECTION B - SOUTH-NORTH

PROJECT NUMBER
10150537

PROJECT MANAGER
M. ROBERTS, P.E.

DATE
FEB. 2019

REFERENCE SHEET

REFERENCE DOCUMENT

EXHIBIT NUMBER
FIGURE 3

LOG OF BORING O. B-1

PROPOSED LANDFILL SITE
OFF STEELFIELD ROAD
BAY COUNTY, FLORIDA

TYPE: ASTM D-1586 LOCATION: Per Plan

SURF. EL: Unknown

DEPTH, FT	SYMBOL	SAMPLES	DESCRIPTION OF MATERIAL	BLOWS PER FT	UNIT DRY WT LB/CU FT
			**	6	
5			Tan Fine SAND	5	
10				9	
15			Dark Brown Slightly Silty Fine SAND	16	
20			Light Brown Slightly Silty Fine SAND	3	
25			White Fine SAND	23	
30			Brown Fine SAND	23	
35			White Silty to Slightly Clayey Fine SAND with Tan Clay Lenses	3	
40				8	
45			Brown Fine SAND	67	
50				77	
			** Topsoil		

BLOWS PER FOOT (N): 5 10 15 20 25 30 35

PLASTIC LIMIT — WATER CONTENT, % — LIQUID LIMIT: 10 20 30 40 50 60 70

ELEVATION, FT

COMPLETION DEPTH: 101.5'
DATE: 4/10/86

DEPTH TO WATER IN BORING: 13'3" DATE: 4/16/86

SOUTHERN EARTH SCIENCES, INC.

File No.: F-86-042

LOG OF BORING O. B-1

PROPOSED LANDFILL SITE
OFF STEELFIELD RAOD
BAY COUNTY, FLORIDA

TYPE: ASTM D-1586 LOCATION: Per Plan

SURF. EL: Unknown

DEPTH, FT	SYMBOL	SAMPLES	DESCRIPTION OF MATERIAL	BLOWS PER FT	UNIT DRY WT LB/CU FT
			Brown Fine SAND		
55			Tan Medium to Fine SAND	88	
60			Tan Fine SAND	50	
65			Tan Medium to Fine SAND		
70					
75			White Fine SAND		
80				50/6"	
85			Tan Medium to Fine SAND	50/5"	
90			Tan Fine SAND	50/5"	
95			Dark Gray CLAY	WOR	
100			Gray Slightly Clayey Fine SAND with shell fragments	32	
			WOR=weight of rods		

BLOWS PER FOOT (N)
5 10 15 20 25 30 35

PLASTIC LIMIT — WATER CONTENT, % — LIQUID LIMIT
10 20 30 40 50 60 70

ELEVATION, FT

COMPLETION DEPTH: 101.5'
DATE: 4/10/86

DEPTH TO WATER IN BORING: 13'3"
DATE: 4/16/86

SOUTHERN EARTH SCIENCES, INC.

File No.: F-86-042

LOG OF BORING J. B-2

PROPOSED LANDFILL SITE
OFF STEELFIELD ROAD
BAY COUNTY, FLORIDA

TYPE: ASTM D-1586 LOCATION: Per Plan

SURF. EL: Unknown

DEPTH, FT	SYMBOL	SAMPLES	DESCRIPTION OF MATERIAL	BLOWS PER FT	UNIT DRY WT LB/CU FT
			**	4	
5				7	
10			Tan Fine SAND	14	
15				26	
20			White Fine SAND	41	
25			Light Gray Slightly Silty Fine SAND with trace Clay	19	
30			White Silty Fine SAND with trace Clay	5	
35			White Fine SAND	45	
40				57	
45			Tan Medium to Fine SAND	56	
50				57	
			** topsoil		

BLOWS PER FOOT (N)
5 10 15 20 25 30 35

PLASTIC LIMIT — WATER CONTENT, % — LIQUID LIMIT
10 20 30 40 50 60 70

ELEVATION, FT

COMPLETION DEPTH: 126.5'
DATE: 4/15/86

DEPTH TO WATER IN BORING: 18.5' DATE: 4/16/86

SOUTHERN EARTH SCIENCES, INC.

File No.: F-86-042

LOG OF BORING NO. B-2

PROPOSED LANDFILL SITE
OFF STEELFIELD RAOD
BAY COUNTY, FLORIDA

TYPE: ASTM D-1586 LOCATION: Per Plan

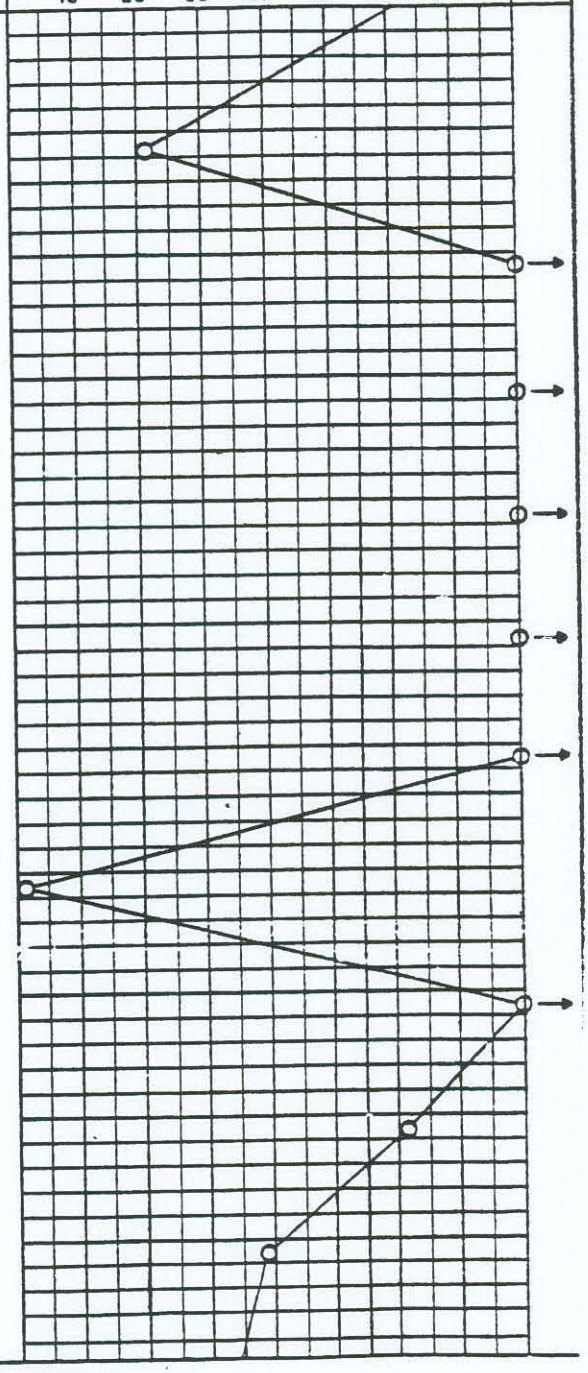
SURF. EL: Unknown

DEPTH, FT	SYMBOL	SAMPLES	DESCRIPTION OF MATERIAL	BLOWS PER FT	UNIT DRY WT LB/CU FT
			Tan Medium to Fine SAND		
55			Light Gray Slightly Sandy Silty CLAY	11	
			White Silty Fine SAND		
60				50/6"	
65				95/ 11"	
70			Tan Medium to Fine SAND	94/ 11"	
75				50/6"	
80				91/11"	
85			Dark Gray Slightly Silty Fine SAND	1	
90			Gray Slightly Clayey Shelly SAND	44	
95				31	
100			Gray Slightly Clayey Silty SAND with some shell fragments	19	

BLOWS PER FOOT (N)
5 10 15 20 25 30 35

PLASTIC LIMIT — WATER CONTENT, % — LIQUID LIMIT
10 20 30 40 50 60 70

ELEVATION, FT



COMPLETION DEPTH: 126.5' DEPTH TO WATER IN BORING: 18.5' DATE: 4/16/86
DATE: 4/15/86

SOUTHERN EARTH SCIENCES, INC.

File No.: F-86-042

LOG OF BORING NO. B-2

PROPOSED LANDFILL SITE
OFF STEELFIELD ROAD
BAY COUNTY, FLORIDA

TYPE: ASTM D-1586 LOCATION: Per Plan

SURF. EL: Unknown

DEPTH, FT	SYMBOL	SAMPLES	DESCRIPTION OF MATERIAL	BLOWS PER FT	UNIT DRY WT LB/CU FT
105			Gray Slightly Clayey Silty SAND with some shell fragments	15	
110				18	
115			Gray Green Slightly Clayey Silty SAND with some shell fragments	20	
120				26	
125				29	

BLOWS PER FOOT (N)
5 10 15 20 25 30 35

PLASTIC LIMIT — WATER CONTENT, % — LIQUID LIMIT
10 20 30 40 50 60 70

ELEVATION, FT

COMPLETION DEPTH: 126.5'
DATE: 4/15/86

DEPTH TO WATER IN BORING: 18.5'
DATE: 4/16/86

LOG OF BORING NO. B-3

PROPOSED LANDFILL SITE
OFF STEELFIELD ROAD
BAY COUNTY, FLORIDA

TYPE: ASTM D-1586 LOCATION: Per Plan

DEPTH, FT	SYMBOL	SAMPLES	DESCRIPTION OF MATERIAL	BLOWS PER FT	UNIT DRY WT LB/CU FT
			SURF. EL:		
5			White, Tan, to Orange Fine SAND	7	
10				10	
15			Brown Slightly Silty Fine SAND	14	
20				15	
25			White, Tan, to Orange Fine SAND	18	
30				33	
35				27	
40			Light Gray Clayey SAND with thin clay lenses	7	
45			Tan Medium to Fine SAND with trace clay	50	
50				78	

BLOWS PER FOOT (N)
5 10 15 20 25 30 35

PLASTIC LIMIT — WATER CONTENT, % — LIQUID LIMIT
10 20 30 40 50 60 70

ELEVATION, FT

COMPLETION DEPTH: 71.5'
DATE: 5/28/86

DEPTH TO WATER IN BORING: 15'±
DATE: 5/28/86

SOUTHERN EARTH SCIENCES INC.

File No.: F-86-042

LOG OF BORING NO. B-3

PROPOSED LANDFILL SITE
OFF STEELFIELD ROAD
BAY COUNTY, FLORIDA

TYPE: ASTM D-1586 LOCATION: Per Plan

DEPTH, FT	SYMBOL	SAMPLES	DESCRIPTION OF MATERIAL	BLOWS PER FT	UNIT DRY WT LB/CU FT	BLOWS PER FOOT (N) 5 10 15 20 25 30 35 / PLASTIC LIMIT — WATER CONTENT, % — LIQUID LIMIT 10 20 30 40 50 60 70	ELEVATION, FT
			SURF. EL:				
55			Tan Medium to Fine SAND with trace clay	68			
60			Tan Medium to Fine SAND	50/3"			
65							
70							
75							
80							

COMPLETION DEPTH: 71.5' DEPTH TO WATER IN BORING: 15'± DATE: 5/28/86
DATE: 5/28/86

LOG OF BORING NO. B-6

PROPOSED LANDFILL SITE
OFF STEELFIELD ROAD
BAY COUNTY, FLORIDA

TYPE: ASTM D-1586 LOCATION: Per Plan

DEPTH, FT	SYMBOL	SAMPLES	DESCRIPTION OF MATERIAL (SURF. EL:)	BLOWS PER FT	UNIT DRY WT LB/CU FT
5			Tan Fine SAND	8	
10				17	
15			Tan Slightly Silty Fine SAND	3	
20			Brown Medium to Fine SAND	25	
25			Tan Fine SAND	18	
30			Brown Slightly Silty Fine SAND	20	
35			Gray Fine SAND with clay lenses	50	
40			Gray Fine SAND	45	
45			Tan Medium to Fine SAND with clay lenses	87/9"	
50			Tan Medium to Fine SAND with trace clay	62	

BLOWS PER FOOT (N)
5 10 15 20 25 30 35

PLASTIC LIMIT — WATER CONTENT, % — LIQUID LIMIT
10 20 30 40 50 60 70

ELEVATION, FT

COMPLETION DEPTH: 101.5'
DATE: 6/2/86

DEPTH TO WATER IN BORING: 15'±
DATE: 6/2/86

LOG OF BORING NO. B-6

PROPOSED LANDFILL SITE
OFF STEELFIELD ROAD
BAY COUNTY, FLORIDA

TYPE: ASTM D-1586 LOCATION: Per Plan

DEPTH, FT	SYMBOL	SAMPLES	DESCRIPTION OF MATERIAL	BLOWS PER FT	UNIT DRY WT LB/CU FT	BLOWS PER FOOT (N) 5 10 15 20 25 30 35 / PLASTIC LIMIT — WATER CONTENT, % — LIQUID LIMIT 10 20 30 40 50 60 70	ELEVATION, FT
			SURF. EL:				
55			Tan Medium to Fine SAND	50/6"			
60			Tan Coarse to Fine SAND with trace clay & wood	54			
65			Tan Fine SAND	56/9"			
70			Tan Medium to Fine SAND	44			
75			Tan Medium to Fine SAND with trace clay	73			
80				50/5"			
85				50/4"			
90			Tan Medium to Fine SAND	50/3"			
95			Dark Gray Silty Slightly Clayey SAND with shell fragments	31			
100				27			

COMPLETION DEPTH: 101.5' DEPTH TO WATER IN BORING: 15'± DATE: 6/2/86
DATE: 6/2/86

LOG OF BORING NO. B-10

PROPOSED LANDFILL SITE
OFF STEELFIELD ROAD
BAY COUNTY, FLORIDA

TYPE: ASTM D-1586 LOCATION: Per Plan

DEPTH, FT	SYMBOL	SAMPLES	DESCRIPTION OF MATERIAL	BLOWS PER FT	UNIT DRY WT LB/CU FT
			SURF. EL:		
5			Tan Fine SAND	4	
				9	
10				15	
15			Tan Slightly Clayey Fine SAND	16	
20			Dark Brown Silty Fine SAND	12	
25			White to tan Fine SAND	18	
30				29	
35			Brown Slightly Silty Medium to Fine SAND	24	
40			Tan Medium to Fine SAND	38	
45				50/6"	
50			Tan Fine SAND	50/5"	

BLOWS PER FOOT (N)
5 10 15 20 25 30 35

PLASTIC LIMIT — WATER CONTENT, % — LIQUID LIMIT
10 20 30 40 50 60 70

ELEVATION, FT

COMPLETION DEPTH: 96.5'
DATE: 7/24/86

DEPTH TO WATER IN BORING: 14.8'
DATE: 8/4/86

LOG OF BORING NO. B-10

PROPOSED LANDFILL SITE
OFF STEELFIELD ROAD
BAY COUNTY, FLORIDA

TYPE: ASTM D-1586 LOCATION: Per Plan

DEPTH, FT	SYMBOL	SAMPLES	DESCRIPTION OF MATERIAL (SURF. EL:)	BLOWS PER FT	UNIT DRY WT LB/CU FT
55			Tan Medium to Fine SAND	89	
60			→ trace clay	46	
65			→ some coarse sand	67	
70			Gray Organic CLAY with wood	15	
75			White to Tan Fine SAND with trace clay	50/6"	
80			Tan Medium to Fine SAND with trace coarse sand	50/6"	
85			→ trace gravel	50/5"	
90				50/5"	
95				50/4"	
100					

BLOWS PER FOOT (N)
5 10 15 20 25 30 35

PLASTIC LIMIT — WATER CONTENT, % — LIQUID LIMIT
10 20 30 40 50 60 70

ELEVATION, FT

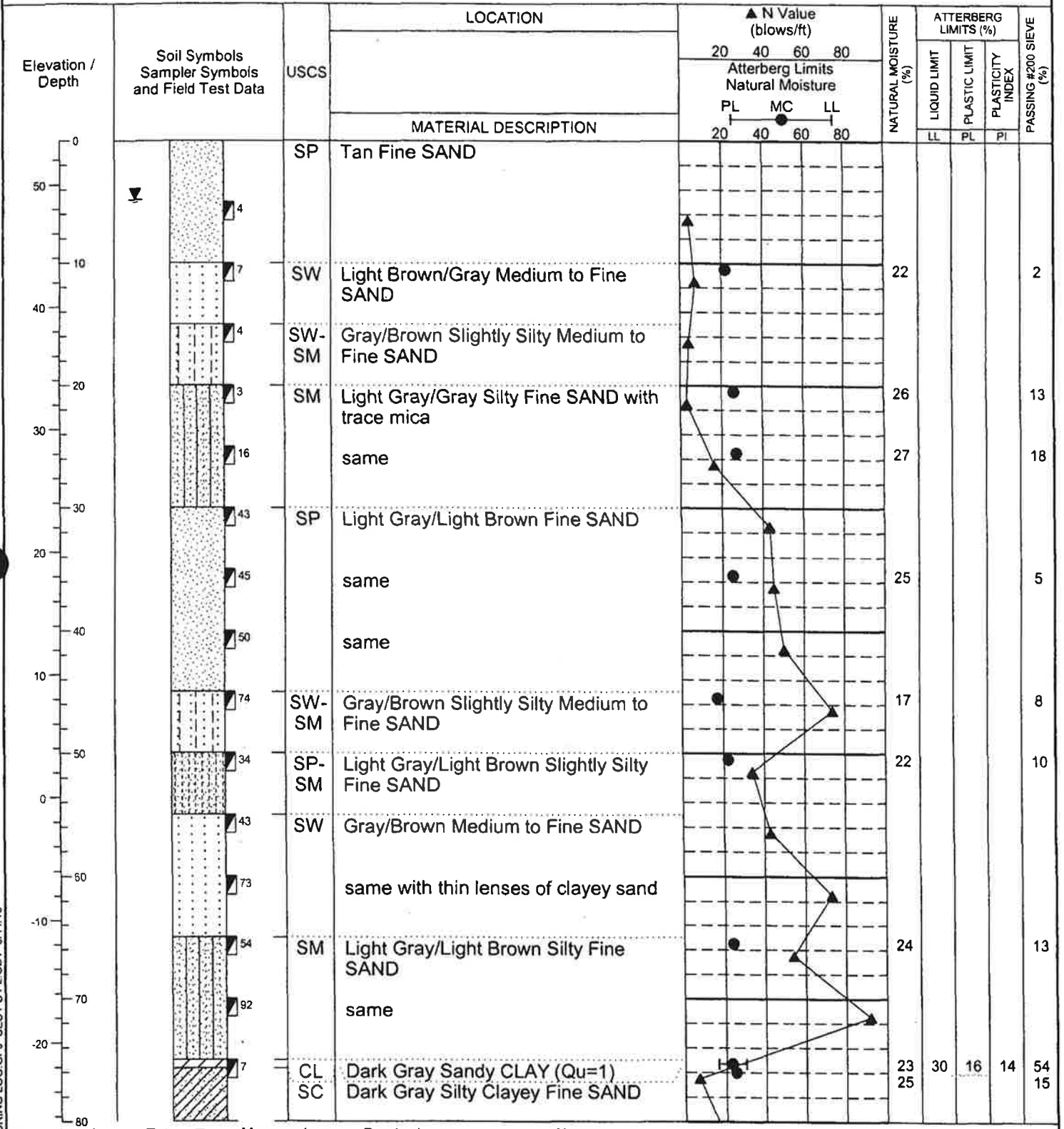
COMPLETION DEPTH: 96.5'
DATE: 7/24/86

DEPTH TO WATER IN BORING: 14.8'
DATE: 8/4/86

LOG OF BORING B-1

PROJECT: Cell #5 Expansion
LOCATION: Steelfield Landfill, Bay County, FL
PROJECT NO.: P10-0285
DATE: 07/06/10

METHOD: Rotary Wash
DRILLER: Jamison Short
ENGR / GEOL: M. Varner
SURFACE ELEVATION: 53.6 ft



Water Level Est.: ▽ Measured: ▼ Perched: ▽
Water Observations: Depth 5 Feet

Notes:
Automatic Hammer
Qu=Unconfined Compressive Strength (tsf)

N - SPT Data (Blows/Ft) P - Pocket Penetrometer (tsf)
Sample Key: SPT Shelby Tube

SOUTHERN EARTH SCIENCES, inc.

LOG OF BORING LOG.GPJ SES PC FL.GDT 8/11/10

LOG OF BORING B-1

PROJECT: Cell #5 Expansion
LOCATION: Steelfield Landfill, Bay County, FL
PROJECT NO.: P10-0285
DATE: 07/06/10

METHOD: Rotary Wash
DRILLER: Jamison Short
ENGR / GEOL: M. Varner
SURFACE ELEVATION: 53.6 ft

Elevation / Depth	Soil Symbols Sampler Symbols and Field Test Data	USCS	LOCATION / MATERIAL DESCRIPTION	▲ N Value (blows/ft) 20 40 60 80 / Atterberg Limits Natural Moisture PL MC LL 20 40 60 80	NATURAL MOISTURE (%)	ATTERBERG LIMITS (%) LIQUID LIMIT LL	PLASTIC LIMIT PL	PLASTICITY INDEX PI	PASSING #200 SIEVE (%)
80	20	SM	Gray/Dark Gray Calcareous Silty Fine SAND with shell		23				23
-30	30		same less clay		25				13
90	16	SM	Gray Calcareous Silty Fine SAND with shell (Partially Cemented)						
-40	9		same		33				35
100	7		same (Partially Cemented)						
-50	6		same						
110	15	SC	Gray Calcareous Clayey Fine SAND		36				39
-60	12		same (Partially Cemented)						
120	8		same		33				44
-70	10		same						
130	8		same		39				49
-80	11	SC	Gray Calcareous Clayey Fine SAND with little shell						
140	11	CL	Gray Calcareous Sandy CLAY with little shell (Qu=2.5)		41 / 38	51 / 42	30	12	71
-90	10		same						
150	7		same (Qu=2)		40				62
-100									
160									

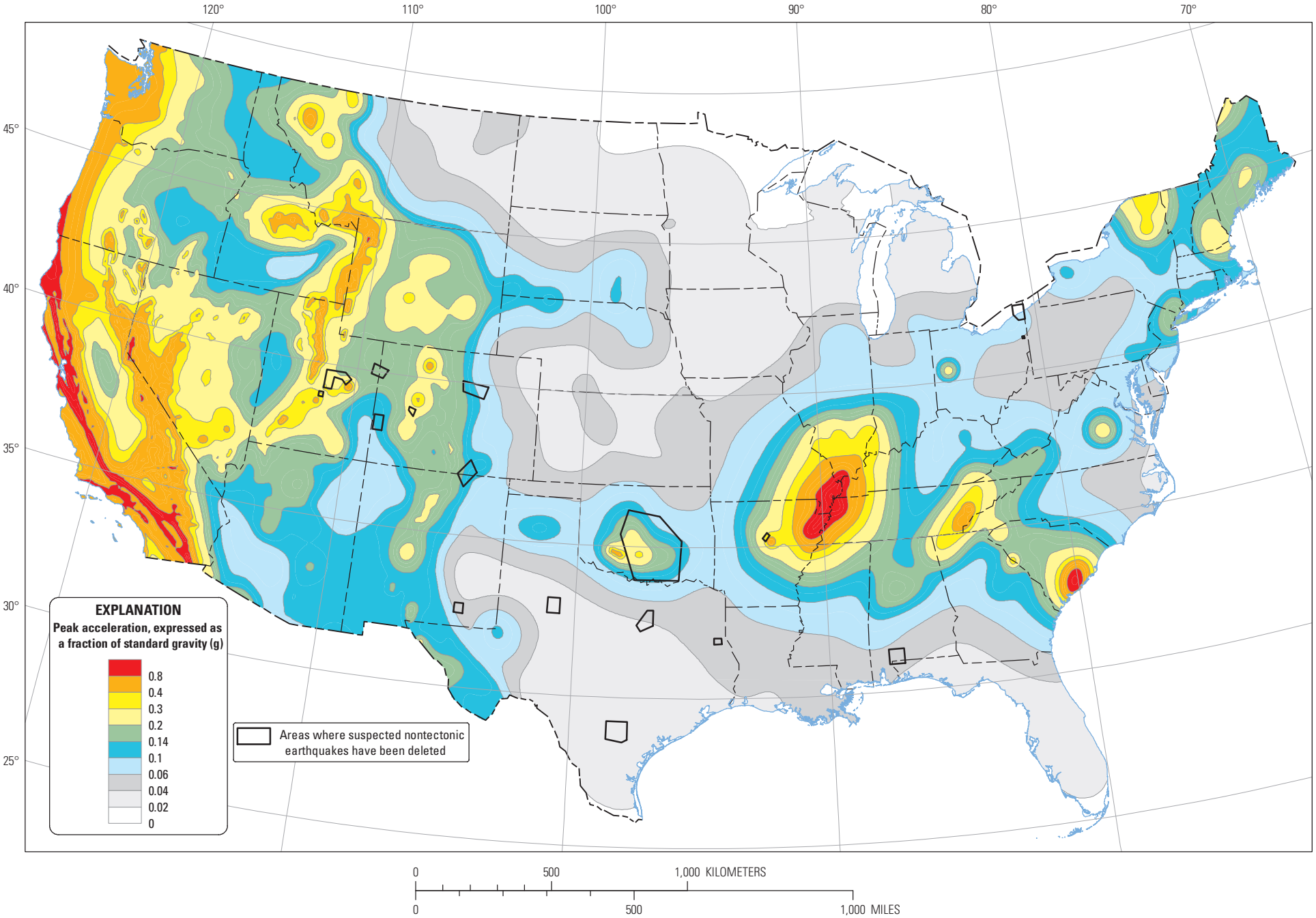
Water Level Est.: ▽ Measured: ▼ Perched: ▽
Water Observations: Depth 5 Feet

Notes:
Automatic Hammer
Qu=Unconfined Compressive Strength (tsf)

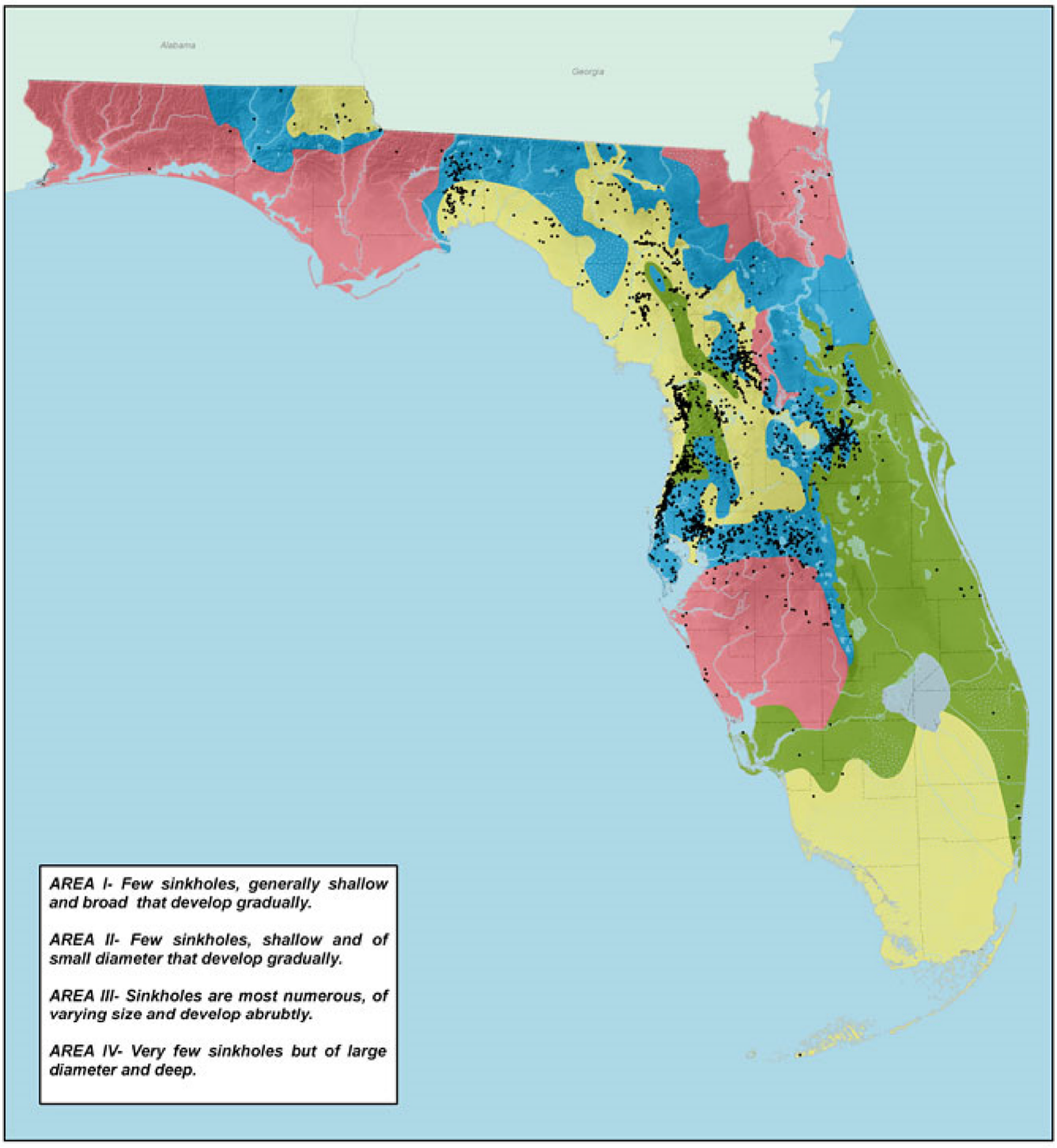
N - SPT Data (Blows/Ft) P - Pocket Penetrometer (tsf)
Sample Key: SPT Shelby Tube

SOUTHERN EARTH SCIENCES, inc.

LOG OF BORING LOG.GPJ SES PC FL.GDT 8/11/10



Two-percent probability of exceedance in 50 years map of peak ground acceleration



Legend

- AREA I
- AREA II
- AREA III
- AREA IV
- Sinkhole

AREAS OF SINKHOLE OCCURANCE FLORIDA

Produced in 2008 by the Florida Center for Instructional Technology (FCIT) using data from the Florida Department of Environmental Protection and the Florida Geological Survey.

N
W E
S

0 25 50 75 Miles

Attachment B-2

Shear Strength and Interface Friction Angle References

8.6 SOIL PARAMETERS AND CORRELATIONS

There are a number of references in the industry that present the correlations between soil parameters. The soil parameters function is useful if users only have a few parameters available and want to estimate the others to complete the calculation. However, one should bear in mind that these correlations are from various sources, references, and statistical results of different soil types under different conditions. The actual value may be different from the estimate given by the correlation. Users should make their own judgment based on local experience and local soil conditions and adjust the values accordingly.

Following are the references used to form the soil correlation in the program:

Table 8-6. General Soil Parameters for Sand

Compactness			Very Loose	Loose	Medium	Dense	Very Dense
	Symbol	Unit					
SPT*	N_{SPT}	--	0-4	4-10	10-30	30-50	>50
Relative Density	Dr	%	0-15	15-35	35-65	65-85	85-100
Friction	ϕ	Deg	<28	28-30	30-36	36-41	>42
Unit Weight							
Moist	γ	pcf	<100	95-125	110-130	110-140	>130
Submerged	γ	pcf	<60	55-65	60-70	65-85	>75

*SPT -- Standard Penetration Test
Reference: Steel Sheet Piling Design Manual, USS, 1975, p.12

Table 8-7. General Soil Parameters for Clay

Consistency			Very Soft	Soft	Medium	Stiff	Very Stiff	Hard
	Symbol	Unit						
SPT	N_{SPT}	--	0-2	2-4	4-8	8-16	16-32	>32
UCS*	q_u	pcf	0-500	500-1000	1000-2000	2000-4000	4000-8000	>8000
Shear Strength	C_u	psf	0-250	250-500	500-1000	1000-2000	2000-4000	>4000
Unit Weight								
Saturated	γ	pcf	<100	100-120	100-130	120-130	120-140	>130

*UCS -- Unconfined Compressive Strength
Reference: Steel Sheet Piling Design Manual, USS, 1975, p.12

and this paper should focus future discussion.

Geosynthetic clay liners (GCLs) are assembled linear structures of geosynthetic materials and bentonite, and are used as primary and/or secondary liners in landfill applications. Gartung & Zanzinger present a comprehensive overview of the engineering properties and use of GCLs. The function and properties of GCLs, together with the importance of quality control are highlighted in the paper; which includes two case histories from Germany on performance observations from GCLs used in capping applications. At the Hamburg-Georgswerder site, problems of root damage, desiccation and cation exchange arose due to lack of sufficient soil cover. However, the second case history, at Nuremberg, reports a successful use of a GCL on a landfill capping system. The authors conclude with advice on the use of GCLs in landfill covers given by the German Institute of Construction; if this guidance is followed in the UK, desiccation can be avoided.

The stability of geosynthetic landfill lining systems

D.R.V. Jones [1] and N. Dixon[2]
1) Golder Associates, Landmere Lane, Nottingham NG12 4DG
2) The Nottingham Trent University, Nottingham NG1 4BU

Introduction

Geosynthetic materials are now commonly used in landfills for many applications such as:

- Geomembranes used as primary liners as barriers to leachate and landfill gas escape.
- Geotextiles used as separation layers, filter layers and as geomembrane protectors.
- Geosynthetic Clay Liners (GCLs) used as primary or secondary liners.
- Geonets and geocomposites used as leachate, landfill gas and groundwater drainage layers.
- Geogrids used for reinforcing applications.

The stability of a geosynthetic landfill lining system is controlled by the shear strength between the various interfaces, i.e. geosynthetic/geosynthetic and geosynthetic/soil interface shear strengths. This paper considers the stability of geosynthetics on landfill side slopes and in sloping capping applications by presenting a summary of available interface shear strength values from the literature, supplemented by testing carried out at The Nottingham Trent University. Design methods promoted by various authors are discussed and modifications suggested.

Background

The shear strength developed at a geosynthetic interface is dependent on both the normal stress applied to the interface and the displacement at the interface. Several authors (e.g. Seed *et al.*, 1988, Byrne 1994 etc.) have indicated that most geosynthetic interfaces are strain softening, i.e. they exhibit a reduction in shear

Geotechnical engineering of landfills. Thomas Telford, London, 1998

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nt By: S.L.R; 01773766021; 21-Apr-04 15:32; Page 2

EPA Export 25-07-2013:13:49:36

stress at displacements beyond peak strengths. Typically for each normal stress, the shear stress increases from the origin with increasing displacement until a peak value is achieved. Subsequent displacement results in a reduction in shear stress to a constant or residual value.

If the peak and residual strengths are plotted against the relevant normal stresses, the resulting failure envelope can be defined. A linear Coulomb-type failure envelope is usually obtained which defines the interface shear strength in terms of the friction angle (δ) and cohesion intercept (α). It should be noted that these parameters only define the failure envelope for the range of normal stresses tested and that extrapolation of both friction angle and cohesion intercept outside the range may not be representative. These interface shear strength parameters can be used to assess the stability of any slope containing a geosynthetic, using a conventional soil mechanics approach.

Measurement of interface shear strength

The measurement of geosynthetic interface shear strength can be carried out by three main methods; direct shear testing, ring shear testing and testing with a tilting table. Direct shear testing can be carried out in standard soil shear boxes with dimensions of 60 mm x 60 mm and 100 mm x 100 mm which can be regarded as index testing, or can be more performance-related using larger 300 mm x 300 mm and 300 mm x 400 mm direct shear apparatus. All direct shear apparatus have limited displacements and it has been shown (Jones, 1998) that even displacements of 100 mm may not mobilise the true residual interface shear strengths.

Ring shear testing can be carried out to investigate the true residual strengths since the apparatus can produce unlimited displacements. It should be recognised, however, that the direction of shearing in a ring shear test is not comparable to the field and thus true residual shear strengths may only be of academic interest and the large strain strengths obtained from a direct shear test in a 300 mm x 400 mm apparatus may be sufficient for design applications. In addition, ring shear testing should not be used to measure peak interface shear strengths (Dixon & Jones, 1995).

The third main method of measurement is the use of a tilting table which has been used predominantly in Europe. There is currently no consensus on the size of apparatus required to provide performance results and its use is limited to low normal stresses. It may be, however, that the tilting table may be more accurate in determining the behaviour of geosynthetic interfaces at low confining stress.

Interface shear strength values

The following paragraphs summarise a literature search carried out to investigate the range of shear strengths published for various geosynthetic interfaces. The results of the literature search have been supplemented by over 200 direct shear

tests carried out at The Nottingham Trent University (Jones, 1998). Peak and residual shear strengths have been plotted against the appropriate normal stress (Figures 1, 2 and 3) and linear regression has been used to generate the failure envelope for each interface. The peak and residual shear strength envelopes are given, together with the correlation coefficient (R^2) which gives a statistical determination of whether the assumed linear regression is strong; a perfect straight line fit giving an R^2 value of 1.0.

Smooth HDPE geomembrane

The results of testing on smooth HDPE geomembranes are presented in Figure 1 and a summary is given in Table 1 below.

Interface	Interface shear strength parameters					
	Peak			Residual		
	δ (°)	α (kPa)	R^2	δ (°)	α (kPa)	R^2
Geonet	9.0	1.0	0.74	6.9	1.8	0.80
Non-woven geotextile	9.8	-0.8	0.88	5.8	0.3	0.88
Sand	26.9	-4.0	0.90	16.2	0.0	0.95
Clay - undrained	10.3	7.1	0.48	2.3	15.0	0.09
Clay - drained	21.5	2.1	0.86	17.1	-6.1	0.97

Table 1 Summary of results for smooth HDPE geomembrane

The summary plot of shear stress vs. normal stress for a smooth geomembrane/geonet interface (Figure 1a) shows a scatter in data points with a poor straight line fit for both peak and residual conditions with R^2 values of 0.74 and 0.80 respectively. This linear regression gives a peak friction angle of 9.0°, which reduces to 6.9° at large displacements. This interface has low cohesion intercepts for both peak (1.0kPa) and residual (1.8kPa) conditions. For the smooth geomembrane/non-woven geotextile interface, a peak interface friction angle of 9.8°, reducing to 5.8° for residual conditions (Figure 1b) is calculated; there is negligible cohesion intercept for this interface. Both peak and residual conditions give strong straight line fits both with correlation coefficient values of 0.88, however there is still a degree of scatter in the results (Figure 1b).

The smooth geomembrane/sand interface has much higher shear strength than the two interfaces discussed above. The peak interface shear strength using linear regression is δ = 26.9° and α = -4.0 kPa, and there is a good straight line fit with R^2 = 0.90 (Figure 1c). The residual values give slightly less scatter and thus a higher correlation coefficient of 0.95, and a residual friction angle of 16.2°.

Testing of the interface shear strength between geosynthetics and cohesive soil is more difficult than the testing of geosynthetic/geosynthetic or geosynthetic/granular interfaces since there is the possibility of pore water pressures at the interface during shearing. Such

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negative (suctions) and will lead to a decrease or increase in effective stress at the interface thus making the assessment of interface shear strength more difficult. The assessment of whether the results quoted in the literature are based on undrained or drained conditions is based on either the various authors' descriptions or on an interpretation of the shearing rates used by the current authors. It is considered that the results presented may not be true undrained or drained conditions and thus caution is required when assessing the results.

For undrained tests it may be that the interface shear strength will be dependent on the undrained shear strength of the clay. However, not all authors reported the clay strength and this makes any accurate assessment of the results difficult if not impossible. The scatter in results for smooth HDPE geomembrane/clay interface (Figure 1d) is not unexpected. Correlation coefficients of 0.48 and 0.09 for the peak and residual envelopes respectively demonstrate this scatter. There is a clear increase in shear strength with increasing normal stress with a peak interface shear strength parameters of δ = 10.3° and α = 7.1 kPa. However, the friction angle of the residual envelope is negligible (δ = 2.3°) and the cohesion intercept is 15.0 kPa.

For the drained case the smooth geomembrane/clay interface has less scatter than the undrained conditions (Figure 1e). This may be associated with no pore pressures at the interface or may be due to the lower number of data points available. Both peak and residual envelopes have strong correlation coefficients of 0.86 and 0.97 respectively, and the peak interface friction angle of 21.5° reduces to a residual value of 17.1°. The cohesion intercept reduces from 2.1 kPa for the peak to -6.1 kPa for the residual shear strength. Since the residual envelope is only based on four data points it is not considered to be representative.

Textured HDPE geomembrane

The results of testing on textured HDPE geomembranes are presented in Figure 2 and a summary is given in Table 2 below.

Interface	Interface shear strength parameters					
	Peak			Residual		
	δ (°)	α (kPa)	R^2	δ (°)	α (kPa)	R^2
Geonet	11.0	3.0	0.98	9.1	9.2	0.96
Non-woven geotextile	25.8	6.9	0.88	13.1	3.6	0.88
Sand	27.4	6.9	0.96	25.5	15.5	0.90
Clay - undrained	4.4	36.0	0.13	3.1	34.0	0.21
Clay - drained	10.7	26.7	0.93	-	-	-

Table 2 Summary of results for textured HDPE geomembrane

The information available on the interface shear strength between textured HDPE geomembranes and geonets is limited and this may be because

the increase in interface shear strength over and above the smooth geomembrane is marginal. Figure 2a summarises the available information, although there are only five data points for the peak strength and three points for the residual strength. The peak interface shear strength based on this data is δ = 11.0° and α = 3.0 kPa with a correlation coefficient of 0.98, which compares with a friction angle of 9.0° for the smooth geomembrane case (Figure 1a). The residual interface shear strength for the textured geomembrane (δ = 9.1° and α = 9.2 kPa) needs to be treated with care since it is only based on three data points.

The majority of data presented for the shear strength of textured geomembrane/non-woven geotextile interfaces are from the results of the testing carried out by the authors (Jones & Dixon, 1998), although other information from the literature has been used to develop Figure 2b. A peak friction angle of 25.8° is obtained together with a cohesion intercept of 6.9 kPa, which reduces to residual values of δ = 13.1° and α = 3.6 kPa, although there is a significant range of values with R^2 values of 0.88 for both the peak and residual case.

The interface shear strength results for the textured geomembrane/sand interface are shown on Figure 2c which give peak parameters of δ = 27.4° and α = 6.9 kPa with a correlation coefficient of 0.96. This interface, although strain softening, does not seem to exhibit a large reduction in shear strength with increased displacement since the residual friction angle is 25.5° with a relatively high cohesion intercept of 15.5 kPa.

From the results of undrained tests on textured HDPE geomembrane against clays (Figure 2d), it can be seen that the dependency of shear strength on normal stress is limited with peak and residual friction angles of 4.4° and 3.1° respectively. Cohesion intercepts for both peak and large strain conditions are similar with a peak value of 36.0 kPa and a residual value of 34.0 kPa, however both envelopes give poor linear relationships with R^2 values of 0.13 and 0.21. The shape of the envelopes suggest that the shear strength between textured geomembrane and a clay tested without an allowance for the dissipation of pore pressures is almost independent of normal stress, and is likely to be related to the undrained shear strength of the clay. Since the data shown on Figure 2d has been obtained from eight separate references with different clay at different remoulding conditions, the extent of the data scatter is not surprising.

The results shown on Figure 2d compare well with the observations made by Orman (1994), who found that failure of a textured HDPE geomembrane/silt interface occurred within the silt along the line of the asperities on the geomembrane sheet. Thus it is to be expected that the undrained interface shear strength of a textured geomembrane/clay is independent of normal stress and probably equal to the undrained shear strength of the clay.

There is little information on geomembrane/clay interfaces tested at strain rates slow enough to dissipate pore waters pressures although the available data indicates that the shear strength of this interface is dependent on normal stress (Figure 2e). Again the small amount of data available means that caution is required when analysing the results, however, I

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interface shear strength corresponding to $\delta = 10.7°$ and $\alpha = 26.7$ kPa. Closer inspection of the plot reveals that a non-linear fit may be more representative for the peak shear strength envelope, possibly curving downwards at lower normal stresses and passing through the origin. There is insufficient data to determine the residual shear strength for this interface, however, it is likely that the residual interface shear strength will be the residual shear strength of the clay. The asperities of the textured geomembrane are very similar to the upper sintered brass platten on the standard Bromhead ring shear apparatus (Bromhead 1979).

Non-woven geotextile

The results of testing on non-woven geotextiles are presented in Figure 3 and a summary is given in Table 3 below.

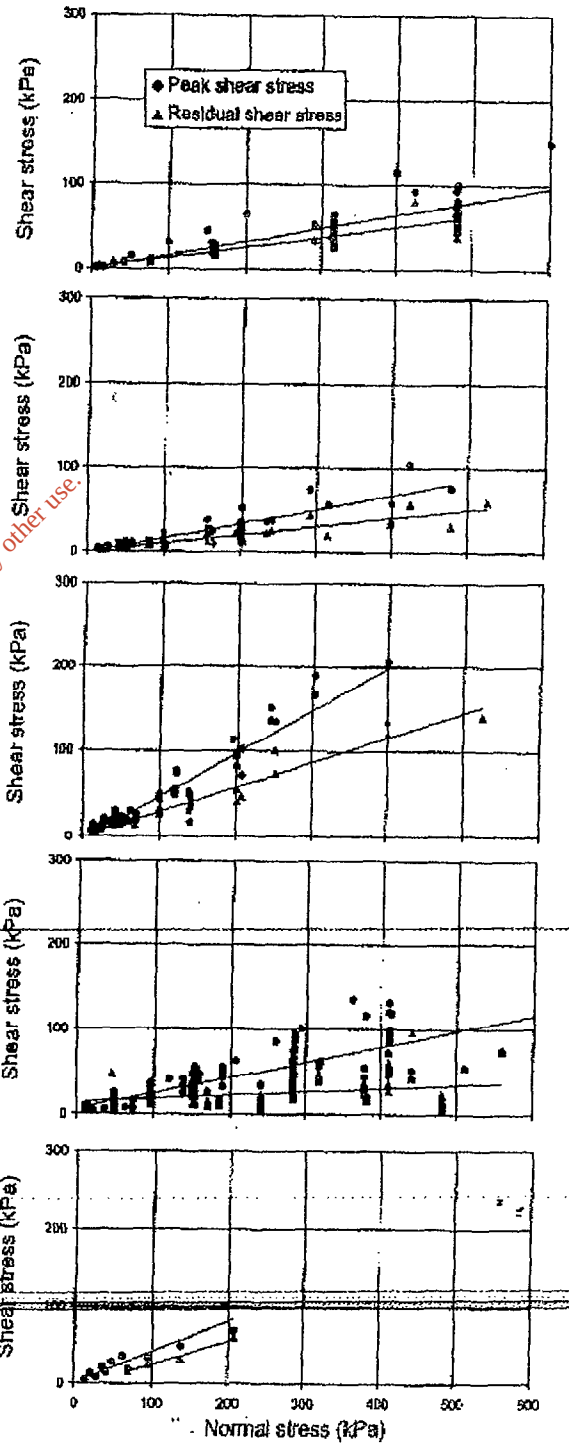
Interface	Interface shear strength parameters					
	Peak			Residual		
	δ (°)	α (kPa)	R^2	δ (°)	α (kPa)	R^2
Geonet	13.1	17.9	0.76	15.4	4.1	0.92
Gravel	35.0	-1.0	0.87	19.9	30.1	0.99
Sand	33.0	-1.3	0.93	28.7	7.7	0.92
Clay - undrained	25.3	5.3	0.91	17.7	55.6	0.98
Clay - drained	32.5	4.4	0.98	-	-	-

Table 3 Summary of results for non-woven geotextile

The results of shear strength testing on non-woven geotextile/geonet interfaces are plotted in Figure 3a and linear regression of all the data points give peak interface shear strengths of $\delta = 13.1°$ and $\alpha = 17.9$ kPa with an R^2 value of 0.76. For the range of normal stresses considered, the residual envelope is similar to the peak in terms of its mobilised shear strength, however the friction angles and cohesion intercept are different. The best fit line through the residual data points is given by $\delta = 15.4°$ and $\alpha = 4.1$ kPa, i.e. a higher friction angle but a lower cohesion intercept with a correlation coefficient of 0.92.

The non-woven geotextile/gravel interface has a high shear strength with some values in the literature reported as high as 48°. Most of the results available are for tests carried out at normal stresses less than 200 kPa (Figure 3b) and linear regression gives a friction angle of 35.0° with a cohesion intercept of -1.0 kPa. This reduces to a residual shear strength corresponding to $\delta = 19.9°$ and $\alpha = 30.1$ kPa. The peak shear strength envelope shows a reasonable strong straight line fit with a correlation coefficient of 0.94, while the residual envelope has a very strong fit with $R^2 = 0.99$, however the residual is based on a small number of data points.

There is much more information available in the literature on the interface shear strength between sand and non-woven geotextiles, and this is also a high strength interface with a peak friction angle of 33.0° and a cohesion



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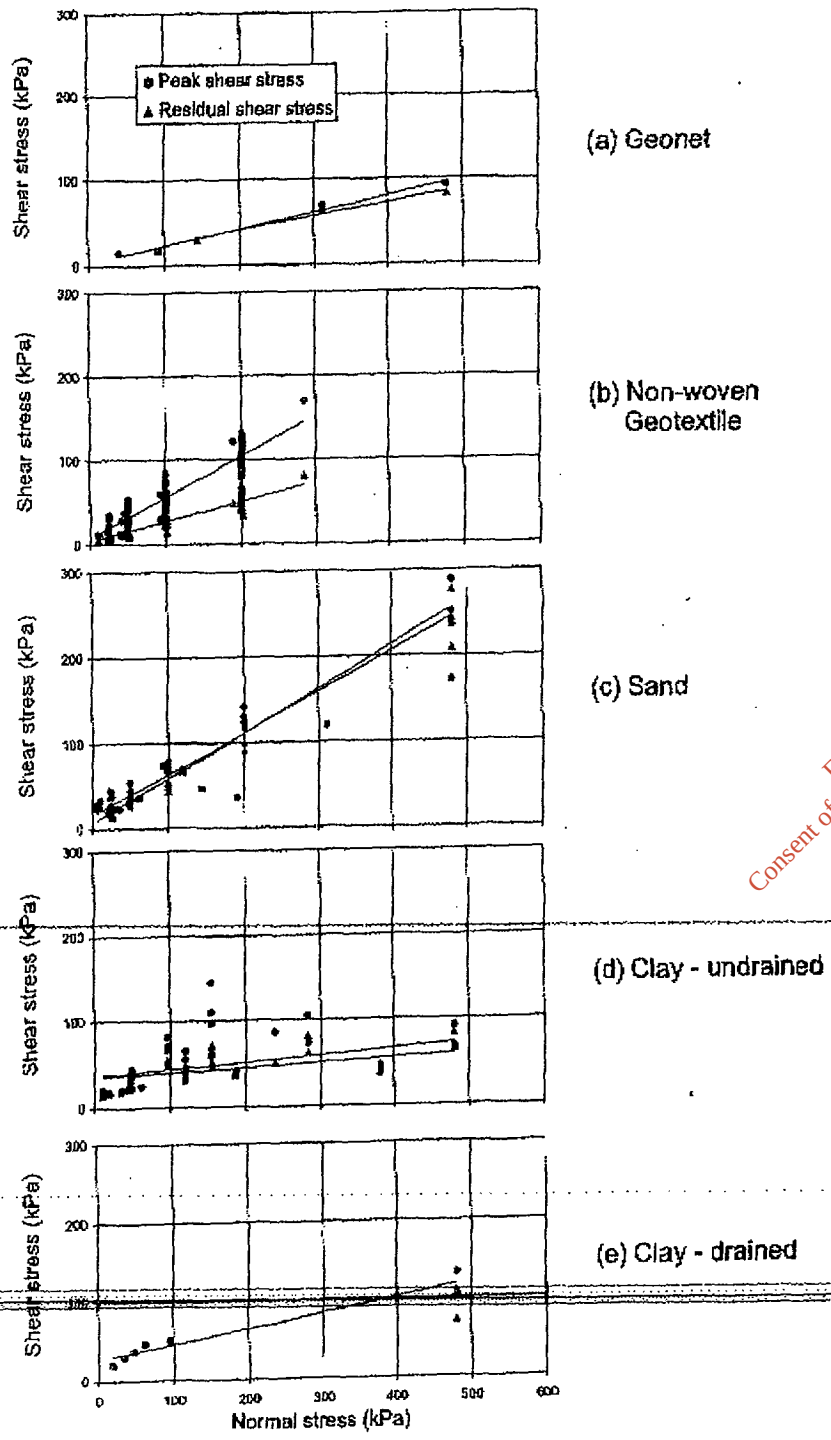

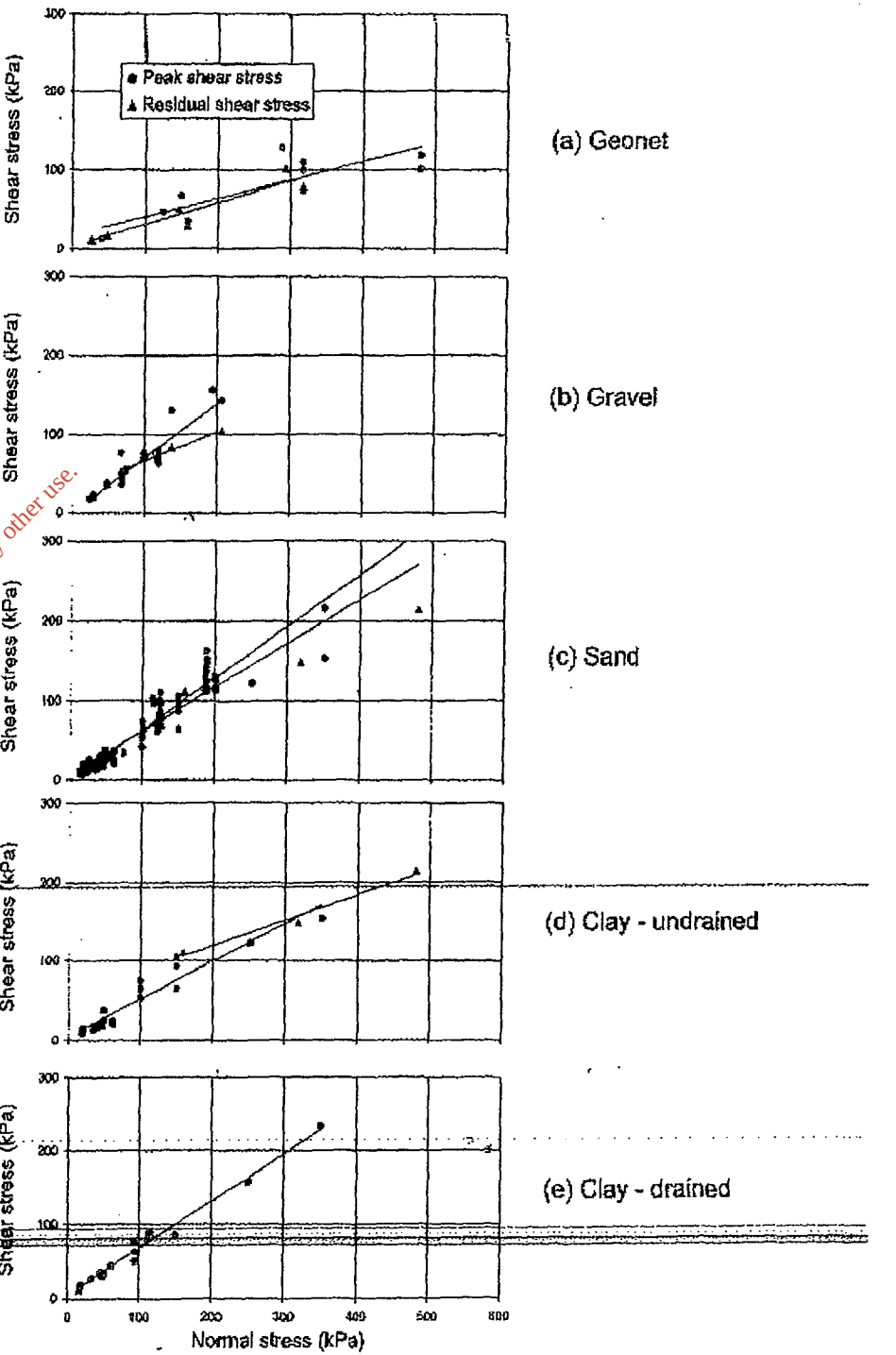

Figure 3. Non-woven Geotextile

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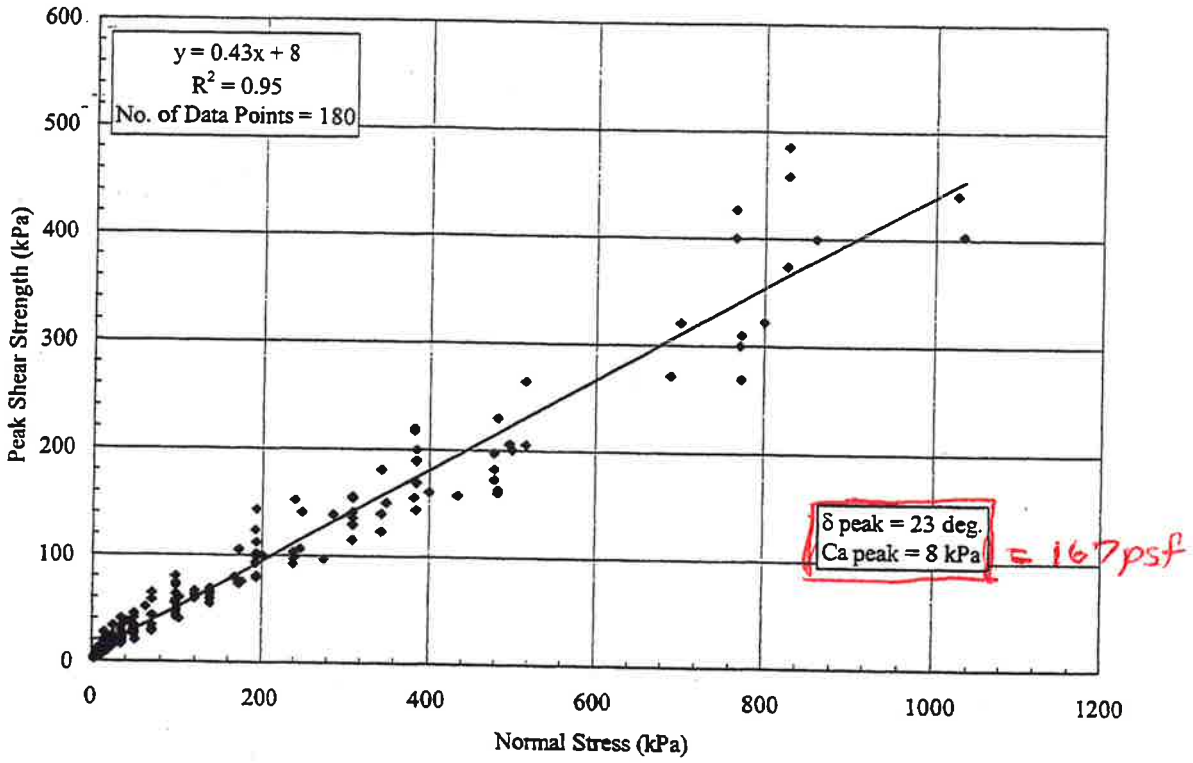
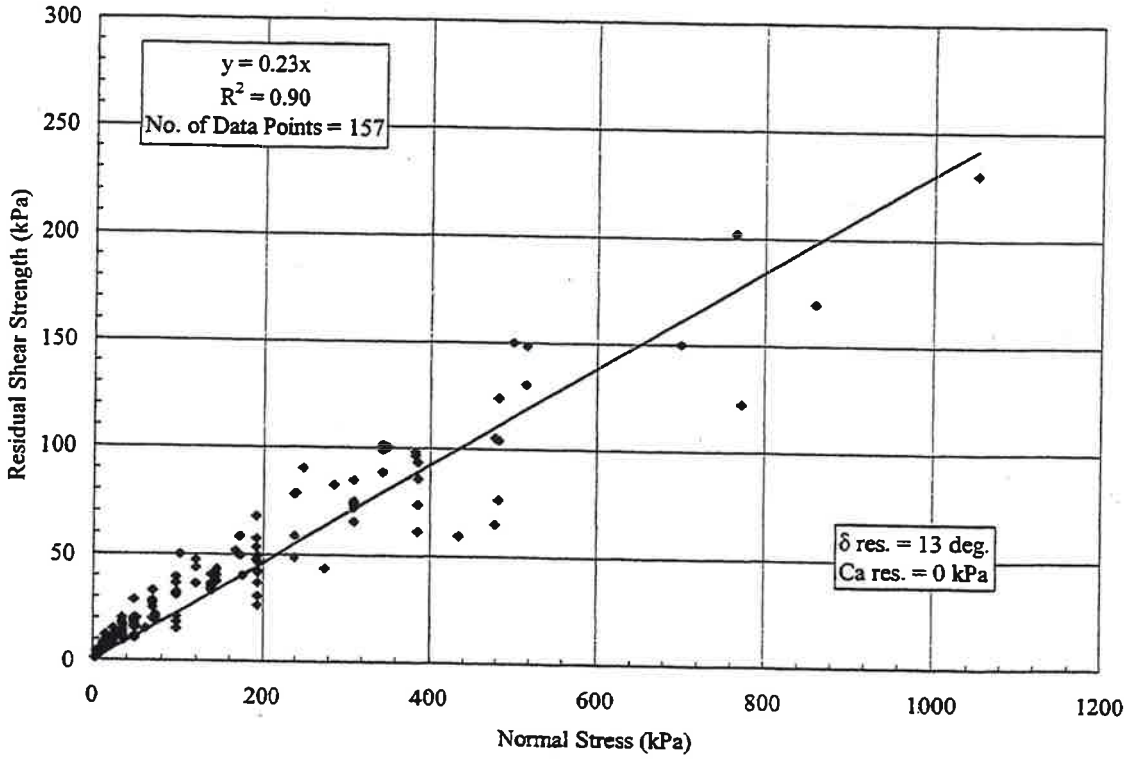


Appendix Figure 11a - Peak Shear Strength; Textured HDPE against NW-NP Side of Fabric-Reinforced GCL.



Appendix Figure 11b - Residual Shear Strength; Textured HDPE against NW-NP Side of Fabric-Reinforced GCL.

BASKERVILLE-DONOVAN, INC.
Innovative Infrastructure Solutions

22219 Panama City Beach Parkway, Suite 200
Panama City Beach, Florida 32413
Phone: 850.230.6150
Fax: 850.230.6155

March 6, 2012

Mr. Jamie Jones – Community Operations Division Manager
Bay County Community Operation Division
8110 John Pitts Road
Panama City, FL 32404

RE: Steelfield Road Landfill Expansion
BDI Project No. 06272.01
Bay County, Florida

Ref: Liner and Geocomposite Testing Results for Purchase Authorization

Dear Mr. Jones:

Baskerville Donovan, Inc. (BDI) has reviewed the Interface Friction Test results for the liner elements that were performed by TRI for Phoenix Construction Services, Inc. (PCS) for the referenced project as submitted on March 5, 2012 (copy enclosed). The following table presents the results of the tested interface and the required interface friction angle as presented in the specifications and our interface shear testing letter dated February 24, 2012.

Interface	Test Method	Result (peak angle, degrees)	Specified Minimum (peak angle, degrees)	Does test result meet the CQA Testing requirements?
Soil/GCL	ASTM D 6243	26.1	23	Yes
GCL/60-mil Agru Microspike Liner	ASTM D 5321	23.4	23	Yes
Double Sided Geocomposite/60-mil Agru Microspike Liner	ASTM D 5321	22.0	17	Yes

Based on these test results, there is reasonable assurance to expect the liner system materials proposed by PCS will be capable of meeting the CQA conformance testing requirements as modified in the referenced letter dated February 24, 2012 from Mark Shaeffer of BDI to Ted Schoppe IV of PCS. Based upon this information, it is believed that the County has reasonable assurances needed to proceed with the purchase authorization for the Bentomat GCL, Skapps Double Sided Geocomposite and 60-mil Agru Microspike Liner materials as submitted by PCS.

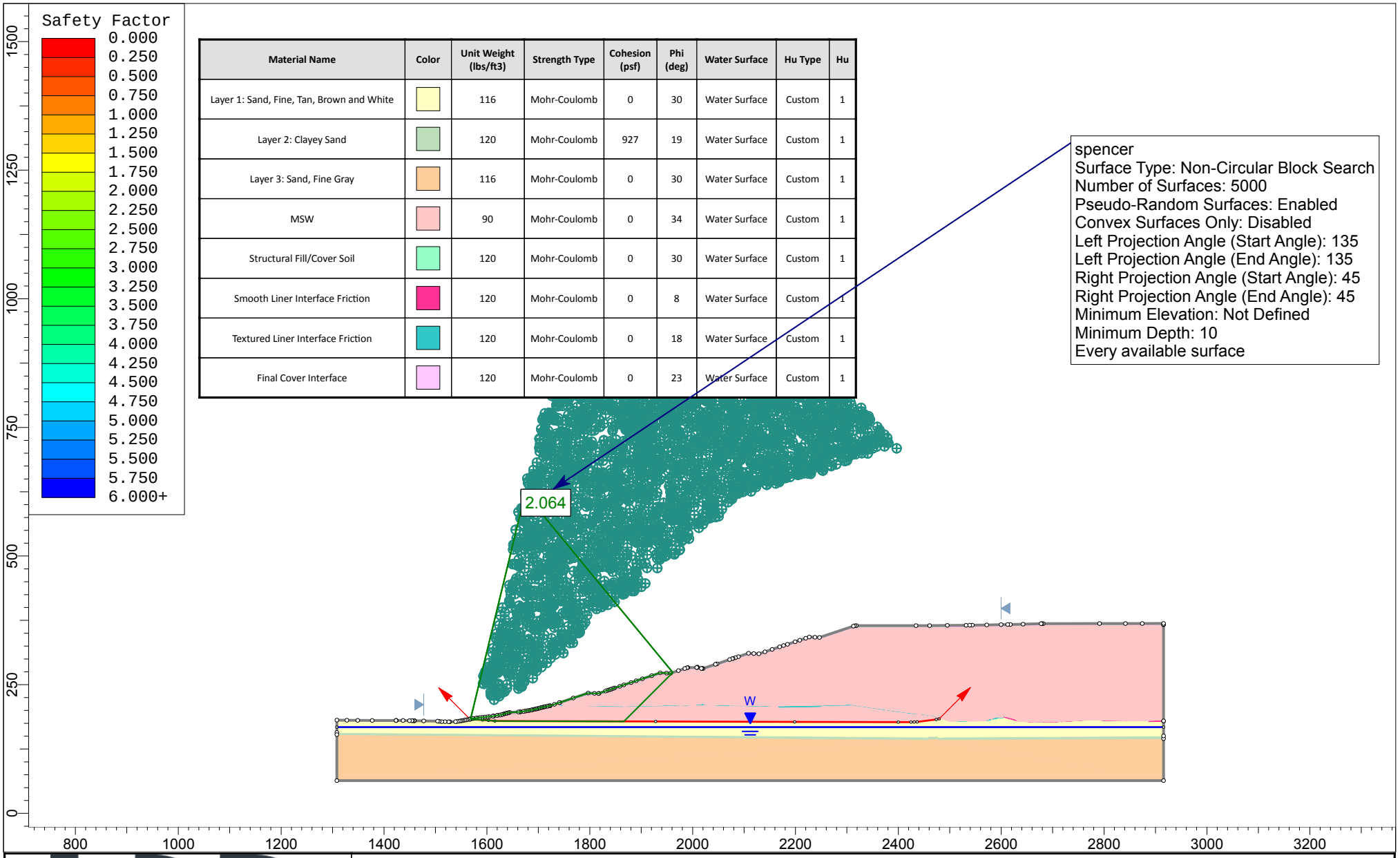
As always, please contact me at the address and telephone number listed on this letterhead should you have any questions regarding this matter.

Sincerely,

Mark E. Shaeffer, P.E.
MES/mes
Enc.
Cc: Mr. Glenn Ogborn, Solid Waste Facilities Manager (w/ enc.)
L:\062 - Bay County\06272.01 Bay Co Landfill\Correspondence\Correspondence\Liner Materials Testing Results 6 Mar 2012.doc

Attachment B-3

SLIDE Output



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface	Hu Type	Hu
Layer 1: Sand, Fine, Tan, Brown and White		116	Mohr-Coulomb	0	30	Water Surface	Custom	1
Layer 2: Clayey Sand		120	Mohr-Coulomb	927	19	Water Surface	Custom	1
Layer 3: Sand, Fine Gray		116	Mohr-Coulomb	0	30	Water Surface	Custom	1
MSW		90	Mohr-Coulomb	0	34	Water Surface	Custom	1
Structural Fill/Cover Soil		120	Mohr-Coulomb	0	30	Water Surface	Custom	1
Smooth Liner Interface Friction		120	Mohr-Coulomb	0	8	Water Surface	Custom	1
Textured Liner Interface Friction		120	Mohr-Coulomb	0	18	Water Surface	Custom	1
Final Cover Interface		120	Mohr-Coulomb	0	23	Water Surface	Custom	1

spencer
Surface Type: Non-Circular Block Search
Number of Surfaces: 5000
Pseudo-Random Surfaces: Enabled
Convex Surfaces Only: Disabled
Left Projection Angle (Start Angle): 135
Left Projection Angle (End Angle): 135
Right Projection Angle (Start Angle): 45
Right Projection Angle (End Angle): 45
Minimum Elevation: Not Defined
Minimum Depth: 10
Every available surface

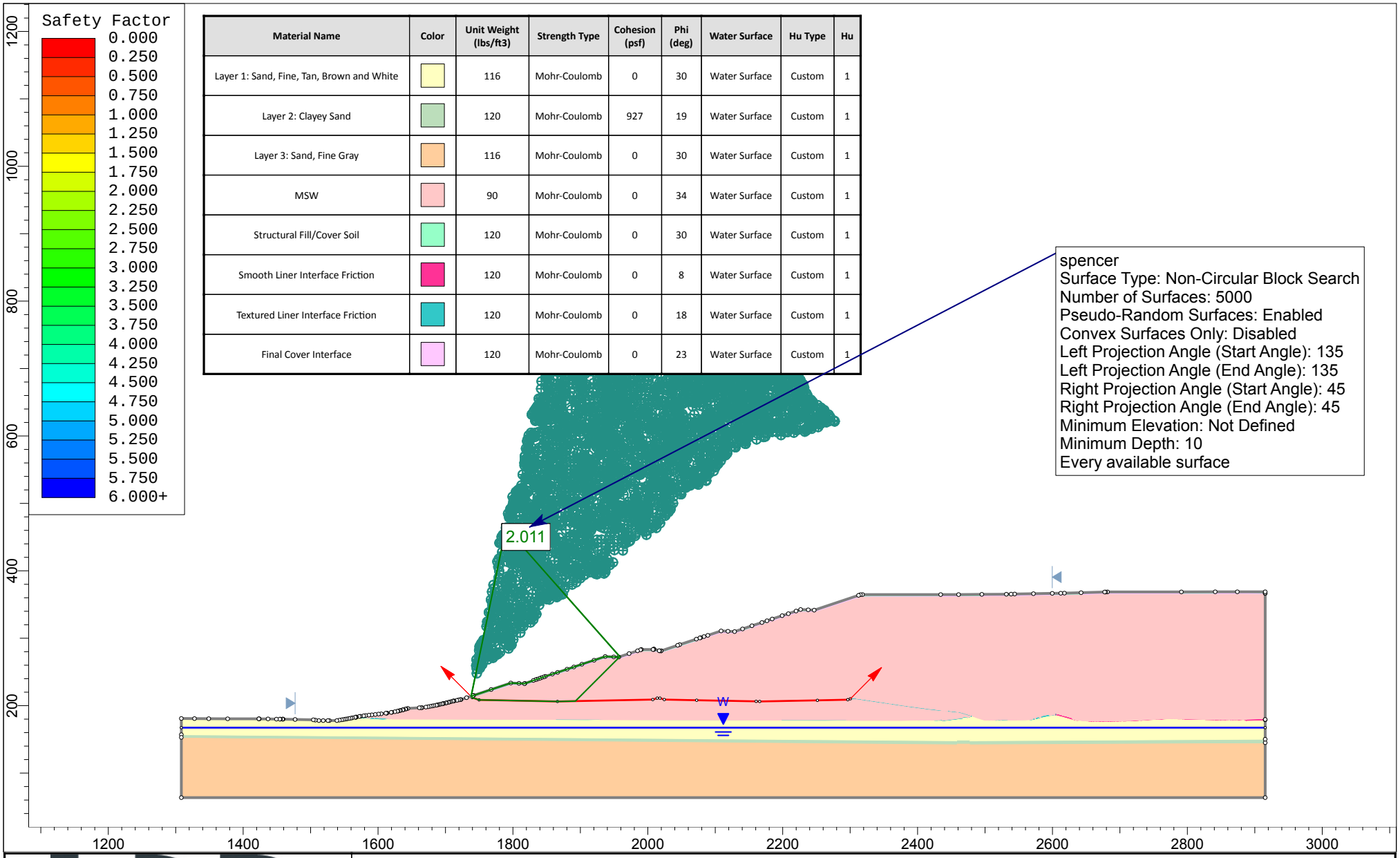
Slope Stability Analysis For East-West Section-Final Condition

Analysis Description	Sliding Block Failure Through Cell 1 and 2 Bottom Liner - Under Static Conditions		
Analyzed By	KP	*Scale* 1:3094	*Checked By* MGR
Date	02-04-2019	*File Name*	East-West ESA-Block-Cell 1 and 2.slim

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface	Hu Type	Hu
Layer 1: Sand, Fine, Tan, Brown and White		116	Mohr-Coulomb	0	30	Water Surface	Custom	1
Layer 2: Clayey Sand		120	Mohr-Coulomb	927	19	Water Surface	Custom	1
Layer 3: Sand, Fine Gray		116	Mohr-Coulomb	0	30	Water Surface	Custom	1
MSW		90	Mohr-Coulomb	0	34	Water Surface	Custom	1
Structural Fill/Cover Soil		120	Mohr-Coulomb	0	30	Water Surface	Custom	1
Smooth Liner Interface Friction		120	Mohr-Coulomb	0	8	Water Surface	Custom	1
Textured Liner Interface Friction		120	Mohr-Coulomb	0	18	Water Surface	Custom	1
Final Cover Interface		120	Mohr-Coulomb	0	23	Water Surface	Custom	1

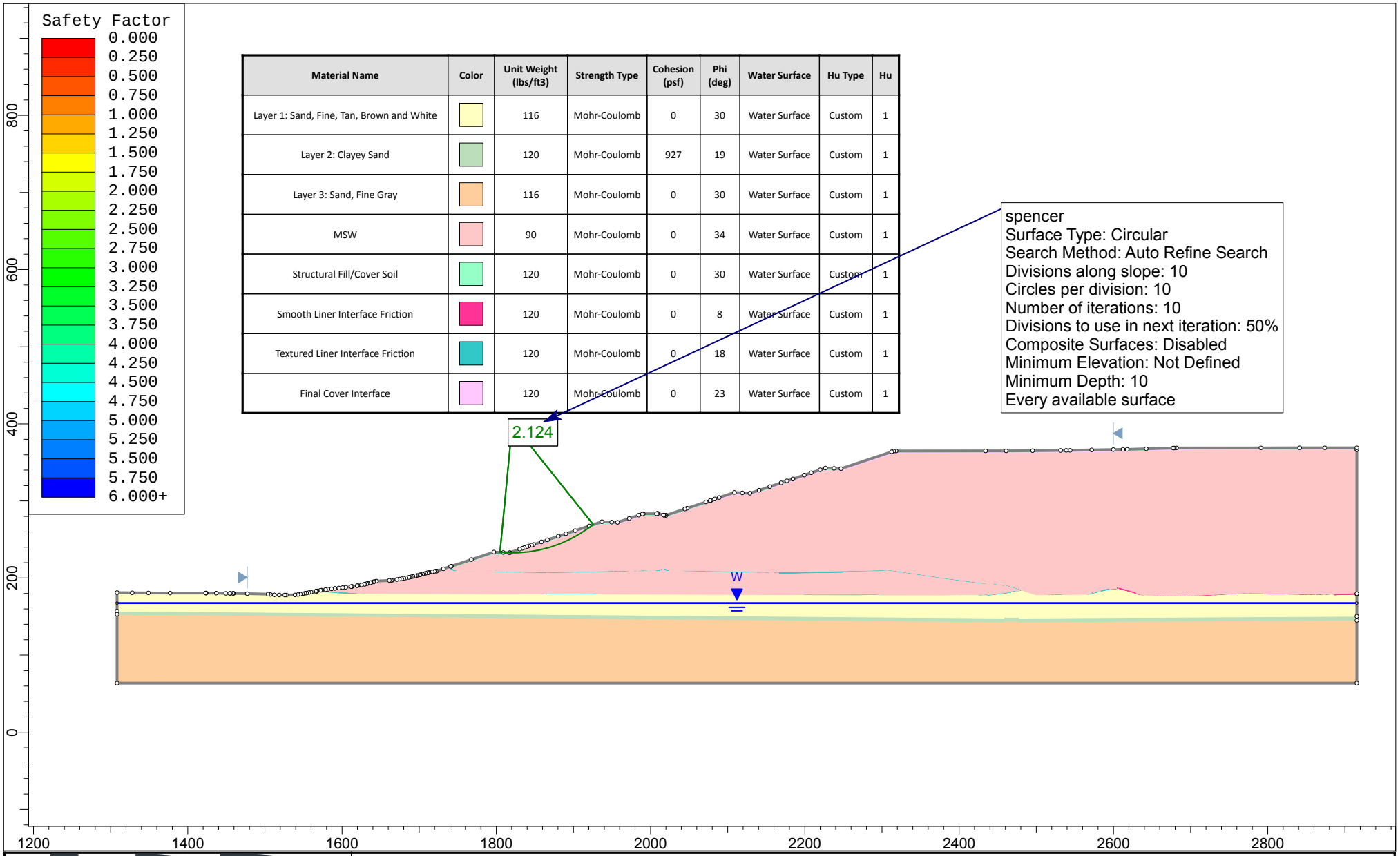
Safety Factor
0.000
0.250
0.500
0.750
1.000
1.250
1.500
1.750
2.000
2.250
2.500
2.750
3.000
3.250
3.500
3.750
4.000
4.250
4.500
4.750
5.000
5.250
5.500
5.750
6.000+

spencer
Surface Type: Non-Circular Block Search
Number of Surfaces: 5000
Pseudo-Random Surfaces: Enabled
Convex Surfaces Only: Disabled
Left Projection Angle (Start Angle): 135
Left Projection Angle (End Angle): 135
Right Projection Angle (Start Angle): 45
Right Projection Angle (End Angle): 45
Minimum Elevation: Not Defined
Minimum Depth: 10
Every available surface



Slope Stability Analysis For East-West Section-Final Condition

Analysis Description	Sliding Block Failure Through Proposed Liner - Under Static Conditions		
Analyzed By	KP	Scale 1:2361	Checked By MGR
Date	02-04-2019		File Name East-West ESA-Block-Proposed Base Liner.slim

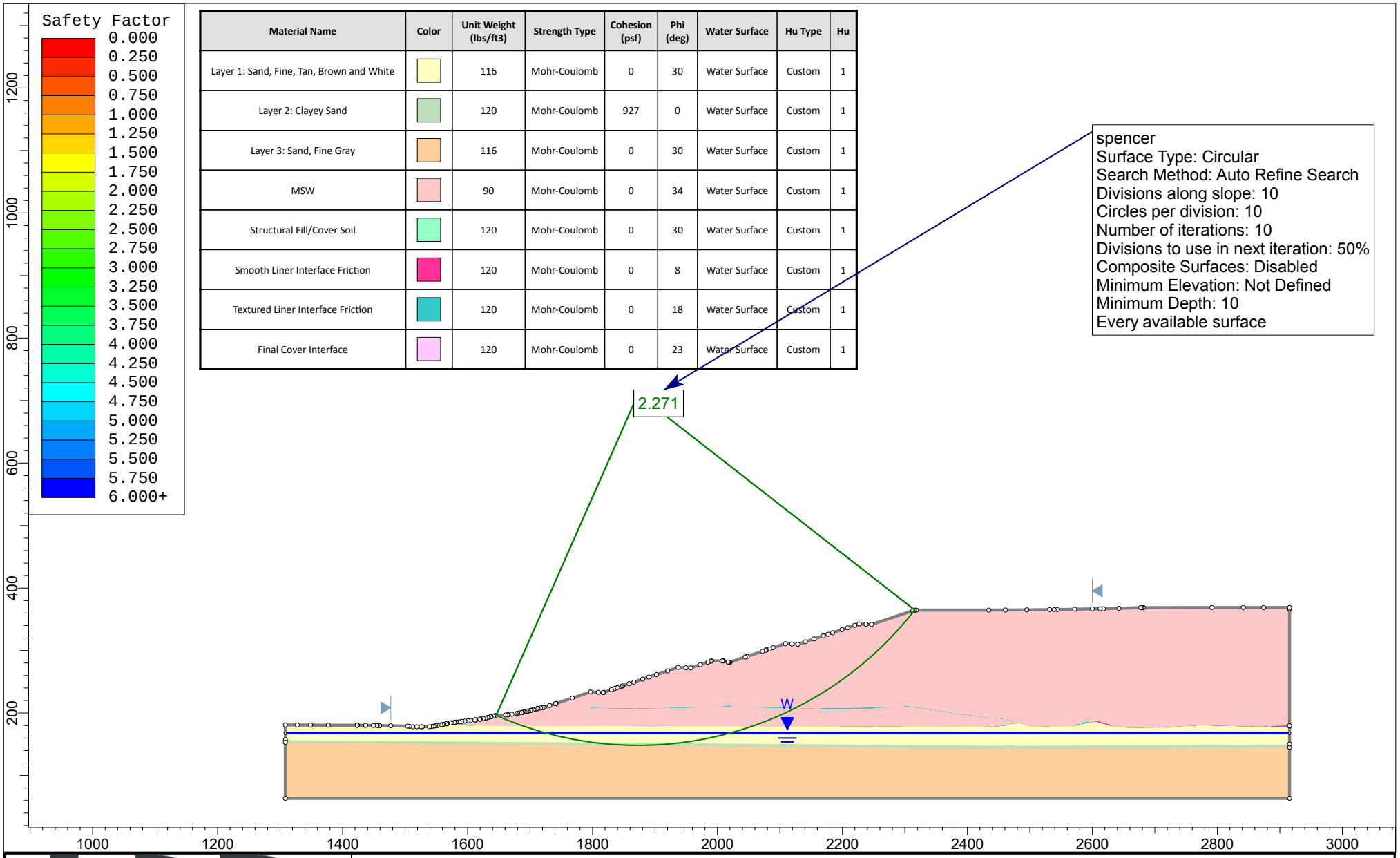


Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface	Hu Type	Hu
Layer 1: Sand, Fine, Tan, Brown and White		116	Mohr-Coulomb	0	30	Water Surface	Custom	1
Layer 2: Clayey Sand		120	Mohr-Coulomb	927	19	Water Surface	Custom	1
Layer 3: Sand, Fine Gray		116	Mohr-Coulomb	0	30	Water Surface	Custom	1
MSW		90	Mohr-Coulomb	0	34	Water Surface	Custom	1
Structural Fill/Cover Soil		120	Mohr-Coulomb	0	30	Water Surface	Custom	1
Smooth Liner Interface Friction		120	Mohr-Coulomb	0	8	Water Surface	Custom	1
Textured Liner Interface Friction		120	Mohr-Coulomb	0	18	Water Surface	Custom	1
Final Cover Interface		120	Mohr-Coulomb	0	23	Water Surface	Custom	1

spencer
Surface Type: Circular
Search Method: Auto Refine Search
Divisions along slope: 10
Circles per division: 10
Number of iterations: 10
Divisions to use in next iteration: 50%
Composite Surfaces: Disabled
Minimum Elevation: Not Defined
Minimum Depth: 10
Every available surface

Slope Stability Analysis For East-West Section-Final Condition

Analysis Description	Circular Failure Mode - Under Static Conditions		
Analyzed By	KP	*Scale* 1:2063	*Checked By* MGR
Date	02-04-2019	*File Name*	East-West ESA-Circular.slim



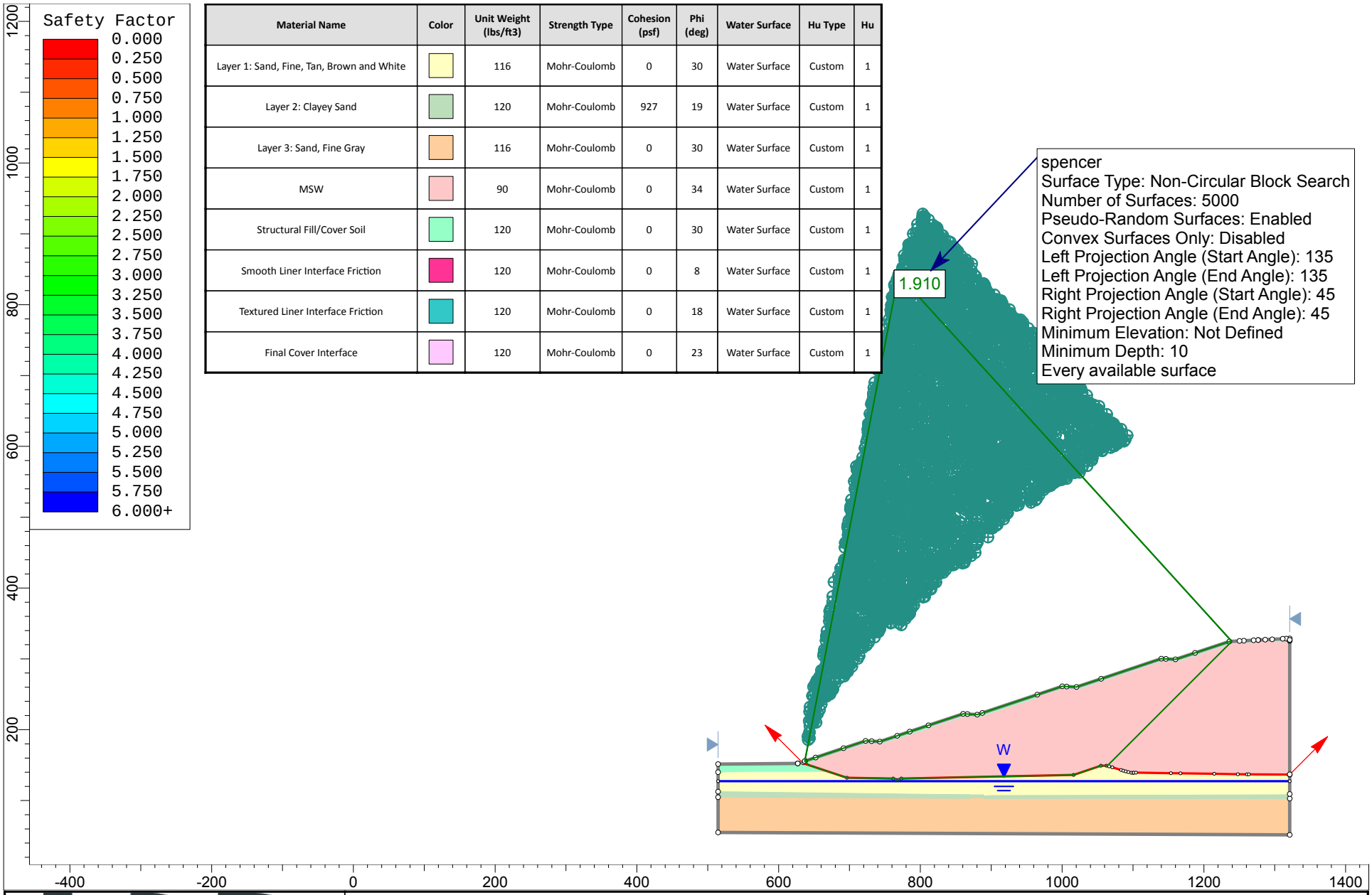
Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface	Hu Type	Hu
Layer 1: Sand, Fine, Tan, Brown and White		116	Mohr-Coulomb	0	30	Water Surface	Custom	1
Layer 2: Clayey Sand		120	Mohr-Coulomb	927	0	Water Surface	Custom	1
Layer 3: Sand, Fine Gray		116	Mohr-Coulomb	0	30	Water Surface	Custom	1
MSW		90	Mohr-Coulomb	0	34	Water Surface	Custom	1
Structural Fill/Cover Soil		120	Mohr-Coulomb	0	30	Water Surface	Custom	1
Smooth Liner Interface Friction		120	Mohr-Coulomb	0	8	Water Surface	Custom	1
Textured Liner Interface Friction		120	Mohr-Coulomb	0	18	Water Surface	Custom	1
Final Cover Interface		120	Mohr-Coulomb	0	23	Water Surface	Custom	1

spencer
Surface Type: Circular
Search Method: Auto Refine Search
Divisions along slope: 10
Circles per division: 10
Number of iterations: 10
Divisions to use in next iteration: 50%
Composite Surfaces: Disabled
Minimum Elevation: Not Defined
Minimum Depth: 10
Every available surface

Slope Stability Analysis For East-West Section-Final Condition

Analysis Description	Circular Failure Under Undrained Subsurface		
Analyzed By	KP	*Scale* 1:2547	*Checked By* MGR
Date	02-04-2019	*File Name*	East-West tTSA-Circular.slim

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface	Hu Type	Hu
Layer 1: Sand, Fine, Tan, Brown and White		116	Mohr-Coulomb	0	30	Water Surface	Custom	1
Layer 2: Clayey Sand		120	Mohr-Coulomb	927	19	Water Surface	Custom	1
Layer 3: Sand, Fine Gray		116	Mohr-Coulomb	0	30	Water Surface	Custom	1
MSW		90	Mohr-Coulomb	0	34	Water Surface	Custom	1
Structural Fill/Cover Soil		120	Mohr-Coulomb	0	30	Water Surface	Custom	1
Smooth Liner Interface Friction		120	Mohr-Coulomb	0	8	Water Surface	Custom	1
Textured Liner Interface Friction		120	Mohr-Coulomb	0	18	Water Surface	Custom	1
Final Cover Interface		120	Mohr-Coulomb	0	23	Water Surface	Custom	1

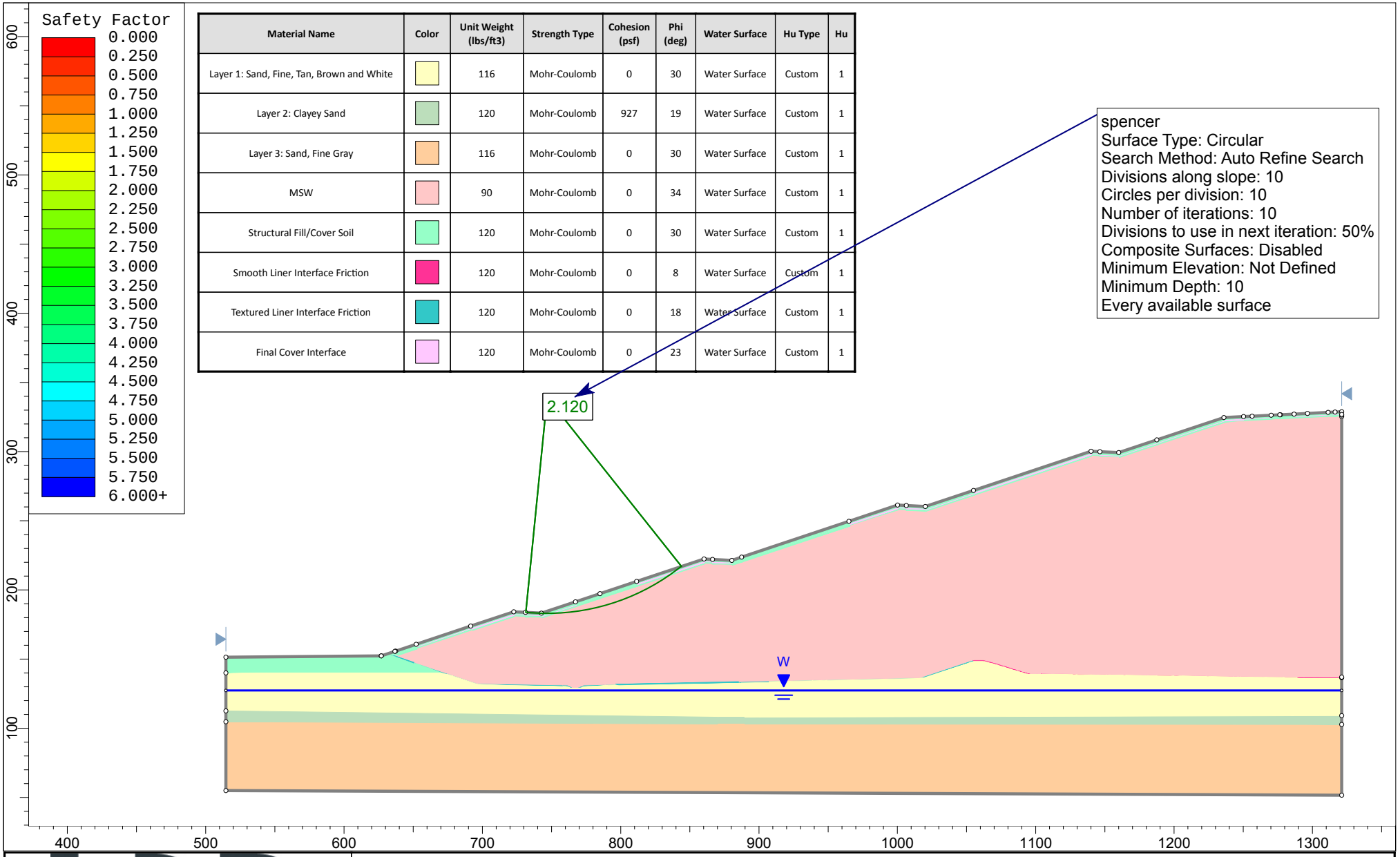


Slope Stability Analysis For North South Section-Final Condition

Analysis Description	Block Failure Through Bottom Liner				
Analyzed By	KP	*Scale*	1:2312	*Checked By*	MGR
Date	02-04-2019			*File Name*	North-South ESA-Block-through bottom liner.slim

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface	Hu Type	Hu
Layer 1: Sand, Fine, Tan, Brown and White		116	Mohr-Coulomb	0	30	Water Surface	Custom	1
Layer 2: Clayey Sand		120	Mohr-Coulomb	927	19	Water Surface	Custom	1
Layer 3: Sand, Fine Gray		116	Mohr-Coulomb	0	30	Water Surface	Custom	1
MSW		90	Mohr-Coulomb	0	34	Water Surface	Custom	1
Structural Fill/Cover Soil		120	Mohr-Coulomb	0	30	Water Surface	Custom	1
Smooth Liner Interface Friction		120	Mohr-Coulomb	0	8	Water Surface	Custom	1
Textured Liner Interface Friction		120	Mohr-Coulomb	0	18	Water Surface	Custom	1
Final Cover Interface		120	Mohr-Coulomb	0	23	Water Surface	Custom	1

spencer
Surface Type: Circular
Search Method: Auto Refine Search
Divisions along slope: 10
Circles per division: 10
Number of iterations: 10
Divisions to use in next iteration: 50%
Composite Surfaces: Disabled
Minimum Elevation: Not Defined
Minimum Depth: 10
Every available surface



Slope Stability Analysis For East-West Section-Final Condition

Analysis Description	Circular Failure -Effective Stress Conditions		
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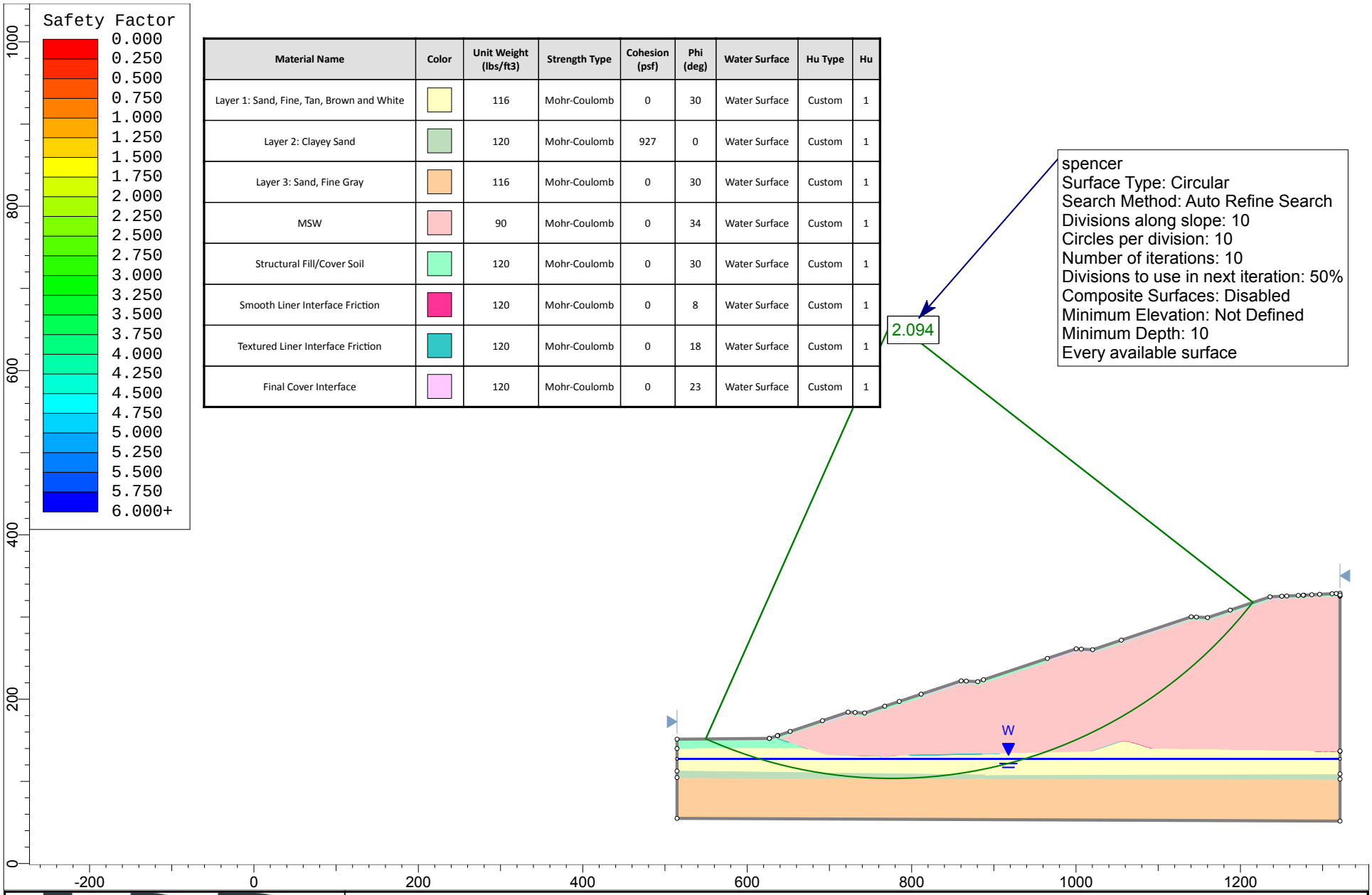
Date	02-04-2019		File Name North-South ESA-Circular.slim



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface	Hu Type	Hu
Layer 1: Sand, Fine, Tan, Brown and White		116	Mohr-Coulomb	0	30	Water Surface	Custom	1
Layer 2: Clayey Sand		120	Mohr-Coulomb	927	0	Water Surface	Custom	1
Layer 3: Sand, Fine Gray		116	Mohr-Coulomb	0	30	Water Surface	Custom	1
MSW		90	Mohr-Coulomb	0	34	Water Surface	Custom	1
Structural Fill/Cover Soil		120	Mohr-Coulomb	0	30	Water Surface	Custom	1
Smooth Liner Interface Friction		120	Mohr-Coulomb	0	8	Water Surface	Custom	1
Textured Liner Interface Friction		120	Mohr-Coulomb	0	18	Water Surface	Custom	1
Final Cover Interface		120	Mohr-Coulomb	0	23	Water Surface	Custom	1

spencer
Surface Type: Circular
Search Method: Auto Refine Search
Divisions along slope: 10
Circles per division: 10
Number of iterations: 10
Divisions to use in next iteration: 50%
Composite Surfaces: Disabled
Minimum Elevation: Not Defined
Minimum Depth: 10
Every available surface

Slope Stability Analysis For North South Section-Final Condition

Analysis Description	Circular Failure - Undrained Conditions				
Analyzed By	KP	*Scale*	1:1993	*Checked By*	MGR
Date	02-04-2019			*File Name*	North-South TSA-Circular.slim

Attachment B-4

Final Cover Geocomposite Hydraulic Conductivity

HDR Engineering, Inc.

Job No.				No.	
Project	Steelfield Landfill Expansion	Computed	K. Perera	Date	2/4/2019
Task	Hydraulic Conductivity and Drainage Layer Capacity	Checked	TY	Date	2/27/2019

Problem

Determine the hydraulic conductivity and drainage capacity of the geocomposite drainage layer in the final cover sideslope for Scenario 4 of HELP model.

Transmissivity Calculations

Final cover liner from bottom to top: 40-mil LLDPE textured - Double-sided Geocomposite - 1.5 Ft Erosion Control Layer- 6-inch top soil

Determine 100-hour transmissivity (Θ_{100}).

Use GSE 300-mil GSE FabriNet or equivalent (see attached cut sheet with product specifications).

$$\Theta_{allow} = \frac{\Theta_{100}}{RF_{CR} * RF_{CC} * RF_{BC}}$$

$$\Theta_{allow} = \Theta_{req} \cdot FS_D$$

$$\Theta_{req} = k \cdot b$$

where,

- Θ_{allow} = minimum allowable transmissivity of geocomposite (cm^2/s)
- Θ_{req} = required transmissivity for site (cm^2/s)
- FS_D = overall factor of safety for drainage
- Θ_{100} = transmissivity after 100 hrs. under expected load (cm^2/s)
- RF_{CR} = reduction factor for creep deformation
- RF_{CC} = reduction factor for chemical clogging
- RF_{BC} = reduction factor for biological clogging
- k = saturated hydraulic conductivity (cm/s)
- b = thickness of geonet (cm)

Information based on product data for GSE FabriNet TRx 300mil. Site specific testing is recommended before selecting the material for installation to confirm minimum transmissivity.

Parameters	Geocomposite (300-mil) @ 1,000 psf (1)
Slope	33.00
FS_D (2)	1.5
RF_{CR} (2)	1.02
RF_{CC} (2)	1.1
RF_{BC} (2)	1.4
Θ_{100} (m^2/s)	0.0015
Θ_{allow} (cm^2/s)	9.55
Θ_{req} (cm^2/s)	6.37
Manufactured Thickness (mils)	300
Manufactured Thickness (cm)	0.76
Design Hydraulic Conductivity (cm/s)	8.35
Retained Final Thickness (mils)	285.0

Notes:

1. psf = pounds per square foot
2. Reduction factors based on HDR experience and information provided by GSE.

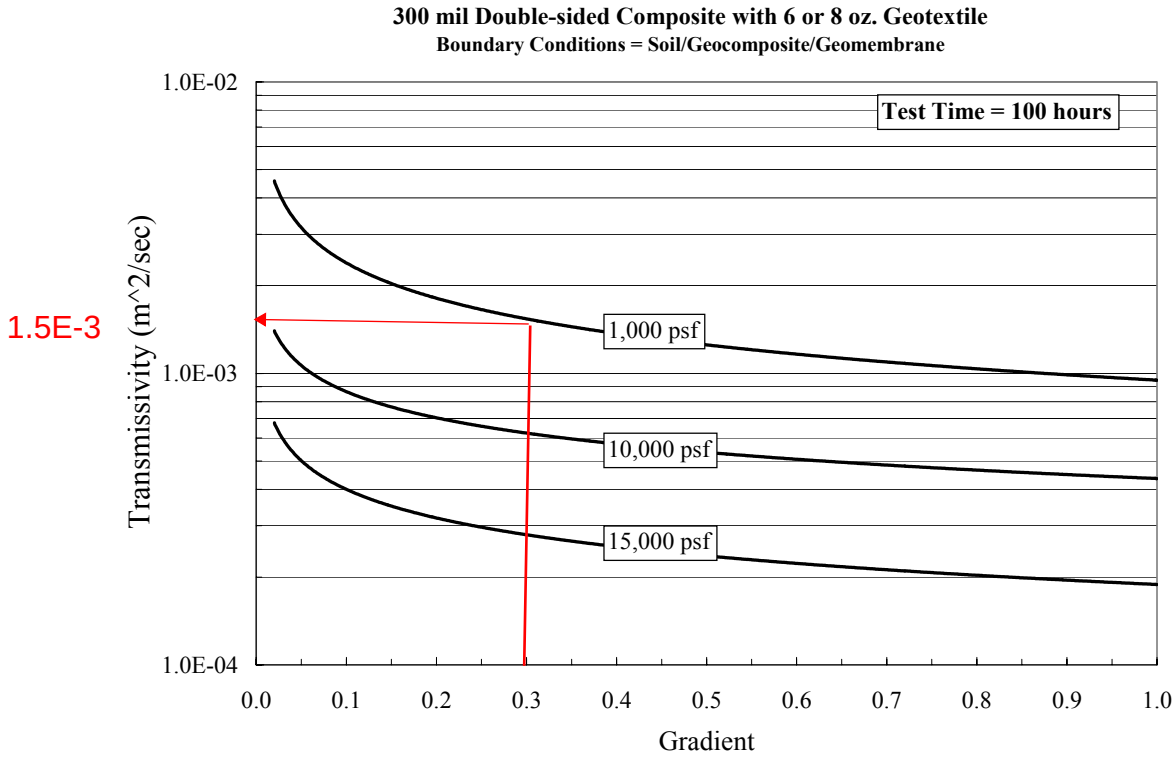


Figure A-9 100-hour transmissivity of 300 mil biplanar geonet geocomposite under soil/geocomposite/geomembrane boundary conditions.

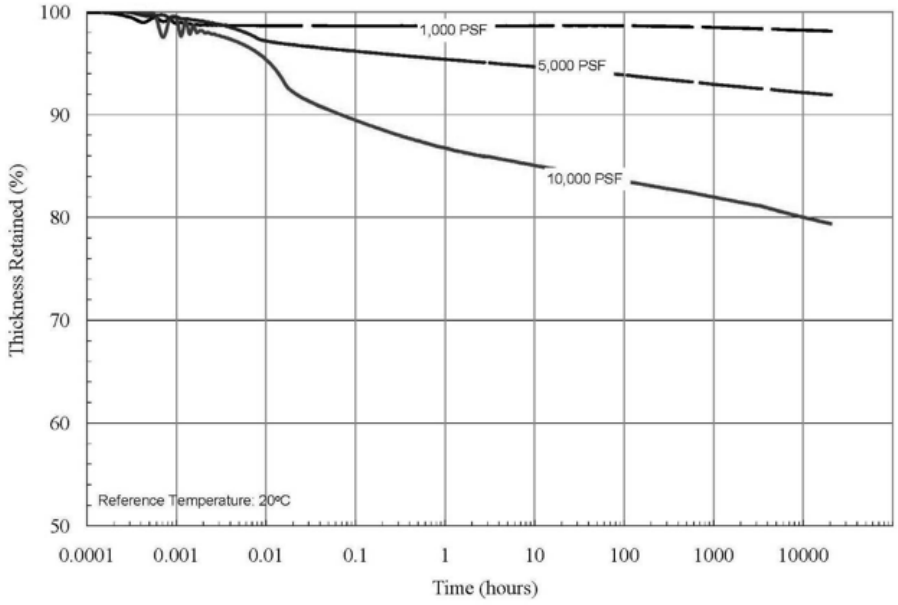


Figure 2 Creep Curves for a 250 mil HyperNet HF Geonet.

Table 2 Creep Reduction Factors for a 250 mil HyperNet HF Geonet from 100 hours to 50 Years.

Stress (psf)	Creep Reduction Factor
1,000	1.01
5,000	1.13
10,000	1.33

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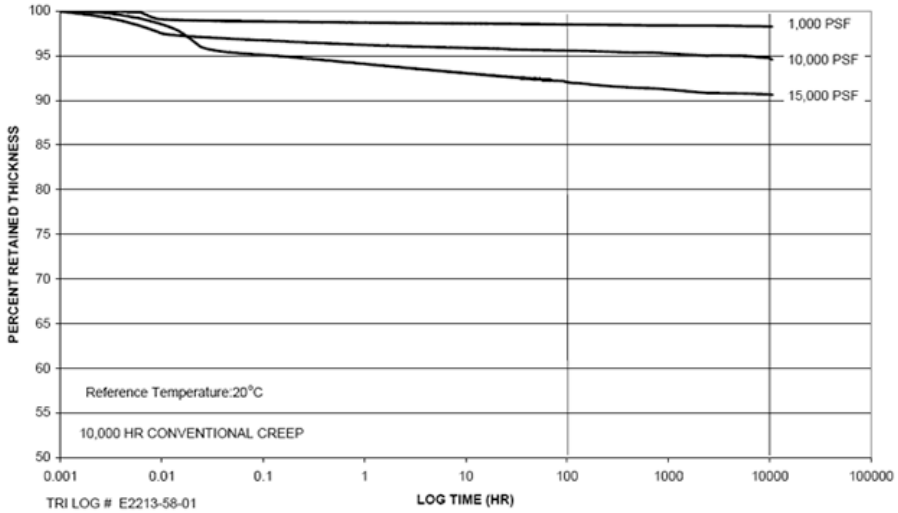

Figure 3 Creep Reduction Factors for a 300 mil HyperNet UF Geonet.

Table 3 Creep Reduction Factors for a 300 mil HyperNet UF Geonet from 100 hours to 50 Years.

Stress (psf)	Creep Reduction Factor
1,000	1.00
5,000	1.06
15,000	1.25

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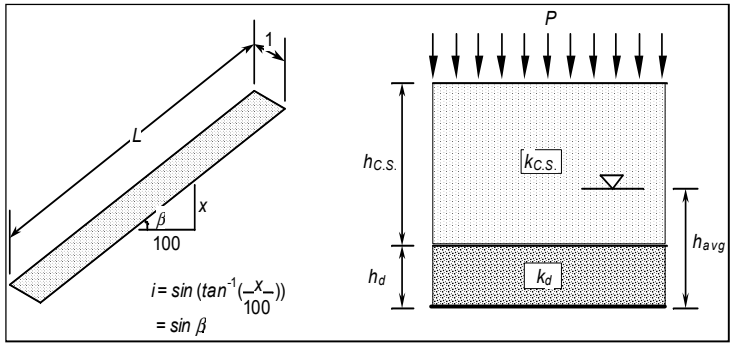
Attachment B-5

Veneer Stability of Side Slopes

Calculation of DLC and PSR: Final Cover-Side Slope



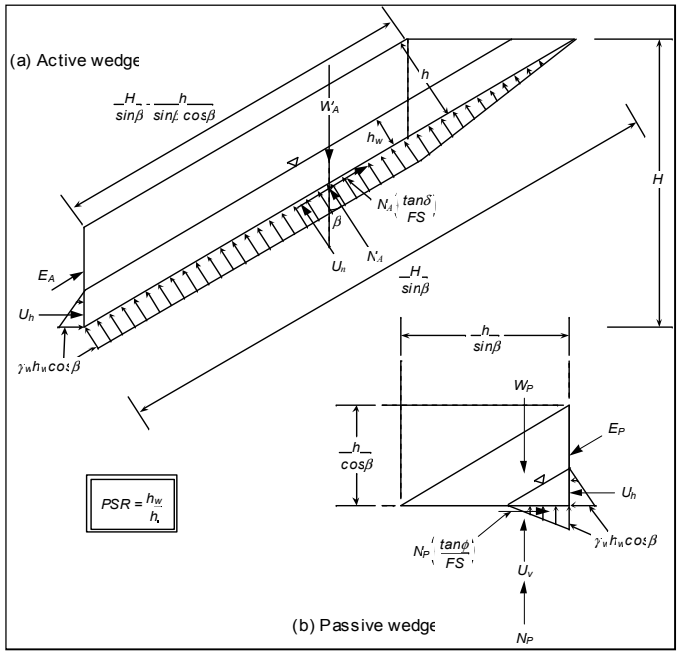
L =	32.0	m	i =	*0.3160*	
β =	18.4	°	$L(\cos\beta)$ =	*30.36*	m
			x =	*10.11*	m
$h_{C.S.}$ =	610	mm	$h_{C.S.}$ =	*0.6*	m
h_d or t_{GS} =	7.2	mm	h_d or t_{GS} =	*0.01*	m
			$h_{C.S.} + h_d$ =	*0.62*	m
$k_{C.S.}$ =	1.0E-04	cm/s	$k_{C.S.}$ =	*1.0E-06*	m/s
k_d or k_{GS} =	6.3E+00	cm/s	k_d or k_{GS} =	*6.3E-02*	m/s
P =	307.34	mm/hr	P (RC) =	*30.7*	mm/hr
RC =	0.1		Actual runoff =	*303.74*	mm/hr
			PERC =	*3.60*	mm/hr
			$FLUX_{actual}$ =	*0.109*	m^3/hr
			$FLUX_{allow}$ =	*0.514*	m^3/hr
			q =	*3.0E-05*	m^3/sec
			h_{avg} =	*0.00*	m

DLC =	***4.699***
PSR	***0.002***
FS	***1.327***

* Note: if only one soil layer above GM, treat it as the drainage layer.

DLC = 4.6988

PSR = 0.002



Calculation of FS

Active Wedge:

W_A =	*298.394*	*kN*
U_n =	*0.45634*	*kN*
U_h =	*1.2E-05*	*kN*
N_A =	*282.647*	*kN*

Passive Wedge:

W_P =	*9.91064*	*kN*
U_V =	*3.5E-05*	*kN*

$$FS = \frac{-b + \sqrt{b^2 - 4ac}}{2a}$$

where a =	*89.5*
b =	*-135.2*
c =	*21.9*

FS= 1.327

thickness of cover soil = h = *0.62* m

length of slope measured along the geomembrane = L = *32* m

soil slope angle beneath the geomembrane = β = *18.4* ° = *0.32* (rad.)

vertical height of the slope measured from the toe = H = *10.1* m

parallel submergence ratio = PSR = *0.00*

depth of the water surface measured from the geomembrane = h_w = *0.00* m

dry unit weight of the protective cover = γ_{dry} =	15.6	kN/m^3	
saturated unit weight of protective cover = $\gamma_{sat'd}$ =	19.6	kN/m^3	
unit weight of water = γ_w =	9.81	kN/m^3	
friction angle of protective cover = ϕ =	30.0	°	= *0.52* (rad.)
interface friction angle between geotextile and geomembrane = δ =	23.0	°	= *0.40* (rad.)

Note: numbers in boxes are input values

numbers in Italics are calculated values

Constructed by Te-Yang Soong

NOAA Atlas 14, Volume 9, Version 2
Location name: Panama City Beach, Florida, USA*
Latitude: 30.3546°, Longitude: -85.9176°
Elevation: 58.8 ft**
* source: ESRI Maps
** source: USGS

POINT PRECIPITATION FREQUENCY ESTIMATES

Sanja Perica, Deborah Martin, Sandra Pavlovic, Ishani Roy, Michael St. Laurent, Carl Trypaluk, Dale Unruh, Michael Yekta, Geoffery Bonnin

NOAA, National Weather Service, Silver Spring, Maryland

PF_tabular | PF_graphical | Maps_&_aerials

PF tabular

PDS-based point precipitation frequency estimates with 90% confidence intervals (in inches/hour)[1]

Duration	Average recurrence interval (years)									
	1	2	5	10	25	50	100	200	500	1000
5-min	6.40 (5.29-7.74)	7.43 (6.14-9.00)	9.08 (7.48-11.0)	10.4 (8.51-12.7)	12.1 (9.52-15.2)	13.4 (10.3-17.1)	14.7 (10.8-19.2)	15.9 (11.2-21.3)	17.4 (11.8-24.1)	18.4 (12.2-26.2)
10-min	4.68 (3.87-5.66)	5.44 (4.49-6.59)	6.65 (5.47-8.08)	7.61 (6.23-9.29)	8.89 (6.97-11.1)	9.83 (7.53-12.5)	10.7 (7.93-14.0)	11.6 (8.20-15.6)	12.7 (8.62-17.6)	13.5 (8.93-19.2)
15-min	3.80 (3.15-4.60)	4.42 (3.66-5.36)	5.40 (4.45-6.57)	6.19 (5.06-7.56)	7.23 (5.67-9.04)	7.99 (6.12-10.2)	8.72 (6.44-11.4)	9.44 (6.67-12.7)	10.3 (7.00-14.3)	11.0 (7.26-15.6)
30-min	2.86 (2.37-3.46)	3.35 (2.76-4.05)	4.11 (3.38-4.99)	4.72 (3.86-5.76)	5.53 (4.33-6.91)	6.12 (4.69-7.78)	6.68 (4.94-8.73)	7.23 (5.11-9.73)	7.92 (5.36-11.0)	8.41 (5.56-11.9)
60-min	2.00 (1.65-2.42)	2.29 (1.89-2.77)	2.77 (2.28-3.37)	3.18 (2.60-3.88)	3.74 (2.95-4.72)	4.18 (3.22-5.36)	4.63 (3.43-6.08)	5.08 (3.60-6.88)	5.69 (3.86-7.93)	6.15 (4.07-8.73)
2-hr	1.28 (1.07-1.54)	1.45 (1.21-1.75)	1.75 (1.45-2.11)	2.00 (1.64-2.42)	2.36 (1.88-2.97)	2.65 (2.06-3.39)	2.96 (2.21-3.87)	3.27 (2.34-4.41)	3.71 (2.54-5.15)	4.05 (2.69-5.71)
3-hr	0.981 (0.820-1.17)	1.10 (0.918-1.32)	1.31 (1.09-1.58)	1.50 (1.24-1.82)	1.79 (1.44-2.26)	2.03 (1.59-2.59)	2.29 (1.72-3.00)	2.56 (1.84-3.46)	2.95 (2.03-4.10)	3.26 (2.18-4.58)
6-hr	0.594 (0.499-0.706)	0.665 (0.559-0.791)	0.798 (0.667-0.952)	0.923 (0.767-1.11)	1.12 (0.906-1.41)	1.28 (1.01-1.64)	1.47 (1.11-1.92)	1.67 (1.21-2.24)	1.95 (1.36-2.71)	2.19 (1.47-3.06)
12-hr	0.338 (0.286-0.399)	0.386 (0.326-0.456)	0.475 (0.399-0.562)	0.558 (0.467-0.664)	0.687 (0.562-0.863)	0.799 (0.634-1.01)	0.920 (0.703-1.20)	1.05 (0.770-1.41)	1.25 (0.872-1.72)	1.41 (0.950-1.95)
24-hr	0.193 (0.165-0.227)	0.225 (0.192-0.264)	0.284 (0.241-0.334)	0.339 (0.285-0.400)	0.423 (0.348-0.528)	0.496 (0.396-0.625)	0.574 (0.441-0.743)	0.660 (0.485-0.878)	0.784 (0.552-1.07)	0.885 (0.603-1.22)
2-day	0.112 (0.096-0.130)	0.131 (0.112-0.153)	0.167 (0.142-0.195)	0.200 (0.170-0.235)	0.252 (0.209-0.313)	0.297 (0.239-0.372)	0.345 (0.267-0.444)	0.399 (0.295-0.527)	0.475 (0.337-0.646)	0.538 (0.369-0.736)
3-day	0.081 (0.070-0.094)	0.095 (0.081-0.110)	0.120 (0.103-0.139)	0.144 (0.122-0.168)	0.181 (0.151-0.224)	0.213 (0.172-0.267)	0.249 (0.194-0.319)	0.288 (0.214-0.380)	0.345 (0.246-0.468)	0.392 (0.270-0.534)
4-day	0.065 (0.056-0.075)	0.075 (0.065-0.087)	0.094 (0.081-0.109)	0.113 (0.096-0.131)	0.142 (0.119-0.175)	0.167 (0.136-0.209)	0.195 (0.152-0.250)	0.226 (0.169-0.298)	0.271 (0.194-0.367)	0.308 (0.213-0.419)
7-day	0.042 (0.036-0.048)	0.048 (0.042-0.055)	0.060 (0.052-0.069)	0.071 (0.061-0.082)	0.089 (0.074-0.109)	0.104 (0.085-0.129)	0.120 (0.094-0.153)	0.139 (0.104-0.181)	0.165 (0.119-0.222)	0.187 (0.130-0.253)
10-day	0.033 (0.029-0.038)	0.038 (0.033-0.043)	0.046 (0.040-0.053)	0.054 (0.047-0.063)	0.067 (0.056-0.081)	0.078 (0.063-0.095)	0.089 (0.070-0.113)	0.102 (0.077-0.133)	0.121 (0.087-0.161)	0.136 (0.094-0.183)
20-day	0.022 (0.019-0.025)	0.025 (0.022-0.028)	0.030 (0.026-0.034)	0.034 (0.030-0.039)	0.041 (0.034-0.049)	0.046 (0.038-0.056)	0.052 (0.041-0.065)	0.058 (0.044-0.075)	0.067 (0.048-0.089)	0.074 (0.052-0.099)
30-day	0.018 (0.016-0.020)	0.020 (0.018-0.023)	0.024 (0.021-0.027)	0.027 (0.024-0.031)	0.032 (0.027-0.037)	0.035 (0.029-0.042)	0.039 (0.031-0.048)	0.043 (0.033-0.055)	0.049 (0.035-0.064)	0.053 (0.037-0.070)
45-day	0.015 (0.013-0.017)	0.017 (0.015-0.019)	0.019 (0.017-0.022)	0.022 (0.019-0.025)	0.025 (0.021-0.029)	0.028 (0.023-0.033)	0.030 (0.024-0.037)	0.033 (0.025-0.041)	0.036 (0.026-0.047)	0.038 (0.027-0.051)
60-day	0.013 (0.011-0.014)	0.014 (0.013-0.016)	0.017 (0.015-0.019)	0.019 (0.017-0.021)	0.022 (0.018-0.025)	0.023 (0.019-0.028)	0.025 (0.020-0.031)	0.027 (0.020-0.034)	0.029 (0.021-0.038)	0.031 (0.022-0.041)

[1] Precipitation frequency (PF) estimates in this table are based on frequency analysis of partial duration series (PDS).

Numbers in parenthesis are PF estimates at lower and upper bounds of the 90% confidence interval. The probability that precipitation frequency estimates (for a given duration and average recurrence interval) will be greater than the upper bound (or less than the lower bound) is 5%. Estimates at upper bounds are not checked against probable maximum precipitation (PMP) estimates and may be higher than currently valid PMP values.

Please refer to NOAA Atlas 14 document for more information.

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APPENDIX C
SETTLEMENT AND FOUNDATION ANALYSIS

Job No.	No.

HDR Engineering, Inc.

Project	Steelfield Road Landfill	Computed	KP	Date	9/21/2019
Subject	Bearing Capacity-Base Liner	Checked	TY	Date	9/22/2019
Task		Sheet	1	Of	2

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Problem

Determine Bearing Capacity of Subgrade due to MSW Fill based on Undrained Stress Parameters- Square Foundation

Discussion

The subgrade shall bear the weight of MSW fill material and equipment operating on the landfill footprint without causing or allowing a failure of the foundation system; and accommodate potential settlement without damage to the foundaton system. The analysis was peformed for the highest thickness of the waste/cover. The results indicated that bearing capacity of the native soil due to over burden ash fill material is adequate. The method of calculating bearing capacity is descibed below. Undrained stress parameters for clay soil is used as determined in the slope stability report.

References

Calculations

Given:

90	lb/cf	Specific weight of MSW fill material (Refer to Slope Stability Report)
187	ft	Thickness of MSW Excluding Cover Soil and Protective Cover (Refer to Figure 1 of Settlement Analysis Section)
125	lb/cf	Specific weight of cover soil/protective cover material
4	ft	Thickness of cover soil/protective cover

q_{actual} = 187 ft*90 lb/ft^3 +4 ft*125 lb/ft3

= 17,285 lb/ft^2

Using Tezargi bearing capacity equation:

$q_u = 1.3\ cN_c + qN_q + 0.4\gamma\ BN\gamma$ (square foundation)

where:

q_u = ultimate bearing capacity

c = cohesion of soil

γ = unit weight of soil

q = γ x depth of foundation

$N_c, N_q, N\gamma$ = bearing capacity factors that are nondimensional and functions of the internal friction angle, *ø*

For Foundation Soil

γ	120	lb/ft^3
Φ	0	degrees
c	927	psf (Conservative assumption used in slope stability)
Nc	5.7	"Principles of Geotechnical Engineering", Das, 2002 (Foundations on Layered Clay)
Nγ	0.0	
Nq	1.0	

Job No.	No.

HDR Engineering, Inc.

Project	Steelfield Road Landfill	Computed	KP	Date	9/21/2019
Subject	Bearing Capacity-Base Liner	Checked	TY	Date	9/22/2019
Task		Sheet	1	Of	2

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Use B = 1 foot

Use maximum MSW/cover height as the surcharge. This means same height is applied to either side of 1 foot strip selected for analysis.This is applicable when there is no sudden variation in overburden MSW thickness.

q_u 24,154 psf

$$FS = \frac{q_u}{q_{actual}} = 1.40$$

Discussion

As shown above, the bearing capacity is adequate for the anticipated load considering short term conditions. However, for an embankment-type loading, such as a landfill, the most appropriate analysis to assess bearing capacity is conducted through a slope stability analysis that allow for a deep-seated failure surface, thereby determining the true bearing capacity failure mechanism. This failure is considered in the slope stability analysis.

Settlement Analysis

Introduction 2
Settlement Computation 2
Results and Conclusions 3

Attachments

C-1 Calculations for Settlement Analysis

C-2 Selected Locations for Settlement Analysis

C-3 Supporting Data

Introduction

The base of the proposed Cells 6 and 7 of Steelfield Road Landfill is graded at a minimum 2% slope toward the leachate collection trench. The cell floor will consist of a geocomposite to facilitate leachate flow to the proposed lateral drainage system which will consist of coarse gravel around a perforated pipe at 1.0% designed slope. The leachate collection pipe conveys leachate to the leachate collection sump for removal outside of the cell footprint. The same concept is followed in previously constructed cells.

The purpose of the settlement analysis is to confirm that minimum 1% slope of baseliner and 0.3% slope of leachate collection pipe is maintained after settlement per 62-701.400(4)(c). By reviewing the proposed landfill basegrades and final cover, several locations were selected for settlement calculations. The locations were selected within the existing landfill footprint that will be impacted from the vertical expansion.

Settlement Computation

The selected locations for settlement analysis shown in Attachment C-2 are based on maximum change in overburden stress within a given distance while considering following:

- Location of leachate collection pipes to determine post settlement pipe slopes
- Worst case proposed liner slope reduction
- Worst case overliner slope reduction
- Worst case existing liner slope reduction

The Table 1 summarizes the purpose of specific locations selected for analysis.

Table 1 Select Locations for Settlement Computations

Location	Purpose
A	Compute basegrade slope after settlement in Cells 1 & 2 waste.
B	
C	Compute slope after settlement at Cell 3 leachate collection pipe.
D	
E	Compute basegrade slope after settlement at Cell 5
F	
G	Compute basegrade slope after settlement over Cell 6
H	
I	Compute slope after settlement at Cell 6 leachate collection pipe.
J	

In general, settlement increases as thickness of the MSW increases since the subsurface at project site is generally uniform or without any drastic changes.

The settlement at selected locations is calculated considering elastic settlement, primary consolidation settlement and secondary consolidation settlement. The settlement calculation methodology, results and assumptions are included in Attachment C-1. The determination of respective settlement parameters for calculations are discussed in the calculation package. The settlement parameters for fine grained soils are based on site specific consolidation tests performed. The support data for settlement calculations and references are provided in

Attachment C-3. Note that no primary consolidation settlement assumed when the over burden load is less that preconsolidation pressure.

The computations associated with settlement impact is provided in the calculation summary table.

Results and Conclusions

Based on the settlement calculations performed following are the main conclusions:

- The minimum leachate pipe slope after settlement within existing landfill is 0.49% at Cell 3.
- The minimum existing base liner slope after settlement is 1.26% at Cell 5.
- The minimum leachate pipe slope after settlement within proposed landfill is 0.36% at Cell 6.
- The minimum proposed base liner slope after settlement is 0.46% at Cell 7.
- The minimum base liner slope over Cells 1 and 2 is 1.54% at Cell 7.

Attachment C-1

Calculations for Settlement Analysis

DETERMINATION OF POST SETTLEMENT GRADES

STEELFIELD CLASS I LANDFILL

		Total Settlement (ft)
Settlement Location	A	0.70
Settlement Location	B	1.16
Settlement Location	C	3.60
Settlement Location	D	4.11
Settlement Location	E	2.21
Settlement Location	F	2.95
Settlement Location	G	0.57
Settlement Location	H	1.03
Settlement Location	I	0.42
Settlement Location	J	1.06

Slope Increment	Horizontal Increment Length (ft)	Original Vertical Change (ft)	Initial Slope Inclination (percent)	Increment Slope Length (ft)	Differential Settlement (ft)	Final Slope Inclination (percent)	New Increm. Slope Length (ft)	Change In Slope Length (ft)	Tensile Strain (+ = Tension) (percent)
A to B	100	2	2.00	100.020	0.460	1.54	100.012	-0.008	-0.008
C to D	100	1	1.00	100.005	0.510	0.49	100.001	-0.004	-0.004
E to F	100	2	2.00	100.020	0.740	1.26	100.008	-0.012	-0.012
G to H	100	2	2.00	100.020	0.460	1.54	100.012	-0.008	-0.008
I to J	100	1	1.00	100.005	0.640	0.36	100.001	-0.004	-0.004

HDR Computation

Project:	Steelfield Road Landfill	Computed By:	KP	Date:	1/25/2019
Subject:	Cells 6 & 7 Expansion	Checked By:	TY	Date:	2/9/2019
Task:	Settlement Calculation - Total Settlement	Sheet:		Of:	

Objective:

Estimate the settlement that will occur under the proposed Landfill when the MSW has reached its final grades.

Background:

As part of the hydrogeological investigation for the site, several borings and testing programs were conducted in 1986, 2001 and 2010 to characterize subsurface conditions. The boring locations are shown in Drawing 00C-01. Foundation settlement was calculated at locations shown in Attachment C-2 . The elastic settlement was based on correlations with soil type and the estimated vertical stress on the soil. It should be noted that settlement computations are based on change in stress at the surface applies to all subsurface layers without considering an influence factor to reduce applied stress with increase in depth. This is a conservative assumption that results in an increase in overall settlement.

References:

1. Naval Facilities Engineering Command (NAVFAC) (1986). *Design Manual 7.01 - Soil Mechanics* .
2. Histrotrical data collected from previous investigations.
3. Qian, X., Koerner, R. M., & Gray, D. H. (). *Geotechnical Aspects of Landfill Design and Construction* . Upper Saddle River, New Jersey: Prentice Hall.

Steps - Primary Consolidation (Clay Soils)

1. Estimate the total settlement (Z_T) for each layer using the elastic settlement (Z_e), primary consolidation settlement (Z_c), secondary consolidation settlement (Z_α) and equation 12.24 (page 470) below from Reference 3.

$$Z_T = Z_e + Z_c + Z_\alpha$$

2. Repeat Step 1 for each soil layer. Add the total settlement for each layer together to get the total estimated settlement at that location based on information from the boring logs, recent surveys, and seasonal high groundwater levels.

$$Z_{T\,Total} = Z_{T\,layer\,n} + Z_{T\,layer\,n+1}$$

HDR Computation

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Calculations

A

Layer	Boring Log Description	From Previous Calculations			Step 1
		Elastic Layer Settlement, Z_e (ft)	Primary Consolidation Settlement of Clay, Z_c (ft)	Secondary Compression Settlement of Caly Layer, Z_α (ft)	Total Settlement, Z_T (ft)
1	Sand - Fine, Medium, Tan, Brown, White	0.15	0.00	0.00	0.15
2	Clayey Sand	0.16	0.05	0.02	0.23
3	Sand - Coarse-Fine	0.09	0.00	0.00	0.09
4	Clayey Sand	0.10	0.09	0.04	0.23

Step 2: Total Estimated Settlement for A 0.7 ft

B

Layer	Boring Log Description	From Previous Calculations			Step 1
		Elastic Layer Settlement, Z_e (ft)	Primary Consolidation Settlement of Clay, Z_c (ft)	Secondary Compression Settlement of Caly Layer, Z_α (ft)	Total Settlement, Z_T (ft)
1	Sand - Fine, Medium, Tan, Brown, White	0.23	0.00	0.00	0.23
2	Clayey Sand	0.24	0.23	0.02	0.49
3	Sand - Coarse-Fine	0.12	0.00	0.00	0.12
4	Clayey Sand	0.14	0.14	0.04	0.32

Step 2: Total Estimated Settlement for B 1.16 ft

C

Layer	Boring Log Description	From Previous Calculations			Step 1
		Elastic Layer Settlement, Z_e (ft)	Primary Consolidation Settlement of Clay, Z_c (ft)	Secondary Compression Settlement of Caly Layer, Z_α (ft)	Total Settlement, Z_T (ft)
1	Sand - Fine, Medium, Tan, Brown, White	0.91	0.00	0.00	0.91
2	Clayey Sand	1.20	0.60	0.02	1.82
3	Sand - Coarse-Fine	0.46	0.00	0.00	0.46
4	Silty Fine Sand with Shell Fragments	0.41	0.00	0.00	0.41

Step 2: Total Estimated Settlement for C 3.6 ft

HDR Computation

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D

Layer	Boring Log Description	From Previous Calculations			Step 1
		Elastic Layer Settlement, Z_e (ft)	Primary Consolidation Settlement of Clay, Z_c (ft)	Secondary Compression Settlement of Caly Layer, Z_α (ft)	Total Settlement, Z_T (ft)
1	Sand - Fine, Medium, Tan, Brown, White	1.16	0.00	0.00	1.16
2	Clayey Sand	1.23	0.66	0.02	1.91
3	Sand - Coarse-Fine	0.55	0.00	0.00	0.55
4	Silty Fine Sand with Shell Fragments	0.49	0.00	0.00	0.49
5					

Step 2: Total Estimated Settlement for D 4.11 ft

E

Layer	Boring Log Description	From Previous Calculations			Step 1
		Elastic Layer Settlement, Z_e (ft)	Primary Consolidation Settlement of Clay, Z_c (ft)	Secondary Compression Settlement of Caly Layer, Z_α (ft)	Total Settlement, Z_T (ft)
1	Sand - Fine, Medium, Tan, Brown, White	0.67	0.00	0.00	0.67
2	Clayey Sand	0.70	0.23	0.02	0.95
3	Sand - Coarse-Fine	0.28	0.00	0.00	0.28
4	Clayey Sand	0.18	0.10	0.03	0.31
5	Silty Fine Sand with Clary Fragments	0.29	0.00	0.00	0.29

Step 2: Total Estimated Settlement for E 2.21 ft

HDR Computation

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Calculations

F

Layer	Boring Log Description	From Previous Calculations			Step 1
		Elastic Layer Settlement, Z_e (ft)	Primary Consolidation Settlement of Clay, Z_c (ft)	Secondary Compression Settlement of Caly Layer, Z_α (ft)	Total Settlement, Z_T (ft)
1	Sand - Fine, Medium, Tan, Brown, White	0.80	0.00	0.00	0.80
2	Clayey Sand	0.72	0.32	0.02	1.06
3	Sand - Coarse-Fine	0.34	0.00	0.00	0.34
4	Clayey Sand	0.23	0.12	0.03	0.38
5	Silty Fine Sand with Shell Fragments	0.37	0.00	0.00	0.37
				Step 2: Total Estimated Settlement for F	2.95 ft

G

Layer	Boring Log Description	From Previous Calculations			Step 1
		Elastic Layer Settlement, Z_e (ft)	Primary Consolidation Settlement of Clay, Z_c (ft)	Secondary Compression Settlement of Caly Layer, Z_α (ft)	Total Settlement, Z_T (ft)
1	Sand - Fine, Medium, Tan, Brown, White	0.10	0.00	0.00	0.10
2	Clayey Sand	0.18	0.00	0.02	0.20
3	Sand - Coarse-Fine	0.06	0.00	0.00	0.06
4	Clayey Sand	0.09	0.00	0.04	0.13
5	Silty Fine Sand with Shell Fragments	0.08	0.00	0.00	0.08
				Step 2: Total Estimated Settlement for G	0.57 ft

H

Layer	Boring Log Description	From Previous Calculations			Step 1
		Elastic Layer Settlement, Z_e (ft)	Primary Consolidation Settlement of Clay, Z_c (ft)	Secondary Compression Settlement of Caly Layer, Z_α (ft)	Total Settlement, Z_T (ft)
1	Sand - Fine, Medium, Tan, Brown, White	0.21	0.00	0.00	0.21
2	Clayey Sand	0.34	0.00	0.02	0.36
3	Sand - Coarse-Fine	0.12	0.00	0.00	0.12
4	Clayey Sand	0.16	0.00	0.04	0.20
5	Silty Fine Sand with Shell Fragments	0.14	0.00	0.00	0.14
				Step 2: Total Estimated Settlement for H	1.03 ft

HDR Computation

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I

Layer	Boring Log Description	From Previous Calculations			Step 1
		Elastic Layer Settlement, Z_e (ft)	Primary Consolidation Settlement of Clay, Z_c (ft)	Secondary Compression Settlement of Caly Layer, Z_α (ft)	Total Settlement, Z_T (ft)
1	Sand - Fine, Medium, Tan, Brown, White	0.10	0.00	0.00	0.10
2	Clayey Sand	0.16	0.00	0.03	0.19
3	Sand - Coarse-Fine	0.07	0.00	0.00	0.07
4	Clayey Sand	0.06	0.00	0.00	0.06
5					

Step 2: Total Estimated Settlement for I 0.42 ft

J

Layer	Boring Log Description	From Previous Calculations			Step 1
		Elastic Layer Settlement, Z_e (ft)	Primary Consolidation Settlement of Clay, Z_c (ft)	Secondary Compression Settlement of Caly Layer, Z_α (ft)	Total Settlement, Z_T (ft)
1	Sand - Fine, Medium, Tan, Brown, White	0.28	0.00	0.00	0.28
2	Clayey Sand	0.41	0.00	0.03	0.44
3	Sand - Coarse-Fine	0.17	0.00	0.00	0.17
4	Clayey Sand	0.17	0.00	0.00	0.17

Step 2: Total Estimated Settlement for J 1.06 ft

Computation

Project:	Steelfield Road Landfill	Computed By:	KP	Date:	1/25/2019
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Steps - Elastic Settlement

1. Determine the soil type and height of layer (H_o) based on the boring logs and groundwater levels. Top of liner and waste/soil thinkness at 100-foot grid are provided in Attachments 1. The select locations for settlement is based on worst case scenarios identified based on 100-foot grid that could impact slope of cell floor or leachate collection pipe.

2. Determine the wet unit weight (γ) based on the soil type and are consistent with slope stability analysis report.

3. Determine the vertical stress (σ_{soil}) of the soil by multiplying the height of the soil by the unit weight from step 2 and subtracting the buoyant force due to the water if the layer is below the water table. If water is not present in the layer, set γ_{water} equal to zero. It is conservatively assumed that all subsurface layers below liner is saturated.

$$\sigma_{soil} = \gamma_{soil} H_o - \gamma_{water} H_o$$

4. Determine the vertical stress ($\sigma_{m\ soil}$) of the soil at the midpoint of the soil layer.

$$\sigma_{m\ soil} = \frac{\sigma_{soil}}{2}$$

5. Determine the increment of vertical effective stress of the waste & cover on the soil ($\Delta\sigma_{waste\ \&\ cover}$) assuming the unit weight of MSW is 90 pcf (from Cell 5 ash test report and adjusting for raw MSW), the height (Hmsw) varies across the site as shown in Attachment 1, unit weight of the final cover, intermediate cover and protective cover (γ_{cover}) is estimated to be 120 pcf and the height of the soil cover(H_{cover}) is 5 feet including final cover thickness, intermediate cover and protective cover.

$$\Delta\sigma_{ash\ \&\ cover} = \gamma_{ash} H_{ash} + \gamma_{cover} H_{cover}$$

6. Determine the total vertical stress due to soil (σ_T) at the midpoint of the soil considering the soil ($\sigma_{m\ soil}$) and soil layers above ($\sigma_{soil\ layers\ above}$).

$$\sigma_T = \sigma_{m\ soil} + \sigma_{soil\ layers\ above}$$

Computation

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7. Convert the total vertical stress (σ_T) from psf to psi.

$$\sigma_{T\ psi} = \sigma_T \times \frac{1\ ft^2}{144\ in^2}$$

8. Using the total vertical stress (σ_T) in psi and soil type, look up the elastic modulus of soil (E_s) and Poisson's ratio of soil (v_s) in the Columns in Table 9.5 (page 310) of Reference 3 (Attachment 2). Note, considering average SPT N value, 95% density column was used for all soil types below upper sandy clay layer. E_s and v_s for upper layers were based on 85% compaction rate .

9. Convert the elastic modulus of soil (E_S) from psi to psf.

$$E_{S\ psf} = E_S \times \frac{144\ in^2}{1\ ft^2}$$

10. Determine the constrained modulus of soil (M_S) using the elastic modulus of soil (E_s) and Poisson's ratio of soil (v_s) and Equation 12.21 (page 470) of Reference 3.

$$M_s = \frac{E_s(1 - v_s)}{(1 + v_s)(1 - 2v_s)}$$

11. Determine the elastic settlement of soil layer (Ze) using the initial thickness of the soil layer (Ho), the increment of vertical effective stress (Δσ) and the constrained modulus of soil (Ms) and Equation 12.20 (page 469) of Reference 3.

$$Z_e = \left(\frac{\Delta\sigma}{M_s}\right) H_o$$

12. Repeat Steps 1 - 11 for each layer. Add the elastic settlement for each layer together to get the total estimated elastic settlement at that location based on information from the boring logs, recent surveys, and seasonal high groundwater levels.

$$Z_{e\ Total} = Z_{e\ layer\ n} + Z_{e\ layer\ n+1}$$

Computation

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Calculations

A

Top of Final Cover Elevation	141	ft
Top of Liner Elevation	88.4	ft
Natural Ground Elevation	90.90	ft
Depth of Excavation to Top of Liner	2.5	ft
Elevation of Partially Weathered Rock (PWR)	-102.00	ft
Groundwater Elevation	44	ft
Height of MSW & Cover	52.6	ft
Cover Thickness+Protective Cover	5	ft
MSW Thickness	47.6	ft
Unit Weight of MSW	90	pcf
Estimated Unit Weight of Cover	120	pcf
Unit Weight of Soil Excavated	116	pcf
Vertical Stress Induced by Exavated Material	290	psf
Net Vertical Stress of MSW & Cover	4,594	psf
Water Density	62.4	pcf

Layer	Boring Log Description	Step 1		Step 2	Step 3	
		Soil Symbol	H_o (ft)	Unit Weight of Soil, γ_{soil} (pcf)	Unit Weight of Water, γ_{water} (pcf)	Vertical stress of Soil, σ_{soil} (psf)
1	MSW	WTE Ash	35.9	90	0	3231
2	Sand - Fine, Medium, Tan, Brown, White	SP, SW, SM	20.20	116.0	62.4	1,083
3	Clayey Sand	CL, SC	4.50	120.0	62.4	259
4	Sand - Coarse-Fine	SM	42.2	116.0	62.4	2,262
5	Clayey Sand	SC, CL	7.9	120.0	62.4	455
6	Silty Fine Sand with Shell Fragments	SM	34.1	116.0	62.4	1,828

A continued

Layer	Boring Log Description	Step 4	Step 5 (see calculation above)	Step 6	Step 7
		Vertical Stress at midpoint of Soil, $\sigma_{m\ soil}$ (psf)	Increment of Vertical Effective Stress of Ash & Cover, $\Delta\sigma ash_{\ \&\ cover}$ (psf)	Total vertical stress due to soil, σ_T (psf)	Total vertical stress on soil, $\sigma_{T\ psi}$ (psi)
2	MSW	1616	4,594	1616	11
2	Sand - Fine, Medium, Tan, Brown, White	542	4,594	3,774	26
3	Clayey Sand	130	4,594	4,446	31
4	Sand - Coarse-Fine	1,131	4,594	5,707	40
5	Clayey Sand	228	4,594	7,066	49
6	Silty Fine Sand with Shell Fragments	914	4,594	8,208	57

A continued

Layer	Boring Log Description	Step 8		Step 9	Step 10	Step 11
		Es (psi)	ν_s	E_s (psf)	M_s (psf)	Z_e (ft)
1	MSW	2600	0.19	374400	411038	0.23
2	Sand - Fine, Medium, Tan, Brown, White	3,700	0.21	532,800	599,761	0.15
3	Clayey Sand	650	0.30	93,600	126,000	0.16
4	Sand - Coarse-Fine	13,000	0.25	1,872,000	2,246,400	0.09
5	Clayey Sand	1,450	0.37	208,800	359,756	0.10
6	Silty Fine Sand with Shell Fragments	16,000	0.29	2,304,000	3,019,269	0.05

Step 12: Total Estimated Elastic Settlement for A 0.55 ft

Computation

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B

Top of Final Cover Elevation	162.9	ft
Top of Liner Elevation	90.7	ft
Natural Ground Elevation	91.1	ft
Depth of Excavation to Top of Liner	0.4	ft
Elevation of Partially Weathered Rock (PWR)	-102.00	ft
Groundwater Elevation	44.00	ft
Height of MSW & Cover	72.2	ft
Cover Thickness+Protective Cover	5	ft
MSW Thickness	67.2	ft
Unit Weight of MSW	90	pcf
Estimated Unit Weight of Cover	120	pcf
Unit Weight of Soil Excavated	116	pcf
Vertical Stress Induced by Exavated Material	46	psf
Net Vertical Stress of MSW & Cover	6,602	psf
Water Density	62.4	pcf

Layer	Boring Log Description	Step 1		Step 2	Step 3	
		Soil Symbol	H_o (ft)	Unit Weight of Soil, γ_{soil} (pcf)	Unit Weight of Water, γ_{water} (pcf)	Vertical stress of Soil, σ_{soil} (psf)
1	MSW	WTE Ash	38.2	90	0	3438
2	Sand - Fine, Medium, Tan, Brown, White	SP, SW, SM	20.9	116.0	62.4	1,120
3	Clayey Sand	CL, SC	4.6	120.0	62.4	265
4	Sand - Coarse-Fine	SM	41.7	116.0	62.4	2,235
5	Clayey Sand	SC, CL	7.8	120.0	62.4	449
6	Silty Fine Sand with Shell Fragments	SM	33.9	116.0	62.4	1,817

B continued

Layer	Boring Log Description	Step 4	Step 5 (see calculation above)	Step 6	Step 7
		Vertical Stress at midpoint of Soil, $\sigma_{m\ soil}$ (psf)	Increment of Vertical Effective Stress of Ash & Cover, $\Delta\sigma ash_{\ \&\ cover}$ (psf)	Total vertical stress due to soil, σ_T (psf)	Total vertical stress on soil, $\sigma_{T\ psi}$ (psi)
1	MSW	1719	6,602	1719	12
2	Sand - Fine, Medium, Tan, Brown, White	560	6,602	3,998	28
3	Clayey Sand	133	6,602	4,691	33
4	Sand - Coarse-Fine	1,118	6,602	5,942	41
5	Clayey Sand	225	6,602	7,285	51
6	Silty Fine Sand with Shell Fragments	909	6,602	8,419	58

B continued

Layer	Boring Log Description	Step 8		Step 9	Step 10	Step 11
		Es (psi)	ν_s	E_s (psf)	M_s (psf)	Z_e (ft)
1	MSW	2600	0.19			
2	Sand - Fine, Medium, Tan, Brown, White	3,700	0.21	532,800	599,761	0.23
3	Clayey Sand	650	0.30	93,600	126,000	0.24
4	Sand - Coarse-Fine	13,000	0.25	1,872,000	2,246,400	0.12
5	Clayey Sand	1,450	0.37	208,800	359,756	0.14
6	Silty Fine Sand with Shell Fragments	16,000	0.29	2,304,000	3,019,269	0.07

Step 12: Total Estimated Elastic Settlement for B 0.8 ft

Computation

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C

Top of Final Cover Elevation	213.8	ft
Top of Liner Elevation	54.7	ft
Natural Ground Elevation	71	ft
Depth of Excavation to Top of Liner	16.3	ft
Elevation of Partially Weathered Rock (PWR)	-102.00	ft
Groundwater Elevation	44.00	ft
Height of MSW & Cover	159.1	ft
Cover Thickness+Protective Cover	5	ft
MSW Thickness	154.1	ft
Unit Weight of MSW	90	pcf
Estimated Unit Weight of Cover	120	pcf
Unit Weight of Soil Excavated	116	pcf
Vertical Stress Induced by Exavated Material	1891	psf
Net Vertical Stress of MSW & Cover	12,578	psf
Water Density	62.4	pcf

Layer	Boring Log Description	Step 1		Step 2	Step 3	
		Soil Symbol	H_o (ft)	Unit Weight of Soil, γ_{soil} (pcf)	Unit Weight of Water, γ_{water} (pcf)	Vertical stress of Soil, σ_{soil} (psf)
1	Sand - Fine, Medium, Tan, Brown, White	SP, SW, SM	24.7	116.0	62.4	1,324
2	Clayey Sand	CL, SC	7	120.0	62.4	403
3	Sand - Coarse-Fine	SM	52	116.0	62.4	2,787
4	Silty Fine Sand with Shell Fragments	SM	73	116.0	62.4	3,913

C continued

Layer	Boring Log Description	Step 4	Step 5 (see calculation above)	Step 6	Step 7
		Vertical Stress at midpoint of Soil, $\sigma_{m\ soil}$ (psf)	Increment of Vertical Effective Stress of Ash & Cover, $\Delta\sigma ash_{\ \&\ cover}$ (psf)	Total vertical stress due to soil, σ_T (psf)	Total vertical stress on soil, $\sigma_{T\ psi}$ (psi)
1	Sand - Fine, Medium, Tan, Brown, White	662	12,578	662	5
2	Clayey Sand	202	12,578	1,526	11
3	Sand - Coarse-Fine	1,394	12,578	3,122	22
4	Silty Fine Sand with Shell Fragments	1,957	12,578	6,473	45

C continued

Layer	Boring Log Description	Step 8		Step 9	Step 10	Step 11
		Es (psi)	ν_s	E_s (psf)	M_s (psf)	Z_e (ft)
1	Sand - Fine, Medium, Tan, Brown, White	2,100	0.21	302,400	340,405	0.91
2	Clayey Sand	400	0.28	57,600	73,636	1.20
3	Sand - Coarse-Fine	8,600	0.23	1,238,400	1,435,664	0.46
4	Silty Fine Sand with Shell Fragments	13,000	0.25	1,872,000	2,246,400	0.41

Step 12: Total Estimated Elastic Settlement for C 2.98 ft

Computation

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D

Top of Final Cover Elevation	242.6	ft
Top of Liner Elevation	55.2	ft
Natural Ground Elevation	72	ft
Depth of Excavation to Top of Liner	16.8	ft
Elevation of Partially Weathered Rock (PWR)	-102.00	ft
Groundwater Elevation	44.00	ft
Height of MSW & Cover	187.4	ft
Cover Thickness+Protective Cover	5	ft
MSW Thickness	182.4	ft
Unit Weight of MSW	90	pcf
Estimated Unit Weight of Cover	120	pcf
Unit Weight of Soil Excavated	116	pcf
Vertical Stress Induced by Exavated Material	1949	psf
Net Vertical Stress of MSW & Cover	15,067	psf
Water Density	62.4	pcf

Layer	Boring Log Description	Step 1		Step 2	Step 3	
		Soil Symbol	H_o (ft)	Unit Weight of Soil, γ_{soil} (pcf)	Unit Weight of Water, γ_{water} (pcf)	Vertical stress of Soil, σ_{soil} (psf)
1	Sand - Fine, Medium, Tan, Brown, White	SP, SW, SM	26.2	116.0	62.4	1,404
2	Clayey Sand	CL, SC	6	120.0	62.4	346
3	Sand - Coarse-Fine	SM	52	116.0	62.4	2,787
4	Silty Fine Sand with Shell Fragments	SM	73	116.0	62.4	3,913

D continued

Layer	Boring Log Description	Step 4	Step 5 (see calculation above)	Step 6	Step 7
		Vertical Stress at midpoint of Soil, $\sigma_{m\ soil}$ (psf)	Increment of Vertical Effective Stress of Ash & Cover, $\Delta\sigma ash_{\ \&\ cover}$ (psf)	Total vertical stress due to soil, σ_T (psf)	Total vertical stress on soil, $\sigma_{T\ psi}$ (psi)
1	Sand - Fine, Medium, Tan, Brown, White	702	15,067	702	5
2	Clayey Sand	173	15,067	1,577	11
3	Sand - Coarse-Fine	1,394	15,067	3,144	22
4	Silty Fine Sand with Shell Fragments	1,957	15,067	6,495	45

D continued

Layer	Boring Log Description	Step 8		Step 9	Step 10	Step 11
		Es (psi)	ν_s	E_s (psf)	M_s (psf)	Z_e (ft)
1	Sand - Fine, Medium, Tan, Brown, White	2,100	0.21	302,400	340,405	1.16
2	Clayey Sand	400	0.28	57,600	73,636	1.23
3	Sand - Coarse-Fine	8,600	0.23	1,238,400	1,435,664	0.55
4	Silty Fine Sand with Shell Fragments	13,000	0.25	1,872,000	2,246,400	0.49

Step 12: Total Estimated Elastic Settlement for D 3.43 ft

Computation

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E

Top of Final Cover Elevation	160.8	ft
Top of Liner Elevation	59.3	ft
Natural Ground Elevation	65	ft
Depth of Excavation to Top of Liner	5.7	ft
Elevation of Partially Weathered Rock (PWR)	-102.00	ft
Groundwater Elevation	52.00	ft
Height of MSW & Cover	101.5	ft
Cover Thickness+Protective Cover	5	ft
MSW Thickness	96.5	ft
Unit Weight of MSW	90	pcf
Estimated Unit Weight of Cover	120	pcf
Unit Weight of Soil Excavated	116	pcf
Vertical Stress Induced by Exavated Material	661	
Net Vertical Stress of MSW & Cover	8,624	psf
Water Density	62.4	pcf

Layer	Boring Log Description	Step 1		Step 2	Step 3	
		Soil Symbol	H_o (ft)	Unit Weight of Soil, γ_{soil} (pcf)	Unit Weight of Water, γ_{water} (pcf)	Vertical stress of Soil, σ_{soil} (psf)
1	Sand - Fine, Medium, Tan, Brown, White	SP, SW, SM	26.30	116.0	62.4	1,410
2	Clayey Sand	CL, SC	6.00	120.0	62.4	346
3	Sand - Coarse-Fine	SM	47	116.0	62.4	2,519
4	Clayey Sand	SC, CL	6	120.0	62.4	346
5	Silty Fine Sand with Clary Fragments	SM	76	116.0	62.4	4,074

E continued

Layer	Boring Log Description	Step 4	Step 5 (see calculation above)	Step 6	Step 7
		Vertical Stress at midpoint of Soil, $\sigma_{m\ soil}$ (psf)	Increment of Vertical Effective Stress of Ash & Cover, $\Delta\sigma ash_{\ \&\ cover}$ (psf)	Total vertical stress due to soil, σ_T (psf)	Total vertical stress on soil, $\sigma_{T\ psi}$ (psi)
1	Sand - Fine, Medium, Tan, Brown, White	705	8,624	705	5
2	Clayey Sand	173	8,624	1,583	11
3	Sand - Coarse-Fine	1,260	8,624	3,016	21
4	Clayey Sand	173	8,624	4,449	31
5	Silty Fine Sand with Clary Fragments	2,037	8,624	6,659	46

E continued

Layer	Boring Log Description	Step 8		Step 9	Step 10	Step 11
		Es (psi)	v_s	E_s (psf)	M_s (psf)	Z_e (ft)
1	Sand - Fine, Medium, Tan, Brown, White	2,100	0.21	302,400	340,405	0.67
2	Clayey Sand	400	0.28	57,600	73,636	0.70
3	Sand - Coarse-Fine	8,600	0.23	1,238,400	1,435,664	0.28
4	Clayey Sand	1,350	0.325	194,400	282,954	0.18
5	Silty Fine Sand with Clary Fragments	13,000	0.25	1,872,000	2,246,400	0.29

Step 12: Total Estimated Elastic Settlement for E 2.12 ft

Computation

Project:	Steelfield Road Landfill	Computed By:	KP	Date:	1/25/2019
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Calculations

F

Top of Final Cover Elevation	186.4	ft
Top of Liner Elevation	57.6	ft
Natural Ground Elevation	67.00	ft
Depth of Excavation to Top of Liner	9.4	ft
Elevation of Partially Weathered Rock (PWR)	-102.00	ft
Groundwater Elevation	50	ft
Height of MSW & Cover	128.8	ft
Cover Thickness+Protective Cover	5	ft
MSW Thickness	123.8	ft
Unit Weight of MSW	90	pcf
Estimated Unit Weight of Cover	120	pcf
Unit Weight of Soil Excavated	116	pcf
Vertical Stress Induced by Exavated Material	1090	psf
Net Vertical Stress of MSW & Cover	10,652	psf
Water Density	62.4	pcf

Layer	Boring Log Description	Step 1		Step 2	Step 3	
		Soil Symbol	H_o (ft)	Unit Weight of Soil, γ_{soil} (pcf)	Unit Weight of Water, γ_{water} (pcf)	Vertical stress of Soil, σ_{soil} (psf)
1	Sand - Fine, Medium, Tan, Brown, White	SP, SW, SM	25.60	116.0	62.4	1,372
2	Clayey Sand	CL, SC	5.00	120.0	62.4	288
3	Sand - Coarse-Fine	SM	46	116.0	62.4	2,466
4	Clayey Sand	SC, CL	6	120.0	62.4	346
5	Silty Fine Sand with Shell Fragments	SM	77	116.0	62.4	4,127

F continued

Layer	Boring Log Description	Step 4	Step 5 (see calculation above)	Step 6	Step 7
		Vertical Stress at midpoint of Soil, $\sigma_{m\ soil}$ (psf)	Increment of Vertical Effective Stress of Ash & Cover, $\Delta\sigma ash_{\ \&\ cover}$ (psf)	Total vertical stress due to soil, σ_T (psf)	Total vertical stress on soil, $\sigma_{T\ psi}$ (psi)
1	Sand - Fine, Medium, Tan, Brown, White	686	10,652	686	5
2	Clayey Sand	144	10,652	1,516	11
3	Sand - Coarse-Fine	1,233	10,652	2,893	20
4	Clayey Sand	173	10,652	4,299	30
5	Silty Fine Sand with Shell Fragments	2,064	10,652	6,536	45

F continued

Layer	Boring Log Description	Step 8		Step 9	Step 10	Step 11
		Es (psi)	v_s	E_s (psf)	M_s (psf)	Z_e (ft)
1	Sand - Fine, Medium, Tan, Brown, White	2,100	0.21	302,400	340,405	0.80
2	Clayey Sand	400	0.28	57,600	73,636	0.72
3	Sand - Coarse-Fine	8,600	0.23	1,238,400	1,435,664	0.34
4	Clayey Sand	1,350	0.33	194,400	282,954	0.23
5	Silty Fine Sand with Shell Fragments	13,000	0.25	1,872,000	2,246,400	0.37

Step 12: Total Estimated Elastic Settlement for F 2.46 ft

Computation

Project:	Steelfield Road Landfill	Computed By:	KP	Date:	1/25/2019
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G

Top of Final Cover Elevation	86.2	ft
Top of Liner Elevation	51.7	ft
Natural Ground Elevation	61.6	ft
Depth of Excavation to Top of Liner	9.9	ft
Elevation of Partially Weathered Rock (PWR)	-102.00	ft
Groundwater Elevation	44.00	ft
Height of MSW & Cover	34.5	ft
Cover Thickness+Protective Cover	5	ft
MSW Thickness	29.5	ft
Unit Weight of MSW	90	pcf
Estimated Unit Weight of Cover	120	pcf
Unit Weight of Soil Excavated	116	pcf
Vertical Stress Induced by Exavated Material	1148	psf
Net Vertical Stress of MSW & Cover	2,107	psf
Water Density	62.4	pcf

Layer	Boring Log Description	Step 1		Step 2	Step 3	
		Soil Symbol	H_o (ft)	Unit Weight of Soil, γ_{soil} (pcf)	Unit Weight of Water, γ_{water} (pcf)	Vertical stress of Soil, σ_{soil} (psf)
1	Sand - Fine, Medium, Tan, Brown, White	SP, SW, SM	16.7	116.0	62.4	895
2	Clayey Sand	CL, SC	4	120.0	62.4	230
3	Sand - Coarse-Fine	SM	44	116.0	62.4	2,358
4	Clayey Sand	CL, SC	9	120.0	62.4	518
5	Silty Fine Sand with Shell Fragments	SM	81	116.0	62.4	4,342

G continued

Layer	Boring Log Description	Step 4	Step 5 (see calculation above)	Step 6	Step 7
		Vertical Stress at midpoint of Soil, $\sigma_{m\ soil}$ (psf)	Increment of Vertical Effective Stress of Ash & Cover, $\Delta\sigma ash_{\ \&\ cover}$ (psf)	Total vertical stress due to soil, σ_T (psf)	Total vertical stress on soil, $\sigma_{T\ psi}$ (psi)
1	Sand - Fine, Medium, Tan, Brown, White	448	2,107	448	3
2	Clayey Sand	115	2,107	1,011	7
3	Sand - Coarse-Fine	1,179	2,107	2,305	16
4	Clayey Sand	259	2,107	3,743	26
5	Silty Fine Sand with Shell Fragments	2,171	2,107	6,173	43

G continued

Layer	Boring Log Description	Step 8		Step 9	Step 10	Step 11
		Es (psi)	ν_s	E_s (psf)	M_s (psf)	Z_e (ft)
1	Sand - Fine, Medium, Tan, Brown, White	2,100	0.21	302,400	340,405	0.10
2	Clayey Sand	250	0.28	36,000	46,023	0.18
3	Sand - Coarse-Fine	8,600	0.23	1,238,400	1,435,664	0.06
4	Clayey Sand	1,300	0.23	187,200	217,019	0.09
5	Silty Fine Sand with Shell Fragments	13,000	0.25	1,872,000	2,246,400	0.08

Step 12: Total Estimated Elastic Settlement for G 0.51 ft

Computation

Project:	Steelfield Road Landfill	Computed By:	KP	Date:	1/25/2019
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H

Top of Final Cover Elevation	111.8	ft
Top of Liner Elevation	53.7	ft
Natural Ground Elevation	66.4	ft
Depth of Excavation to Top of Liner	12.7	ft
Elevation of Partially Weathered Rock (PWR)	-102.00	ft
Groundwater Elevation	44.00	ft
Height of MSW & Cover	58.1	ft
Cover Thickness+Protective Cover	5	ft
MSW Thickness	53.1	ft
Unit Weight of MSW	90	pcf
Estimated Unit Weight of Cover	120	pcf
Unit Weight of Soil Excavated	116	pcf
Vertical Stress Induced by Exavated Material	1473	psf
Net Vertical Stress of MSW & Cover	3,906	psf
Water Density	62.4	pcf

Layer	Boring Log Description	Step 1		Step 2	Step 3	
		Soil Symbol	H_o (ft)	Unit Weight of Soil, γ_{soil} (pcf)	Unit Weight of Water, γ_{water} (pcf)	Vertical stress of Soil, σ_{soil} (psf)
1	Sand - Fine, Medium, Tan, Brown, White	SP, SW, SM	18.7	116.0	62.4	1,002
2	Clayey Sand	CL, SC	4	120.0	62.4	230
3	Sand - Coarse-Fine	SM	44	116.0	62.4	2,358
4	Clayey Sand	CL, SC	9	120.0	62.4	518
5	Clayey Sand	SM	80	116.0	62.4	4,288

H continued

Layer	Boring Log Description	Step 4	Step 5 (see calculation above)	Step 6	Step 7
		Vertical Stress at midpoint of Soil, $\sigma_{m\ soil}$ (psf)	Increment of Vertical Effective Stress of Ash & Cover, $\Delta\sigma ash_{\ \&\ cover}$ (psf)	Total vertical stress due to soil, σ_T (psf)	Total vertical stress on soil, $\sigma_{T\ psi}$ (psi)
1	Sand - Fine, Medium, Tan, Brown, White	501	3,906	501	3
2	Clayey Sand	115	3,906	1,117	8
3	Sand - Coarse-Fine	1,179	3,906	2,411	17
4	Clayey Sand	259	3,906	3,849	27
5	Clayey Sand	2,144	3,906	6,252	43

H continued

Layer	Boring Log Description	Step 8		Step 9	Step 10	Step 11
		Es (psi)	v_s	E_s (psf)	M_s (psf)	Z_e (ft)
1	Sand - Fine, Medium, Tan, Brown, White	2,100	0.21	302,400	340,405	0.21
2	Clayey Sand	250	0.28	36,000	46,023	0.34
3	Sand - Coarse-Fine	8,600	0.23	1,238,400	1,435,664	0.12
4	Clayey Sand	1,300	0.23	187,200	217,019	0.16
5	Clayey Sand	13,000	0.25	1,872,000	2,246,400	0.14

Step 12: Total Estimated Elastic Settlement for H 0.97 ft

Computation

Project:	Steelfield Road Landfill	Computed By:	KP	Date:	1/25/2019
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I

Top of Final Cover Elevation	96.7	ft
Top of Liner Elevation	47.0	ft
Natural Ground Elevation	70.3	ft
Depth of Excavation to Top of Liner	23.3	ft
Elevation of Partially Weathered Rock (PWR)	-102.00	ft
Groundwater Elevation	44.00	ft
Height of MSW & Cover	49.7	ft
Cover Thickness+Protective Cover	5	ft
MSW Thickness	44.7	ft
Unit Weight of MSW	90	pcf
Estimated Unit Weight of Cover	120	pcf
Unit Weight of Soil Excavated	116	pcf
Vertical Stress Induced by Exavated Material	2703	psf
Net Vertical Stress of MSW & Cover	1,920	psf
Water Density	62.4	pcf

Layer	Boring Log Description	Step 1		Step 2	Step 3	
		Soil Symbol	H_o (ft)	Unit Weight of Soil, γ_{soil} (pcf)	Unit Weight of Water, γ_{water} (pcf)	Vertical stress of Soil, σ_{soil} (psf)
1	Sand - Fine, Medium, Tan, Brown, White	SP, SW, SM	17.0	116.0	62.4	911
2	Clayey Sand	CL, SC	6	120.0	62.4	346
3	Sand - Coarse-Fine	SM	50	116.0	62.4	2,680
4	Clayey Sand	SM	76	120.0	62.4	4,378

I continued

Layer	Boring Log Description	Step 4	Step 5 (see calculation above)	Step 6	Step 7
		Vertical Stress at midpoint of Soil, $\sigma_{m\ soil}$ (psf)	Increment of Vertical Effective Stress of Ash & Cover, $\Delta\sigma ash_{\ \&\ cover}$ (psf)	Total vertical stress due to soil, σ_T (psf)	Total vertical stress on soil, $\sigma_{T\ psi}$ (psi)
1	Sand - Fine, Medium, Tan, Brown, White	456	1,920	456	3
2	Clayey Sand	173	1,920	1,085	8
3	Sand - Coarse-Fine	1,340	1,920	2,598	18
4	Clayey Sand	2,189	1,920	6,127	43

I continued

Layer	Boring Log Description	Step 8		Step 9	Step 10	Step 11
		Es (psi)	ν_s	E_s (psf)	M_s (psf)	Z_e (ft)
1	Sand - Fine, Medium, Tan, Brown, White	2,100	0.21	302,400	340,405	0.10
2	Clayey Sand	400	0.28	57,600	73,636	0.16
3	Sand - Coarse-Fine	8,600	0.23	1,238,400	1,435,664	0.07
4	Clayey Sand	13,000	0.25	1,872,000	2,246,400	0.06

Step 12: Total Estimated Elastic Settlement for I 0.39 ft

Computation

Project:	Steelfield Road Landfill	Computed By:	KP	Date:	1/25/2019
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J

Top of Final Cover Elevation	129.9	ft
Top of Liner Elevation	48.0	ft
Natural Ground Elevation	69	ft
Depth of Excavation to Top of Liner	21	ft
Elevation of Partially Weathered Rock (PWR)	-102.00	ft
Groundwater Elevation	44.00	ft
Height of MSW & Cover	81.9	ft
Cover Thickness+Protective Cover	5	ft
MSW Thickness	76.9	ft
Unit Weight of MSW	90	pcf
Estimated Unit Weight of Cover	120	pcf
Unit Weight of Soil Excavated	116	pcf
Vertical Stress Induced by Exavated Material	2436	
Net Vertical Stress of MSW & Cover	5,085	psf
Water Density	62.4	pcf

Layer	Boring Log Description	Step 1		Step 2	Step 3	
		Soil Symbol	H_o (ft)	Unit Weight of Soil, γ_{soil} (pcf)	Unit Weight of Water, γ_{water} (pcf)	Vertical stress of Soil, σ_{soil} (psf)
1	Sand - Fine, Medium, Tan, Brown, White	SP, SW, SM	19.00	116.0	62.4	1,018
2	Clayey Sand	CL, SC	6.00	120.0	62.4	346
3	Sand - Coarse-Fine	SM	49	116.0	62.4	2,626
4	Silty Fine Sand with Clary Fragments	SM	76	116.0	62.4	4,074

J continued

Layer	Boring Log Description	Step 4	Step 5 (see calculation above)	Step 6	Step 7
		Vertical Stress at midpoint of Soil, $\sigma_{m\ soil}$ (psf)	Increment of Vertical Effective Stress of Ash & Cover, $\Delta\sigma ash_{\ \&\ cover}$ (psf)	Total vertical stress due to soil, σ_T (psf)	Total vertical stress on soil, $\sigma_{T\ psi}$ (psi)
1	Sand - Fine, Medium, Tan, Brown, White	509	5,085	509	4
2	Clayey Sand	173	5,085	1,191	8
3	Sand - Coarse-Fine	1,313	5,085	2,677	19
4	Silty Fine Sand with Clary Fragments	2,037	5,085	6,027	42

J continued

Layer	Boring Log Description	Step 8		Step 9	Step 10	Step 11
		Es (psi)	ν_s	E_s (psf)	M_s (psf)	Z_e (ft)
1	Sand - Fine, Medium, Tan, Brown, White	2,100	0.21	302,400	340,405	0.28
2	Clayey Sand	400	0.28	57,600	73,636	0.41
3	Sand - Coarse-Fine	8,600	0.23	1,238,400	1,435,664	0.17
4	Silty Fine Sand with Clary Fragments	13,000	0.25	1,872,000	2,246,400	0.17

Step 12: Total Estimated Elastic Settlement for J 1.03 ft

HDR Computation

Project:	Steelfield Road Landfill	Computed By:	KP	Date:	1/25/2019
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Steps - Primary Consolidation (Clay Soils)

1.Initial void ratio (e_{oi}) of the clay soil layers are based on laboratory consolidations tests performed (Refer to Attachment 2).

2. Primary compression index (C_c) and recompression index(C_r) for CL, SC and CH are based on laboratory consolidations tests performed (Refer to Attachment 2)

3. Preconsolidation pressures for fine grain soils are based on consolidation tests performed during site investigation.

4. Determine the primary consolidation using the Equation 12.22 (page 470) below from Reference 3.

$$Z_c = C_r \times \frac{H_{oi}}{1+e_{oi}} \times \log\frac{p_c}{\sigma_o} + C_c \times \frac{H_{oi}}{1+e_{oi}} \times \log\frac{\sigma_o + \Delta\sigma}{p_c}$$

Where:

Z_c = primary consolidation settlement of clay layer (ft)
H_o = initial thickness of clay layer (ft)
e_{oi} = initial void ratio of clay layer
C_r = recompression index
C_c = primary compression index
σ_o = initial vertical effective stress (lb/ft^2)
p_c = preconsolidation pressure (lb/ft^2)
$\Delta\sigma$ = increment of vertical effective stress (lb/ft^2)

4. Repeat Steps 1 - 4 for each clay layer. Add the primary consolidation settlement for each clay layer together to get the total estimated primary consolidation settlement at that location based on information from the boring logs, recent surveys, and seasonal high groundwater levels.

$$Z_{c\,Total} = Z_{c\,layer\,n} + Z_{c\,layer\,n+1}$$

HDR Computation

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Calculations

A

Layer	Boring Log Description	From Previous Calculation				
		Soil Symbol	H_o (ft)	Increment of Vertical Effective Stress, Δσ (psf)	Vertical Stress at midpoint, $\sigma_{m\ soil}$ (psf)	Vertical Stress in soil layer, $\sigma_{soil\ layers}$ (psf)
1	MSW	WTE Ash	35.9	4,594	1616	3231
2	Sand - Fine, Medium, Tan, Brown, White	SP, SW, SM	20.2	4,594	542	1,083
3	Clayey Sand	CL, SC	4.5	4,594	130	259
4	Sand - Coarse-Fine	SM	42.2	4,594	1,131	2,262
5	Clayey Sand	SC, CL	7.9	4,594	228	455
6	Silty Fine Sand with Shell Fragments	SM	34.1	4,594	914	1,828

A continued

Layer	Boring Log Description	Step 1			Step 2	Step 3
		Minimum Void Ratio, e_{min}	Maximum Void Ratio, e_{max}	Initial Void Ratio, e_{oi}	Primary Compression Index, C_c	Preconsolidation Pressure, p_c (psf)
1	MSW	WTE Ash				
2	Sand - Fine, Medium, Tan, Brown, White	SP, SW, SM				
3	Clayey Sand	CL, SC		3.10	1.40	8,630
4	Sand - Coarse-Fine	SM				
5	Clayey Sand	SC, CL		1.01	0.16	8,360
6	Silty Fine Sand with Shell Fragments	SM				

A continued

Layer	Boring Log Description	Step 3, Continued	Step 4	
		Initial Vertical Effective Stress on Soil, σ_o (psf)	Recompression Index, C_r	Primary Consolidation Settlement of Clay Layer, Z_c (ft)
1	MSW	1616		
2	Sand - Fine, Medium, Tan, Brown, White	3,774		
3	Clayey Sand	4,446	0.071	0.05
4	Sand - Coarse-Fine	5,707		
5	Clayey Sand	7,066	0.003	0.09
6	Silty Fine Sand with Shell Fragments			

Step 4: Total Estimated Primary Consolidation Settlement for A 0.14 ft

HDR Computation

Project:	Steelfield Road Landfill	Computed By:	KP	Date:	1/25/2019
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B

Layer	Boring Log Description	From Previous Calculation				
		Soil Symbol	H_o (ft)	Increment of Vertical Effective Stress, Δσ (psf)	Vertical Stress at midpoint, $\sigma_{m\ soil}$ (psf)	Vertical Stress in soil layer, $\sigma_{soil\ layers}$ (psf)
1	MSW	WTE Ash	38.2	6,602	1,719	3438
2	Sand - Fine, Medium, Tan, Brown, White	SP, SW, SM	21	6,602	560	1,120
3	Clayey Sand	CL, SC	5	6,602	133	265
4	Sand - Coarse-Fine	SM	42	6,602	1,118	2,235
5	Clayey Sand	SC, CL	8	6,602	225	449
6	Silty Fine Sand with Shell Fragments	SM	34	6,602	909	1,817

B continued

Layer	Boring Log Description	Step 1			Step 2	Step 3
		Minimum Void Ratio, e_{min}	Maximum Void Ratio, e_{max}	Initial Void Ratio, e_{oi}	Primary Compression Index, C_c	Preconsolidation Pressure, p_c (psf)
1	MSW					
2	Sand - Fine, Medium, Tan, Brown, White					
3	Clayey Sand			3.10	1.40	8,360
4	Sand - Coarse-Fine					
5	Clayey Sand			1.010	0.16	8,360
6	Silty Fine Sand with Shell Fragments					

B continued

Layer	Boring Log Description	Step 3, Continued	Step 4	
		Initial Vertical Effective Stress on Soil, σ_o (psf)	Recompression Index, C_r	Primary Consolidation Settlement of Clay Layer, Z_c (ft)
1	MSW	1719		
2	Sand - Fine, Medium, Tan, Brown, White	3,998		
3	Clayey Sand	4,691	0.071	0.23
4	Sand - Coarse-Fine	5,942		
5	Clayey Sand	7,285	0.003	0.14
6	Silty Fine Sand with Shell Fragments	8,419		

Step 4: Total Estimated Primary Consolidation Settlement for B 0.37 ft

HDR Computation

Project:	Steelfield Road Landfill	Computed By:	KP	Date:	1/25/2019
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C

Layer	Boring Log Description	From Previous Calculation				
		Soil Symbol	H_o (ft)	Increment of Vertical Effective Stress, Δσ (psf)	Vertical Stress at midpoint, $\sigma_{m\ soil}$ (psf)	Vertical Stress in soil layer, $\sigma_{soil\ layers}$ (psf)
1	Sand - Fine, Medium, Tan, Brown, White	SP, SW, SM	25	12,578	662	1,324
2	Clayey Sand	CL, SC	7	12,578	202	403
3	Sand - Coarse-Fine	SM	52	12,578	1,394	2,787
4	Silty Fine Sand with Shell Fragments	SM	73	12,578	1,957	3,913
5	0	0	0	0	0	0

C continued

Layer	Boring Log Description	Step 1			Step 2	Step 3
		Minimum Void Ratio, e_{min}	Maximum Void Ratio, e_{max}	Initial Void Ratio, e_{oi}	Primary Compression Index, C_c	Preconsolidation Pressure, p_c (psf)
1	Sand - Fine, Medium, Tan, Brown, White					
2	Clayey Sand			3.100	1.40	8,630
3	Sand - Coarse-Fine					
4	Silty Fine Sand with Shell Fragments					
5	0					

C continued

Layer	Boring Log Description	Step 3, Continued	Step 4		
		Initial Vertical Effective Stress on Soil, σ_o (psf)	Recompression Index, C_r	Primary Consolidation Settlement of Clay Layer, Z_c (ft)	
1	Sand - Fine, Medium, Tan, Brown, White	662	0		
2	Clayey Sand	1,526	0.071		0.60
3	Sand - Coarse-Fine	3,122			
4	Silty Fine Sand with Shell Fragments	6,473			
5	0	0			

Step 4: Total Estimated Primary Consolidation Settlement for C 0.6 ft

HDR Computation

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D

Layer	Boring Log Description	From Previous Calculation				
		Soil Symbol	H_o (ft)	Increment of Vertical Effective Stress, Δσ (psf)	Vertical Stress at midpoint, $\sigma_{m\ soil}$ (psf)	Vertical Stress in soil layer, $\sigma_{soil\ layers}$ (psf)
1	Sand - Fine, Medium, Tan, Brown, White	SP, SW, SM	26	15,067	702	1,404
2	Clayey Sand	CL, SC	6	15,067	173	346
3	Sand - Coarse-Fine	SM	52	15,067	1,394	2,787
4	Silty Fine Sand with Shell Fragments	SM	73	15,067	1,957	3,913
5	0	0	0	0	0	0

D continued

Layer	Boring Log Description	Step 1			Step 2	Step 3
		Minimum Void Ratio, e_{min}	Maximum Void Ratio, e_{max}	Initial Void Ratio, e_{oi}	Primary Compression Index, C_c	Preconsolidation Pressure, p_c (psf)
1	Sand - Fine, Medium, Tan, Brown, White					
2	Clayey Sand			3.100	1.40	8,630
3	Sand - Coarse-Fine					
4	Silty Fine Sand with Shell Fragments					
5	0					

D continued

Layer	Boring Log Description	Step 3, Continued	Step 4	
		Initial Vertical Effective Stress on Soil, σ_o (psf)	Recompression Index, C_r	Primary Consolidation Settlement of Clay Layer, Z_c (ft)
1	Sand - Fine, Medium, Tan, Brown, White	702		
2	Clayey Sand	1,577	0.071	0.66
3	Sand - Coarse-Fine	3,144		
4	Silty Fine Sand with Shell Fragments	6,495		
5	0	0		

Step 4: Total Estimated Primary Consolidation Settlement for D 0.66 ft

HDR Computation

Project:	Steelfield Road Landfill	Computed By:	KP	Date:	1/25/2019
Subject:	Cells 6 & 7 Expansion	Checked By:	TY	Date:	2/9/2019
Task:	Settlement Calculation - Primary Consolidation Settlement	Sheet:		Of:	

E

Layer	Boring Log Description	From Previous Calculation				
		Soil Symbol	H_o (ft)	Increment of Vertical Effective Stress, Δσ (psf)	Vertical Stress at midpoint, $\sigma_{m\ soil}$ (psf)	Vertical Stress in soil layer, $\sigma_{soil\ layers}$ (psf)
1	Sand - Fine, Medium, Tan, Brown, White	SP, SW, SM	26	8,624	705	1,410
2	Clayey Sand	CL, SC	6	8,624	173	346
3	Sand - Coarse-Fine	SM	47	8,624	1,260	2,519
4	Clayey Sand	SC, CL	6	8,624	173	346
5	Silty Fine Sand with Clary Fragments	SM	76	8,624	2,037	4,074

E continued

Layer	Boring Log Description	Step 1			Step 2	Step 3
		Minimum Void Ratio, e_{min}	Maximum Void Ratio, e_{max}	Initial Void Ratio, e_{oi}	Primary Compression Index, C_c	Preconsolidation Pressure, p_c (psf)
1	Sand - Fine, Medium, Tan, Brown, White					
2	Clayey Sand			3.100	1.40	8,630
3	Sand - Coarse-Fine					
4	Clayey Sand			1.010	0.16	8,360
5	Silty Fine Sand with Clary Fragments					

E continued

Layer	Boring Log Description	Step 3, Continued	Step 4	
		Initial Vertical Effective Stress on Soil, σ_o (psf)	Recompression Index, C_r	Primary Consolidation Settlement of Clay Layer, Z_c (ft)
1	Sand - Fine, Medium, Tan, Brown, White	705		
2	Clayey Sand	1,583	0.071	0.23
3	Sand - Coarse-Fine	3,016		
4	Clayey Sand	4,449	0.003	0.10
5	Silty Fine Sand with Clary Fragments	6,659		

Step 4: Total Estimated Primary Consolidation Settlement for E 0.33 ft

HDR Computation

Project:	Steelfield Road Landfill	Computed By:	KP	Date:	1/25/2019
Subject:	Cells 6 & 7 Expansion	Checked By:	TY	Date:	2/9/2019
Task:	Settlement Calculation - Primary Consolidation Settlement	Sheet:		Of:	

Calculations

F

Layer	Boring Log Description	From Previous Calculation				
		Soil Symbol	H_o (ft)	Increment of Vertical Effective Stress, Δσ (psf)	Vertical Stress at midpoint, $\sigma_{m\ soil}$ (psf)	Vertical Stress in soil layer, $\sigma_{soil\ layers}$ (psf)
1	Sand - Fine, Medium, Tan, Brown, White	SP, SW, SM	25.6	10,652	686	1,372
2	Clayey Sand	CL, SC	5.0	10,652	144	288
3	Sand - Coarse-Fine	SM	46.0	10,652	1,233	2,466
4	Clayey Sand	SC, CL	6.0	10,652	173	346
5	Silty Fine Sand with Shell Fragments	SM	77.0	10,652	2,064	4,127

F continued

Layer	Boring Log Description	Step 1			Step 2	Step 3
		Minimum Void Ratio, e_{min}	Maximum Void Ratio, e_{max}	Initial Void Ratio, e_{oi}	Primary Compression Index, C_c	Preconsolidation Pressure, p_c (psf)
1	Sand - Fine, Medium, Tan, Brown, White	SP, SW, SM				
2	Clayey Sand	CL, SC		3.10	1.40	8,630
3	Sand - Coarse-Fine	SM				
4	Clayey Sand	SC, CL		1.01	0.16	8,360
5	Silty Fine Sand with Shell Fragments	SM				

F continued

Layer	Boring Log Description	Step 3, Continued	Step 4	
		Initial Vertical Effective Stress on Soil, σ_o (psf)	Recompression Index, C_r	Primary Consolidation Settlement of Clay Layer, Z_c (ft)
1	Sand - Fine, Medium, Tan, Brown, White	686		
2	Clayey Sand	1,516	0.071	0.32
3	Sand - Coarse-Fine	2,893		
4	Clayey Sand	4,299	0.003	0.12
5	Silty Fine Sand with Shell Fragments			

Step 4: Total Estimated Primary Consolidation Settlement for F 0.44 ft

HDR Computation

Project:	Steelfield Road Landfill	Computed By:	KP	Date:	1/25/2019
Subject:	Cells 6 & 7 Expansion	Checked By:	TY	Date:	2/9/2019
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G

Layer	Boring Log Description	From Previous Calculation				
		Soil Symbol	H_o (ft)	Increment of Vertical Effective Stress, Δσ (psf)	Vertical Stress at midpoint, $\sigma_{m\ soil}$ (psf)	Vertical Stress in soil layer, $\sigma_{soil\ layers}$ (psf)
1	Sand - Fine, Medium, Tan, Brown, White	SP, SW, SM	17	2,107	448	895
2	Clayey Sand	CL, SC	4	2,107	115	230
3	Sand - Coarse-Fine	SM	44	2,107	1,179	2,358
4	Clayey Sand	CL, SC	9	2,107	259	518
5	Silty Fine Sand with Shell Fragments	SM	81	2,107	2,171	4,342

G continued

Layer	Boring Log Description	Step 1			Step 2	Step 3
		Minimum Void Ratio, e_{min}	Maximum Void Ratio, e_{max}	Initial Void Ratio, e_{oi}	Primary Compression Index, C_c	Preconsolidation Pressure, p_c (psf)
1	Sand - Fine, Medium, Tan, Brown, White					
2	Clayey Sand			3.10	1.40	8,630
3	Sand - Coarse-Fine					
4	Clayey Sand			1.01	0.16	8,360
5	Silty Fine Sand with Shell Fragments					

G continued

Layer	Boring Log Description	Step 3, Continued	Step 4	
		Initial Vertical Effective Stress on Soil, σ_o (psf)	Recompression Index, C_r	Primary Consolidation Settlement of Clay Layer, Z_c (ft)
1	Sand - Fine, Medium, Tan, Brown, White	448		
2	Clayey Sand	1,011	0.071	-0.54
3	Sand - Coarse-Fine	2,305		
4	Clayey Sand	3,743	0.003	-0.11
5	Silty Fine Sand with Shell Fragments	6,173		

Step 4: Total Estimated Primary Consolidation Settlement for G -0.65 ft

HDR Computation

Project:	Steelfield Road Landfill	Computed By:	KP	Date:	1/25/2019
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Task:	Settlement Calculation - Primary Consolidation Settlement	Sheet:		Of:	

H

Layer	Boring Log Description	From Previous Calculation				
		Soil Symbol	H_o (ft)	Increment of Vertical Effective Stress, Δσ (psf)	Vertical Stress at midpoint, $\sigma_{m\ soil}$ (psf)	Vertical Stress in soil layer, $\sigma_{soil\ layers}$ (psf)
1	Sand - Fine, Medium, Tan, Brown, White	SP, SW, SM	19	3,906	501	1,002
2	Clayey Sand	CL, SC	4	3,906	115	230
3	Sand - Coarse-Fine	SM	44	3,906	1,179	2,358
4	Clayey Sand	CL, SC	9	3,906	259	518
5	Clayey Sand	SM	80	3,906	2,144	4,288
	0	0	0	0	0	0

H continued

Layer	Boring Log Description	Step 1			Step 2	Step 3
		Minimum Void Ratio, e_{min}	Maximum Void Ratio, e_{max}	Initial Void Ratio, e_{oi}	Primary Compression Index, C_c	Preconsolidation Pressure, p_c (psf)
1	Sand - Fine, Medium, Tan, Brown, White					
2	Clayey Sand			3.100	1.40	8,630
3	Sand - Coarse-Fine					
4	Clayey Sand			1.010	0.16	8,360
5	Clayey Sand					
	0					

H continued

Layer	Boring Log Description	Step 3, Continued	Step 4		
		Initial Vertical Effective Stress on Soil, σ_o (psf)	Recompression Index, C_r	Primary Consolidation Settlement of Clay Layer, Z_c (ft)	
1	Sand - Fine, Medium, Tan, Brown, White	501	0		
2	Clayey Sand	1,117	0.071		-0.26
3	Sand - Coarse-Fine	2,411			
4	Clayey Sand	3,849	0.003		-0.02
5	Clayey Sand	6,252			
	0	0			

Step 4: Total Estimated Primary Consolidation Settlement for C -0.28 ft

HDR Computation

Project:	Steelfield Road Landfill	Computed By:	KP	Date:	1/25/2019
Subject:	Cells 6 & 7 Expansion	Checked By:	TY	Date:	2/9/2019
Task:	Settlement Calculation - Primary Consolidation Settlement	Sheet:		Of:	

I

Layer	Boring Log Description	From Previous Calculation				
		Soil Symbol	H_o (ft)	Increment of Vertical Effective Stress, Δσ (psf)	Vertical Stress at midpoint, $\sigma_{m\ soil}$ (psf)	Vertical Stress in soil layer, $\sigma_{soil\ layers}$ (psf)
1	Sand - Fine, Medium, Tan, Brown, White	SP, SW, SM	17	1,920	456	911
2	Clayey Sand	CL, SC	6	1,920	173	346
3	Sand - Coarse-Fine	SM	50	1,920	1,340	2,680
4	Clayey Sand	SM	76	1,920	2,189	4,378
5	0	0	0	0	0	0

I continued

Layer	Boring Log Description	Step 1			Step 2	Step 3
		Minimum Void Ratio, e_{min}	Maximum Void Ratio, e_{max}	Initial Void Ratio, e_{oi}	Primary Compression Index, C_c	Preconsolidation Pressure, p_c (psf)
1	Sand - Fine, Medium, Tan, Brown, White					
2	Clayey Sand			3.100	1.40	8,630
3	Sand - Coarse-Fine					
4	Clayey Sand					
5	0					

I continued

Layer	Boring Log Description	Step 3, Continued	Step 4	
		Initial Vertical Effective Stress on Soil, σ_o (psf)	Recompression Index, C_r	Primary Consolidation Settlement of Clay Layer, Z_c (ft)
1	Sand - Fine, Medium, Tan, Brown, White	456		
2	Clayey Sand	1,085	0.071	-0.85
3	Sand - Coarse-Fine	2,598		
4	Clayey Sand	6,127		
5	0	0		

Step 4: Total Estimated Primary Consolidation Settlement for H -0.85 ft

HDR Computation

Project:	Steelfield Road Landfill	Computed By:	KP	Date:	1/25/2019
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J

Layer	Boring Log Description	From Previous Calculation				
		Soil Symbol	H_o (ft)	Increment of Vertical Effective Stress, Δσ (psf)	Vertical Stress at midpoint, $\sigma_{m\ soil}$ (psf)	Vertical Stress in soil layer, $\sigma_{soil\ layers}$ (psf)
1	Sand - Fine, Medium, Tan, Brown, White	SP, SW, SM	19	5,085	509	1,018
2	Clayey Sand	CL, SC	6	5,085	173	346
3	Sand - Coarse-Fine	SM	49	5,085	1,313	2,626
4	Silty Fine Sand with Clary Fragments	SM	76	5,085	2,037	4,074

J continued

Layer	Boring Log Description	Step 1			Step 2	Step 3
		Minimum Void Ratio, e_{min}	Maximum Void Ratio, e_{max}	Initial Void Ratio, e_{oi}	Primary Compression Index, C_c	Preconsolidation Pressure, p_c (psf)
1	Sand - Fine, Medium, Tan, Brown, White					
2	Clayey Sand			3.100	1.40	8,630
3	Sand - Coarse-Fine					
4	Silty Fine Sand with Clary Fragments					

J continued

Layer	Boring Log Description	Step 3, Continued	Step 4	
		Initial Vertical Effective Stress on Soil, σ_o (psf)	Recompression Index, C_r	Primary Consolidation Settlement of Clay Layer, Z_c (ft)
1	Sand - Fine, Medium, Tan, Brown, White	509		
2	Clayey Sand	1,191	0.071	-0.19
3	Sand - Coarse-Fine	2,677		
4	Silty Fine Sand with Clary Fragments	6,027		

Step 4: Total Estimated Primary Consolidation Settlement for J -0.19 ft

HDR Computation

Project:	Steelfield Road Landfill	Computed By:	KP	Date:	1/25/2019
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Steps - Secondary Consolidation (Clay Soils)

1. Secondary consolidation compression index (C_α) is estimated based on natural water content (w_n) of 30% (Refer to Cell 5 investigation, Boring Log B-1).

2. Estimate the initial thickness of the clay layer before starting secondary consolidation settlement (H_{os}) using the initial layer thickness (H_o),the elastic settlement (Z_e), the primary consolidation settlement (Z_c) and the equation below.

$$H_{os} = H_o - Z_e - Z_c$$

3. Determine the secondary compression using the Equation 12.23 (page 470) below from Reference 3. Assume the initial void ratio of the clay layer before secondary consolidation settlement is equal to the minimum void ratio in Table 6 (page 7.1-22) of Reference 1 (Attachment 2).

$$Z_\alpha = C_\alpha \times \frac{H_{os}}{1 + e_{os}} \times \log\frac{t_2}{t_1}$$

Where:

Z_α = long-term secondary compression (ft)
e_{os} = initial void ratio of clay layer before starting secondary consolidation settlement
C_α = secondary consolidation compression index
H_{os} = initial thickness of clay layer before starting secondary consolidation settlement (ft)
t_1 = starting time of the time period for which long-term settlement of the layer is desired
t_2 = ending time of the time period for which long-term settlement of the layer is desired

assume t_1 = 1 year
assume t_2 = 100 years

4. Repeat Steps 1 - 3 for each clay layer. Add the secondary compression settlement for each clay layer together to get the total estimated secondary compression settlement at that location based on information from the boring logs, recent surveys, and seasonal high groundwater levels.

$$Z_{\alpha\,Total} = Z_{\alpha\,layer\,n} + Z_{\alpha\,layer\,n+1}$$

HDR Computation

Project:	Steelfield Road Landfill	Computed By:	KP	Date:	1/25/2019
Subject:	Cells 6 & 7 Expansion	Checked By:	TY	Date:	2/9/2019
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Calculations

A

Layer	Boring Log Description	From Previous Calculations				Step 1
		Initial Layer Thickness, H_o (ft)	Elastic Layer Settlement, Z_e (ft)	Primary Consolidation Settlement of Clay, Z_c (ft)	Wet Unit Weight, γ_{wet} (pcf)	Minimum dry unit weight, $\gamma_{dry\ min}$ (pcf)
1	Sand - Fine, Medium, Tan, Brown, White	20.20	0.15	0.00	116	
2	Clayey Sand	4.50	0.16	0.05	120	
3	Sand - Coarse-Fine	42	0.09	0.00	116	
4	Clayey Sand	8	0.10	0.09	120	
5	Silty Fine Sand with Shell Fragments	34	0.05	0.00	116	

A continued

Layer	Boring Log Description	Step 1, continued				Step 2
		Maximum dry unit weight, $\gamma_{dry\ max}$ (pcf)	Average dry unit weight, $\gamma_{dry\ avg}$ (pcf)	water content, w_n	Secondary Consolidation Compression Index, C_α	Initial Thickness of Clay Layer before Secondary Settlement, H_{os} (ft)
1	Sand - Fine, Medium, Tan, Brown, White					20.05
2	Clayey Sand				0.003	4.29
3	Sand - Coarse-Fine					42.11
4	Clayey Sand				0.003	7.71
5	Silty Fine Sand with Shell Fragments					34.05

A continued

Layer	Boring Log Description	Step 3	
		Initial Void Ratio of Clay before Secondary Settlement, e_{os}	Secondary Compression Settlement of Clay Layer, Z_α (ft)
1	Sand - Fine, Medium, Tan, Brown, White		0.00
2	Clayey Sand	0.25	0.02
3	Sand - Coarse-Fine		0.00
4	Clayey Sand	0.25	0.04
5	Silty Fine Sand with Shell Fragments		0.00

Step 4: Total Estimated Secondary Compression Settlement for A 0.06 ft

HDR Computation

Project:	Steelfield Road Landfill	Computed By:	KP	Date:	1/25/2019
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B

Layer	Boring Log Description	From Previous Calculations				Step 1
		Initial Layer Thickness, H_o (ft)	Elastic Layer Settlement, Z_e (ft)	Primary Consolidation Settlement of Clay, Z_c (ft)	Wet Unit Weight, γ_{wet} (pcf)	Minimum dry unit weight, $\gamma_{dry\ min}$ (pcf)
1	Sand - Fine, Medium, Tan, Brown, White	21	0.23	0.00	116	
2	Clayey Sand	5	0.24	0.23	120	
3	Sand - Coarse-Fine	42	0.12	0.00	116	
4	Clayey Sand	8	0.14	0.14	120	
5	Silty Fine Sand with Shell Fragments	34	0.07	0.00	116	

B continued

Layer	Boring Log Description	Step 1, continued				Step 2
		Maximum dry unit weight, $\gamma_{dry\ max}$ (pcf)	Average dry unit weight, $\gamma_{dry\ avg}$ (pcf)	water content, w_n	Secondary Consolidation Compression Index, C_α	Initial Thickness of Clay Layer before Secondary Settlement, H_{os} (ft)
1	Sand - Fine, Medium, Tan, Brown, White				0	20.67
2	Clayey Sand				0.003	4.13
3	Sand - Coarse-Fine				0	41.58
4	Clayey Sand				0.003	7.52
5	Silty Fine Sand with Shell Fragments				0	33.83

B continued

Layer	Boring Log Description	Step 3	
		Initial Void Ratio of Clay before Secondary Settlement, e_{os}	Secondary Compression Settlement of Clay Layer, Z_α (ft)
1	Sand - Fine, Medium, Tan, Brown, White		0.00
2	Clayey Sand	0.25	0.02
3	Sand - Coarse-Fine		0.00
4	Clayey Sand	0.25	0.04
5	Silty Fine Sand with Shell Fragments		0.00

Step 4: Total Estimated Secondary Compression Settlement for B 0.06 ft

HDR Computation

Project:	Steelfield Road Landfill	Computed By:	KP	Date:	1/25/2019
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C

Layer	Boring Log Description	From Previous Calculations				Step 1
		Initial Layer Thickness, H_o (ft)	Elastic Layer Settlement, Z_e (ft)	Primary Consolidation Settlement of Clay, Z_c (ft)	Wet Unit Weight, γ_{wet} (pcf)	Minimum dry unit weight, $\gamma_{dry\ min}$ (pcf)
1	Sand - Fine, Medium, Tan, Brown, White	25	0.91	0.00	116	
2	Clayey Sand	7	1.20	0.60	120	
3	Sand - Coarse-Fine	52	0.46	0.00	116	
4	Silty Fine Sand with Shell Fragments	73	0.41	0.00	116	
5	0	0	0.00	0.00	0	

C continued

Layer	Boring Log Description	Step 1, continued				Step 2
		Maximum dry unit weight, $\gamma_{dry\ max}$ (pcf)	Average dry unit weight, $\gamma_{dry\ avg}$ (pcf)	water content, w_n	Secondary Consolidation Compression Index, C_α	Initial Thickness of Clay Layer before Secondary Settlement, H_{os} (ft)
1	Sand - Fine, Medium, Tan, Brown, White					
2	Clayey Sand				0.003	5.20
3	Sand - Coarse-Fine					
4	Silty Fine Sand with Shell Fragments					
5	0					

C continued

Layer	Boring Log Description	Step 3	
		Initial Void Ratio of Clay before Secondary Settlement, e_{os}	Secondary Compression Settlement of Clay Layer, Z_α (ft)
1	Sand - Fine, Medium, Tan, Brown, White		0.00
2	Clayey Sand	0.25	0.02
3	Sand - Coarse-Fine		
4	Silty Fine Sand with Shell Fragments		
5	0		

Step 4: Total Estimated Secondary Compression Settlement for C 0.02 ft

HDR Computation

Project:	Steelfield Road Landfill	Computed By:	KP	Date:	1/25/2019
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D

Layer	Boring Log Description	From Previous Calculations				Step 1
		Initial Layer Thickness, H_o (ft)	Elastic Layer Settlement, Z_e (ft)	Primary Consolidation Settlement of Clay, Z_c (ft)	Wet Unit Weight, γ_{wet} (pcf)	Minimum dry unit weight, $\gamma_{dry\ min}$ (pcf)
1	Sand - Fine, Medium, Tan, Brown, White	26	1.16	0.00	116	
2	Clayey Sand	6	1.23	0.66	120	
3	Sand - Coarse-Fine	52	0.55	0.00	116	
4	Silty Fine Sand with Shell Fragments	73	0.49	0.00	116	
5	0	0	0.00	0.00	0	

D continued

Layer	Boring Log Description	Step 1, continued				Step 2
		Maximum dry unit weight, $\gamma_{dry\ max}$ (pcf)	Average dry unit weight, $\gamma_{dry\ avg}$ (pcf)	water content, w_n	Secondary Consolidation Compression Index, C_α	Initial Thickness of Clay Layer before Secondary Settlement, H_{os} (ft)
1	Sand - Fine, Medium, Tan, Brown, White					
2	Clayey Sand				0.003	4.11
3	Sand - Coarse-Fine					
4	Silty Fine Sand with Shell Fragments					
5	0					

D continued

Layer	Boring Log Description	Step 3	
		Initial Void Ratio of Clay before Secondary Settlement, e_{os}	Secondary Compression Settlement of Clay Layer, Z_α (ft)
1	Sand - Fine, Medium, Tan, Brown, White		0.00
2	Clayey Sand	0.250	0.02
3	Sand - Coarse-Fine	0.000	0.00
4	Silty Fine Sand with Shell Fragments	0.000	0.00
5	0		

Step 4: Total Estimated Secondary Compression Settlement for D 0.02 ft

HDR Computation

Project:	Steelfield Road Landfill	Computed By:	KP	Date:	1/25/2019
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E

Layer	Boring Log Description	From Previous Calculations				Step 1
		Initial Layer Thickness, H_o (ft)	Elastic Layer Settlement, Z_e (ft)	Primary Consolidation Settlement of Clay, Z_c (ft)	Wet Unit Weight, γ_{wet} (pcf)	Minimum dry unit weight, $\gamma_{dry\ min}$ (pcf)
1	Sand - Fine, Medium, Tan, Brown, White	26	0.67	0.00	116	
2	Clayey Sand	6	0.70	0.23	120	
3	Sand - Coarse-Fine	47	0.28	0.00	116	
4	Clayey Sand	6	0.18	0.10	120	
5	Silty Fine Sand with Clary Fragments	76	0.29	0.00	116	

E continued

Layer	Boring Log Description	Step 1, continued				Step 2
		Maximum dry unit weight, $\gamma_{dry\ max}$ (pcf)	Average dry unit weight, $\gamma_{dry\ avg}$ (pcf)	water content, w_n	Secondary Consolidation Compression Index, C_α	Initial Thickness of Clay Layer before Secondary Settlement, H_{os} (ft)
1	Sand - Fine, Medium, Tan, Brown, White					
2	Clayey Sand				0.003	5.07
3	Sand - Coarse-Fine					
4	Clayey Sand				0.003	5.72
5	Silty Fine Sand with Clary Fragments					

E continued

Layer	Boring Log Description	Step 3	
		Initial Void Ratio of Clay before Secondary Settlement, e_{os}	Secondary Compression Settlement of Clay Layer, Z_α (ft)
1	Sand - Fine, Medium, Tan, Brown, White		
2	Clayey Sand	0.250	0.02
3	Sand - Coarse-Fine		
4	Clayey Sand	0.250	0.03
5	Silty Fine Sand with Clary Fragments		

Step 4: Total Estimated Secondary Compression Settlement for E 0.05 ft

HDR Computation

Project:	Steelfield Road Landfill	Computed By:	KP	Date:	1/25/2019
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F

Layer	Boring Log Description	From Previous Calculations				Step 1
		Initial Layer Thickness, H_o (ft)	Elastic Layer Settlement, Z_e (ft)	Primary Consolidation Settlement of Clay, Z_c (ft)	Wet Unit Weight, γ_{wet} (pcf)	Minimum dry unit weight, $\gamma_{dry\ min}$ (pcf)
1	Sand - Fine, Medium, Tan, Brown, White	25.60	0.80	0.00	116	
2	Clayey Sand	5.00	0.72	0.32	120	
3	Sand - Coarse-Fine	46	0.34	0.00	116	
4	Clayey Sand	6	0.23	0.12	120	
5	Silty Fine Sand with Shell Fragments	77	0.37	0.00	116	

F continued

Layer	Boring Log Description	Step 1, continued				Step 2
		Maximum dry unit weight, $\gamma_{dry\ max}$ (pcf)	Average dry unit weight, $\gamma_{dry\ avg}$ (pcf)	water content, w_n	Secondary Consolidation Compression Index, C_α	Initial Thickness of Clay Layer before Secondary Settlement, H_{os} (ft)
1	Sand - Fine, Medium, Tan, Brown, White					24.80
2	Clayey Sand				0.003	3.96
3	Sand - Coarse-Fine					45.66
4	Clayey Sand				0.003	5.65
5	Silty Fine Sand with Shell Fragments					76.63

F continued

Layer	Boring Log Description	Step 3	
		Initial Void Ratio of Clay before Secondary Settlement, e_{os}	Secondary Compression Settlement of Clay Layer, Z_α (ft)
1	Sand - Fine, Medium, Tan, Brown, White		0.00
2	Clayey Sand	0.25	0.02
3	Sand - Coarse-Fine		0.00
4	Clayey Sand	0.25	0.03
5	Silty Fine Sand with Shell Fragments		0.00

Step 4: Total Estimated Secondary Compression Settlement for F 0.05 ft

HDR Computation

Project:	Steelfield Road Landfill	Computed By:	KP	Date:	1/25/2019
Subject:	Cells 6 & 7 Expansion	Checked By:	TY	Date:	2/9/2019
Task:	Settlement Calculation - Secondary Compression Settlement	Sheet:		Of:	

G

Layer	Boring Log Description	From Previous Calculations				Step 1
		Initial Layer Thickness, H_o (ft)	Elastic Layer Settlement, Z_e (ft)	Primary Consolidation Settlement of Clay, Z_c (ft)	Wet Unit Weight, γ_{wet} (pcf)	Minimum dry unit weight, $\gamma_{dry\ min}$ (pcf)
1	Sand - Fine, Medium, Tan, Brown, White	17	0.10	0.00	116	
2	Clayey Sand	4	0.18	-0.54	120	
3	Sand - Coarse-Fine	44	0.06	0.00	116	
4	Clayey Sand	9	0.09	-0.11	120	
5	Silty Fine Sand with Shell Fragments	81	0.08	0.00	116	

G continued

Layer	Boring Log Description	Step 1, continued				Step 2
		Maximum dry unit weight, $\gamma_{dry\ max}$ (pcf)	Average dry unit weight, $\gamma_{dry\ avg}$ (pcf)	water content, w_n	Secondary Consolidation Compression Index, C_α	Initial Thickness of Clay Layer before Secondary Settlement, H_{os} (ft)
1	Sand - Fine, Medium, Tan, Brown, White				0	16.60
2	Clayey Sand				0.003	4.36
3	Sand - Coarse-Fine				0	43.94
4	Clayey Sand				0.003	9.02
5	Silty Fine Sand with Shell Fragments				0	80.92

G continued

Layer	Boring Log Description	Step 3	
		Initial Void Ratio of Clay before Secondary Settlement, e_{os}	Secondary Compression Settlement of Clay Layer, Z_α (ft)
1	Sand - Fine, Medium, Tan, Brown, White		0.00
2	Clayey Sand	0.25	0.02
3	Sand - Coarse-Fine		0.00
4	Clayey Sand	0.25	0.04
5	Silty Fine Sand with Shell Fragments		0.00

Step 4: Total Estimated Secondary Compression Settlement for G 0.06 ft

HDR Computation

Project:	Steelfield Road Landfill	Computed By:	KP	Date:	1/25/2019
Subject:	Cells 6 & 7 Expansion	Checked By:	TY	Date:	2/9/2019
Task:	Settlement Calculation - Secondary Compression Settlement	Sheet:		Of:	

H

Layer	Boring Log Description	From Previous Calculations				Step 1
		Initial Layer Thickness, H_o (ft)	Elastic Layer Settlement, Z_e (ft)	Primary Consolidation Settlement of Clay, Z_c (ft)	Wet Unit Weight, γ_{wet} (pcf)	Minimum dry unit weight, $\gamma_{dry\ min}$ (pcf)
1	Sand - Fine, Medium, Tan, Brown, White	19	0.21	0.00	116	
2	Clayey Sand	4	0.34	-0.26	120	
3	Sand - Coarse-Fine	44	0.12	0.00	116	
4	Clayey Sand	9	0.16	-0.02	120	
5	Clayey Sand	80	0.14	0.00	116	
5	0	0	0.00	0.00	0	

H continued

Layer	Boring Log Description	Step 1, continued				Step 2
		Maximum dry unit weight, $\gamma_{dry\ max}$ (pcf)	Average dry unit weight, $\gamma_{dry\ avg}$ (pcf)	water content, w_n	Secondary Consolidation Compression Index, C_α	Initial Thickness of Clay Layer before Secondary Settlement, H_{os} (ft)
1	Sand - Fine, Medium, Tan, Brown, White					
2	Clayey Sand				0.003	3.92
3	Sand - Coarse-Fine					
4	Clayey Sand				0.003	8.86
5	Clayey Sand					
5	0					

H continued

Layer	Boring Log Description	Step 3	
		Initial Void Ratio of Clay before Secondary Settlement, e_{os}	Secondary Compression Settlement of Clay Layer, Z_α (ft)
1	Sand - Fine, Medium, Tan, Brown, White		0.00
2	Clayey Sand	0.25	0.02
3	Sand - Coarse-Fine		0.00
4	Clayey Sand	0.250	0.04
5	Clayey Sand		
5	0		

Step 4: Total Estimated Secondary Compression Settlement for H 0.06 ft

HDR Computation

Project:	Steelfield Road Landfill	Computed By:	KP	Date:	1/25/2019
Subject:	Cells 6 & 7 Expansion	Checked By:	TY	Date:	2/9/2019
Task:	Settlement Calculation - Secondary Compression Settlement	Sheet:		Of:	

I

Layer	Boring Log Description	From Previous Calculations				Step 1
		Initial Layer Thickness, H_o (ft)	Elastic Layer Settlement, Z_e (ft)	Primary Consolidation Settlement of Clay, Z_c (ft)	Wet Unit Weight, γ_{wet} (pcf)	Minimum dry unit weight, $\gamma_{dry\ min}$ (pcf)
1	Sand - Fine, Medium, Tan, Brown, White	17	0.10	0.00	116	
2	Clayey Sand	6	0.16	-0.85	120	
3	Sand - Coarse-Fine	50	0.07	0.00	116	
4	Clayey Sand	76	0.06	0.00	120	
5	0	0	0.00	0.00	0	

I continued

Layer	Boring Log Description	Step 1, continued				Step 2
		Maximum dry unit weight, $\gamma_{dry\ max}$ (pcf)	Average dry unit weight, $\gamma_{dry\ avg}$ (pcf)	water content, w_n	Secondary Consolidation Compression Index, C_α	Initial Thickness of Clay Layer before Secondary Settlement, H_{os} (ft)
1	Sand - Fine, Medium, Tan, Brown, White					
2	Clayey Sand				0.003	6.69
3	Sand - Coarse-Fine					
4	Clayey Sand					
5	0					

I continued

Layer	Boring Log Description	Step 3	
		Initial Void Ratio of Clay before Secondary Settlement, e_{os}	Secondary Compression Settlement of Clay Layer, Z_α (ft)
1	Sand - Fine, Medium, Tan, Brown, White		0.00
2	Clayey Sand	0.250	0.03
3	Sand - Coarse-Fine	0.000	0.00
4	Clayey Sand	0.000	0.00
5	0		

Step 4: Total Estimated Secondary Compression Settlement for I 0.03 ft

HDR Computation

Project:	Steelfield Road Landfill	Computed By:	KP	Date:	1/25/2019
Subject:	Cells 6 & 7 Expansion	Checked By:	TY	Date:	2/9/2019
Task:	Settlement Calculation - Secondary Compression Settlement	Sheet:		Of:	

J

Layer	Boring Log Description	From Previous Calculations				Step 1
		Initial Layer Thickness, H_o (ft)	Elastic Layer Settlement, Z_e (ft)	Primary Consolidation Settlement of Clay, Z_c (ft)	Wet Unit Weight, γ_{wet} (pcf)	Minimum dry unit weight, $\gamma_{dry\ min}$ (pcf)
1	Sand - Fine, Medium, Tan, Brown, White	19	0.28	0.00	116	
2	Clayey Sand	6	0.41	-0.19	120	
3	Sand - Coarse-Fine	49	0.17	0.00	116	
4	Silty Fine Sand with Clary Fragments	76	0.17	0.00	116	

J continued

Layer	Boring Log Description	Step 1, continued				Step 2
		Maximum dry unit weight, $\gamma_{dry\ max}$ (pcf)	Average dry unit weight, $\gamma_{dry\ avg}$ (pcf)	water content, w_n	Secondary Consolidation Compression Index, C_α	Initial Thickness of Clay Layer before Secondary Settlement, H_{os} (ft)
1	Sand - Fine, Medium, Tan, Brown, White					
2	Clayey Sand				0.003	5.78
3	Sand - Coarse-Fine					
4	Silty Fine Sand with Clary Fragments					

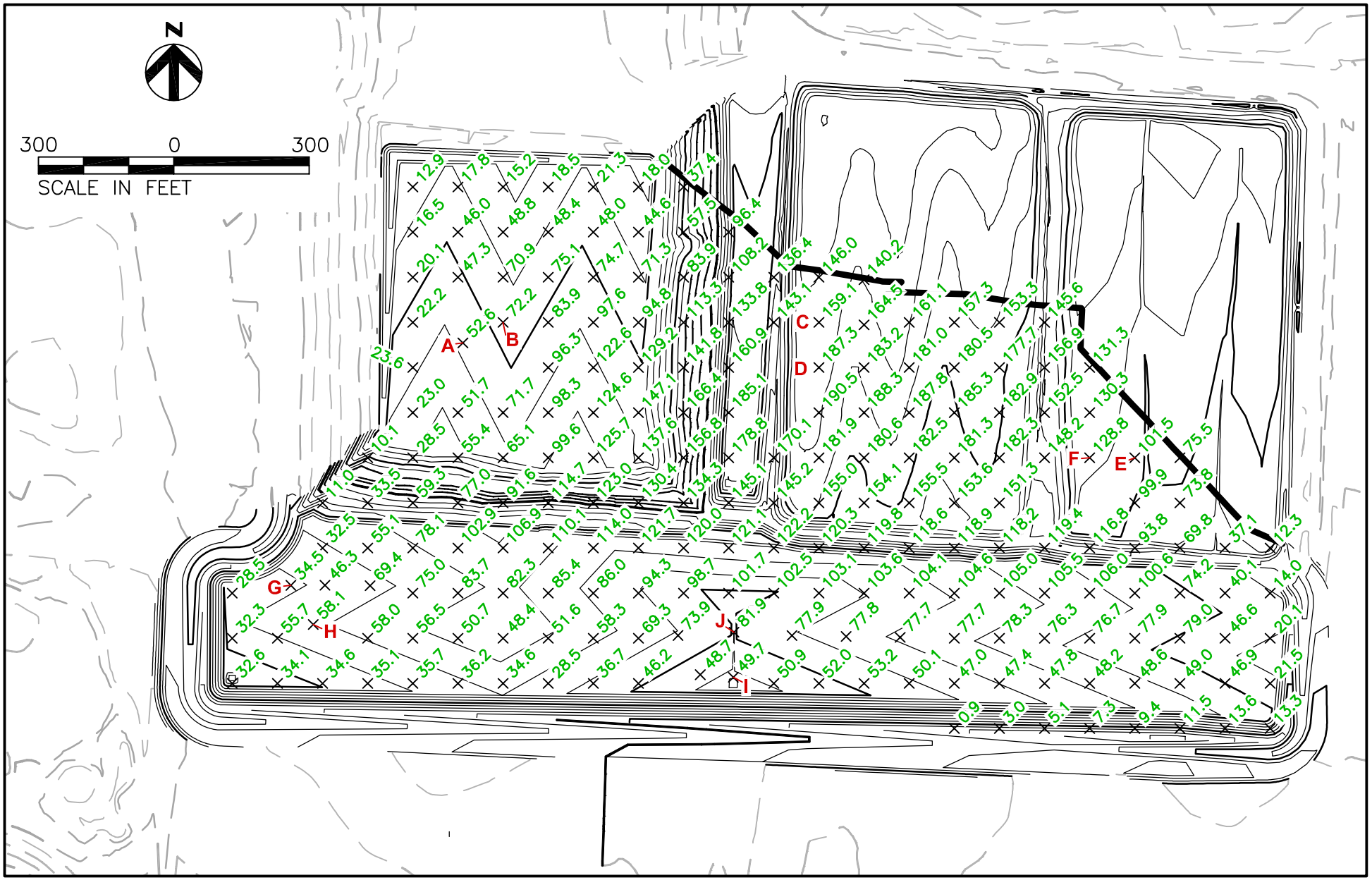
J continued

Layer	Boring Log Description	Step 3	
		Initial Void Ratio of Clay before Secondary Settlement, e_{os}	Secondary Compression Settlement of Clay Layer, Z_α (ft)
1	Sand - Fine, Medium, Tan, Brown, White		
2	Clayey Sand	0.250	0.03
3	Sand - Coarse-Fine		
4	Silty Fine Sand with Clary Fragments		

Step 4: Total Estimated Secondary Compression Settlement for J 0.03 ft

Attachment C-2

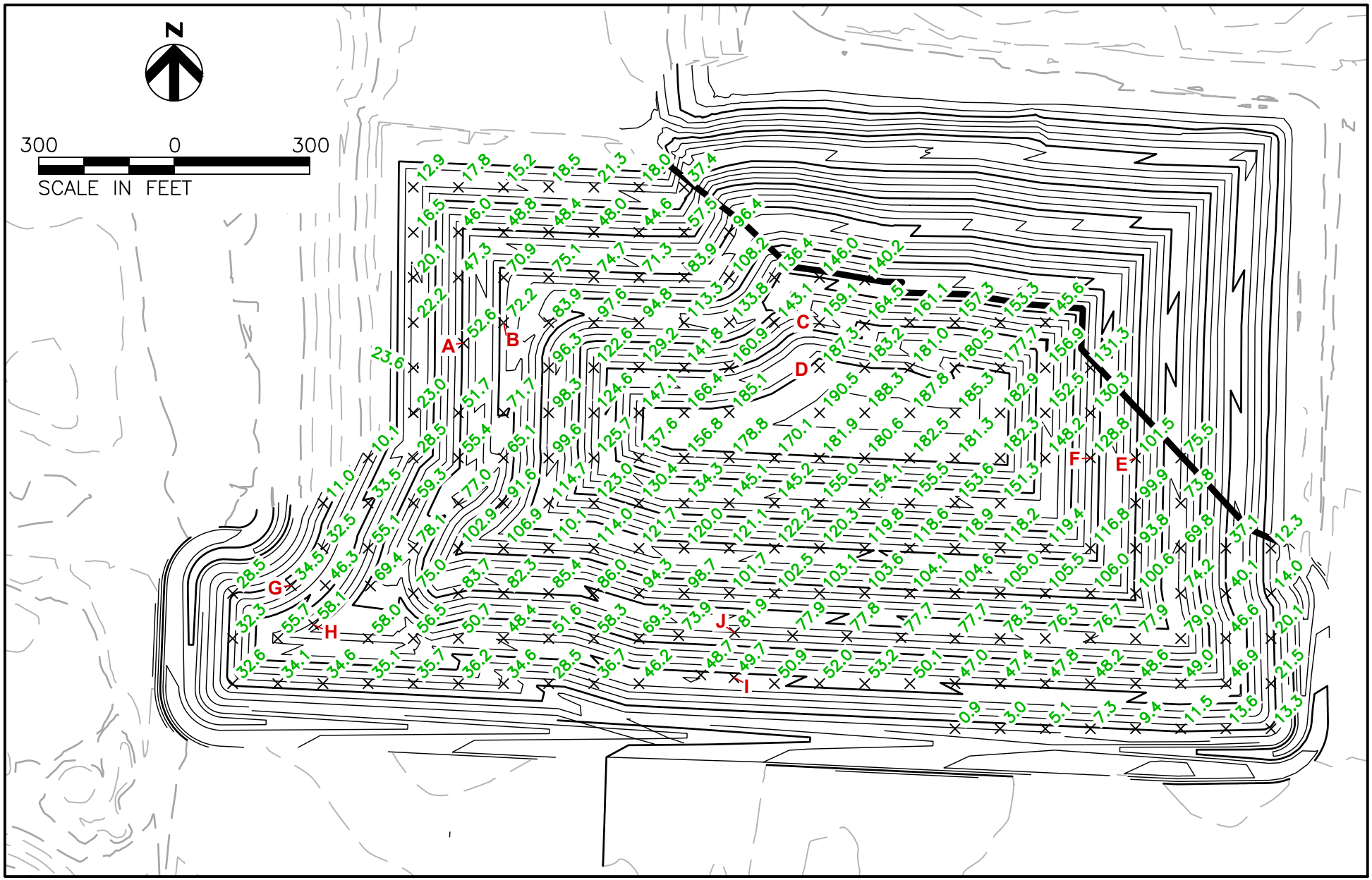
Selected Locations for Settlement Analysis



PROJECT TITLE
STEELFIELD ROAD LANDFILL
PRELIMINARY DESIGN BUILD BID - BAY COUNTY, FLORIDA

SHEET TITLE
SETTLEMENT ANALYSIS
BASE GRADES SHOWING WASTE THICKNESS

PROJECT NUMBER	10150537
PROJECT MANAGER	M. ROBERTS, P.E.
DATE	FEB. 2019
REFERENCE SHEET	
REFERENCE DOCUMENT	
EXHIBIT NUMBER	FIGURE 1



PROJECT TITLE
STEELFIELD ROAD LANDFILL
PRELIMINARY DESIGN BUILD BID - BAY COUNTY, FLORIDA

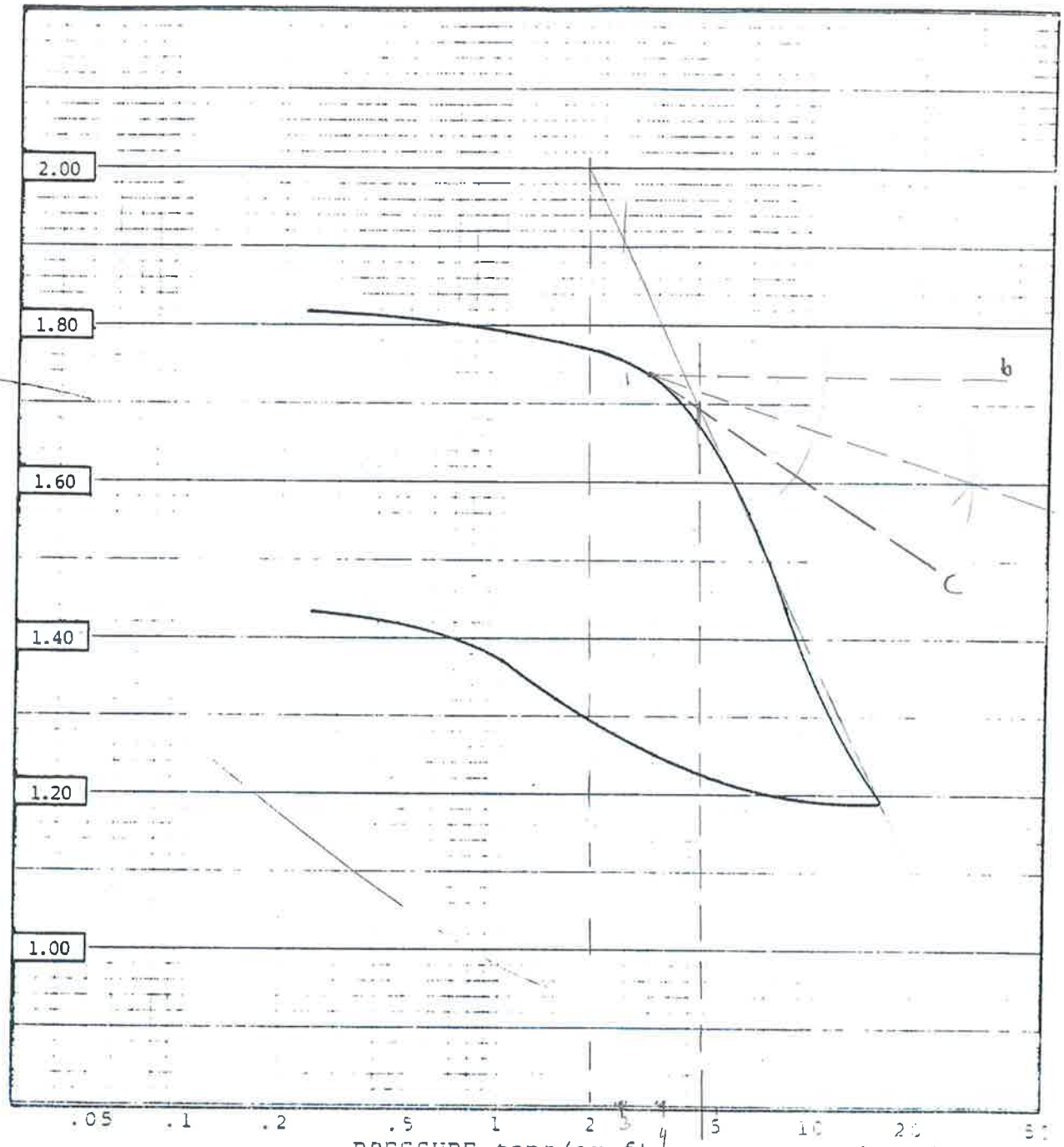
SHEET TITLE
SETTLEMENT ANALYSIS
FINAL COVER SHOWING WASTE THICKNESS

PROJECT NUMBER	10150537
PROJECT MANAGER	M. ROBERTS, P.E.
DATE	FEB. 2019
REFERENCE SHEET	
REFERENCE DOCUMENT	
EXHIBIT NUMBER	FIGURE 2

Attachment C-3

Supporting Data

upper clay layer (Refer to Section on 1986 permit App)



$C_c = \frac{2 - 1.7}{\log\left(\frac{4.75}{2}\right)} = 0.80$

$P_c = 4.75 \text{ tons/sq ft}$

TEST DATA

BORING NUMBER	B-7
SAMPLE NUMBER	US-1
DEPTH (FEET)	55-57 Approx Elev -7
CLASSIFICATION	CH
INITIAL MOISTURE (%)	77
INITIAL DRY DENSITY (LB/CU.FT.)	52
SPECIFIC GRAVITY	2.36
FINAL MOISTURE (%)	
LIQUID LIMIT (%)	90
PLASTIC LIMIT (%)	32

VOID RATIO - LOG PRESSURE

Bay County Landfill
off Steelfield Road

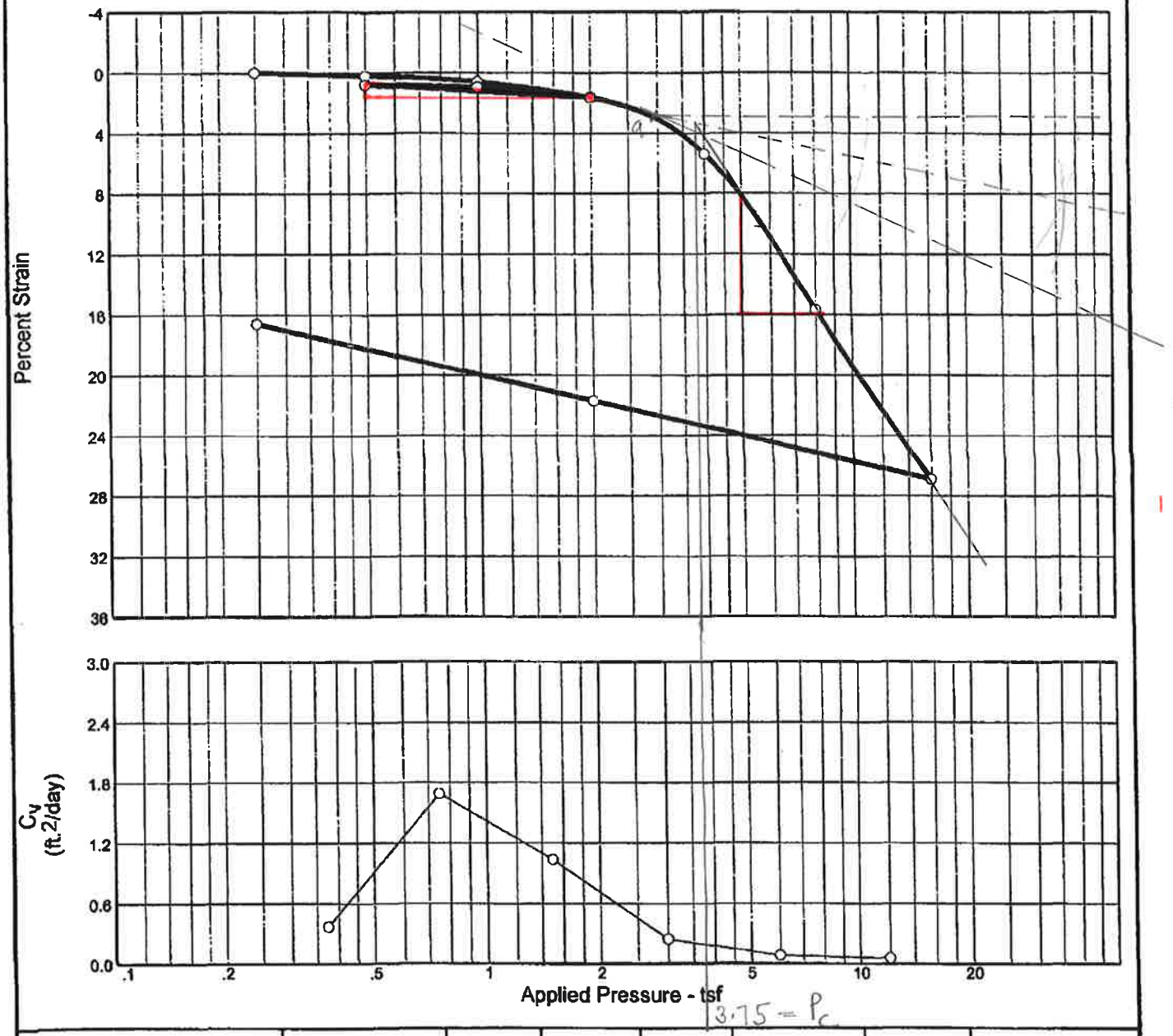
FILE NO.	DATE	DRAWN BY
F-86-042	7/7/86	MKV

SOUTHERN EARTH SCIENCES, INC.
CONSULTING GEOTECHNICAL ENGINEERS

PLATE I

Upper clay layer

CONSOLIDATION/SWELL TEST REPORT



Natural		Dry Dens. (pcf)	LL	PI	Sp. Gr.	P_c (tsf)	C_c	Initial Void Ratio
Saturation	Moisture							
95.0 %	148.6 %	32.5	192	90	2.80	3.88	2.00	4.383

MATERIAL DESCRIPTION	USCS	AASHTO
Dark Brown fat CLAY with wood and organics (OH)	(OH)	

Project No. P10-0285 **Client:**

Project: Steelfield Landfill

Source: C-1 **Elev./Depth:** 20.5-22.5

CONSOLIDATION/SWELL TEST REPORT

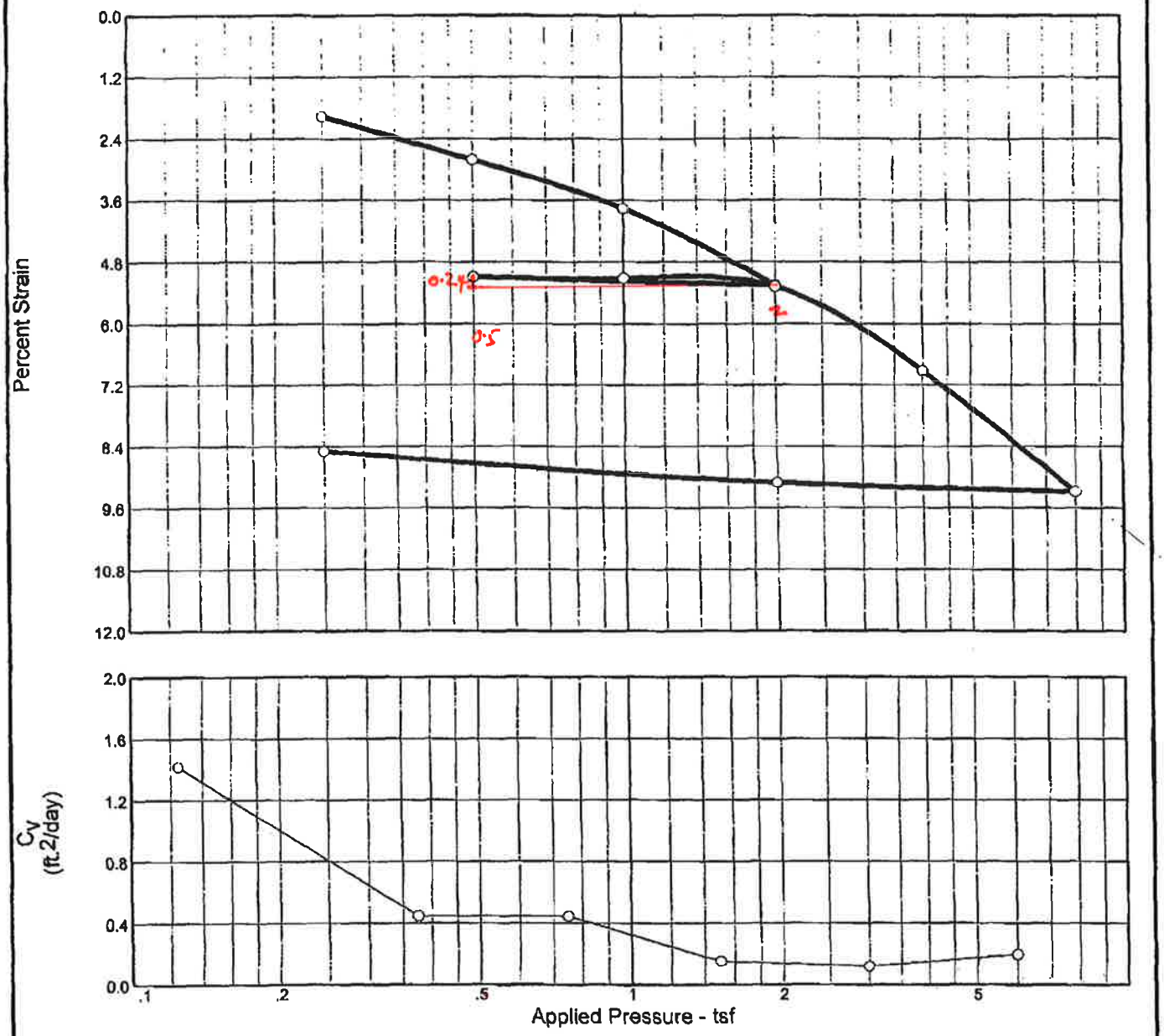
Southern Earth Sciences, Inc.

Remarks:
ASTM D2435

$C_c = \frac{0.8\,(1+4.383)}{100 \times \log\left(\frac{12}{0.5}\right)} + 0.032 = 0.071$

Lower clay layer

CONSOLIDATION/SWELL TEST REPORT



Natural		Dry Dens. (pcf)	LL	PI	Sp. Gr.	P_c (tsf)	C_c	Initial Void Ratio
Saturation	Moisture							
99.3 %	37.7 %	82.4	42	12	2.65	4.18	0.16	1.007

MATERIAL DESCRIPTION	USCS	AASHTO
Gray Lean CLAY with sand and shell fragments (CL)	(CL)	

Project No. P10-0285 **Client:**

Project: P10-0285

Source: B-1 **Elev./Depth:** 141.5-143.5

CONSOLIDATION/SWELL TEST REPORT

Southern Earth Sciences, Inc.

Remarks:
ASTM D2435

$$C_r = \frac{0.0024\,(1+1.007)}{\log(2/0.5)}$$

$$= 0.003$$

SOUTHERN EARTH SCIENCES, INC.

CONSULTING GEOTECHNICAL ENGINEERS

SUMMARY OF PERMEABILITY TESTS

FILE NO. F-86-042

LOCATION DESCRIPTION			CLASSIFICATION	TYPE OF TEST *	SOIL TYPE **	DRY DENSITY(PCF)	DEGREE OF SATURATION	COEFF. OF PERMEABILITY (CM/SEC)
TEST NO.	TEST HOLE NO.	DEPTH (FEET)						
1	B-1	40-41.5	Slightly Clayey Fine Sand with Clay lenses	C	R	102.6		4.0×10^{-7}
2	B-2	45-46.5	Medium to Fine Sand	C	R	103.6		2.6×10^{-4}
3	B-2	55-56.5	Slightly Sandy Silty Clay	C	R	79.2		6.34×10^{-8}
4	B-2	100-101.5	Slightly Clayey Silty Sand with shell frag.	C	R	93.2		2.1×10^{-5}
5	B-3	35-36.5	Fine Sand	C	R	97.2		1.6×10^{-4}
6	B-3	40-41.5	Clayey Sand with thin clay lenses	C	R	104.1		1.5×10^{-6}
7	B-4	20-21.5	Slightly Silty Fine Sand	C	R	102.5		5.8×10^{-4}
8	B-4	30-31.5	Silty Orange Fine Sand	C	R	82.2		3.8×10^{-6}
9	B-4	100-101.5	Clay with Sand lenses	C	R	86.9		2.5×10^{-7}
10	B-5	25-26.5	Medium to Fine Sand	C	R	95.9		1.4×10^{-3}
11	B-5	30-31.5	Silty Fine Sand with trace clay	C	R	102.6		3.1×10^{-5}
12	B-5	35-36.5	Medium to Fine Sand	C	R	88.8		1.5×10^{-4}
13	B-5	60-61.5	Silty Clay	C	R	52.3		7.7×10^{-8}
14	B-6	40-41.5	Fine Sand	C	R	96.9		5.1×10^{-5}

* A CONSTANT HEAD - TRIAXIAL CELL
B CONSTANT HEAD - BRASS PERMEAMETER
C FALLING HEAD - BRASS PERMEAMETER

** U UNDISTURBED NATURAL GROUND
R REMOLDED NATURAL GROUND

Young's modulus of soil, E_s, and the constrained modulus of soil, M_s, where E_s and D_s are related through Poisson's ratio of soil, v_s, by

$$M_s = \frac{E_s \cdot (1 - v_s)}{(1 + v_s)(1 - 2 \cdot v_s)} \tag{9.27}$$

where M_s = constrained modulus of soil, lb/ft^2 or kN/m^2;
E_s = elastic modulus of soil, lb/ft^2 or kN/m^2; and
v_s = Poisson's ratio of soil.

The studies and analyses by Neilson (1967), Allgood and Takahashi (1972), and Hartely and Duncan (1987) indicated that for

$$E' = k \cdot M_s \tag{9.28}$$

the value of k may vary from 0.7 to 2.3. Using $k = 1.5$ as a representative value and $v_s = 0.3$, in addition to combining Equations 9.27 and 9.28 yields the following relationship between the elastic modulus of the pipe and soil (Selig, 1990):

$$E' = 2 \cdot E_s \tag{9.29}$$

The values of elastic parameters, E_s and v_s, can be found in Table 9.5 according to different percents of density from a standard Proctor compaction test (ASTM D698).

TABLE 9.5 Elastic Soil Parameters (Selig, 1990)

Soil Type	Stress Level		85% Standard Density			95% Standard Density		
			E_s			E_s		
	psi	kPa	psi	MPa	v_s	psi	MPa	v_s
SW, SP, GW, GP	1	7	1,300	9	0.26	1,600	11	0.40
	5	35	2,100	14	0.21	4,100	28	0.29
	10	70	2,600	18	0.19	6,000	41	0.24
	20	140	3,300	23	0.19	8,600	59	0.23
	40	280	4,100	28	0.23	13,000	90	0.25
	60	420	4,700	32	0.28	16,000	110	0.29
GM, SM, ML, and GC, SC with < 20% fines	1	7	600	4	0.25	1,800	12	0.34
	5	35	700	5	0.24	2,500	17	0.29
	10	70	800	6	0.23	2,900	20	0.27
	20	140	850	6	0.30	3,200	22	0.29
	40	280	900	6	0.38	3,700	25	0.32
	60	420	1,000	7	0.41	4,100	28	0.35
CH CL, MH, GC, SC	1	7	100	1	0.33	400	3	0.42
	5	35	250	2	0.29	800	6	0.35
	10	70	400	3	0.28	1,100	8	0.32
	20	140	600	4	0.25	1,300	9	0.30
	40	280	700	5	0.35	1,400	10	0.35
	60	420	800	6	0.40	1,500	10	0.38

Source: Reference 3

9.4.2 Pipe Wall Buckling

Buckling can occur
flexible pipes subject
pressures in compact
conduit (e.g., high v
resisting buckling.

Most conduits
resistance. An exact
elastic medium entai
of uncertainties in t
solution is not neces
cal formula for comp

Where,
P_{cr} = critical
E' = modul
μ = Poisso
E = modul
I = mome
in^4/in
r = mean

Because $I = t^3$,

where

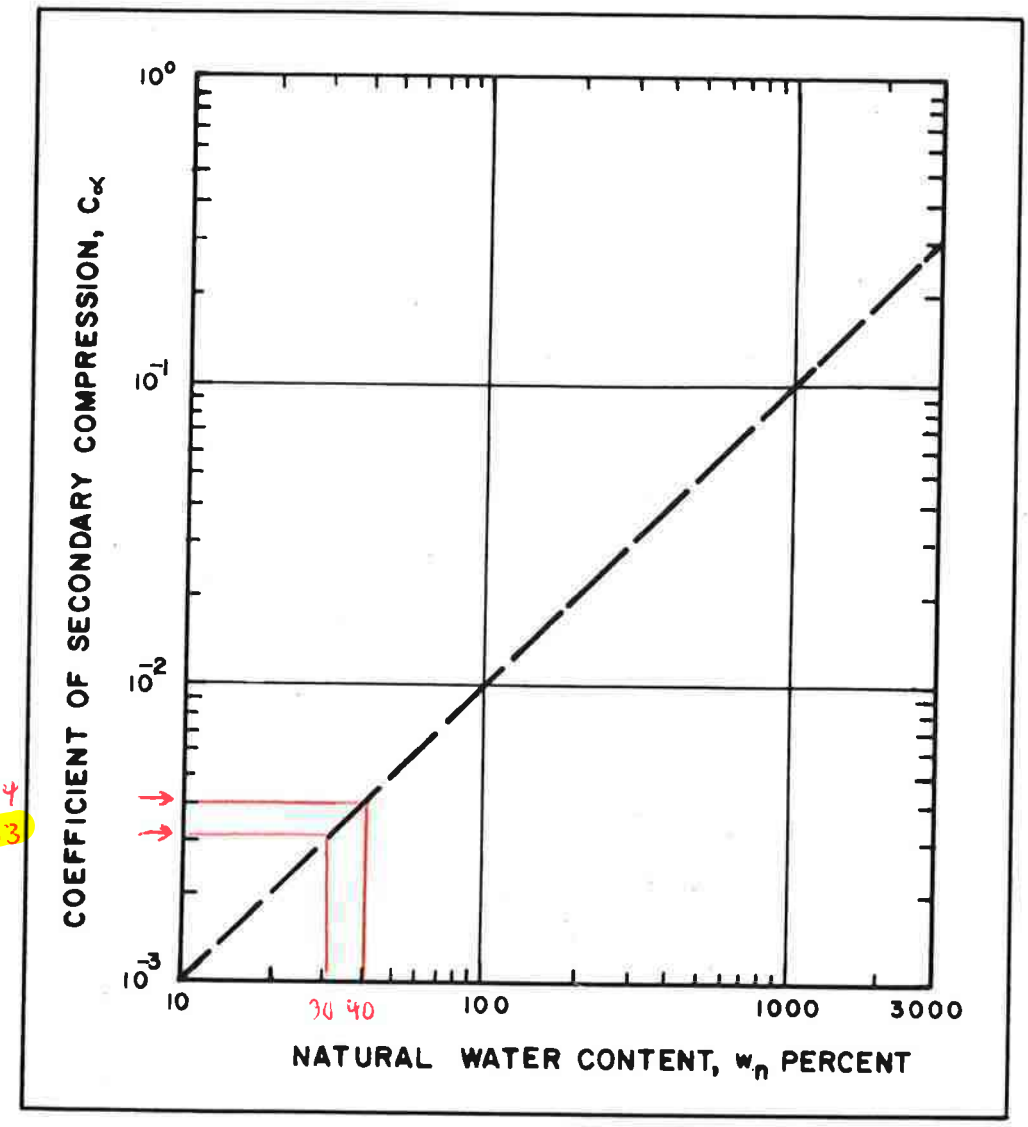


FIGURE 16
Coefficient of Secondary Compression as Related to
Natural Water Content

Source: Reference 1

TABLE 6
Typical Values of Soil Index Properties

	Particle Size and Gradation				Voids[1]					Unit Weight[2] (lb./cu.ft.)						
	Approximate Size Range (mm)		Approx. D_{10} (mm)	Approx. Range Uniform Coefficient C_u	Void Ratio			Porosity (%)		Dry Weight			Wet Weight		Submerged Weight	
	D_{max}	D_{min}			e_{max} loose	e_{cr}	e_{min} dense	n_{max} loose	n_{min} dense	Min loose	100% Mod. AASHO	Max dense	Min loose	Max dense	Min loose	Max dense
GRANULAR MATERIALS																
Uniform Materials																
a. Equal spheres (theoretical values)	-	-	-	1.0	0.92	-	0.35	47.6	26	-	-	-	-	-	-	-
b. Standard Ottawa SAND	0.84	0.59	0.67	1.1	0.80	0.75	0.50	44	33	92	-	110	93	131	57	69
c. Clean, uniform SAND (fine or medium)	-	-	-	1.2 to 2.0	1.0	0.80	0.40	50	29	83	115	118	84	136	52	73
d. Uniform, inorganic SILT	0.05	0.005	0.012	1.2 to 2.0	1.1	-	0.40	52	29	80	-	118	81	136	51	73
Well-graded Materials																
a. Silty SAND	2.0	0.005	0.02	5 to 10	0.90	-	0.30	47	23	87	122	127	88	142	54	79
b. Clean, fine to coarse SAND	2.0	0.05	0.09	4 to 6	0.95	0.70	0.20	49	17	85	132	138	86	148	53	86
c. Micaceous SAND	-	-	-	-	1.2	-	0.40	55	29	76	-	120	77	138	48	76
d. Silty SAND & GRAVEL	100	0.005	0.02	15 to 300	0.85	-	0.14	46	12	89	-	146[3]	90	155[3]	56	92
MIXED SOILS																
Sandy or Silty CLAY	2.0	0.001	0.003	10 to 30	1.8	-	0.25	64	20	60	130	135	100	147	38	85
Skip-graded Silty CLAY with stones or rk fgmts	250	0.001	-	-	1.0	-	0.20	50	17	84	-	140	115	151	53	89
Well-graded GRAVEL, SAND, SILT & CLAY mixture	250	0.001	0.002	25 to 1000	0.70	-	0.13	41	11	100	140	148[4]	125	156[4]	62	94
CLAY SOILS																
CLAY (30%-50% clay sizes)	0.05	0.5μ	0.001	-	2.4	-	0.50	71	33	50	105	112	94	133	31	71
Colloidal CLAY (-0.002 mm: 50%)	0.01	10Å	-	-	12	-	0.60	92	37	13	90	106	71	128	8	66
ORGANIC SOILS																
Organic SILT	-	-	-	-	3.0	-	0.55	75	35	40	-	110	87	131	25	69
Organic CLAY (30% - 50% clay sizes)	-	-	-	-	4.4	-	0.70	81	41	30		100	81	125	18	62

Source: Reference 1

APPENDIX D
LEACHATE COLLECTION SYSTEM

Leachate Collection System

Leachate Collection System Design 2
Containment Systems 2
HELP Model Scenarios and Input Data 3
Material Properties and Landfill Geometry 4
Model Outputs and Conclusions 5

Attachments

D-1 Summary of Model Input Data and Results

D-2 Support Data

D-3 Geocomposite Hydraulic Conductivity Determination

D-4 HELP Model Output Files (Scenarios 1-4)

D-5 Final Cover Hydraulic Conductivity Determination

Leachate Collection System

The Universität Hamburg updated HELP model Version 3.95D, which was used as part of the design of the leachate collection systems for the Steelfield Road Landfill (SFRL) expansion. The site is located in Panama City Beach, FL approximately 100 miles southwest of Tallahassee. This section presents the design assumptions, decisions, background data, and calculations for the water balance model. The section outlines efforts to establish leachate generation rates, leachate leakage rates and maximum hydraulic heads on the liner. The leachate generation rates established per acre should be used to design the leachate collection and removal system (LCRS).

Containment Systems

The SFRL proposed footprint will receive a double liner system consistent with FAC 62-701.400(3)(c). The alternative landfill liner will consist of the following components, from top to bottom:

- 24 inches of $1x10^{-3}$ cm/s protective cover soil
- 300-mil drainage geocomposite
- 60-mil HDPE geomembrane
- 300-mil drainage geocomposite
- 60-mil HDPE geomembrane
- Geosynthetic Clay Liner (GCL)

The final cover is consistent with the prescribed final cover and will consist of the following materials from top to bottom:

- 6 inches of topsoil layer
- 18 inches of erosion control soil with $1x10^{-4}$ cm/s (maximum) permeability
- 300 mil drainage geocomposite
- 40 mil low density polyethylene (LLDPE) geomembrane
- 12-inch intermediate cover soil

The leachate collection system design intends to maintain the leachate head less than one foot above the liner or within the thickness of the drainage layer, whichever is less, except in the sump.

HELP Model Scenarios and Input Data

Each scenario was modeled as a 1-acre area. A major goal for the modeling was to demonstrate that the drainage layer capacity is not exceeded. The second aim of the modeling was to estimate leachate production. The table provided in Attachment D-1 summarizes the model input data, and the results of the scenarios. The Attachment D-2 provide supporting data used in the HELP model. The bottom liner hydraulic conductivity is determined as shown in Attachment D-3. HELP model output files for Scenarios 1-4 are presented in Attachment D-4.

- Scenario 1 modeled a 10-foot depth of MSW on the floor of prepared liner based on 5-year simulation period. The purpose of this model was to demonstrate that the head on the liner would be less than the thickness of the geocomposite when there was a relatively shallow lift of waste.
- Scenario 2 modeled 60 feet (720-inches) of average MSW depth with intermediate cover soil. The maximum thickness of MSW measured from the top of protective cover to top of intermediate cover is 145 feet. However, since the landfill is a pyramid shape, 60 feet is the average depth or typical depth of MSW in the proposed landfill footprint. The average depth was obtained by dividing fill volume by expansion footprint area. The objective of the HELP model is to estimate leachate generation rate based on a 1-acre area (i.e. gallons/acre). The total leachate generation is then estimated by multiplying the total waste footprint by leachate generation rate. The entire MSW column is assumed to have an initial moisture content of 30% which is the default moisture content of MSW within HELP model. The model was simulated over a 30 year period. The purpose of this model was to demonstrate that the head on the liner would be less than the thickness of the geocomposite.
- Scenario 3 modeled 102 feet (1224-inches) average MSW depth with intermediate cover soil within existing liner footprint. The average depth is obtained by dividing total waste volume within existing liner footprint divided by the area of existing liner footprint. The purpose is to confirm that existing leachate collection system is capable of handling leachate due to proposed expansion. The HELP model is used to estimate leachate generation rate based on a 1-acre area (i.e. gallons/acre). The total leachate generation is then estimated by multiplying the total existing waste footprint by leachate generation rate. The entire MSW column is assumed to have an initial moisture content of 30% which is the default moisture content of MSW within HELP model. The model was simulated over a 30 year period. The purpose of this model was to demonstrate that the head on the liner would be less than the thickness of the existing geocomposite. Note that 1×10^{-4} cm/s permeability is used for existing protective cover soil.
- Scenario 4 modeled leachate production after closure based on average MSW thickness for the entire footprint which is 82 feet (984 inches). The drainage layer permeability for the existing landfill is used in the model. The HELP model confirmed that the drainage layer capacity of the final cover is not exceeded. The model was simulated over a 30 year post closure period.

The HELP Model requires weather data for the analysis. The data may be manually input or synthetically generated. The synthetic generation option provides parameters based on many locations around the country. The closest location to the site available for synthetically generated weather data is Tallahassee, Florida. The synthetically generated temperature, solar radiation and evapotranspiration data was reviewed and used without modification in all scenarios.

Material Properties and Landfill Geometry

The floor of the facility will slope towards the leachate collection sumps with 2.0% minimum grade. The maximum flow path for leachate in the leachate drainage layer geocomposite will be 171 feet for proposed cells and is 165 feet for the existing footprint, at which point the leachate will enter an interceptor trench containing a perforated pipe surrounded by gravel for conveyance to the sumps/manholes.

The final grades of the SFRL will average 5% on the top deck and 33% on the side slopes. The sideslopes consist of 33% slope with terraces at every 40 ft vertical interval. A 300 mil thick geocomposite is included in both the top deck and side slope final cover design to convey subsurface drainage.

Material properties were set as follows:

Initial moisture content for MSW after placement at the cell is assumed at approximately 30% which is the HELP model default for MSW.

- Refer to Attachment D-2 for physical properties of materials.
- Default model parameters were used for the final cover and intermediate cover layers.
- Landfill base liner and final cover liner was modeled as material texture 35, HDPE and 36, LLDPE, respectively.
- Pinhole density for the membrane liners was set at 2 per acre.
- Installation defects were set at 2 per acre for the membrane liners.
- Membrane liner placement quality was assumed to be “good”.
- Landfill base leachate drainage layer was modeled as material texture 34, except the transmissivity was modified to reflect select material properties (Refer to Attachment D-3).

A detailed calculation of the hydraulic conductivity required for the bottom liner geocomposite is contained in Attachment D-3. The simplified allowable transmissivity determination equation per GRI GC-8 was used based on the 100 hr measured transmissivity. The maximum overburden pressure, based on 90 pcf density and approximately 82 foot average height is 7,860 psf. The permeability is conservatively estimated based on 15,000 psf maximum loading considering future lateral expansions.

The geocomposite permeability determination for final cover is included with model data and supporting information in Attachment D-5.

Model Outputs and Conclusions

The table provided in Attachment D-1 summarizes the model outputs for each of the scenarios considered. The model demonstrates that the proposed design will comply with applicable design standards. More specifically:

- The peak head on HDPE primary base liner for all scenarios is 0.252 inches (refer to Scenario 1, Peak Daily Values for Years 1 to 5), which is less than the retained thickness of the geocomposite.
- The peak head on the final cover LLDPE is 0.091 inches (Refer to Scenario 4, Peak Daily Values for Years 1 Through 30) less than the thickness of the drainage layer with is 0.29 inches.
- Peak leachate generated during filling is 1774 cf/acre/day.
- Peak leachate generated after closure is 0.014 cf/acre/day.
- Annual average leachate generation after closure is 0.163 cf or almost zero.

The following design criteria were established to prevent exceedance of drainage layer capacity:

- Maximum drainage length on the bottom liner is 171 feet.
- Maximum drainage length necessary on the sideslope of the landfill cover is 127 feet.

Attachment D-1

Summary of Model Input Data and Results

Summary of HELP Model Input Data and Results

Input Data	Scenario 1	Scenario 2	Scenario 3	Scenario 4
Cover thickness (inches)	0	12	12	36
SCS runoff curve number	80.17	63.03	63.03	57.82
Depth of Waste (feet)	10	60	102	82
fraction of area allowing runoff (%)	0	100	100	100
Evaporative zone depth (inches)	6			
Leaf Area Index	0	2		3.5
Area simulated (acres)	1			
MSW hydraulic conductivity, k (cm/sec)	1.0×10^{-3}			
Initial moisture content of MSW	0.2920			
Saturated hydraulic conductivity of geocomposite drainage layer (cm/s)	5.7		5.2	
Bottom liner drainage length with single liner (feet)	171		165	
Bottom HDPE liner thickness (mils)	60			
liner pinhole density (holes/acre)	2			
liner installation defects (holes/acre)	2			
liner placement quality	good			
recirculation? (amount recirculated)	No			
number of years simulated	5	30		
Output Data				
average annual leachate collected in collection layer (ft^3)	105,670	110,073	108,822	0.185
average annual head on primary base liner (inches)	0.021	0.022	0.023	0.000
average annual percolation through primary liner (ft^3)	11608	13093	13421	3.015
peak day leachate collected in collection layer (ft^3)	1774	891	924	0.015
peak day head on primary base liner (inches)	0.252	0.128	0.140	0.007
peak day head on final cover liner (inches)	N/A			0.091
Total average leachate generation from collection + detection (ft^3)	117,278	123,166	122,243	3.200

Attachment D-2

Support Data

Source: HELP Model User Guide for Version 3

DEFAULT SOIL, WASTE, AND GEOSYNTHETIC CHARACTERISTICS

Classification			Total Porosity	Field Capacity	Wilting Point	Saturated Hydraulic Conductivity
HELP	USDA	USCS	vol/vol	vol/vol	vol/vol	cm/sec
1	CoS	SP	0.417	0.045	0.018	1.0×10^{-2}
2	S	SW	0.437	0.062	0.024	5.8×10^{-3}
3	FS	SW	0.457	0.083	0.033	3.1×10^{-3}
4	LS	SM	0.437	0.105	0.047	1.7×10^{-3}
5	LFS	SM	0.457	0.131	0.058	1.0×10^{-3}
6	SL	SM	0.453	0.190	0.085	7.2×10^{-4}
7	FSL	SM	0.473	0.222	0.104	5.2×10^{-4}
8	L	ML	0.463	0.232	0.116	3.7×10^{-4}
9	SiL	ML	0.501	0.284	0.135	1.9×10^{-4}
10	SCL	SC	0.398	0.244	0.136	1.2×10^{-4}
11	CL	CL	0.464	0.310	0.187	6.4×10^{-5}
12	SiCL	CL	0.471	0.342	0.210	4.2×10^{-5}
13	SC	SC	0.430	0.321	0.221	3.3×10^{-5}
14	SiC	CH	0.479	0.371	0.251	2.5×10^{-5}
15	C	CH	0.475	0.378	0.265	1.7×10^{-5}
16	Barrier Soil		0.427	0.418	0.367	1.0×10^{-7}
17	Bentonite Mat (0.6 cm)		0.750	0.747	0.400	3.0×10^{-9}
18	Municipal Waste (900 lb/yd^3 or 312 kg/m^3)		0.671	0.292	0.077	1.0×10^{-3}
19	Municipal Waste (channeling and dead zones)		0.168	0.073	0.019	1.0×10^{-3}
20	Drainage Net (0.5 cm)		0.850	0.010	0.005	$1.0 \times 10^{+1}$
21	Gravel		0.397	0.032	0.013	3.0×10^{-1}
22	L*	ML	0.419	0.307	0.180	1.9×10^{-5}
23	SiL*	ML	0.461	0.360	0.203	9.0×10^{-6}
24	SCL*	SC	0.365	0.305	0.202	2.7×10^{-6}
25	CL*	CL	0.437	0.373	0.266	3.6×10^{-6}
26	SiCL*	CL	0.445	0.393	0.277	1.9×10^{-6}
27	SC*	SC	0.400	0.366	0.288	7.8×10^{-7}
28	SiC*	CH	0.452	0.411	0.311	1.2×10^{-6}
29	C*	CH	0.451	0.419	0.332	6.8×10^{-7}
30	Coal-Burning Electric Plant Fly Ash*		0.541	0.187	0.047	5.0×10^{-5}
31	Coal-Burning Electric Plant Bottom Ash*		0.578	0.076	0.025	4.1×10^{-3}
32	Municipal Incinerator Fly Ash*		0.450	0.116	0.049	1.0×10^{-2}
33	Fine Copper Slag*		0.375	0.055	0.020	4.1×10^{-2}
34	Drainage Net (0.6 cm)		0.850	0.010	0.005	$3.3 \times 10^{+1}$

* Moderately Compacted (Continued)

Attachment D-3

Geocomposite Hydraulic Conductivity Determination

HDR Engineering, Inc.

Job No.		No.	
Project	Steelfield Road Landfill Expansion	Computed	K. Perera
Task	Hydraulic Conductivity and Drainage Layer Capacity	Checked	
		Date	2/4/2019
		Date	

Problem

Determine the hydraulic conductivity and drainage capacity of the geocomposite drainage layer in the bottom liner system.

Transmissivity Calculations

Landfill section from bottom to top: GCL-60-mil HDPE-Double-sided Geocomposite- 60-mil HDPE - Double-sided Geocomposite - MSW

Determine 100-hour transmissivity (Θ_{100}).

Use GSE 300-mil Permanet UL or equivalent (see attached cut sheet with product specifications). The allowable transmissivity is determined based on GRI-GC-8 as below.

$$\Theta_{allow} = \frac{\Theta_{100}}{RF_{CR} * RF_{CC} * RF_{BC}}$$

$$\Theta_{allow} = \Theta_{req} \cdot FS_D$$

$$\Theta_{req} = k \cdot b$$

where,

Θ_{allow}	= minimum allowable transmissivity of geocomposite (cm^2/s)
Θ_{req}	= required transmissivity for site (cm^2/s)
FS_D	= overall factor of safety for drainage
Θ_{100}	= transmissivity after 100 hrs. under expected load (cm^2/s)
RF_{CR}	= reduction factor for creep deformation
RF_{CC}	= reduction factor for chemical clogging
RF_{BC}	= reduction factor for biological clogging
k	= saturated hydraulic conductivity (cm/s)
b	= thickness of geonet (cm)

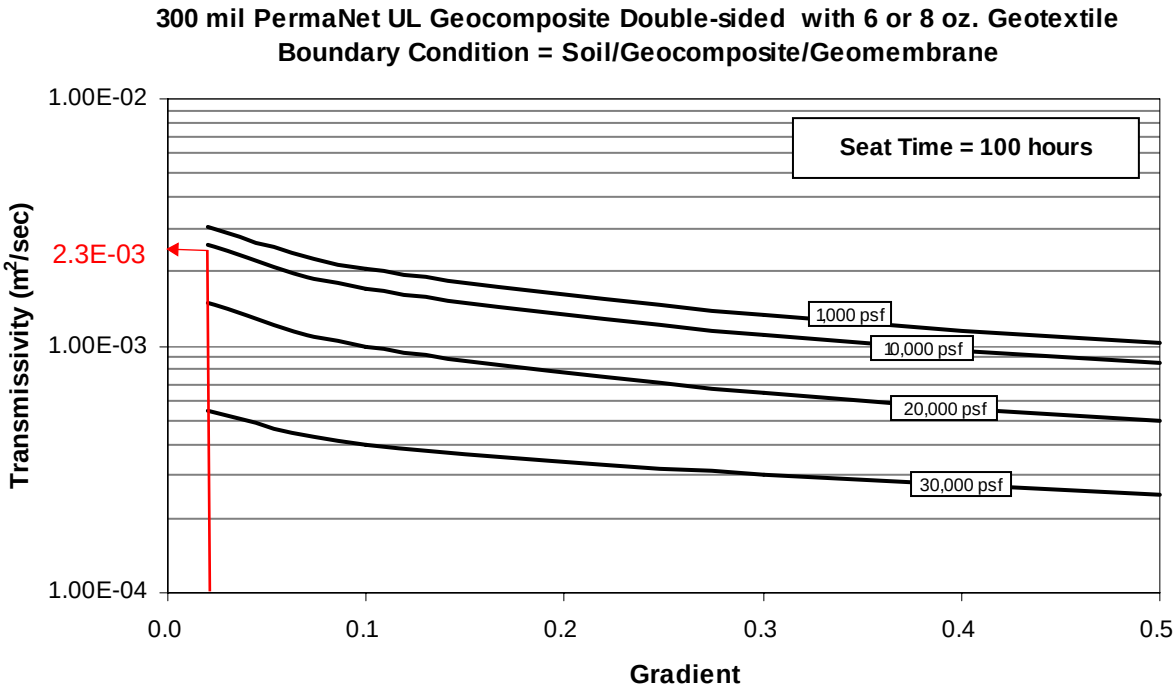
Information based on product data for GSE Permanet 300mil.

Maximum Height of MSW	60.00	ft
Density of MSW, Daily Cover and Intermediate Cover	90	pcf
Combined Final Cover and Protective Cover Thickness	4.00	ft
Density of Cover Soil and Protective Cover	120.00	pcf
Overburden Pressure (with cover soil)	5880.00	psf
Overburden Pressure (without cover soil)	5640.00	psf

Parameters	Geocomposite (300-mil) @ 10,000 psf (1)
Slope	0.02
FS_D (2)	2.0
RF_{CR} (2)	1.10
RF_{CC} (2)	2.00
RF_{BC} (2)	1.2
Θ_{100} (m^2/s)	0.0023
Θ_{allow} (cm^2/s)	8.71
Θ_{req} (cm^2/s)	4.36
Manufactured Thickness (mils)	300
Manufactured Thickness (cm)	0.76
Design Hydraulic Conductivity (cm/s)	5.7
Retained Final Thickness (mils)	270.0

Notes:

1. psf = pounds per square foot.
2. Reduction factors based on HDR experience and information provided by GSE.



A-17 Performance Transmissivity of PermaNet UL Geocomposite under Soil

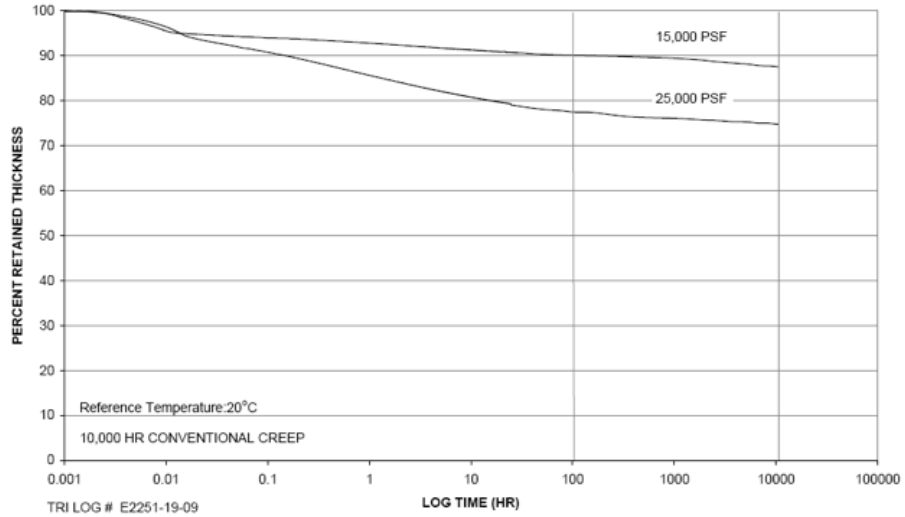


Figure 6 Creep Curves for PermaNet HL Geonet.

Table 6 Creep Reduction Factors for GSE PermaNet HL Geonet from 100 hours to 50 Years.

Stress (psf)	Creep Reduction Factor
15,000	1.12
25,000	1.16

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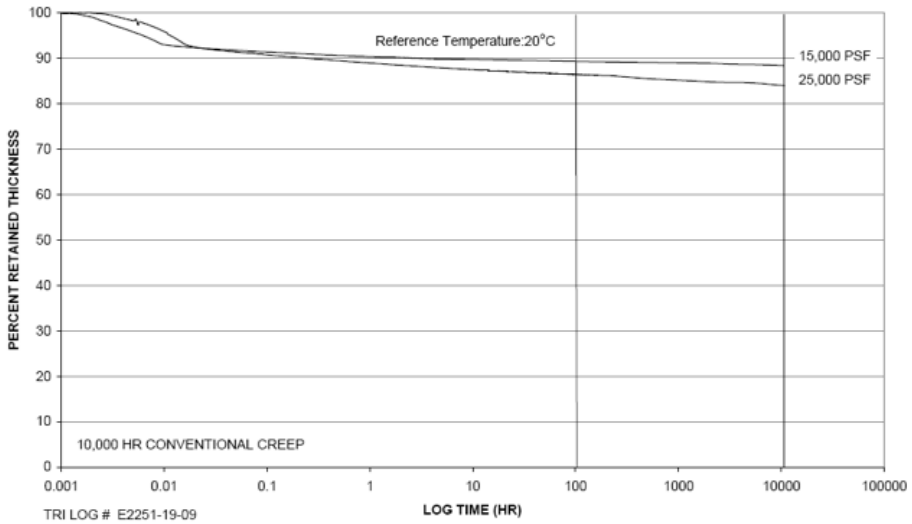

Figure 7 Creep Curves for PermaNet UL Geonet.

Table 7 Creep Reduction Factors for PermaNet UL from 100 hours to 50 Years.

Stress (psf)	Creep Reduction Factor
15,000	1.10
25,000	1.14

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HDR Engineering, Inc.

Job No.				No.	
Project	Steelfield Road Landfill Expansion	Computed	K. Perera	Date	2/4/2019
Task	Hydraulic Conductivity and Drainage Layer Capacity (Existing Landfill)	Checked	TY	Date	2/27/2019

Problem

Determine the hydraulic conductivity and drainage capacity of the geocomposite drainage layer in the bottom liner system.

Transmissivity Calculations

Landfill section from bottom to top: GCL-60-mil HDPE-Double-sided Geocomposite- 60-mil HDPE - Double-sided Geocomposite - MSW

Determine 100-hour transmissivity (Θ_{100}).

Use GSE 300-mil Permanet UL or equivalent (see attached cut sheet with product specifications). The allowable transmissivity is determined based on GRI-GC-8 as below.

$$\Theta_{allow} = \frac{\Theta_{100}}{RF_{CR} * RF_{CC} * RF_{BC}}$$

$$\Theta_{allow} = \Theta_{req} \cdot FS_D$$

$$\Theta_{req} = k \cdot b$$

where,

Θ_{allow}	= minimum allowable transmissivity of geocomposite (cm^2/s)
Θ_{req}	= required transmissivity for site (cm^2/s)
FS_D	= overall factor of safety for drainage
Θ_{100}	= transmissivity after 100 hrs. under expected load (cm^2/s)
RF_{CR}	= reduction factor for creep deformation
RF_{CC}	= reduction factor for chemical clogging
RF_{BC}	= reduction factor for biological clogging
k	= saturated hydraulic conductivity (cm/s)
b	= thickness of geonet (cm)

Information based on product data for SKAPS 330 mil.

Average Height of MSW	82.00	ft
Density of MSW, Daily Cover and Intermediate Cover	90	pcf
Combined Final Cover and Protective Cover Thickness	4.00	ft
Density of Cover Soil and Protective Cover	120.00	pcf
Overburden Pressure (with cover soil)	7860.00	psf
Overburden Pressure (without cover soil)	7620.00	psf

Parameters	Geocomposite (300-mil) @ 15,000 psf (1)
Slope	0.02
FS_D (2)	2.0
RF_{CR} (2)	1.18
RF_{CC} (2)	1.50
RF_{BC} (2)	1.2
Θ_{100} (m^2/s)	0.0018
Θ_{allow} (cm^2/s)	8.66
Θ_{req} (cm^2/s)	4.33
Manufactured Thickness (mils)	330
Manufactured Thickness (cm)	0.84
Design Hydraulic Conductivity (cm/s)	5.2
Retained Final Thickness (mils)	297.0

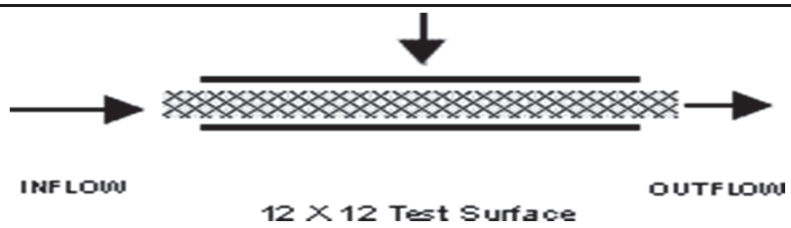
Notes:

1. psf = pounds per square foot.
2. Reduction factors based on HDR experience and information provided by SKAPS.

ASTM D 4716

Client:	T & K Construction, LLC	**Job #**	6358
Project:	Tomoka Farms Road Landfill Phase 2 Area 3, FL		
Product:	TN 330-2-6		

Test Configuration:



Test Information:

Boundary Conditions:	Ottawa Sand Geocomposite 60 Mil HDPE Geomembrane	**Normal Load:** **Gradient:** **Seating Time:** **Flow Direction:**	15000 0.1 100 hours MD

Test Results:

Roll No.	Pressure, psf	Gradient	Transmissivity, m^2/sec
			100 hours
63581010001	15000	0.1	1.84×10^{-3}
63581010215			2.09×10^{-3}
63581010425			1.96×10^{-3}

571 Industrial Parkway, Commerce, GA 30529 Ph : 706-336-7000 Fax : 706-336-7007 Email : contact@skaps.com

March 24, 2014

Mr. Nilay Patel
Skaps Industries
571 Industrial Parkway
Commerce, GA 30529

Email: nilay@skaps.com

Project: Creep Reduction Factor

Dear Mr. Patel:

TRI/Environmental, Inc. (TRI) is pleased to present this final report for accelerated and conventional compression creep testing including the determination of the creep reduction factor in accordance with relevant portions of GRI GC8. All work was registered and performed under TRI log number E2280-93-06. The following sections describe the work and present the results.

INTRODUCTION

Objective

The objective of this effort was to develop long-term compressive creep data and the resulting reduction factor at 40 degrees C under 15,000 psf loading for a Skaps Industries HDPE biplaner geonet product. An estimate of long-term compressive creep was obtained using both the Stepped Isothermal Method (SIM) of time-temperature superposition (TTS) and conventional isothermal compressive creep testing.

MATERIAL AND METHODS

Material

The material tested was a nominal 330-mil high-density polyethylene biplaner geonet product manufactured by Skaps Industries identified as TN330.

Test Equipment used in this study is listed below:

- Testing platforms: Instron Model 5583 load frame under computer control.
- Environmental chamber: TRI Model SIW - stepped isothermal, wide chamber.
- Extensometers: Epsilon Model SW3542-0200-050-ST
- Temperature controller: Watlow Series 982 programmable temperature controller.
- Heating/cooling - Electrical/liquid CO2
- Data acquisition: Instron Bluehill 2 software.

Method

Accelerated compressive creep testing was performed using ASTM D7361-07(2012), *Accelerated Compressive Creep of Geosynthetic Materials Based on Time-Temperature Superposition Using the Stepped Isothermal Method*. Test specimens measured 6 x 6 in (152.4 x 152.4 mm). The temperature regimen employed used a ramp from 20°C to 83°C via 10,000-second steps of 7°C each. All shifting was performed in accordance with ASTM D7361-07.

Conventional isothermal compressive creep testing was performed using ASTM D7406-07(2012), *Standard Test Method for Time-Dependent (Creep) Deformation Under Constant Pressure for Geosynthetic Drainage Products*. Test specimens measured 6 x 6 in (152.4 x 152.4 mm). Testing was conducted at a reference temperature of 40°C.

RESULTS

A presentation of all compressive creep test results may be found in the attached graphs and tables. 10,000-hour creep reduction factors and reduction factors for 10, 20 and 30 year design lives were determined using accelerated creep testing via SIM and conventional creep testing in accordance with GRI GC8 can be found in Table 1.

Table 1: Compressive Creep Reduction Factor @ 40°C

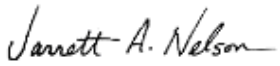
Compressive Load (psf)	Design Life	Retained Thickness @ 100 Hours (%)	Retained Thickness @ Design Life (%)	Calculated RF_{CR} (Design Life)
15,000	10,000 hour	83.87	80.44	1.18
	10 year		78.27	1.32
	20 year		77.48	1.38
	30 year		77.02	1.42
	50 year		76.28	1.47

In addition, the geonet was measured for thickness (ASTM D 5199), density (ASTM D 1505) and mass/unit area (ASTM D 5261) with results as follows.

Thickness (mm)	7.9
Density (kg/m^3)	954
Mass/unit area (kg/m^2)	1.606

CONCLUSION

TRI is pleased to be of service to this work effort. If you have any questions or require any additional information, please contact me at JNelson@tri-env.com or telephone to 512 263 2101.
Sincerely,



Jarrett A. Nelson
Laboratory Director
TRI Geosynthetic Services
www.GeosyntheticTesting.com

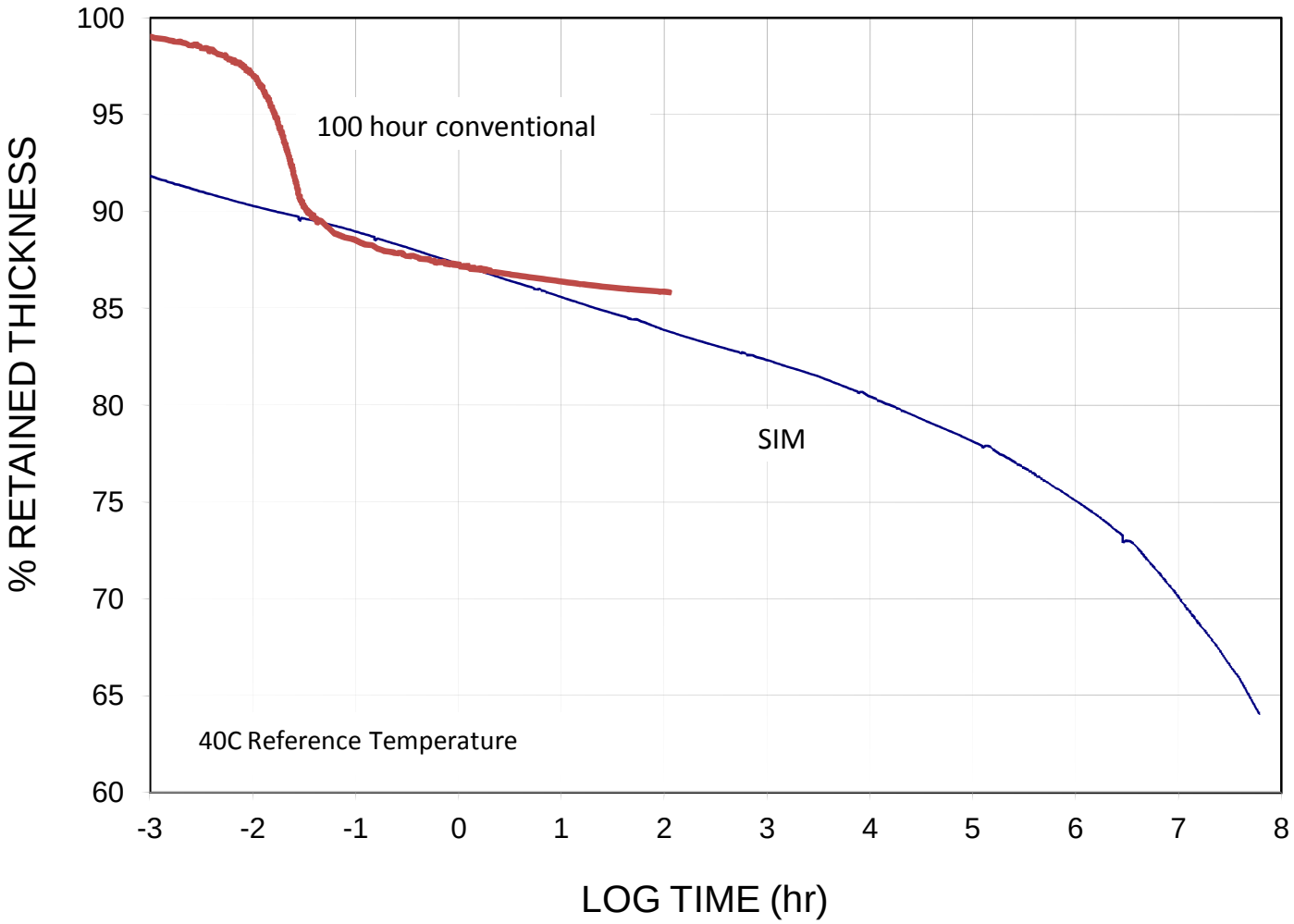
[Attachments]

TRI/Environmental, Inc.
A Texas Research International Company

SUMMARY COMPRESSIVE CREEP PARAMETERS
TRI Client: Skaps Industries

Specimen: TN 330 Test Date: 08-Mar-14 Method: SIM (10^4s, 7C)
Average Creep Stress: 15000.0 psf Initial Thickness (in): 0.312
Mass/Unit Area (g/sq.m): 1605.5

Dwell Seq	t'	t	$(t-t')_i$	Vshift(%)	$logA_T$	Temp	$logA_T/T$
1	0	0.5	0.5	-	-	21.41	-
2	7800	10020	2220	-0.13	0.6532	26.62	0.1254
3	17800	20010	2210	-0.14	0.7413	33.75	0.1039
4	28200	30000	1800	-0.17	0.8300	41.07	0.1134
5	38600	39990	1390	-0.18	0.9274	48.49	0.1251
6	49100	49980	880	-0.15	1.1106	56.35	0.1413
7	59200	59970	770	-0.13	1.1486	63.00	0.1728
8	69300	69960	660	-0.1	1.2111	70.12	0.1699
9	79500	79950	450	-0.1	1.3730	77.33	0.1904
10	89600	89940	340	0.2	1.4861	84.82	0.1986
				.		AVG	0.1430



9063 Bee Caves Road / Austin, TX 78733 / 512-263-2101 / FAX 263-2558 / 800-880-TEST

Summary of Testing for Skaps Industries
TRI Log No. E2280-93-06
Geonet Drainage Study
Product Code: TN330 HDPE Geonet

Creep Reduction Factor @ 40°C

Normal Load (psf)	Time (yrs)	Original Thickness (mils)	Original Thickness (m)	Percent Retained Thickness at 100 hours	Thickness at 100 hours (m)	Long Term Percent Retained Thickness	Long Term Thickness (m)	Mass (g) of 6x6 in specimen	Mass/Area (kg/m^2)	Density (g/cm^3)	Density (kg/m^3)	Original Porosity	RF_{CR}
15000	1.14 (10,000 hrs)	312	0.0079	83.87	0.00665	80.44	0.00637	37.29	1.606	0.954	954	0.7876	1.18
	10	312	0.0079	83.87	0.00665	78.27	0.00620	37.29	1.606	0.954	954	0.7876	1.32
	20	312	0.0079	83.87	0.00665	77.48	0.00614	37.29	1.606	0.954	954	0.7876	1.38
	30	312	0.0079	83.87	0.00665	77.02	0.00610	37.29	1.606	0.954	954	0.7876	1.42
	50	312	0.0079	83.87	0.00665	76.28	0.00605	37.29	1.606	0.954	954	0.7876	1.47

$$RF_{CR} = \left[\frac{\left(t_{CO} / t_{original}\right) - \left(1 - n_{original}\right)}{\left(t_{CR} / t_{original}\right) - \left(1 - n_{original}\right)} \right]^3$$

RF_{CR} = reduction factor for creep
$t_{original}$ = original thickness (m)
t_{CO} = thickness at 100-hours (m)
t_{CR} = thickness at >>100-hours, e.g., at 10,000 hours (m)
$n_{original}$ = original porosity (see Equation 7)

$$n_{original} = 1 - \frac{\mu}{\rho t_{original}}$$

where

μ = mass per unit area (kg/m^2)
ρ = density of the formulation (kg/m^3)

Attachment D-4

HELP Model Output Files (Scenarios 1 - 4)

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**********************************************************************************
**********************************************************************************
**                                                                              **
**                                                                              **
**               HYDROLOGIC EVALUATION OF LANDFILL PERFORMANCE                  **
**                                                                              **
**               HELP Version 3.95 D          (10 August 2012)                  **
**                                developed at                                  **
**         Institute of Soil Science, University of Hamburg, Germany            **
**                                  based on                                    **
**               US HELP MODEL VERSION 3.07  (1 NOVEMBER 1997)                  **
**                   DEVELOPED BY ENVIRONMENTAL LABORATORY                      **
**                     USAE WATERWAYS EXPERIMENT STATION                        **
**                FOR USEPA RISK REDUCTION ENGINEERING LABORATORY               **
**                                                                              **
**                                                                              **
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 TIME:  15.40     DATE:   6.02.2019

 PRECIPITATION DATA FILE:      C:\HDR_Projects\steelfield\Engineering Design\HELP
Model\Filling Stage\Precipitation-5-year.d4
 TEMPERATURE DATA FILE:        C:\HDR_Projects\steelfield\Engineering Design\HELP
Model\Filling Stage\temperature.d7
 SOLAR RADIATION DATA FILE:    C:\HDR_Projects\steelfield\Engineering Design\HELP
Model\Filling Stage\solar radiation.d13
 EVAPOTRANSPIRATION DATA F. 1: C:\HDR_Projects\steelfield\Engineering Design\HELP
Model\Filling Stage\Evapotranspiration.d11
 SOIL AND DESIGN DATA FILE  1: C:\HDR_Projects\steelfield\Engineering Design\HELP
Model\Filling Stage\Initial Lift-10ft.d10
 OUTPUT DATA FILE:             C:\HDR_Projects\steelfield\Engineering Design\HELP
Model\Filling Stage\Initial Lift-10ft.out
 YEARLY OUTPUT DATA FILE:      C:\HDR_Projects\steelfield\Engineering Design\HELP
Model\Filling Stage\Initial Lift-10ft.YR

 COLUMNS OF YEARLY OUTPUT DATA FILE:

      1    DATE OF ULTIMO (yyyy1231, years 2101 to 2200 from weather generator)
      2    PRECIPITATION (INCH)
      3    RUNOFF (INCH)
      4    POTENTIAL EVAPOTRANSPIRATION (INCH)
      5    ACTUAL EVAPOTRANSPIRATION (INCH)
      6    DRAIN #1: LATERAL DRAINAGE FROM LAYER  3 (INCH)
      7    LEAK  #1: PERCOLATION/LEAKAGE THROUGH LAYER  4 (INCH)
      8    DRAIN #2: LATERAL DRAINAGE FROM LAYER  5 (INCH)
      9    LEAK  #2: PERCOLATION/LEAKAGE THROUGH LAYER  6 (INCH)
     10    LEAK  #3: PERCOLATION/LEAKAGE THROUGH LAYER  7 (INCH)
     11    CHANGE IN TOTAL WATER STORAGE (INCH)
     12    CHANGE IN SOIL WATER STORAGE (INCH)
     13    CHANGE IN INTERCEPTION WATER STORAGE (INCH)
     14    CHANGE IN SNOW WATER STORAGE (INCH)
     15    ANNUAL WATER BUDGET BALANCE (INCH)

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      TITLE:  Scenario 1: Proposed Landfill Initial Lift

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WEATHER DATA SOURCES

NOTE: PRECIPITATION DATA WAS SYNTHETICALLY GENERATED USING
COEFFICIENTS FOR TALLAHASSEE FLORIDA

NORMAL MEAN MONTHLY PRECIPITATION (INCHES)

JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
4.66	5.00	5.60	4.13	5.16	6.55
8.75	7.30	6.45	3.10	3.31	4.58

NOTE: TEMPERATURE DATA WAS SYNTHETICALLY GENERATED USING
COEFFICIENTS FOR TALLAHASSEE FLORIDA

NORMAL MEAN MONTHLY TEMPERATURE (DEGREES FAHRENHEIT)

JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
51.60	53.60	60.20	67.10	74.00	79.50
81.20	81.10	78.30	68.40	58.80	53.00

NOTE: SOLAR RADIATION DATA WAS SYNTHETICALLY GENERATED USING
COEFFICIENTS FOR TALLAHASSEE FLORIDA
AND STATION LATITUDE = 30.23 DEGREES

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LAYER DATA 1

VALID FOR 5 YEARS

NOTE: INITIAL MOISTURE CONTENT OF THE LAYERS AND SNOW WATER WERE
COMPUTED AS NEARLY STEADY-STATE VALUES BY THE PROGRAM.

LAYER 1

TYPE 1 - VERTICAL PERCOLATION LAYER
MATERIAL TEXTURE NUMBER 18

THICKNESS	=	120.00	INCHES
POROSITY	=	0.6710	VOL/VOL
FIELD CAPACITY	=	0.2920	VOL/VOL
WILTING POINT	=	0.0770	VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.3205	VOL/VOL
EFFECTIVE SAT. HYD. CONDUCT.	=	0.1000E-02	CM/SEC

LAYER 2

TYPE 1 - VERTICAL PERCOLATION LAYER
MATERIAL TEXTURE NUMBER 5

THICKNESS	=	24.00	INCHES
POROSITY	=	0.4570	VOL/VOL
FIELD CAPACITY	=	0.1310	VOL/VOL
WILTING POINT	=	0.0580	VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.2759	VOL/VOL
EFFECTIVE SAT. HYD. CONDUCT.	=	0.1000E-02	CM/SEC

LAYER 3

TYPE 2 - LATERAL DRAINAGE LAYER
MATERIAL TEXTURE NUMBER 0

THICKNESS	=	0.27	INCHES
POROSITY	=	0.8500	VOL/VOL
FIELD CAPACITY	=	0.0100	VOL/VOL
WILTING POINT	=	0.0050	VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.1596	VOL/VOL
EFFECTIVE SAT. HYD. CONDUCT.	=	5.700	CM/SEC
SLOPE	=	2.00	PERCENT
DRAINAGE LENGTH	=	171.0	FEET

LAYER 4

TYPE 4 - FLEXIBLE MEMBRANE LINER
MATERIAL TEXTURE NUMBER 35

THICKNESS	=	0.06	INCHES
EFFECTIVE SAT. HYD. CONDUCT.	=	0.2000E-12	CM/SEC
FML PINHOLE DENSITY	=	2.00	HOLES/ACRE
FML INSTALLATION DEFECTS	=	2.00	HOLES/ACRE
FML PLACEMENT QUALITY	=	3 - GOOD	

LAYER 5

TYPE 2 - LATERAL DRAINAGE LAYER
MATERIAL TEXTURE NUMBER 0

THICKNESS	=	0.27	INCHES
POROSITY	=	0.8500	VOL/VOL
FIELD CAPACITY	=	0.0100	VOL/VOL
WILTING POINT	=	0.0050	VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.0214	VOL/VOL
EFFECTIVE SAT. HYD. CONDUCT.	=	5.700	CM/SEC
SLOPE	=	2.00	PERCENT
DRAINAGE LENGTH	=	171.0	FEET

LAYER 6

Initial Lift-10ft
TYPE 4 - FLEXIBLE MEMBRANE LINER
MATERIAL TEXTURE NUMBER 35

THICKNESS	=	0.06	INCHES
EFFECTIVE SAT. HYD. CONDUCT.	=	0.2000E-12	CM/SEC
FML PINHOLE DENSITY	=	2.00	HOLES/ACRE
FML INSTALLATION DEFECTS	=	2.00	HOLES/ACRE
FML PLACEMENT QUALITY	=	3 - GOOD	

LAYER 7

TYPE 1 - VERTICAL PERCOLATION LAYER
MATERIAL TEXTURE NUMBER 17

THICKNESS	=	0.24	INCHES
POROSITY	=	0.7500	VOL/VOL
FIELD CAPACITY	=	0.7470	VOL/VOL
WILTING POINT	=	0.4000	VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.7470	VOL/VOL
EFFECTIVE SAT. HYD. CONDUCT.	=	0.3000E-08	CM/SEC

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GENERAL DESIGN AND EVAPORATIVE ZONE DATA 1

VALID FOR 5 YEARS

NOTE: SCS RUNOFF CURVE NUMBER WAS COMPUTED FROM DEFAULT SOIL DATA BASE USING SOIL TEXTURE # 3 WITH BARE GROUND CONDITIONS, A SURFACE SLOPE OF 2.% AND A SLOPE LENGTH OF 400. FEET.

SCS RUNOFF CURVE NUMBER	=	80.17	
FRACTION OF AREA ALLOWING RUNOFF	=	0.0	PERCENT
AREA PROJECTED ON HORIZONTAL PLANE	=	1.000	ACRES
EVAPORATIVE ZONE DEPTH	=	6.0	INCHES
INITIAL WATER IN EVAPORATIVE ZONE	=	2.118	INCHES
UPPER LIMIT OF EVAPORATIVE STORAGE	=	4.026	INCHES
FIELD CAPACITY OF EVAPORATIVE ZONE	=	1.752	INCHES
LOWER LIMIT OF EVAPORATIVE STORAGE	=	0.462	INCHES
SOIL EVAPORATION ZONE DEPTH	=	6.000	INCHES
INITIAL SNOW WATER	=	0.000	INCHES
INITIAL INTERCEPTION WATER	=	0.000	INCHES
INITIAL WATER IN LAYER MATERIALS	=	45.308	INCHES
TOTAL INITIAL WATER	=	45.308	INCHES
TOTAL SUBSURFACE INFLOW	=	0.00	INCHES/YEAR

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EVAPOTRANSPIRATION DATA 1

VALID FOR 5 YEARS

NOTE: EVAPOTRANSPIRATION DATA WAS OBTAINED FROM

Initial Lift-10ft

TALLAHASSEE FLORIDA

STATION LATITUDE	= 30.23	DEGREES
MAXIMUM LEAF AREA INDEX	= 0.00	
START OF GROWING SEASON (JULIAN DATE)	= 36	
END OF GROWING SEASON (JULIAN DATE)	= 350	
EVAPORATIVE ZONE DEPTH	= 6.0	INCHES
AVERAGE ANNUAL WIND SPEED	= 6.60	MPH
AVERAGE 1ST QUARTER RELATIVE HUMIDITY	= 73.0	%
AVERAGE 2ND QUARTER RELATIVE HUMIDITY	= 73.0	%
AVERAGE 3RD QUARTER RELATIVE HUMIDITY	= 80.0	%
AVERAGE 4TH QUARTER RELATIVE HUMIDITY	= 77.0	%

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ANNUAL TOTALS FOR YEAR 1

	INCHES	CU. FEET	PERCENT
PRECIPITATION	48.59	176381.703	100.00
RUNOFF	0.000	0.000	0.00
POTENTIAL EVAPOTRANSPIRATION	54.031	196131.859	
ACTUAL EVAPOTRANSPIRATION	28.638	103957.523	58.94
DRAINAGE COLLECTED FROM LAYER 3	17.6004	63889.531	36.22
PERC./LEAKAGE THROUGH LAYER 4	2.351157	8534.701	4.84
AVG. HEAD ON TOP OF LAYER 4	0.0127		
DRAINAGE COLLECTED FROM LAYER 5	2.3512	8534.690	4.84
PERC./LEAKAGE THROUGH LAYER 6	0.000002	0.009	0.00
AVG. HEAD ON TOP OF LAYER 6	0.0017		
PERC./LEAKAGE THROUGH LAYER 7	0.000002	0.009	0.00
CHANGE IN WATER STORAGE	0.000	-0.014	0.00
SOIL WATER AT START OF YEAR	45.490	165128.500	
SOIL WATER AT END OF YEAR	45.490	165128.484	
INTERCEPTION WATER AT START OF YEAR	0.000	0.000	
INTERCEPTION WATER AT END OF YEAR	0.000	0.000	
SNOW WATER AT START OF YEAR	0.000	0.000	0.00
SNOW WATER AT END OF YEAR	0.000	0.000	0.00
ANNUAL WATER BUDGET BALANCE	0.0000	-0.035	0.00

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ANNUAL TOTALS FOR YEAR 2

	INCHES	CU. FEET	PERCENT
PRECIPITATION	80.32	291561.531	100.00
RUNOFF	0.000	0.000	0.00
POTENTIAL EVAPOTRANSPIRATION	53.954	195854.750	
ACTUAL EVAPOTRANSPIRATION	37.847	137385.547	47.12
DRAINAGE COLLECTED FROM LAYER 3	37.3968	135750.203	46.56
PERC./LEAKAGE THROUGH LAYER 4	3.803491	13806.672	4.74
AVG. HEAD ON TOP OF LAYER 4	0.0271		
DRAINAGE COLLECTED FROM LAYER 5	3.8062	13816.551	4.74
PERC./LEAKAGE THROUGH LAYER 6	0.000003	0.010	0.00
AVG. HEAD ON TOP OF LAYER 6	0.0028		
PERC./LEAKAGE THROUGH LAYER 7	0.000003	0.010	0.00
CHANGE IN WATER STORAGE	1.270	4609.257	1.58
SOIL WATER AT START OF YEAR	45.490	165128.484	
SOIL WATER AT END OF YEAR	46.760	169737.734	
INTERCEPTION WATER AT START OF YEAR	0.000	0.000	
INTERCEPTION WATER AT END OF YEAR	0.000	0.000	
SNOW WATER AT START OF YEAR	0.000	0.000	0.00
SNOW WATER AT END OF YEAR	0.000	0.000	0.00
ANNUAL WATER BUDGET BALANCE	0.0000	-0.066	0.00

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ANNUAL TOTALS FOR YEAR 3

	INCHES	CU. FEET	PERCENT
PRECIPITATION	69.92	253809.578	100.00

RUNOFF	0.000	0.000	0.00
POTENTIAL EVAPOTRANSPIRATION	54.644	198359.312	
ACTUAL EVAPOTRANSPIRATION	34.734	126083.719	49.68
DRAINAGE COLLECTED FROM LAYER 3	34.8709	126581.445	49.87
PERC./LEAKAGE THROUGH LAYER 4	3.480930	12635.776	4.98
AVG. HEAD ON TOP OF LAYER 4	0.0253		
DRAINAGE COLLECTED FROM LAYER 5	3.4801	12632.610	4.98
PERC./LEAKAGE THROUGH LAYER 6	0.000003	0.009	0.00
AVG. HEAD ON TOP OF LAYER 6	0.0025		
PERC./LEAKAGE THROUGH LAYER 7	0.000003	0.009	0.00
CHANGE IN WATER STORAGE	-3.165	-11488.137	-4.53
SOIL WATER AT START OF YEAR	46.760	169737.734	
SOIL WATER AT END OF YEAR	43.595	158249.609	
INTERCEPTION WATER AT START OF YEAR	0.000	0.000	
INTERCEPTION WATER AT END OF YEAR	0.000	0.000	
SNOW WATER AT START OF YEAR	0.000	0.000	0.00
SNOW WATER AT END OF YEAR	0.000	0.000	0.00
ANNUAL WATER BUDGET BALANCE	0.0000	-0.061	0.00

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ANNUAL TOTALS FOR YEAR 4

	INCHES	CU. FEET	PERCENT
PRECIPITATION	58.15	211084.484	100.00
RUNOFF	0.000	0.000	0.00
POTENTIAL EVAPOTRANSPIRATION	54.811	198963.734	
ACTUAL EVAPOTRANSPIRATION	34.303	124521.062	58.99
DRAINAGE COLLECTED FROM LAYER 3	24.5749	89206.852	42.26
PERC./LEAKAGE THROUGH LAYER 4	3.009459	10924.337	5.18
AVG. HEAD ON TOP OF LAYER 4	0.0178		
DRAINAGE COLLECTED FROM LAYER 5	3.0100	10926.140	5.18

PERC./LEAKAGE THROUGH LAYER 6	0.000003	0.009	0.00
AVG. HEAD ON TOP OF LAYER 6	0.0022		
PERC./LEAKAGE THROUGH LAYER 7	0.000003	0.009	0.00
CHANGE IN WATER STORAGE	-3.738	-13569.519	-6.43
SOIL WATER AT START OF YEAR	43.595	158249.609	
SOIL WATER AT END OF YEAR	39.857	144680.078	
INTERCEPTION WATER AT START OF YEAR	0.000	0.000	
INTERCEPTION WATER AT END OF YEAR	0.000	0.000	
SNOW WATER AT START OF YEAR	0.000	0.000	0.00
SNOW WATER AT END OF YEAR	0.000	0.000	0.00
ANNUAL WATER BUDGET BALANCE	0.0000	-0.068	0.00

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ANNUAL TOTALS FOR YEAR 5

	INCHES	CU. FEET	PERCENT
PRECIPITATION	70.21	254862.266	100.00
RUNOFF	0.000	0.000	0.00
POTENTIAL EVAPOTRANSPIRATION	53.436	193973.516	
ACTUAL EVAPOTRANSPIRATION	35.798	129945.188	50.99
DRAINAGE COLLECTED FROM LAYER 3	31.1079	112921.617	44.31
PERC./LEAKAGE THROUGH LAYER 4	3.344393	12140.147	4.76
AVG. HEAD ON TOP OF LAYER 4	0.0224		
DRAINAGE COLLECTED FROM LAYER 5	3.3443	12139.715	4.76
PERC./LEAKAGE THROUGH LAYER 6	0.000003	0.009	0.00
AVG. HEAD ON TOP OF LAYER 6	0.0024		
PERC./LEAKAGE THROUGH LAYER 7	0.000003	0.009	0.00
CHANGE IN WATER STORAGE	-0.040	-144.109	-0.06
SOIL WATER AT START OF YEAR	39.857	144680.078	
SOIL WATER AT END OF YEAR	39.817	144535.984	

INTERCEPTION WATER AT START OF YEAR	0.000	0.000	
INTERCEPTION WATER AT END OF YEAR	0.000	0.000	
SNOW WATER AT START OF YEAR	0.000	0.000	0.00
SNOW WATER AT END OF YEAR	0.000	0.000	0.00
ANNUAL WATER BUDGET BALANCE	0.0000	-0.158	0.00

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FINAL WATER STORAGE AT END OF YEAR 5

LAYER	(INCHES)	(VOL/VOL)
1	34.6784	0.2890
2	4.7688	0.1987
3	0.0050	0.0187
4	0.0000	0.0000
5	0.0035	0.0131
6	0.0000	0.0000
7	0.1793	0.7470
TOTAL WATER IN LAYERS	39.635	
SNOW WATER	0.000	
INTERCEPTION WATER	0.000	
TOTAL FINAL WATER	39.635	

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PEAK DAILY VALUES FOR YEARS 1 THROUGH 5

	(INCHES)	(CU. FT.)
PRECIPITATION	4.34	15754.200
RUNOFF	0.000	0.0000
DRAINAGE COLLECTED FROM LAYER 3	0.48881	1774.38379
PERCOLATION/LEAKAGE THROUGH LAYER 4	0.026356	95.67166

AVERAGE HEAD ON TOP OF LAYER 4		0.129	
MAXIMUM HEAD ON TOP OF LAYER 4		0.252	
LOCATION OF MAXIMUM HEAD IN LAYER 3 (DISTANCE FROM DRAIN)		4.4 FEET	
DRAINAGE COLLECTED FROM LAYER 5		0.02597	94.27908
PERCOLATION/LEAKAGE THROUGH LAYER 6		0.000000	0.00003
AVERAGE HEAD ON TOP OF LAYER 6		0.007	
MAXIMUM HEAD ON TOP OF LAYER 6		0.014	
LOCATION OF MAXIMUM HEAD IN LAYER 5 (DISTANCE FROM DRAIN)		0.0 FEET	
PERCOLATION/LEAKAGE THROUGH LAYER 7		0.000000	0.00003
SNOW WATER		0.31	1126.4432
MAXIMUM VEG. SOIL WATER (VOL/VOL)		0.5084	
MINIMUM VEG. SOIL WATER (VOL/VOL)		0.0770	

*** Maximum heads are computed using McEnroe's equations. ***

Reference: Maximum Saturated Depth over Landfill Liner
by Bruce M. McEnroe, University of Kansas
ASCE Journal of Environmental Engineering
Vol. 119, No. 2, March 1993, pp. 262-270.

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AVERAGE MONTHLY VALUES IN INCHES FOR YEARS 1 THROUGH 5

	JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
PRECIPITATION						
TOTALS	4.32 9.44	5.01 8.08	4.43 7.36	3.72 2.45	5.07 1.84	8.32 5.40
STD. DEVIATIONS	2.35 4.08	2.10 2.93	2.66 2.62	4.55 1.77	4.30 0.49	3.23 3.13
RUNOFF						
TOTALS	0.000 0.000	0.000 0.000	0.000 0.000	0.000 0.000	0.000 0.000	0.000 0.000
STD. DEVIATIONS	0.000 0.000	0.000 0.000	0.000 0.000	0.000 0.000	0.000 0.000	0.000 0.000

POTENTIAL EVAPOTRANSPIRATION

TOTALS	2.376	2.703	3.829	5.296	6.636	6.855
	6.955	6.056	4.828	3.839	2.805	1.997
STD. DEVIATIONS	0.133	0.137	0.201	0.415	0.431	0.212
	0.293	0.381	0.266	0.225	0.149	0.159

ACTUAL EVAPOTRANSPIRATION

TOTALS	2.017	2.266	2.867	2.106	2.864	4.100
	5.421	3.990	3.673	1.981	1.333	1.647
STD. DEVIATIONS	0.251	0.300	0.675	1.375	1.270	0.898
	0.560	0.616	0.496	0.506	0.814	0.375

LATERAL DRAINAGE COLLECTED FROM LAYER 3

TOTALS	3.6382	2.2312	2.3715	1.7185	1.6270	1.8915
	3.8089	3.7978	3.7800	2.8829	0.9888	0.3736
STD. DEVIATIONS	2.4667	1.6352	1.2799	1.1367	2.0520	2.1466
	2.2114	1.9592	1.6665	1.5323	0.6010	0.1769

PERCOLATION/LEAKAGE THROUGH LAYER 4

TOTALS	0.3251	0.2530	0.2844	0.2354	0.2032	0.2078
	0.3483	0.3692	0.3624	0.3216	0.1785	0.1089
STD. DEVIATIONS	0.1621	0.1325	0.0953	0.0990	0.1467	0.1551
	0.1555	0.1129	0.0933	0.1150	0.0599	0.0182

LATERAL DRAINAGE COLLECTED FROM LAYER 5

TOTALS	0.3241	0.2539	0.2844	0.2361	0.2024	0.2073
	0.3477	0.3695	0.3620	0.3232	0.1790	0.1087
STD. DEVIATIONS	0.1615	0.1329	0.0943	0.0993	0.1459	0.1555
	0.1566	0.1135	0.0932	0.1149	0.0602	0.0179

PERCOLATION/LEAKAGE THROUGH LAYER 6

TOTALS	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
STD. DEVIATIONS	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000

PERCOLATION/LEAKAGE THROUGH LAYER 7

TOTALS	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
STD. DEVIATIONS	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000

AVERAGES OF MONTHLY AVERAGED DAILY HEADS (INCHES)

DAILY AVERAGE HEAD ON TOP OF LAYER 4

Initial Lift-10ft

DAILY AVERAGE HEAD ON TOP OF LAYER 6

AVERAGES	0.0311	0.0209	0.0203	0.0152	0.0139	0.0167
	0.0325	0.0324	0.0334	0.0246	0.0087	0.0032
STD. DEVIATIONS	0.0211	0.0154	0.0109	0.0100	0.0175	0.0189
	0.0189	0.0167	0.0147	0.0131	0.0053	0.0015

DAILY AVERAGE HEAD ON TOP OF LAYER 6

AVERAGES	0.0028	0.0024	0.0024	0.0021	0.0017	0.0018
	0.0030	0.0032	0.0032	0.0028	0.0016	0.0009
STD. DEVIATIONS	0.0014	0.0012	0.0008	0.0009	0.0012	0.0014
	0.0013	0.0010	0.0008	0.0010	0.0005	0.0002

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AVERAGE ANNUAL TOTALS & (STD. DEVIATIONS) FOR YEARS 1 THROUGH 5

	INCHES		CU. FEET	PERCENT
PRECIPITATION	65.44	(12.261)	237539.9	100.00
RUNOFF	0.000	(0.0000)	0.00	0.000
POTENTIAL EVAPOTRANSPIRATION	54.175	(0.5573)	196656.62	
ACTUAL EVAPOTRANSPIRATION	34.264	(3.4300)	124378.62	52.361
LATERAL DRAINAGE COLLECTED FROM LAYER 3	29.11017	(8.04499)	105669.930	44.48512
PERCOLATION/LEAKAGE THROUGH LAYER 4	3.19789	(0.55245)	11608.325	4.88689
AVERAGE HEAD ON TOP OF LAYER 4	0.021	(0.006)		
LATERAL DRAINAGE COLLECTED FROM LAYER 5	3.19833	(0.55303)	11609.941	4.88757
PERCOLATION/LEAKAGE THROUGH LAYER 6	0.00000	(0.00000)	0.009	0.00000
AVERAGE HEAD ON TOP OF LAYER 6	0.002	(0.000)		
PERCOLATION/LEAKAGE THROUGH LAYER 7	0.00000	(0.00000)	0.009	0.00000
CHANGE IN WATER STORAGE	-1.135	(2.1890)	-4118.50	-1.734

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**               HYDROLOGIC EVALUATION OF LANDFILL PERFORMANCE                      **
**                                                                                  **
**               HELP Version 3.95 D          (10 August 2012)                      **
**                                developed at                                      **
**         Institute of Soil Science, University of Hamburg, Germany                **
**                                  based on                                        **
**               US HELP MODEL VERSION 3.07  (1 NOVEMBER 1997)                      **
**                   DEVELOPED BY ENVIRONMENTAL LABORATORY                          **
**                     USAE WATERWAYS EXPERIMENT STATION                            **
**                FOR USEPA RISK REDUCTION ENGINEERING LABORATORY                   **
**                                                                                  **
**                                                                                  **
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 TIME:  15.42     DATE:   6.02.2019


 PRECIPITATION DATA FILE:      C:\HDR_Projects\steelfield\Engineering Design\HELP
Model\Intermediate-expansion Area\precipitation.d4
 TEMPERATURE DATA FILE:        C:\HDR_Projects\steelfield\Engineering Design\HELP
Model\Intermediate-expansion Area\temperature.d7
 SOLAR RADIATION DATA FILE:    C:\HDR_Projects\steelfield\Engineering Design\HELP
Model\Intermediate-expansion Area\solar radiation.d13
 EVAPOTRANSPIRATION DATA F. 1: C:\HDR_Projects\steelfield\Engineering Design\HELP
Model\Intermediate-expansion Area\evapotranspiration.d11
 SOIL AND DESIGN DATA FILE  1: C:\HDR_Projects\steelfield\Engineering Design\HELP
Model\Intermediate-expansion Area\Soil and Design-Proposed Landfill.d10
 OUTPUT DATA FILE:             C:\HDR_Projects\steelfield\Engineering Design\HELP
Model\Intermediate-expansion Area\Proposed LF-Intermediate Cover.out
 YEARLY OUTPUT DATA FILE:      C:\HDR_Projects\steelfield\Engineering Design\HELP
Model\Intermediate-expansion Area\Proposed LF-Intermediate Cover.YR

 COLUMNS OF YEARLY OUTPUT DATA FILE:

      1    DATE OF ULTIMO (yyyy1231, years 2101 to 2200 from weather generator)
      2    PRECIPITATION (INCH)
      3    RUNOFF (INCH)
      4    POTENTIAL EVAPOTRANSPIRATION (INCH)
      5    ACTUAL EVAPOTRANSPIRATION (INCH)
      6    DRAIN #1: LATERAL DRAINAGE FROM LAYER  4 (INCH)
      7    LEAK  #1: PERCOLATION/LEAKAGE THROUGH LAYER  5 (INCH)
      8    DRAIN #2: LATERAL DRAINAGE FROM LAYER  6 (INCH)
      9    LEAK  #2: PERCOLATION/LEAKAGE THROUGH LAYER  7 (INCH)
     10    LEAK  #3: PERCOLATION/LEAKAGE THROUGH LAYER  8 (INCH)
     11    CHANGE IN TOTAL WATER STORAGE (INCH)
     12    CHANGE IN SOIL WATER STORAGE (INCH)
     13    CHANGE IN INTERCEPTION WATER STORAGE (INCH)
     14    CHANGE IN SNOW WATER STORAGE (INCH)
     15    ANNUAL WATER BUDGET BALANCE (INCH)


 **************************************************************************************

      TITLE:  Scenario 3: Proposed Landfill-Intermediate Cover

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Proposed LF-Intermediate Cover

WEATHER DATA SOURCES

NOTE: PRECIPITATION DATA WAS SYNTHETICALLY GENERATED USING
COEFFICIENTS FOR TALLAHASSEE FLORIDA

NORMAL MEAN MONTHLY PRECIPITATION (INCHES)

JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
4.66	5.00	5.60	4.13	5.16	6.55
8.75	7.30	6.45	3.10	3.31	4.58

NOTE: TEMPERATURE DATA WAS SYNTHETICALLY GENERATED USING
COEFFICIENTS FOR TALLAHASSEE FLORIDA

NORMAL MEAN MONTHLY TEMPERATURE (DEGREES FAHRENHEIT)

JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
51.60	53.60	60.20	67.10	74.00	79.50
81.20	81.10	78.30	68.40	58.80	53.00

NOTE: SOLAR RADIATION DATA WAS SYNTHETICALLY GENERATED USING
COEFFICIENTS FOR TALLAHASSEE FLORIDA
AND STATION LATITUDE = 30.23 DEGREES

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LAYER DATA 1

VALID FOR 30 YEARS

NOTE: INITIAL MOISTURE CONTENT OF THE LAYERS AND SNOW WATER WERE
COMPUTED AS NEARLY STEADY-STATE VALUES BY THE PROGRAM.

LAYER 1

TYPE 1 - VERTICAL PERCOLATION LAYER
MATERIAL TEXTURE NUMBER 7

THICKNESS	=	12.00	INCHES
POROSITY	=	0.4730	VOL/VOL
FIELD CAPACITY	=	0.2220	VOL/VOL
WILTING POINT	=	0.1040	VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.3020	VOL/VOL
EFFECTIVE SAT. HYD. CONDUCT.	=	0.5200E-03	CM/SEC

NOTE: SATURATED HYDRAULIC CONDUCTIVITY IS MULTIPLIED BY 3.00
FOR ROOT CHANNELS IN TOP HALF OF EVAPORATIVE ZONE.

LAYER 2

TYPE 1 - VERTICAL PERCOLATION LAYER
MATERIAL TEXTURE NUMBER 18

THICKNESS	=	720.00	INCHES
POROSITY	=	0.6710	VOL/VOL
FIELD CAPACITY	=	0.2920	VOL/VOL
WILTING POINT	=	0.0770	VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.3035	VOL/VOL
EFFECTIVE SAT. HYD. CONDUCT.	=	0.1000E-02	CM/SEC

LAYER 3

TYPE 1 - VERTICAL PERCOLATION LAYER
MATERIAL TEXTURE NUMBER 5

THICKNESS	=	24.00	INCHES
POROSITY	=	0.4570	VOL/VOL
FIELD CAPACITY	=	0.1310	VOL/VOL
WILTING POINT	=	0.0580	VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.2201	VOL/VOL
EFFECTIVE SAT. HYD. CONDUCT.	=	0.1000E-02	CM/SEC

LAYER 4

TYPE 2 - LATERAL DRAINAGE LAYER
MATERIAL TEXTURE NUMBER 0

THICKNESS	=	0.27	INCHES
POROSITY	=	0.8500	VOL/VOL
FIELD CAPACITY	=	0.0100	VOL/VOL
WILTING POINT	=	0.0050	VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.0296	VOL/VOL
EFFECTIVE SAT. HYD. CONDUCT.	=	5.700	CM/SEC
SLOPE	=	2.00	PERCENT
DRAINAGE LENGTH	=	171.0	FEET

LAYER 5

TYPE 4 - FLEXIBLE MEMBRANE LINER
MATERIAL TEXTURE NUMBER 35

THICKNESS	=	0.06	INCHES
EFFECTIVE SAT. HYD. CONDUCT.	=	0.2000E-12	CM/SEC
FML PINHOLE DENSITY	=	2.00	HOLES/ACRE
FML INSTALLATION DEFECTS	=	2.00	HOLES/ACRE
FML PLACEMENT QUALITY	=	3 - GOOD	

LAYER 6

Proposed LF-Intermediate Cover
TYPE 2 - LATERAL DRAINAGE LAYER
MATERIAL TEXTURE NUMBER 0

THICKNESS	=	0.27	INCHES
POROSITY	=	0.8500	VOL/VOL
FIELD CAPACITY	=	0.0100	VOL/VOL
WILTING POINT	=	0.0050	VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.0149	VOL/VOL
EFFECTIVE SAT. HYD. CONDUCT.	=	5.700	CM/SEC
SLOPE	=	2.00	PERCENT
DRAINAGE LENGTH	=	171.0	FEET

LAYER 7

TYPE 4 - FLEXIBLE MEMBRANE LINER
MATERIAL TEXTURE NUMBER 35

THICKNESS	=	0.06	INCHES
EFFECTIVE SAT. HYD. CONDUCT.	=	0.2000E-12	CM/SEC
FML PINHOLE DENSITY	=	2.00	HOLES/ACRE
FML INSTALLATION DEFECTS	=	2.00	HOLES/ACRE
FML PLACEMENT QUALITY	=	3 - GOOD	

LAYER 8

TYPE 1 - VERTICAL PERCOLATION LAYER
MATERIAL TEXTURE NUMBER 17

THICKNESS	=	0.25	INCHES
POROSITY	=	0.7500	VOL/VOL
FIELD CAPACITY	=	0.7470	VOL/VOL
WILTING POINT	=	0.4000	VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.7470	VOL/VOL
EFFECTIVE SAT. HYD. CONDUCT.	=	0.3000E-08	CM/SEC

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GENERAL DESIGN AND EVAPORATIVE ZONE DATA 1

VALID FOR 30 YEARS

NOTE: SCS RUNOFF CURVE NUMBER WAS COMPUTED FROM DEFAULT SOIL DATA BASE USING SOIL TEXTURE # 4 WITH A FAIR STAND OF GRASS, A SURFACE SLOPE OF 5.% AND A SLOPE LENGTH OF 100. FEET.

SCS RUNOFF CURVE NUMBER	=	63.03	
FRACTION OF AREA ALLOWING RUNOFF	=	100.0	PERCENT
AREA PROJECTED ON HORIZONTAL PLANE	=	1.000	ACRES
EVAPORATIVE ZONE DEPTH	=	6.0	INCHES
INITIAL WATER IN EVAPORATIVE ZONE	=	1.861	INCHES
UPPER LIMIT OF EVAPORATIVE STORAGE	=	2.838	INCHES
FIELD CAPACITY OF EVAPORATIVE ZONE	=	1.332	INCHES
LOWER LIMIT OF EVAPORATIVE STORAGE	=	0.624	INCHES
SOIL EVAPORATION ZONE DEPTH	=	6.000	INCHES
INITIAL SNOW WATER	=	0.000	INCHES

Proposed LF-Intermediate Cover

INITIAL INTERCEPTION WATER	=	0.002	INCHES
INITIAL WATER IN LAYER MATERIALS	=	227.641	INCHES
TOTAL INITIAL WATER	=	227.643	INCHES
TOTAL SUBSURFACE INFLOW	=	0.00	INCHES/YEAR

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EVAPOTRANSPIRATION DATA 1

VALID FOR 30 YEARS

NOTE: EVAPOTRANSPIRATION DATA WAS OBTAINED FROM
TALLAHASSEE FLORIDA

STATION LATITUDE	=	30.23	DEGREES
MAXIMUM LEAF AREA INDEX	=	2.00	
START OF GROWING SEASON (JULIAN DATE)	=	36	
END OF GROWING SEASON (JULIAN DATE)	=	350	
EVAPORATIVE ZONE DEPTH	=	6.0	INCHES
AVERAGE ANNUAL WIND SPEED	=	6.60	MPH
AVERAGE 1ST QUARTER RELATIVE HUMIDITY	=	73.0	%
AVERAGE 2ND QUARTER RELATIVE HUMIDITY	=	73.0	%
AVERAGE 3RD QUARTER RELATIVE HUMIDITY	=	80.0	%
AVERAGE 4TH QUARTER RELATIVE HUMIDITY	=	77.0	%

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ANNUAL TOTALS FOR YEAR 1

	INCHES	CU. FEET	PERCENT
PRECIPITATION	48.59	176381.703	100.00
RUNOFF	0.037	136.076	0.08
POTENTIAL EVAPOTRANSPIRATION	54.031	196131.859	
ACTUAL EVAPOTRANSPIRATION	26.949	97825.938	55.46
DRAINAGE COLLECTED FROM LAYER 4	18.8882	68564.219	38.87
PERC./LEAKAGE THROUGH LAYER 5	2.704132	9815.999	5.57
AVG. HEAD ON TOP OF LAYER 5	0.0138		
DRAINAGE COLLECTED FROM LAYER 6	2.7041	9816.021	5.57
PERC./LEAKAGE THROUGH LAYER 7	0.000003	0.009	0.00
AVG. HEAD ON TOP OF LAYER 7	0.0020		
PERC./LEAKAGE THROUGH LAYER 8	0.000003	0.009	0.00
CHANGE IN WATER STORAGE	0.011	39.394	0.02

SOIL WATER AT START OF YEAR	227.831	827026.062	
SOIL WATER AT END OF YEAR	227.841	827063.250	
INTERCEPTION WATER AT START OF YEAR	0.002	6.527	
INTERCEPTION WATER AT END OF YEAR	0.002	8.763	
SNOW WATER AT START OF YEAR	0.000	0.000	0.00
SNOW WATER AT END OF YEAR	0.000	0.000	0.00
ANNUAL WATER BUDGET BALANCE	0.0000	0.041	0.00

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ANNUAL TOTALS FOR YEAR 2

	INCHES	CU. FEET	PERCENT
PRECIPITATION	80.32	291561.531	100.00
RUNOFF	0.151	548.701	0.19
POTENTIAL EVAPOTRANSPIRATION	53.954	195854.750	
ACTUAL EVAPOTRANSPIRATION	33.181	120448.805	41.31
DRAINAGE COLLECTED FROM LAYER 4	35.5901	129192.070	44.31
PERC./LEAKAGE THROUGH LAYER 5	3.913497	14205.996	4.87
AVG. HEAD ON TOP OF LAYER 5	0.0258		
DRAINAGE COLLECTED FROM LAYER 6	3.9131	14204.493	4.87
PERC./LEAKAGE THROUGH LAYER 7	0.000003	0.010	0.00
AVG. HEAD ON TOP OF LAYER 7	0.0028		
PERC./LEAKAGE THROUGH LAYER 8	0.000003	0.010	0.00
CHANGE IN WATER STORAGE	7.484	27167.506	9.32
SOIL WATER AT START OF YEAR	227.841	827063.250	
SOIL WATER AT END OF YEAR	235.328	854239.500	
INTERCEPTION WATER AT START OF YEAR	0.002	8.763	
INTERCEPTION WATER AT END OF YEAR	0.000	0.000	
SNOW WATER AT START OF YEAR	0.000	0.000	0.00
SNOW WATER AT END OF YEAR	0.000	0.000	0.00

ANNUAL WATER BUDGET BALANCE	0.0000	-0.061	0.00

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ANNUAL TOTALS FOR YEAR 3

	INCHES	CU. FEET	PERCENT
PRECIPITATION	69.92	253809.578	100.00
RUNOFF	0.237	859.039	0.34
POTENTIAL EVAPOTRANSPIRATION	54.644	198359.312	
ACTUAL EVAPOTRANSPIRATION	30.335	110116.250	43.39
DRAINAGE COLLECTED FROM LAYER 4	36.5904	132822.984	52.33
PERC./LEAKAGE THROUGH LAYER 5	3.978231	14440.979	5.69
AVG. HEAD ON TOP OF LAYER 5	0.0266		
DRAINAGE COLLECTED FROM LAYER 6	3.9783	14441.114	5.69
PERC./LEAKAGE THROUGH LAYER 7	0.000003	0.010	0.00
AVG. HEAD ON TOP OF LAYER 7	0.0029		
PERC./LEAKAGE THROUGH LAYER 8	0.000003	0.010	0.00
CHANGE IN WATER STORAGE	-1.220	-4429.823	-1.75
SOIL WATER AT START OF YEAR	235.328	854239.500	
SOIL WATER AT END OF YEAR	234.107	849809.688	
INTERCEPTION WATER AT START OF YEAR	0.000	0.000	
INTERCEPTION WATER AT END OF YEAR	0.000	0.000	
SNOW WATER AT START OF YEAR	0.000	0.000	0.00
SNOW WATER AT END OF YEAR	0.000	0.000	0.00
ANNUAL WATER BUDGET BALANCE	0.0000	0.025	0.00

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ANNUAL TOTALS FOR YEAR 4

	INCHES	CU. FEET	PERCENT

Proposed LF-Intermediate Cover

PRECIPITATION	58.15	211084.484	100.00
RUNOFF	0.156	567.807	0.27
POTENTIAL EVAPOTRANSPIRATION	54.811	198963.734	
ACTUAL EVAPOTRANSPIRATION	31.950	115977.070	54.94
DRAINAGE COLLECTED FROM LAYER 4	33.3242	120966.914	57.31
PERC./LEAKAGE THROUGH LAYER 5	3.849976	13975.413	6.62
AVG. HEAD ON TOP OF LAYER 5	0.0241		
DRAINAGE COLLECTED FROM LAYER 6	3.8494	13973.415	6.62
PERC./LEAKAGE THROUGH LAYER 7	0.000003	0.010	0.00
AVG. HEAD ON TOP OF LAYER 7	0.0028		
PERC./LEAKAGE THROUGH LAYER 8	0.000003	0.010	0.00
CHANGE IN WATER STORAGE	-11.130	-40400.645	-19.14
SOIL WATER AT START OF YEAR	234.107	849809.688	
SOIL WATER AT END OF YEAR	222.978	809409.000	
INTERCEPTION WATER AT START OF YEAR	0.000	0.000	
INTERCEPTION WATER AT END OF YEAR	0.000	0.000	
SNOW WATER AT START OF YEAR	0.000	0.000	0.00
SNOW WATER AT END OF YEAR	0.000	0.000	0.00
ANNUAL WATER BUDGET BALANCE	0.0000	-0.095	0.00

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ANNUAL TOTALS FOR YEAR 5

	INCHES	CU. FEET	PERCENT
PRECIPITATION	70.21	254862.266	100.00
RUNOFF	0.124	451.807	0.18
POTENTIAL EVAPOTRANSPIRATION	53.436	193973.516	
ACTUAL EVAPOTRANSPIRATION	33.082	120089.180	47.12
DRAINAGE COLLECTED FROM LAYER 4	27.0631	98238.992	38.55
PERC./LEAKAGE THROUGH LAYER 5	3.372091	12240.691	4.80

AVG. HEAD ON TOP OF LAYER 5	0.0196		
DRAINAGE COLLECTED FROM LAYER 6	3.3715	12238.714	4.80
PERC./LEAKAGE THROUGH LAYER 7	0.000003	0.010	0.00
AVG. HEAD ON TOP OF LAYER 7	0.0024		
PERC./LEAKAGE THROUGH LAYER 8	0.000003	0.010	0.00
CHANGE IN WATER STORAGE	6.569	23843.697	9.36
SOIL WATER AT START OF YEAR	222.978	809409.000	
SOIL WATER AT END OF YEAR	229.546	833252.750	
INTERCEPTION WATER AT START OF YEAR	0.000	0.000	
INTERCEPTION WATER AT END OF YEAR	0.000	0.000	
SNOW WATER AT START OF YEAR	0.000	0.000	0.00
SNOW WATER AT END OF YEAR	0.000	0.000	0.00
ANNUAL WATER BUDGET BALANCE	0.0000	-0.133	0.00

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ANNUAL TOTALS FOR YEAR 6

	INCHES	CU. FEET	PERCENT
PRECIPITATION	73.90	268256.906	100.00
RUNOFF	0.048	173.497	0.06
POTENTIAL EVAPOTRANSPIRATION	54.288	197066.312	
ACTUAL EVAPOTRANSPIRATION	34.014	123469.812	46.03
DRAINAGE COLLECTED FROM LAYER 4	29.2508	106180.258	39.58
PERC./LEAKAGE THROUGH LAYER 5	3.551738	12892.810	4.81
AVG. HEAD ON TOP OF LAYER 5	0.0212		
DRAINAGE COLLECTED FROM LAYER 6	3.5528	12896.761	4.81
PERC./LEAKAGE THROUGH LAYER 7	0.000003	0.010	0.00
AVG. HEAD ON TOP OF LAYER 7	0.0026		
PERC./LEAKAGE THROUGH LAYER 8	0.000003	0.010	0.00
CHANGE IN WATER STORAGE	7.035	25536.619	9.52
SOIL WATER AT START OF YEAR	229.546	833252.750	

Proposed LF-Intermediate Cover

SOIL WATER AT END OF YEAR	236.581	858789.312	
INTERCEPTION WATER AT START OF YEAR	0.000	0.000	
INTERCEPTION WATER AT END OF YEAR	0.000	0.000	
SNOW WATER AT START OF YEAR	0.000	0.000	0.00
SNOW WATER AT END OF YEAR	0.000	0.000	0.00
ANNUAL WATER BUDGET BALANCE	0.0000	-0.043	0.00

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ANNUAL TOTALS FOR YEAR 7

	INCHES	CU. FEET	PERCENT
PRECIPITATION	68.37	248183.109	100.00
RUNOFF	0.591	2144.042	0.86
POTENTIAL EVAPOTRANSPIRATION	54.391	197439.922	
ACTUAL EVAPOTRANSPIRATION	29.658	107659.906	43.38
DRAINAGE COLLECTED FROM LAYER 4	35.3828	128439.633	51.75
PERC./LEAKAGE THROUGH LAYER 5	3.897793	14148.988	5.70
AVG. HEAD ON TOP OF LAYER 5	0.0257		
DRAINAGE COLLECTED FROM LAYER 6	3.8961	14142.886	5.70
PERC./LEAKAGE THROUGH LAYER 7	0.000003	0.010	0.00
AVG. HEAD ON TOP OF LAYER 7	0.0028		
PERC./LEAKAGE THROUGH LAYER 8	0.000003	0.010	0.00
CHANGE IN WATER STORAGE	-1.158	-4203.391	-1.69
SOIL WATER AT START OF YEAR	236.581	858789.312	
SOIL WATER AT END OF YEAR	235.423	854585.938	
INTERCEPTION WATER AT START OF YEAR	0.000	0.000	
INTERCEPTION WATER AT END OF YEAR	0.000	0.000	
SNOW WATER AT START OF YEAR	0.000	0.000	0.00
SNOW WATER AT END OF YEAR	0.000	0.000	0.00
ANNUAL WATER BUDGET BALANCE	0.0000	0.010	0.00

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ANNUAL TOTALS FOR YEAR 8

	INCHES	CU. FEET	PERCENT
PRECIPITATION	66.99	243173.688	100.00
RUNOFF	0.369	1339.651	0.55
POTENTIAL EVAPOTRANSPIRATION	54.866	199164.266	
ACTUAL EVAPOTRANSPIRATION	29.881	108466.375	44.60
DRAINAGE COLLECTED FROM LAYER 4	33.2825	120815.617	49.68
PERC./LEAKAGE THROUGH LAYER 5	3.851950	13982.579	5.75
AVG. HEAD ON TOP OF LAYER 5	0.0241		
DRAINAGE COLLECTED FROM LAYER 6	3.8537	13988.839	5.75
PERC./LEAKAGE THROUGH LAYER 7	0.000003	0.010	0.00
AVG. HEAD ON TOP OF LAYER 7	0.0028		
PERC./LEAKAGE THROUGH LAYER 8	0.000003	0.010	0.00
CHANGE IN WATER STORAGE	-0.396	-1436.746	-0.59
SOIL WATER AT START OF YEAR	235.423	854585.938	
SOIL WATER AT END OF YEAR	235.027	853149.188	
INTERCEPTION WATER AT START OF YEAR	0.000	0.000	
INTERCEPTION WATER AT END OF YEAR	0.000	0.000	
SNOW WATER AT START OF YEAR	0.000	0.000	0.00
SNOW WATER AT END OF YEAR	0.000	0.000	0.00
ANNUAL WATER BUDGET BALANCE	0.0000	-0.065	0.00

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ANNUAL TOTALS FOR YEAR 9

	INCHES	CU. FEET	PERCENT
PRECIPITATION	67.42	244734.578	100.00

RUNOFF	0.148	539.023	0.22
POTENTIAL EVAPOTRANSPIRATION	54.852	199112.859	
ACTUAL EVAPOTRANSPIRATION	32.482	117910.719	48.18
DRAINAGE COLLECTED FROM LAYER 4	34.2935	124485.508	50.87
PERC./LEAKAGE THROUGH LAYER 5	3.846149	13961.521	5.70
AVG. HEAD ON TOP OF LAYER 5	0.0248		
DRAINAGE COLLECTED FROM LAYER 6	3.8448	13956.708	5.70
PERC./LEAKAGE THROUGH LAYER 7	0.000003	0.010	0.00
AVG. HEAD ON TOP OF LAYER 7	0.0028		
PERC./LEAKAGE THROUGH LAYER 8	0.000003	0.010	0.00
CHANGE IN WATER STORAGE	-3.349	-12157.421	-4.97
SOIL WATER AT START OF YEAR	235.027	853149.188	
SOIL WATER AT END OF YEAR	231.678	840991.750	
INTERCEPTION WATER AT START OF YEAR	0.000	0.000	
INTERCEPTION WATER AT END OF YEAR	0.000	0.000	
SNOW WATER AT START OF YEAR	0.000	0.000	0.00
SNOW WATER AT END OF YEAR	0.000	0.000	0.00
ANNUAL WATER BUDGET BALANCE	0.0000	0.037	0.00

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ANNUAL TOTALS FOR YEAR 10

	INCHES	CU. FEET	PERCENT
PRECIPITATION	71.32	258891.562	100.00
RUNOFF	0.308	1117.734	0.43
POTENTIAL EVAPOTRANSPIRATION	54.532	197951.062	
ACTUAL EVAPOTRANSPIRATION	31.134	113015.211	43.65
DRAINAGE COLLECTED FROM LAYER 4	33.9514	123243.445	47.60
PERC./LEAKAGE THROUGH LAYER 5	3.831333	13907.741	5.37
AVG. HEAD ON TOP OF LAYER 5	0.0247		

DRAINAGE COLLECTED FROM LAYER 6	3.8312	13907.231	5.37
PERC./LEAKAGE THROUGH LAYER 7	0.000003	0.010	0.00
AVG. HEAD ON TOP OF LAYER 7	0.0028		
PERC./LEAKAGE THROUGH LAYER 8	0.000003	0.010	0.00
CHANGE IN WATER STORAGE	2.096	7608.012	2.94
SOIL WATER AT START OF YEAR	231.678	840991.750	
SOIL WATER AT END OF YEAR	233.774	848599.812	
INTERCEPTION WATER AT START OF YEAR	0.000	0.000	
INTERCEPTION WATER AT END OF YEAR	0.000	0.000	
SNOW WATER AT START OF YEAR	0.000	0.000	0.00
SNOW WATER AT END OF YEAR	0.000	0.000	0.00
ANNUAL WATER BUDGET BALANCE	0.0000	-0.074	0.00

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ANNUAL TOTALS FOR YEAR 11

	INCHES	CU. FEET	PERCENT
PRECIPITATION	65.26	236893.766	100.00
RUNOFF	0.125	455.422	0.19
POTENTIAL EVAPOTRANSPIRATION	54.803	198934.250	
ACTUAL EVAPOTRANSPIRATION	30.643	111235.500	46.96
DRAINAGE COLLECTED FROM LAYER 4	33.2071	120541.812	50.88
PERC./LEAKAGE THROUGH LAYER 5	3.801236	13798.485	5.82
AVG. HEAD ON TOP OF LAYER 5	0.0241		
DRAINAGE COLLECTED FROM LAYER 6	3.8013	13798.660	5.82
PERC./LEAKAGE THROUGH LAYER 7	0.000003	0.010	0.00
AVG. HEAD ON TOP OF LAYER 7	0.0028		
PERC./LEAKAGE THROUGH LAYER 8	0.000003	0.010	0.00
CHANGE IN WATER STORAGE	-2.517	-9137.609	-3.86
SOIL WATER AT START OF YEAR	233.774	848599.812	
SOIL WATER AT END OF YEAR	231.250	839437.500	

Proposed LF-Intermediate Cover

INTERCEPTION WATER AT START OF YEAR	0.000	0.000	
INTERCEPTION WATER AT END OF YEAR	0.007	24.684	
SNOW WATER AT START OF YEAR	0.000	0.000	0.00
SNOW WATER AT END OF YEAR	0.000	0.000	0.00
ANNUAL WATER BUDGET BALANCE	0.0000	-0.010	0.00

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ANNUAL TOTALS FOR YEAR 12

	INCHES	CU. FEET	PERCENT
PRECIPITATION	52.45	190393.484	100.00
RUNOFF	0.000	0.000	0.00
POTENTIAL EVAPOTRANSPIRATION	55.172	200276.109	
ACTUAL EVAPOTRANSPIRATION	28.595	103798.125	54.52
DRAINAGE COLLECTED FROM LAYER 4	25.7660	93530.680	49.12
PERC./LEAKAGE THROUGH LAYER 5	3.367026	12222.305	6.42
AVG. HEAD ON TOP OF LAYER 5	0.0187		
DRAINAGE COLLECTED FROM LAYER 6	3.3672	12222.766	6.42
PERC./LEAKAGE THROUGH LAYER 7	0.000003	0.010	0.00
AVG. HEAD ON TOP OF LAYER 7	0.0024		
PERC./LEAKAGE THROUGH LAYER 8	0.000003	0.010	0.00
CHANGE IN WATER STORAGE	-5.278	-19158.031	-10.06
SOIL WATER AT START OF YEAR	231.250	839437.500	
SOIL WATER AT END OF YEAR	225.979	820304.188	
INTERCEPTION WATER AT START OF YEAR	0.007	24.684	
INTERCEPTION WATER AT END OF YEAR	0.000	0.000	
SNOW WATER AT START OF YEAR	0.000	0.000	0.00
SNOW WATER AT END OF YEAR	0.000	0.000	0.00
ANNUAL WATER BUDGET BALANCE	0.0000	-0.048	0.00

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ANNUAL TOTALS FOR YEAR 13

	INCHES	CU. FEET	PERCENT
PRECIPITATION	54.63	198306.891	100.00
RUNOFF	0.211	767.420	0.39
POTENTIAL EVAPOTRANSPIRATION	55.320	200811.297	
ACTUAL EVAPOTRANSPIRATION	26.389	95793.414	48.31
DRAINAGE COLLECTED FROM LAYER 4	24.7974	90014.648	45.39
PERC./LEAKAGE THROUGH LAYER 5	3.298777	11974.562	6.04
AVG. HEAD ON TOP OF LAYER 5	0.0180		
DRAINAGE COLLECTED FROM LAYER 6	3.3000	11979.062	6.04
PERC./LEAKAGE THROUGH LAYER 7	0.000003	0.009	0.00
AVG. HEAD ON TOP OF LAYER 7	0.0024		
PERC./LEAKAGE THROUGH LAYER 8	0.000003	0.009	0.00
CHANGE IN WATER STORAGE	-0.068	-247.812	-0.12
SOIL WATER AT START OF YEAR	225.979	820304.188	
SOIL WATER AT END OF YEAR	225.911	820056.312	
INTERCEPTION WATER AT START OF YEAR	0.000	0.000	
INTERCEPTION WATER AT END OF YEAR	0.000	0.000	
SNOW WATER AT START OF YEAR	0.000	0.000	0.00
SNOW WATER AT END OF YEAR	0.000	0.000	0.00
ANNUAL WATER BUDGET BALANCE	0.0000	0.164	0.00

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ANNUAL TOTALS FOR YEAR 14

	INCHES	CU. FEET	PERCENT
PRECIPITATION	59.92	217509.609	100.00
RUNOFF	0.558	2026.976	0.93

POTENTIAL EVAPOTRANSPIRATION	54.090	196346.109	
ACTUAL EVAPOTRANSPIRATION	28.210	102400.586	47.08
DRAINAGE COLLECTED FROM LAYER 4	27.7481	100725.648	46.31
PERC./LEAKAGE THROUGH LAYER 5	3.481380	12637.407	5.81
AVG. HEAD ON TOP OF LAYER 5	0.0202		
DRAINAGE COLLECTED FROM LAYER 6	3.4813	12637.090	5.81
PERC./LEAKAGE THROUGH LAYER 7	0.000003	0.010	0.00
AVG. HEAD ON TOP OF LAYER 7	0.0025		
PERC./LEAKAGE THROUGH LAYER 8	0.000003	0.010	0.00
CHANGE IN WATER STORAGE	-0.077	-280.603	-0.13
SOIL WATER AT START OF YEAR	225.911	820056.312	
SOIL WATER AT END OF YEAR	225.834	819775.750	
INTERCEPTION WATER AT START OF YEAR	0.000	0.000	
INTERCEPTION WATER AT END OF YEAR	0.000	0.000	
SNOW WATER AT START OF YEAR	0.000	0.000	0.00
SNOW WATER AT END OF YEAR	0.000	0.000	0.00
ANNUAL WATER BUDGET BALANCE	0.0000	-0.087	0.00

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ANNUAL TOTALS FOR YEAR 15

	INCHES	CU. FEET	PERCENT
PRECIPITATION	56.04	203425.203	100.00
RUNOFF	0.153	556.658	0.27
POTENTIAL EVAPOTRANSPIRATION	56.640	205602.625	
ACTUAL EVAPOTRANSPIRATION	25.575	92836.570	45.64
DRAINAGE COLLECTED FROM LAYER 4	29.4994	107082.945	52.64
PERC./LEAKAGE THROUGH LAYER 5	3.610497	13106.105	6.44
AVG. HEAD ON TOP OF LAYER 5	0.0214		
DRAINAGE COLLECTED FROM LAYER 6	3.6111	13108.460	6.44

PERC./LEAKAGE THROUGH LAYER 7	0.000003	0.010	0.00
AVG. HEAD ON TOP OF LAYER 7	0.0026		
PERC./LEAKAGE THROUGH LAYER 8	0.000003	0.010	0.00
CHANGE IN WATER STORAGE	-2.799	-10159.414	-4.99
SOIL WATER AT START OF YEAR	225.834	819775.750	
SOIL WATER AT END OF YEAR	223.035	809616.312	
INTERCEPTION WATER AT START OF YEAR	0.000	0.000	
INTERCEPTION WATER AT END OF YEAR	0.000	0.000	
SNOW WATER AT START OF YEAR	0.000	0.000	0.00
SNOW WATER AT END OF YEAR	0.000	0.000	0.00
ANNUAL WATER BUDGET BALANCE	0.0000	-0.023	0.00

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ANNUAL TOTALS FOR YEAR 16

	INCHES	CU. FEET	PERCENT
PRECIPITATION	50.58	183605.375	100.00
RUNOFF	0.043	155.588	0.08
POTENTIAL EVAPOTRANSPIRATION	55.167	200257.641	
ACTUAL EVAPOTRANSPIRATION	26.671	96816.680	52.73
DRAINAGE COLLECTED FROM LAYER 4	21.5276	78145.031	42.56
PERC./LEAKAGE THROUGH LAYER 5	3.014374	10942.180	5.96
AVG. HEAD ON TOP OF LAYER 5	0.0156		
DRAINAGE COLLECTED FROM LAYER 6	3.0126	10935.641	5.96
PERC./LEAKAGE THROUGH LAYER 7	0.000003	0.009	0.00
AVG. HEAD ON TOP OF LAYER 7	0.0022		
PERC./LEAKAGE THROUGH LAYER 8	0.000003	0.009	0.00
CHANGE IN WATER STORAGE	-0.674	-2447.547	-1.33
SOIL WATER AT START OF YEAR	223.035	809616.312	
SOIL WATER AT END OF YEAR	222.361	807168.812	
INTERCEPTION WATER AT START OF YEAR	0.000	0.000	

INTERCEPTION WATER AT END OF YEAR	0.000	0.000	
SNOW WATER AT START OF YEAR	0.000	0.000	0.00
SNOW WATER AT END OF YEAR	0.000	0.000	0.00
ANNUAL WATER BUDGET BALANCE	0.0000	-0.034	0.00

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ANNUAL TOTALS FOR YEAR 17

	INCHES	CU. FEET	PERCENT
PRECIPITATION	70.44	255697.203	100.00
RUNOFF	0.234	849.494	0.33
POTENTIAL EVAPOTRANSPIRATION	55.309	200771.188	
ACTUAL EVAPOTRANSPIRATION	32.765	118936.992	46.51
DRAINAGE COLLECTED FROM LAYER 4	26.7162	96979.930	37.93
PERC./LEAKAGE THROUGH LAYER 5	3.326207	12074.132	4.72
AVG. HEAD ON TOP OF LAYER 5	0.0194		
DRAINAGE COLLECTED FROM LAYER 6	3.3273	12077.927	4.72
PERC./LEAKAGE THROUGH LAYER 7	0.000003	0.009	0.00
AVG. HEAD ON TOP OF LAYER 7	0.0024		
PERC./LEAKAGE THROUGH LAYER 8	0.000003	0.009	0.00
CHANGE IN WATER STORAGE	7.397	26852.838	10.50
SOIL WATER AT START OF YEAR	222.361	807168.812	
SOIL WATER AT END OF YEAR	229.758	834021.688	
INTERCEPTION WATER AT START OF YEAR	0.000	0.000	
INTERCEPTION WATER AT END OF YEAR	0.000	0.000	
SNOW WATER AT START OF YEAR	0.000	0.000	0.00
SNOW WATER AT END OF YEAR	0.000	0.000	0.00
ANNUAL WATER BUDGET BALANCE	0.0000	0.010	0.00

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ANNUAL TOTALS FOR YEAR 18

	INCHES	CU. FEET	PERCENT
PRECIPITATION	57.83	209922.891	100.00
RUNOFF	0.504	1828.946	0.87
POTENTIAL EVAPOTRANSPIRATION	54.640	198342.125	
ACTUAL EVAPOTRANSPIRATION	28.945	105072.148	50.05
DRAINAGE COLLECTED FROM LAYER 4	30.5657	110953.383	52.85
PERC./LEAKAGE THROUGH LAYER 5	3.668350	13316.110	6.34
AVG. HEAD ON TOP OF LAYER 5	0.0221		
DRAINAGE COLLECTED FROM LAYER 6	3.6678	13314.117	6.34
PERC./LEAKAGE THROUGH LAYER 7	0.000003	0.010	0.00
AVG. HEAD ON TOP OF LAYER 7	0.0027		
PERC./LEAKAGE THROUGH LAYER 8	0.000003	0.010	0.00
CHANGE IN WATER STORAGE	-5.853	-21245.713	-10.12
SOIL WATER AT START OF YEAR	229.758	834021.688	
SOIL WATER AT END OF YEAR	223.905	812775.938	
INTERCEPTION WATER AT START OF YEAR	0.000	0.000	
INTERCEPTION WATER AT END OF YEAR	0.000	0.000	
SNOW WATER AT START OF YEAR	0.000	0.000	0.00
SNOW WATER AT END OF YEAR	0.000	0.000	0.00
ANNUAL WATER BUDGET BALANCE	0.0000	0.014	0.00

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ANNUAL TOTALS FOR YEAR 19

	INCHES	CU. FEET	PERCENT
PRECIPITATION	75.35	273520.531	100.00
RUNOFF	0.203	735.085	0.27
POTENTIAL EVAPOTRANSPIRATION	52.829	191769.641	

ACTUAL EVAPOTRANSPIRATION	34.824	126411.914	46.22
DRAINAGE COLLECTED FROM LAYER 4	28.7316	104295.586	38.13
PERC./LEAKAGE THROUGH LAYER 5	3.473348	12608.254	4.61
AVG. HEAD ON TOP OF LAYER 5	0.0208		
DRAINAGE COLLECTED FROM LAYER 6	3.4729	12606.489	4.61
PERC./LEAKAGE THROUGH LAYER 7	0.000003	0.010	0.00
AVG. HEAD ON TOP OF LAYER 7	0.0025		
PERC./LEAKAGE THROUGH LAYER 8	0.000003	0.010	0.00
CHANGE IN WATER STORAGE	8.119	29471.422	10.77
SOIL WATER AT START OF YEAR	223.905	812775.938	
SOIL WATER AT END OF YEAR	232.016	842219.000	
INTERCEPTION WATER AT START OF YEAR	0.000	0.000	
INTERCEPTION WATER AT END OF YEAR	0.008	28.353	
SNOW WATER AT START OF YEAR	0.000	0.000	0.00
SNOW WATER AT END OF YEAR	0.000	0.000	0.00
ANNUAL WATER BUDGET BALANCE	0.0000	0.063	0.00

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ANNUAL TOTALS FOR YEAR 20

	INCHES	CU. FEET	PERCENT
PRECIPITATION	59.49	215948.734	100.00
RUNOFF	0.043	156.247	0.07
POTENTIAL EVAPOTRANSPIRATION	57.884	210118.578	
ACTUAL EVAPOTRANSPIRATION	30.391	110318.258	51.09
DRAINAGE COLLECTED FROM LAYER 4	34.3470	124679.781	57.74
PERC./LEAKAGE THROUGH LAYER 5	3.912986	14204.138	6.58
AVG. HEAD ON TOP OF LAYER 5	0.0249		
DRAINAGE COLLECTED FROM LAYER 6	3.9136	14206.434	6.58
PERC./LEAKAGE THROUGH LAYER 7	0.000003	0.010	0.00

AVG. HEAD ON TOP OF LAYER 7	0.0028		
PERC./LEAKAGE THROUGH LAYER 8	0.000003	0.010	0.00
CHANGE IN WATER STORAGE	-9.204	-33412.051	-15.47
SOIL WATER AT START OF YEAR	232.016	842219.000	
SOIL WATER AT END OF YEAR	222.820	808835.312	
INTERCEPTION WATER AT START OF YEAR	0.008	28.353	
INTERCEPTION WATER AT END OF YEAR	0.000	0.000	
SNOW WATER AT START OF YEAR	0.000	0.000	0.00
SNOW WATER AT END OF YEAR	0.000	0.000	0.00
ANNUAL WATER BUDGET BALANCE	0.0000	0.062	0.00

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ANNUAL TOTALS FOR YEAR 21

	INCHES	CU. FEET	PERCENT
PRECIPITATION	65.64	238273.188	100.00
RUNOFF	0.000	0.254	0.00
POTENTIAL EVAPOTRANSPIRATION	55.173	200277.875	
ACTUAL EVAPOTRANSPIRATION	31.339	113760.883	47.74
DRAINAGE COLLECTED FROM LAYER 4	27.1046	98389.719	41.29
PERC./LEAKAGE THROUGH LAYER 5	3.423182	12426.151	5.22
AVG. HEAD ON TOP OF LAYER 5	0.0196		
DRAINAGE COLLECTED FROM LAYER 6	3.4242	12430.017	5.22
PERC./LEAKAGE THROUGH LAYER 7	0.000003	0.010	0.00
AVG. HEAD ON TOP OF LAYER 7	0.0025		
PERC./LEAKAGE THROUGH LAYER 8	0.000003	0.010	0.00
CHANGE IN WATER STORAGE	3.772	13692.149	5.75
SOIL WATER AT START OF YEAR	222.820	808835.312	
SOIL WATER AT END OF YEAR	226.592	822527.438	
INTERCEPTION WATER AT START OF YEAR	0.000	0.000	
INTERCEPTION WATER AT END OF YEAR	0.000	0.000	

SNOW WATER AT START OF YEAR	0.000	0.000	0.00
SNOW WATER AT END OF YEAR	0.000	0.000	0.00
ANNUAL WATER BUDGET BALANCE	0.0000	0.154	0.00

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ANNUAL TOTALS FOR YEAR 22

	INCHES	CU. FEET	PERCENT
PRECIPITATION	71.54	259690.203	100.00
RUNOFF	0.052	189.340	0.07
POTENTIAL EVAPOTRANSPIRATION	53.971	195913.875	
ACTUAL EVAPOTRANSPIRATION	29.923	108620.281	41.83
DRAINAGE COLLECTED FROM LAYER 4	25.2108	91515.148	35.24
PERC./LEAKAGE THROUGH LAYER 5	3.209039	11648.813	4.49
AVG. HEAD ON TOP OF LAYER 5	0.0183		
DRAINAGE COLLECTED FROM LAYER 6	3.2085	11646.823	4.48
PERC./LEAKAGE THROUGH LAYER 7	0.000003	0.009	0.00
AVG. HEAD ON TOP OF LAYER 7	0.0023		
PERC./LEAKAGE THROUGH LAYER 8	0.000003	0.009	0.00
CHANGE IN WATER STORAGE	13.146	47718.637	18.38
SOIL WATER AT START OF YEAR	226.592	822527.438	
SOIL WATER AT END OF YEAR	239.737	870246.062	
INTERCEPTION WATER AT START OF YEAR	0.000	0.000	
INTERCEPTION WATER AT END OF YEAR	0.000	0.000	
SNOW WATER AT START OF YEAR	0.000	0.000	0.00
SNOW WATER AT END OF YEAR	0.000	0.000	0.00
ANNUAL WATER BUDGET BALANCE	0.0000	-0.051	0.00

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Proposed LF-Intermediate Cover

ANNUAL TOTALS FOR YEAR 23

	INCHES	CU. FEET	PERCENT
PRECIPITATION	64.39	233735.734	100.00
RUNOFF	0.022	80.953	0.03
POTENTIAL EVAPOTRANSPIRATION	54.842	199077.625	
ACTUAL EVAPOTRANSPIRATION	30.604	111092.367	47.53
DRAINAGE COLLECTED FROM LAYER 4	38.1070	138328.328	59.18
PERC./LEAKAGE THROUGH LAYER 5	4.095190	14865.540	6.36
AVG. HEAD ON TOP OF LAYER 5	0.0276		
DRAINAGE COLLECTED FROM LAYER 6	4.0944	14862.655	6.36
PERC./LEAKAGE THROUGH LAYER 7	0.000003	0.010	0.00
AVG. HEAD ON TOP OF LAYER 7	0.0030		
PERC./LEAKAGE THROUGH LAYER 8	0.000003	0.010	0.00
CHANGE IN WATER STORAGE	-8.438	-30628.514	-13.10
SOIL WATER AT START OF YEAR	239.737	870246.062	
SOIL WATER AT END OF YEAR	231.300	839617.562	
INTERCEPTION WATER AT START OF YEAR	0.000	0.000	
INTERCEPTION WATER AT END OF YEAR	0.000	0.000	
SNOW WATER AT START OF YEAR	0.000	0.000	0.00
SNOW WATER AT END OF YEAR	0.000	0.000	0.00
ANNUAL WATER BUDGET BALANCE	0.0000	-0.085	0.00

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ANNUAL TOTALS FOR YEAR 24

	INCHES	CU. FEET	PERCENT
PRECIPITATION	80.16	290980.875	100.00
RUNOFF	0.483	1754.090	0.60
POTENTIAL EVAPOTRANSPIRATION	55.334	200861.797	
ACTUAL EVAPOTRANSPIRATION	32.455	117813.086	40.49

DRAINAGE COLLECTED FROM LAYER 4	35.9782	130600.992	44.88
PERC./LEAKAGE THROUGH LAYER 5	3.938642	14297.271	4.91
AVG. HEAD ON TOP OF LAYER 5	0.0260		
DRAINAGE COLLECTED FROM LAYER 6	3.9378	14294.122	4.91
PERC./LEAKAGE THROUGH LAYER 7	0.000003	0.010	0.00
AVG. HEAD ON TOP OF LAYER 7	0.0028		
PERC./LEAKAGE THROUGH LAYER 8	0.000003	0.010	0.00
CHANGE IN WATER STORAGE	7.305	26518.619	9.11
SOIL WATER AT START OF YEAR	231.300	839617.562	
SOIL WATER AT END OF YEAR	238.605	866136.188	
INTERCEPTION WATER AT START OF YEAR	0.000	0.000	
INTERCEPTION WATER AT END OF YEAR	0.000	0.000	
SNOW WATER AT START OF YEAR	0.000	0.000	0.00
SNOW WATER AT END OF YEAR	0.000	0.000	0.00
ANNUAL WATER BUDGET BALANCE	0.0000	-0.055	0.00

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ANNUAL TOTALS FOR YEAR 25

	INCHES	CU. FEET	PERCENT
PRECIPITATION	64.20	233046.047	100.00
RUNOFF	0.074	270.189	0.12
POTENTIAL EVAPOTRANSPIRATION	53.792	195263.141	
ACTUAL EVAPOTRANSPIRATION	34.213	124194.602	53.29
DRAINAGE COLLECTED FROM LAYER 4	35.8956	130301.039	55.91
PERC./LEAKAGE THROUGH LAYER 5	3.995844	14504.913	6.22
AVG. HEAD ON TOP OF LAYER 5	0.0261		
DRAINAGE COLLECTED FROM LAYER 6	3.9968	14508.226	6.23
PERC./LEAKAGE THROUGH LAYER 7	0.000003	0.010	0.00
AVG. HEAD ON TOP OF LAYER 7	0.0029		

PERC./LEAKAGE THROUGH LAYER 8	0.000003	0.010	0.00
CHANGE IN WATER STORAGE	-9.980	-36228.160	-15.55
SOIL WATER AT START OF YEAR	238.605	866136.188	
SOIL WATER AT END OF YEAR	228.625	829908.000	
INTERCEPTION WATER AT START OF YEAR	0.000	0.000	
INTERCEPTION WATER AT END OF YEAR	0.000	0.000	
SNOW WATER AT START OF YEAR	0.000	0.000	0.00
SNOW WATER AT END OF YEAR	0.000	0.000	0.00
ANNUAL WATER BUDGET BALANCE	0.0000	0.129	0.00

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ANNUAL TOTALS FOR YEAR 26

	INCHES	CU. FEET	PERCENT
PRECIPITATION	67.32	244371.625	100.00
RUNOFF	0.231	838.847	0.34
POTENTIAL EVAPOTRANSPIRATION	55.491	201433.438	
ACTUAL EVAPOTRANSPIRATION	30.107	109287.789	44.72
DRAINAGE COLLECTED FROM LAYER 4	26.2398	95250.500	38.98
PERC./LEAKAGE THROUGH LAYER 5	3.328543	12082.610	4.94
AVG. HEAD ON TOP OF LAYER 5	0.0191		
DRAINAGE COLLECTED FROM LAYER 6	3.3282	12081.441	4.94
PERC./LEAKAGE THROUGH LAYER 7	0.000003	0.009	0.00
AVG. HEAD ON TOP OF LAYER 7	0.0024		
PERC./LEAKAGE THROUGH LAYER 8	0.000003	0.009	0.00
CHANGE IN WATER STORAGE	7.414	26912.992	11.01
SOIL WATER AT START OF YEAR	228.625	829908.000	
SOIL WATER AT END OF YEAR	236.039	856821.000	
INTERCEPTION WATER AT START OF YEAR	0.000	0.000	
INTERCEPTION WATER AT END OF YEAR	0.000	0.000	
SNOW WATER AT START OF YEAR	0.000	0.000	0.00

SNOW WATER AT END OF YEAR	0.000	0.000	0.00
ANNUAL WATER BUDGET BALANCE	0.0000	0.041	0.00

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ANNUAL TOTALS FOR YEAR 27

	INCHES	CU. FEET	PERCENT
PRECIPITATION	61.15	221974.500	100.00
RUNOFF	0.561	2035.103	0.92
POTENTIAL EVAPOTRANSPIRATION	55.851	202738.734	
ACTUAL EVAPOTRANSPIRATION	30.187	109578.812	49.37
DRAINAGE COLLECTED FROM LAYER 4	32.0280	116261.586	52.38
PERC./LEAKAGE THROUGH LAYER 5	3.763540	13661.649	6.15
AVG. HEAD ON TOP OF LAYER 5	0.0233		
DRAINAGE COLLECTED FROM LAYER 6	3.7647	13666.020	6.16
PERC./LEAKAGE THROUGH LAYER 7	0.000003	0.010	0.00
AVG. HEAD ON TOP OF LAYER 7	0.0027		
PERC./LEAKAGE THROUGH LAYER 8	0.000003	0.010	0.00
CHANGE IN WATER STORAGE	-5.390	-19567.082	-8.82
SOIL WATER AT START OF YEAR	236.039	856821.000	
SOIL WATER AT END OF YEAR	230.648	837253.938	
INTERCEPTION WATER AT START OF YEAR	0.000	0.000	
INTERCEPTION WATER AT END OF YEAR	0.000	0.000	
SNOW WATER AT START OF YEAR	0.000	0.000	0.00
SNOW WATER AT END OF YEAR	0.000	0.000	0.00
ANNUAL WATER BUDGET BALANCE	0.0000	0.042	0.00

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ANNUAL TOTALS FOR YEAR 28

Proposed LF-Intermediate Cover

	INCHES	CU. FEET	PERCENT
PRECIPITATION	65.06	236167.797	100.00
RUNOFF	0.462	1678.811	0.71
POTENTIAL EVAPOTRANSPIRATION	53.479	194130.438	
ACTUAL EVAPOTRANSPIRATION	31.379	113905.242	48.23
DRAINAGE COLLECTED FROM LAYER 4	26.3729	95733.727	40.54
PERC./LEAKAGE THROUGH LAYER 5	3.363435	12209.269	5.17
AVG. HEAD ON TOP OF LAYER 5	0.0191		
DRAINAGE COLLECTED FROM LAYER 6	3.3635	12209.488	5.17
PERC./LEAKAGE THROUGH LAYER 7	0.000003	0.009	0.00
AVG. HEAD ON TOP OF LAYER 7	0.0024		
PERC./LEAKAGE THROUGH LAYER 8	0.000003	0.009	0.00
CHANGE IN WATER STORAGE	3.482	12640.638	5.35
SOIL WATER AT START OF YEAR	230.648	837253.938	
SOIL WATER AT END OF YEAR	234.131	849894.562	
INTERCEPTION WATER AT START OF YEAR	0.000	0.000	
INTERCEPTION WATER AT END OF YEAR	0.000	0.000	
SNOW WATER AT START OF YEAR	0.000	0.000	0.00
SNOW WATER AT END OF YEAR	0.000	0.000	0.00
ANNUAL WATER BUDGET BALANCE	0.0000	-0.106	0.00

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ANNUAL TOTALS FOR YEAR 29

	INCHES	CU. FEET	PERCENT
PRECIPITATION	62.48	226802.375	100.00
RUNOFF	0.179	651.299	0.29
POTENTIAL EVAPOTRANSPIRATION	54.992	199620.500	
ACTUAL EVAPOTRANSPIRATION	27.296	99083.523	43.69
DRAINAGE COLLECTED FROM LAYER 4	32.1546	116721.023	51.46

PERC./LEAKAGE THROUGH LAYER 5	3.742116	13583.880	5.99
AVG. HEAD ON TOP OF LAYER 5	0.0233		
DRAINAGE COLLECTED FROM LAYER 6	3.7417	13582.358	5.99
PERC./LEAKAGE THROUGH LAYER 7	0.000003	0.010	0.00
AVG. HEAD ON TOP OF LAYER 7	0.0027		
PERC./LEAKAGE THROUGH LAYER 8	0.000003	0.010	0.00
CHANGE IN WATER STORAGE	-0.891	-3235.904	-1.43
SOIL WATER AT START OF YEAR	234.131	849894.562	
SOIL WATER AT END OF YEAR	233.239	846658.688	
INTERCEPTION WATER AT START OF YEAR	0.000	0.000	
INTERCEPTION WATER AT END OF YEAR	0.000	0.000	
SNOW WATER AT START OF YEAR	0.000	0.000	0.00
SNOW WATER AT END OF YEAR	0.000	0.000	0.00
ANNUAL WATER BUDGET BALANCE	0.0000	0.065	0.00

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ANNUAL TOTALS FOR YEAR 30

	INCHES	CU. FEET	PERCENT
PRECIPITATION	61.55	223426.484	100.00
RUNOFF	0.115	416.920	0.19
POTENTIAL EVAPOTRANSPIRATION	53.835	195422.859	
ACTUAL EVAPOTRANSPIRATION	29.889	108496.633	48.56
DRAINAGE COLLECTED FROM LAYER 4	30.0779	109182.656	48.87
PERC./LEAKAGE THROUGH LAYER 5	3.594167	13046.828	5.84
AVG. HEAD ON TOP OF LAYER 5	0.0218		
DRAINAGE COLLECTED FROM LAYER 6	3.5932	13043.313	5.84
PERC./LEAKAGE THROUGH LAYER 7	0.000003	0.010	0.00
AVG. HEAD ON TOP OF LAYER 7	0.0026		
PERC./LEAKAGE THROUGH LAYER 8	0.000003	0.010	0.00

CHANGE IN WATER STORAGE	-2.125	-7712.919	-3.45
SOIL WATER AT START OF YEAR	233.239	846658.688	
SOIL WATER AT END OF YEAR	231.115	838945.750	
INTERCEPTION WATER AT START OF YEAR	0.000	0.000	
INTERCEPTION WATER AT END OF YEAR	0.000	0.000	
SNOW WATER AT START OF YEAR	0.000	0.000	0.00
SNOW WATER AT END OF YEAR	0.000	0.000	0.00
ANNUAL WATER BUDGET BALANCE	0.0000	-0.141	0.00

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FINAL WATER STORAGE AT END OF YEAR 30

LAYER	(INCHES)	(VOL/VOL)
1	2.7819	0.2318
2	221.6291	0.3078
3	6.2911	0.2621
4	0.0305	0.1131
5	0.0000	0.0000
6	0.0056	0.0208
7	0.0000	0.0000
8	0.1867	0.7470
TOTAL WATER IN LAYERS	230.925	
SNOW WATER	0.000	
INTERCEPTION WATER	0.000	
TOTAL FINAL WATER	230.925	

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PEAK DAILY VALUES FOR YEARS 1 THROUGH 30

	(INCHES)	(CU. FT.)

Proposed LF-Intermediate Cover

PRECIPITATION	4.69	17024.701
RUNOFF	0.375	1362.0471
DRAINAGE COLLECTED FROM LAYER 4	0.24540	890.80957
PERCOLATION/LEAKAGE THROUGH LAYER 5	0.018263	66.29525
AVERAGE HEAD ON TOP OF LAYER 5	0.065	
MAXIMUM HEAD ON TOP OF LAYER 5	0.128	
LOCATION OF MAXIMUM HEAD IN LAYER 4 (DISTANCE FROM DRAIN)	2.4 FEET	
DRAINAGE COLLECTED FROM LAYER 6	0.01779	64.56208
PERCOLATION/LEAKAGE THROUGH LAYER 7	0.000000	0.00003
AVERAGE HEAD ON TOP OF LAYER 7	0.005	
MAXIMUM HEAD ON TOP OF LAYER 7	0.012	
LOCATION OF MAXIMUM HEAD IN LAYER 6 (DISTANCE FROM DRAIN)	0.0 FEET	
PERCOLATION/LEAKAGE THROUGH LAYER 8	0.000000	0.00003
SNOW WATER	0.91	3304.4434
MAXIMUM VEG. SOIL WATER (VOL/VOL)	0.3691	
MINIMUM VEG. SOIL WATER (VOL/VOL)	0.1040	

*** Maximum heads are computed using McEnroe's equations. ***

Reference: Maximum Saturated Depth over Landfill Liner
by Bruce M. McEnroe, University of Kansas
ASCE Journal of Environmental Engineering
Vol. 119, No. 2, March 1993, pp. 262-270.

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AVERAGE MONTHLY VALUES IN INCHES FOR YEARS 1 THROUGH 30

	JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
PRECIPITATION						
TOTALS	4.34 9.00	4.81 7.43	5.62 7.41	4.25 3.08	4.20 2.95	6.59 5.00
STD. DEVIATIONS	2.11 3.04	2.39 2.65	2.68 3.03	3.05 2.35	3.00 2.17	2.48 2.74

RUNOFF

TOTALS	0.014	0.007	0.007	0.012	0.010	0.021
	0.025	0.017	0.061	0.015	0.017	0.010
STD. DEVIATIONS	0.053	0.021	0.025	0.029	0.035	0.075
	0.083	0.046	0.101	0.070	0.050	0.027

POTENTIAL EVAPOTRANSPIRATION

TOTALS	2.417	2.706	4.163	5.384	6.702	7.082
	6.587	6.153	5.024	3.848	2.634	2.047
STD. DEVIATIONS	0.151	0.180	0.264	0.294	0.292	0.328
	0.413	0.311	0.324	0.273	0.172	0.182

ACTUAL EVAPOTRANSPIRATION

TOTALS	1.507	1.819	2.592	2.256	2.716	3.876
	5.031	4.096	3.086	1.319	0.997	1.140
STD. DEVIATIONS	0.218	0.285	0.673	1.143	1.144	1.026
	0.823	0.822	0.646	0.531	0.317	0.188

LATERAL DRAINAGE COLLECTED FROM LAYER 4

TOTALS	2.0633	2.2233	2.5912	2.6781	2.9826	2.7323
	2.1398	2.1531	2.3007	2.9252	2.9515	2.5820
STD. DEVIATIONS	0.7904	0.6406	0.9136	0.8698	0.7439	0.8376
	0.7915	0.9279	0.7776	0.9631	0.7126	0.8737

PERCOLATION/LEAKAGE THROUGH LAYER 5

TOTALS	0.2663	0.2705	0.3053	0.3107	0.3383	0.3188
	0.2720	0.2718	0.2809	0.3303	0.3323	0.3097
STD. DEVIATIONS	0.0637	0.0479	0.0690	0.0610	0.0560	0.0692
	0.0665	0.0731	0.0579	0.0685	0.0482	0.0626

LATERAL DRAINAGE COLLECTED FROM LAYER 6

TOTALS	0.2664	0.2704	0.3051	0.3104	0.3385	0.3192
	0.2720	0.2717	0.2813	0.3295	0.3324	0.3098
STD. DEVIATIONS	0.0637	0.0475	0.0694	0.0608	0.0562	0.0691
	0.0664	0.0731	0.0578	0.0682	0.0478	0.0625

PERCOLATION/LEAKAGE THROUGH LAYER 7

TOTALS	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
STD. DEVIATIONS	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000

PERCOLATION/LEAKAGE THROUGH LAYER 8

TOTALS	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
STD. DEVIATIONS	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000

AVERAGES OF MONTHLY AVERAGED DAILY HEADS (INCHES)

DAILY AVERAGE HEAD ON TOP OF LAYER 5

AVERAGES	0.0176	0.0209	0.0221	0.0236	0.0255	0.0241
	0.0183	0.0184	0.0203	0.0250	0.0260	0.0220
STD. DEVIATIONS	0.0067	0.0060	0.0078	0.0077	0.0064	0.0074
	0.0068	0.0079	0.0069	0.0082	0.0063	0.0075

DAILY AVERAGE HEAD ON TOP OF LAYER 7

AVERAGES	0.0023	0.0025	0.0026	0.0027	0.0029	0.0028
	0.0023	0.0023	0.0025	0.0028	0.0029	0.0026
STD. DEVIATIONS	0.0005	0.0004	0.0006	0.0005	0.0005	0.0006
	0.0006	0.0006	0.0005	0.0006	0.0004	0.0005

AVERAGE ANNUAL TOTALS & (STD. DEVIATIONS) FOR YEARS 1 THROUGH 30

	INCHES		CU. FEET	PERCENT
PRECIPITATION	64.69	(7.987)	234821.1	100.00
RUNOFF	0.214	(0.1826)	777.50	0.331
POTENTIAL EVAPOTRANSPIRATION	54.747	(0.9852)	198732.92	
ACTUAL EVAPOTRANSPIRATION	30.436	(2.4037)	110481.07	47.049
LATERAL DRAINAGE COLLECTED FROM LAYER 4	30.32308	(4.75846)	110072.789	46.87518
PERCOLATION/LEAKAGE THROUGH LAYER 5	3.60683	(0.32372)	13092.778	5.57564
AVERAGE HEAD ON TOP OF LAYER 5	0.022	(0.003)		
LATERAL DRAINAGE COLLECTED FROM LAYER 6	3.60677	(0.32372)	13092.578	5.57555
PERCOLATION/LEAKAGE THROUGH LAYER 7	0.00000	(0.00000)	0.010	0.00000
AVERAGE HEAD ON TOP OF LAYER 7	0.003	(0.000)		
PERCOLATION/LEAKAGE THROUGH LAYER 8	0.00000	(0.00000)	0.010	0.00000

Proposed LF-Intermediate Cover

CHANGE IN WATER STORAGE	0.109	(6.1181)	397.10	0.169

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**                                                                              **
**               HYDROLOGIC EVALUATION OF LANDFILL PERFORMANCE                  **
**                                                                              **
**               HELP Version 3.95 D          (10 August 2012)                  **
**                                developed at                                  **
**        Institute of Soil Science, University of Hamburg, Germany             **
**                                  based on                                    **
**               US HELP MODEL VERSION 3.07  (1 NOVEMBER 1997)                  **
**                   DEVELOPED BY ENVIRONMENTAL LABORATORY                      **
**                     USAE WATERWAYS EXPERIMENT STATION                        **
**              FOR USEPA RISK REDUCTION ENGINEERING LABORATORY                 **
**                                                                              **
**                                                                              **
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 TIME:  15.41     DATE:   6.02.2019


 PRECIPITATION DATA FILE:      C:\HDR_Projects\steelfield\Engineering Design\HELP
Model\Existing Landfill\precipitation.d4
 TEMPERATURE DATA FILE:        C:\HDR_Projects\steelfield\Engineering Design\HELP
Model\Existing Landfill\temperature.d7
 SOLAR RADIATION DATA FILE:    C:\HDR_Projects\steelfield\Engineering Design\HELP
Model\Existing Landfill\solar radiation.d13
 EVAPOTRANSPIRATION DATA F. 1: C:\HDR_Projects\steelfield\Engineering Design\HELP
Model\Existing Landfill\evapotranspiration.d11
 SOIL AND DESIGN DATA FILE  1: C:\HDR_Projects\steelfield\Engineering Design\HELP
Model\Existing Landfill\soil and design data-existing landfill.d10
 OUTPUT DATA FILE:             C:\HDR_Projects\steelfield\Engineering Design\HELP
Model\Existing Landfill\existing landfill.out
 YEARLY OUTPUT DATA FILE:      C:\HDR_Projects\steelfield\Engineering Design\HELP
Model\Existing Landfill\existing landfill.YR

 COLUMNS OF YEARLY OUTPUT DATA FILE:

      1    DATE OF ULTIMO (yyyy1231, years 2101 to 2200 from weather generator)
      2    PRECIPITATION (INCH)
      3    RUNOFF (INCH)
      4    POTENTIAL EVAPOTRANSPIRATION (INCH)
      5    ACTUAL EVAPOTRANSPIRATION (INCH)
      6    DRAIN #1: LATERAL DRAINAGE FROM LAYER  4 (INCH)
      7    LEAK  #1: PERCOLATION/LEAKAGE THROUGH LAYER  5 (INCH)
      8    DRAIN #2: LATERAL DRAINAGE FROM LAYER  6 (INCH)
      9    LEAK  #2: PERCOLATION/LEAKAGE THROUGH LAYER  7 (INCH)
     10    LEAK  #3: PERCOLATION/LEAKAGE THROUGH LAYER  8 (INCH)
     11    CHANGE IN TOTAL WATER STORAGE (INCH)
     12    CHANGE IN SOIL WATER STORAGE (INCH)
     13    CHANGE IN INTERCEPTION WATER STORAGE (INCH)
     14    CHANGE IN SNOW WATER STORAGE (INCH)
     15    ANNUAL WATER BUDGET BALANCE (INCH)


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      TITLE:  Senario 2: Existing Landfill-Intermediate Cover

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WEATHER DATA SOURCES

NOTE: PRECIPITATION DATA WAS SYNTHETICALLY GENERATED USING
COEFFICIENTS FOR TALLAHASSEE FLORIDA

NORMAL MEAN MONTHLY PRECIPITATION (INCHES)

JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
4.66	5.00	5.60	4.13	5.16	6.55
8.75	7.30	6.45	3.10	3.31	4.58

NOTE: TEMPERATURE DATA WAS SYNTHETICALLY GENERATED USING
COEFFICIENTS FOR TALLAHASSEE FLORIDA

NORMAL MEAN MONTHLY TEMPERATURE (DEGREES FAHRENHEIT)

JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
51.60	53.60	60.20	67.10	74.00	79.50
81.20	81.10	78.30	68.40	58.80	53.00

NOTE: SOLAR RADIATION DATA WAS SYNTHETICALLY GENERATED USING
COEFFICIENTS FOR TALLAHASSEE FLORIDA
AND STATION LATITUDE = 30.23 DEGREES

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LAYER DATA 1

VALID FOR 30 YEARS

NOTE: INITIAL MOISTURE CONTENT OF THE LAYERS AND SNOW WATER WERE
COMPUTED AS NEARLY STEADY-STATE VALUES BY THE PROGRAM.

LAYER 1

TYPE 1 - VERTICAL PERCOLATION LAYER
MATERIAL TEXTURE NUMBER 7

THICKNESS	=	12.00	INCHES
POROSITY	=	0.4730	VOL/VOL
FIELD CAPACITY	=	0.2220	VOL/VOL
WILTING POINT	=	0.1040	VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.2988	VOL/VOL
EFFECTIVE SAT. HYD. CONDUCT.	=	0.5200E-03	CM/SEC

LAYER 2

TYPE 1 - VERTICAL PERCOLATION LAYER
MATERIAL TEXTURE NUMBER 18

THICKNESS	=	1224.00	INCHES
POROSITY	=	0.6710	VOL/VOL
FIELD CAPACITY	=	0.2920	VOL/VOL
WILTING POINT	=	0.0770	VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.2990	VOL/VOL
EFFECTIVE SAT. HYD. CONDUCT.	=	0.1000E-02	CM/SEC

LAYER 3

TYPE 1 - VERTICAL PERCOLATION LAYER
MATERIAL TEXTURE NUMBER 0

THICKNESS	=	24.00	INCHES
POROSITY	=	0.5010	VOL/VOL
FIELD CAPACITY	=	0.2840	VOL/VOL
WILTING POINT	=	0.1350	VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.3262	VOL/VOL
EFFECTIVE SAT. HYD. CONDUCT.	=	0.1000E-03	CM/SEC

LAYER 4

TYPE 2 - LATERAL DRAINAGE LAYER
MATERIAL TEXTURE NUMBER 0

THICKNESS	=	0.30	INCHES
POROSITY	=	0.8500	VOL/VOL
FIELD CAPACITY	=	0.0100	VOL/VOL
WILTING POINT	=	0.0050	VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.0294	VOL/VOL
EFFECTIVE SAT. HYD. CONDUCT.	=	5.200	CM/SEC
SLOPE	=	2.00	PERCENT
DRAINAGE LENGTH	=	165.0	FEET

LAYER 5

TYPE 4 - FLEXIBLE MEMBRANE LINER
MATERIAL TEXTURE NUMBER 35

THICKNESS	=	0.06	INCHES
EFFECTIVE SAT. HYD. CONDUCT.	=	0.2000E-12	CM/SEC
FML PINHOLE DENSITY	=	2.00	HOLES/ACRE
FML INSTALLATION DEFECTS	=	2.00	HOLES/ACRE
FML PLACEMENT QUALITY	=	3 - GOOD	

LAYER 6

TYPE 2 - LATERAL DRAINAGE LAYER
MATERIAL TEXTURE NUMBER 0

existing landfill

THICKNESS	=	0.30	INCHES
POROSITY	=	0.8500	VOL/VOL
FIELD CAPACITY	=	0.0100	VOL/VOL
WILTING POINT	=	0.0050	VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.0149	VOL/VOL
EFFECTIVE SAT. HYD. CONDUCT.	=	5.200	CM/SEC
SLOPE	=	2.00	PERCENT
DRAINAGE LENGTH	=	165.0	FEET

LAYER 7

TYPE 4 - FLEXIBLE MEMBRANE LINER
MATERIAL TEXTURE NUMBER 35

THICKNESS	=	0.06	INCHES
EFFECTIVE SAT. HYD. CONDUCT.	=	0.2000E-12	CM/SEC
FML PINHOLE DENSITY	=	2.00	HOLES/ACRE
FML INSTALLATION DEFECTS	=	2.00	HOLES/ACRE
FML PLACEMENT QUALITY	=	3 - GOOD	

LAYER 8

TYPE 1 - VERTICAL PERCOLATION LAYER
MATERIAL TEXTURE NUMBER 17

THICKNESS	=	0.25	INCHES
POROSITY	=	0.7500	VOL/VOL
FIELD CAPACITY	=	0.7470	VOL/VOL
WILTING POINT	=	0.4000	VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.7470	VOL/VOL
EFFECTIVE SAT. HYD. CONDUCT.	=	0.3000E-08	CM/SEC

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GENERAL DESIGN AND EVAPORATIVE ZONE DATA 1

VALID FOR 30 YEARS

NOTE: SCS RUNOFF CURVE NUMBER WAS COMPUTED FROM DEFAULT SOIL DATA BASE USING SOIL TEXTURE # 4 WITH A FAIR STAND OF GRASS, A SURFACE SLOPE OF 5.% AND A SLOPE LENGTH OF 100. FEET.

SCS RUNOFF CURVE NUMBER	=	63.03	
FRACTION OF AREA ALLOWING RUNOFF	=	100.0	PERCENT
AREA PROJECTED ON HORIZONTAL PLANE	=	1.000	ACRES
EVAPORATIVE ZONE DEPTH	=	6.0	INCHES
INITIAL WATER IN EVAPORATIVE ZONE	=	1.866	INCHES
UPPER LIMIT OF EVAPORATIVE STORAGE	=	2.838	INCHES
FIELD CAPACITY OF EVAPORATIVE ZONE	=	1.332	INCHES
LOWER LIMIT OF EVAPORATIVE STORAGE	=	0.624	INCHES
SOIL EVAPORATION ZONE DEPTH	=	6.000	INCHES
INITIAL SNOW WATER	=	0.000	INCHES
INITIAL INTERCEPTION WATER	=	0.000	INCHES
INITIAL WATER IN LAYER MATERIALS	=	377.648	INCHES

TOTAL INITIAL WATER = 377.648 INCHES
TOTAL SUBSURFACE INFLOW = 0.00 INCHES/YEAR

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EVAPOTRANSPIRATION DATA 1

VALID FOR 30 YEARS

NOTE: EVAPOTRANSPIRATION DATA WAS OBTAINED FROM
TALLAHASSEE FLORIDA

STATION LATITUDE	= 30.23	DEGREES
MAXIMUM LEAF AREA INDEX	= 0.00	
START OF GROWING SEASON (JULIAN DATE)	= 36	
END OF GROWING SEASON (JULIAN DATE)	= 350	
EVAPORATIVE ZONE DEPTH	= 6.0	INCHES
AVERAGE ANNUAL WIND SPEED	= 6.60	MPH
AVERAGE 1ST QUARTER RELATIVE HUMIDITY	= 73.0	%
AVERAGE 2ND QUARTER RELATIVE HUMIDITY	= 73.0	%
AVERAGE 3RD QUARTER RELATIVE HUMIDITY	= 80.0	%
AVERAGE 4TH QUARTER RELATIVE HUMIDITY	= 77.0	%

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ANNUAL TOTALS FOR YEAR 1

	INCHES	CU. FEET	PERCENT
PRECIPITATION	48.59	176381.703	100.00
RUNOFF	0.044	159.800	0.09
POTENTIAL EVAPOTRANSPIRATION	54.031	196131.859	
ACTUAL EVAPOTRANSPIRATION	26.345	95632.633	54.22
DRAINAGE COLLECTED FROM LAYER 4	19.2979	70051.531	39.72
PERC./LEAKAGE THROUGH LAYER 5	2.902968	10537.775	5.97
AVG. HEAD ON TOP OF LAYER 5	0.0149		
DRAINAGE COLLECTED FROM LAYER 6	2.9030	10537.765	5.97
PERC./LEAKAGE THROUGH LAYER 7	0.000003	0.009	0.00
AVG. HEAD ON TOP OF LAYER 7	0.0022		
PERC./LEAKAGE THROUGH LAYER 8	0.000003	0.009	0.00
CHANGE IN WATER STORAGE	0.000	-0.111	0.00
SOIL WATER AT START OF YEAR	377.837	1371549.375	

SOIL WATER AT END OF YEAR	377.837	1371549.375	
INTERCEPTION WATER AT START OF YEAR	0.000	0.000	
INTERCEPTION WATER AT END OF YEAR	0.000	0.000	
SNOW WATER AT START OF YEAR	0.000	0.000	0.00
SNOW WATER AT END OF YEAR	0.000	0.000	0.00
ANNUAL WATER BUDGET BALANCE	0.0000	0.069	0.00

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ANNUAL TOTALS FOR YEAR 2

	INCHES	CU. FEET	PERCENT
PRECIPITATION	80.32	291561.531	100.00
RUNOFF	0.253	916.985	0.31
POTENTIAL EVAPOTRANSPIRATION	53.954	195854.750	
ACTUAL EVAPOTRANSPIRATION	35.514	128915.617	44.22
DRAINAGE COLLECTED FROM LAYER 4	28.1454	102167.750	35.04
PERC./LEAKAGE THROUGH LAYER 5	3.524379	12793.495	4.39
AVG. HEAD ON TOP OF LAYER 5	0.0216		
DRAINAGE COLLECTED FROM LAYER 6	3.5237	12791.069	4.39
PERC./LEAKAGE THROUGH LAYER 7	0.000003	0.009	0.00
AVG. HEAD ON TOP OF LAYER 7	0.0027		
PERC./LEAKAGE THROUGH LAYER 8	0.000003	0.009	0.00
CHANGE IN WATER STORAGE	12.884	46770.145	16.04
SOIL WATER AT START OF YEAR	377.837	1371549.375	
SOIL WATER AT END OF YEAR	390.722	1418319.500	
INTERCEPTION WATER AT START OF YEAR	0.000	0.000	
INTERCEPTION WATER AT END OF YEAR	0.000	0.000	
SNOW WATER AT START OF YEAR	0.000	0.000	0.00
SNOW WATER AT END OF YEAR	0.000	0.000	0.00
ANNUAL WATER BUDGET BALANCE	0.0000	-0.049	0.00

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ANNUAL TOTALS FOR YEAR 3

	INCHES	CU. FEET	PERCENT
PRECIPITATION	69.92	253809.578	100.00
RUNOFF	0.390	1415.960	0.56
POTENTIAL EVAPOTRANSPIRATION	54.644	198359.312	
ACTUAL EVAPOTRANSPIRATION	31.575	114618.711	45.16
DRAINAGE COLLECTED FROM LAYER 4	32.4582	117823.273	46.42
PERC./LEAKAGE THROUGH LAYER 5	3.857963	14004.406	5.52
AVG. HEAD ON TOP OF LAYER 5	0.0249		
DRAINAGE COLLECTED FROM LAYER 6	3.8581	14004.724	5.52
PERC./LEAKAGE THROUGH LAYER 7	0.000003	0.010	0.00
AVG. HEAD ON TOP OF LAYER 7	0.0030		
PERC./LEAKAGE THROUGH LAYER 8	0.000003	0.010	0.00
CHANGE IN WATER STORAGE	1.638	5947.049	2.34
SOIL WATER AT START OF YEAR	390.722	1418319.500	
SOIL WATER AT END OF YEAR	392.360	1424266.500	
INTERCEPTION WATER AT START OF YEAR	0.000	0.000	
INTERCEPTION WATER AT END OF YEAR	0.000	0.000	
SNOW WATER AT START OF YEAR	0.000	0.000	0.00
SNOW WATER AT END OF YEAR	0.000	0.000	0.00
ANNUAL WATER BUDGET BALANCE	0.0000	-0.122	0.00

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ANNUAL TOTALS FOR YEAR 4

	INCHES	CU. FEET	PERCENT
PRECIPITATION	58.15	211084.484	100.00

RUNOFF	0.156	567.879	0.27
POTENTIAL EVAPOTRANSPIRATION	54.811	198963.734	
ACTUAL EVAPOTRANSPIRATION	32.337	117383.000	55.61
DRAINAGE COLLECTED FROM LAYER 4	33.3001	120879.477	57.27
PERC./LEAKAGE THROUGH LAYER 5	3.976194	14433.583	6.84
AVG. HEAD ON TOP OF LAYER 5	0.0255		
DRAINAGE COLLECTED FROM LAYER 6	3.9754	14430.681	6.84
PERC./LEAKAGE THROUGH LAYER 7	0.000003	0.010	0.00
AVG. HEAD ON TOP OF LAYER 7	0.0030		
PERC./LEAKAGE THROUGH LAYER 8	0.000003	0.010	0.00
CHANGE IN WATER STORAGE	-11.619	-42176.371	-19.98
SOIL WATER AT START OF YEAR	392.360	1424266.500	
SOIL WATER AT END OF YEAR	380.741	1382090.125	
INTERCEPTION WATER AT START OF YEAR	0.000	0.000	
INTERCEPTION WATER AT END OF YEAR	0.000	0.000	
SNOW WATER AT START OF YEAR	0.000	0.000	0.00
SNOW WATER AT END OF YEAR	0.000	0.000	0.00
ANNUAL WATER BUDGET BALANCE	-0.0001	-0.190	0.00

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ANNUAL TOTALS FOR YEAR 5

	INCHES	CU. FEET	PERCENT
PRECIPITATION	70.21	254862.266	100.00
RUNOFF	0.164	593.762	0.23
POTENTIAL EVAPOTRANSPIRATION	53.436	193973.516	
ACTUAL EVAPOTRANSPIRATION	34.058	123631.688	48.51
DRAINAGE COLLECTED FROM LAYER 4	26.9041	97661.977	38.32
PERC./LEAKAGE THROUGH LAYER 5	3.489461	12666.743	4.97
AVG. HEAD ON TOP OF LAYER 5	0.0207		

DRAINAGE COLLECTED FROM LAYER 6	3.4893	12666.266	4.97
PERC./LEAKAGE THROUGH LAYER 7	0.000003	0.010	0.00
AVG. HEAD ON TOP OF LAYER 7	0.0027		
PERC./LEAKAGE THROUGH LAYER 8	0.000003	0.010	0.00
CHANGE IN WATER STORAGE	5.595	20308.635	7.97
SOIL WATER AT START OF YEAR	380.741	1382090.125	
SOIL WATER AT END OF YEAR	386.336	1402398.875	
INTERCEPTION WATER AT START OF YEAR	0.000	0.000	
INTERCEPTION WATER AT END OF YEAR	0.000	0.000	
SNOW WATER AT START OF YEAR	0.000	0.000	0.00
SNOW WATER AT END OF YEAR	0.000	0.000	0.00
ANNUAL WATER BUDGET BALANCE	0.0000	-0.087	0.00

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ANNUAL TOTALS FOR YEAR 6

	INCHES	CU. FEET	PERCENT
PRECIPITATION	73.90	268256.906	100.00
RUNOFF	0.039	143.212	0.05
POTENTIAL EVAPOTRANSPIRATION	54.288	197066.312	
ACTUAL EVAPOTRANSPIRATION	35.798	129948.070	48.44
DRAINAGE COLLECTED FROM LAYER 4	28.6476	103990.805	38.77
PERC./LEAKAGE THROUGH LAYER 5	3.598630	13063.027	4.87
AVG. HEAD ON TOP OF LAYER 5	0.0220		
DRAINAGE COLLECTED FROM LAYER 6	3.5995	13066.005	4.87
PERC./LEAKAGE THROUGH LAYER 7	0.000003	0.009	0.00
AVG. HEAD ON TOP OF LAYER 7	0.0028		
PERC./LEAKAGE THROUGH LAYER 8	0.000003	0.009	0.00
CHANGE IN WATER STORAGE	5.815	21108.902	7.87
SOIL WATER AT START OF YEAR	386.336	1402398.875	
SOIL WATER AT END OF YEAR	392.151	1423507.625	

INTERCEPTION WATER AT START OF YEAR	0.000	0.000	
INTERCEPTION WATER AT END OF YEAR	0.000	0.000	
SNOW WATER AT START OF YEAR	0.000	0.000	0.00
SNOW WATER AT END OF YEAR	0.000	0.000	0.00
ANNUAL WATER BUDGET BALANCE	0.0000	-0.080	0.00

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ANNUAL TOTALS FOR YEAR 7

	INCHES	CU. FEET	PERCENT
PRECIPITATION	68.37	248183.109	100.00
RUNOFF	0.622	2257.355	0.91
POTENTIAL EVAPOTRANSPIRATION	54.391	197439.922	
ACTUAL EVAPOTRANSPIRATION	31.810	115469.969	46.53
DRAINAGE COLLECTED FROM LAYER 4	31.5267	114441.906	46.11
PERC./LEAKAGE THROUGH LAYER 5	3.795331	13777.052	5.55
AVG. HEAD ON TOP OF LAYER 5	0.0242		
DRAINAGE COLLECTED FROM LAYER 6	3.7938	13771.324	5.55
PERC./LEAKAGE THROUGH LAYER 7	0.000003	0.010	0.00
AVG. HEAD ON TOP OF LAYER 7	0.0029		
PERC./LEAKAGE THROUGH LAYER 8	0.000003	0.010	0.00
CHANGE IN WATER STORAGE	0.618	2242.717	0.90
SOIL WATER AT START OF YEAR	392.151	1423507.625	
SOIL WATER AT END OF YEAR	392.769	1425750.375	
INTERCEPTION WATER AT START OF YEAR	0.000	0.000	
INTERCEPTION WATER AT END OF YEAR	0.000	0.000	
SNOW WATER AT START OF YEAR	0.000	0.000	0.00
SNOW WATER AT END OF YEAR	0.000	0.000	0.00
ANNUAL WATER BUDGET BALANCE	0.0000	-0.176	0.00

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ANNUAL TOTALS FOR YEAR 8

	INCHES	CU. FEET	PERCENT
PRECIPITATION	66.99	243173.688	100.00
RUNOFF	0.406	1473.358	0.61
POTENTIAL EVAPOTRANSPIRATION	54.866	199164.266	
ACTUAL EVAPOTRANSPIRATION	31.554	114539.531	47.10
DRAINAGE COLLECTED FROM LAYER 4	32.2761	117162.320	48.18
PERC./LEAKAGE THROUGH LAYER 5	3.927659	14257.401	5.86
AVG. HEAD ON TOP OF LAYER 5	0.0247		
DRAINAGE COLLECTED FROM LAYER 6	3.9293	14263.212	5.87
PERC./LEAKAGE THROUGH LAYER 7	0.000003	0.010	0.00
AVG. HEAD ON TOP OF LAYER 7	0.0030		
PERC./LEAKAGE THROUGH LAYER 8	0.000003	0.010	0.00
CHANGE IN WATER STORAGE	-1.175	-4264.874	-1.75
SOIL WATER AT START OF YEAR	392.769	1425750.375	
SOIL WATER AT END OF YEAR	391.594	1421485.500	
INTERCEPTION WATER AT START OF YEAR	0.000	0.000	
INTERCEPTION WATER AT END OF YEAR	0.000	0.000	
SNOW WATER AT START OF YEAR	0.000	0.000	0.00
SNOW WATER AT END OF YEAR	0.000	0.000	0.00
ANNUAL WATER BUDGET BALANCE	0.0000	0.127	0.00

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ANNUAL TOTALS FOR YEAR 9

	INCHES	CU. FEET	PERCENT
PRECIPITATION	67.42	244734.578	100.00
RUNOFF	0.148	536.978	0.22

POTENTIAL EVAPOTRANSPIRATION	54.852	199112.859	
ACTUAL EVAPOTRANSPIRATION	32.030	116269.867	47.51
DRAINAGE COLLECTED FROM LAYER 4	31.1890	113215.898	46.26
PERC./LEAKAGE THROUGH LAYER 5	3.750374	13613.856	5.56
AVG. HEAD ON TOP OF LAYER 5	0.0239		
DRAINAGE COLLECTED FROM LAYER 6	3.7495	13610.597	5.56
PERC./LEAKAGE THROUGH LAYER 7	0.000003	0.010	0.00
AVG. HEAD ON TOP OF LAYER 7	0.0029		
PERC./LEAKAGE THROUGH LAYER 8	0.000003	0.010	0.00
CHANGE IN WATER STORAGE	0.303	1101.363	0.45
SOIL WATER AT START OF YEAR	391.594	1421485.500	
SOIL WATER AT END OF YEAR	391.897	1422586.875	
INTERCEPTION WATER AT START OF YEAR	0.000	0.000	
INTERCEPTION WATER AT END OF YEAR	0.000	0.000	
SNOW WATER AT START OF YEAR	0.000	0.000	0.00
SNOW WATER AT END OF YEAR	0.000	0.000	0.00
ANNUAL WATER BUDGET BALANCE	0.0000	-0.118	0.00

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ANNUAL TOTALS FOR YEAR 10

	INCHES	CU. FEET	PERCENT
PRECIPITATION	71.32	258891.562	100.00
RUNOFF	0.445	1616.601	0.62
POTENTIAL EVAPOTRANSPIRATION	54.532	197951.062	
ACTUAL EVAPOTRANSPIRATION	30.647	111249.594	42.97
DRAINAGE COLLECTED FROM LAYER 4	33.7606	122550.914	47.34
PERC./LEAKAGE THROUGH LAYER 5	3.938928	14298.309	5.52
AVG. HEAD ON TOP OF LAYER 5	0.0259		
DRAINAGE COLLECTED FROM LAYER 6	3.9389	14298.054	5.52

PERC./LEAKAGE THROUGH LAYER 7	0.000003	0.010	0.00
AVG. HEAD ON TOP OF LAYER 7	0.0030		
PERC./LEAKAGE THROUGH LAYER 8	0.000003	0.010	0.00
CHANGE IN WATER STORAGE	2.528	9176.362	3.54
SOIL WATER AT START OF YEAR	391.897	1422586.875	
SOIL WATER AT END OF YEAR	394.425	1431763.375	
INTERCEPTION WATER AT START OF YEAR	0.000	0.000	
INTERCEPTION WATER AT END OF YEAR	0.000	0.000	
SNOW WATER AT START OF YEAR	0.000	0.000	0.00
SNOW WATER AT END OF YEAR	0.000	0.000	0.00
ANNUAL WATER BUDGET BALANCE	0.0000	0.042	0.00

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ANNUAL TOTALS FOR YEAR 11

	INCHES	CU. FEET	PERCENT
PRECIPITATION	65.26	236893.766	100.00
RUNOFF	0.173	626.438	0.26
POTENTIAL EVAPOTRANSPIRATION	54.803	198934.250	
ACTUAL EVAPOTRANSPIRATION	31.541	114495.031	48.33
DRAINAGE COLLECTED FROM LAYER 4	33.5626	121832.289	51.43
PERC./LEAKAGE THROUGH LAYER 5	3.942528	14311.379	6.04
AVG. HEAD ON TOP OF LAYER 5	0.0257		
DRAINAGE COLLECTED FROM LAYER 6	3.9422	14310.208	6.04
PERC./LEAKAGE THROUGH LAYER 7	0.000003	0.010	0.00
AVG. HEAD ON TOP OF LAYER 7	0.0030		
PERC./LEAKAGE THROUGH LAYER 8	0.000003	0.010	0.00
CHANGE IN WATER STORAGE	-3.959	-14370.116	-6.07
SOIL WATER AT START OF YEAR	394.425	1431763.375	
SOIL WATER AT END OF YEAR	390.466	1417393.125	
INTERCEPTION WATER AT START OF YEAR	0.000	0.000	

INTERCEPTION WATER AT END OF YEAR	0.000	0.000	
SNOW WATER AT START OF YEAR	0.000	0.000	0.00
SNOW WATER AT END OF YEAR	0.000	0.000	0.00
ANNUAL WATER BUDGET BALANCE	0.0000	-0.065	0.00

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ANNUAL TOTALS FOR YEAR 12

	INCHES	CU. FEET	PERCENT
PRECIPITATION	52.45	190393.484	100.00
RUNOFF	0.000	0.000	0.00
POTENTIAL EVAPOTRANSPIRATION	55.172	200276.109	
ACTUAL EVAPOTRANSPIRATION	29.849	108353.117	56.91
DRAINAGE COLLECTED FROM LAYER 4	28.6020	103825.383	54.53
PERC./LEAKAGE THROUGH LAYER 5	3.670013	13322.148	7.00
AVG. HEAD ON TOP OF LAYER 5	0.0219		
DRAINAGE COLLECTED FROM LAYER 6	3.6706	13324.401	7.00
PERC./LEAKAGE THROUGH LAYER 7	0.000003	0.010	0.00
AVG. HEAD ON TOP OF LAYER 7	0.0028		
PERC./LEAKAGE THROUGH LAYER 8	0.000003	0.010	0.00
CHANGE IN WATER STORAGE	-9.672	-35109.461	-18.44
SOIL WATER AT START OF YEAR	390.466	1417393.125	
SOIL WATER AT END OF YEAR	380.794	1382283.625	
INTERCEPTION WATER AT START OF YEAR	0.000	0.000	
INTERCEPTION WATER AT END OF YEAR	0.000	0.000	
SNOW WATER AT START OF YEAR	0.000	0.000	0.00
SNOW WATER AT END OF YEAR	0.000	0.000	0.00
ANNUAL WATER BUDGET BALANCE	0.0000	0.053	0.00

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ANNUAL TOTALS FOR YEAR 13

	INCHES	CU. FEET	PERCENT
PRECIPITATION	54.63	198306.891	100.00
RUNOFF	0.509	1848.962	0.93
POTENTIAL EVAPOTRANSPIRATION	55.320	200811.297	
ACTUAL EVAPOTRANSPIRATION	25.802	93660.211	47.23
DRAINAGE COLLECTED FROM LAYER 4	25.8398	93798.469	47.30
PERC./LEAKAGE THROUGH LAYER 5	3.465009	12577.981	6.34
AVG. HEAD ON TOP OF LAYER 5	0.0198		
DRAINAGE COLLECTED FROM LAYER 6	3.4658	12580.816	6.34
PERC./LEAKAGE THROUGH LAYER 7	0.000003	0.010	0.00
AVG. HEAD ON TOP OF LAYER 7	0.0027		
PERC./LEAKAGE THROUGH LAYER 8	0.000003	0.010	0.00
CHANGE IN WATER STORAGE	-0.987	-3581.368	-1.81
SOIL WATER AT START OF YEAR	380.794	1382283.625	
SOIL WATER AT END OF YEAR	379.808	1378702.375	
INTERCEPTION WATER AT START OF YEAR	0.000	0.000	
INTERCEPTION WATER AT END OF YEAR	0.000	0.000	
SNOW WATER AT START OF YEAR	0.000	0.000	0.00
SNOW WATER AT END OF YEAR	0.000	0.000	0.00
ANNUAL WATER BUDGET BALANCE	-0.0001	-0.200	0.00

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ANNUAL TOTALS FOR YEAR 14

	INCHES	CU. FEET	PERCENT
PRECIPITATION	59.92	217509.609	100.00
RUNOFF	0.609	2211.771	1.02
POTENTIAL EVAPOTRANSPIRATION	54.090	196346.109	

ACTUAL EVAPOTRANSPIRATION	27.448	99636.180	45.81
DRAINAGE COLLECTED FROM LAYER 4	26.3553	95669.844	43.98
PERC./LEAKAGE THROUGH LAYER 5	3.468497	12590.644	5.79
AVG. HEAD ON TOP OF LAYER 5	0.0203		
DRAINAGE COLLECTED FROM LAYER 6	3.4679	12588.431	5.79
PERC./LEAKAGE THROUGH LAYER 7	0.000003	0.010	0.00
AVG. HEAD ON TOP OF LAYER 7	0.0027		
PERC./LEAKAGE THROUGH LAYER 8	0.000003	0.010	0.00
CHANGE IN WATER STORAGE	2.039	7403.348	3.40
SOIL WATER AT START OF YEAR	379.808	1378702.375	
SOIL WATER AT END OF YEAR	381.847	1386105.625	
INTERCEPTION WATER AT START OF YEAR	0.000	0.000	
INTERCEPTION WATER AT END OF YEAR	0.000	0.000	
SNOW WATER AT START OF YEAR	0.000	0.000	0.00
SNOW WATER AT END OF YEAR	0.000	0.000	0.00
ANNUAL WATER BUDGET BALANCE	0.0000	0.029	0.00

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ANNUAL TOTALS FOR YEAR 15

	INCHES	CU. FEET	PERCENT
PRECIPITATION	56.04	203425.203	100.00
RUNOFF	0.197	716.388	0.35
POTENTIAL EVAPOTRANSPIRATION	56.640	205602.625	
ACTUAL EVAPOTRANSPIRATION	26.624	96644.305	47.51
DRAINAGE COLLECTED FROM LAYER 4	28.8684	104792.398	51.51
PERC./LEAKAGE THROUGH LAYER 5	3.704112	13445.927	6.61
AVG. HEAD ON TOP OF LAYER 5	0.0221		
DRAINAGE COLLECTED FROM LAYER 6	3.7048	13448.351	6.61
PERC./LEAKAGE THROUGH LAYER 7	0.000003	0.010	0.00

AVG. HEAD ON TOP OF LAYER 7	0.0028		
PERC./LEAKAGE THROUGH LAYER 8	0.000003	0.010	0.00
CHANGE IN WATER STORAGE	-3.354	-12176.363	-5.99
SOIL WATER AT START OF YEAR	381.847	1386105.625	
SOIL WATER AT END OF YEAR	378.493	1373929.375	
INTERCEPTION WATER AT START OF YEAR	0.000	0.000	
INTERCEPTION WATER AT END OF YEAR	0.000	0.000	
SNOW WATER AT START OF YEAR	0.000	0.000	0.00
SNOW WATER AT END OF YEAR	0.000	0.000	0.00
ANNUAL WATER BUDGET BALANCE	0.0000	0.120	0.00

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ANNUAL TOTALS FOR YEAR 16

	INCHES	CU. FEET	PERCENT
PRECIPITATION	50.58	183605.375	100.00
RUNOFF	0.057	205.602	0.11
POTENTIAL EVAPOTRANSPIRATION	55.167	200257.641	
ACTUAL EVAPOTRANSPIRATION	27.371	99356.062	54.11
DRAINAGE COLLECTED FROM LAYER 4	23.5557	85507.367	46.57
PERC./LEAKAGE THROUGH LAYER 5	3.295202	11961.582	6.51
AVG. HEAD ON TOP OF LAYER 5	0.0180		
DRAINAGE COLLECTED FROM LAYER 6	3.2937	11956.145	6.51
PERC./LEAKAGE THROUGH LAYER 7	0.000003	0.010	0.00
AVG. HEAD ON TOP OF LAYER 7	0.0025		
PERC./LEAKAGE THROUGH LAYER 8	0.000003	0.010	0.00
CHANGE IN WATER STORAGE	-3.697	-13419.745	-7.31
SOIL WATER AT START OF YEAR	378.493	1373929.375	
SOIL WATER AT END OF YEAR	374.796	1360509.625	
INTERCEPTION WATER AT START OF YEAR	0.000	0.000	
INTERCEPTION WATER AT END OF YEAR	0.000	0.000	

SNOW WATER AT START OF YEAR	0.000	0.000	0.00
SNOW WATER AT END OF YEAR	0.000	0.000	0.00
ANNUAL WATER BUDGET BALANCE	0.0000	-0.072	0.00

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ANNUAL TOTALS FOR YEAR 17

	INCHES	CU. FEET	PERCENT
PRECIPITATION	70.44	255697.203	100.00
RUNOFF	0.349	1265.196	0.49
POTENTIAL EVAPOTRANSPIRATION	55.309	200771.188	
ACTUAL EVAPOTRANSPIRATION	34.409	124905.398	48.85
DRAINAGE COLLECTED FROM LAYER 4	23.8383	86533.195	33.84
PERC./LEAKAGE THROUGH LAYER 5	3.236137	11747.179	4.59
AVG. HEAD ON TOP OF LAYER 5	0.0183		
DRAINAGE COLLECTED FROM LAYER 6	3.2380	11753.858	4.60
PERC./LEAKAGE THROUGH LAYER 7	0.000003	0.009	0.00
AVG. HEAD ON TOP OF LAYER 7	0.0025		
PERC./LEAKAGE THROUGH LAYER 8	0.000003	0.009	0.00
CHANGE IN WATER STORAGE	8.606	31239.625	12.22
SOIL WATER AT START OF YEAR	374.796	1360509.625	
SOIL WATER AT END OF YEAR	383.402	1391749.125	
INTERCEPTION WATER AT START OF YEAR	0.000	0.000	
INTERCEPTION WATER AT END OF YEAR	0.000	0.000	
SNOW WATER AT START OF YEAR	0.000	0.000	0.00
SNOW WATER AT END OF YEAR	0.000	0.000	0.00
ANNUAL WATER BUDGET BALANCE	0.0000	-0.074	0.00

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ANNUAL TOTALS FOR YEAR 18

	INCHES	CU. FEET	PERCENT
PRECIPITATION	57.83	209922.891	100.00
RUNOFF	0.534	1937.940	0.92
POTENTIAL EVAPOTRANSPIRATION	54.640	198342.125	
ACTUAL EVAPOTRANSPIRATION	28.751	104364.344	49.72
DRAINAGE COLLECTED FROM LAYER 4	28.1514	102189.523	48.68
PERC./LEAKAGE THROUGH LAYER 5	3.627437	13167.595	6.27
AVG. HEAD ON TOP OF LAYER 5	0.0216		
DRAINAGE COLLECTED FROM LAYER 6	3.6263	13163.464	6.27
PERC./LEAKAGE THROUGH LAYER 7	0.000003	0.010	0.00
AVG. HEAD ON TOP OF LAYER 7	0.0028		
PERC./LEAKAGE THROUGH LAYER 8	0.000003	0.010	0.00
CHANGE IN WATER STORAGE	-3.232	-11732.473	-5.59
SOIL WATER AT START OF YEAR	383.402	1391749.125	
SOIL WATER AT END OF YEAR	380.170	1380016.625	
INTERCEPTION WATER AT START OF YEAR	0.000	0.000	
INTERCEPTION WATER AT END OF YEAR	0.000	0.000	
SNOW WATER AT START OF YEAR	0.000	0.000	0.00
SNOW WATER AT END OF YEAR	0.000	0.000	0.00
ANNUAL WATER BUDGET BALANCE	0.0000	0.084	0.00

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ANNUAL TOTALS FOR YEAR 19

	INCHES	CU. FEET	PERCENT
PRECIPITATION	75.35	273520.531	100.00
RUNOFF	0.226	821.544	0.30
POTENTIAL EVAPOTRANSPIRATION	52.829	191769.641	
ACTUAL EVAPOTRANSPIRATION	35.042	127200.992	46.51

DRAINAGE COLLECTED FROM LAYER 4	26.6256	96650.969	35.34
PERC./LEAKAGE THROUGH LAYER 5	3.453948	12537.832	4.58
AVG. HEAD ON TOP OF LAYER 5	0.0204		
DRAINAGE COLLECTED FROM LAYER 6	3.4534	12535.765	4.58
PERC./LEAKAGE THROUGH LAYER 7	0.000003	0.010	0.00
AVG. HEAD ON TOP OF LAYER 7	0.0026		
PERC./LEAKAGE THROUGH LAYER 8	0.000003	0.010	0.00
CHANGE IN WATER STORAGE	10.003	36311.410	13.28
SOIL WATER AT START OF YEAR	380.170	1380016.625	
SOIL WATER AT END OF YEAR	390.173	1416328.125	
INTERCEPTION WATER AT START OF YEAR	0.000	0.000	
INTERCEPTION WATER AT END OF YEAR	0.000	0.000	
SNOW WATER AT START OF YEAR	0.000	0.000	0.00
SNOW WATER AT END OF YEAR	0.000	0.000	0.00
ANNUAL WATER BUDGET BALANCE	0.0000	-0.132	0.00

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ANNUAL TOTALS FOR YEAR 20

	INCHES	CU. FEET	PERCENT
PRECIPITATION	59.49	215948.734	100.00
RUNOFF	0.050	182.026	0.08
POTENTIAL EVAPOTRANSPIRATION	57.884	210118.578	
ACTUAL EVAPOTRANSPIRATION	30.566	110952.992	51.38
DRAINAGE COLLECTED FROM LAYER 4	33.0740	120058.500	55.60
PERC./LEAKAGE THROUGH LAYER 5	3.943154	14313.648	6.63
AVG. HEAD ON TOP OF LAYER 5	0.0253		
DRAINAGE COLLECTED FROM LAYER 6	3.9435	14314.770	6.63
PERC./LEAKAGE THROUGH LAYER 7	0.000003	0.010	0.00
AVG. HEAD ON TOP OF LAYER 7	0.0030		

PERC./LEAKAGE THROUGH LAYER 8	0.000003	0.010	0.00
CHANGE IN WATER STORAGE	-8.143	-29559.553	-13.69
SOIL WATER AT START OF YEAR	390.173	1416328.125	
SOIL WATER AT END OF YEAR	382.030	1386768.625	
INTERCEPTION WATER AT START OF YEAR	0.000	0.000	
INTERCEPTION WATER AT END OF YEAR	0.000	0.000	
SNOW WATER AT START OF YEAR	0.000	0.000	0.00
SNOW WATER AT END OF YEAR	0.000	0.000	0.00
ANNUAL WATER BUDGET BALANCE	0.0000	-0.016	0.00

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ANNUAL TOTALS FOR YEAR 21

	INCHES	CU. FEET	PERCENT
PRECIPITATION	65.64	238273.188	100.00
RUNOFF	0.000	0.012	0.00
POTENTIAL EVAPOTRANSPIRATION	55.173	200277.875	
ACTUAL EVAPOTRANSPIRATION	32.560	118191.500	49.60
DRAINAGE COLLECTED FROM LAYER 4	28.2545	102563.977	43.04
PERC./LEAKAGE THROUGH LAYER 5	3.628052	13169.827	5.53
AVG. HEAD ON TOP OF LAYER 5	0.0217		
DRAINAGE COLLECTED FROM LAYER 6	3.6294	13174.633	5.53
PERC./LEAKAGE THROUGH LAYER 7	0.000003	0.010	0.00
AVG. HEAD ON TOP OF LAYER 7	0.0028		
PERC./LEAKAGE THROUGH LAYER 8	0.000003	0.010	0.00
CHANGE IN WATER STORAGE	1.196	4342.862	1.82
SOIL WATER AT START OF YEAR	382.030	1386768.625	
SOIL WATER AT END OF YEAR	383.226	1391111.375	
INTERCEPTION WATER AT START OF YEAR	0.000	0.000	
INTERCEPTION WATER AT END OF YEAR	0.000	0.000	
SNOW WATER AT START OF YEAR	0.000	0.000	0.00

SNOW WATER AT END OF YEAR	0.000	0.000	0.00
ANNUAL WATER BUDGET BALANCE	0.0001	0.208	0.00

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ANNUAL TOTALS FOR YEAR 22

	INCHES	CU. FEET	PERCENT
PRECIPITATION	71.54	259690.203	100.00
RUNOFF	0.099	357.898	0.14
POTENTIAL EVAPOTRANSPIRATION	53.971	195913.875	
ACTUAL EVAPOTRANSPIRATION	31.719	115138.352	44.34
DRAINAGE COLLECTED FROM LAYER 4	25.0507	90934.156	35.02
PERC./LEAKAGE THROUGH LAYER 5	3.334351	12103.694	4.66
AVG. HEAD ON TOP OF LAYER 5	0.0192		
DRAINAGE COLLECTED FROM LAYER 6	3.3341	12102.925	4.66
PERC./LEAKAGE THROUGH LAYER 7	0.000003	0.009	0.00
AVG. HEAD ON TOP OF LAYER 7	0.0026		
PERC./LEAKAGE THROUGH LAYER 8	0.000003	0.009	0.00
CHANGE IN WATER STORAGE	11.338	41156.988	15.85
SOIL WATER AT START OF YEAR	383.226	1391111.375	
SOIL WATER AT END OF YEAR	394.564	1432268.375	
INTERCEPTION WATER AT START OF YEAR	0.000	0.000	
INTERCEPTION WATER AT END OF YEAR	0.000	0.000	
SNOW WATER AT START OF YEAR	0.000	0.000	0.00
SNOW WATER AT END OF YEAR	0.000	0.000	0.00
ANNUAL WATER BUDGET BALANCE	0.0000	-0.119	0.00

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ANNUAL TOTALS FOR YEAR 23

existing landfill

	INCHES	CU. FEET	PERCENT
PRECIPITATION	64.39	233735.734	100.00
RUNOFF	0.035	126.891	0.05
POTENTIAL EVAPOTRANSPIRATION	54.842	199077.625	
ACTUAL EVAPOTRANSPIRATION	31.863	115661.844	49.48
DRAINAGE COLLECTED FROM LAYER 4	34.2373	124281.258	53.17
PERC./LEAKAGE THROUGH LAYER 5	3.944024	14316.806	6.13
AVG. HEAD ON TOP OF LAYER 5	0.0262		
DRAINAGE COLLECTED FROM LAYER 6	3.9424	14311.063	6.12
PERC./LEAKAGE THROUGH LAYER 7	0.000003	0.010	0.00
AVG. HEAD ON TOP OF LAYER 7	0.0030		
PERC./LEAKAGE THROUGH LAYER 8	0.000003	0.010	0.00
CHANGE IN WATER STORAGE	-5.687	-20645.293	-8.83
SOIL WATER AT START OF YEAR	394.564	1432268.375	
SOIL WATER AT END OF YEAR	388.877	1411623.125	
INTERCEPTION WATER AT START OF YEAR	0.000	0.000	
INTERCEPTION WATER AT END OF YEAR	0.000	0.000	
SNOW WATER AT START OF YEAR	0.000	0.000	0.00
SNOW WATER AT END OF YEAR	0.000	0.000	0.00
ANNUAL WATER BUDGET BALANCE	0.0000	-0.076	0.00

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ANNUAL TOTALS FOR YEAR 24

	INCHES	CU. FEET	PERCENT
PRECIPITATION	80.16	290980.875	100.00
RUNOFF	0.509	1848.232	0.64
POTENTIAL EVAPOTRANSPIRATION	55.334	200861.797	
ACTUAL EVAPOTRANSPIRATION	34.714	126010.992	43.31
DRAINAGE COLLECTED FROM LAYER 4	32.3954	117595.219	40.41

PERC./LEAKAGE THROUGH LAYER 5	3.846412	13962.477	4.80
AVG. HEAD ON TOP OF LAYER 5	0.0248		
DRAINAGE COLLECTED FROM LAYER 6	3.8465	13962.718	4.80
PERC./LEAKAGE THROUGH LAYER 7	0.000003	0.010	0.00
AVG. HEAD ON TOP OF LAYER 7	0.0029		
PERC./LEAKAGE THROUGH LAYER 8	0.000003	0.010	0.00
CHANGE IN WATER STORAGE	8.695	31563.430	10.85
SOIL WATER AT START OF YEAR	388.877	1411623.125	
SOIL WATER AT END OF YEAR	397.572	1443186.500	
INTERCEPTION WATER AT START OF YEAR	0.000	0.000	
INTERCEPTION WATER AT END OF YEAR	0.000	0.000	
SNOW WATER AT START OF YEAR	0.000	0.000	0.00
SNOW WATER AT END OF YEAR	0.000	0.000	0.00
ANNUAL WATER BUDGET BALANCE	0.0001	0.259	0.00

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ANNUAL TOTALS FOR YEAR 25

	INCHES	CU. FEET	PERCENT
PRECIPITATION	64.20	233046.047	100.00
RUNOFF	0.086	313.867	0.13
POTENTIAL EVAPOTRANSPIRATION	53.792	195263.141	
ACTUAL EVAPOTRANSPIRATION	34.795	126307.461	54.20
DRAINAGE COLLECTED FROM LAYER 4	35.3610	128360.336	55.08
PERC./LEAKAGE THROUGH LAYER 5	4.099065	14879.605	6.38
AVG. HEAD ON TOP OF LAYER 5	0.0272		
DRAINAGE COLLECTED FROM LAYER 6	4.0992	14880.195	6.39
PERC./LEAKAGE THROUGH LAYER 7	0.000003	0.010	0.00
AVG. HEAD ON TOP OF LAYER 7	0.0031		
PERC./LEAKAGE THROUGH LAYER 8	0.000003	0.010	0.00

CHANGE IN WATER STORAGE	-10.142	-36815.785	-15.80
SOIL WATER AT START OF YEAR	397.572	1443186.500	
SOIL WATER AT END OF YEAR	387.430	1406370.875	
INTERCEPTION WATER AT START OF YEAR	0.000	0.000	
INTERCEPTION WATER AT END OF YEAR	0.000	0.000	
SNOW WATER AT START OF YEAR	0.000	0.000	0.00
SNOW WATER AT END OF YEAR	0.000	0.000	0.00
ANNUAL WATER BUDGET BALANCE	0.0000	-0.040	0.00

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ANNUAL TOTALS FOR YEAR 26

	INCHES	CU. FEET	PERCENT
PRECIPITATION	67.32	244371.625	100.00
RUNOFF	0.261	945.744	0.39
POTENTIAL EVAPOTRANSPIRATION	55.491	201433.438	
ACTUAL EVAPOTRANSPIRATION	30.994	112509.555	46.04
DRAINAGE COLLECTED FROM LAYER 4	27.8468	101083.711	41.36
PERC./LEAKAGE THROUGH LAYER 5	3.560481	12924.547	5.29
AVG. HEAD ON TOP OF LAYER 5	0.0214		
DRAINAGE COLLECTED FROM LAYER 6	3.5605	12924.661	5.29
PERC./LEAKAGE THROUGH LAYER 7	0.000003	0.010	0.00
AVG. HEAD ON TOP OF LAYER 7	0.0027		
PERC./LEAKAGE THROUGH LAYER 8	0.000003	0.010	0.00
CHANGE IN WATER STORAGE	4.658	16907.947	6.92
SOIL WATER AT START OF YEAR	387.430	1406370.875	
SOIL WATER AT END OF YEAR	392.088	1423278.625	
INTERCEPTION WATER AT START OF YEAR	0.000	0.000	
INTERCEPTION WATER AT END OF YEAR	0.000	0.000	
SNOW WATER AT START OF YEAR	0.000	0.000	0.00
SNOW WATER AT END OF YEAR	0.000	0.000	0.00

ANNUAL WATER BUDGET BALANCE	0.0000	-0.008	0.00

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ANNUAL TOTALS FOR YEAR 27

	INCHES	CU. FEET	PERCENT
PRECIPITATION	61.15	221974.500	100.00
RUNOFF	0.702	2548.619	1.15
POTENTIAL EVAPOTRANSPIRATION	55.851	202738.734	
ACTUAL EVAPOTRANSPIRATION	31.162	113119.844	50.96
DRAINAGE COLLECTED FROM LAYER 4	30.9003	112167.977	50.53
PERC./LEAKAGE THROUGH LAYER 5	3.815775	13851.265	6.24
AVG. HEAD ON TOP OF LAYER 5	0.0237		
DRAINAGE COLLECTED FROM LAYER 6	3.8164	13853.642	6.24
PERC./LEAKAGE THROUGH LAYER 7	0.000003	0.010	0.00
AVG. HEAD ON TOP OF LAYER 7	0.0029		
PERC./LEAKAGE THROUGH LAYER 8	0.000003	0.010	0.00
CHANGE IN WATER STORAGE	-5.431	-19715.637	-8.88
SOIL WATER AT START OF YEAR	392.088	1423278.625	
SOIL WATER AT END OF YEAR	386.656	1403563.000	
INTERCEPTION WATER AT START OF YEAR	0.000	0.000	
INTERCEPTION WATER AT END OF YEAR	0.000	0.000	
SNOW WATER AT START OF YEAR	0.000	0.000	0.00
SNOW WATER AT END OF YEAR	0.000	0.000	0.00
ANNUAL WATER BUDGET BALANCE	0.0000	0.051	0.00

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ANNUAL TOTALS FOR YEAR 28

existing landfill

	INCHES	CU. FEET	PERCENT
PRECIPITATION	65.06	236167.797	100.00
RUNOFF	0.512	1859.633	0.79
POTENTIAL EVAPOTRANSPIRATION	53.479	194130.438	
ACTUAL EVAPOTRANSPIRATION	32.283	117188.344	49.62
DRAINAGE COLLECTED FROM LAYER 4	26.7316	97035.742	41.09
PERC./LEAKAGE THROUGH LAYER 5	3.497284	12695.141	5.38
AVG. HEAD ON TOP OF LAYER 5	0.0205		
DRAINAGE COLLECTED FROM LAYER 6	3.4987	12700.169	5.38
PERC./LEAKAGE THROUGH LAYER 7	0.000003	0.010	0.00
AVG. HEAD ON TOP OF LAYER 7	0.0027		
PERC./LEAKAGE THROUGH LAYER 8	0.000003	0.010	0.00
CHANGE IN WATER STORAGE	2.034	7383.961	3.13
SOIL WATER AT START OF YEAR	386.656	1403563.000	
SOIL WATER AT END OF YEAR	388.691	1410947.000	
INTERCEPTION WATER AT START OF YEAR	0.000	0.000	
INTERCEPTION WATER AT END OF YEAR	0.000	0.000	
SNOW WATER AT START OF YEAR	0.000	0.000	0.00
SNOW WATER AT END OF YEAR	0.000	0.000	0.00
ANNUAL WATER BUDGET BALANCE	0.0000	-0.079	0.00

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ANNUAL TOTALS FOR YEAR 29

	INCHES	CU. FEET	PERCENT
PRECIPITATION	62.48	226802.375	100.00
RUNOFF	0.231	837.722	0.37
POTENTIAL EVAPOTRANSPIRATION	54.992	199620.500	
ACTUAL EVAPOTRANSPIRATION	28.018	101704.242	44.84
DRAINAGE COLLECTED FROM LAYER 4	29.4592	106936.812	47.15
PERC./LEAKAGE THROUGH LAYER 5	3.657419	13276.430	5.85

AVG. HEAD ON TOP OF LAYER 5	0.0226		
DRAINAGE COLLECTED FROM LAYER 6	3.6562	13271.870	5.85
PERC./LEAKAGE THROUGH LAYER 7	0.000003	0.010	0.00
AVG. HEAD ON TOP OF LAYER 7	0.0028		
PERC./LEAKAGE THROUGH LAYER 8	0.000003	0.010	0.00
CHANGE IN WATER STORAGE	1.116	4051.735	1.79
SOIL WATER AT START OF YEAR	388.691	1410947.000	
SOIL WATER AT END OF YEAR	389.807	1414998.875	
INTERCEPTION WATER AT START OF YEAR	0.000	0.000	
INTERCEPTION WATER AT END OF YEAR	0.000	0.000	
SNOW WATER AT START OF YEAR	0.000	0.000	0.00
SNOW WATER AT END OF YEAR	0.000	0.000	0.00
ANNUAL WATER BUDGET BALANCE	0.0000	-0.008	0.00

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ANNUAL TOTALS FOR YEAR 30

	INCHES	CU. FEET	PERCENT
PRECIPITATION	61.55	223426.484	100.00
RUNOFF	0.124	451.608	0.20
POTENTIAL EVAPOTRANSPIRATION	53.835	195422.859	
ACTUAL EVAPOTRANSPIRATION	30.337	110121.773	49.29
DRAINAGE COLLECTED FROM LAYER 4	29.2752	106269.031	47.56
PERC./LEAKAGE THROUGH LAYER 5	3.658418	13280.058	5.94
AVG. HEAD ON TOP OF LAYER 5	0.0225		
DRAINAGE COLLECTED FROM LAYER 6	3.6576	13277.142	5.94
PERC./LEAKAGE THROUGH LAYER 7	0.000003	0.010	0.00
AVG. HEAD ON TOP OF LAYER 7	0.0028		
PERC./LEAKAGE THROUGH LAYER 8	0.000003	0.010	0.00
CHANGE IN WATER STORAGE	-1.844	-6692.812	-3.00

existing landfill

SOIL WATER AT START OF YEAR	389.807	1414998.875	
SOIL WATER AT END OF YEAR	387.963	1408306.000	
INTERCEPTION WATER AT START OF YEAR	0.000	0.000	
INTERCEPTION WATER AT END OF YEAR	0.000	0.000	
SNOW WATER AT START OF YEAR	0.000	0.000	0.00
SNOW WATER AT END OF YEAR	0.000	0.000	0.00
ANNUAL WATER BUDGET BALANCE	-0.0001	-0.260	0.00

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FINAL WATER STORAGE AT END OF YEAR 30

LAYER	(INCHES)	(VOL/VOL)
1	2.3699	0.1975
2	376.0079	0.3072
3	9.1710	0.3821
4	0.0317	0.1068
5	0.0000	0.0000
6	0.0061	0.0206
7	0.0000	0.0000
8	0.1867	0.7470
TOTAL WATER IN LAYERS	387.773	
SNOW WATER	0.000	
INTERCEPTION WATER	0.000	
TOTAL FINAL WATER	387.773	

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PEAK DAILY VALUES FOR YEARS 1 THROUGH 30

	(INCHES)	(CU. FT.)
PRECIPITATION	4.69	17024.701

RUNOFF	0.462	1677.9515
DRAINAGE COLLECTED FROM LAYER 4	0.25558	927.73846
PERCOLATION/LEAKAGE THROUGH LAYER 5	0.019329	70.16337
AVERAGE HEAD ON TOP OF LAYER 5	0.072	
MAXIMUM HEAD ON TOP OF LAYER 5	0.141	
LOCATION OF MAXIMUM HEAD IN LAYER 4 (DISTANCE FROM DRAIN)	2.6 FEET	
DRAINAGE COLLECTED FROM LAYER 6	0.01858	67.45229
PERCOLATION/LEAKAGE THROUGH LAYER 7	0.000000	0.00003
AVERAGE HEAD ON TOP OF LAYER 7	0.005	
MAXIMUM HEAD ON TOP OF LAYER 7	0.012	
LOCATION OF MAXIMUM HEAD IN LAYER 6 (DISTANCE FROM DRAIN)	0.0 FEET	
PERCOLATION/LEAKAGE THROUGH LAYER 8	0.000000	0.00003
SNOW WATER	0.91	3304.4434
MAXIMUM VEG. SOIL WATER (VOL/VOL)	0.3923	
MINIMUM VEG. SOIL WATER (VOL/VOL)	0.1040	

*** Maximum heads are computed using McEnroe's equations. ***

Reference: Maximum Saturated Depth over Landfill Liner
by Bruce M. McEnroe, University of Kansas
ASCE Journal of Environmental Engineering
Vol. 119, No. 2, March 1993, pp. 262-270.

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AVERAGE MONTHLY VALUES IN INCHES FOR YEARS 1 THROUGH 30

	JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
PRECIPITATION						
TOTALS	4.34 9.00	4.81 7.43	5.62 7.41	4.25 3.08	4.20 2.95	6.59 5.00
STD. DEVIATIONS	2.11 3.04	2.39 2.65	2.68 3.03	3.05 2.35	3.00 2.17	2.48 2.74
RUNOFF						

TOTALS	0.016	0.008	0.007	0.017	0.015	0.027
	0.030	0.030	0.074	0.014	0.016	0.010
STD. DEVIATIONS	0.059	0.027	0.025	0.041	0.054	0.093
	0.089	0.083	0.110	0.069	0.046	0.030

POTENTIAL EVAPOTRANSPIRATION

TOTALS	2.417	2.706	4.163	5.384	6.702	7.082
	6.587	6.153	5.024	3.848	2.634	2.047
STD. DEVIATIONS	0.151	0.180	0.264	0.294	0.292	0.328
	0.413	0.311	0.324	0.273	0.172	0.182

ACTUAL EVAPOTRANSPIRATION

TOTALS	1.926	2.166	2.744	2.082	2.387	3.532
	4.696	3.859	3.246	1.627	1.300	1.684
STD. DEVIATIONS	0.387	0.409	0.841	1.166	1.105	1.055
	0.891	0.977	0.844	0.857	0.686	0.369

LATERAL DRAINAGE COLLECTED FROM LAYER 4

TOTALS	2.1970	2.1549	2.4356	2.4074	2.7386	2.5397
	1.9480	1.9637	2.1566	2.7756	3.0362	2.8298
STD. DEVIATIONS	0.8466	0.5289	0.8234	0.6675	0.7029	0.7885
	0.6038	0.7976	0.6560	0.8073	0.6133	0.7531

PERCOLATION/LEAKAGE THROUGH LAYER 5

TOTALS	0.2870	0.2761	0.3066	0.3027	0.3321	0.3146
	0.2672	0.2667	0.2805	0.3289	0.3506	0.3406
STD. DEVIATIONS	0.0711	0.0416	0.0681	0.0529	0.0573	0.0684
	0.0523	0.0642	0.0540	0.0615	0.0443	0.0546

LATERAL DRAINAGE COLLECTED FROM LAYER 6

TOTALS	0.2873	0.2762	0.3063	0.3025	0.3323	0.3148
	0.2673	0.2667	0.2806	0.3280	0.3507	0.3408
STD. DEVIATIONS	0.0710	0.0417	0.0684	0.0528	0.0577	0.0683
	0.0524	0.0639	0.0540	0.0616	0.0440	0.0544

PERCOLATION/LEAKAGE THROUGH LAYER 7

TOTALS	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
STD. DEVIATIONS	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000

PERCOLATION/LEAKAGE THROUGH LAYER 8

TOTALS	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
STD. DEVIATIONS	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000

AVERAGES OF MONTHLY AVERAGED DAILY HEADS (INCHES)

DAILY AVERAGE HEAD ON TOP OF LAYER 5

AVERAGES	0.0198	0.0214	0.0220	0.0225	0.0247	0.0237
	0.0176	0.0177	0.0201	0.0251	0.0283	0.0256
STD. DEVIATIONS	0.0076	0.0053	0.0074	0.0062	0.0063	0.0074
	0.0055	0.0072	0.0061	0.0073	0.0057	0.0068

DAILY AVERAGE HEAD ON TOP OF LAYER 7

AVERAGES	0.0026	0.0027	0.0028	0.0028	0.0030	0.0029
	0.0024	0.0024	0.0026	0.0030	0.0033	0.0031
STD. DEVIATIONS	0.0006	0.0004	0.0006	0.0005	0.0005	0.0006
	0.0005	0.0006	0.0005	0.0006	0.0004	0.0005

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AVERAGE ANNUAL TOTALS & (STD. DEVIATIONS) FOR YEARS 1 THROUGH 30

	INCHES		CU. FEET	PERCENT
PRECIPITATION	64.69	(7.987)	234821.1	100.00
RUNOFF	0.264	(0.2100)	959.60	0.409
POTENTIAL EVAPOTRANSPIRATION	54.747	(0.9852)	198732.92	
ACTUAL EVAPOTRANSPIRATION	31.251	(2.7947)	113439.37	48.309
LATERAL DRAINAGE COLLECTED FROM LAYER 4	29.18303	(3.69786)	105934.406	45.11282
PERCOLATION/LEAKAGE THROUGH LAYER 5	3.65364	(0.26308)	13262.714	5.64801
AVERAGE HEAD ON TOP OF LAYER 5	0.022	(0.003)		
LATERAL DRAINAGE COLLECTED FROM LAYER 6	3.65358	(0.26300)	13262.499	5.64792
PERCOLATION/LEAKAGE THROUGH LAYER 7	0.00000	(0.00000)	0.010	0.00000
AVERAGE HEAD ON TOP OF LAYER 7	0.003	(0.000)		
PERCOLATION/LEAKAGE THROUGH LAYER 8	0.00000	(0.00000)	0.010	0.00000
CHANGE IN WATER STORAGE	0.338	(6.3370)	1225.22	0.522

existing landfill
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 **               HYDROLOGIC EVALUATION OF LANDFILL PERFORMANCE               **
 **                                                                          **
 **               HELP Version 3.95 D          (10 August 2012)             **
 **                                developed at                              **
 **        Institute of Soil Science, University of Hamburg, Germany         **
 **                                  based on                                **
 **               US HELP MODEL VERSION 3.07  (1 NOVEMBER 1997)              **
 **                   DEVELOPED BY ENVIRONMENTAL LABORATORY                  **
 **                     USAE WATERWAYS EXPERIMENT STATION                    **
 **                FOR USEPA RISK REDUCTION ENGINEERING LABORATORY           **
 **                                                                          **
 **                                                                          **
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 TIME:  15.43     DATE:   6.02.2019


 PRECIPITATION DATA FILE:      C:\HDR_Projects\steelfield\Engineering Design\HELP
Model\Final Stage\precipitation.d4
 TEMPERATURE DATA FILE:        C:\HDR_Projects\steelfield\Engineering Design\HELP
Model\Final Stage\temperature.d7
 SOLAR RADIATION DATA FILE:    C:\HDR_Projects\steelfield\Engineering Design\HELP
Model\Final Stage\solar radiation.d13
 EVAPOTRANSPIRATION DATA F. 1: C:\HDR_Projects\steelfield\Engineering Design\HELP
Model\Final Stage\evapotranspiration.d11
 SOIL AND DESIGN DATA FILE  1: C:\HDR_Projects\steelfield\Engineering Design\HELP
Model\Final Stage\Soil and Design-Final Cover.d10
 OUTPUT DATA FILE:             C:\HDR_Projects\steelfield\Engineering Design\HELP
Model\Final Stage\Final Cover.out
 YEARLY OUTPUT DATA FILE:      C:\HDR_Projects\steelfield\Engineering Design\HELP
Model\Final Stage\Final Cover.YR

 COLUMNS OF YEARLY OUTPUT DATA FILE:

      1    DATE OF ULTIMO (yyyy1231, years 2101 to 2200 from weather generator)
      2    PRECIPITATION (INCH)
      3    RUNOFF (INCH)
      4    POTENTIAL EVAPOTRANSPIRATION (INCH)
      5    ACTUAL EVAPOTRANSPIRATION (INCH)
      6    DRAIN #1: LATERAL DRAINAGE FROM LAYER  3 (INCH)
      7    LEAK  #1: PERCOLATION/LEAKAGE THROUGH LAYER  4 (INCH)
      8    DRAIN #2: LATERAL DRAINAGE FROM LAYER  8 (INCH)
      9    LEAK  #2: PERCOLATION/LEAKAGE THROUGH LAYER  9 (INCH)
     10    DRAIN #3: LATERAL DRAINAGE FROM LAYER 10 (INCH)
     11    LEAK  #3: PERCOLATION/LEAKAGE THROUGH LAYER 11 (INCH)
     12    LEAK  #4: PERCOLATION/LEAKAGE THROUGH LAYER 12 (INCH)
     13    CHANGE IN TOTAL WATER STORAGE (INCH)
     14    CHANGE IN SOIL WATER STORAGE (INCH)
     15    CHANGE IN INTERCEPTION WATER STORAGE (INCH)
     16    CHANGE IN SNOW WATER STORAGE (INCH)
     17    ANNUAL WATER BUDGET BALANCE (INCH)


 ******************************************************************************

      TITLE:  Scenario 4:Final Cover-Overall Landfill
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WEATHER DATA SOURCES

NOTE: PRECIPITATION DATA WAS SYNTHETICALLY GENERATED USING
COEFFICIENTS FOR TALLAHASSEE FLORIDA

NORMAL MEAN MONTHLY PRECIPITATION (INCHES)

JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
4.66	5.00	5.60	4.13	5.16	6.55
8.75	7.30	6.45	3.10	3.31	4.58

NOTE: TEMPERATURE DATA WAS SYNTHETICALLY GENERATED USING
COEFFICIENTS FOR TALLAHASSEE FLORIDA

NORMAL MEAN MONTHLY TEMPERATURE (DEGREES FAHRENHEIT)

JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
51.60	53.60	60.20	67.10	74.00	79.50
81.20	81.10	78.30	68.40	58.80	53.00

NOTE: SOLAR RADIATION DATA WAS SYNTHETICALLY GENERATED USING
COEFFICIENTS FOR TALLAHASSEE FLORIDA
AND STATION LATITUDE = 30.23 DEGREES

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LAYER DATA 1

VALID FOR 30 YEARS

NOTE: INITIAL MOISTURE CONTENT OF THE LAYERS AND SNOW WATER WERE
COMPUTED AS NEARLY STEADY-STATE VALUES BY THE PROGRAM.

LAYER 1

TYPE 1 - VERTICAL PERCOLATION LAYER
MATERIAL TEXTURE NUMBER 7

THICKNESS	=	6.00	INCHES
POROSITY	=	0.4730	VOL/VOL
FIELD CAPACITY	=	0.2220	VOL/VOL
WILTING POINT	=	0.1040	VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.3170	VOL/VOL
EFFECTIVE SAT. HYD. CONDUCT.	=	0.5200E-03	CM/SEC

Final Cover
NOTE: SATURATED HYDRAULIC CONDUCTIVITY IS MULTIPLIED BY 4.63
FOR ROOT CHANNELS IN TOP HALF OF EVAPORATIVE ZONE.

LAYER 2

TYPE 1 - VERTICAL PERCOLATION LAYER
MATERIAL TEXTURE NUMBER 10

THICKNESS	=	18.00	INCHES
POROSITY	=	0.3980	VOL/VOL
FIELD CAPACITY	=	0.2440	VOL/VOL
WILTING POINT	=	0.1360	VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.3152	VOL/VOL
EFFECTIVE SAT. HYD. CONDUCT.	=	0.1200E-03	CM/SEC

LAYER 3

TYPE 2 - LATERAL DRAINAGE LAYER
MATERIAL TEXTURE NUMBER 0

THICKNESS	=	0.28	INCHES
POROSITY	=	0.8500	VOL/VOL
FIELD CAPACITY	=	0.0100	VOL/VOL
WILTING POINT	=	0.0050	VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.0240	VOL/VOL
EFFECTIVE SAT. HYD. CONDUCT.	=	8.350	CM/SEC
SLOPE	=	33.00	PERCENT
DRAINAGE LENGTH	=	127.0	FEET

LAYER 4

TYPE 4 - FLEXIBLE MEMBRANE LINER
MATERIAL TEXTURE NUMBER 36

THICKNESS	=	0.04	INCHES
EFFECTIVE SAT. HYD. CONDUCT.	=	0.4000E-12	CM/SEC
FML PINHOLE DENSITY	=	2.00	HOLES/ACRE
FML INSTALLATION DEFECTS	=	2.00	HOLES/ACRE
FML PLACEMENT QUALITY	=	3 - GOOD	

LAYER 5

TYPE 1 - VERTICAL PERCOLATION LAYER
MATERIAL TEXTURE NUMBER 7

THICKNESS	=	12.00	INCHES
POROSITY	=	0.4730	VOL/VOL
FIELD CAPACITY	=	0.2220	VOL/VOL
WILTING POINT	=	0.1040	VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.2220	VOL/VOL
EFFECTIVE SAT. HYD. CONDUCT.	=	0.5200E-03	CM/SEC

LAYER 6

Final Cover

TYPE 1 - VERTICAL PERCOLATION LAYER
MATERIAL TEXTURE NUMBER 18

THICKNESS	=	984.00	INCHES
POROSITY	=	0.6710	VOL/VOL
FIELD CAPACITY	=	0.2920	VOL/VOL
WILTING POINT	=	0.0770	VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.2920	VOL/VOL
EFFECTIVE SAT. HYD. CONDUCT.	=	0.1000E-02	CM/SEC

LAYER 7

TYPE 1 - VERTICAL PERCOLATION LAYER
MATERIAL TEXTURE NUMBER 5

THICKNESS	=	24.00	INCHES
POROSITY	=	0.4570	VOL/VOL
FIELD CAPACITY	=	0.1310	VOL/VOL
WILTING POINT	=	0.0580	VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.1310	VOL/VOL
EFFECTIVE SAT. HYD. CONDUCT.	=	0.1000E-02	CM/SEC

LAYER 8

TYPE 2 - LATERAL DRAINAGE LAYER
MATERIAL TEXTURE NUMBER 0

THICKNESS	=	0.30	INCHES
POROSITY	=	0.8500	VOL/VOL
FIELD CAPACITY	=	0.0100	VOL/VOL
WILTING POINT	=	0.0050	VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.0100	VOL/VOL
EFFECTIVE SAT. HYD. CONDUCT.	=	5.200	CM/SEC
SLOPE	=	2.00	PERCENT
DRAINAGE LENGTH	=	165.0	FEET

LAYER 9

TYPE 4 - FLEXIBLE MEMBRANE LINER
MATERIAL TEXTURE NUMBER 35

THICKNESS	=	0.06	INCHES
EFFECTIVE SAT. HYD. CONDUCT.	=	0.2000E-12	CM/SEC
FML PINHOLE DENSITY	=	2.00	HOLES/ACRE
FML INSTALLATION DEFECTS	=	2.00	HOLES/ACRE
FML PLACEMENT QUALITY	=	3 - GOOD	

LAYER 10

TYPE 2 - LATERAL DRAINAGE LAYER
MATERIAL TEXTURE NUMBER 0

THICKNESS	=	0.30	INCHES
POROSITY	=	0.8500	VOL/VOL

Final Cover

FIELD CAPACITY	=	0.0100	VOL/VOL
WILTING POINT	=	0.0050	VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.0100	VOL/VOL
EFFECTIVE SAT. HYD. CONDUCT.	=	5.200	CM/SEC
SLOPE	=	2.00	PERCENT
DRAINAGE LENGTH	=	165.0	FEET

LAYER 11

TYPE 4 - FLEXIBLE MEMBRANE LINER
MATERIAL TEXTURE NUMBER 35

THICKNESS	=	0.06	INCHES
EFFECTIVE SAT. HYD. CONDUCT.	=	0.2000E-12	CM/SEC
FML PINHOLE DENSITY	=	2.00	HOLES/ACRE
FML INSTALLATION DEFECTS	=	2.00	HOLES/ACRE
FML PLACEMENT QUALITY	=	3 - GOOD	

LAYER 12

TYPE 1 - VERTICAL PERCOLATION LAYER
MATERIAL TEXTURE NUMBER 17

THICKNESS	=	0.25	INCHES
POROSITY	=	0.7500	VOL/VOL
FIELD CAPACITY	=	0.7470	VOL/VOL
WILTING POINT	=	0.4000	VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.7470	VOL/VOL
EFFECTIVE SAT. HYD. CONDUCT.	=	0.3000E-08	CM/SEC

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GENERAL DESIGN AND EVAPORATIVE ZONE DATA 1

VALID FOR 30 YEARS

NOTE: SCS RUNOFF CURVE NUMBER WAS COMPUTED FROM DEFAULT SOIL DATA BASE USING SOIL TEXTURE # 4 WITH A GOOD STAND OF GRASS, A SURFACE SLOPE OF 33.% AND A SLOPE LENGTH OF 127. FEET.

SCS RUNOFF CURVE NUMBER	=	57.82	
FRACTION OF AREA ALLOWING RUNOFF	=	100.0	PERCENT
AREA PROJECTED ON HORIZONTAL PLANE	=	1.000	ACRES
EVAPORATIVE ZONE DEPTH	=	6.0	INCHES
INITIAL WATER IN EVAPORATIVE ZONE	=	1.902	INCHES
UPPER LIMIT OF EVAPORATIVE STORAGE	=	2.838	INCHES
FIELD CAPACITY OF EVAPORATIVE ZONE	=	1.332	INCHES
LOWER LIMIT OF EVAPORATIVE STORAGE	=	0.624	INCHES
SOIL EVAPORATION ZONE DEPTH	=	6.000	INCHES
INITIAL SNOW WATER	=	0.000	INCHES
INITIAL INTERCEPTION WATER	=	0.016	INCHES
INITIAL WATER IN LAYER MATERIALS	=	300.910	INCHES
TOTAL INITIAL WATER	=	300.927	INCHES
TOTAL SUBSURFACE INFLOW	=	0.00	INCHES/YEAR

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EVAPOTRANSPIRATION DATA 1

VALID FOR 30 YEARS

NOTE: EVAPOTRANSPIRATION DATA WAS OBTAINED FROM
TALLAHASSEE FLORIDA

STATION LATITUDE	=	30.23	DEGREES
MAXIMUM LEAF AREA INDEX	=	3.50	
START OF GROWING SEASON (JULIAN DATE)	=	36	
END OF GROWING SEASON (JULIAN DATE)	=	350	
EVAPORATIVE ZONE DEPTH	=	6.0	INCHES
AVERAGE ANNUAL WIND SPEED	=	6.60	MPH
AVERAGE 1ST QUARTER RELATIVE HUMIDITY	=	73.0	%
AVERAGE 2ND QUARTER RELATIVE HUMIDITY	=	73.0	%
AVERAGE 3RD QUARTER RELATIVE HUMIDITY	=	80.0	%
AVERAGE 4TH QUARTER RELATIVE HUMIDITY	=	77.0	%

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ANNUAL TOTALS FOR YEAR 1

	INCHES	CU. FEET	PERCENT
PRECIPITATION	48.59	176381.703	100.00
RUNOFF	0.000	0.000	0.00
POTENTIAL EVAPOTRANSPIRATION	54.031	196131.859	
ACTUAL EVAPOTRANSPIRATION	27.365	99334.445	56.32
DRAINAGE COLLECTED FROM LAYER 3	21.2239	77042.602	43.68
PERC./LEAKAGE THROUGH LAYER 4	0.000600	2.180	0.00
AVG. HEAD ON TOP OF LAYER 4	0.0005		
DRAINAGE COLLECTED FROM LAYER 8	0.0000	0.114	0.00
PERC./LEAKAGE THROUGH LAYER 9	0.000569	2.066	0.00
AVG. HEAD ON TOP OF LAYER 9	0.0000		
DRAINAGE COLLECTED FROM LAYER 10	0.0006	2.058	0.00
PERC./LEAKAGE THROUGH LAYER 11	0.000002	0.008	0.00
AVG. HEAD ON TOP OF LAYER 11	0.0000		
PERC./LEAKAGE THROUGH LAYER 12	0.000002	0.008	0.00

CHANGE IN WATER STORAGE	0.001	2.217	0.00
SOIL WATER AT START OF YEAR	303.764	1102664.125	
SOIL WATER AT END OF YEAR	303.764	1102664.750	
INTERCEPTION WATER AT START OF YEAR	0.016	59.438	
INTERCEPTION WATER AT END OF YEAR	0.017	61.041	
SNOW WATER AT START OF YEAR	0.000	0.000	0.00
SNOW WATER AT END OF YEAR	0.000	0.000	0.00
ANNUAL WATER BUDGET BALANCE	0.0001	0.258	0.00

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ANNUAL TOTALS FOR YEAR 2

	INCHES	CU. FEET	PERCENT
PRECIPITATION	80.32	291561.531	100.00
RUNOFF	0.012	44.966	0.02
POTENTIAL EVAPOTRANSPIRATION	53.954	195854.750	
ACTUAL EVAPOTRANSPIRATION	33.826	122787.602	42.11
DRAINAGE COLLECTED FROM LAYER 3	45.2096	164110.766	56.29
PERC./LEAKAGE THROUGH LAYER 4	0.001073	3.894	0.00
AVG. HEAD ON TOP OF LAYER 4	0.0012		
DRAINAGE COLLECTED FROM LAYER 8	0.0001	0.251	0.00
PERC./LEAKAGE THROUGH LAYER 9	0.001004	3.643	0.00
AVG. HEAD ON TOP OF LAYER 9	0.0000		
DRAINAGE COLLECTED FROM LAYER 10	0.0010	3.633	0.00
PERC./LEAKAGE THROUGH LAYER 11	0.000002	0.009	0.00
AVG. HEAD ON TOP OF LAYER 11	0.0000		
PERC./LEAKAGE THROUGH LAYER 12	0.000002	0.009	0.00
CHANGE IN WATER STORAGE	1.271	4614.380	1.58
SOIL WATER AT START OF YEAR	303.764	1102664.750	
SOIL WATER AT END OF YEAR	305.052	1107340.125	

INTERCEPTION WATER AT START OF YEAR	0.017	61.041	
INTERCEPTION WATER AT END OF YEAR	0.000	0.000	
SNOW WATER AT START OF YEAR	0.000	0.000	0.00
SNOW WATER AT END OF YEAR	0.000	0.000	0.00
ANNUAL WATER BUDGET BALANCE	0.0000	-0.118	0.00

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ANNUAL TOTALS FOR YEAR 3

	INCHES	CU. FEET	PERCENT
PRECIPITATION	69.92	253809.578	100.00
RUNOFF	0.081	294.127	0.12
POTENTIAL EVAPOTRANSPIRATION	54.644	198359.312	
ACTUAL EVAPOTRANSPIRATION	31.465	114218.055	45.00
DRAINAGE COLLECTED FROM LAYER 3	40.2763	146202.984	57.60
PERC./LEAKAGE THROUGH LAYER 4	0.001023	3.712	0.00
AVG. HEAD ON TOP OF LAYER 4	0.0010		
DRAINAGE COLLECTED FROM LAYER 8	0.0001	0.227	0.00
PERC./LEAKAGE THROUGH LAYER 9	0.000960	3.485	0.00
AVG. HEAD ON TOP OF LAYER 9	0.0000		
DRAINAGE COLLECTED FROM LAYER 10	0.0010	3.476	0.00
PERC./LEAKAGE THROUGH LAYER 11	0.000002	0.009	0.00
AVG. HEAD ON TOP OF LAYER 11	0.0000		
PERC./LEAKAGE THROUGH LAYER 12	0.000002	0.009	0.00
CHANGE IN WATER STORAGE	-1.903	-6909.274	-2.72
SOIL WATER AT START OF YEAR	305.052	1107340.125	
SOIL WATER AT END OF YEAR	303.149	1100430.875	
INTERCEPTION WATER AT START OF YEAR	0.000	0.000	
INTERCEPTION WATER AT END OF YEAR	0.000	0.000	
SNOW WATER AT START OF YEAR	0.000	0.000	0.00
SNOW WATER AT END OF YEAR	0.000	0.000	0.00

ANNUAL WATER BUDGET BALANCE	0.0000	-0.006	0.00

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ANNUAL TOTALS FOR YEAR 4

	INCHES	CU. FEET	PERCENT
PRECIPITATION	58.15	211084.484	100.00
RUNOFF	0.024	86.355	0.04
POTENTIAL EVAPOTRANSPIRATION	54.811	198963.734	
ACTUAL EVAPOTRANSPIRATION	32.231	116997.930	55.43
DRAINAGE COLLECTED FROM LAYER 3	26.2058	95127.062	45.07
PERC./LEAKAGE THROUGH LAYER 4	0.000741	2.688	0.00
AVG. HEAD ON TOP OF LAYER 4	0.0007		
DRAINAGE COLLECTED FROM LAYER 8	0.0000	0.144	0.00
PERC./LEAKAGE THROUGH LAYER 9	0.000701	2.544	0.00
AVG. HEAD ON TOP OF LAYER 9	0.0000		
DRAINAGE COLLECTED FROM LAYER 10	0.0007	2.538	0.00
PERC./LEAKAGE THROUGH LAYER 11	0.000002	0.009	0.00
AVG. HEAD ON TOP OF LAYER 11	0.0000		
PERC./LEAKAGE THROUGH LAYER 12	0.000002	0.009	0.00
CHANGE IN WATER STORAGE	-0.311	-1129.611	-0.54
SOIL WATER AT START OF YEAR	303.149	1100430.875	
SOIL WATER AT END OF YEAR	302.838	1099301.375	
INTERCEPTION WATER AT START OF YEAR	0.000	0.000	
INTERCEPTION WATER AT END OF YEAR	0.000	0.000	
SNOW WATER AT START OF YEAR	0.000	0.000	0.00
SNOW WATER AT END OF YEAR	0.000	0.000	0.00
ANNUAL WATER BUDGET BALANCE	0.0000	0.060	0.00

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ANNUAL TOTALS FOR YEAR 5

	INCHES	CU. FEET	PERCENT
PRECIPITATION	70.21	254862.266	100.00
RUNOFF	0.007	25.442	0.01
POTENTIAL EVAPOTRANSPIRATION	53.436	193973.516	
ACTUAL EVAPOTRANSPIRATION	33.905	123074.180	48.29
DRAINAGE COLLECTED FROM LAYER 3	36.5859	132806.797	52.11
PERC./LEAKAGE THROUGH LAYER 4	0.000951	3.453	0.00
AVG. HEAD ON TOP OF LAYER 4	0.0009		
DRAINAGE COLLECTED FROM LAYER 8	0.0001	0.204	0.00
PERC./LEAKAGE THROUGH LAYER 9	0.000895	3.249	0.00
AVG. HEAD ON TOP OF LAYER 9	0.0000		
DRAINAGE COLLECTED FROM LAYER 10	0.0009	3.241	0.00
PERC./LEAKAGE THROUGH LAYER 11	0.000002	0.009	0.00
AVG. HEAD ON TOP OF LAYER 11	0.0000		
PERC./LEAKAGE THROUGH LAYER 12	0.000002	0.009	0.00
CHANGE IN WATER STORAGE	-0.289	-1047.414	-0.41
SOIL WATER AT START OF YEAR	302.838	1099301.375	
SOIL WATER AT END OF YEAR	302.549	1098253.875	
INTERCEPTION WATER AT START OF YEAR	0.000	0.000	
INTERCEPTION WATER AT END OF YEAR	0.000	0.000	
SNOW WATER AT START OF YEAR	0.000	0.000	0.00
SNOW WATER AT END OF YEAR	0.000	0.000	0.00
ANNUAL WATER BUDGET BALANCE	-0.0001	-0.192	0.00

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ANNUAL TOTALS FOR YEAR 6

	INCHES	CU. FEET	PERCENT

Final Cover			
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PRECIPITATION	73.90	268256.906	100.00
RUNOFF	0.002	9.054	0.00
POTENTIAL EVAPOTRANSPIRATION	54.288	197066.312	
ACTUAL EVAPOTRANSPIRATION	35.260	127994.914	47.71
DRAINAGE COLLECTED FROM LAYER 3	38.4767	139670.359	52.07
PERC./LEAKAGE THROUGH LAYER 4	0.001010	3.665	0.00
AVG. HEAD ON TOP OF LAYER 4	0.0010		
DRAINAGE COLLECTED FROM LAYER 8	0.0001	0.216	0.00
PERC./LEAKAGE THROUGH LAYER 9	0.000950	3.449	0.00
AVG. HEAD ON TOP OF LAYER 9	0.0000		
DRAINAGE COLLECTED FROM LAYER 10	0.0009	3.439	0.00
PERC./LEAKAGE THROUGH LAYER 11	0.000002	0.009	0.00
AVG. HEAD ON TOP OF LAYER 11	0.0000		
PERC./LEAKAGE THROUGH LAYER 12	0.000002	0.009	0.00
CHANGE IN WATER STORAGE	0.159	578.930	0.22
SOIL WATER AT START OF YEAR	302.549	1098253.875	
SOIL WATER AT END OF YEAR	302.709	1098832.875	
INTERCEPTION WATER AT START OF YEAR	0.000	0.000	
INTERCEPTION WATER AT END OF YEAR	0.000	0.000	
SNOW WATER AT START OF YEAR	0.000	0.000	0.00
SNOW WATER AT END OF YEAR	0.000	0.000	0.00
ANNUAL WATER BUDGET BALANCE	0.0000	-0.012	0.00

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ANNUAL TOTALS FOR YEAR 7

	INCHES	CU. FEET	PERCENT
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PRECIPITATION	68.37	248183.109	100.00
RUNOFF	0.188	681.451	0.27
POTENTIAL EVAPOTRANSPIRATION	54.391	197439.922	

ACTUAL EVAPOTRANSPIRATION	30.457	110558.539	44.55
DRAINAGE COLLECTED FROM LAYER 3	37.5005	136126.969	54.85
PERC./LEAKAGE THROUGH LAYER 4	0.000912	3.312	0.00
AVG. HEAD ON TOP OF LAYER 4	0.0010		
DRAINAGE COLLECTED FROM LAYER 8	0.0001	0.209	0.00
PERC./LEAKAGE THROUGH LAYER 9	0.000855	3.103	0.00
AVG. HEAD ON TOP OF LAYER 9	0.0000		
DRAINAGE COLLECTED FROM LAYER 10	0.0009	3.094	0.00
PERC./LEAKAGE THROUGH LAYER 11	0.000002	0.008	0.00
AVG. HEAD ON TOP OF LAYER 11	0.0000		
PERC./LEAKAGE THROUGH LAYER 12	0.000002	0.008	0.00
CHANGE IN WATER STORAGE	0.224	812.784	0.33
SOIL WATER AT START OF YEAR	302.709	1098832.875	
SOIL WATER AT END OF YEAR	302.933	1099645.625	
INTERCEPTION WATER AT START OF YEAR	0.000	0.000	
INTERCEPTION WATER AT END OF YEAR	0.000	0.000	
SNOW WATER AT START OF YEAR	0.000	0.000	0.00
SNOW WATER AT END OF YEAR	0.000	0.000	0.00
ANNUAL WATER BUDGET BALANCE	0.0000	0.035	0.00

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ANNUAL TOTALS FOR YEAR 8

	INCHES	CU. FEET	PERCENT
PRECIPITATION	66.99	243173.688	100.00
RUNOFF	0.075	273.145	0.11
POTENTIAL EVAPOTRANSPIRATION	54.866	199164.266	
ACTUAL EVAPOTRANSPIRATION	31.228	113358.531	46.62
DRAINAGE COLLECTED FROM LAYER 3	35.4308	128613.750	52.89
PERC./LEAKAGE THROUGH LAYER 4	0.000920	3.341	0.00
AVG. HEAD ON TOP OF LAYER 4	0.0009		

DRAINAGE COLLECTED FROM LAYER 8	0.0001	0.196	0.00
PERC./LEAKAGE THROUGH LAYER 9	0.000867	3.146	0.00
AVG. HEAD ON TOP OF LAYER 9	0.0000		
DRAINAGE COLLECTED FROM LAYER 10	0.0009	3.137	0.00
PERC./LEAKAGE THROUGH LAYER 11	0.000002	0.009	0.00
AVG. HEAD ON TOP OF LAYER 11	0.0000		
PERC./LEAKAGE THROUGH LAYER 12	0.000002	0.009	0.00
CHANGE IN WATER STORAGE	0.255	925.003	0.38
SOIL WATER AT START OF YEAR	302.933	1099645.625	
SOIL WATER AT END OF YEAR	303.188	1100570.625	
INTERCEPTION WATER AT START OF YEAR	0.000	0.000	
INTERCEPTION WATER AT END OF YEAR	0.000	0.000	
SNOW WATER AT START OF YEAR	0.000	0.000	0.00
SNOW WATER AT END OF YEAR	0.000	0.000	0.00
ANNUAL WATER BUDGET BALANCE	0.0000	-0.092	0.00

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ANNUAL TOTALS FOR YEAR 9

	INCHES	CU. FEET	PERCENT
PRECIPITATION	67.42	244734.578	100.00
RUNOFF	0.015	52.872	0.02
POTENTIAL EVAPOTRANSPIRATION	54.852	199112.859	
ACTUAL EVAPOTRANSPIRATION	34.116	123840.875	50.60
DRAINAGE COLLECTED FROM LAYER 3	33.3745	121149.258	49.50
PERC./LEAKAGE THROUGH LAYER 4	0.000908	3.295	0.00
AVG. HEAD ON TOP OF LAYER 4	0.0008		
DRAINAGE COLLECTED FROM LAYER 8	0.0001	0.183	0.00
PERC./LEAKAGE THROUGH LAYER 9	0.000857	3.112	0.00
AVG. HEAD ON TOP OF LAYER 9	0.0000		

DRAINAGE COLLECTED FROM LAYER 10	0.0009	3.105	0.00
PERC./LEAKAGE THROUGH LAYER 11	0.000002	0.009	0.00
AVG. HEAD ON TOP OF LAYER 11	0.0000		
PERC./LEAKAGE THROUGH LAYER 12	0.000002	0.009	0.00
CHANGE IN WATER STORAGE	-0.086	-311.842	-0.13
SOIL WATER AT START OF YEAR	303.188	1100570.625	
SOIL WATER AT END OF YEAR	303.102	1100258.750	
INTERCEPTION WATER AT START OF YEAR	0.000	0.000	
INTERCEPTION WATER AT END OF YEAR	0.000	0.000	
SNOW WATER AT START OF YEAR	0.000	0.000	0.00
SNOW WATER AT END OF YEAR	0.000	0.000	0.00
ANNUAL WATER BUDGET BALANCE	0.0000	0.132	0.00

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ANNUAL TOTALS FOR YEAR 10

	INCHES	CU. FEET	PERCENT
PRECIPITATION	71.32	258891.562	100.00
RUNOFF	0.114	412.835	0.16
POTENTIAL EVAPOTRANSPIRATION	54.532	197951.062	
ACTUAL EVAPOTRANSPIRATION	31.681	115003.664	44.42
DRAINAGE COLLECTED FROM LAYER 3	39.5512	143570.922	55.46
PERC./LEAKAGE THROUGH LAYER 4	0.000982	3.565	0.00
AVG. HEAD ON TOP OF LAYER 4	0.0010		
DRAINAGE COLLECTED FROM LAYER 8	0.0001	0.220	0.00
PERC./LEAKAGE THROUGH LAYER 9	0.000921	3.345	0.00
AVG. HEAD ON TOP OF LAYER 9	0.0000		
DRAINAGE COLLECTED FROM LAYER 10	0.0009	3.334	0.00
PERC./LEAKAGE THROUGH LAYER 11	0.000002	0.009	0.00
AVG. HEAD ON TOP OF LAYER 11	0.0000		
PERC./LEAKAGE THROUGH LAYER 12	0.000002	0.009	0.00

CHANGE IN WATER STORAGE	-0.027	-99.369	-0.04
SOIL WATER AT START OF YEAR	303.102	1100258.750	
SOIL WATER AT END OF YEAR	303.074	1100159.375	
INTERCEPTION WATER AT START OF YEAR	0.000	0.000	
INTERCEPTION WATER AT END OF YEAR	0.000	0.000	
SNOW WATER AT START OF YEAR	0.000	0.000	0.00
SNOW WATER AT END OF YEAR	0.000	0.000	0.00
ANNUAL WATER BUDGET BALANCE	0.0000	-0.064	0.00

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ANNUAL TOTALS FOR YEAR 11

	INCHES	CU. FEET	PERCENT
PRECIPITATION	65.26	236893.766	100.00
RUNOFF	0.001	3.262	0.00
POTENTIAL EVAPOTRANSPIRATION	54.803	198934.250	
ACTUAL EVAPOTRANSPIRATION	31.059	112743.445	47.59
DRAINAGE COLLECTED FROM LAYER 3	33.6095	122002.336	51.50
PERC./LEAKAGE THROUGH LAYER 4	0.000878	3.189	0.00
AVG. HEAD ON TOP OF LAYER 4	0.0009		
DRAINAGE COLLECTED FROM LAYER 8	0.0001	0.186	0.00
PERC./LEAKAGE THROUGH LAYER 9	0.000827	3.003	0.00
AVG. HEAD ON TOP OF LAYER 9	0.0000		
DRAINAGE COLLECTED FROM LAYER 10	0.0008	2.998	0.00
PERC./LEAKAGE THROUGH LAYER 11	0.000002	0.009	0.00
AVG. HEAD ON TOP OF LAYER 11	0.0000		
PERC./LEAKAGE THROUGH LAYER 12	0.000002	0.009	0.00
CHANGE IN WATER STORAGE	0.590	2141.524	0.90
SOIL WATER AT START OF YEAR	303.074	1100159.375	
SOIL WATER AT END OF YEAR	303.630	1102176.125	

INTERCEPTION WATER AT START OF YEAR	0.000	0.000	
INTERCEPTION WATER AT END OF YEAR	0.034	124.795	
SNOW WATER AT START OF YEAR	0.000	0.000	0.00
SNOW WATER AT END OF YEAR	0.000	0.000	0.00
ANNUAL WATER BUDGET BALANCE	0.0000	0.016	0.00

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ANNUAL TOTALS FOR YEAR 12

	INCHES	CU. FEET	PERCENT
PRECIPITATION	52.45	190393.484	100.00
RUNOFF	0.000	0.000	0.00
POTENTIAL EVAPOTRANSPIRATION	55.172	200276.109	
ACTUAL EVAPOTRANSPIRATION	29.169	105882.445	55.61
DRAINAGE COLLECTED FROM LAYER 3	23.3483	84754.383	44.52
PERC./LEAKAGE THROUGH LAYER 4	0.000699	2.537	0.00
AVG. HEAD ON TOP OF LAYER 4	0.0006		
DRAINAGE COLLECTED FROM LAYER 8	0.0000	0.123	0.00
PERC./LEAKAGE THROUGH LAYER 9	0.000665	2.414	0.00
AVG. HEAD ON TOP OF LAYER 9	0.0000		
DRAINAGE COLLECTED FROM LAYER 10	0.0007	2.403	0.00
PERC./LEAKAGE THROUGH LAYER 11	0.000002	0.009	0.00
AVG. HEAD ON TOP OF LAYER 11	0.0000		
PERC./LEAKAGE THROUGH LAYER 12	0.000002	0.009	0.00
CHANGE IN WATER STORAGE	-0.068	-246.040	-0.13
SOIL WATER AT START OF YEAR	303.630	1102176.125	
SOIL WATER AT END OF YEAR	303.596	1102054.875	
INTERCEPTION WATER AT START OF YEAR	0.034	124.795	
INTERCEPTION WATER AT END OF YEAR	0.000	0.000	
SNOW WATER AT START OF YEAR	0.000	0.000	0.00
SNOW WATER AT END OF YEAR	0.000	0.000	0.00

ANNUAL WATER BUDGET BALANCE	0.0000	0.168	0.00

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ANNUAL TOTALS FOR YEAR 13

	INCHES	CU. FEET	PERCENT
PRECIPITATION	54.63	198306.891	100.00
RUNOFF	0.041	147.907	0.07
POTENTIAL EVAPOTRANSPIRATION	55.320	200811.297	
ACTUAL EVAPOTRANSPIRATION	27.638	100324.258	50.59
DRAINAGE COLLECTED FROM LAYER 3	28.1070	102028.352	51.45
PERC./LEAKAGE THROUGH LAYER 4	0.000763	2.771	0.00
AVG. HEAD ON TOP OF LAYER 4	0.0007		
DRAINAGE COLLECTED FROM LAYER 8	0.0000	0.157	0.00
PERC./LEAKAGE THROUGH LAYER 9	0.000720	2.614	0.00
AVG. HEAD ON TOP OF LAYER 9	0.0000		
DRAINAGE COLLECTED FROM LAYER 10	0.0007	2.607	0.00
PERC./LEAKAGE THROUGH LAYER 11	0.000002	0.009	0.00
AVG. HEAD ON TOP OF LAYER 11	0.0000		
PERC./LEAKAGE THROUGH LAYER 12	0.000002	0.009	0.00
CHANGE IN WATER STORAGE	-1.156	-4196.301	-2.12
SOIL WATER AT START OF YEAR	303.596	1102054.875	
SOIL WATER AT END OF YEAR	302.440	1097858.625	
INTERCEPTION WATER AT START OF YEAR	0.000	0.000	
INTERCEPTION WATER AT END OF YEAR	0.000	0.000	
SNOW WATER AT START OF YEAR	0.000	0.000	0.00
SNOW WATER AT END OF YEAR	0.000	0.000	0.00
ANNUAL WATER BUDGET BALANCE	0.0000	-0.084	0.00

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ANNUAL TOTALS FOR YEAR 14

	INCHES	CU. FEET	PERCENT
PRECIPITATION	59.92	217509.609	100.00
RUNOFF	0.245	889.886	0.41
POTENTIAL EVAPOTRANSPIRATION	54.090	196346.109	
ACTUAL EVAPOTRANSPIRATION	28.847	104713.461	48.14
DRAINAGE COLLECTED FROM LAYER 3	30.4305	110462.586	50.79
PERC./LEAKAGE THROUGH LAYER 4	0.000803	2.917	0.00
AVG. HEAD ON TOP OF LAYER 4	0.0008		
DRAINAGE COLLECTED FROM LAYER 8	0.0000	0.168	0.00
PERC./LEAKAGE THROUGH LAYER 9	0.000757	2.749	0.00
AVG. HEAD ON TOP OF LAYER 9	0.0000		
DRAINAGE COLLECTED FROM LAYER 10	0.0008	2.740	0.00
PERC./LEAKAGE THROUGH LAYER 11	0.000002	0.008	0.00
AVG. HEAD ON TOP OF LAYER 11	0.0000		
PERC./LEAKAGE THROUGH LAYER 12	0.000002	0.008	0.00
CHANGE IN WATER STORAGE	0.397	1440.678	0.66
SOIL WATER AT START OF YEAR	302.440	1097858.625	
SOIL WATER AT END OF YEAR	302.837	1099299.375	
INTERCEPTION WATER AT START OF YEAR	0.000	0.000	
INTERCEPTION WATER AT END OF YEAR	0.000	0.000	
SNOW WATER AT START OF YEAR	0.000	0.000	0.00
SNOW WATER AT END OF YEAR	0.000	0.000	0.00
ANNUAL WATER BUDGET BALANCE	0.0000	0.083	0.00

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ANNUAL TOTALS FOR YEAR 15

	INCHES	CU. FEET	PERCENT

Final Cover			
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PRECIPITATION	56.04	203425.203	100.00
RUNOFF	0.003	12.372	0.01
POTENTIAL EVAPOTRANSPIRATION	56.640	205602.625	
ACTUAL EVAPOTRANSPIRATION	25.475	92475.180	45.46
DRAINAGE COLLECTED FROM LAYER 3	30.1096	109297.969	53.73
PERC./LEAKAGE THROUGH LAYER 4	0.000779	2.829	0.00
AVG. HEAD ON TOP OF LAYER 4	0.0008		
DRAINAGE COLLECTED FROM LAYER 8	0.0000	0.165	0.00
PERC./LEAKAGE THROUGH LAYER 9	0.000734	2.664	0.00
AVG. HEAD ON TOP OF LAYER 9	0.0000		
DRAINAGE COLLECTED FROM LAYER 10	0.0007	2.653	0.00
PERC./LEAKAGE THROUGH LAYER 11	0.000002	0.009	0.00
AVG. HEAD ON TOP OF LAYER 11	0.0000		
PERC./LEAKAGE THROUGH LAYER 12	0.000002	0.009	0.00
CHANGE IN WATER STORAGE	0.451	1636.979	0.80
SOIL WATER AT START OF YEAR	302.837	1099299.375	
SOIL WATER AT END OF YEAR	303.288	1100936.375	
INTERCEPTION WATER AT START OF YEAR	0.000	0.000	
INTERCEPTION WATER AT END OF YEAR	0.000	0.000	
SNOW WATER AT START OF YEAR	0.000	0.000	0.00
SNOW WATER AT END OF YEAR	0.000	0.000	0.00
ANNUAL WATER BUDGET BALANCE	0.0000	-0.104	0.00

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ANNUAL TOTALS FOR YEAR 16

	INCHES	CU. FEET	PERCENT
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PRECIPITATION	50.58	183605.375	100.00
RUNOFF	0.000	0.000	0.00
POTENTIAL EVAPOTRANSPIRATION	55.167	200257.641	

ACTUAL EVAPOTRANSPIRATION	27.460	99678.023	54.29
DRAINAGE COLLECTED FROM LAYER 3	23.4936	85281.758	46.45
PERC./LEAKAGE THROUGH LAYER 4	0.000697	2.529	0.00
AVG. HEAD ON TOP OF LAYER 4	0.0006		
DRAINAGE COLLECTED FROM LAYER 8	0.0000	0.126	0.00
PERC./LEAKAGE THROUGH LAYER 9	0.000662	2.403	0.00
AVG. HEAD ON TOP OF LAYER 9	0.0000		
DRAINAGE COLLECTED FROM LAYER 10	0.0007	2.395	0.00
PERC./LEAKAGE THROUGH LAYER 11	0.000002	0.009	0.00
AVG. HEAD ON TOP OF LAYER 11	0.0000		
PERC./LEAKAGE THROUGH LAYER 12	0.000002	0.009	0.00
CHANGE IN WATER STORAGE	-0.374	-1357.040	-0.74
SOIL WATER AT START OF YEAR	303.288	1100936.375	
SOIL WATER AT END OF YEAR	302.914	1099579.250	
INTERCEPTION WATER AT START OF YEAR	0.000	0.000	
INTERCEPTION WATER AT END OF YEAR	0.000	0.000	
SNOW WATER AT START OF YEAR	0.000	0.000	0.00
SNOW WATER AT END OF YEAR	0.000	0.000	0.00
ANNUAL WATER BUDGET BALANCE	0.0000	0.095	0.00

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ANNUAL TOTALS FOR YEAR 17

	INCHES	CU. FEET	PERCENT
PRECIPITATION	70.44	255697.203	100.00
RUNOFF	0.054	197.606	0.08
POTENTIAL EVAPOTRANSPIRATION	55.309	200771.188	
ACTUAL EVAPOTRANSPIRATION	33.922	123138.648	48.16
DRAINAGE COLLECTED FROM LAYER 3	36.6317	132973.109	52.00
PERC./LEAKAGE THROUGH LAYER 4	0.000940	3.413	0.00
AVG. HEAD ON TOP OF LAYER 4	0.0009		

DRAINAGE COLLECTED FROM LAYER 8	0.0001	0.203	0.00
PERC./LEAKAGE THROUGH LAYER 9	0.000884	3.210	0.00
AVG. HEAD ON TOP OF LAYER 9	0.0000		
DRAINAGE COLLECTED FROM LAYER 10	0.0009	3.202	0.00
PERC./LEAKAGE THROUGH LAYER 11	0.000002	0.009	0.00
AVG. HEAD ON TOP OF LAYER 11	0.0000		
PERC./LEAKAGE THROUGH LAYER 12	0.000002	0.009	0.00
CHANGE IN WATER STORAGE	-0.170	-615.376	-0.24
SOIL WATER AT START OF YEAR	302.914	1099579.250	
SOIL WATER AT END OF YEAR	302.745	1098963.875	
INTERCEPTION WATER AT START OF YEAR	0.000	0.000	
INTERCEPTION WATER AT END OF YEAR	0.000	0.000	
SNOW WATER AT START OF YEAR	0.000	0.000	0.00
SNOW WATER AT END OF YEAR	0.000	0.000	0.00
ANNUAL WATER BUDGET BALANCE	-0.0001	-0.192	0.00

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ANNUAL TOTALS FOR YEAR 18

	INCHES	CU. FEET	PERCENT
PRECIPITATION	57.83	209922.891	100.00
RUNOFF	0.171	620.557	0.30
POTENTIAL EVAPOTRANSPIRATION	54.640	198342.125	
ACTUAL EVAPOTRANSPIRATION	29.201	105998.680	50.49
DRAINAGE COLLECTED FROM LAYER 3	28.9187	104974.992	50.01
PERC./LEAKAGE THROUGH LAYER 4	0.000777	2.820	0.00
AVG. HEAD ON TOP OF LAYER 4	0.0007		
DRAINAGE COLLECTED FROM LAYER 8	0.0000	0.161	0.00
PERC./LEAKAGE THROUGH LAYER 9	0.000733	2.659	0.00
AVG. HEAD ON TOP OF LAYER 9	0.0000		

DRAINAGE COLLECTED FROM LAYER 10	0.0007	2.653	0.00
PERC./LEAKAGE THROUGH LAYER 11	0.000002	0.009	0.00
AVG. HEAD ON TOP OF LAYER 11	0.0000		
PERC./LEAKAGE THROUGH LAYER 12	0.000002	0.009	0.00
CHANGE IN WATER STORAGE	-0.461	-1674.200	-0.80
SOIL WATER AT START OF YEAR	302.745	1098963.875	
SOIL WATER AT END OF YEAR	302.284	1097289.625	
INTERCEPTION WATER AT START OF YEAR	0.000	0.000	
INTERCEPTION WATER AT END OF YEAR	0.000	0.000	
SNOW WATER AT START OF YEAR	0.000	0.000	0.00
SNOW WATER AT END OF YEAR	0.000	0.000	0.00
ANNUAL WATER BUDGET BALANCE	0.0000	0.053	0.00

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ANNUAL TOTALS FOR YEAR 19

	INCHES	CU. FEET	PERCENT
PRECIPITATION	75.35	273520.531	100.00
RUNOFF	0.029	104.880	0.04
POTENTIAL EVAPOTRANSPIRATION	52.829	191769.641	
ACTUAL EVAPOTRANSPIRATION	36.807	133609.891	48.85
DRAINAGE COLLECTED FROM LAYER 3	37.7037	136864.500	50.04
PERC./LEAKAGE THROUGH LAYER 4	0.000979	3.555	0.00
AVG. HEAD ON TOP OF LAYER 4	0.0010		
DRAINAGE COLLECTED FROM LAYER 8	0.0001	0.209	0.00
PERC./LEAKAGE THROUGH LAYER 9	0.000922	3.346	0.00
AVG. HEAD ON TOP OF LAYER 9	0.0000		
DRAINAGE COLLECTED FROM LAYER 10	0.0009	3.336	0.00
PERC./LEAKAGE THROUGH LAYER 11	0.000002	0.009	0.00
AVG. HEAD ON TOP OF LAYER 11	0.0000		
PERC./LEAKAGE THROUGH LAYER 12	0.000002	0.009	0.00

CHANGE IN WATER STORAGE	0.809	2937.640	1.07
SOIL WATER AT START OF YEAR	302.284	1097289.625	
SOIL WATER AT END OF YEAR	303.074	1100158.625	
INTERCEPTION WATER AT START OF YEAR	0.000	0.000	
INTERCEPTION WATER AT END OF YEAR	0.019	68.690	
SNOW WATER AT START OF YEAR	0.000	0.000	0.00
SNOW WATER AT END OF YEAR	0.000	0.000	0.00
ANNUAL WATER BUDGET BALANCE	0.0000	0.069	0.00

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ANNUAL TOTALS FOR YEAR 20

	INCHES	CU. FEET	PERCENT
PRECIPITATION	59.49	215948.734	100.00
RUNOFF	0.000	1.224	0.00
POTENTIAL EVAPOTRANSPIRATION	57.884	210118.578	
ACTUAL EVAPOTRANSPIRATION	30.694	111420.375	51.60
DRAINAGE COLLECTED FROM LAYER 3	29.5560	107288.320	49.68
PERC./LEAKAGE THROUGH LAYER 4	0.000803	2.914	0.00
AVG. HEAD ON TOP OF LAYER 4	0.0007		
DRAINAGE COLLECTED FROM LAYER 8	0.0000	0.163	0.00
PERC./LEAKAGE THROUGH LAYER 9	0.000758	2.751	0.00
AVG. HEAD ON TOP OF LAYER 9	0.0000		
DRAINAGE COLLECTED FROM LAYER 10	0.0008	2.743	0.00
PERC./LEAKAGE THROUGH LAYER 11	0.000002	0.008	0.00
AVG. HEAD ON TOP OF LAYER 11	0.0000		
PERC./LEAKAGE THROUGH LAYER 12	0.000002	0.008	0.00
CHANGE IN WATER STORAGE	-0.762	-2764.264	-1.28
SOIL WATER AT START OF YEAR	303.074	1100158.625	
SOIL WATER AT END OF YEAR	302.331	1097463.125	

INTERCEPTION WATER AT START OF YEAR	0.019	68.690	
INTERCEPTION WATER AT END OF YEAR	0.000	0.000	
SNOW WATER AT START OF YEAR	0.000	0.000	0.00
SNOW WATER AT END OF YEAR	0.000	0.000	0.00
ANNUAL WATER BUDGET BALANCE	0.0000	0.175	0.00

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ANNUAL TOTALS FOR YEAR 21

	INCHES	CU. FEET	PERCENT
PRECIPITATION	65.64	238273.188	100.00
RUNOFF	0.000	0.000	0.00
POTENTIAL EVAPOTRANSPIRATION	55.173	200277.875	
ACTUAL EVAPOTRANSPIRATION	31.674	114977.117	48.25
DRAINAGE COLLECTED FROM LAYER 3	33.3555	121080.617	50.82
PERC./LEAKAGE THROUGH LAYER 4	0.000897	3.257	0.00
AVG. HEAD ON TOP OF LAYER 4	0.0008		
DRAINAGE COLLECTED FROM LAYER 8	0.0001	0.186	0.00
PERC./LEAKAGE THROUGH LAYER 9	0.000846	3.071	0.00
AVG. HEAD ON TOP OF LAYER 9	0.0000		
DRAINAGE COLLECTED FROM LAYER 10	0.0008	3.060	0.00
PERC./LEAKAGE THROUGH LAYER 11	0.000002	0.009	0.00
AVG. HEAD ON TOP OF LAYER 11	0.0000		
PERC./LEAKAGE THROUGH LAYER 12	0.000002	0.009	0.00
CHANGE IN WATER STORAGE	0.609	2212.364	0.93
SOIL WATER AT START OF YEAR	302.331	1097463.125	
SOIL WATER AT END OF YEAR	302.941	1099675.375	
INTERCEPTION WATER AT START OF YEAR	0.000	0.000	
INTERCEPTION WATER AT END OF YEAR	0.000	0.000	
SNOW WATER AT START OF YEAR	0.000	0.000	0.00
SNOW WATER AT END OF YEAR	0.000	0.000	0.00

ANNUAL WATER BUDGET BALANCE	0.0000	-0.158	0.00

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ANNUAL TOTALS FOR YEAR 22

	INCHES	CU. FEET	PERCENT
PRECIPITATION	71.54	259690.203	100.00
RUNOFF	0.002	8.067	0.00
POTENTIAL EVAPOTRANSPIRATION	53.971	195913.875	
ACTUAL EVAPOTRANSPIRATION	30.312	110031.352	42.37
DRAINAGE COLLECTED FROM LAYER 3	38.8449	141006.969	54.30
PERC./LEAKAGE THROUGH LAYER 4	0.001019	3.699	0.00
AVG. HEAD ON TOP OF LAYER 4	0.0010		
DRAINAGE COLLECTED FROM LAYER 8	0.0001	0.212	0.00
PERC./LEAKAGE THROUGH LAYER 9	0.000960	3.486	0.00
AVG. HEAD ON TOP OF LAYER 9	0.0000		
DRAINAGE COLLECTED FROM LAYER 10	0.0010	3.470	0.00
PERC./LEAKAGE THROUGH LAYER 11	0.000002	0.009	0.00
AVG. HEAD ON TOP OF LAYER 11	0.0000		
PERC./LEAKAGE THROUGH LAYER 12	0.000002	0.009	0.00
CHANGE IN WATER STORAGE	2.380	8640.083	3.33
SOIL WATER AT START OF YEAR	302.941	1099675.375	
SOIL WATER AT END OF YEAR	305.305	1108257.875	
INTERCEPTION WATER AT START OF YEAR	0.000	0.000	
INTERCEPTION WATER AT END OF YEAR	0.016	57.605	
SNOW WATER AT START OF YEAR	0.000	0.000	0.00
SNOW WATER AT END OF YEAR	0.000	0.000	0.00
ANNUAL WATER BUDGET BALANCE	0.0000	0.043	0.00

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ANNUAL TOTALS FOR YEAR 23

	INCHES	CU. FEET	PERCENT
PRECIPITATION	64.39	233735.734	100.00
RUNOFF	0.000	1.251	0.00
POTENTIAL EVAPOTRANSPIRATION	54.842	199077.625	
ACTUAL EVAPOTRANSPIRATION	31.258	113467.383	48.55
DRAINAGE COLLECTED FROM LAYER 3	36.2145	131458.453	56.24
PERC./LEAKAGE THROUGH LAYER 4	0.000948	3.441	0.00
AVG. HEAD ON TOP OF LAYER 4	0.0009		
DRAINAGE COLLECTED FROM LAYER 8	0.0001	0.203	0.00
PERC./LEAKAGE THROUGH LAYER 9	0.000892	3.238	0.00
AVG. HEAD ON TOP OF LAYER 9	0.0000		
DRAINAGE COLLECTED FROM LAYER 10	0.0009	3.238	0.00
PERC./LEAKAGE THROUGH LAYER 11	0.000002	0.009	0.00
AVG. HEAD ON TOP OF LAYER 11	0.0000		
PERC./LEAKAGE THROUGH LAYER 12	0.000002	0.009	0.00
CHANGE IN WATER STORAGE	-3.084	-11194.974	-4.79
SOIL WATER AT START OF YEAR	305.305	1108257.875	
SOIL WATER AT END OF YEAR	302.237	1097120.500	
INTERCEPTION WATER AT START OF YEAR	0.016	57.605	
INTERCEPTION WATER AT END OF YEAR	0.000	0.000	
SNOW WATER AT START OF YEAR	0.000	0.000	0.00
SNOW WATER AT END OF YEAR	0.000	0.000	0.00
ANNUAL WATER BUDGET BALANCE	0.0000	0.159	0.00

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ANNUAL TOTALS FOR YEAR 24

	INCHES	CU. FEET	PERCENT

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PRECIPITATION	80.16	290980.875	100.00
RUNOFF	0.139	505.373	0.17
POTENTIAL EVAPOTRANSPIRATION	55.334	200861.797	
ACTUAL EVAPOTRANSPIRATION	33.204	120531.281	41.42
DRAINAGE COLLECTED FROM LAYER 3	46.3170	168130.609	57.78
PERC./LEAKAGE THROUGH LAYER 4	0.001143	4.149	0.00
AVG. HEAD ON TOP OF LAYER 4	0.0012		
DRAINAGE COLLECTED FROM LAYER 8	0.0001	0.259	0.00
PERC./LEAKAGE THROUGH LAYER 9	0.001072	3.890	0.00
AVG. HEAD ON TOP OF LAYER 9	0.0000		
DRAINAGE COLLECTED FROM LAYER 10	0.0011	3.880	0.00
PERC./LEAKAGE THROUGH LAYER 11	0.000002	0.009	0.00
AVG. HEAD ON TOP OF LAYER 11	0.0000		
PERC./LEAKAGE THROUGH LAYER 12	0.000002	0.009	0.00
CHANGE IN WATER STORAGE	0.499	1809.572	0.62
SOIL WATER AT START OF YEAR	302.237	1097120.500	
SOIL WATER AT END OF YEAR	302.736	1098930.125	
INTERCEPTION WATER AT START OF YEAR	0.000	0.000	
INTERCEPTION WATER AT END OF YEAR	0.000	0.000	
SNOW WATER AT START OF YEAR	0.000	0.000	0.00
SNOW WATER AT END OF YEAR	0.000	0.000	0.00
ANNUAL WATER BUDGET BALANCE	0.0000	-0.109	0.00

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ANNUAL TOTALS FOR YEAR 25

	INCHES	CU. FEET	PERCENT
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PRECIPITATION	64.20	233046.047	100.00
RUNOFF	0.000	1.378	0.00
POTENTIAL EVAPOTRANSPIRATION	53.792	195263.141	

ACTUAL EVAPOTRANSPIRATION	34.782	126259.586	54.18
DRAINAGE COLLECTED FROM LAYER 3	29.3128	106405.633	45.66
PERC./LEAKAGE THROUGH LAYER 4	0.000836	3.034	0.00
AVG. HEAD ON TOP OF LAYER 4	0.0007		
DRAINAGE COLLECTED FROM LAYER 8	0.0000	0.159	0.00
PERC./LEAKAGE THROUGH LAYER 9	0.000792	2.875	0.00
AVG. HEAD ON TOP OF LAYER 9	0.0000		
DRAINAGE COLLECTED FROM LAYER 10	0.0008	2.866	0.00
PERC./LEAKAGE THROUGH LAYER 11	0.000002	0.009	0.00
AVG. HEAD ON TOP OF LAYER 11	0.0000		
PERC./LEAKAGE THROUGH LAYER 12	0.000002	0.009	0.00
CHANGE IN WATER STORAGE	0.104	376.205	0.16
SOIL WATER AT START OF YEAR	302.736	1098930.125	
SOIL WATER AT END OF YEAR	302.839	1099306.375	
INTERCEPTION WATER AT START OF YEAR	0.000	0.000	
INTERCEPTION WATER AT END OF YEAR	0.000	0.000	
SNOW WATER AT START OF YEAR	0.000	0.000	0.00
SNOW WATER AT END OF YEAR	0.000	0.000	0.00
ANNUAL WATER BUDGET BALANCE	0.0001	0.208	0.00

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ANNUAL TOTALS FOR YEAR 26

	INCHES	CU. FEET	PERCENT
PRECIPITATION	67.32	244371.625	100.00
RUNOFF	0.047	172.063	0.07
POTENTIAL EVAPOTRANSPIRATION	55.491	201433.438	
ACTUAL EVAPOTRANSPIRATION	30.508	110745.664	45.32
DRAINAGE COLLECTED FROM LAYER 3	35.7456	129756.508	53.10
PERC./LEAKAGE THROUGH LAYER 4	0.000933	3.385	0.00
AVG. HEAD ON TOP OF LAYER 4	0.0009		

DRAINAGE COLLECTED FROM LAYER 8	0.0001	0.196	0.00
PERC./LEAKAGE THROUGH LAYER 9	0.000879	3.189	0.00
AVG. HEAD ON TOP OF LAYER 9	0.0000		
DRAINAGE COLLECTED FROM LAYER 10	0.0009	3.182	0.00
PERC./LEAKAGE THROUGH LAYER 11	0.000002	0.008	0.00
AVG. HEAD ON TOP OF LAYER 11	0.0000		
PERC./LEAKAGE THROUGH LAYER 12	0.000002	0.008	0.00
CHANGE IN WATER STORAGE	1.018	3694.030	1.51
SOIL WATER AT START OF YEAR	302.839	1099306.375	
SOIL WATER AT END OF YEAR	303.857	1103000.375	
INTERCEPTION WATER AT START OF YEAR	0.000	0.000	
INTERCEPTION WATER AT END OF YEAR	0.000	0.000	
SNOW WATER AT START OF YEAR	0.000	0.000	0.00
SNOW WATER AT END OF YEAR	0.000	0.000	0.00
ANNUAL WATER BUDGET BALANCE	0.0000	-0.027	0.00

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ANNUAL TOTALS FOR YEAR 27

	INCHES	CU. FEET	PERCENT
PRECIPITATION	61.15	221974.500	100.00
RUNOFF	0.147	532.527	0.24
POTENTIAL EVAPOTRANSPIRATION	55.851	202738.734	
ACTUAL EVAPOTRANSPIRATION	31.180	113184.617	50.99
DRAINAGE COLLECTED FROM LAYER 3	31.2430	113411.938	51.09
PERC./LEAKAGE THROUGH LAYER 4	0.000813	2.950	0.00
AVG. HEAD ON TOP OF LAYER 4	0.0008		
DRAINAGE COLLECTED FROM LAYER 8	0.0000	0.174	0.00
PERC./LEAKAGE THROUGH LAYER 9	0.000765	2.777	0.00
AVG. HEAD ON TOP OF LAYER 9	0.0000		

DRAINAGE COLLECTED FROM LAYER 10	0.0008	2.767	0.00
PERC./LEAKAGE THROUGH LAYER 11	0.000002	0.009	0.00
AVG. HEAD ON TOP OF LAYER 11	0.0000		
PERC./LEAKAGE THROUGH LAYER 12	0.000002	0.009	0.00
CHANGE IN WATER STORAGE	-1.421	-5157.529	-2.32
SOIL WATER AT START OF YEAR	303.857	1103000.375	
SOIL WATER AT END OF YEAR	302.436	1097842.875	
INTERCEPTION WATER AT START OF YEAR	0.000	0.000	
INTERCEPTION WATER AT END OF YEAR	0.000	0.000	
SNOW WATER AT START OF YEAR	0.000	0.000	0.00
SNOW WATER AT END OF YEAR	0.000	0.000	0.00
ANNUAL WATER BUDGET BALANCE	0.0000	-0.012	0.00

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ANNUAL TOTALS FOR YEAR 28

	INCHES	CU. FEET	PERCENT
PRECIPITATION	65.06	236167.797	100.00
RUNOFF	0.119	430.995	0.18
POTENTIAL EVAPOTRANSPIRATION	53.479	194130.438	
ACTUAL EVAPOTRANSPIRATION	32.151	116708.148	49.42
DRAINAGE COLLECTED FROM LAYER 3	32.3459	117415.602	49.72
PERC./LEAKAGE THROUGH LAYER 4	0.000837	3.038	0.00
AVG. HEAD ON TOP OF LAYER 4	0.0008		
DRAINAGE COLLECTED FROM LAYER 8	0.0000	0.178	0.00
PERC./LEAKAGE THROUGH LAYER 9	0.000788	2.860	0.00
AVG. HEAD ON TOP OF LAYER 9	0.0000		
DRAINAGE COLLECTED FROM LAYER 10	0.0008	2.850	0.00
PERC./LEAKAGE THROUGH LAYER 11	0.000002	0.009	0.00
AVG. HEAD ON TOP OF LAYER 11	0.0000		
PERC./LEAKAGE THROUGH LAYER 12	0.000002	0.009	0.00

CHANGE IN WATER STORAGE	0.444	1610.059	0.68
SOIL WATER AT START OF YEAR	302.436	1097842.875	
SOIL WATER AT END OF YEAR	302.880	1099452.875	
INTERCEPTION WATER AT START OF YEAR	0.000	0.000	
INTERCEPTION WATER AT END OF YEAR	0.000	0.000	
SNOW WATER AT START OF YEAR	0.000	0.000	0.00
SNOW WATER AT END OF YEAR	0.000	0.000	0.00
ANNUAL WATER BUDGET BALANCE	0.0000	-0.051	0.00

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ANNUAL TOTALS FOR YEAR 29

	INCHES	CU. FEET	PERCENT
PRECIPITATION	62.48	226802.375	100.00
RUNOFF	0.040	143.841	0.06
POTENTIAL EVAPOTRANSPIRATION	54.992	199620.500	
ACTUAL EVAPOTRANSPIRATION	27.419	99529.539	43.88
DRAINAGE COLLECTED FROM LAYER 3	35.6216	129306.312	57.01
PERC./LEAKAGE THROUGH LAYER 4	0.000934	3.391	0.00
AVG. HEAD ON TOP OF LAYER 4	0.0009		
DRAINAGE COLLECTED FROM LAYER 8	0.0001	0.197	0.00
PERC./LEAKAGE THROUGH LAYER 9	0.000880	3.194	0.00
AVG. HEAD ON TOP OF LAYER 9	0.0000		
DRAINAGE COLLECTED FROM LAYER 10	0.0009	3.186	0.00
PERC./LEAKAGE THROUGH LAYER 11	0.000002	0.009	0.00
AVG. HEAD ON TOP OF LAYER 11	0.0000		
PERC./LEAKAGE THROUGH LAYER 12	0.000002	0.009	0.00
CHANGE IN WATER STORAGE	-0.601	-2180.681	-0.96
SOIL WATER AT START OF YEAR	302.880	1099452.875	
SOIL WATER AT END OF YEAR	302.279	1097272.125	

INTERCEPTION WATER AT START OF YEAR	0.000	0.000	
INTERCEPTION WATER AT END OF YEAR	0.000	0.000	
SNOW WATER AT START OF YEAR	0.000	0.000	0.00
SNOW WATER AT END OF YEAR	0.000	0.000	0.00
ANNUAL WATER BUDGET BALANCE	0.0000	-0.032	0.00

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ANNUAL TOTALS FOR YEAR 30

	INCHES	CU. FEET	PERCENT
PRECIPITATION	61.55	223426.484	100.00
RUNOFF	0.012	44.161	0.02
POTENTIAL EVAPOTRANSPIRATION	53.835	195422.859	
ACTUAL EVAPOTRANSPIRATION	30.369	110238.500	49.34
DRAINAGE COLLECTED FROM LAYER 3	30.8817	112100.648	50.17
PERC./LEAKAGE THROUGH LAYER 4	0.000849	3.082	0.00
AVG. HEAD ON TOP OF LAYER 4	0.0008		
DRAINAGE COLLECTED FROM LAYER 8	0.0000	0.166	0.00
PERC./LEAKAGE THROUGH LAYER 9	0.000803	2.915	0.00
AVG. HEAD ON TOP OF LAYER 9	0.0000		
DRAINAGE COLLECTED FROM LAYER 10	0.0008	2.907	0.00
PERC./LEAKAGE THROUGH LAYER 11	0.000002	0.009	0.00
AVG. HEAD ON TOP OF LAYER 11	0.0000		
PERC./LEAKAGE THROUGH LAYER 12	0.000002	0.009	0.00
CHANGE IN WATER STORAGE	0.287	1040.087	0.47
SOIL WATER AT START OF YEAR	302.279	1097272.125	
SOIL WATER AT END OF YEAR	302.557	1098280.375	
INTERCEPTION WATER AT START OF YEAR	0.000	0.000	
INTERCEPTION WATER AT END OF YEAR	0.009	31.889	
SNOW WATER AT START OF YEAR	0.000	0.000	0.00
SNOW WATER AT END OF YEAR	0.000	0.000	0.00

ANNUAL WATER BUDGET BALANCE	0.0000	0.008	0.00

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FINAL WATER STORAGE AT END OF YEAR 30

LAYER	(INCHES)	(VOL/VOL)
1	1.5067	0.2511
2	4.8644	0.2702
3	0.0031	0.0107
4	0.0000	0.0000
5	2.6640	0.2220
6	287.3280	0.2920
7	3.1440	0.1310
8	0.0030	0.0100
9	0.0000	0.0000
10	0.0030	0.0100
11	0.0000	0.0000
12	0.1867	0.7470
TOTAL WATER IN LAYERS	299.703	
SNOW WATER	0.000	
INTERCEPTION WATER	0.009	
TOTAL FINAL WATER	299.712	

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PEAK DAILY VALUES FOR YEARS 1 THROUGH 30

	(INCHES)	(CU. FT.)
PRECIPITATION	4.69	17024.701
RUNOFF	0.176	639.6636

Final Cover

DRAINAGE COLLECTED FROM LAYER 3	2.49640	9061.93848
PERCOLATION/LEAKAGE THROUGH LAYER 4	0.000031	0.11172
AVERAGE HEAD ON TOP OF LAYER 4	0.027	
MAXIMUM HEAD ON TOP OF LAYER 4	0.091	
LOCATION OF MAXIMUM HEAD IN LAYER 3 (DISTANCE FROM DRAIN)	0.0 FEET	
DRAINAGE COLLECTED FROM LAYER 8	0.00000	0.01472
PERCOLATION/LEAKAGE THROUGH LAYER 9	0.000027	0.09700
AVERAGE HEAD ON TOP OF LAYER 9	0.000	
MAXIMUM HEAD ON TOP OF LAYER 9	0.007	
LOCATION OF MAXIMUM HEAD IN LAYER 8 (DISTANCE FROM DRAIN)	0.0 FEET	
DRAINAGE COLLECTED FROM LAYER 10	0.00002	0.08551
PERCOLATION/LEAKAGE THROUGH LAYER 11	0.000000	0.00002
AVERAGE HEAD ON TOP OF LAYER 11	0.000	
MAXIMUM HEAD ON TOP OF LAYER 11	0.007	
LOCATION OF MAXIMUM HEAD IN LAYER 10 (DISTANCE FROM DRAIN)	0.0 FEET	
PERCOLATION/LEAKAGE THROUGH LAYER 12	0.000000	0.00002
SNOW WATER	0.91	3304.4434
MAXIMUM VEG. SOIL WATER (VOL/VOL)	0.3657	
MINIMUM VEG. SOIL WATER (VOL/VOL)	0.1040	

*** Maximum heads are computed using McEnroe's equations. ***

Reference: Maximum Saturated Depth over Landfill Liner
by Bruce M. McEnroe, University of Kansas
ASCE Journal of Environmental Engineering
Vol. 119, No. 2, March 1993, pp. 262-270.

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AVERAGE MONTHLY VALUES IN INCHES FOR YEARS 1 THROUGH 30

	JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
PRECIPITATION						

TOTALS	4.34	4.81	5.62	4.25	4.20	6.59
	9.00	7.43	7.41	3.08	2.95	5.00
STD. DEVIATIONS	2.11	2.39	2.68	3.05	3.00	2.48
	3.04	2.65	3.03	2.35	2.17	2.74

RUNOFF

TOTALS	0.004	0.001	0.001	0.001	0.002	0.008
	0.005	0.003	0.017	0.005	0.004	0.001
STD. DEVIATIONS	0.018	0.006	0.007	0.004	0.008	0.033
	0.025	0.015	0.036	0.025	0.015	0.003

POTENTIAL EVAPOTRANSPIRATION

TOTALS	2.417	2.706	4.163	5.384	6.702	7.082
	6.587	6.153	5.024	3.848	2.634	2.047
STD. DEVIATIONS	0.151	0.180	0.264	0.294	0.292	0.328
	0.413	0.311	0.324	0.273	0.172	0.182

ACTUAL EVAPOTRANSPIRATION

TOTALS	1.315	1.696	2.803	2.411	2.790	4.004
	5.157	4.292	3.427	1.414	0.870	0.977
STD. DEVIATIONS	0.233	0.285	0.677	1.169	1.188	1.069
	0.837	0.881	0.751	0.614	0.269	0.196

LATERAL DRAINAGE COLLECTED FROM LAYER 3

TOTALS	3.3888	3.3167	2.8915	2.1904	1.7618	2.1094
	3.4925	3.2606	3.9640	1.8865	1.7991	3.4596
STD. DEVIATIONS	2.1844	2.3119	1.8349	1.8103	1.9016	1.6893
	2.1196	2.2272	2.2859	1.7753	1.7259	2.3775

PERCOLATION/LEAKAGE THROUGH LAYER 4

TOTALS	0.0001	0.0001	0.0001	0.0001	0.0000	0.0001
	0.0001	0.0001	0.0001	0.0001	0.0001	0.0001
STD. DEVIATIONS	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000

LATERAL DRAINAGE COLLECTED FROM LAYER 8

TOTALS	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
STD. DEVIATIONS	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000

PERCOLATION/LEAKAGE THROUGH LAYER 9

TOTALS	0.0001	0.0001	0.0001	0.0001	0.0000	0.0001
	0.0001	0.0001	0.0001	0.0001	0.0000	0.0001
STD. DEVIATIONS	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000

LATERAL DRAINAGE COLLECTED FROM LAYER 10

TOTALS	0.0001	0.0001	0.0001	0.0001	0.0000	0.0001
	0.0001	0.0001	0.0001	0.0001	0.0000	0.0001
STD. DEVIATIONS	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000

PERCOLATION/LEAKAGE THROUGH LAYER 11

TOTALS	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
STD. DEVIATIONS	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000

PERCOLATION/LEAKAGE THROUGH LAYER 12

TOTALS	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
STD. DEVIATIONS	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000

AVERAGES OF MONTHLY AVERAGED DAILY HEADS (INCHES)

DAILY AVERAGE HEAD ON TOP OF LAYER 4

AVERAGES	0.0010	0.0011	0.0009	0.0007	0.0005	0.0006
	0.0010	0.0010	0.0012	0.0006	0.0006	0.0010
STD. DEVIATIONS	0.0007	0.0008	0.0005	0.0006	0.0006	0.0005
	0.0006	0.0007	0.0007	0.0006	0.0006	0.0007

DAILY AVERAGE HEAD ON TOP OF LAYER 9

AVERAGES	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
STD. DEVIATIONS	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000

DAILY AVERAGE HEAD ON TOP OF LAYER 11

AVERAGES	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
STD. DEVIATIONS	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000

AVERAGE ANNUAL TOTALS & (STD. DEVIATIONS) FOR YEARS 1 THROUGH 30

	INCHES	CU. FEET	PERCENT

	Final Cover			
PRECIPITATION	64.69	(7.987)	234821.1	100.00
RUNOFF	0.052	(0.0677)	189.92	0.081
POTENTIAL EVAPOTRANSPIRATION	54.747	(0.9852)	198732.92	
ACTUAL EVAPOTRANSPIRATION	31.155	(2.6409)	113094.21	48.162
LATERAL DRAINAGE COLLECTED FROM LAYER 3	33.52088	(5.97521)	121680.789	51.81851
PERCOLATION/LEAKAGE THROUGH LAYER 4	0.00088	(0.00012)	3.200	0.00136
AVERAGE HEAD ON TOP OF LAYER 4	0.001	(0.000)		
LATERAL DRAINAGE COLLECTED FROM LAYER 8	0.00005	(0.00001)	0.185	0.00008
PERCOLATION/LEAKAGE THROUGH LAYER 9	0.00083	(0.00011)	3.015	0.00128
AVERAGE HEAD ON TOP OF LAYER 9	0.000	(0.000)		
LATERAL DRAINAGE COLLECTED FROM LAYER 10	0.00083	(0.00011)	3.006	0.00128
PERCOLATION/LEAKAGE THROUGH LAYER 11	0.00000	(0.00000)	0.009	0.00000
AVERAGE HEAD ON TOP OF LAYER 11	0.000	(0.000)		
PERCOLATION/LEAKAGE THROUGH LAYER 12	0.00000	(0.00000)	0.009	0.00000
CHANGE IN WATER STORAGE	-0.041	(0.9946)	-147.05	-0.063

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Attachment D-5

Final Cover Hydraulic Conductivity Determination

HDR Engineering, Inc.

Job No.				No.	
Project	Steelfield Landfill Expansion	Computed	K. Perera	Date	2/4/2019
Task	Hydraulic Conductivity and Drainage Layer Capacity	Checked	TY	Date	2/27/2019

Problem

Determine the hydraulic conductivity and drainage capacity of the geocomposite drainage layer in the final cover sideslope for Scenario 4 of HELP model.

Transmissivity Calculations

Final cover liner from bottom to top: 40-mil LLDPE textured - Double-sided Geocomposite - 1.5 Ft Erosion Control Layer- 6-inch top soil

Determine 100-hour transmissivity (Θ_{100}).

Use GSE 300-mil GSE FabriNet or equivalent (see attached cut sheet with product specifications).

$$\Theta_{allow} = \frac{\Theta_{100}}{RF_{CR} * RF_{CC} * RF_{BC}}$$

$$\Theta_{allow} = \Theta_{req} \cdot FS_D$$

$$\Theta_{req} = k \cdot b$$

where,

- Θ_{allow} = minimum allowable transmissivity of geocomposite (cm^2/s)
- Θ_{req} = required transmissivity for site (cm^2/s)
- FS_D = overall factor of safety for drainage
- Θ_{100} = transmissivity after 100 hrs. under expected load (cm^2/s)
- RF_{CR} = reduction factor for creep deformation
- RF_{CC} = reduction factor for chemical clogging
- RF_{BC} = reduction factor for biological clogging
- k = saturated hydraulic conductivity (cm/s)
- b = thickness of geonet (cm)

Information based on product data for GSE FabriNet TRx 300mil. Site specific testing is recommended before selecting the material for installation to confirm minimum transmissivity.

Parameters	Geocomposite (300-mil) @ 1,000 psf (1)
Slope	33.00
FS_D (2)	1.5
RF_{CR} (2)	1.02
RF_{CC} (2)	1.1
RF_{BC} (2)	1.4
Θ_{100} (m^2/s)	0.0015
Θ_{allow} (cm^2/s)	9.55
Θ_{req} (cm^2/s)	6.37
Manufactured Thickness (mils)	300
Manufactured Thickness (cm)	0.76
Design Hydraulic Conductivity (cm/s)	8.35
Retained Final Thickness (mils)	285.0

Notes:

1. psf = pounds per square foot
2. Reduction factors based on HDR experience and information provided by GSE.

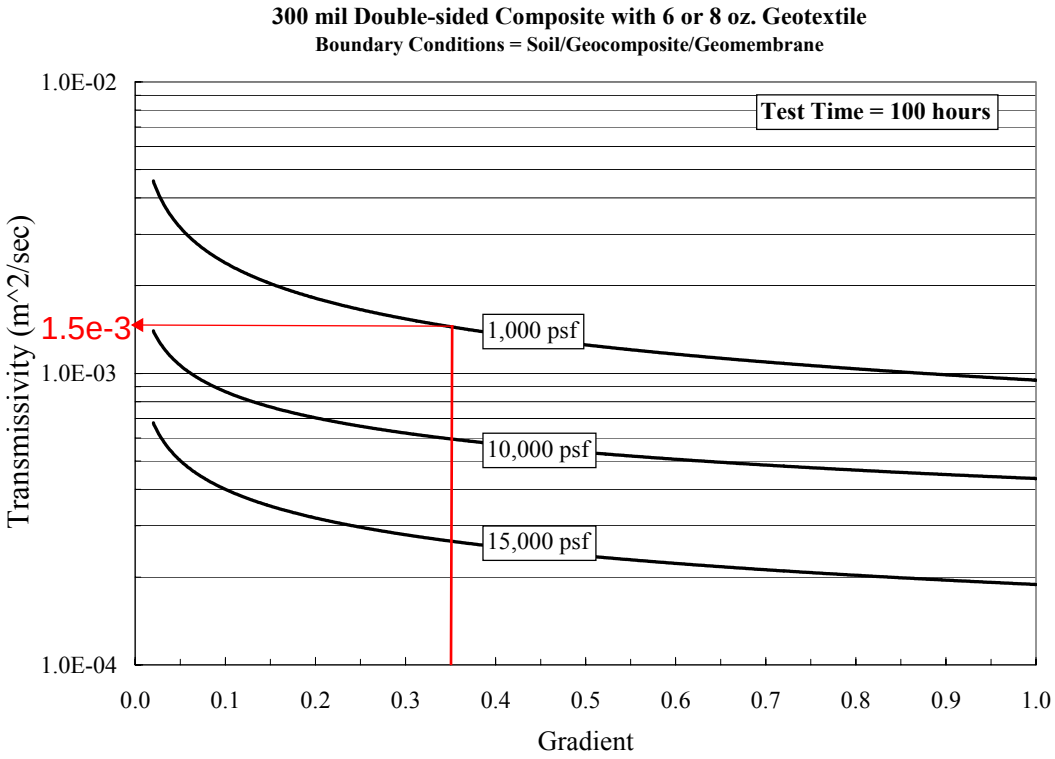

Figure A-9 100-hour transmissivity of 300 mil biplanar geonet geocomposite under soil/geocomposite/geomembrane boundary conditions.

APPENDIX E
TECHNICAL SPECIFICATIONS

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01050 FIELD ENGINEERING (SURVEYS)
01200 DESCRIPTION FOR SCHEDULE OF VALUE ITEMS

DIVISION 2 – SITE CONSTRUCTION

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02225 TRENCHING
02300 EARTHWORK
02310 DEWATERING
02775 HDPE GEOMEMBRANE
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DIVISION 15 – MECHANICAL

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15160 FLOW METER AND PRESSURE GAUGES
15200 PIPE AND PIPE FITTINGS
15300 VALVES

DIVISION 1
GENERAL REQUIREMENTS

SECTION 01050
FIELD ENGINEERING (SURVEYS)

PART 1 GENERAL

1.01 SCOPE OF WORK

A. OWNER shall provide surveys to establish control points that, in OWNER'S judgment, are necessary to enable DBT to proceed with the Work. The DBT shall establish the location of all the Work from control points that are shown on the Drawings, or as modified by the DBE.

B. The DBT shall provide record drawings, sealed and signed by a Florida Registered Land Surveyor, who shall verify that all portions of the work are located vertically and horizontally as shown on the design drawings. The surveying and associated work shall be performed in accordance with this Specification.

C. The DBT shall have the responsibility to carefully preserve the benchmarks, reference points and stakes and, in the case of destruction thereof, DBT shall replace at his own cost all such damaged markers.

D. Existing or new control points, property markers and monuments that are destroyed during the normal course of construction shall be reestablished by the DBT at his expense, and all reference ties shall be recorded and submitted to the OWNER. All computations necessary to establish the exact position of the work shall be made and preserved by the DBT.

E. The OWNER may check all or any portion of the work, and the DBT shall afford all necessary assistance to the OWNER in carrying out such checks. Any necessary corrections to the work shall be immediately made by the DBT. Such checking by the OWNER shall not relieve the DBT of any responsibilities for the accuracy or completeness of his work.

F. All surveying and associated work shall be performed in accordance with this Specification.

1.02 RELATED WORK

A. The General and Supplementary Conditions of these specifications are a part of this section as if incorporated herein.

B. Other related specification sections contained herein are as listed below.

1. Section 01200 – Description of Schedule of Values
2. Section 02300 – Earthwork

3. Section 02775 - HDPE Liner
4. Section 02276 – Geosynthetic Clay Liner
5. Section 15200 - Pipe and Pipe Fittings

1.03 SUBMITTALS

A. See Part 1, Paragraph 1.01 B Surveyor name, address, telephone number, and Florida State Registration Number.

B. See Part 3, Paragraph 3.01 C: preconstruction survey.

C. See Part 3, Paragraph 3.01 for a complete listing.

PART 2 PRODUCTS

(Not Applicable)

PART 3 EXECUTION

3.01 LAYOUT AND LEVELS

A. The DBT shall layout the features and structures as shown on the Drawings, and shall be responsible for any damage caused the OWNER due to incorrect laying out of the work.

B. The Surveyor shall locate and clearly mark in the field all property boundaries within 100 feet of the work area. The Surveyor shall submit to OWNER a description of the evidence and procedures used to locate the property boundary or boundaries. The Surveyor shall immediately notify OWNER of any discrepancy between the boundary location shown on the Drawings and the location indicated by the evidence and procedures used by the Surveyor.

C. The DBT shall, prior to start of construction, perform a preconstruction survey in order to verify all grades, lines, levels and dimensions as shown on the drawings, and shall report any errors or inconsistencies in them to the OWNER in writing within seven (7) days before commencing of the Work. Failure to notify OWNER prior to commencing of Work will be construed as DBT's acceptance of the survey data presented in the Construction Drawings.

D. The DBT shall employ a Florida Registered Land Surveyor to layout the Work, establish bench marks and control points.

E. The Surveyor shall maintain lines and levels, layout, and locate the work utilizing recognized engineering survey practices. A complete and accurate log of control and survey work must be maintained. The Surveyor shall establish a 50 foot by 50 foot survey grid or grade breaks whichever is closer for construction and record documentation. All

survey control points shown on the Drawings shall be included in the Surveyor's documentation. Within seven (7) days after completion of each of the following construction item, the DBT shall prepare and deliver to the OWNER three (3) copies to review that all dimensions, elevation grades, slopes and contours [two foot (2') contour interval or closer] were constructed in accordance with the Contract Documents:

a. A preconstruction survey of the area within the proposed development including storm water control structures, future MSW landfill cell, and at least 100 feet in all directions beyond the limits of disturbance prior to commencing site construction.

b. All items that DBT requires to complete its work and establish pay quantities.

c. All reasonable items requested by OWNER to verify that DBT's work meets the requirements of the Drawings and Specifications apart from the items described below.

1. The location of the anchor trenches and dimensions;

2. Tie-in to existing liner system;

3. Top of prepared subgrade.

4. The surface and limits of primary and secondary base liner geomembrane, including locations of seam samples, repairs, and edge of liner;

5. Top of protective cover including leachate collection piping;

6. Waste limit markers;

7. Location of electrical system components including control panel and buried conduits;

9. Locations and invert elevations related to gravity leachate conveyance pipes, manholes, pump stations, leachate storage tanks, forcemains, structures, including sumps and cleanouts. Note that invert elevations shall be obtained at tie-in locations, intersections and end points and every 50 feet between the intersections and end points;

10. Locations and invert elevations related to drainage channels, underground culverts, inlets, grates and manholes; and

11. Top and toe of all berms, roads and channels.

d. Topographic surveys shall extend a minimum of 100 ft in all directions beyond the limits of construction.

These drawings shall include the horizontal and vertical location and record data of pipes, inverts, slopes, penetrations, and other pertinent features as shown on the Drawings. DBT submitted drawings shall also include the location of the electrical line and outlets, access roads, channels and stockpiles and any revisions made to any of the "details" included in the Contract Drawings.

F. The survey instruments used shall be capable of reading to a precision of 0.01 ft and with a setting accuracy of 20 seconds.

G. Topographic surveys shall be referenced to the site grid system and shall show a minimum of three grid points.

H. The OWNER will review the record documentation within 48 hours of receipt from the Surveyor. The OWNER will notify DBT within this 48 hour period as to whether the record documentation for the subject component demonstrates construction to the specified grades, elevations, etc. DBT shall not proceed with construction of subsequent components until it has received notification from the OWNER. Any work on subsequent components performed prior to this notification may be subject to removal and replacement, at no additional cost to OWNER.

I. A minimum of five sets, or otherwise requested by OWNER, of final certified copies of the record documentation shall be prepared within 14 days after final review and approval by the OWNER. DBT shall also submit electronic files of all record documents to the Owner along with the hard copies.

3.02 TOLERANCES

A. Construction tolerances shall be as given in the various sections of the Specifications and Drawings, and as given below.

B. Subbase construction survey tolerance shall be 0.10 feet.

C. Protective cover layer minimum thickness shall be 24 inches with a construction survey tolerance of plus 0.10 feet.

D. On piping for leachate collection/detection lines, stormwater conveyance pipes/culverts and inlet structures the maximum tolerance shall be 0.02 feet. This tolerance must be set to the record elevation of the surface below it and not the design elevation.

E. For cleanout risers, the tolerance shall be 0.10 foot. This tolerance must be set to the record elevation of the surface below it and not the design elevation.

F. No surveying will be allowed between one hour before sunset and one hour after sunrise, unless approved by OWNER.

G. DBT shall be aware of the surveying activities and shall account for them in the construction schedule.

H. The Surveyor shall cooperate fully with OWNER at all times.

I. Proposed modifications to stormwater management system shall meet the invert elevations and minimum slopes defined on the Contract Drawings.

J. DBT shall meet the minimum slopes for the leachate collection/detection pipe trench, stormwater conveyance pipes, top of subbase and top of protective layer shown in the Contract Drawings and Specifications. The DBT's compliance with these slopes shall be calculated using elevation points that are measured by the Surveyor to the 0.01 feet between every 50 feet. If the surveyed elevations are not in-line with the flow path, interpolated elevations shall be used.

K. Unless otherwise specified on the Drawings, completed slopes shall be no steeper than 33%.

END OF SECTION

SECTION 01200
DESCRIPTION FOR SCHEDULE OF VALUE ITEMS

PART 1 GENERAL

1.01 DESCRIPTION

A completed, quality project, as intended by the general nature of the drawings and specifications, shall be produced whether or not any particular wording or direction is inadvertently omitted. Pay items listed on the Bid Form are for comparison of bids and may be used as a method of determining the value of work performed for partial payment requests.

1.02 LUMP SUM QUANTITIES

DBT shall be solely responsible for determining the quantities of each lump sum pay item necessary to complete the Work as required by the Contract Documents. When lump sum items are broken into components, the sum of the components shall be the total Contract Price for the Work. The lump sum price stated on the Bid Form shall constitute full compensation for each pay item completed in accordance with the drawings and specifications. No other payments will be made to the DBT except as specifically authorized by change order.

1.03 LUMP SUM SCHEDULE

The descriptions provided in the following paragraphs are to be used by the Bidder in preparation of his bid proposal. They generally indicate how the major workscope items and their respective costs are to be separated into the line items listed in the Bid Form. These descriptions are neither fully representative nor all inclusive of the work required for completing the project in accordance with the Contract Documents. It is the Bidder's responsibility to include costs within the most appropriate line item(s) of the Lump Sum Schedule. The following descriptions are ordered in the same numeric sequence of the Lump Sum Schedule.

Item 1 – Design, Permitting and Preparation of Construction Documents
This item shall include review of OWNER supplied design information documents and fill in the gaps/missing data needed to obtain permit, take responsibility for the construction documents, prepare responses to additional information requested by FDEP, prepare construction level drawings and specifications, electrical design, structural design of concrete components, sign and seal permit application, submit permit application fees, participate meetings, assist DBC during construction, provide construction supervision and clarify the construction documents as needed. This task does not allow for design changes for value engineering after the award of the contract.

Item 2 – Mobilization, Demobilization and General Conditions
This item shall include and cover the costs for performance of construction preparatory and overhead operations, including, but not limited to, movement of personnel and equipment to and from the site, temporary facilities, sanitary facilities, project administration and management, dust control,

insurance, bonds, OWNER and OWNER's representative indemnification, temporary utilities, site health and safety, environmental protection, permits related to construction and all other similar activities and facilities necessary for execution of this project. This item shall not exceed 7% of the total contract amount. DBT will be paid up to 75% of this item upon completion of mobilization and general conditions with the remainder paid upon demobilization.

Item 3 – Erosion and Sediment Control

This Work shall consist of furnishing all equipment, materials, labor, and appurtenances required for sedimentation and erosion control at the site as designed by DBE, compliance with applicable laws and regulations, and adhering to standards of practice. This Work includes but is not limited to furnishing, installing, and maintaining silt fence, temporary or intermediate sodding and mulching, temporary stormwater channels and diversions, sediment traps, final sodding of completed berms, channels and side slopes, temporary check dams, temporary construction entrances, maintenance of the stormwater swales during groundwater dewatering and construction and supply, install and maintain temporary stormwater pumps at sump locations to pump out stormwater during entire construction period. The lump sum price proposed shall be full compensation for designing and performing sedimentation and erosion control work required. Payment for this bid item will be on a lump sum basis based on the value of materials furnished or services and work completed using estimates provided by DBT and approved by OWNER's representative.

Item 4 – Surveying

This work shall include furnishing of all labor, materials, tools, supervision, transportation, and equipment necessary to perform project surveying work as specified in the CQA Plan and Section 01050 of these Specifications including establishing survey control points, providing survey control during construction, preconstruction survey, partial and final as-built drawings, and obtaining all survey information required to compile Record (As-Built) Drawings as required by the Contract Documents. Unless OWNER has directed in writing that surveying shall vary from that indicated on the Construction Drawings and these Specifications, the payment quantity shall not exceed the price in the Bid Schedule. The lump sum price proposed shall be full compensation for performing the surveying work required. Payment for this item shall be made on a lump sum basis, wherein no measurement will be made. Payments for this lump sum item will be made based on the DBT submitted schedule and approved by ENGINEER for sub-survey items. Full payment for each milestone will be made upon full completion of the item. Surveys shall only be submitted by DBT after previous surveys have been approved by OWNER's representative.

Item 5 – Site Preparation and Demolition

This Work shall consist of furnishing all equipment, materials, labor, and appurtenances required for demolition and/or relocation of existing structures within the project limits, construction of landfill and debris management site access roads, construction of perimeter access road, construction of relocated citizens recycle drop off center, relocated overhead electrical network, demolish existing leachate storage tank and secondary containment area, abandon cleanouts, remove irrigation system, design and construction of sub-liner gas management system, gravel, paving, removal of wetwells and manholes as shown and all other work associated with site preparation activities as required by the Contract Documents, applicable laws and regulations, and standards of practice. Payment for this

bid item will be on a lump sum basis based on the value of materials furnished or services and work completed using estimates provided by DBT and approved by OWNER's representative.

Item 6 – Subsurface Dewatering within Project Limits

This Work shall consist of furnishing all equipment, materials, labor, and appurtenances required for subsurface dewatering within the project limits to facilitate construction activities as required by the Contract Documents, applicable laws and regulations, and standards of practice. This Work includes but is not limited to preparation of proposed dewatering plan, furnishing, installing, maintaining and monitoring dewatering equipment and conveyance system during the construction period. The lump sum price proposed shall be full compensation for performing all relevant requirements to achieve successful dewatering to proceed with excavation/backfilling to proposed grades and compaction. Payment for this bid item will be on a lump sum basis based on the value of materials furnished or services and work completed using estimates provided by DBT and approved by OWNER's representative.

Item 7 – Installation of Leachate Conveyance System

This Work shall consist of furnishing all equipment, materials, labor, appurtenances and installations required for proposed extension of leachate conveyance system as required by the Contract Documents. This Work includes but is not limited to excavation of trenches, management of oncoming leachate, supply and installation of HDPE and PVC pipes at inverts shown on the drawings, dual containment pipe, gravel bedding, design and installation of pump stations and manholes, supply and installation of pumps, control panels and design and installation of electrical system. The lump sum price proposed shall be full compensation for performing all relevant requirements associated with extension of leachate conveyance system including excavation per Trench Safety Act. Payment for this bid item will be on a lump sum basis based on the value of DBT and approved by OWNER's representative.

Item 8 – Supply and Installation of Leachate Tanks and Piping

This Work shall consist of furnishing all equipment, materials, labor, appurtenances and installations required for design and install above ground leachate storage tanks and truck loading station as required by the Contract Documents. This Work includes but is not limited to prepare construction drawings, foundation investigation, design concrete slab and secondary containment area, supply and install tanks, pipes, valves, float switches, control panels and pumps for truck loading station. Payment for this bid item will be on a lump sum basis based on the value of DBT and approved by OWNER's representative.

Item 9 – Installation of Stormwater Conveyance System

This Work shall consist of furnishing all equipment, materials, labor, appurtenances and installations required for proposed stormwater conveyance system as required by the Contract Documents. This Work includes but is not limited to construction of perimeter channels, supply and installation of pipe culverts at inverts shown on the drawings and establish the road, gravel bedding and end walls.

The lump sum price proposed shall be full compensation for performing all relevant requirements associated with construction of leachate conveyance system. Payment for this bid item will be on a lump sum basis based on the value of materials furnished or services and work completed using estimates provided by DBT and approved by OWNER's representative.

Item 10 – Subgrade

This item shall include furnishing all necessary materials, equipment, transportation, labor, and appurtenances necessary to prepare subgrade of Cells 6 and 7 to the lines and grades shown on the plans. The Work includes, but is not limited to, removal of vegetation, removal of unsuitable on-site soil, cutting and re-locating of existing soil, backfilling as required with on-site structural fill, compaction, fine grading to achieve design elevations, verification of subgrade competency, and disposal of unsuitable soil at an OWNER-designated location and vegetation, excavation and transport Class I waste to active landfill area, remove and dispose geomembrane cover liner within Cells 1 and 2. Conformance testing will be performed by the CQA firm.

Item 11 – Low Permeability Soil and Anchor Trenches

This item shall include furnishing all necessary materials, equipment, transportation, labor, and appurtenances necessary to prepare the anchor trench corridor. The Work includes off-site low permeability soil hauling, backfill, excavation of anchor trenches, soil placement, compaction, and finish grading to achieve design grades. This item also includes supply and installation of low-permeability soil required in the anchor trench detail and in accordance with the earthwork specifications for the anchor trench installation. Prequalification testing of offsite material shall be performed by the DBT. Conformance testing will be performed by the CQA firm.

Item 12 –GCL

This item shall include all equipment, materials, labor and appurtenance necessary to install the GCL where shown on the Drawings and in accordance with the technical specifications and CQA Plan. This Work will include but not be limited to all material delivery, stockpiling, protection, placement, connections, tie-in to existing liner, boots and connections required for liner penetrations, anchor trenches, necessary overlaps and testing involving GCL. No payment will be made for GCL placed outside of the anchor trench, replacing hydrated GCL or where testing has indicated that the installed material does not meet the specified requirements without prior approval of the OWNER.

Item 13 – Secondary HDPE Geomembrane

This item shall include all equipment, materials, labor and appurtenance necessary to install the secondary HDPE where shown on the Drawings and in accordance with the technical specifications and CQA Plan. This Work will include but not be limited to all material delivery, stockpiling, protection, placement, connections, cleaning, securing, tie-in to existing liner, necessary overlaps, boots and connections required for liner penetrations, anchor trenches, and testing involving HDPE secondary geomembrane. No payment will be made for HDPE geomembrane placed outside of the anchor trench, or where testing has indicated that the installed material does not meet the specified requirements without prior approval of the OWNER's representative.

Item 14 – Leak Detection Geocomposite

This item shall include all equipment, materials, labor and appurtenance necessary to install the leak detection geocomposite where shown on the Drawings and in accordance with the technical specifications and CQA Plan. This Work will include but not be limited to all material delivery, stockpiling, protection, placement, connections, cleaning, securing, protecting, existing liner tie-in, boots and connections required for liner penetrations, anchor trenches, and testing involving leak detection geocomposite. No payment will be made for geocomposite placed outside of the anchor trench, or where testing has indicated that the installed material does not meet the specified requirements without prior approval of the OWNER'S representative.

Item 15 – Primary HDPE Geomembrane

This item shall include all equipment, materials, labor and appurtenance necessary to install the primary HDPE where shown on the Drawings and in accordance with the technical specifications and CQA Plan. This Work will include but not be limited to all material delivery, stockpiling, protection, placement, connections, cleaning, securing, existing liner tie-in, boots and connections required for liner penetrations, anchor trenches, and testing involving HDPE primary geomembrane. No payment will be made for HDPE placed outside of the anchor trench, or where testing has indicated that the installed material does not meet the specified requirements without prior approval of the OWNER's representative.

Item 16 – Leachate Collection Geocomposite

This item shall include all equipment, materials, labor and appurtenance necessary to install the leachate collection geocomposite where shown on the Drawings and in accordance with the technical specifications and CQA Plan. This Work will include but not be limited to all material delivery, stockpiling, protection, placement, connections, cleaning, securing, existing liner tie-in, boots and connections required for liner penetrations, anchor trenches, and testing involving leachate collection geocomposite. No payment will be made for geocomposite placed outside of the construction baseline shown on the Drawings, or where testing has indicated that the installed material does not meet the specified requirements without prior approval of the OWNER's representative.

Item 17 –Protective Cover Layer

This item is to include, but not be limited to, all necessary materials, equipment, transportation, and labor necessary to prepare and install protective cover layer above the leachate collection geocomposite. The Work will include but not be limited to hauling from on-site borrow pits as determined by the OWNER and installation of the protective cover layer where shown on the Drawings and in accordance with the Specifications and CQA Plan. No payment will be made for protective drainage layer placed outside of the construction baseline shown on the Drawings or where testing has indicated that the installed material does not meet the specified requirements or required depth without prior approval by the OWNER's representative. Costs for QA/QC testing shall be included in this item. Conformance testing will be performed by the CQA firm.

Item 18 –Leachate Collection/Detection Swale

This item is to include, but not be limited to, all costs associated with supply and installation of the

leachate collection/leak detection swale including perforated HDPE piping, pipe fittings, pipe welding, gravel intra-cell berm, geonet, gravel and geotextile. Costs for HDPE piping, fittings, gravel, geotextile, cleanouts, testing shall be included in this item.

Item 19 –Leachate Collection and Leak Detection Sump

The lump sum price for this item includes all costs required for supply and installation of the leachate collection and detection sump. The work shall include, but not limited to, perforated and solid HDPE sideslope risers, blind flanges, piping and connections to proposed leachate forcemain, leachate forcemain, aggregate, and HDPE boot to the leak detection sideslope riser pipes. Costs for QA/QC testing shall be included in this item. This work shall include but is not limited to supply and installation of pumps, control panel, pipes, pipe supports, valves, grading, concrete pads, meters, pressure gauges, trenching, access points, bollards, mounting of control panel and appurtenances.

Item 20 – Electrical System Relocation and Supply Electricity

The lump sum price for this item includes all work required to construct the control and electrical system. The work shall include, but not be limited to design electrical system, pump controls, pump connections, meter connection, wiring from the power poles to control panel, running electrical conduit, below grade pull box installation, relocation of existing electrical network and coordination with the power company. All necessary certifications, demonstrations, QA/QC and production of record documents shall be included with this item.

Item 21 – Rain Liner

This item shall include all equipment, materials, labor and appurtenance necessary to install the rain liner where shown on the Drawings and in accordance with the technical specifications. This Work will include but not be limited to all material delivery, stockpiling, protection, placement, connections, cleaning, securing, boots and connections required for liner penetrations, anchor trenches, and testing involving rain liner. No payment will be made for rain liner placed outside of the limits shown, or where testing has indicated that the installed material does not meet the specified requirements without prior approval of the OWNER's representative.

END OF SECTION

DIVISION 2
SITE CONSTRUCTION

SECTION 02221

REINFORCED LANDFILL COVER

Rain Liner (RL): The Rain Liner shall be placed on top of protective soil layer within the footprint shown in the drawing to prevent infiltration of stormwater into the leachate collection system.

PART 1 - GENERAL

1.1 SUMMARY

The DBT shall furnish all labor, material, and equipment to install RL including all necessary and incidental items as detailed or required to complete the installation in accordance with the Contract Drawing and these Specifications

1.2 RELATED WORK

Related Contract Work is described in the following section of the specification.

1.3 REFERENCE STANDARDS

The latest revision of the following standard of the Geosynthetic Research Institute (GRI) are hereby made a part of these specifications

GRI GM22 Standard Specification for Test Methods, Required Properties and Testing Frequencies for Scrim Reinforced Polyethylene Geomembranes Used in Exposed Temporary Applications

1.4 QUALITY ASSURANCE

Quality Assurance during installation of RL will be provided by the CQA Consultant.

1.5 MANUFACTURERS QUALIFICATIONS

a. The Manufacturer shall have previously demonstrated his ability to produce the required RL by having successfully manufactured a minimum of 20,000,000 ft^2 of string reinforced RL.

b. Manufacturer must be ISO 9001 certified

1.6 INSTALLER QUALIFICATIONS

The RL Installer shall have installed a minimum of 500,000 ft2 of RL (or similar material) in similar landfill installations.

1.7 WARRANTIES

Manufacturer of the RL shall warrant the material to the installer on a pro rata basis for five years after the final acceptance of the Work under exposed Florida weather conditions. This warranty shall include but not be limited to defects related to workmanship and manufacturing.

PART 2 - MATERIALS

2.1 GENERAL

The materials supplied under these Specifications shall consist of first-quality products designed and manufactured specifically for the purpose of this work, which shall have been satisfactorily demonstrated, by prior use, to be suitable and durable for such purposes.

The RL shall be supplied in panels which shall be of maximum size to provide the largest manageable sheet for the fewest seams (45,000 ft^2 typical). Labels on the panels shall identify the thickness, length, width, lot and panel numbers, and name of Manufacturer.

2.2 RL MATERIALS

a. RL shall be manufactured to meet the following requirements:

(1) Provide finished product free from holes, pin holes, bubbles, blisters, excessive gels, undispersed resins and/or carbon black, or contamination by foreign matter.

(2) RL shall be a geomembrane composed of a four layer reinforced extrusion laminate, with the outer black layers being a high-strength polyethylene film, with a minimum average thickness of 4 mils; which are laminated together using molten polyethylene incorporating a 1000 denier scrim. RL shall have the physical properties as shown in Table 1 of this section. Properties of RL shall meet or exceed the requirements of GRI GM22 (Category 2 - Moderate - 12 mil nominal) or shall be an approved RL as listed below.

TABLE 1: REQUIRED RL PROPERTIES

PROPERTY	TEST METHOD	UNITS	VALUE
Thickness (Nominal)	ASTM D 5199	mils	12
Weight	ASTM D 751	lbs	58
Grab Tensile Strength	ASTM D 7004	lbs	100
Elongation at Break	ASTM D 7003	%	600
Tongue Tear	ASTM D 5884	lbf	40

(3) RL shall be impermeable, capable of repelling water with no absorption.

b. Approved RL:

(1) Dura-Skrim 12BV as manufactured by Raven Industries of Sioux Falls, SD.

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(2) Equal material, as approved by the OWNER.

2.3 SEAMING MATERIALS

Thread for sewing the RL seams shall be black polyester, or as approved by the Engineer.

2.4 BALLAST SYSTEM

The sandbags shall be from the same material as the (parent) RL. The DBT shall place ballast on top of the RL as need to prevent uplift from 90 miles/hour wind event.

PART 3 - SUBMITTALS

The DBT shall submit the following to the OWNER:

3.1 PRE-INSTALLATION REQUIREMENTS

Prior to RL installation the DBT shall submit the following:

a. Certificate of Conformance and Sample: Prior to shipping to the site, the DBT shall submit a certificate or affidavit signed by a legally authorized official of the Manufacturer for the RL attesting that the RL meets the physical and manufacturing requirements stated in these Specifications. The DBT shall also submit a sample of the RL to be used. The sample shall be labeled with the product name and be accompanied by the Manufacturer's specifications.

b. Shipping, Handling, and Storage Instructions: The Manufacturer's plan for shipping, handling, and storage shall be submitted for review.

c. Installation Procedures:

Submit installation procedures for carrying out the work. Installation procedures to be addressed shall include but not be limited to material installation, repair, and protection to be provided in the event of rain or strong winds. With regard to protection, the DBT shall provide a plan of anchoring the RL sufficient to satisfy the DBT's Performance Warranty. This plan shall be approved by the OWNER prior to construction.

d. Furnish copies of the delivery tickets or other approved receipts as evidence for materials received that will be incorporated into the construction.

3.2 POST-INSTALLATION REQUIREMENTS

Upon completion of the RL installation, the DBT shall submit the following:

a. Completed material performance warranty.

PART 4 - EXECUTION

4.1 SHIPPING, HANDLING, AND STORAGE

The RL shall be shipped, handled, and stored in strict accordance with the Manufacturer's recommendations.

4.2 SUBGRADE PREPARATION

The surface of the subgrade shall be approved top of protective cover as determined by the CQA Consultant.

4.3 RL PLACEMENT

a. Weather Conditions:

RC placement shall not proceed at an ambient temperature below 32° F or above 100° F unless otherwise authorized, in writing, by the Engineer. RL placement shall not be performed during precipitation, excessive moisture, in an area of ponded water, or in excessive winds. Any portion of RL or subgrade damaged due to weather conditions shall be repaired at the DBT's cost.

b. Method of Placement:

(1) Each panel of the RL shall be installed in accordance with the Manufacturer's recommendations.

(2) The CQA Consultant shall inspect each panel, after placement and prior to seaming, for damage and/or defects. Defective or damaged panels shall be replaced or repaired, as approved by the CQA Engineer and as described in this section.

(3) The Installer shall avoid dragging the RL on rough soil subgrades

(4) Personnel working on the RL shall not smoke, wear damaging shoes, or involve themselves in any activity that may damage the RL, in the opinion of the CQA Engineer.

(5) The RL shall be properly weighted to avoid uplift due to wind.

(6) Vehicular traffic across the RL shall not be allowed.

(7) The RL shall be kept free of debris, unnecessary tools, and materials. In general, the RL area shall remain neat in appearance.

(8) The RL shall be weighted with filled sandbags, each weighing a minimum of 30 pounds.

(9) The sandbags shall be spaced as needed by DBE to prevent designed wind uplift.

(10) Rope securing the sandbags shall be as approved by the CQA Consultant.

(11) Along its perimeter, the RL shall be anchored in a backfilled and compacted anchor trench.

c. Pipe Penetrations:

Pipe penetrations through the RL shall be as approved by the CQA Consultant.

4.4 FIELD SEAMS

a. Field seams shall be made according to the Manufacturer's recommendations and may include sewing, fusion welding, or other methods as appropriate for the selected material and as approved by the Engineer.

b. Individual panels of RL shall be laid out and overlapped by a minimum of 4 inches prior to seaming. The area to be seamed shall be cleaned and prepared in accordance with the Manufacturer's recommendations.

4.5 REPAIR PROCEDURES

Any portion of the RL exhibiting signs of defect shall be repaired. Several procedures exist for the repair of these areas. The final decision as to the appropriate repair procedure shall be made by the CQA Consultant.

4.6 BALLASTING

The DBT shall add ballasting in accordance with the approved plan.

No overlying material shall be placed over the RL until approved by the CQA Consultant.

END OF SECTION

SECTION 02225
TRENCHING

PART 1 GENERAL

1.01 SCOPE OF WORK

A. DBT shall furnish all labor, materials, tools, supervision, transportation, and installation equipment to perform all trenching work as specified herein and as shown on the Drawings.

B. DBT shall be prepared to construct all trenches in conjunction with other aspects of the Work.

C. Work of this section includes, but is not necessarily limited to, trenching for subsurface drainage, stormwater pipe/culvert installation, leachate transmission pipe installation, anchor trenches and installation of backfill material.

1.02 RELATED SECTIONS

A. Section 01200 – Description of Schedule of Values
B. Section 01050 - Surveying
C. Section 02300 - Earthwork

1.03 SUBMITTALS

A. DBT shall notify OWNER in writing a minimum of 7 days prior to starting trenching activities. The notice shall state the material to be used, shoring, the equipment to be used, the date and time that placement operations will start, methods taken to avoid interruptions to daily landfill traffic and the name of the person in the field who will be in charge of the trenching activities.

B. DBT shall submit to OWNER the proposed layout for each trench 7 days prior to the start of trenching.

C. If work is interrupted for reasons other than inclement weather, DBT shall notify OWNER a minimum of 24 hours prior to the resumption of work.

1.04 CONSTRUCTION QUALITY ASSURANCE

A. DBT shall be aware of the activities outlined in the CQA Plan and shall account for these activities in the construction schedule.

B. Trenching activities will be visually monitored by the CQA Consultant.

1.05 CONSTRUCTION LAYOUT

DBT shall be responsible for all construction layouts.

PART 2 PRODUCTS

2.01 EQUIPMENT

A. DBT shall furnish, operate, and maintain equipment as is necessary to produce uniform trenches.

B. Dewatering equipment is required when excavating below groundwater table.

PART 3 EXECUTION

3.01 FAMILIARIZATION

A. Contours of existing ground elevations shown on the Drawings are believed to be reasonably correct for the date that the topographic survey was obtained. The DBT is advised that grades depicted on the Drawings may vary from field conditions at the time of construction and that such variations will not be a justification for additional costs to OWNER.

B. DBT shall maintain, abandon, relocate and/or protect existing utilities that may pass through the work area without interrupting to daily landfill operations.

C. DBT shall protect benchmarks and existing structures from excavation equipment and vehicular traffic.

D. DBT shall protect above and below grade utilities that are to remain.

E. DBT shall allow uninterrupted traffic on the paved hauling road north and east of proposed leachate transmission pipe trenches and assure paved road stability during excavation activities.

3.02 TRENCHING

A. DBT shall take all necessary safety precautions during construction activities and shall conform to all applicable OSHA regulations, OWNER's safety requirements, and DBT's health and safety plan.

B. Prior to starting trenching operations, DBT shall remove topsoil found suitable by OWNER for reuse. Such material shall be removed in a manner to separate it clearly from underlying material and shall be stored on site where directed by OWNER. Topsoil shall remain the property of OWNER.

C. Trenches shall be constructed to the dimensions and alignments shown in approved trench layout plan. Alignments shall be adjusted by DBT, if necessary and based on CQA Consultant's approval, to achieve the specified slope and spacing requirements.

D. Any excavated soil not suitable for reuse shall be loaded and hauled to the operating portion of the landfill by DBT for disposal by OWNER.

E. The minimum anchor trench dimensions shall always be maintained. Anchor trenches shall be compacted with 2 passes using a walk-behind vibrating plate compactor (refer to Section 02300).

G. DBT shall minimize open overnight trenches.

H. When performing trench excavation in excess of 5 feet in depth, comply with the Occupational Safety and Health Administration's (OSHA) trench safety standards, 29 CFR 1926, Subpart P, and all subsequent revisions or updates adopted by the Department of Labor and Employment Security. Ensure that trench boxes are wide enough to accommodate compaction and density testing.

3.03 LIQUIDS AND WATER

A. Groundwater will be encountered during trenching operations. DBT shall meet Section 02310 requirements before begin trench excavations.

B. DBT shall take every precaution to prevent water from entering an open trench. Should water enter the trench, the water shall be removed so as to return the trench bottom to a firm, dry condition.

3.04 FIELD QUALITY ASSURANCE

The CQA Consultant will visually observe trenching operations, dimensions, dewatering and monitor compaction.

3.05 TOLERANCE

Anchor trenches shall be constructed to within the tolerances in Section 01050.

3.06 SAFETY PROVISIONS

Trenching shall be performed in strict accordance with OSHA and all other applicable laws and regulations, as well as DBT's health and safety plan for this project. Job site safety is the sole responsibility of DBT.

END OF SECTION

SECTION 02300
EARTHWORK

PART 1 - GENERAL

1.1 SUMMARY

This section covers the work necessary to perform earthwork associated with construction of the landfill liner system and site work. The work includes, but is not limited to, earthwork, excavating and removal of unsatisfactory soils and replacement with backfill material, clearing and grubbing, trench excavation, providing satisfactory material for fill and backfill as required, and soil placement and compaction.

1.2 RELATED WORK

A. The General and Supplementary Conditions of these specifications are a part of this section as if incorporated herein.

B. Other related specification sections contained herein are as listed below:
   1. Section 02775 – HDPE Geomembrane
   2. Section 02777 – Drainage Geocomposite
   3. Section 02776 – Geosynthetic Clay Liner
   4. Construction Quality Assurance Plan.

1.3 DEFINITIONS

A. "Relative compaction" is defined as the ratio, in percent, of the as-compacted field dry soil density to the laboratory maximum dry density as determined by the Standard Proctor Method, ASTM D 698. Corrections for oversize material may be applied to either the as-compacted field dry density or the maximum dry density, as determined by the DBE.

B. "Optimum moisture content" is defined as the moisture content corresponding to the maximum dry density obtained by the Standard Proctor test, ASTM D 698. Field moisture content shall be determined on the basis of the fraction passing the No. 4 sieve.

C. "Completed course" is defined as a course or layer that is complete and ready for testing and/or the next layer or phase of construction.

D. "Clearing" shall consist of the felling, trimming, and cutting of trees into sections and the satisfactory disposal of the trees and other vegetation designated for removal, including down timber, snags, brush, and rubbish occurring in the areas to be cleared.

E. "Grubbing" shall consist of the removal and disposal of stumps, roots, and matted roots from the designated grubbing areas.

1.4 REFERENCE STANDARDS

A. Reference standards and recommended practices referred to herein shall be the latest revision of any such document.

B. Standards referenced herein are as listed below:
   1. ASTM D 422 Particle Size - Analysis of Soils
   2. ASTM D 698 Moisture - Density Relations of Soils and Soil Aggregate Mixtures, Using a 5.5 lb Rammer and 12-Inch Drop
   3. ASTM D 1140 Amount of Material in Soils Finer than the No. 200 Sieve
   4. ASTM D 2216 Laboratory Determination of Water (Moisture) Content of Soil, Rock, and Soil Aggregate Mixtures
   5. ASTM D 2487 Classification of Soils for Engineering Purposes (Unified Soil Classification System)
   6. ASTM D 2937 Density of Soil In-Place by the Drive Cylinder Method
   7. ASTM D 3017 Moisture Content of Soil and Soil - Aggregate in Place by Nuclear Methods (Shallow Depth)
   8. ASTM D 4318 Liquid Limit, Plastic Limit, and Plasticity Index of Soils
   9. ASTM D 5084 Hydraulic Conductivity of Saturated Porous Materials Using a Flexible Wall Permeameter
   10. ASTM D 6938 Standard Test Method for In-Place Density and Water Content of Soil and Soil-Aggregate by Nuclear Methods (Shallow Depth)
   11. EPA 9100 Saturated Hydraulic Conductivity, Saturated Leachate Conductivity, and Intrinsic Permeability

C. Florida Department of Transportation (FDOT) Standard Specifications for Road and Bridge Construction (Standard Specifications).

1.5 QUALITY ASSURANCE, ACCEPTANCE, AND TOLERANCES

A. Prior to commencing any excavation or grading, the DBT shall satisfy himself as to the accuracy of all survey data as indicated on the drawings and in the specifications and/or as provided by the OWNER. Should the DBT discover any inaccuracies, errors, or omissions in the survey data, he shall immediately notify the OWNER that proper adjustments can be anticipated or ordered. Commencement by the DBT of any excavation or grading shall be held as an acceptance of the survey data by him after which time the DBT has no claim against the OWNER resulting from alleged errors, omissions, or inaccuracies of the survey data.

B. All grading shall be performed to strictly maintain slopes and drainage as shown on the drawings. Allowable tolerances are provided in Section 01050.

1.6 SUBMITTALS

Submittals shall be in sufficient detail to show full compliance with the specifications.

A. Manufacturer's catalog data sheets for equipment scheduled for use in proof-rolling the subgrade and for operating over geosynthetic layers shall be submitted.

B. Certificates of compliance for proposed materials shall be submitted in accordance with at least 10 days before the material is required for use. Certificates of Compliance for all sources of soil materials shall clearly reference the source as described in paragraph 1.6F of this section.

C. SAMPLING AND TESTING

1. Tests required for location of an acceptable material source shall be done by an independent soils testing agency which shall be approved by the OWNER and retained by the DBT as illustrated in Table 1. Certification for all materials indicating that the material conforms to the specification requirements along with copies of the test results from a qualified independent testing laboratory, approved by the OWNER and retained by the DBT, shall be submitted to the OWNER for approval at least 10 days before the material is required for use. The DBT shall coordinate material location and delivery with the testing agency and the OWNER. No materials shall be delivered to the site or used in construction until the proposed source and materials tests have been tentatively accepted in writing by the OWNER. The DBT shall notify the OWNER, in writing, a minimum of 24 hours prior to the time materials approval is required.
2. Tests required for in-place final acceptance shall be performed by an independent testing agency as retained by the CQA Consultant. The number and location of the tests shall be as specified herein and as directed by the CQA Consultant throughout the project time period. The DBT shall coordinate activity with the CQA Consultant and the testing agency to permit testing as directed in the presence of the CQA Consultant. The cost of any and all retests due to failure to achieve specified requirements are not included in the cost indicated in the Contract Documents, shall be invoiced to the DBT separately, and shall be borne by the DBT. All testing agency activities shall be performed under the direction and supervision of a Professional Engineer, licensed in the state of Florida.
3. Test results shall be sealed and signed by the testing agency representative who shall be a Professional Engineer, licensed in the state of Florida. Copies of test results shall be transmitted to the CQA Consultant for review and approval. The reports shall include, as a minimum, project title; project location; location of sample, time, and date of testing or sampling; and test results.
4. The DBT shall provide certification that proposed material is clean and meets gradation and other parameters herein specified. The certification tests shall be performed at a minimum frequency of one per borrow source and/or change in material, except as noted per Table 1.

Table 1 Prequalification Testing

Item	Test	ASTM No.
Structural Fill (Liner subgrade)	Soil Classification (USCS Classification System)	D2487
	Sieve and Hydrometer Analysis	D 422
	Atterberg Limits	D 4318
	Natural Moisture Content	D 2216
	Standard Proctor	D698
Protective cover layer (Sand Drainage Layer) (See Note 1)	Soil Classification (USCS Classification System)	D2487
	Sieve and Hydrometer Analysis	D 422
	Atterberg Limits	D 4318
	Natural Moisture Content	D 2216
	Standard Proctor	D 698
	Permeability of Granular Soils	D2434
	Calcium Carbonate	D3042
Low Permeability Soil	Soil Classification (USCS Classification System)	D 2487
	Sieve and Hydrometer Analysis	D 422
	Atterberg Limits	D 4318
	Standard Proctor	D 698
	Natural Moisture Content	D 2216
	Triaxial Permeability	D5084
Leachate collection gravel	Soil Classification (USCS Classification System)	D 448
	Permeability of Granular Soils	D2434
	Resistance to Degradation (LA Abrasion)	C535
	Calcium Carbonate	D3042
Topsoil	Soil Classification	D 2487
	Organic Content	D 2974
	pH	D 4972

5. The CQA Consultant shall provide conformance tests during construction as specified in the CQA Plan (Refer to Tables 7 and 8).
6. Classification tests on all materials as specified herein shall be made on samples of each material at its place of production prior to shipment. Classification tests in excess of those listed above may be required on the finished product if variation in gradation is apparent or if the material appears to depart from the specifications. Additional testing shall be as directed by the OWNER.
7. The soil material source will only be considered suitable if the hydraulic conductivity of the materials, as documented on laboratory test specimens, can be shown to meet the requirements of the project specifications at the 98% confidence level.

8. In-place density and moisture content shall be performed by the approved independent soil testing lab using the methods specified herein, or other methods selected by the CQA Consultant. The DBT shall cooperate with this testing work by leveling small test areas designated by the CQA Consultant.
9. Final acceptance will be based on tests made on samples of material taken from the completed and compacted course. Acceptance will be made only on completed courses.
10. The OWNER and the CQA Consultant reserve the right to perform additional tests at the OWNER's expense of any material at any time, location, or elevation.
11. Any areas which do not meet compaction requirements shall be reworked, as necessary, to achieve the specified compaction and retested, at the OWNER's expense, to confirm that compaction requirements have been met.

PART 2 - PRODUCTS

2.1 PROTECTIVE COVER SOIL (SAND DRAINAGE LAYER)

A. Sand shall be obtained from on-site sources approved by the OWNER and shall be clean, excavated, natural sand, free from roots, sharp objects, or organic material, with a maximum size of 1/4 inch and 5% maximum by weight passing the No. 200 Sieve. The sand shall be classified as SW or SP by the Unified Soil Classification System. The sand drainage layer shall have a laboratory permeability not less than 1×10^{-3} cm/sec at 90% relative compaction (ASTM D 698). The material shall not be calcareous or of any other material that would interact with leachate. The sand shall be less than 10 percent loss of weight when tested in accordance with ASTM D3042 (Calcium Carbonate).

2.2 LEACHATE COLLECTION GRAVEL

A. Gravel fill shall be imported from off-site sources, washed granite, AASHTO #3 as approved by OWNER, free from roots, organic material. Leachate collection gravel will have a hydraulic conductivity of 1×10^{-1} cm/s (1×10^{-3} m/s) or greater. Material shall not be limerock or other material that would react and/or break down when in contact with solid waste leachate. The material shall not be calcareous or of any other material that would interact with leachate. The aggregate shall be less than 10 percent loss of weight when tested in accordance with ASTM D3042 (Calcium Carbonate).

B. Aggregate shall be of sound, hard and durable quality. Aggregate shall have less than 15 percent loss when tested in accordance with ASTM C535. It shall not contain any soapstone, shale, or other material that easily disintegrates.

2.3 STRUCTURAL FILL AND SUBGRADE

A. Structural fill will be used for the subgrade, perimeter berms and channels and inter-cell berm as shown on the Drawings. The structural fill will consist of on-site relatively homogenous natural soils that are free of debris, foreign objects, organics and other deleterious materials. Structural fill shall be classified according to ASTM D 2487 as SP, GW, GM, GC, SW, SM, SC, ML, CL or their combinations and will have no particles or soil clods larger than 4 in. (100 mm) except for the top 6 inches of the subgrade. The maximum particle size within top 6-inches of subgrade shall not exceed 0.5 in. (12 mm). Fill material shall be free from alkali, salt, petroleum products, roots, stones, rocks, and building debris. The structural fill shall be placed and compacted in lifts to the lines and grades shown on the Drawings. The compacted thickness of each lift will be 12 in. maximum (300 mm). Each lift will be compacted to at least 95 percent of the standard proctor maximum dry density (ASTM D698) within ±3% of optimum moisture content.

2.4 LOW-PERMEABILITY SOIL

A. The low-permeability soil shall consist of relatively homogeneous, natural soils that are free of debris, foreign objects, excess silt, and organics imported from off-site sources. No particles larger than 1-inch shall be allowed. The soil shall be classified according to the Unified Soil Classification System (USCS, ASTM D 2487) as MH, ML, CL, or CH material. Regardless of the classification requirements, the material shall meet all of the requirements of this section. The selected soil shall not be gap-graded or susceptible to piping. Substandard materials shall be segregated at the source and will not be permitted at the work area. Any material that is found by the OWNER to be substandard shall be removed from the work area by the DBT at no extra cost to the OWNER.

B. The low permeability soil layers shall have a hydraulic conductivity of less than 1.0×10^{-6} cm/s tested in the laboratory in accordance with ASTM D 5084. The material shall be tested at an effective stress of 250 psf. The remolded sample shall be compacted to 95% of standard proctor compaction (ASTM D698).

2.5 TOPSOIL

The topsoil shall be obtained from on-site sources that are capable of promoting the growth of vegetation. The topsoil will be relatively homogenous natural soils that are free of debris, foreign objects, excess silt, clay lumps, brush, roots, weeds, or other deleterious materials. The topsoil will have at least 75 percent by weight passing the U.S. Standard No. 4 (4.75 mm) sieve, and at least 60 percent by weight passing the U.S. Standard No. 10 (2 mm) sieve. For the portion passing the U.S. Standard No. 10 (2 mm) sieve, the material will be classified as sandy loam, loam, or silt loam in accordance with the USDA classification system. In addition, the topsoil will have a pH in the range of 5.8 to 7; a minimum of 7 percent by weight of organic matter; and a maximum of 4 milliohms per centimeter of soluble salts. The material shall comply with the requirements of FDOT's Standard Specifications for Road and Bridge Construction (2004), Section 987 for Topsoil. If necessary, nutrients (including

agricultural lime and fertilizer) shall be added to the topsoil to enhance its ability to promote vegetation growth. The OWNER shall be consulted prior to use of nutrients.

2.6 WATER AND MOISTURE CONTROL EQUIPMENT

A. The DBT shall provide water for compaction as necessary to achieve proper moisture conditions.

B. Equipment for applying water shall be of a type and quality adequate for the work, shall not leak excessively, and shall be equipped with a distributor bar or other approved device to assure uniform application. Equipment for mixing and drying out material shall include blades, discs, or other approved equipment.

PART 3 - EXECUTION

3.1 EXCAVATION SAFETY AND LEGISLATION

A. Protect bench marks and existing structures, roads, sidewalks, monitoring wells, piezometers, paving, and curbs against damage from equipment, vehicular or foot traffic, settlement, lateral movement, undermining, and washout.

B. Install and maintain shoring, sheeting, bracing, and sloping necessary to support the sides of the excavation, to keep and to prevent any movement which may damage adjacent pavements, utilities, or structures; damage or delay the work; or endanger life and health. Install and maintain shoring, sheeting, bracing, and sloping as required by the Occupational Safety and Health Administration (OSHA) and other applicable governmental regulations and agencies.

C. The DBT shall be solely responsible for making all excavations in a safe manner. Excavations shall be barricaded and posted with warning signs for the safety of persons. Warning lights shall be provided during hours of darkness.

D. Excavated materials suitable for backfill shall be piled in an orderly manner sufficiently distant from excavations to prevent overloading, slides, cave-ins, and obstruction of access ways and roadways.

E. Underpin adjacent structure(s) which may be damaged by excavation work, including service lines.

F. Notify OWNER of unexpected subsurface conditions and discontinue work in area until DBE provides notification to resume work.

G. Excavations shall be done in ways that will prevent surface and subsurface water from flowing into excavations and will also prevent flooding of the site and surrounding area.

H. The DBT shall familiarize themselves with, and comply with, all applicable codes, ordinances, and statutes, and bear sole responsibility for the penalties imposed for noncompliance.

I. When performing trench excavation in excess of 5 feet in depth, comply with the Occupational Safety and Health Administration's (OSHA) trench safety standards, 29 CFR 1926, Subpart P, and all subsequent revisions or updates adopted by the Department of Labor and Employment Security. Ensure that trench boxes are wide enough to accommodate compaction and density testing.

3.2 TEMPORARY EROSION CONTROL

A. It is the DBT's responsibility to provide temporary erosion control procedures to protect slopes and other areas from erosion. Measures such as straw bales, temporary slope flumes, sediment traps or other methods shall be used to protect completed work from damage due to erosion. Damage to facilities under construction shall be repaired at the DBT's sole expense.

3.3 REMOVAL OF WATER

A. At all times during construction, DBT shall provide and maintain proper equipment and facilities to remove all water entering the construction area so as to obtain satisfactory working conditions.

B. Avoid settlement or damage to adjacent property. When dewatering open excavations, dewater from outside the structural limits and from a point below the bottom of the excavation when possible.

C. Design dewatering system to prevent removal of fines from existing ground. DBT shall comply with the facility FDEP Permit requirements for all discharges of groundwater to the stormwater conveyance system.

D. The DBT shall be responsible for the control and maintenance of stormwater and dewatering of groundwater within project limits through all phases of construction. Specific groundwater dewatering requirements are illustrated in Section 02310. Under no circumstances shall stormwater be allowed to run into the excavation or pond therein. The DBT shall prevent stormwater runoff from the existing/active landfill cell area entering the proposed work area at all times during the construction and for the duration of the contract. The DBT shall provide temporary stormwater control methods including berms, swales, ponds, and employ other such methods and devices to prevent stormwater runoff from entering and ponding in the proposed cell limits. Any work in the proposed cell limits shall be coordinated with the OWNER 24 hours in advance of when needed. Ponded stormwater of any kind shall be pumped out of the cell and into the stormwater conveyance system within 24 hours of being identified.

3.4 SUBGRADE PREPARATION

A. Heavy growths of grass and other vegetation, roots, debris, stones, objects larger than 2 inches in any dimension, and other materials undesirable to the subsurface construction shall be removed by mowing, grubbing, raking, or other methods from the surface of areas to be stripped.

B. Topsoil Stripping
   1. Topsoil shall be stripped from the surface of those areas to receive fills/excavation or embankments.
   2. Excavated topsoil shall be transported to, and stockpiled in, designated topsoil storage areas on site. Stripped topsoil may be used as topsoil for the Project if approved by the OWNER.

C. Clearing Operations
   1. Clearing shall consist of the felling, trimming, and cutting of trees into sections and the satisfactory disposal of the trees and other vegetation designated for removal, including downed timber, snags, brush, and rubbish occurring within the areas to be cleared. Trees, stumps, roots, brush, and other vegetation in areas to be cleared shall be completely removed except such trees and vegetation as may be indicated or directed to be left standing. Vegetation to be left standing shall be protected from damage incident to clearing, grubbing, and construction operations by the erection of barriers or by such other means as the circumstances require. Clearing shall also include removal and disposal of structures that obtrude, encroach upon, or otherwise obstruct the work.

D. Grubbing Operations
   1. The DBT shall completely remove material to be grubbed, including logs, metallic and concrete debris, boulders, stumps, roots, and organic or other debris protruding through the ground surface.

E. Filling Depressions
   1. Depressions resulting from grubbing operations shall be completely filled with structural fill material, unless further excavation or earthwork is required.
   2. Prior to filling, subgrade surfaces of depressions shall be free of standing water. Unsatisfactory soil materials shall be removed.

F. Subgrade Preparation over Cells 1 and 2
   1. Perform items discussed in Parts 3.4 A and B.
   2. The existing final cover of Cells 1 and 2 require soil/Class 1 waste removal/relocation and compaction to meet designed grades. The operation requires removal of final cover geomembrane liner and disposed at the landfill after cutting into dimensions requested by OWNER.
   3. Once the grading portion is complete, it should be compacted and proof rolled before placing 1-foot structural fill layer

4. A minimum of 1-foot structural fill layer should be placed over waste surface before placing the GCL liner. Placement and compaction are discussed in below sections.

G. The DBT shall excavate/fill the subgrade as required to achieve the subgrade elevations indicated on the drawings. Fill shall be placed not to exceed 6-inch compacted lifts and shall be compacted to at least 95% of the Standard Proctor (ASTM D 698) maximum dry density, within ±3% of optimum moisture content. The subgrade shall be proof-rolled with a vibratory drum roller. The DBT shall provide the equipment and techniques necessary to produce a well-mixed, homogenous layer of the specified thickness, density, moisture content, and permeability. The subgrade lifts shall not pump, rut, or crack under equipment loadings. Any subgrade areas that show evidence of "pumping" shall be remediated by waiting for it to dry, ripping, or discing the soil before re-compacting and testing. Upon completion of proof rolling the subgrade, CQA monitor will perform tests to confirm that the subgrade is compacted to at least 95% of the Standard Proctor (ASTM D 698) maximum dry density, within ±3% of optimum moisture content

H. Upon completion of the compacted subgrade by the DBT, the double liner system shall be placed by DBT in accordance with Section 02776, GCL, Section 02775, HDPE Geomembrane, and Section 02777, Drainage Geocomposite. The DBT shall be responsible for maintaining moisture, density, and integrity of the completed compacted base. Any damage shall be repaired at the DBT's sole expense.

I. Exposure to construction traffic after completion of compacted subgrade placement shall be minimized and shall include only equipment submitted to and approved by the OWNER. The GCL shall not be placed on saturated soil. Saturated subgrade surfaces shall be aerated, dried back to acceptable moisture levels, and re-compacted immediately prior to installing the GCL.

J. The DBT shall sequence operations to allow for systematic completion of the liner system as soon as possible after compacted base completion.

3.5 LEACHATE COLLECTION GRAVEL AT LEACHATE COLLECTION/DETECTION PIPES

A. Leachate drainage aggregate shall be placed around leachate pipes as detailed on the Drawings. Leachate pipes shall be stabilized as necessary to prevent displacement or movement of piping during placement of aggregate backfill. Aggregate shall be placed in loose lifts and the leachate piping shall not be harmed or damaged during placement. Material shall be worked in and shall provide constant contact with and support to the pipe. Gravel shall be covered with geotextile separation layer as specified on the Drawings.

3.6 STRUCTURAL FILL AND LOW-PERMEABILITY SOIL

A. DBT shall place and compact structural fill to the lines and grades shown on the Drawings and as specified in this Section.

B. Structural fill/low permeability soil shall be placed in lifts that results in a compacted lift thickness not more than 12-in. Anchor trenches shall be compacted with 2 passes using a walk-behind vibrating plate compactor.

C. Each lift of structural fill and low permeability soil shall be compacted to at least 95 percent of the maximum dry unit weight determined from the standard Proctor compaction test (ASTM D 698). The soil material shall be compacted at moisture content within plus or minus 3 percent of the optimum moisture content.

D. If the defect is related to adverse site conditions, such as overly wet soils or surface desiccation, the CQA Consultant will define the limits and nature of the defect. If the moisture content of the structural fill is outside of the acceptable range, the soil shall be wetted or dried as appropriate. Wetting shall be accomplished using a water truck and spray nozzle, unless the CQA Consultant approves an alternative method. During wetting or drying, the soil shall be regularly disced or otherwise mixed so that uniform moisture conditions are obtained.

E. DBT shall moisture-condition structural fill in either the stockpile area or work area. Any delays in progress due to moisture conditioning (wetting or drying) of soil, however, are the responsibility of DBT.

F. No structural fill shall be placed over a lift that has not been tested and approved by the CQA Consultant. Should the CQA tests indicate that the dry density of any layer of fill, or portion thereof is below the minimum acceptable value, the particular layer, or portion thereof, shall be reworked and recompacted at no cost to OWNER.

3.7 PROTECTIVE COVER LAYER (SAND DRAINAGE LAYER)

A. Place protective cover layer in one lift with a minimum thickness as specified herein and as shown on the drawings. The DBT shall exercise caution in keeping construction equipment off the geomembrane liner system. Provide sufficient thickness of protective cover layer to maintain the minimum specified thickness and to maintain the surface grades shown. The protective cover layer shall not be compacted or tested for compaction.

B. Track-mounted equipment with low ground pressure treads (less than 6 psi), no larger than a Caterpillar Model D-6 or equal, shall be used for spreading. No tracked equipment shall be allowed to operate on less than 24 inches of protective cover over the geomembrane liner system. No other equipment, including dump trucks or scrapers, will be permitted to travel on the liner and sand layer. The DBT shall avoid sharp turns, sudden starts or stops, spinning and digging of tracks, or any other operation that could damage the lining system. Articulated trucks are permitted to travel over the protective cover layer provided a minimum 4 foot thickness is maintained.

C. The protective cover layer shall be placed in such a manner that no air is trapped underneath the lining. The DBT shall provide and maintain a means of continuously observing the depth of the sand drainage layer, such as by freestanding markers at intervals of 50 feet maximum each way or other means as approved by the OWNER. Sharpened stakes shall not be allowed. Markers shall be removed after use and shall not be abandoned in-place.

D. The DBT shall take precautions necessary to preclude any damage to the liner system due to thermal expansion or contraction during all phases of liner construction and especially during placement of the protective cover layers.

3.8 TOPSOIL AND VEGETATION

A. All unlined areas affected by construction shall be graded and sodded in accordance with the Contract specifications.

B. The DBT shall place topsoil to the lines and grades shown on the Drawings and/or as specified in this Section.

C. The total thickness of the vegetative topsoil shall be a minimum of 6 in.

D. Vegetative topsoil shall not be over compacted so that it inhibits growth of vegetation.

E. DBT shall moisture-condition vegetative topsoil in either the stockpile area or work area. Any delays in progress due to moisture conditioning (wetting or drying) of soil, however, are the responsibility of DBT.

F. No vegetative topsoil shall be placed over a lift that has not been tested and approved by the CQA CONSULTANT.

3.9 DISPOSAL OF EXCESS EXCAVATION

A. Excavated materials shall be stored in area approved by OWNER and disposed at the active working face as directed by the OWNER. The OWNER will not apply a tipping fee for on-site disposal of wastes generated during construction.

3.10 GROUND PRESSURE RESTRICTIONS OVER GEOSYNTHETICS

A. Rubber tired light equipment driven directly on geosynthetics (e.g., GCL, geomembrane, geotextile, geocomposite) shall be preapproved by OWNER.

B. Unless otherwise specified, all equipment operating on earthen materials overlying geosynthetics shall comply with the following:

Allowable Equipment Overlying Pressure (psi)	**Thickness of Compacted Soil (ft)**
<5	1.0
<10	1.5
<20	2.0
>20	3.0

END OF SECTION

SECTION 02310
DEWATERING

PART 1 - GENERAL

1.1 SUMMARY

This section covers the work necessary to dewater the working area as needed.

1.2 QUALITY ASSURANCE

A. Referenced Standards:
   1. All work in this Section shall be performed in accordance with the requirements determined by the Florida Department of Environmental Protection (FDEP).

B. The General and Special Conditions of these specifications are a part of this section as if incorporated herein.

1.3 SUBMITTALS

A. Within 15 days of receiving the notice to proceed, the DBT shall submit to the OWNER a dewatering plan. At a minimum, the plan shall include the following:
   1. Duration of dewatering for each area.
   2. Number and size of pumps.
   3. Method of dewatering each area.
   4. Methods for routing/containing the discharge.
   5. Methods of isolating dewatering areas.
   6. Time dewatering structure will be in place.
   7. Proposed discharge points.
   8. Monitoring requirements

B. Five copies of the plan shall be submitted to the OWNER for record purposes only.

1.4 QUALIFICATIONS

A. At least one person shall be present at all times during the execution of this portion of the work and who shall be thoroughly familiar with the dewatering system being installed, the referenced standards, the requirements of this work, and who shall direct all work performed under this section.

B. It shall be the responsibility of the DBT to determine the water level at the time prior to beginning excavation and construction.

1.5 REFERENCE MATERIAL

A. Design of the dewatering system shall be the sole responsibility of the DBT. The DBT shall be responsible for performing whatever investigations they see necessary, prior to bidding, to design the dewatering system. Inaccuracies in this information shall not be the basis for a claim by the DBT as it is the DBT's responsibility to verify these data with their own investigations.

PART 2 - PRODUCTS (NOT USED)

PART 3 - EXECUTION

3.1 DEWATERING SYSTEM

A. The dewatering system shall be adequate to pre-drain the soils to be excavated to the extent that the piezometric water level in the construction area is a minimum of 3 feet below the bottom of the excavation, side slopes of excavations, or bottom of the footings at all times, or as otherwise required to obtain the specified compaction and installation conditions. Pipeline trenches must be dewatered at least 6 inches below the trench bottom.

B. In the event of layered soils, the hydrostatic head in the zone below the subgrade elevation shall be relieved to prevent uplift.

C. Unless otherwise noted and prior to any excavating below or within 2 feet above the groundwater level, a dewatering system shall be placed into operation to lower water levels to the extent specified previously, and then shall be operated continuously 24 hours per day, 7 days a week, throughout the duration of the contract to maintain and protect all work, until the work has been completed to the satisfaction of the OWNER.

D. Where used, well points shall be installed in an approved manner and in sufficient numbers to provide the necessary removal of water as stated previously. Well points and header piping shall be installed in such a manner that traffic on public thoroughfares and site access roads will not be impeded.

E. The DBT shall be solely responsible for the arrangement, location, and depths of the dewatering system necessary to accomplish the specified work. The dewatering system shall stay in full operation until leachate transmission pipe network and the manholes/pump stations are installed, backfilled and compacted.

F. The DBT shall install observation wells/piezometers as needed to monitor groundwater levels within the excavation footprint. The CQA monitor will review water levels to confirm requirements in this section are met before and during excavation. Upon completion of work, the piezometer casing shall be pulled out of ground and the open hole filled with granular bentonite followed by hydration.

G. To prevent excessive noise, exhaust from all pumps and engines shall be silenced and muffled.

H. Wellpoint or surface water pump discharge shall be controlled to prevent erosion, undermining, and all other damage, and be piped to approved locations.

I. The DBT shall perform all dewatering work in strict compliance with FDEP permit requirements.

J. The DBT shall dewater in such a manner as to prevent damage to existing work. Any damage resulting from the dewatering activities of the DBT shall be repaired or replaced at no additional cost or time to the OWNER.

3.2 OBSERVATION WELLS

A. The DBT shall install observation wells as may be required to record accurate water levels.

B. The DBT shall be responsible for maintaining all observation wells and observing and recording the elevation of the piezometric water levels daily.

C. Wells damaged or destroyed shall be replaced at no additional cost to the OWNER.

3.3 CLEANUP

A. Upon completion of dewatering work elsewhere on the Project, the DBT shall remove all equipment and leave the project site in a neat, clean, and acceptable condition, satisfactory to the OWNER. Wellpoint holes, observation wells, and excavations shall be adequately backfilled bentonite and compacted to prevent settlement. No well points shall be left underneath the geosynthetic liner.

END OF SECTION

SECTION 02775
HDPE GEOMEMBRANE

PART 1 - GENERAL

1.1 SUMMARY

A. Section Includes:
1. Purchasing, delivery, installation, quality control, and testing of a HDPE geomembrane for a landfill bottom liner system.

B. Related Sections include but are not necessarily limited to:
1. Section 02300 – Earthwork
2. Section 02777 – Drainage Composite
3. Section 02776 – Geosynthetic Clay Liner
4. Construction Quality Assurance Plan.

1.2 QUALITY STANDARDS

A. Referenced Standards:
1. ASTM International (ASTM).
a. ASTM D638, Standard Test Method for Tensile Properties of Plastics.
b. ASTM D792, Standard Test Method for Density and Specific Gravity (Relative Density) of Plastics by Displacement.
c. ASTM D1004, Standard Test Method for Initial Tear Resistance of Plastic Film and Sheeting.
d. ASTM D1238 Standard Test Method for Flow Rates of Thermoplastics by Extrusion Plastometer.
e. ASTM D1603 Standard Test Method for Carbon Black in Olefin Plastics.
f. ASTM D3015 Standard Practice for Microscopic Examination of Pigment Dispersion in Plastic Compounds. Refer to Subpart 2.2 for property to be tested.
g. ASTM D3895 Test Method for Oxidative Induction Time of Polyolefins by Thermal Analysis.
h. ASTM D4218 Test Method for Determination of Carbon Black Content in Polyethylene Compounds by the Muffle-Furnace Technique.
i. ASTM D4437 Determining the Integrity of Field Seam Used in Joining Flexible Polymeric Sheets of Geomembrane
j. ASTM D4833 Test Method for Index Puncture Resistance of Geotextiles, Geomembranes, and Related Products.
k. ASTM D5199 Test Method for Measuring Nominal Thickness of Geotextiles and Geomembranes.
l. ASTM D5321, Test Method for Determining the Coefficient of Soil and Geosynthetic or Geosynthetic and Geosynthetic Friction by the Direct Shear Method.
m. ASTM D5397 Procedure to Perform a Single Point Notched Constant Tensile Load – Appendix (SP-NCTL) Test.

n. ASTM D5596 Test Method for Microscopic Evaluation of the Dispersion of Carbon Black in Polyolefin Geosynthetics.
o. ASTM D5721 Practice for Air-Oven Aging of Polyolefin Geomembranes.
p. ASTM D5820, Standard Practice for Pressured Air Channel Evaluation of Dual Seamed Geomembrane.
q. ASTM D5885 Test Method for Oxidative Induction Time of Polyolefin Geosynthetics by High Pressure Differential Scanning Calorimetry.
r. ASTM D5994 Test Method for Measuring the Core Thickness of Textured Geomembranes.
s. ASTM E-96-00, Procedure BW, Standard Test Methods for Water Vapor Transmission of Materials.

2. The Geosynthetic Research Institute (GRI).
a. GRI GM6 Pressurized Air Channel Test for Dual Seam Geomembranes.
b. GRI GM10 Specification for the Stress Crack Resistance of Geomembrane Sheet.
c. GRI GM11 Accelerated Weathering of Geomembranes Using a Fluorescent UVA-Condensation Exposure Device.
d. GRI GM12 Measurement of the Asperity Height of Textured Geomembranes Using a Depth Gauge.
e. GRI GM13 Standard Specification for Test Properties, Testing Frequency, and Recommended Warranty for High Density Polyethylene (HDPE) Smooth and Textured Geomembrane.
f. GRI GM14 Selection Variable Intervals for Taking Geomembrane Destructive Seam Samples Using the Method of Attributes.

B. Qualifications:
1. Each manufacturing and fabricating firm shall demonstrate 5 years continuous experience with a minimum of 10,000,000 SF of HDPE geomembranes.
2. INSTALLER:
a. Demonstrate 5 years continuous experience on at least 20 completed facilities with a minimum 10,000,000 SF of HDPE geomembranes.
b. Trained and certified by at least one of the named manufacturers in this Specification (not necessarily the manufacturer supplying materials for this Project).
c. Geomembrane INSTALLER Personnel Qualifications:
1) Installation Superintendent shall have worked in a similar capacity on at least five HDPE geomembrane liner jobs similar in size and complexity to the project described in the Contract Documents.
2) The Master Welder shall have completed a minimum of 5,000,000 sf of HDPE geomembrane seaming work using the type of seaming apparatus proposed for use on this Project.
3) Other welders shall have seamed a minimum of 1,000,000 sf of HDPE geomembrane.
3. CQA Testing Laboratory shall demonstrate 3 years of continuous experience in geosynthetic materials testing similar to the testing contained herein.

C. Quality Assurance:
   1. The OWNER or OWNER's representative will conduct independent testing to support construction quality assurance program and to provide documentation of such to appropriate regulatory agencies.
   2. Unless specifically superseded by these contract documents or approved plans submitted by the DBT, the geosynthetic materials shall be manufactured, stored, placed, seamed, tested and protected as described in EPA/600/R-93/182 and EPA/530/SW-91/051.
      a. This specifically includes:
         1) Material Composition.
         2) Manufacturing.
         3) Handling and Packaging.
         4) Shipment.
         5) Storage (Manufacturer and Site).
         6) Placement:
            a) Seaming and Joining.
            b) Destructive and Non Destructive Testing.
            c) Protection, Backfilling and Covering.
         7) Conformance Testing.
         8) Anchoring and Anchor Trenches.
         9) Access Roads/Ramps.

D. CQA Plan Implementation: Construction Quality Assurance will be performed in accordance with the Specifications and permitted CQA Plan prepared for this project. The DBT and Geomembrane INSTALLER should familiarize themselves with the CQA Plan.

E. Certifications:
   1. Certifications are required for various aspects of the project related to the HDPE geomembrane liner system construction.
      a. Unless alternately approved, the certificates provided at the end of this Section shall be used and no alterations, additions, deletions, or exception shall be made to the specified language.

F. INSTALLER's construction quality control programs to include, but not be limited to, installation testing, including both nondestructive and destructive quality control field testing of the sheets and seams during installation of the geomembrane, proposed methods of testing geosynthetic joints and connections at appurtenances for continuity, documentation and changes, alterations, repairs, retests, and acceptance.

G. Geomembrane INSTALLER's installation manual to include:
   1. Ambient temperature at which the seams are made
   2. Control of panel lift up by wind
   3. Acceptable condition of the subsurface beneath the geomembrane
   4. Quality and consistency of the welding material
   5. Proper preparation of the liner surfaces to be joined

6. Cleanliness of the seam interface (e.g., the amount of airborne dust and debris present)
7. Proposed details for connecting the HDPE liner to appurtenances, i.e. penetrations of the containment facilities.
8. A complete description of seaming by extrusion welding and hot-wedge welding.
9. Requirements of the MANUFACTURER's Installation Manual unless exceptions are noted.

1.3 DEFINITIONS AND RESPONSIBILITIES

A. Geomembrane MANUFACTURER: MANUFACTURER of geomembranes producing geomembrane sheets from resin and additives. The manufacturer is responsible for producing geomembrane sheet which complies with these Specifications. These responsibilities include but are not limited to:
   1. Acceptance of the resin and additives from chemical formulators. Testing of the raw resin and additives to ensure compliance with the manufacturer's specifications and with this Specification.
   2. Formulation of the resin and additives into geomembrane sheeting using mixing and extrusion equipment.
   3. Testing of the geomembrane sheet to ensure compliance with manufacturer's specification and this Specification.
   4. Shipping of the geomembrane sheet to INSTALLER or OWNER designated facilities.
   5. Certification of the raw materials and finished geomembrane sheet to comply with this Specification.
   6. Certification of installer's training, experience, and methods for welding and inspection of geomembrane installations in compliance with manufacturer's standards.

B. Geomembrane INSTALLER. INSTALLER of geomembranes is responsible for handling, fitting, welding, and testing of geomembrane sheets or blankets in the field. These responsibilities include but are not limited to:
   1. Acceptance (in writing) of the geomembrane from the manufacturer.
   2. Acceptance (in writing) subgrade followed by GCL surface layer will serve as a base for the geomembrane. This acceptance shall precede installation of the geomembrane, and shall state that the installer has inspected the surface, and reviewed the Specifications for material and placement, and finds all conditions acceptable for placement of geomembrane liners. The written acceptance shall explicitly state any and all exceptions to acceptance.
   3. Handling, welding, testing, and repair geomembrane liners in compliance with this Specification and the Geomembrane INSTALLER's Installation Procedures Manual.
   4. Performance of QC testing and record keeping as required by the approved Geomembrane INSTALLER's Field Installation Procedures Manual.
   5. Repair or replacement of defects in the geomembrane as required by the CQA Officer.

C. Inspector:
   1. Inspectors of HDPE geomembrane are the individuals responsible for observing field installation of the geosynthetic materials and providing the MANUFACTURER, INSTALLER, CQA Officer, and OWNER with verbal and written documentation of the compliance of the installation with this specification and with written procedures manuals prepared by the MANUFACTURER or INSTALLER.

D. CQA Testing Laboratory shall:
   1. Perform conformance testing and destructive testing of the HDPE geomembrane.

E. INSTALLER's QC Consultant: Responsible for observing field installation of the geomembrane and performance of material conformance and CQC testing to provide the DBT with verbal and written documentation of the compliance of the installation with these Specifications. The CQC Consultant reports to the DBT and is part of this contract.

F. CQA Officer: Responsible for implementing CQA Plan including reviewing material conformance testing, review destructive testing, field installation of the geomembrane, and CQC activities, and to perform limited CQA conformance testing to provide OWNER with verbal and written documentation of the compliance of the installation with these Specifications. The CQA Officer will use the written results of the CQC program and the CQA program in the preparation of the Project Certification Document. The CQA Officer reports to the OWNER and is not part of this contract.

G. Refer to the accompanying CQA Plan for additional definitions.

1.4 SUBMITTALS

A. Shop Drawings:
   1. At least 14 days prior to installation submit for OWNER's approval Shop Drawings, including:
      a. MANUFACTURER's certification that raw materials and sheet materials comply with required materials, mil thickness, and material properties. Original certificates are required.
      b. MANUFACTURER/Fabricator/INSTALLER quality control requirements.
      c. Qualifications and experience of key personnel per 1.2 B of this section.
      d. MANUFACTURER's written acceptance of Geomembrane INSTALLER's qualifications for installation of the HDPE geomembrane.
      e. HDPE Geomembrane panel layout plan with proposed size, number, position and sequencing of liner panels and showing the location and direction of all field or factory joints.
         1) Proposed details for connecting the geosynthetic materials to appurtenances.
         2) Proposed methods of welding, seaming or jointing geosynthetic materials.
         3) Proposed method and sequencing for placement of geomembrane on top of GCL.

4) Proposed method and sequencing for placement of geocomposite on top of the HDPE geomembrane liner.
5) Proposed method of testing HDPE geomembrane and other geosynthetic materials, joints and connections at appurtenances for continuity.
6) Proposed details for anchor trench if different than included in Contract Documents.

B. Miscellaneous:
1. Test results:
a. Resin test, tests of sheet material and factory seam tests at frequency specified in respective quality control manuals.
1) Results shall include or bracket the rolls delivered for use in the Work.
b. Daily test seam results.
c. Daily results of production seam testing.
d. At least 14 days prior to geomembrane deployment, manufacturing quality control certificates for each shift's production shall be submitted. The certificates shall identify the origin of the resin and the manufacturer of geomembrane. The certificates shall be signed by responsible parties employed by the manufacture (such as the production manager). The quality control certificate shall include:
a. roll numbers and identification;
b. sampling procedures; and
c. results of quality control tests, including a description of the test methods used.
e. At least 14 days prior to shipment of the geomembrane, leachate compatibility test data shall be submitted to OWNER.
f. Warranties as described below.
2. Submit written certifications that:
a. Utilize certification forms from this Section unless alternately approved. Make appropriate number of copies, as required.
b. The HDPE geomembrane material delivered to site meets the requirements of this Specification.
c. The HDPE geomembrane were received and accepted in undamaged condition from shipper.
d. The subgrade has been properly prepared and acceptable for the placement of the HDPE geomembrane.
e. The HDPE geomembrane liner was installed in accordance with this Specification, Project Drawings and with approved Shop Drawings.
f. The HDPE geomembrane joints were inspected, tested for strength and continuity, and passed all inspections and tests.
1) All test and inspection data shall be incorporated into this certification.
g. The geocomposite layer on top of the HDPE geomembrane liner was placed properly and carefully.
3. MANUFACTURER/INSTALLER's Field Installation Procedures Manual shall clearly identify any exceptions taken to the specified execution of the Work.

4. At least 14 days prior to shipment of the geomembrane, recommendations for unloading, field handling, and stockpiling of the geomembrane shall be submitted.
5. During the installation, the Geosynthetics INSTALLER shall be responsible for the timely submission to the CQA Officer of:
   a. quality control documentation;
   b. panel layout with destructive tests, panel boundaries and repairs shown; and
   c. subgrade acceptance certificates, signed by the INSTALLER, for each area to be covered by the geomembrane
5. Record Drawings: Submit reproducible drawings of record showing changes from the approved installation drawings. The record drawings shall include the identity and location of each repair, cap strip, penetration, boot, and sample taken from the installed geosynthetic for testing. The record drawings shall show locations of each type of material, anchor trenches and the construction baseline.

C. Provide all submittals in a single coordinated transmittal. Partial submittals will not be accepted. All submittals must be submitted prior to the Geomembrane Preconstruction Meeting.

1.5 DELIVERY, STORAGE, AND HANDLING

A. Unused or stockpiled HDPE geomembrane shall be stored in accordance with the manufacturer's recommendations.

B. Each roll shall be labeled with the manufacturers name, type, lot number, roll number, and roll dimensions (length, width, gross weight).
   1. HDPE geomembrane or plastic wrapping damaged as a result of storage or handling shall be repaired or replaced, as directed.
   2. HDPE geomembrane shall not be exposed to temperatures in excess of 60 DegC (140 DegF) or less if authorized by the MANUFACTURER.

C. No hooks, tongs or other sharp instruments shall be used for handling the HDPE geomembrane.
   1. Rolls shall not be lifted by use of cables or chains in contact with the HDPE geomembrane.
   2. HDPE geomembrane shall not be dragged along the ground.

1.6 PROJECT CONDITIONS

A. When the weather is of such a nature as to endanger the integrity and quality of the installation, whether this is due to rain, high winds, cold temperatures, or other weather elements, the installation of the geomembrane shall be halted at the direction of, or with the concurrence of, the OWNER until the weather conditions are satisfactory.
B. Ensure that adequate dust control methods are in effect to prevent the unnecessary accumulation of dust and dirt on geosynthetic surfaces which hamper the efficient field seaming of geosynthetic panels or performance.

C. Maintain natural surface water drainage diversions around the work area and provide for the disposal of water which may collect in the work area directly from precipitation falling within the area or from inadequate diversion structures or practices.
D. Coordinate the installation of the leachate collection system which shall be in accordance with Geomembrane INSTALLER's Installation Manual and as specified in these Specifications and shown on the Contract Drawings.
E. Vehicles, other than those specifically approved, will not be allowed on HDPE membrane liner unless at least 24 inch of protective soil cover has been placed over these materials.
    1. No vehicle shall access the completed Work unless it can be demonstrated that its weight, movement or activities will not damage the Work.
    2. When damage is suspected uncover area, repair damage if required, and recover area at no cost to OWNER.
    3. Suspect areas may be identified by OWNER.

1.7 WARRANTIES

A. Written warranties addressing HDPE geomembrane material and installation workmanship shall be furnished by the DBT and shall be made to the OWNER.

B. Submit material samples and warranties prior to shipment.

PART 2 - PRODUCTS

2.1 ACCEPTABLE MANUFACTURERS AND/OR GEOMEMBRANE INSTALLERS

A. Subject to compliance with the Contract Documents, the following manufacturers and installers are acceptable:
    1. HDPE Geomembrane liners manufacturers:
        a. Agru/America, Inc.
        b. Solmax.
        c. Poly-Flex Inc.
    2. HDPE Geomembrane Liner Installers:
        a. Authorized installers of approved manufacturers.
        b. Other installers may qualify by providing references for a minimum of 10,000,000 SF of liner installations.

B. Submit requests for substitution to OWNER for approval.

2.2 Resin

A. Reclaimed polymer shall not be added to the resin; however, the use of polymer recycled during the manufacturing process shall be permitted if approved by OWNER and if the recycled polymer does not exceed 2 percent by weight of the total polymer weight. The geomembrane shall be manufactured from new, first-quality polyethylene resin, and shall be designed and manufactured specifically for use in geomembranes. No additives or fillers may be added to the resin prior to or during manufacture of the HDPE geomembrane.

B. The resin shall comply with the following HDPE specified properties.
   1. Specific Gravity: 0.932 (ASTM D 792 Method A)
   2. Melt Index: 1.0 g/10 minute, maximum (ASTM D 1238 Condition E 190°C, 2.16 kg)

2.3 MATERIALS

A. HDPE Geomembrane Liner:
   1. Consists of unreinforced polyethylene.
   2. Thickness: 60 mils for bottom liner.
   3. No additives or fillers may be added to the resin prior to or during manufacture of the HDPE geomembrane.
   4. Manufactured to be free of holes, blisters, undispersed raw materials, or any sign of contamination by foreign matter.
      a. Any such defects shall be cause for rejection of the material.
      b. Minor defects may be repaired in accordance with Section 3.2.
   5. Manufactured as seamless rolls.
      a. Minimum width: 22 feet as delivered to the site.
   6. Prior to shipment, the geomembrane manufacturer will provide the CQA Officer and the INSTALLER's QC Consultant with a quality control certificate for each roll of geomembrane provided. The quality control certificate will be signed by a responsible party employed by the geomembrane manufacturer and will include:
      a. Roll numbers and identification; and
      b. The results of quality control tests performed under the MQC program.
   7. The INSTALLER's QC Consultant will verify that a control certificate has been received for each roll and that the certified roll properties meet the requirements of these Specifications.
   8. HDPE sheet with **texture** on both sides shall be used for the primary and secondary geomembranes.

B. Manufacturing Quality Assurance (QA): The geomembrane liner shall be manufactured in accordance with a written quality assurance/quality control program (QC). This QA/QC program shall be submitted to the OWNER or CQA Consultant, together with shop drawings showing the layout of geomembrane liner in the containment facility. After this QA/QC program has been approved by the OWNER or CQA Consultant, the MANUFACTURER shall not deviate from the program without written approval of the OWNER or CQA Consultant. The geomembrane liner material shall meet or exceed GRI GM13 and the requirements in Table 1.

Table 1: Geomembrane Property Requirements

Item	ASTM Test Method	Test Value	Min. Test Frequency	
			MQC	CQA
Density (Resin)	D1505	>0.932 g/cc	1/180,000 lb	---
Sheet Thickness (Note 1) • Minimum Average • Lowest Individual 8 of 10 • Lowest Individual 10 of 10	D5994	60 mil nominal - 5% nominal - 10% nominal - 15%	1/50,000 ft^2	1/100,000 ft^2
Asperity Height, Minimum Average	GRI GM12	10 mil	1/45,000 ft^2	---
Sheet Density (min. ave.)	D1505	≥ 0.940 g/cc	1/180,000 lb	1/100,000 ft^2
Melt Flow Index (Resin) (g/10 min.)	D1238 Condition 190/2.16	≤ 1.00	1/200,000 ft^2	---
Minimum Tensile Properties • Yield Stress • Break Stress • Elongation at Yield • Elongation at Break (2-inch gage length)	D638, Type IV, Dumb-bell at 2 imp. (each direction) or ASTM D6693	126 ppi 90 ppi 12% 100%	1/20,000 ft^2	1/100,000 ft^2
Min. Tear Resistance Initiation	D1004, Die C	42 lbs	1/45,000 ft^2	---
Carbon Black Content	D1603 or D4218	2.0-3.0%	1/20,000 ft^2	1/100,000 ft^2
Carbon Black Dispersion	D5596	Category 1 or 2	1/45,000 ft^2	1/100,000 ft^2
Puncture Resistance, Minimum Average	D4833	90 lbs	1/45,000 ft^2	---
Dimensional Stability	D1204	+/- 3.0%	1/lot	---
Oxidative Induction Time, Minimum Average	D3895 or D5885	100 min. 400 min.	1/lot, and 1/200,000 lb	---
Max. Water Vapor Transmission Rate	E96	0.24 g/m^2 per day	1/lot	---
Stress Crack Resistance	D5397	200 hr	1/lot	---

Notes:

1. Thickness shall be monitored continuously through the manufacturing process, or measured physically at a frequency of not less than one per roll of manufactured geomembrane.
2. Routine testing of fabricated seams, where applicable, shall include seam shear strength and seam peel strength at a frequency of not less than one per 500 liner feet of seam. The Fabricator shall test all seams using one of the methods specified in Paragraph 3.2.C.2.

C. Extrusion rod shall be manufactured from identical resin to that used in geomembrane manufacture. Manufactured extrusion rod shall be tested for carbon black, specific gravity and melt index at a frequency of not less than one test per batch.

D. The MANUFACTURER shall reject resin shipments which do not conform with the density and melt index requirements of the approved QA/QC program. The MANUFACTURER shall reject manufactured geomembrane which does not conform to the sheet physical requirements of the approved QA/QC program. The MANUFACTURER shall reject fabricated seams, where applicable, which do not exhibit seam shear strength of greater than 120 lbs per inch of width, and seam peel strength of greater than 91 lbs per inch of width (hot wedge) or 78 lbs per inch of width (extrusion).

2.4 MANUFACTURING QUALITY CONTROL

A. Resin:
1. The MANUFACTURER shall sample and test the resin to demonstrate that the resin complies with the Specifications. The MANUFACTURER shall certify in writing that the resin meets the Specifications, and shall be held liable for any non-compliance.
2. Any geomembrane manufactured from non-complying resin shall be rejected.
3. Conformance testing, as defined in Part 2.7 of this Section and in the CQA Plan, will be carried out by the CQA Officer. If the MANUFACTURER's and CQA Officer's test results differ, the tests will be repeated by the CQA Officer, and the MANUFACTURER will be allowed to monitor this testing. The results of this latter series of tests will prevail, provided that the applicable test methods, as detailed in the CQA Plan, have been followed.

B. Rolls:
1. The MANUFACTURER shall continuously monitor geomembranes during the manufacturing process for inclusions, bubbles, or other defects.
2. The MANUFACTURER shall continuously spark test geomembranes during the manufacturing process to confirm that no defects were found.
3. No geomembrane shall be accepted that exhibits any defects.
4. The MANUFACTURER shall continuously monitor the geomembrane thickness during the manufacturing process.
5. No geomembrane shall be accepted that fails to meet the specified minimum thickness.
6. The MANUFACTURER shall sample and test the geomembrane, at a minimum, test frequencies specified in GRI Test Method GM13, subject to any revisions.
    a. Samples taken from stored rolls shall be taken across the entire width of the roll and shall not include the first wrapping or outer layer of the roll (about 3.3 feet).
    b. Samples taken at the time of manufacturing can be obtained from the end of the roll.
    c. Unless otherwise specified, samples shall be 2 feet long by the roll width. The MANUFACTURER shall mark the machine direction on the samples with an arrow.
7. Any geomembrane sample that does not comply with the Specifications will result in rejection of the roll from which the sample was obtained. DBT shall replace any rejected rolls at no additional cost to OWNER.

8. If a geomembrane sample fails to meet the quality control requirements of this Section, the MANUFACTURER shall sample and test each roll manufactured, in the same resin batch, or at the same time, as the failing roll. Sampling and testing of rolls shall continue until a pattern of acceptable test results is established.
9. Additional testing may be performed at the MANUFACTURER's discretion and expense, to more closely identify the non-complying rolls and/or to qualify individual rolls.

2.5 EQUIPMENT AND ACCESSORIES

A. Welding and Seaming Equipment:
   1. Equipped with gages showing temperatures at the nozzle (extrusion welder) or at the wedge (wedge welder).
   2. Maintained in adequate numbers to avoid delaying work.
   3. Supplied by a power source capable of providing constant voltage under a combined-line load.
   4. Electric generator shall not be placed directly on the HDPE geomembrane.

B. Field Tensiometer:
   1. Provide a tensiometer for on-site shear and peel testing of HDPE geomembrane seams.
      a. Tensiometer shall be in good working order.
      b. Built to ASTM specifications.
      c. Accompanied by evidence of calibration of equipment and gages within the past six months. The evidence of calibration shall be submitted to the CQA consultant.
   2. Tension meter:
      a. Motor driven.
      b. Jaws capable of traveling a measure rate of 2 inch per minute.
      c. Equipped with a gauge that measures the force in unit pounds exerted between the jaws.
      d. Digital readout.

C. Punch Press:
   1. Provide a punch press for the onsite preparation of specimens for testing.
   2. Capable of cutting specimens in accordance with ASTM D4437.

D. Vacuum Box:
   1. Provide a vacuum box for onsite testing of HDPE geomembrane seams in accordance with ASTM D5641.

E. Equipment necessary to perform "Pressurized Air Channel Evaluation of Dual Seamed Geomembranes" in accordance with ASTM D5820.

F. Gages:
   1. Calibrated within past six months.

2. Specified test values reading near mid-range of the gage scale.

2.6 MANUFACTURER'S RESPONSIBILITIES

A. The MANUFACTURER is responsible for producing geomembrane sheet that complies with this Specification. These responsibilities include but are not limited to:
   1. Resin and additive quality control:
      a. Acceptance of the resin and additives from chemical formulators.
      b. Testing of the raw resin and additives to ensure compliance with the MANUFACTURER's specifications and with this Specification.
   2. Formulation of the resin and additives into sheeting using mixing and extrusion equipment.
   3. Testing of the sheet material to ensure compliance with MANUFACTURER's specifications and this Specification.
   4. Shipping of the sheet material to INSTALLER.
   5. Certification of the raw materials and finished sheet to comply with this Specification.
   6. Certification of INSTALLER's training (unless INSTALLER is certified by other acceptable manufacturer list herein), experience and methods for welding, seaming, joining and inspecting geosynthetic materials installations in compliance with MANUFACTURER's standards and with Quality Assurance requirements of this Specification Part 1.2.

2.7 CONFORMANCE TESTING

A. Samples of the geomembrane will be removed by the CQA or MANUFACTURER's QC Officer at the MANUFACTURER's plant during production of the geomembrane. Samples will be sent to a geosynthetics CQA laboratory for testing to assure conformance with the requirements of this section. If mutually agreed upon by OWNER, DBT, and FDEP, samples may be shipped from MANUFACTURER to the CQA Officer's designated laboratory, provided that adequate chain-of-custody documentation is prepared and submitted, including origin and destination of sample, lot number, roll number, product name, project name, date of production, and date of shipment, at a minimum.
B. Samples and tests will be selected by the CQA or MANUFACTURER's QC Officer in accordance with this section and with the procedures outlined in the CQA Plan.
C. Samples will be taken at a minimum frequency of one sample per 100,000 square feet with a minimum of one sample per lot. If the MANUFACTURER ships geomembrane that requires sampling and testing at a frequency greater than one per 100,000 square feet, then the MANUFACTURER shall pay for the cost of the additional CQA sampling and testing beyond one sample per 100,000 square feet.
D. The CQA Officer may increase the frequency of sampling in the event that test results do not comply with the requirements of Parts 2.2 and 2.3 of this section. DBT shall bear the expense of this additional testing.
E. At a minimum, the geomembrane tests will include those identified in Table 1.

F. Any geomembranes that are not certified in accordance with Part 1.3 of this section, or that do not comply with Parts 2.2, 2.3 and 2.4 of this section, will be rejected by OWNER. DBT shall replace the rejected material with new material, at no additional cost to OWNER.

PART 3 - EXECUTION

3.1 GEOSYNTHETIC LINER SYSTEM

A. Subgrade:
   1. Protect GCL and subgrade at all times from damage until such time as the placement of HDPE geomembrane liner and other components of the geosynthetic liner system are complete.
   2. The subgrade shall be prepared in a manner consistent with proper subgrade preparation techniques for the installation of HDPE Geomembrane.
      a. The subgrade shall be properly compacted so as not to settle and cause excessive strains in the HDPE Geomembrane or other synthetic liner materials.
      b. Prior to installation, ensure a surface free of debris, roots, or angular stones larger than 1/2 inch.
      c. In addition, ensure that the subgrade has been rolled to provide a uniform surface.
      d. During installation, ensure that rutting or ravelling is not caused by installation equipment or weathering.
      e. Do not drag geomembrane above GCL. Damages to the GCL caused by the DBT shall be repaired by the DBT at the DBT's expense.

B. Anchor Trenches:
   1. Geosynthetic materials placed on side slopes shall be anchored into trenches as detailed on the Contract Drawings.
   2. Excavation, backfill and compaction shall be in accordance with Section 02300.

C. HDPE Geomembrane:
   1. Do not deploy geomembrane until all inspections of the GCL layer are complete and approval has been given by the OWNER. The OWNER shall be notified a week in advance of initial deployment of geomembrane.
   2. General:
      a. INSTALLER of HDPE geomembranes is responsible for handling, fitting, welding, seaming, jointing and testing of geosynthetic materials sheets or blankets in the field in accordance with the Construction Quality Assurance (CQA) Plan.
      b. These responsibilities include but are not limited to:
         1) Acceptance (in writing) of the geosynthetic materials sheets or blankets from the transporter.
         2) Acceptance (in writing) of the subgrade soil layer which will serve as a base for the HDPE geomembrane.
            a) This acceptance shall precede installation of the HDPE geomembrane.

b) Shall state that the INSTALLER has inspected the surface, and reviewed the Specifications for material and placement, and finds all conditions acceptable for placement of HDPE geomembrane liners.
c) Shall explicitly state any and all exceptions to acceptance.

3) Handling, welding, seaming, jointing, testing and repair of HDPE geomembrane liners and other geosynthetic materials are in compliance with this Specification and with written procedures manuals prepared by the MANUFACTURER.
a) Manual shall be submitted to the OWNER together with Shop Drawings showing the layout of HDPE geomembrane within the facility.
(1) Do not deviate from the procedures included in the manual.
b) HDPE Geomembrane shall not be placed upon frozen foundation, standing water or other conditions which will result in deterioration of the foundation.
c) HDPE Geomembrane liner materials shall be laid out according to plans previously approved by the OWNER.
d) Adjacent rolls of HDPE geomembrane shall overlap a minimum of 3 inches, provided that greater overlap may be required to allow seaming in accordance with the MANUFACTURER's instructions.

4) Repair or replacement of defects in the geosynthetic materials as required by the Inspector or the OWNER.
5) INSTALLER and MANUFACTURER may be the same firm.

3. Panel deployment:
a. Only those panel/sheets that can be seamed in 1 day shall be deployed.
b. Place panels with minimal handling.
1) Orient sheets to eliminate or minimize number of horizontal seams on side slopes.
2) Protect panels from tear, puncture or abrasion.
3) No seams will be permitted in the leachate collection trench.
c. Equipment used to deploy the geomembrane shall not damage or rut the subgrade layer.
1) A rut is defined as a 1/4 inch depression over a 10 feet straight-edged length.
d. No vehicular traffic is permitted on unprotected HDPE geomembrane, except for low-ground pressure equipment necessary for deployment and installation of overlying geosynthetic layers.
e. Minimize foot traffic.
1) Do not allow personnel access to wet or slippery liners without adequate safety precautions.
2) Do not allow footwear that may damage the geomembrane.
f. Ballast with sandbags to prevent wind uplift as recommended by MANUFACTURER based on local climatic conditions.
1) Remove and replace all wind damaged panels at no additional cost to OWNER.
2) If wind causes panels to be displaced, displaced panel may not be reused.
g. Install HDPE geomembrane in stress free, tension free and relaxed condition.

1) Account for temperature and weather-related impacts when deploying and covering.
2) Stretching to fit and folding are not permitted.

h. Do not allow HDPE geomembrane to bubble, fold, or create ripples as a result of deployment of drainage layer or protective soil cover placement.
1) Except as noted on Contract Drawings no folds in HDPE geomembrane will be allowed.

i. Any panel exhibiting stretching caused by placement, covering techniques, or wind shall be removed and may not be incorporated in the final construction.

j. Field seaming:
1) Field seaming shall be done in accordance with seaming recommendations furnished by the geomembrane MANUFACTURER and referenced EPA documents.
2) Each piece of seaming equipment and each operator shall perform demonstration seams at the start of a shift, whenever equipment is switched on or seaming is interrupted for more than ten minutes, and at other times at the discretion of the INSTALLER and Inspector.
3) Demonstration seams shall use the same seaming materials and methods to be used in the actual construction.
4) Surfaces to be seamed shall be clean and dry at the time of seaming.
a) Precipitation and ponding of water on the HDPE geomembrane shall cause termination of seaming operations.
b) HDPE geomembrane shall not be seamed when ambient temperatures are below 41 DegF or above 104 DegF, without written consent of HDPE geomembrane MANUFACTURER and OWNER.
5) HDPE geomembrane sheets shall be seamed continuously without fishmouths or breaks in the seam.
a) Where fishmouths are unavoidable, the sheet shall be slit to a point such that the sheet lies flat and with no remaining wrinkle.
b) The two edges of the slit shall be seamed together provided that the overlap for this seam shall be a minimum of 6 inch.
c) Areas of the slit which do not achieve an overlap of 6 inch, including the terminus of the slit, shall be provided with a patch as discussed below.
6) All HDPE geomembranes shall be seamed by thermal fusion methods as recommended by the HDPE geomembrane MANUFACTURER.
a) HDPE geomembrane seaming shall be double wedge weld unless otherwise approved or prohibited by construction.
7) MANUFACTURER's seaming instructions shall specifically address subgrade preparation, seaming materials, temporary and permanent jointing, seaming temperatures including temperatures for seaming materials, seam finishing and curing.
8) A copy of MANUFACTURER's seaming instructions shall be available on site at all times and shall not be deviated from without written approval of the MANUFACTURER and OWNER.
9) All panels/sheets should be overlapped a minimum of 3 inch.

a) If horizontal seams are required on side slopes, the upper panel should be lapped over the lower panel.

10) Seaming shall not be conducted in the presence of standing water and/or soft subgrades.
    a) The seamed area shall be cleaned of dust, dirt and foreign material prior to and during the seaming operation.
11) Seaming shall extend to the outside edge of panels/sheets to be placed in anchor and/or drainage trenches.
12) Tack welds shall conform with manufacturers seaming techniques and shall not damage underlying membrane.

k. Patching:
    1) Defects in and damage to HDPE geomembrane sheets shall be repaired by seaming a patch over the defect.
        a) The patch material shall consist of an undamaged piece of HDPE geomembrane cut to provide a minimum of 6 inch of overlap in all directions from the defect.
        b) Round corners shall be utilized on all patches. No bead or spot patching will be accepted.
        c) Torn or permanently twisted HDPE geomembrane shall be replaced at no expense to the OWNER.
    2) Test all patch seams using one of the following nondestructive tests: vacuum tests; spark tests; or ultrasonic tests.
        a) Test patch seams destructively at a frequency of ten percent or a minimum of one test per seaming personnel per day.
        b) This destructive testing may be accomplished using demonstration seams performed adjacent to the liner installation.
l. Smoking is not permitted while on the geomembrane.
m. Field Panel Identification: The INSTALLER's QC Consultant will document that the Geomembrane INSTALLER labels each field panel with an "identification code" consistent with the approved panel layout plan. The location of the label and the color of marker used must be as agreed to in the QA/QC Preconstruction Meeting.

3.2 FIELD QUALITY CONTROL

A. Inspector shall not be a part of the installation program and shall not serve as a substitute for performing the duties or certification required of the MANUFACTURER and INSTALLER.
    1. Inspector's responsibilities include, but are not limited to:
    a. Inspection of the material and the handling and field installation of the geomembranes. Inspection of all welds, repairs and quality control test results.
    b. All exceptions to material or installation shall be documented and furnished to the CQA Officer in writing within 48 hours of discovery.
    c. Inspection and Certification of HDPE geomembrane integrity until completion of placement of protective soil cover.

B. Trial Seam Testing:
   1. Trial seams shall be made each half-day prior to production seaming.
      a. The location of trial seam shall be in an area proposed for the day's production seaming.
      b. Equipment, methods and personnel shall be the same as proposed for the day's seaming.
   2. Test five replicates (1 inch wide specimens) cut from trial seam. Field seaming cannot begin until successful trial seams have been completed.
      a. To be acceptable, five of five replicate test specimens must meet specified seam strength requirements. The break must occur in the liner material itself, not through peel separation (FTB).
      b. If the field tests fail to meet these requirements, the entire operation shall be repeated.
      c. If the additional test seams fail, the seaming apparatus or seamer shall not be accepted or used for seaming until the deficiencies are corrected and two consecutive successful test seams are achieved.

C. Non-Destructive Seam Testing:
   1. All field seams shall be non-destructively tested over their full length.
      a. Seam testing shall be performed as the seaming work progresses, not at the completion of field seaming.
      b. All testing shall be documented. Any seams which fail shall be repaired and documented.
   2. Non-destructively test all field seams continuously using one of the following nondestructive seam tests: vacuum box; and pressurized air channel test.

      Vacuum Testing:
      a. The equipment shall comprise the following.
         i. A vacuum box assembly consisting of a rigid housing, a transparent viewing window, a soft neoprene gasket attached to the bottom, port hole or valve assembly, and a vacuum gauge.
         ii. A steel vacuum tank and pump assembly equipped with a pressure controller and pipe connections.
         iii. A rubber pressure/vacuum hose with fittings and connections.
         iv. A bucket and applicator.
         v. A soapy solution.

      b. The following procedures shall be followed.
         i. Energize the vacuum pump and reduce the tank pressure to approximately 5 psi gauge.
         ii. Wet a strip of geomembrane seam having an area larger than the vacuum box assembly with the soapy solution.
         iii. Place the box over the wetted area.
         iv. Close the bleed valve and open the vacuum valve.
         v. Ensure that a leak tight seal is created.

vi. Examine the geomembrane through the viewing window for the presence of soap bubbles for not less than 10 seconds.
vii. If no bubbles appear after 10 seconds, close the vacuum valve and open the bleed valve, move the box over the next adjoining area with a minimum 3-in. overlap, and repeat the process.
viii. All areas where soap bubbles appear shall be marked with a marker that will not damage the geomembrane and repaired in accordance with Paragraph 3.3 of this section.

3. Air Pressure Testing (For Double Fusion Seams Only):

a. The following procedures are applicable to those processes that produce a double seam with an enclosed space.

b. The equipment shall comprise the following.

i. An air pump (manual or motor driven), equipped with a pressure gauge, capable of generating and sustaining a pressure between 25 and 30 psi, mounted on a cushion to protect the geomembrane.
ii. A rubber hose with fittings and connections.
iii. A sharp hollow needle, or other approved pressure feed device.

c. The following procedures shall be followed.

i. Seal both ends of the seam to be tested.
ii. Insert needle, or other approved pressure feed device, into the channel created by the fusion weld.
iii. Insert a protective cushion between the air pump and the geomembrane.
iv. Energize the air pump to a pressure between 25 and 30 psi, close valve, allow two minutes for pressure to stabilize, and sustain the pressure for not less than 5 minutes.
v. If loss of pressure exceeds 4 psi, or if the pressure does not stabilize, locate faulty area and repair in accordance with Paragraph 3.3 of this section.
vi. Cut opposite end to verify continuity of seam, remove needle, or other approved pressure feed device, and seal repair in accordance with Paragraph 3.3 of this section.

D. Destructive Seam Testing:
1. A minimum of one destructive test per 500 linear feet of seam, and as many other samples as CQA Officer determines appropriate, shall be obtained at locations specified by the CQA Officer. Minimum frequency shall be extended up to 1000 feet if electrical leak location survey method is used.
a. Sample locations shall not be identified prior to seaming.

b. The samples shall be a minimum of 12 inch wide by 48 inch long with the seam centered lengthwise. Sample size will be adjusted for double track seams or dual hot wedge seams to include both sets of seams.
c. Each sample shall be cut into three equal pieces with one piece retained by the INSTALLER, one piece given to a CQA Testing Laboratory, and the remaining piece given to the CQA Officer for quality assurance testing and/or permanent record.
d. Each sample shall be numbered and recorded on the final panel layout record drawing, and cross-referenced to a field log which identifies:
   1) Panel/sheet number.
   2) Seam number.
   3) Top sheet.
   4) Date and time cut.
   5) Ambient temperature.
   6) Seaming unit designation.
   7) Name of seamer.
   8) Seaming apparatus temperature and pressures (where applicable).

2. A minimum of four 1 inch wide replicate specimens shall be cut from the INSTALLER's sample.
   a. A minimum of 2 specimens shall be tested for shear strength and 2 for peel adhesion using an approved field quantitative tensiometer. Jaw separation speed shall be 2 inch per minute.
   b. To be acceptable, all replicate test specimens must meet the specified seam strength requirements and fail as Film Tear Bond.
   c. If the field tests pass, 5 specimens shall be tested at the CQA Testing Laboratory for shear strength and 5 for peel adhesion in accordance with ASTM D4437.
   d. To be acceptable, 4 out of 5 replicate test specimens must meet the specified seam strength requirements and fail as Film Tear Bond and the fifth sample meets 80% of the required strength.
3. The minimum required seam strengths are:

		Required Value (lbs/in width)
Description	Test Method	
HDPE Peel (Hot Wedge)	ASTM D4437	91
HDPE Shear (all)	ASTM D4437	120
HDPE Peel (Extrusion)	ASTM D4437	78

4. If the field tests pass, 5 specimens shall be tested at the CQA Testing Laboratory for shear strength and 5 for peel adhesion in accordance with ASTM D4437.
   a. To be acceptable, 4 out of 5 replicate test specimens must meet the specified seam strength requirements and fail as Film Tear Bond.
   b. If the field or laboratory tests fail, the seam shall be repaired in accordance with the MANUFACTURER's Quality Control manual.
   c. In addition, all destructive seam sample holes shall be repaired the same day as cut.

d. Certified test results on all field seams shall be submitted to and approved by the CQA Officer prior to acceptance of the seam.

5. Repaired areas shall be destructively tested at a minimum of every 500 linear feet of extrusion weld. Minimum frequency shall be extended up to 1000 feet if electrical leak location survey method is used.
   a. All repaired areas shall be non-destructively tested.
6. Destructive testing shall be performed by a CQA Testing Laboratory not employed by the INSTALLER.
7. A map showing the locations, number and type of all patches shall be prepared and provided to the OWNER.
8. Documentation: The following documentation must be maintained at the project site for review by the CQA Officer, OWNER, or Inspector.
   a. Geomembrane INSTALLER's Documentation:
      1) Daily Log: daily record that summarizes panels deployed, seams completed, seam testing, seam repair, personnel on site, weather conditions, and equipment on site.
      2) Material Conformance: maintain original conformance certificate(s) from geomembrane manufacturer.
      3) Subgrade Acceptance Log: maintained originals of subgrade acceptance forms for each panel and signed by the Geomembrane INSTALLER.
      4) Panel Log: provides geomembrane roll number used and subgrade acceptance for each panel deployed.
      5) Seam Testing Log: provides a complete record of all nondestructive and destructive seam tests performed as part of the Geomembrane INSTALLER's QC program.
      6) Seam/Panel Repair Log: provides a complete record of all repairs and vacuum box testing of repairs made to defective seams or panels.
      7) As-Built Drawing: maintain an as-built drawing updated on a weekly basis.

E. Destructive Test Failure:
   1) The following procedures shall apply whenever a sample fails a destructive test, whether the test is conducted by the CQA laboratory, the Geosynthetics Installer’s laboratory, or by a field tensiometer. The Geosynthetics Installer shall have two options, as described in b and c below.
   2) The Geosynthetics Installer can reconstruct the seam (e.g., remove the old seam and reseam) between any two passed test locations.

3) The Geosynthetics Installer can trace the welding path to an intermediate location, a minimum of 10 ft from the location of the failed test (in each direction) and take a small sample for an additional field test at each location. If these additional samples pass the tests, then full laboratory samples shall be taken. If these laboratory samples pass the tests, then the seam shall be reconstructed between these locations. If either sample fails, then the process shall be repeated to establish the zone in which the seam should be reconstructed. In any case, all acceptable seams must be bounded by two locations from which samples passing laboratory destructive tests have been taken. In cases where the length of reconstructed seam exceeds 150 ft, a destructive sample taken from within the reconstructed zone must pass destructive testing. Whenever a sample fails, the CQA Officer may require additional tests for seams that were formed by the same seamer and/or seaming apparatus or seamed during the same time shift.
4) DBT shall bear the cost of the destructive testing by the CQA laboratory for those seam samples associated with b. and c. above.

3.3 Defects and Repairs:

1. All seams and non-seam areas of the geomembrane will be examined by the CQA Officer for evidence of defects, holes, blisters, undispersed raw materials and any sign of contamination by foreign matter. The surface of the geomembrane shall be clean at the time of examination. The geomembrane surface shall be swept or washed by the Geosynthetics INSTALLER if surface contamination inhibits examination. The Geosynthetics INSTALLER shall ensure that this examination of the geomembrane precedes any seaming of that section.

2. Each suspect location, both in seam and non-seam areas, shall be nondestructively tested using the methods described Paragraph 3.2 of this section, as appropriate. Each location that fails nondestructive testing shall be marked by the CQA Officer and repaired by the Geosynthetics INSTALLER. Work shall not proceed with any materials that will cover repaired locations until laboratory test results with passing values are available.

3. When seaming of a geomembrane is completed (or when seaming of a large area of a geomembrane is completed) and prior to placing overlying materials, the CQA Officer shall identify excessive geomembrane wrinkles. The Geosynthetics INSTALLER shall cut and reseam all wrinkles so identified. The seams thus produced shall be tested like any other seams.

4. Repair Procedures:
   a. Any portion of the geomembrane exhibiting a flaw, or failing a destructive or nondestructive test, shall be repaired by the Geosynthetics INSTALLER. Several repair procedures are specified below. The final decision as to the appropriate repair procedure shall be agreed upon between the CQA Officer and the Geosynthetics INSTALLER. The procedures available include:

i. patching - used to repair large holes, tears, undispersed raw materials, and contamination by foreign matter;
ii. abrading and reseaming - used to repair small sections of extruded seams;
iii. spot seaming - used to repair small tears, pinholes, or other minor, localized flaws;
iv. capping - used to repair long lengths of failed seams; and
v. removing bad seam and replacing with a strip of new material seamed into place (used with long lengths of fusion seams).

b. In addition, the following shall be satisfied:

i. surfaces of the geomembrane that are to be repaired shall be abraded no more than one hour prior to the repair;
ii. all surfaces must be clean and dry at the time of repair;
iii. all seaming equipment used in repair procedures must be approved by OWNER;
iv. the repair procedures, materials, and techniques shall be approved in advance, for the specific repair, by the CQA Officer and Geosynthetics INSTALLER;
v. patches or caps shall extend at least 6 inches beyond the edge of the defect, and all corners of patches shall be rounded with a radius of at least 3 inches.; and
vi. the geomembrane below large caps shall be appropriately cut to avoid water or gas collection between the two sheets.

5. Each repair shall be numbered and logged and shall be nondestructively tested using the methods described in Paragraph 3.2 of this section, as appropriate. Repairs that pass the nondestructive test shall be taken as an indication of an adequate repair. Failed tests will require the repair to be redone and retested until a passing test result is achieved. At the discretion of the CQA Officer, destructive testing may be required on large caps at no additional cost to OWNER.

6. The Geosynthetics INSTALLER shall repair damage to the existing HDPE liner tie-in locations. Repairs shall be made according to this Specification.

3.4 MATERIAL IN CONTACT WITH GEOMEMBRANE

A. The Geosynthetics INSTALLER shall take all necessary precautions to ensure that the geomembrane is not damaged during its installation or during the installation of other components of the liner system or by other construction activities. If approved by the CQA Officer, additional loosely placed geotextile sections may be used by the Geosynthetics INSTALLER to protect the geomembrane.

B. No granular materials shall be placed directly on the geomembranes at any time. A nonwoven geotextile or geocomposite shall be installed between aggregate and geomembrane and between geonet edges and geomembrane.

C. Unless otherwise specified by OWNER, all equipment operating on earthen materials overlying geosynthetics shall comply with the following:

Allowable Equipment Ground Pressure (psi)	Thickness of Overlying Compacted Soil (ft)
<5	1.0
<10	1.5
<20	2.0
>20	3.0

D. In heavily trafficked areas such as access ramps, and in areas trafficked by rubber tire vehicles, the thickness of overlying compacted fill shall be at least 3 ft.

E. Connection of the geomembrane to appurtenances shall be made according to these Specifications and as shown on the Drawings. Extreme care shall be taken while seaming around appurtenances since neither nondestructive nor destructive testing may be feasible in these areas. The Geosynthetics INSTALLER shall ensure that the geomembrane has not been damaged while making connections to appurtenances.

3.5 GEOSYNTHETIC LINER SYSTEM ACCEPTANCE

A. DBT shall retain all ownership and responsibility for the geosynthetic bottom liner system until final acceptance by the OWNER.

1. OWNER will accept the geosynthetic liner system installation when the installation is finished and all required warranties, test results, and documentation from the DBT, MANUFACTURER, Inspector and INSTALLER has been received and approved, and verification of the adequacy of all field seams and repairs, including associated testing, is complete.

END OF SECTION

SECTION 02776
GEOSYNTHETIC CLAY LINER (GCL)

PART 1 - GENERAL

1.1 SUMMARY

A. Section Includes:
   1. Purchasing, delivery, installation, quality control, and testing of a GCL for a landfill bottom liner system.

B. Related Sections include but are not necessarily limited to:
   1. Section 02300 – Earthwork
   2. Section 02775 – HDPE Geomembrane Liner

1.2 QUALITY ASSURANCE

A. Referenced Standards:
   1. ASTM International (ASTM):
      a. D2216, Moisture Content of Soil.
      b. D4632, Test Method for Breaking Load and Elongation of Geotextiles (Grab Method).
      c. D4643, Determination of Water Content of Soil by Microwave Owen Method.
      d. D4833, Test Method for Index Puncture Resistance of Geotextiles, Geomembranes, and Related Products.
      e. D4873, Identification, Storage and Handling of Geosynthetic Rolls.
      f. D5261, Measuring Mass Per Unit Area of Geotextiles.
      g. D5321, Test Method for Determining the Coefficient of Soil and Geosynthetic or Geosynthetic and Geosynthetic Friction by the Direct Shear Method.
      h. D5887, Test Method for Measurement of Index Flux Through Saturated Geosynthetic Clay Liner Specimens Using a Flexible Wall Permeater.
      i. D5888, Guide for Storage and Handling of Geosynthetic Clay Liners.
      j. D5889, Quality Control of GCL.
      k. D5890, Swell Index of Clay Mineral Component of Geosynthetic Clay Liners.
      l. D5891, Fluid Loss of Clay Component of Geosynthetic Clay Liners.
      m. D5993, Test Method for Measuring Mass Per Unit Area of Geosynthetic Clay Liners.
      n. D6072, Obtaining Samples of GCL
      o. D6102, Guide for Installation of Geosynthetic Clay Liners.
      p. D6243, Test Method for Determining the Internal and Interface Shear Resistance of Geosynthetic Clay Liner by the Direct Shear Method.
      q. D6496, Determining Average Bonding Peel Strength Between Top and Bottom Layers of Needle Punched GCLs.
      r. D6766, Evaluation of Hydraulic Properties of GCLs permeated with Potentially Incompatible Liquids.
      s. D6768, Tensile Strength of Geosynthetic Clay Liner.
   2. Geosynthetic Research Institute (GRI):

a. GCL-3, Test Methods, Required Properties, and Testing Frequencies of Geosynthetic Clay Liners (GCLs).

B. Quality Assurance:
1. The CQA Inspector will direct testing to support construction quality assurance program and to provide documentation of such to appropriate regulatory agencies.
2. The CQA Inspector will observe that the GCL is stored, placed, seamed, and protected as described in ASTM D4873, D5888 and D6102.

C. Qualifications:
1. Each manufacturing firm shall demonstrate 5 years continuous experience, including a minimum of 5,000,000 SF of manufacture/fabrication in geosynthetic lining systems. Aggregate experience shall not apply.
2. Installer shall attend pre-installation conference.

1.3 DEFINITIONS

A. Manufacturer: Manufacturer produces geosynthetic clay liner panels from first quality geotextiles and sodium bentonite. The manufacturer is responsible for producing panels which comply with this Specification.

B. Installer: Installers of GCLs are responsible for storing, handling, fitting, seaming and testing of GCL panels in the field. Installer and manufacturer may be the same firm.

C. Hydrated GCL is defined as material which has become soft as determined by squeezing the material with finger pressure, material which has exhibited swelling, or material which as a moisture content greater than 100 percent as determined by ASTM D2216.

1.4 SUBMITTALS

A. Shop Drawings:
1. Manufacturer's documentation that bentonite, geotextiles and GCL comply with required material properties.
2. Manufacturer and Installer quality control manuals.

B. Miscellaneous Submittals:
1. Test results:
a. Bentonite, geotextile and GCL tests at frequency specified in respective quality control manuals. Results shall include or bracket the rolls delivered for use in the Work.
2. Qualification documentation specified in Article 1.2.
3. Submit written certifications that:
a. The GCL delivered to site meets the requirements of this Specification.
b. The GCL was received and accepted in undamaged condition from shipper.
c. The subgrade has been properly prepared and acceptable for the placement of the GCL.
d. The GCL was installed in accordance with this Specification and with approved shop drawings.
e. The geomembrane on top of the GCL was placed properly and carefully.

1.5 DELIVERY, STORAGE, AND HANDLING

A. Do not place GCL rolls directly on the ground.

B. Store and protect GCL from dirt, water, ultraviolet light and other sources of damage.

C. Label, handle, and store GCL in accordance with ASTM D4873 and as specified herein.
   1. Wrap each roll in an opaque and waterproof layer of plastic during shipment and storage.
   2. Do not remove the plastic wrapping until deployment.

D. Label each roll with the manufacturers name, lot number, roll number, and roll dimensions (length, width, gross weight).
   1. Repair or replace GCL or plastic wrapping damaged as a result of storage or handling, as directed.
   2. Do not expose GCL to temperatures in excess of 71 DegC (160 DegF) or less than 0 DegC (32 DegF) unless recommended by the Manufacturer.

E. Do not use hooks, tongs or other sharp instruments for handling the GCL. Do not lift rolls by use of cables or chains in contact with the GCL. Do not drag GCL along the ground.

F. Damaged rolls may be rejected. If rejected, it must be verified that rejected material is removed from the site or stored at a location separate from accepted rolls. GCL rolls that do not have proper manufacturer's documentation must be stored at a separate location until all documentation has been received and approved.

1.6 WARRANTY

A. The Manufacturer shall provide a 5-year warranty to the Owner against manufacturing defects. The warranty shall include defective product found to be not in compliance with the requirements of this specification. Warranty shall not be prorated.

PART 2 - PRODUCTS

2.1 ACCEPTABLE MANUFACTURERS

A. Subject to compliance with the Contract Documents, the following Manufacturers are acceptable:
   1. Geosynthetic Clay Liners:
      a. CETCO, 1500 W. Shore Drive, Arlington Heights, Illinois 60004.
      b. Solmax, 23 Truman Road, Barrie, Ontario L4M 3V7

2.2 MATERIALS

A. Geosynthetic Clay Liner:
   1. The active ingredient of the GCL shall be natural sodium bentonite. Encapsulate bentonite between two geotextiles.
   2. Lock-stitch or heat-seal needle punched geotextile backed GCL with high strength polypropylene thread to provide internal shear strength reinforcing. The internal shear reinforcing mechanism shall resist failure due to thread pull-out over long-term creep situations.

3. Continuously adhere the bentonite to both geotextiles to ensure that the bentonite will not be displaced during handling, transportation, storage and installation, including cutting, patching and fitting around penetrations. The bentonite sealing compound or bentonite granules used to seal penetrations and make repairs shall be made of the same natural sodium bentonite as the GCL and shall be as recommended by the GCL manufacturer. The permeability of the GCL overlap seams shall be equal to or less than the permeability of the body of the GCL sheet.
4. Provide material meeting GRI-GCL 3 or as specified in Table 1 below.

Table 1: GCL Properties

Item	ASTM Test Method	Test Value	Min. Test Frequency	
			MQC	CQA
Geotextile Property				
Cap Nonwoven, mass/unit area	ASTM D5261	6 oz/sy	1/200,000 ft^2	1/100,000 ft^2
Carrier Scrim Nonwoven, mass/unit area	ASTM D5261	6 oz/sy	1/200,000 ft^2	1/100,000 ft^2
Bentonite Property				
Minimum Free Swell	ASTM D5890	24 ml/2 g min	1/100,000 lbs	1/100,000 ft^2
Maximum Moisture Content	ASTM D4643 or D2216	12%	1/100,000 ft^2	---
Maximum Fluid Loss	ASTM D5891	18 mL	1/100,000 lbs	---
Finished GCL Property				
Maximum Hydraulic Conductivity (MARV)	ASTM D5887 @5 psi max. confining stress	$5x10^{-9}$ cm/s	1/100,000 ft^2	1/100,000 ft^2
Minimum Bentonite Content (MARV)	ASTM D5993 (@ 0% moisture)	0.75 lb/sf	1/40,000 ft^2	1/100,000 ft^2
Typical Internal Shear Strength	ASTM D6243	500 psf (when hydrated)	1/project	---
Minimum peel strength, MD (MARV)	ASTM D6496	5.3 lbs/in	1/40,000 ft^2	---
Tensile strength, MD (MARV)	ASTM D6768	45 ppi	1/40,000 ft^2	---

2.3 SOURCE QUALITY CONTROL

A. Interface Friction Tests.
   1. This material is part of a system. The system shall meet the requirements before the component material can be deemed acceptable. Testing will include the interfaces between the following adjacent materials:
      - Subgrade/GCL
      - GCL/60 mil Textured HDPE Geomembrane

- 60 mil HDPE Geomembrane/Geocomposite
- Geocomposite/Protective Cover

2. Interface friction angle testing is the responsibility of the DBT. All failing tests shall be the CONTRACTOR's responsibility. Requirements are based on standard practice and permit conditions.

B. The testing shall be performed as follows.
   1. Conduct one set of three direct-shear interface friction tests on each of the interfaces listed above. Normal stresses of 5,000 psf, 10,000 psf, and 20,000 psf shall be used during hydration and shearing. Samples shall be allowed to hydrate for a minimum of 3 days prior to shear testing. Orient all geosynthetic materials such that the shear force is parallel to the downslope orientation of these components in the field.
   2. A minimum friction angle of 23 degrees is required for each interface in the system defined in Paragraph A.
   3. Test interface friction between the GCL and adjacent materials in accordance with ASTM D6243, Procedure B.

PART 3 - EXECUTION

3.1 INSTALLATION

A. Construct, inspect and test geosynthetic clay liner in accordance with manufacturer's recommendations, this specification and ASTM D6102.

B. Halt installation when the weather is of such a nature as to endanger the integrity and quality of the installation.

C. Assure adequate dust control methods are in effect to prevent the unnecessary accumulation of dust and dirt.

D. Maintain natural surface water drainage diversions around the work area and provide for the disposal of water which may collect in the work area directly from precipitation falling within the area or from inadequate diversion structures or practices.

E. Do not allow vehicles in direct contact with the deployed GCL.

F. Subgrade Preparation:
   1. Prepare subgrade in a manner consistent with proper subgrade preparation techniques for the installation of GCL.
   2. Properly compact the subgrade so as not to settle and cause excessive strains in the GCL or other synthetic liner materials.
   3. Prior to installation, ensure a surface free of debris, roots, or angular stones larger than 1/2 IN.

4. Subgrade soils proof-rolled with a ten (10) ton drum roller, two (2) passes in each of two (2) perpendicular directions or as directed by CQA Inspector. The subgrade shall be compacted and proof-rolled under observation of the CQA Inspector to assure the maximum practical compaction under the existing field conditions has been achieved. See specification Section 02220 for project specific compaction requirement.
5. Ensure rutting or raveling is not caused by installation equipment or weather.
6. Ensure that lines and grades have been verified by the Contractor and a subgrade acceptance form has been submitted.

G. Construct and backfill anchor trenches.

H. Deploy GCL in a manner to ensure it is not damaged.

I. On slopes, anchor the GCL securely and deploy it down the slope in a controlled manner.

J. Weight the GCL with sandbags or equivalent in the presence of wind.

K. Minimize cutting GCL. Whenever possible, overlap instead of cutting material. If cutting is required, cut GCL with a cutter or other approved device. Seal all cut edges, as recommended by Manufacturer, to prevent loss of bentonite. Protect adjacent materials from potential damage due to cutting of GCL.

L. During GCL deployment, do not entrap in or beneath GCL, stones, trash, or moisture that could damage GCL.

M. Visually examine entire GCL surface. Ensure no potentially harmful foreign objects, such as needles, are present.

N. Do not place GCL in the rain or at times of impending rain.

O. Do not place GCL in areas of ponded water.

P. Remove and replace GCL which has been hydrated (above 100% per ASTM 4643) prior to being covered.

Q. In general, only deploy GCL that can be covered during that day.

R. If applicable, face the GCL's woven carrier geotextile against soil of prepared subgrade.

S. Overlaps:
   1. Reinforced GCL shall be overlapped a minimum of 12 inches along the sides and a minimum of 18 inches on butt seams or as recommended by manufacturer, whichever is greater.
   2. For reinforced needle-punched GCLs, add additional granular bentonite to overlapped area at a rate specified by the manufacturer.

T. Defects and Repairs:
   1. Repair all flaws or damaged areas by placing a patch of the same material extending at least 1 FT beyond the flaw or damaged area. For needle-punched GCLs, add granular bentonite to the overlapped edges of the patch at the manufacturer's recommended rate.

2. Ensure that all defects and defect corrective actions (panel rejected, patch installed, etc.) are recorded, and corrective actions are performed in accordance with this specification.

U. The CQA Inspector shall observe that the equipment used to install geocomposite does not damage it during handling, deployment, or due to leakage of hydrocarbons or other means.

V. Crews working on the GCL may not smoke, wear shoes that could damage the GCL, or engage in activities that could damage the GCL.

END OF SECTION

SECTION 02777
DRAINAGE COMPOSITE

PART 1 - GENERAL

1.1 SUMMARY

A. Section Includes:

1. Purchasing, delivery, installation, quality control, and testing of a geocomposite (heat bonded geonet/geotextile) for a landfill bottom liner system.

B. Related sections include but are not necessarily limited to:

1. Section 02300 – Earthwork
2. Section 02775 – HDPE Geomembrane Liner
3. Section 02778 - Geotextiles
4. Construction Quality Assurance Plan

1.2 QUALITY ASSURANCE

A. Referenced Standards:

1. ASTM International (ASTM):
   a. ASTM D1238 - Standard Test Method for Melt Flow Rates of Thermoplastics by Extrusion Plastometer.
   b. ASTM D 1505-98 Standard Test Method for Density of Plastics by the Density-Gradient Technique.
   c. ASTM D4218 - Standard Test Method for Determination of Carbon Black Content in Polyethylene Compounds by the Muffle-Furnace Technique.
   d. ASTM D4355 - Standard Test Method for Deterioration of Geotextiles by Exposure to Light, Moisture and Heat in a Xenon Arc Type Apparatus.
   e. ASTM D4491 - Test Method for Water Permeability of Geotextiles by Permittivity.
   f. ASTM D4533 - Standard Test Method for Trapezoid Tearing Strength of Geotextiles.
   g. ASTM D4632 - Standard Test Method for Grab Breaking Load and Elongation of Geotextiles.
   h. ASTM D4716 - Standard Test Method for Determining the (In-Plane) Flow Rate Per Unit Width and Hydraulic Transmissivity of a Geosynthetic Using a Constant Head.
   i. ASTM D4751 - Standard Test Method for Determining Apparent Opening Size of a Geotextile.
   j. ASTM D5035 - Test Method for Breaking Force and Elongation of Textile Fabrics (Strip Method).
   k. ASTM D5199 - Standard Test Method for Measuring Nominal Thickness of Geosynthetics.
   l. ASTM D5261 - Standard Test Method for Mass Per Unit Area of Geotextiles.
   m. ASTM D5321 - Standard Test for Determining the Coefficient of Soil and Geosynthetic or Geosynthetic and Geosynthetic Friction by the Direct Shear Method.
   n. ASTM D6241 - Standard Test Method for the Static Puncture Strength of Geotextiles and Geotextile-Related Products Using a 50-mm Probe.
   o. ASTM D6364 - Test Method for Determining the Short-Term Compressive Behavior of Geosynthetics.

p. ASTM D7005 - Standard Test Method for Determining the Bond Strength (Ply Adhesion) of Geocomposites.

2. Geosynthetic Research Institute (GRI).

a. GRI GC8 – Determination of the Allowable Flow Rate of a Drainage Geocomposite.

B. Qualifications:

1. INSTALLER shall attend pre-installation conference.

1.3 DEFINITIONS

A. MANUFACTURER: Manufacturer producing drainage composites from geonet cores and geotextiles.

B. INSTALLER: The Installers are the individuals actually performing the hands-on work in the field.

1.4 SUBMITTALS

INSTALLER/MANUFACTURER shall submit to OWNER the following documentation on the geocomposite or equal, as indicated below. Approval of the geocomposite will be made by OWNER, based on the documentation, at the sole discretion of OWNER. The geocomposite manufacturer quality control tests to be performed are outlined in Part 2.4 of this section.

A. As part of the bid a written statement listing:

1. A certification, accompanied by test results, that the geocomposite or equal material meets or exceed the minimum average roll property values listed in Table -1, for leachate collection and detection geocomposites; and

2. Production capacity available for this Contract.

B. A least 14 days prior to manufacturing the geonet component of the geocomposite, the following resin quality control data for each batch of resin assigned for use under this Contract shall be submitted. The certificates shall be signed by responsible parties employed by the manufacturer (such as the production manager). The following shall be included:

1. Copies of quality control certificates issued by the resin supplier including the production dates of the raw material and origin of the raw materials used to manufacture the geonet for this Contract;

2. Results of resin density and polymer melt index tests conducted by the resin supplier to verify the quality of the resin used to manufacture the geonet rolls assigned to this Contract and the origin of the resin and quality control certificates issued by the resin supplier; and

3. A written certification stating that no reclaimed polymer is added to the resin during the manufacture of the geonet assigned for this Contract.

C. At least 14 days prior to manufacture of the geotextile component of the geocomposite, manufacturing quality control certificates for each batch of resin and each shift's production shall be submitted. The certificates shall identify the origin of the resin and the manufacturer of the resin. The certificates shall be signed by responsible parties employed by the manufacturer (such as the production manager). The quality control certificate shall include:

1. roll numbers and identification;
2. sampling procedures; and
3. results of quality control tests, including a description of the test methods used.

D. Prior to shipment of the geocomposite, manufacturer's quality control certificates for each shift's production of geotextile, geonet, and geocomposite, signed by a responsible party employed by the manufacturer (such as the production manager) shall be submitted. The quality control certificate shall include roll numbers of the material manufactured.

E. Prior to shipment of the geocomposite, documentation shall be submitted that the geocomposite furnished meets the property values listed in Table -1, for leachate collection and leak detection geocomposites, that will:

1. Retain their structure during handling, placement, and long-term service; and
2. Be capable of withstanding direct exposure to sunlight for a minimum of 30 days with no measurable deterioration.

F. At least 14 days prior to shipment of the geocomposite, recommendations for unloading, field handling, and stockpiling of the geocomposite shall be submitted.

G. At least 14 days prior to shipment of the geocomposite, leachate compatibility test data shall be submitted to OWNER.

H. At least 14 days prior to shipping geocomposite, INSTALLER shall provide the following information to OWNER in writing, regarding the Geosynthetic INSTALLER:

1. Corporate background, qualifications and other information contained herein; and
2. Copy of installer's letter of approval or license by geocomposite manufacturer allowing the installer to install the geocomposite.

I. Friction angles of selected interfaces of liner for selected geosynthetics and site specific soil shall be submitted as specified in Part 2.3 at least 14 days prior to shipment of geosynthetics. The friction angles shall be above the minimum angles specified.

1.5 DELIVERY, STORAGE AND HANDLING

A. Label, handle, and store drainage composites in accordance with ASTM D4873 and as specified herein.

B. Wrap each roll in an opaque and waterproof layer of plastic during shipment and storage. Do not remove the plastic wrapping until deployment.

C. Label each roll with the manufacturer's name, drainage composite type, lot number, roll number, and roll dimensions (length, width, gross weight).

D. Repair or replace, as directed by the OWNER, drainage composite or plastic wrapping damaged as a result of storage or handling.

E. Do not expose drainage composite to temperatures in excess of 71 DegC (160 DegF) or below 0 DegC (32 DegF) unless recommended by the MANUFACTURER.

F. Do not use hooks, tongs or other sharp instruments for handling the drainage composite.

G. Do not lift rolls by use of cables or chains in contact with the drainage composite.

H. Do not drag drainage composite along the ground or across textured geomembranes.

PART 2 - PRODUCTS

2.1 ACCEPTABLE MANUFACTURERS

A. The recommended product for the project is 300 mil PermaNet UL Geocomposite as manufactured by Solmax. If an equivalent product is proposed, then the product information should be submitted with the bid per Part 2.2 D.

2.2 MATERIALS AND MANUFACTURE

A. Geonet Core:

1. The geonet core should be a standard product of the manufacturer. Specifically manufactured cores that meet project specifications are not acceptable.
2. Use non-thermally degraded polyethylene polymer which is clean and free of any foreign contaminants.
3. Manufactured geonet to conform to the property requirements listed in Table -1, and be free of defects including tears, nodules or other manufacturing defects which may affect its serviceability.

B. Geotextile:

1. Cover geonet core on both sides with a geotextile complying with requirements specified in Table -1.

C. Drainage Composite:

1. Create a composite by heat bonding geotextiles to the geonet. Minimum requirements for geocomposite are specified in Table -1, for leachate collection and leak detection geocomposites.

D. Equivalent Material:

1. DBT may submit a substitution request for equivalent material that meets the minimum requirements as specified in Table -1. Following should be considered when estimating the equivalency:
2. Minimum thickness 300 mil +/-15%.
3. Compression strength 45,000 psf (ASTM D6364)
4. Minimum transmissivity based on Table -1.
5. 10,000 hr Creep Reduction Factor for geonet core at 10,000 PSF and 40°C shall be less than 1.10.

2.3 SOURCE QUALITY CONTROL

A. Transmissivity Testing:

1. Measure transmissivity according to ASTM D 4716 as described in Table 1.
2. Attach geotextiles to the geonet in the same configuration as will be used in the field.
3. Boundary conditions for geocomposite are protective cover (or Ottawa Sand) interface on the upper geotextile and 60 mil HDPE geomembrane against the lower geotextile.
4. The testing shall be conducted at 10,000 psf loading for a minimum period of 100 hour.

B. Interface Friction Tests.

1. Refer to Section 02776-Geosynthetic Clay Layer (GCL) for test requirements.

2.4 MANUFACTURING QUALITY CONTROL

A. The gecomposite and its components shall be manufactured with quality control procedures that meet generally accepted industry standards.

B. The geocomposite MANUFACTURER shall sample and test the geocomposite, and its components, to demonstrate that the material complies with this Specification. Sampling shall, in general, be performed on sacrificial portions of the geocomposite material such that repair is not required.

C. The geocomposite MANUFACTURER shall sample and test the geocomposites, at the specified frequency, to demonstrate that its properties confirm to the values specified in Table-1for leachate collection and leak detection zones.

The geocomposite MANUFACTURER shall provide test results to OWNER demonstrating that the geocomposite MANUFACTURER performed the tests and that results were obtained that meet or exceed the transmissivity value required in Table 1, for leachate collection and leak detection zones.

D. Any geocomposite sample that does not comply with the Specifications will result in rejection of the roll from which the sample was obtained. MANUFACTURER shall replace any rejected rolls at no additional cost to OWNER.

E. If a geocomposite sample fails to meet the quality control requirements of this section, the geocomposite MANUFACTURER shall sample and test each roll manufactured in the same lot, or at the same time, as the failing roll. Sampling and testing of rolls shall continue until a pattern of acceptable test results is established.

F. Additional sample testing may be performed, at the geocomposite MANUFACTURER's discretion and expense, to more closely identify any non-complying rolls and/or to qualify individual rolls.

2.5 CONFORMANCE TESTING

Samples of the geocomposite will be removed by the CQA Consultant or CQA Laboratory Representative at MANUFACTURER's plant during production of the geocomposite. The CQA Consultant or CQA Laboratory Representative will sample the geotextile and geonet components before the components are bonded together and will sample the geocomposite material after the individual components are bonded. Samples will be sent to a geosynthetics CQA laboratory for testing to assure conformance with the requirements of this section. If mutually agreed upon by OWNER, DBT, and FDEP, samples may be shipped from MANUFACTURER to the CQA Officer's designated laboratory, provided that adequate chain-of-custody documentation is prepared and submitted, including origin and destination of sample, lot number, roll number, product name, project name, date of production, and date of shipment, at a minimum.

B. Samples and tests shall be selected by the CQA Officer in accordance with the procedures outlined in the CQA Plan.

C. Samples shall be taken at a minimum frequency of one sample per 100,000 square feet with a minimum of one sample per lot. If DBT ships geocomposite that requires sampling and testing at a frequency greater than one per 90,000 square feet, then DBT shall pay for the additional CQA sampling and testing beyond one per 90,000 square feet.

D. The CQA Officer may increase the frequency of sampling in the event that test results do not comply with the requirements of this section. DBT shall bear the expense of this additional testing.

E. As a minimum, tests listed in Table 1 shall be performed.

F. Any geocomposite material that is not certified in accordance with Part 1.4 of this section, or that conformance testing indicates do not comply with Part 2 of this section, will be rejected by OWNER. DBT shall replace the rejected material with new material, at no additional cost to OWNER

PART 3 - EXECUTION

3.1 EXAMINATION

A. Prior to placement of the drainage composite, clean the surface of all soil, rock, and other materials which could damage the composite.

B. The geocomposite drainage media shall be placed only on geomembrane that has been approved by the Geomembrane INSTALLER and accepted by the CQA Consultant.

3.2 INSTALLATION

A. Install geocomposite drain in accordance with manufacturer's written recommendations.

B. Deploy the drainage composite ensuring that the drainage composite and underlying materials are not damaged. Replace or repair faulty or damaged drainage composite as directed by OWNER.

C. Unroll drainage composite downslope keeping in slight tension to minimize wrinkles and folds.

D. Maintain free of dirt, mud, or any other foreign materials at all times during construction. Clean or replace rolls which are contaminated.

E. Place adequate ballast to prevent uplift by wind.

F. Overlap adjacent rolls a minimum of 6 inches. Overlap new drainage composite over existing as shown on the drawings.

G. Use manufacturer's fasteners to join adjacent rolls. Metallic fasteners will not be allowed. Space fasteners a maximum of 5 feet along downslope roll overlaps and a maximum of 1 foot along cross slope roll overlaps. Use fasteners of contrasting color from the drainage composite to facilitate visual inspection. Do not weld drainage composite to geomembranes.

H. Heat tack overlap of the upper geotextile to the upper geotextile of the adjacent rolls.

I. Any holes or tears in the geocomposite material shall be repaired by placing a patch extending 1 foot beyond the edges of the hole or tear. The patch shall be secured by tying fasteners through the bottom geotextile and the geonet of the patch, and through the top geotextile and geonet on the slope. The patch shall be secured every 6 inches with approved tying devices. The top geotextile component of the patch shall be thermally bonded to the top geotextile of the geocomposite needing repair. If the hole or tear width across the roll is more than 50 percent of the width of the roll, the entire damaged geocomposite panel shall be removed and replaced.

J. Penetration details shall be as recommended by the MANUFACTURER and as approved by the OWNER.

Table 1 Double Sided Gecomposite Property Values For Leachate Collection And Leak Detection Zone[1]

Properties	Qualifiers	Units	Values	Test Method	Test Frequency	
					MQC	CQA
Geonet Component						
Resin Polymer Melt Index	maximum	g/10 min.	1.0	ASTM D1238	1 per batch	n/a
Polymer Density (geonet)	Minimum	g/cc	0.94	ASTM D 1505	1/100,000 SF	1/100,000 SF
Carbon Black Content	range	%	2.0 - 3.0	ASTM D 1603	1/100,000 SF	1/100,000 SF
Thickness	minimum	mils	300 +/-15%	ASTM D 5199	1/100,000 SF	1/100,000 SF
Tensile Strength (MD and TD)	minimum	lb/in	100	ASTM D 5035	1/100,000 SF	1/100,000 SF
Geotextile Component						
Polymer Composition	minimum	%	95 polyester or polypropylene	-	n/a	n/a
Mass Per Unit Area	minimum	oz/yd^2	6	ASTM D 3776	1/100,000 SF	1/100,000 SF
Apparent Opening Size	maximum	US Sieve	#70	ASTM D 4751	1/100,000 SF	1/100,000 SF
Flow Rate[5]	minimum	$gal/min./ft^2$	110	ASTM D 4491	1/500,000 SF	n/a
Permittivity[5]	minimum	sec^{-1}	1.2	ASTM D 4491		1/500,000 SF
Grab Strength	minimum	lb	160	ASTM D 4632	1/100,000 SF	1/100,000 SF
CBR Puncture Strength	minimum	lb	435	ASTM D 6241	1/100,000 SF	1/100,000 SF
Tear Strength	minimum	lb	65	ASTM 4533	1/100,000 SF	1/100,000 SF
UV Resistance	minimum	% retained	70	ASTM D 4355 (after 300 hours)	n/a	n/a
Geocomposite						
Transmissivity@10,000 psf[3]	minimum	m^2/s	2.0×10^{-3}	ASTM D 4716	1/500,000 SF	1/500,000 SF
Ply Adhesion	minimum	lb/in.	1	ASTM F 904[4]	1/100,000 SF	1/100,000 SF

Notes:

(1) All values represent minimum average roll values (i.e., test results for samples collected from any roll in a lot should meet or exceed these values).

(2) The density of the net resin shall not exceed that of the geomembrane (see Section 02775).

(3) Transmissivity @ 100hour: Measure transmissivity using water at 68 DegF with a normal compressive load of 10,000 psf and a hydraulic gradient of 0.02 (ASTM D 4716). Boundary conditions are provided in Section 2.3. Measurements shall be taken 100 hour after application of confining pressure if satisfies the requirements given in Part 2.3A.

(4) Minimum of values measured in machine and cross machine directions with 1 in. clamp on Constant Rate of Extension (CRE) machine.

(5) Either flow rate requirement or permittivity requirement shall be met.

END OF SECTION

SECTION 02778
GEOTEXTILES

PART 1 - GENERAL

1.1 SUMMARY

A. Section Includes:
   1. Non-woven geotextile material (except for the geotextile bonded to geocomposite).
   2. Woven geotextile material.

B. Related Sections:
   1. Section 02300 – Earthwork
   2. Construction Quality Assurance Plan.

1.2 QUALITY ASSURANCE

A. Referenced Standards:
   1. American Association of State Highway Transportation Officials (AASHTO):
      a. M288, Standard Specification for Geotextile Specification for Highway Application.
   2. ASTM International (ASTM):
      a. D1987, Biological Clogging of Geotextile or Soil/Geotextile Filters.
      b. D3776, Test Method for Mass Per Unit Area of Woven Fabric.
      c. D3786, Test Method for Hydraulic Bursting Strength of Knitted Goods and Nonwoven Fabrics - Diaphragm Bursting Strength Tester Method.
      d. D4354, Sampling of Geosynthetics for Testing.
      e. D4355, Deterioration of Geotextiles from Exposure to Ultraviolet Light and Water (Xenon-Arc Type Apparatus).
      f. D4491, Water Permeability of Geotextiles by Permittivity.
      g. D4533, Trapezoid Tearing Strength of Geotextiles.
      h. D4595, Tensile Properties of Geotextiles by the Wide-Width Strip Method.
      i. D4632, Grab Breaking Load and Elongation of Geotextiles.
      j. D4751, Determining Apparent Opening Size of A Geotextile.
      k. D4759, Determining the Specification Conformance of Geosynthetics.
      l. D4833, Index Puncture Resistance of Geotextiles, Geomembranes, and Related Products.
      m. D4873, Identification, Storage, and Handling of Geosynthetic Rolls.
      n. D5261, Test Method for Measuring Mass Per Unit Area of Geotextiles.
      o. D6193, Standard Practice for Stitches and Seams.

B. Qualifications:
   1. Each manufacturing, fabricating firm shall demonstrate 5 years continuous experience, including a minimum of 10,000,000 SF of geotextile installation in the past 3 years.

2. Installing firm shall demonstrate that the site Superintendent or Foreman has had responsible charge for installation geotextile on several projects of a similar scope.
3. Installer shall attend pre-installation conference.

1.3 DEFINITIONS

A. Manufacturer: Manufacturer producing geotextile sheets from resin and additives.

B. Installer: The Installers are the individuals actually performing the hands-on work in the field.

C. MARV: Minimum Average Roll Value

1.4 SUBMITTALS

A. Shop Drawings:
1. Manufacturer's documentation that raw materials and roll materials comply with required geotextile physical properties.
2. Manufacturer and Installer quality control manuals.
3. Original test results for resins, roll material and factory seam tests at frequency specified in respective quality control manuals. Results shall include or bracket the rolls delivered for use in the Work.
4. Proposed details of anchoring and overlapping if different than included in Contract Documents.

B. Miscellaneous Submittals:
1. Provide same certifications specified in Section 02777. No alterations, additions, deletions, or exception shall be made to the specified language.
2. For needle punched geotextiles, the Manufacturer shall certify that the geotextile has been continuously inspected using permanent on-line full-width metal detectors and does not contain any needles which could damage other geosynthetic layers.
3. Qualification documentation.

1.5 DELIVERY, STORAGE AND HANDLING

A. Label, handle, and store geotextiles in accordance with ASTM D4873 and as specified herein.

B. Wrap each roll in an opaque and waterproof layer of plastic during shipment and storage. Do not remove the plastic wrapping until deployment.

C. Label each roll with the manufacturer's name, geotextile type, lot number, roll number, and roll dimensions (length, width, gross weight).

D. Repair or replace geotextile or plastic wrapping damaged as a result of storage or handling, as directed.

E. Do not expose geotextile to temperatures in excess of 71 DegC (160 DegF) or less than 0 DegC (32 DegF) unless recommended by the manufacturer.

F. Do not use hooks, tongs or other sharp instruments for handling geotextile. Do not lift rolls lifted by use of cables or chains in contact with the geotextile. Do not drag geotextile along the ground.

PART 2 - PRODUCTS

2.1 ACCEPTABLE MANUFACTURERS

A. Subject to compliance with the Contract Documents, the following Manufacturers are acceptable:
   1. Solmax
   2. Thrace-LINQ
   3. Mirafi (Ten Cate Geosynthetics)
   4. Propex
   5. SKAPS Industries
   6. Tenax Corp

2.2 MATERIALS AND MANUFACTURE

A. Geotextile:
   1. Non-woven pervious sheet of polymeric material.
   2. Geotextile fibers:
      a. Long-chain synthetic polymer composed of at least 85 percent by weight polyolefins, polyesters, or polyamides.
      b. Filaments resistant to deterioration by ultraviolet light, oxidation, and heat exposure.
      c. Do not use reclaimed or recycled fibers or polymer to the formulation.
   3. Form geotextile into a network such that the filaments or yarns retain dimensional stability relative to each other, including the selvages.
   4. The geotextile physical properties shall equal or exceed the minimum average roll values listed in Tables 1 and 2. Values shown are for the weaker principal direction. Acceptance of geotextile shall be in accordance with ASTM D4759.

Table 1 Cushion Geotextile (nonwoven, needlepunched) Properties

PROPERTY	TEST METHOD	MARV	MIN. MQC TEST FREQUENCY
Mass per Unit Area, oz/sy	ASTM D5261	16	1 per 100,000 ft^2
Ultraviolet Degradation, % retained @ 500 hrs	ASTM D4355	70	1per 100,000 ft^2
Trapezoidal Tear, lbs	ASTM D4533	145	1 per 100,000 ft^2
Grab Tensile, lbs	ASTM D4632	370	1 per 100,000 ft^2
Grab Elongation, %	ASTM D4632	50	1 per 100,000 ft^2
Puncture Strength, lbs	ASTM D4833	170	1 per 100,000 ft^2

Table 2 Separation Geotextile (woven, monofilament) Properties

PROPERTY	TEST METHOD	MARV	MIN. MQC TEST FREQUENCY
Ultraviolet Stability, % retained @ 500 hrs	ASTM D4355	50%	1per formulation
Tear Strength, lbs	ASTM D4533	110	1 per 100,000 ft^2
Puncture Strength , lbs	ASTMD4833	110	1 per 540,000 ft^2
Grab Tensile, lbs	ASTM D4632	310	1 per 100,000 ft^2
Sewn Seam Strength, lbs	ASTM D4632	283	1 per 100,000 ft^2
AOS, in. (max ave. roll value)	ASTM D4751	0.024	1 per 540,000 ft^2
Burst Strength, psi	ASTM D3786	500	1 per 540,000 ft^2
Permitivity, sec^{-1}	ASTM D4491	0.02	1 per 540,000 ft^2

B. Thread:
   1. High-strength polyester, nylon, or other approved thread type.
   2. Equivalent chemical compatibility and ultraviolet light stability as the geotextile.
   3. Contrasting color with the geotextile.

2.06 CONFORMANCE TESTING

A. Samples of the geotextile shall be removed by the CQA Consultant and sent to a geosynthetics CQA laboratory for testing to assure conformance with the requirements of this section. This testing shall be completed prior to deployment of geotextile. If mutually agreed upon by OWNER, ENGINEER, CONTRACTOR, and FDEP, samples may be shipped from Manufacturer to the CQA Consultant's designated laboratory, provided that adequate chain-of-custody documentation is prepared and submitted, including origin and destination of sample, lot number, roll number, product name, project name, date of production, and date of shipment, at a minimum.

B. Samples and tests will be selected by the CQA Consultant in accordance with the procedures outlined in the CQA Plan.

C. Samples shall be taken at a minimum frequency of one sample per 100,000 ft^2 with a minimum of one sample per lot. If the Geotextile Manufacturer ships geotextile that requires sampling and testing at a frequency greater than one per 100,000 ft^2, then the Geotextile Manufacturer shall pay for the cost of the additional CQA sampling and testing beyond one sample per 100,000 ft^2.

D. The CQA Consultant may increase the frequency of sampling in the event that test results do not comply with requirements of Part 2.01 of this section. The Geotextile Manufacturer or CONTRACTOR shall bear the expense of this additional testing.

E. Any geotextile that is not certified in accordance with Part 1.03 of this section, or that conformance testing indicates does not comply with Part 2.01 of this section, will be rejected by the CQA Consultant. The Geotextile Manufacturer shall replace the rejected material with new material, at no additional cost to OWNER.

PART 3 - EXECUTION

3.1 PREPARATION

A. Construct the surface underlying the geotextiles smooth and free of ruts or protrusions which could damage the geotextiles.

3.2 INSTALLATION

A. Install geotextiles in accordance with manufacturer's written recommendations.

B. Handling shall be in accordance with ASTM D4873.

C. No equipment will be permitted to traffic in direct contact with the geotextile.

D. Lay geotextile smooth so as to be free of tensile stresses, folds, and wrinkles.

E. Protect geotextiles from clogging, tears, and other damage during installation.

F. Overlapping geotextiles shall be thermally bonded.

G. Geotextile Repair:
   1. Place a patch of the same type of geotextile which extends a minimum of 12 inches beyond the edge of the damage or defect.
   2. Fasten patches continuously using thermal boding, sewn seam or other approved method.
   3. Align machine direction of the patch with the machine direction of the geotextile being repaired.
   4. Replace geotextile which cannot be repaired.

H. Use adequate ballast (e.g. sand bags) to prevent uplift by wind.

I. Do not use staples or pins to hold the geotextile in place.

J. Geotextile left uncovered for more than 14 days shall be removed and replaced at the Contractor’s expense.

END OF SECTION

DIVISION 15

MECHANICAL

SECTION 15100

PUMPS

Part 1 – General

1.01 Summary

1. This section covers the work necessary to supply and installed leachate collection, leachate detection and pump station pumps

1.02 Quality Assurance

A. Referenced Standards:
   1. American Iron and Steel Institute (AISI)
   2. American Standards Institute (ANSI)
   3. American Society for Testing and Materials (ASTM)
   4. Factory Mutual (FM)
   5. Hydraulic Institute Standards for Centrifugal, Rotary and Reciprocating Pumps (HI)
   6. National Electrical Manufacturer's Association (NEMA)
      a. 250, Enclosures for Electrical Equipment (1000) Volts Maximum)
      b. ICS 6, Enclosures for Industrial Controls
   7. National Fire Protection Agency (NAFPA)
   8. Underwriters Laboratories, Inc. (UL)
   9. It is the intentions of these Specifications that the pump, control panel, level controls and flow meter are provided by a sole manufacturer with in-house testing facilities.

1.03 Submittals

A. Shop Drawings
B. Operation and Maintenance Manuals

Part 2 – Products

2.01 Leachate pumping system, including sensors and control panels will be designed and supplied by EPG Companies Inc., Maple Grove, MN (800-443-7426).

2.02 Materials

A. Furnish unit component meeting the following material specifications.
   1. EPG Model WSDPT SurePump™ Wheeled Sump Drainer with center mounted transducer holder for the sideslope risers and EPG Model VSDPT SurePumpTM Vertical Sump Drainer with built in level sensor for pump stations.
      a. Major components made of 304 stainless Steel

b. Built –in check valve with nonmetallic seat
c. E-Glide™ bearings and seals. Teflon or rubber not acceptable.
d. Stainless steel shaft rotating on E-Glide™ bearings that are fluid lubricated.
e. The diffuser chambers for each impeller shall be 304 stainless steel and fitted with E-Glide™ impeller seal rings.
f. The motor shall be submersible, stainless steel, hermetically sealed and capable of sustaining up to 100 starts per day.
g. The motor shall be connected to the pump by a motor adaptor and coupling of 304 stainless steel.
h. The motor lead shall be of non -splice with EPG's "CP" waterproof and chemically resistant jacket over 600-Volt insulation and be of the length as determined by DBT.

2.03 Equipment

A. Performance Requirements

1. Submersible Cell 6 Leachate Collection Pump EPG model WSDPT 46-1
   a. Design condition: 176 gpm at 40 ft. TDH
   b. Maximum horse power 5.0 HP
   c. Drive Type: Constant speed.
   d. Threaded discharge nozzle: 3 inch NPT
   e. Submersible pump motor: 460 Volts, 3 phase, 60 Hertz

2. Submersible Cell 7 Leachate Collection Pump EPG model WSDPT 30-1
   a. Design condition: 116 gpm at 40 ft. TDH
   b. Maximum horse power 2.0 HP
   c. Drive Type: Constant speed.
   d. Threaded discharge nozzle: 3 inch MNPT
   e. Submersible pump motor: 460 Volts, 3 phase, 60 Hertz

3. Submersible Leachate Detection Pump EPG model WSDPT 8-2
   a. Design condition: 34 gpm at 40 ft. TDH
   b. Maximum horse power 0.75 HP
   c. Drive Type: Constant speed.
   d. Threaded discharge nozzle: 1. 5 inch NPT
   e. Submersible pump motor: 460 Volts, 3 phase, 60 Hertz

4. Submersible Vertical Sump Drainer at PS#1 EPG model VSDPT 95-2
   a. Design condition: 390 gpm at 120 ft. TDH
   b. Maximum horse power 20 HP
   c. Drive Type: Constant speed.

d. Threaded discharge nozzle: 6 inch NPT
e. Submersible pump motor: 460 Volts, 3 phase, 60 Hertz

5. Submersible Vertical Sump Drainer at PS#3 EPG model VSDPT 8-2
   a. Design condition: 34 gpm at 40 ft. TDH
   b. Maximum horse power 0.75 HP
   c. Drive Type: Constant speed.
   d. Threaded discharge nozzle: 1.5 inch NPT
   e. Submersible pump motor: 460 Volts, 3 phase, 60 Hertz

2.04 Accessories

A. Sump Drainer
   1. Pump shall include a stainless steel sump drainer housing with wheels on both ends.
   2. A removable transmitter mount shall be installed at the center bottom of the Sump Drainer for liquid level control.
   3. Provide stainless steel suspension cable of sufficient strength with stainless steel connections.
   4. The stainless steel horizontal Sump Drainer shall permit the unit to "pump down" to within 10 inches of the sump bottom for collection sump and within 8 inches of the sump bottom for detection sump. The vertical pumps shall be pump down to minimum levels recommended by the manufacturer.

B. Level Sensor
   1. The submersible pressure transmitter level sensor shall have adequate range of operation with a 4-20MA output signal. Transmitter constructed with a stainless steel body, stainless steel diaphragm and Viton seals. Signal cable must be vented and chemical resistant. Transmitter circuit must be protected by intrinsically safe barriers.

C. Breakout Junction Boxes
   1. Furnish breakout box EPG Model BJBP500 for single pump system and BJB525 dual pump system and one EPG Model BJBL625B dual pump system breakout box for the two level sensors in the sump including a vented cable with desiccant-filled filter and bellows. Supply and install a separate EPG Model BJBL600B breakout box for level sensor in PS#3 and EPG Model BJBL625B breakout box for level sensor in PS#1. Enclosures shall be NEMA4X non-metallic or stainless steel.

D. Discharge Hose and Fittings
   1. Furnish and provide a complete discharge piping assembly to connect the pump to the existing piping as shown on the plans.
   2. Discharge hose shall be rated at 300 psi and -20 to 180 degrees Fahrenheit.
   3. Furnish stainless steel discharge adapters with NPT fittings.
   4. Provide EPG Guide rail assembly as shown on the drawings.

2.05 Control Panel

A. The leachate pump control panel shall be manufactured by the pump manufacturer. Furnish and install three EPG Companies Inc., UL listed 508A/698A, Series YCL975PT Control Panel with the single Operator Control Station (OCS) with YCL Series pump controller/liquid level meter to operate two pump motor and auxiliary equipment in manual or automatic mode for each Cells 6 Sump, Cell 7 Sump and YCL950PT Series pump controller/liquid level meter to operate two pump motor in lead/lag alternating and auxiliary equipment in manual or automatic mode for PS#1 pump station. The pump control panel at PS#3 shall be EPG Companies Inc., UL listed 508A/698A, Series YCL925PT with the single Operator Control Station (OCS) with YCL Series pump controller/liquid level meter to operate pump motor and auxiliary equipment in manual or automatic mode
B. The control panel enclosure shall be NEMA type 4X. The enclosure shall be equipped with a window in the outer door, an inner door, a stainless steel drip shield, and a tamper resistant latch. The NEMA 4X enclosure shall be stainless steel. Type 2 coordinated protection shall be provided for pump control components and will be sized to operate pump motors of the specified horsepower.
C. The main power and motor protection shall be fused per National Electrical Code (NEC) Article 409 to limit Arc Flash potential. The main power switch shall prevent opening of the control panel while power is on.
D. The "Hand-Off-Auto" selector switch allows manual or automatic operation. The selector switch shall be heavy duty; oil tight, NEMA 4 rated and shall be mounted on the inner door. The hand position shall be momentary with a spring return.
E. The motor starter shall be sized to the pump motor horsepower, and shall be equipped with built in, single phasing protection and an ambient compensated, quick-trip, Class 10, adjustable thermal overload.
F. The control transformer with fused primary and secondary shall isolate control circuit from power circuit and provide easier and safer field wiring of accessories. It shall lower incoming voltage to 120 Volts.
G. The run light shall indicate energization of motor circuit. It shall be heavy duty, oil tight, and NEMA 4 rated. The light shall be mounted on the inner door and will be green in color.
H. The motor overload light (three phase only) shall indicate motor not running due to overload condition. It shall be heavy duty, oil tight, and NEMA 4 rated. The light shall be mounted on the inner door and will be red in color.
I. Alarm lights shall indicate high sump level or pump remote disable. They shall be heavy duty, oil tight, and NEMA 4 rated. The lights shall be mounted on the inner door and will be red in color.
J. All interior panel lights to be LED.

K. An optional red top mounted light shall be provided and activated by any alarm condition.
L. An optional non-resettable elapsed time meter shall be mounted on the inner door and will record total pump run time in hours.
M. Control Panel must display a SCCR (short circuit current rating) per UL specifications.
N. The level simulator shall be mounted on the inner door. The level simulator shall be a built-in test circuit designed to simulate 4-20 mA loads to assist in level setup and troubleshooting.
O. The level simulator for the dewatering riser shall be mounted on the inner door. The dewatering level simulator shall be connected to a light that indicate high water level of the dewatering riser.
P. The level sensor circuit shall be protected by intrinsically safe barriers.
Q. A heater with adjustable thermostat shall promote even distribution of heat and elimination of hot spots and condensation. The heater element shall be mounted in space between the sub-panel and the back of the enclosure and provide a minimum of 100 inches square of heating area.
R. A lightning arrestor shall be internally mounted and shall be grounded, metal-to-metal, to water strata. When properly grounded, the lightning arrestor will protect electrical equipment against lightning induced surges.
S. A labeled and numbered terminal strip shall provide easy connection of external components.
T. Inclusion of an industrial corrosion inhibitor emitter shall protect internal components of control panel from corrosion for up to one year and shall be replaceable.

2.06 System Logic and Function

A. The controller is designed to start and stop a pump using the level control meter with a submersible pressure transmitter. The pump starts at the pump start level set point and continues to run until the liquid level decreases to the pump stop level set point as programmed in the level control meter. If the liquid level rises to the high level alarm set point, a high level will be annunciated. If the liquid level rises to the high-high level fail-safe set point, the pump motor will shut off. The pressure transmitter level sensor shall have a range of 0 to 11.5 feet with a 4-20 mA output signal with 125 foot Poly Lead. An elapsed time meter monitors total pump run time in hours. A pump remote enable (i.e. tank full) dry input contact with pass-along is also provided. The flow meter display flow rate and total flow from a paddlewheel flow sensor. The pump at PS#1 should have a mechanism to automatic shut down when the leachate storage tanks are full to maximum storage elevation with an alarming mechanism.

Part 3 – Execution

3.01 Source Quality Control

A. Test pump for 30 minutes in a tank operated by control panel.
B. Attach motor to cable and test for moisture seal to 150 psi.

3.02 Installation

A. Pump and control systems shall be installed by a factory trained installer.
B. Installation shall be properly grounded to meet all electrical codes.

3.03 Warranty

A. Wheeled Sump Drainer, Control Panel and accessories manufactured by EPG Companies and installed as a complete system will be warranted for a period of 36 months from date of manufacturer.
B. Manufacturer to submit warranty with any required “as built shop drawings” and Operation and Maintenance Manuals.

END OF SECTION

SECTION 15160

FLOW METER AND PRESSURE GAUGES

PART 1 GENERAL

1.01 WORK INCLUDED

This section covers the work necessary to furnish and install flow meter assemblies and pressured gauges in the proposed detection and leachate collection pumping systems as indicated on the drawings.

1.02 RELATED WORK

A. The Standard General and Supplemental Conditions of these specifications are a part of this section as if incorporated herein.

B. Other related specification sections contained herein are listed below:

1. Section 15200, Pipe and Fittings

1.03 SUBMITTALS

Contractor shall submit printed manufacturer's product data, Schedules and Reports.

1.04 PROTECTION DURING STORAGE

The flow meter assemblies shall be stored in a manner that will protect them from the environment.

PART 2 PRODUCTS

2.01 GENERAL

A. The flow meter shall consist of the following:
   a. McCROMETER MW502 2-inch (Serial No: 15-07034) flow meter for leachate detection riser outlet pipe.
   b. McCROMETER MW504 4-inch (Serial No: 15-07035) flow meter for leachate collection riser outlet pipe.
   c. McCROMETER MZ506 6-inch flow meter for PS#1pump station.

B. The pressure gauge shall be 2-inch stainless steel encased Bourdon tube pressure gauges with an accuracy of ± 1.6%.

2.02 FLOW METER FOR LEACHATE COLLECTION AND LEAK DETECTION PUMPS

A. Install flow meters in the locations shown on the drawings and as herein specified with all equipment and necessary appurtenances to provide complete and working systems.

2.03 PUMP STATION PRESSURE GAUGES

The gauges shall be glycerin filled, 2-inch stainless steel encased Bourdon tube pressure gauges with an accuracy of ± 1.6%. Other requirements are listed below.

- Range for the pump discharge shall be 0 - 60 psig except for the pump at PS#1 where the range shall be 0-100 psig.
- Dial shall be white painted aluminum with a black anodized pointer and a Plexiglas window.
- Wetted parts shall be brass, but the sensing tube shall be 316L SS.
- Exterior movement and housing shall be 304 SS.
- Seals shall be Buna N for the housing, EPDM for the block, and Monoprene for the lens.
- Protection shall be NEMA 4, UL listed.
- The bottom fitting shall be ¼" NPT.

The gauges shall be as manufactured by Kobold Instruments, Inc. Pittsburgh PA., or an OWNER approved equal.

PART 3 EXECUTION

3.01 INSTALLATION OF FLOW METER

A. The flow meter assemblies shall be installed in the HDPE pipe as shown on the drawings.

B. The Contractor shall be responsible for installation and in-place calibration of flow meters and related equipment. The Contractor shall provide written certification that the flow meter assemblies are properly calibrated and tested and meet the accuracy limits previously stated herein.

3.02 INSTALLATION OF PRESSURE GAUGES

The gauges shall be installed where indicated on the drawings. The gauges shall be mounted in a vertical position and shall face perimeter road. The threaded connection to the piping system shall be sealed to prevent any leaks due to expansion/contraction.

END OF SECTION

SECTION 15200
PIPE AND PIPE FITTINGS

PART 1 - GENERAL

1.1 WORK INCLUDED

Provide and install piping systems as shown on the Drawings including fittings, accessories, and other appurtenances.

1.2 RELATED WORK

A. The General and Supplemental Conditions of these specifications are a part of this section as if incorporated herein.

B. Other related specification sections contained herein are as listed below.
   1. Section 02300 Earthwork

1.3 QUALITY ASSURANCE

A. Piping materials and manufacture shall adhere to the standards referenced herein.

B. Manufacturer's written installation, joining, and connection instructions shall be kept on site during all piping operations and shall be strictly adhered to.

C. Joining and laying of HDPE pipe is to be performed only by a CONTRACTOR or subcontractor with proven experience in working with HDPE pipe.

D. Upon completion of the non-pressurized HDPE piping, protective cover, and leachate collection gravel installation, the non-pressurized HDPE piping systems shall be inspected via videotaping and cleaned via high-pressure water-jetting. The CONTRACTOR or subcontractor shall have demonstrated experience water-jetting and video inspection of landfill leachate collection pipes. The videotape(s) shall be accompanied by a certification from the video operator that the piping systems are free of blockages and soil deposits. Cameras and equipment used for piping system cleaning and videotaping shall be explosion-proof, NEC Class 1, Division 1, Groups C and D. A letter report addressed from the water-jetting firm which contains a summary of the work and the results from the water-jetting for the new cell shall be submitted to the Engineer two weeks after water-jetting is completed. The Engineer and Owner shall be notified seven days ahead of time of the scheduled video inspection and water-jetting.

1.4 SUBMITTALS AND TESTING

Submittals shall be in sufficient detail to show full compliance with these specifications.

A. The manufacturer's product data shall be provided for each pipe material which will become a permanent part of the work and shall include all fittings and accessories.

B. The CONTRACTOR shall submit installer qualifications and experience for himself or subcontractor with regard to HDPE pipe installation for record purposes.

C. The CONTRACTOR shall provide qualifications of the proposed firm(s) to be used in the jet-cleaning and video inspection of the HDPE piping for record purposes. The CONTRACTOR shall also submit the results of the cleaning two weeks after the jet-cleaning. Videotape shall be submitted upon system completion and shall be clearly labeled with video taping date, pipe starting and ending point, name, address, and phone number of firm performing the videotaping. Each section of pipe that is videotaped shall have a complete dialog dubbed onto the video tape which, as a minimum, describes: length of pipe videotaped; location of pipe videotaped; all welds; deflections; other features of interest.

D. Pressure test reports which include time, date, section tested, pressure reading, ambient temperature at time of pressure reading and witnesses to testing.

E. Drawings containing as-constructed pipe invert elevations shall be submitted for record purposes as specified herein.

1.5 HANDLING, STORAGE, INSPECTION, AND PROTECTION

A. Care shall be taken during transport of all pipes to protect pipe from kinks, cuts, end damage, and other defects. Binding and tie-down methods shall not damage or deflect pipe in any way. Pipe damaged during shipment shall be rejected.

B. Pipe shall be stored on level ground, preferably turf or sand, free of sharp objects which could damage the pipe. Stacking of any pipe shall be limited to a height that will not cause deformation of the lower layers of pipe under anticipated temperature conditions. When necessary due to ground conditions, the pipe shall be stored on wooden sleepers, spaced suitably and of such widths to prevent deformation of the pipe at the point of contact with the sleeper or between supports. Pipe shall not be removed from storage until bedding or subgrade work is complete and ready to receive the pipe.

C. The handling of the joined pipe shall be in such a manner that the pipe is not damaged by dragging it over sharp and cutting objects. Ropes, fabric, or rubber-protected slings and straps shall be used when handling pipe. Chains, cables, or hooks inserted into the pipe ends shall not be used. Two slings spread apart shall be used for lifting each length of pipe. Pipe or fittings shall not be dropped. Slings for handling the joined pipe shall not be positioned at butt-fused joints. Sections of the pipes with cuts and gouges shall be removed and the ends of the pipe rejoined.

D. Any plastic-type pipe stored on the job site shall be covered with canvas or other opaque material to protect it from sunlight. Air circulation shall be provided under the covering. HDPE pipe may be excluded from this requirement as recommended by the manufacturer.

E. All pipe, fittings, and other accessories shall be inspected upon delivery and during the course of the work. Any defective or damaged materials found during field inspection or during tests shall be removed from the site and replaced by, and at the expense of, the CONTRACTOR.

F. The interior of all pipe, fittings, and other accessories shall be kept free from dirt and foreign matter at all times. Fittings shall be drained and stored in a manner that will protect them from damage by freezing.

G. Manufacturer's written instructions regarding handling and storage of pipe and fittings shall be adhered to and shall be kept on site for inspection as required.

1.6 REFERENCE STANDARDS

Submittals shall be in sufficient detail to show full compliance with these specifications.

A. Reference standards and recommended practices referred to herein shall be the latest revision of any such document.

B. Standards referenced herein are as listed below.

1. ANSI B16.5 Pipe Flanges and Flanged Fittings
2. ANSI B18.2.1 Square and Hex Bolts and Screws
3. ASTM A 182 Forged or Rolled Alloy-Steel Pipe Flanges, Forged Fittings, and Valves and Parts for High-Temperature Service
4. ASTM A 193 Alloy-Steel and Stainless Steel Bolting Materials for High-Temperature Service
5. ASTM A 194 Carbon and Alloy Steel Nuts for Bolts for High-Pressure and High-Temperature Service
6. ASTM A 276 Stainless and Heat-Resisting Steel Bars and Shapes
7. ASTM A 312 Seamless and Welded Austenitic Stainless Steel Pipe
8. ASTM A 774 As-Welded Wrought Austenitic Stainless Steel Fittings for General Corrosive Service at Low and Moderate Temperatures
9. ASTM A 778 Welded, Unannealed Austenitic Stainless Steel Tubular Products
10. ASTM D 256 Impact Resistance of Plastics and Electrical Insulating Materials
11. ASTM D 638 Tensile Properties of Plastics
12. ASTM D 696 Coefficient of Linear Thermal Expansion of Plastics
13. ASTM D 746 Brittleness Temperature of Plastics and Elastomers by impact
14. ASTM D 1248 Polyethylene Plastics Molding and Extrusion Materials
15. ASTM D 1525 Vicat Softening Temperature of Plastics
16. ASTM D 2240 Rubber Property-Durometer Hardness
17. ASTM D 2837 Obtaining Hydrostatic Design Basis for Thermoplastic Pipe Materials
18. ASTM D 3350 Polyethylene Plastics Pipe and Fittings Material
19. ASTM F 714 Polyethylene (PE) Plastic Pipe (SDR-PR) Based on Outside Diameter

20. ASTM D 1784 Rigid Poly(Vinyl Chloride) (PVC) Compounds and Chlorinated Poly(Vinyl Chloride) (CPVC) Compounds.
21. ASTM D 1785 Poly(Vinyl Chloride) PVC Plastic Pipe, Schedules 40, 80 and 120.
23. ASTM D 2467 Poly(Vinyl Chloride) (PVC) Plastic Pipe Fittings, Schedule 80.
24. ASTM D 3034 Type PSM Poly(Vinyl Chloride) (PVC) Sewer Pipe and Fittings.
25. ASTM D 3139 Joints for Plastic Pressure Pipes Using Flexible Elastomeric Seals.
26. ASTM D 3212 Joints for Drain and Sewer Plastic Pipes Using Flexible Elastomeric Seals.
27. ASTM F 593 Stainless Steel Bolts, Hex Cap Screws, and Studs.
28. ASTM F 679, Poly(Vinyl Chloride) (PVC) Large-Diameter Plastic Gravity Sewer Pipe and Fittings.
29. ASTM F 794 Poly(Vinyl Chloride) (PVC) Profile Gravity Sewer Pipe and Fittings Based on Controlled Inside Diameter.
30. ASTM F 949 Poly(Vinyl Chloride) (PVC) Corrugated Sewer Pipe with a Smooth Interior and Fittings.

PART 2 - PRODUCTS

2.1 GENERAL

A. All pipe joints and fittings shall have the same or greater strength, pressure ratings, thermal resistance, chemical resistance, and other pertinent properties as the pipe being joined or connected. Plastic pipe fittings shall be manufactured of the same resin as used in the manufacture of the pipe being joined.

B. Fittings for use with plastic-type pipe material herein specified shall be provided by the same manufacturer providing the pipe.

C. Each pipe length shall be clearly marked with manufacturer's name or trademark, applicable ASTM ratings, size, and, where applicable, standard dimension ratio.

D. All flange bolts, nuts, and washers shall be AISI Type 316 stainless steel, ASTM A 193, Grade B8M hex head bolts and ASTM A 194, Grade 8M hex head nuts. Bolts shall be fabricated in accordance with ANSI B18.2.1 and shall be provided with washers.

E. Field threading of pipes is not allowed.

F. Where threaded fittings are permitted, thread lubricant shall be Teflon tape.

G. Identify each length of pipe clearly at intervals of 5 FT or less.
   1. Include manufacturer's name and trademark.
   2. Nominal size of pipe, standard dimension ratio, appurtenant information regarding polymer cell classification and critical identifications regarding performance specifications and NSF approvals when applicable.

2.2 POLYVINYL CHLORIDE (PVC) PIPING MATERIALS

A. General:
   1. Provide Schedule 80 pipe with Schedule 80 fittings and appurtenances at locations shown on Drawings. Note that the proposed pipe is connected to existing pipe 8-inch Schedule 35 PVC.
   2. Furnish materials in full compliance to following material specifications:
      a. Manufacture pipe, fittings and appurtenances from polyvinyl chloride (PVC) compound which meets the requirements of Type 1, Grade 1 (12454-B) Polyvinyl Chloride as outlined in ASTM D1784.
      b. Manufacture pipe, fittings and valves from materials that have been tested and approved for conveying potable water by the NSF.

B. Pipe:
   1. Furnish pipe meeting requirements of ASTM D1785.
   2. Pipe 2 IN and less to be solvent welded.
   3. Pipe larger than 2 IN may be either flanged or solvent welded unless shown otherwise on Drawings.

C. Fittings: Provide ASTM D2467 PVC socket type fittings having the same pressure and temperature rating as the pipe.

D. Flanges/Unions:
   1. Furnish flanges and unions at locations shown on Drawings.
   2. Provide either flanges or unions at valves, penetrations through structures and equipment connections.
   3. For pipe larger than 2 IN, provide 150 LB socket type PVC flange.
   4. For pipe 2 IN and less, provide socket type PVC union with Buna O-rings.
   5. Use flat, full faced natural rubber gaskets at flanged connections.
      a. Furnish heavy hex head bolts, each with one (1) heavy hex nut, ASTM F593 Type 316 stainless steel.
   6. Use spacers supplied by pipe manufacturer when mating raised-faced flanges to other flanges.

2.3 HIGH DENSITY POLYETHYLENE (HDPE) PIPING MATERIALS

A. Black PE materials used for the manufacture of polyethylene pipe, tube and fittings shall be PE 4710 high density polyethylene meeting ASTM D3350 cell classification 445574C and shall be listed in the name of the pipe and fitting Manufacturer in PPI (Plastics Pipe Institute) TR-4 with a standard grade HDB rating of 1600 psi at 73°F.. Additional requirements are listed below.

Table 1 Pipe Material Physical Properties

Property	Standard	Typical Value
Cell Classification	ASTM D3350	445574C (black)
Density	ASTM D1505	0.960 g/cc (black)
Melt Index	ASTM D1238	0.08 g/10 min
Flexural Modulus	ASTM D790	>120,000 psi
Tensile Strength	ASTM D635 Type IV	> 3500 psi
SCG (PENT)	ASTM F1473	>500 hours
HDB at 73^0F	ASTM D2837	1600 psi
Color; UV stabilizer	ASTM D3350	Black

B. The HDPE pipe and fittings shall be homogeneous throughout and free of visible cracks, holes, foreign inclusions, or other injurious defects. Any pipe or fitting with nicks, scrapes, or gouges deeper than 5 percent of the nominal wall thickness shall be rejected.

C. HDPE piping shall be provided in the sizes and shall have a Standard Dimension Ratio (SDR) as indicated on the drawings. Dimensions and workmanship shall conform to the requirements of ASTM F 714.

D. HDPE pipe and fitting joints shall be butt, heat fusion weld type only. No extrusion welds shall be permitted unless specifically noted on the drawings or specified herein. These criteria shall also apply to both the carrier piping and the containment piping for double-wall piping systems. All welds shall achieve full penetration.

E. HDPE fittings for single-wall piping systems shall be either molded or shop-fabricated type. Shop-fabricated fittings shall be made from pipe sections using butt, heat fusion welding. Fittings with extrusion welding shall not be permitted.

F. HDPE fittings used in single-wall piping systems and in carrier piping of double-wall piping systems shall be free of internal projections. Any and all internal projections resulting from molding or heat fusion fabrication shall be removed flush to the interior surface of the fitting at the factory or shop.

G. Flanged joints, where required, shall utilize molded HDPE flange adapters and 316 stainless steel backup rings with ANSI B16.5 drilling. Flange bolting shall be as specified herein. Flanged HDPE joints shall have flat ring type Viton gaskets with gasket thickness and hardness as recommended by the manufacturer.

H. Molded fittings shall be manufactured and tested in accordance with ASTM D 3261 and shall be so marked. Molded fittings shall be tested in accordance with AWWA C906.

I. Fabricated fittings shall be made by heat fusion joining specially machined shapes cut from pipe, polyethylene sheet stock or molded fittings. Fabricated fittings shall be rated for internal pressure service at least equal to the full service pressure rating of the mating pipe. Fabricated fittings shall be tested in accordance with AWWA C906.

J. All fusion welding beads shall be ground smooth upon completion of welding.

2.4 PERFORATED HDPE PIPING MATERIALS

A. Perforated HDPE pipe shall conform to the requirements specified for HDPE pipe.

B. Perforations shall be drilled into the pipe at the pipe extrusion plant or fabrication shop. Any burrs remaining after drilling shall be removed. Perforations shall be drilled and deburred prior to pipe delivery to the job site. Job site perforation or perforation by the CONTRACTOR shall not be permitted.

C. Provide perforations as indicated on the Drawings.

2.5 DOUBLE WALL HDPE PIPING MATERIALS

A. Double wall HDPE piping, including the carrier pipe, the containment pipe and all related fittings shall conform to the requirements specified for HDPE piping. Double-wall pipe shall not use spacers to position or attach the carrier pipe to the containment pipe except where necessary at butt, heat fusion joints to maintain alignment during joining and with no more than two spacers on each side of and adjacent to the joint. Piping systems shall be assembled in a manner which will minimize the number of joints with spacers.

B. HDPE fittings for double-wall piping systems shall be either molded or shop-fabricated type. Shop-fabricated fittings shall be made from pipe sections using butt, heat fusion welding. Double-wall fittings shall not use spacers to position or attach the carrier fitting to the containment fitting except where necessary to maintain alignment during fabrication and joining. Fittings with extrusion welding shall not be permitted.

C. Any and all double-wall piping spacers shall have a configuration which will permit passage/flow of liquid through the containment piping and shall be designed to preclude the application of damaging stress to the piping system walls as a result of external and/or internal loads.

D. Containment pipe sizes shall be as indicated on the drawings or in accordance with the pipe manufacturer's recommendations for the carrier pipe size indicated. Prefabrication of double wall systems shall be utilized to the greatest extent possible in order to minimize field assembly.

2.6 FLEXIBLE CONNECTORS

Flexible connectors shall have Viton interiors, neoprene exteriors, 316 stainless steel retaining rings, and 316 stainless steel control units. Flexible connectors shall be Metraflex Style No. 300, or equal as approved by the Engineer.

2.7 APPURTENANCES

A. HDPE Flange Adapter: HDPE flange adapter shall be the same SDR and from the same resin as the pipe.

B. HDPE to Steel Pipe Transition: Use HDPE manufacturer's factory made transition.

C. All fasteners and hardware shall be stainless steel type 304 unless otherwise specified.

PART 3 - EXECUTION

3.1 GENERAL INSTALLATION REQUIREMENTS

A. All pipe shall be laid and maintained straight and true to line in strict conformance with the lines, grades, and elevations indicated on the drawings. Line and grade tolerances, where applicable, shall be in accordance with limits given for specific material. Grade shall be measured at the pipe invert, not at the top of the pipe, due to permissible variations in pipe wall thickness. CONTRACTOR shall supply the Engineer drawings showing as-constructed pipe invert elevations at every 50 feet of pipeline.

B. Trenching, bedding, and backfill shall be in accordance with Section 02300, Earthwork for Liner Construction.

C. During laying operations, do not permit debris, tools, clothing, or similar items to be placed in pipes. Pipe interior shall be free of mud and kept clean at all times.

D. Pipe ends shall be kept clear and clean and the CONTRACTOR shall ensure that inside surfaces are maintained smooth and free from any projections that may interfere with joint assembly or flow through the completed line.

E. Care shall be exercised when lowering pipe into trenches or on subgrade to prevent damage or twisting of the pipe. After laying and prior to completion of backfill or cover operations, pipe shall be protected from any vehicular traffic.

F. Existing piping flanged joints which are disassembled by the CONTRACTOR shall be fitted with new gaskets, as specified, upon reassembly.

G. Laying of pipe and backfilling shall not be conducted in areas of standing water.

3.2 HDPE PIPING INSTALLATION

A. Line or grade shall not deviate from dimensions and elevations given on the drawings by more than 1/2 inch for line and 1/4 inch for grade at any point, provided that such variation does not result in a level or reverse sloping invert.

B. Pipe and fittings shall be joined together using butt heat-fusion techniques in accordance with the pipe manufacturer's recommendations. The heat-fusion welds shall not project into the interior of the piping high enough to interfere with passage of cameras and equipment used or piping system cleaning and videotaping inspection, and in no event shall such projections exceed 0.25 inch.

C. Prior to final acceptance, completely flush and clean all parts of the system. Flushing water shall be properly disposed. Flushing water shall not be discharged to the leachate aeration/storage basins. The leachate collection and leak detection pumps shall not be used to handle water resulting from flushing operations. Remove all accumulated construction debris, rocks, sand, gravel, and other foreign material.

D. Pipe Testing – General (refer to individual piping sections for pipe-specific testing requirements):
   1. Test piping systems as follows:
      a. Test exposed, non-insulated piping systems upon completion of system.
      b. Test exposed, insulated piping systems upon completion of system but prior to application of insulation.
      c. Test concealed interior piping systems prior to concealment and, if system is insulated, prior to application of insulation.
      d. Test buried piping after backfilling has been complete.
   2. Utilize pressures, media and pressure test durations as specified on Piping Specification Schedules.
   3. Isolate equipment which may be damaged by the specified pressure test conditions.
   4. Perform pressure test using calibrated pressure gages and calibrated volumetric measuring equipment to determine leakage rates.
      a. Select each gage so that the specified test pressure falls within the upper half of the gage's range.
      b. Notify the Engineer 24 HRS prior to each test.
   5. Completely assemble and test new piping systems prior to connection to existing pipe systems.
   6. Acknowledge satisfactory performance of tests and inspections in writing to Engineer prior to final acceptance.
   7. Bear the cost of all testing and inspecting, locating and remedying of leaks and any necessary retesting and re-examination.

E.Pressure Testing:

1. Testing medium: Unless otherwise specified in the Piping Specification Schedules, utilize the following test media.
   a. Liquid systems:

PIPE LINE SIZE (DIA)	GRAVITY OR PUMPED	SPECIFIED TEST PRESSURE	TESTING MEDIUM
Up to and including 48 IN	Gravity	25 psig or less	Air or water
Above 48 IN	Gravity	25 psig or less	Water
All sizes	Pumped	200 psig or less	Water

2. Allowable leakage rates:
   a. Leachate systems, groundwater pumping systems, all exposed piping systems, all pressure piping systems, and all buried, insulated piping systems which are hydrostatically pressure tested shall have zero leakage at the specified test pressure throughout the duration of the test.
   b. Hydrostatic exfiltration and infiltration for sanitary and stormwater sewers (groundwater level is below the top of pipe):
      1) Leakage rate: 200 GAL per inch diameter per mile of pipe per day at average head on test section of 3 FT.
      2) Average head is defined from groundwater elevation to average pipe crown.
      3) Acceptable test head leakage rate for heads greater than 3 FT: Acceptable leakage rate (gallons per inch diameter per mile per day) = 115 x (actual test head to the 1/2 power).
   c. Hydrostatic infiltration test for sanitary and stormwater sewers (groundwater level is above the top of pipe):
      1) Allowable leakage rate: 200 GAL per inch diameter per mile of pipe per day when depth of groundwater over top of pipe is 2 to 6 FT.
      2) Leakage rate at heads greater than 6 FT: Allowable leakage rate (gallons per inch diameter per mile of pipe per day) = 82 x (actual head to the 1/2 power).
   d. For low pressure (less than 25 psig) air testing, the acceptable time for loss of 1 psig of air pressure shall be:

3. Hydrostatic pressure testing methodology:
   a. General:
      1) All joints, including welds, are to be left exposed for examination during the test.
      2) Provide additional temporary supports for piping systems designed for vapor or gas to support the weight of the test water.
      3) Provide temporary restraints for expansion joints for additional pressure load under test.

4) Isolate equipment in piping system with rated pressure lower than pipe test pressure.
5) Do not paint or insulate exposed piping until successful performance of pressure test.

PIPE SIZE (IN DIA)	TIME (MINUTES/100 FT)
2	0.2
4	0.3
6	0.7
8	1.2
10	1.5
12	1.8
15	2.1
18	2.4
21	3.0
24	3.6
27	4.2
30	4.8
33	5.4
36	6.0
42	7.3
48	7.6

4. Air testing methodology:
   a. General:
      1) Assure air is ambient temperature.
   b. Low pressure air testing:
      1) Place plugs in line and inflate to 25 psig.
      2) Check pneumatic plugs for proper sealing.
      3) Introduce low pressure air into sealed line segment until air pressure reaches 4 psig greater than ground water that may be over the pipe.
         a) Use test gage conforming to ANSI B40.1 with 0 to 15 psi scale and accuracy of 1 percent of full range.
      4) Allow 2 minutes for air pressure to stabilize.
      5) After stabilization period (3.5 psig minimum pressure in pipe) discontinue air supply to line segment.
      6) Record pressure at beginning and end of test.
   c. Repeat test procedure for verification.

3.3 PERFORATED HDPE PIPING INSTALLATION

A. All sections of perforated HDPE pipe shall be thoroughly cleaned and deburred after perforating and before welding or delivery to the job site to ensure all drill cuttings are removed from the pipe.

B. Pipe shall be installed in gravel fill as specified in Section 02300, Earthwork for Liner Construction.

C. Install perforated pipe in accordance with drawing details and as specified for HDPE pipe.

D. Prior to final acceptance, completely flush and clean all parts of the system. Flushing water shall be disposed of properly. Flushing water shall not be discharged to the leachate storage/aeration impoundment. The leachate collection and leak detection pumps shall not be used to handle water resulting from flushing operations. Remove all accumulated construction debris, rocks, sand, gravel, and other foreign material.

3.4 DOUBLE-WALL HDPE PIPING INSTALLATION

A. Install containment and carrier piping as per manufacturer's directions and in accordance with specifications herein for HDPE piping installation.

3.5 PVC PIPE INSTALLATION

A. Installation:
   1. Field threading PVC will not be permitted.
      a. Perform required threaded connections or attachments by the use of factory molded socket by threaded adapters.
      b. Female adapters are not acceptable.
   2. Employ installation and pipe support practices and solvent welding all in compliance to the manufacturer's printed recommendation.
      a. Continuously support PVC piping at liquid operating temperatures in excess of 100 DegF.
      b. For vertical piping, band the pipe at intervals to rigidly support load of twice vertical load.
      c. Support riser clamps on spring hangers.
      d. Do not clamp PVC tightly or restrict movement for expansion and contraction.
   3. Perform installation procedures, handling, thrust blocking, connections, and other appurtenant operations in full compliance to the manufacturer's printed recommendations and in full observance to plan details when more stringent.

B. Deflection:
   1. After backfilling, each section of pipe shall be checked for deflection by pulling a mandrel through the pipe.
   2. Pipe with deflection exceeding 5 percent of the inside diameter shall have backfill removed and replaced to provide a deflection of less than 5 percent.
   3. Any repaired pipe shall be retested.

C. PVC Tubing Fittings:

1. Install tubing with nylon fittings and connectors.
2. Use barbed type adapters with stainless steel clamps.
3. Provide fittings capable of withstanding temperatures from a -70 to 250 DegF.
4. Ensure fittings have the same pressure and temperature rating as the tubing.

END OF SECTION

SECTION 15300
VALVES

PART 1 GENERAL

1.01 WORK INCLUDED

This section covers the work necessary to furnish and install valves at the leachate collection and leak detection pumping systems, and at other locations as indicated on the drawings.

1.02 RELATED WORK

A. The General and Supplemental Conditions of these Specifications are a part of this section as if incorporated herein.

B. Other related specification sections contained herein are listed below:

1. Section 15200, Pipe and Pipe Fittings

1.03 SUBMITTALS

DBT shall submit printed manufacturer's product data for review and approval by OWNER.

1.04 PROTECTION DURING STORAGE

The interior of valves shall be kept free from dirt and foreign material at all times. Valves shall be drained and stored in a manner that will protect them from damage by freezing.

1.05 REFERENCE STANDARDS

A. Reference standards and recommended practices referred to herein shall be the latest revision of any such document.

B. Standards referenced herein are as listed below:

1.	ASTM A 473	Stainless and Heat-Resisting Steel Forgings
2.	ASTM D2000	Classification System for Rubber Products in Automated Applications
3.	AWWA C504	Rubber-Seated Butterfly Valves
4.	AWWA C509	Resilient Wedge Gate Valves for Water Service

5. AWWA C512 Air-Release, Air/Vacuum, and Combination Air Valves for Waterworks Service

PART 2 PRODUCTS

2.01 GENERAL

Each valve shall have the manufacturer's name, address, model number, and serial number engraved or embossed on a nameplate securely affixed in a conspicuous place; the nameplate of the distributing agent will not be acceptable. The nameplate shall be 316 stainless steel, or other corrosion resistant material as approved by the OWNER.

2.02 CHECK VALVES FOR LEACHATE SERVICE

A. Silent check valves shall be provided for leachate service and shall protect against water hammer by closing while there is a positive pressure on the pump side of the valve. The valve shall be wafer-style designed for mounting between ANSI Class 150 flanges. Valves shall permit free flow forward and provide a positive check against backflow.

B. The valve shall be global-style with spring actuator. The flow through area of the valve shall be greater than the cross-sectional area of the upstream piping. The valve body shall be ductile iron with stainless steel trim, resilient Buna-N seating and stainless steel spring.

C. Check valves shall have a working differential pressure rating of at least 150 psi.

E. Check valves shall be Series 304, as manufactured by APCO Valve and Primer Corporation, or equal as approved by the OWNER.

2.03 BALL VALVES

Ball valves shall be style A36TT as manufactured by Jamesbury, or equal as approved by the OWNER. Size shall be as shown on the drawings.

2.04 COMBINATION AIR VALVE

A. The combination air valve (CAV) shall have 1-inch NPT inlet and 1-inch NPT outlet connections. The outlet shall be routed to discharge into either the riser pipe or leachate monitoring station pad. The body and cover shall be of cast iron. The lever frame shall be Debrin. All internal cast iron components shall be epoxy coated by the manufacturer. All other internal components and trim shall be of 300-series stainless steel. The seat shall be of Buna-N. The cover bolts shall be of 300-series stainless steel.

B. CAV valves shall automatically open to permit large quantities of air to enter the pipeline when the internal pressure of the pipeline approaches a negative value not to exceed negative 0.25-psi gauge pressure due to column separation in the pipeline, power outage, pipeline break, etc.

C. The exterior surfaces of the valves shall be coated with an epoxy coating by the manufacturer.

D. The CAV valves shall be Series No. 142 as manufactured by Valve and Primer Corporation, or equal as approved by the OWNER.

PART 3 EXECUTION

3.01 GENERAL VALVE INSTALLATION

A. Valves shall be installed at locations indicated on the drawings and in general accordance with the requirements of Section 15200, Pipe and Pipe Fittings.

B. Vertical and horizontal valve stems shall be plumb and level, respectively.

C. Settings of valve actuator stops shall be checked/adjusted in accordance with valve manufacturer's instructions.

D. Trench excavation shall be performed in accordance with Section 02300, Earthwork for Pipelines and Utilities, with bottom of trench hand trimmed such that valve flanges do not rest on subgrade.

3.02 COMBINATION AIR VALVE INSTALLATION

CAV valves shall be installed vertically and in accordance with the manufacturer's directions.

END OF SECTION

APPENDIX F
CQA PLAN

STEELFIELD ROAD LANDFILL
CELLS 6 AND 7 CONSTRUCTION

BAY COUNTY, FLORIDA

CONSTRUCTION QUALITY ASSURANCE (CQA) PLAN

WACS ID: 16
Permit Number: 0007875-009-SO-01

Prepared for:

BAY COUNTY SOLID WASTE DIVISION
311411 Landfill Road, West Bay
Florida 32413

Prepared by:

HDR ENGINEERING
76 South Laura Street, Ste. 1600
Jacksonville, Florida 32202
Telephone: (904)598-8900

HDR Project No.: 10150537

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CONSTRUCTION QUALITY ASSURANCE PLAN

1.0 INTRODUCTION

1.1 Purpose and Scope of the Construction Quality Assurance Plan

This Construction Quality Assurance (CQA) Plan addresses the soil and geosynthetic components of the bottom liner system for the Cells 6 and 7 of the Steelfield Road Landfill (SFRL). This CQA Plan describes the procedures to be followed for monitoring construction associated with these materials. In addition, it presents CQA procedures for survey control. The scope of this CQA Plan includes the CQA of the soil and geosynthetic components of the bottom liner system. This CQA Plan does not address design guidelines, installation specifications, or selection of soils, geomembranes, and other geosynthetics.

1.2 Units

In this CQA Plan, properties and dimensions are expressed in U.S. units with "approximate equivalent" SI units in parentheses. It should be noted that the conversion is typically only accurate within 10 percent due to rounding. In cases of conflict or clarification, the U.S. units will be deemed to govern.

1.3 References

The CQA Plan includes references to test procedures of the American Society for Testing and Materials (ASTM), Corps of Engineers (COE), the "Standards for Flexible Membrane Liners" of the National Sanitation Foundation (NSF), the Geosynthetics Research Institute (GRI), the American Association of State Highway and Transportation Officials (AASHTO), and the Plastic Pipe Institute (PPI). The Technical Specifications of the Contract Documents shall take precedence in cases where they are more stringent than this CQA Plan. The CQA Plan cites requirements that are in conformance with applicable regulations. However, in some cases, the specific requirements of the Project are more restrictive than the regulations and, therefore, are specified elsewhere in the Contract Documents and shall govern the Work.

1.4 Organization of the Construction Quality Assurance Plan

The remainder of the CQA Plan is organized as follows:

- Section 2 presents definitions relating to CQA;

- Section 3 describes the parties involved with the CQA;
- Section 4 describes site and project control requirements of the CQA;
- Section 5 describes CQA documentation;
- Section 6 describes CQA for high-density polyethylene (HDPE) geomembranes;
- Section 7 describes CQA for geotextile;
- Section 8 describes CQA for geonet;
- Section 9 describes CQA for geocomposite;
- Section 10 describes CQA for geosynthetic clay liners;
- Section 11 describes CQA for HDPE pipe and fittings;
- Section 12 describes CQA for soils; and
- Section 13 describes surveying requirements of the CQA.

2.0 DEFINITIONS RELATING TO CONSTRUCTION QUALITY ASSURANCE

2.1 Construction Quality Assurance and Construction Quality Control

This CQA Plan is devoted to Construction Quality Assurance. In the context of this plan, Construction Quality Assurance (CQA) and Construction Quality Control (CQC) are defined as follows:

CQA - A planned and systematic pattern of the means and actions designed to provide adequate confidence that items or services meet contractual and regulatory requirements, and will perform satisfactorily in service.

CQC - Those actions which provide a means to measure and regulate the characteristics of an item or service to contractual and regulatory requirements.

2.2 Use of The Terms in This Plan

In the context of this document:

- *CQA* refers to means and actions employed by the CQA Consultant and testing laboratories to assure that the components of the liner system is produced and installed to conform with the requirements of this CQA Plan, the drawings, and the specifications. CQA is provided by a consultant who is independent of the Owner, DBT, Manufacturer, and Fabricator.

- *CQC* refers to those actions taken by Manufacturers, Fabricators, Installers, DBCs, or Owners to ensure that the materials and the workmanship meet the requirements of the plans and specifications. In the case of soil components, CQC may be combined with CQA and then may be provided by the CQA Consultant; otherwise, CQC of soil components is provided by the DBC. In the case of the geosynthetic and other non-soil components, CQC is provided by the Manufacturers and Installers of the various geosynthetics.

- This Document was prepared as a Permit document to show intent to the Florida Department of Environmental Protection that the protocols set forth herein will be carried out. Therefore, the DBC, all Manufacturers, Fabricators, Installers, Suppliers, Transporters, Laboratories, Surveyors

and other parties subcontracted by or affiliated with the DBC shall interpret the term “will” to be “shall”, meaning a requirement.

3.0 PARTIES TO CONSTRUCTION QUALITY ASSURANCE

3.1 Description of the Parties

3.1.1 Design Build Engineer

The Design Build Engineer (DBE) is responsible for the design, drawings, CQA plan, and specifications for the liner system.

3.1.2 Design Build Contractor

The Design Build Contractor (DBC) is responsible for the preparation of the site and installation of the soil and non-soil components of the liner system. DBC shall also be responsible for all damage and related repair costs to the liner system, electrical system or LFG management systems associated with general construction activities (e.g., earthwork, geosynthetic deployment).

3.1.3 Soils Supplier

The Soils Supplier excavates and delivers off-site soils to the DBC.

3.1.4 Pipe Supplier

The Pipe Supplier manufactures and delivers pipe to the site.

3.1.5 Resin Supplier

The Resin Supplier produces and delivers the resin to the Geosynthetics or Pipe Manufacturer.

3.1.6 Geosynthetics Manufacturer

The Geosynthetics (e.g., geomembrane, geotextile, geonet, GCL, geocomposite, in the liner system) Manufacturer (Manufacturer) is responsible for the production of rolls of geosynthetics to be used on this project.

3.1.7 Geosynthetics Fabricator

The Geosynthetics Fabricator (Fabricator) is responsible for the fabrication (if required) of factory panels of geosynthetics from rolls received from the Manufacturer. The Fabricator may also be responsible for the delivery of the factory fabricated panels to the site.

3.1.8 Geosynthetics Installer

The Geosynthetics Installer (Installer) is responsible for field handling, storing, placing, seaming, loading (against wind), and other aspects of the geosynthetics installation. The Installer may also be responsible for specialized construction tasks (e.g., construction of anchor trenches for the geosynthetics and pipe boots).

3.1.9 Transporter

The Transporter transports the geosynthetics, including rolls of factory panels of geosynthetics and lengths of pipe from the manufacturing facility and/or fabrication facility to the site.

3.1.10 Soils CQA Consultant

The Soils CQA Consultant is HDR Engineering (HDR), independent from the Owner, DBC, Manufacturer, Fabricator, and Installer, that is responsible for observing, testing, and documenting activities related to the CQA of the earthworks at the site. The Soils CQA Consultant may also be responsible for CQA of other components of the liner system. The Soils CQA Consultant is also responsible for issuing a CQA report.

3.1.11 Geosynthetics CQA Consultant

The Geosynthetics CQA Consultant is HDR, independent from the Owner, DBC, Manufacturer, Fabricator, and Installer, that is responsible for observing and documenting activities related to the CQA of the production and installation of the geosynthetic components of the liner system. The Geosynthetics CQA Consultant will also be responsible for CQA of pipes associated with the landfill liner system. The Geosynthetics CQA Consultant is also responsible for issuing a CQA report.

3.1.12 Soils CQA Laboratory

The Soils CQA Laboratory is independent from the Owner, DBC, Soils Supplier, and DBC and is responsible for conducting tests in accordance with ASTM standards on samples of soil in the field and in either an on-site or off-site laboratory.

3.1.13 Geosynthetics CQA Laboratory

The Geosynthetics CQA Laboratory is a party, independent from the Owner, Manufacturer, Fabricator, and Installer, that is responsible for conducting tests in accordance with ASTM standards on samples of geosynthetic materials used in construction of the landfill liner system.

3.1.14 Owner

The Owner owns and operates the SFRL. In this CQA Plan, the term "Owner" will refer specifically to the Bay County Solid Waste Division.

3.1.15 Project Manager

The Project Manager will be the official representative of the Owner. In this CQA Plan, the term "Project Manager" will apply equally to "Owner's Representative" (i.e., the individual in charge of coordinating field activities).

3.1.16 Certifying Surveyor

The Certifying Surveyor is responsible for producing as-built documentation of the components of the landfill liner system and is part of Design Build Team (DBT).

3.2 Qualifications of the Parties

3.2.1 Introduction

The following qualifications are required of all parties involved with the design, manufacture, installation, transportation, and CQA of all components of the landfill liner system.

3.2.2 Design Build Engineer

The DBE shall be a qualified engineer, licensed as a Professional Engineer in the State of Florida and by regulation. The Engineer shall have a history that demonstrates familiarity with the components of the landfill liner system including detailed design methods and procedures.

3.2.3 Design Build Contractor

Qualifications of the DBC are specific to the construction contract. The DBC shall have a demonstrated history of successful roadway, landfill, pipe work and earthworks construction and shall have current state and federal licenses as appropriate.

3.2.4 Soils Supplier

The qualifications of the Soils Supplier are specific to the construction contract. The Soils Supplier shall have a demonstrated history of providing soils with consistent properties and should be a licensed contractor in the State of Florida.

3.2.5 Pipe Supplier

The qualifications of the Pipe Supplier are specific to the construction contract. The Pipe Supplier shall have a demonstrated history of providing pipe with consistent properties.

3.2.6 Resin Supplier

Qualifications of the Resin Supplier are specific to the Manufacturer's requirements. The Resin Supplier shall have a demonstrated history of providing resin with consistent properties.

3.2.7 Geosynthetics Manufacturers

The Geosynthetics Manufacturers shall be able to provide sufficient production capacity and qualified personnel to meet the demands of the project. The Geomembrane Manufacturer, in particular, must be pre-qualified and approved by the Project Manager. The qualifications required of the Geomembrane Manufacturer may include:

- corporate background and information; and
- a list of at least 20 completed facilities totaling a minimum of 10,000,000 ft^2 (100,000 m^2), for which the Manufacturer has manufactured a geomembrane of the same material to be used for this project; for each facility, the following information will be provided:
    - name and purpose of facility, its location and date of installation;
    - name of owner, project manager, DBE, fabricator (if any), and installer; and
    - thickness of geomembrane and surface area of geomembrane manufactured.

3.2.8 Geosynthetics Fabricator

The Geosynthetics Fabricator shall be trained and qualified to fabricate geosynthetic panels to be used for this project. The Geosynthetics Fabricator shall be a well-established firm able to provide sufficient fabrication capacity and qualified personnel to meet the demands of the project.

The Geomembrane Fabricator, in particular, shall be approved and/or licensed by the Geomembrane Manufacturer. A copy of the approval letter or license must be submitted by the Geomembrane Fabricator to the Project Manager prior to the fabrication of materials.

Prior to the confirmation of any contractual agreements, the Geomembrane Fabricator must provide the Project Manager with the following written information:

- corporate background and information;
- fabrication capabilities including:
    - information on factory size, equipment, personnel, number of shifts per day and capacity per shift;
    - daily fabrication quantity available for this contract;
    - quality control manual for fabrication; and
    - samples of fabricated seams and certified test results; and
- a list of at least 10 completed facilities, totaling a minimum of 10,000,000 ft^2 (100,000 m^2), for which the Geomembrane Fabricator has fabricated polyethylene geomembrane panels; for each fabrication, the following information will be provided:
    - name and purpose of facility, its location, and date of installation;
    - name of owner, project manager, designer, manufacturer, and installer;
    - thickness of geomembrane and surface area of geomembrane fabricated;
    - type of seaming and type of seaming apparatus used; and
    - available information on the performance of the geosynthetics and the facility.

All personnel performing seaming operations should be qualified by experience or by successfully passing seaming tests. At least one seamer must have experience seaming a minimum of 100,000 linear ft (30,000 linear m) of geomembrane seams using the same method of seaming that will be used on this project. The most experienced seamer, the "master seamer", shall provide direct supervision, as required, over less experienced seamers. No field seaming shall take place without the master seamer being present.

3.2.9 Geosynthetics Installer

The Geosynthetics Installer is part of DBT and shall be trained and qualified to install geosynthetics, as well as other liner system components such as pipe. The Geomembrane Installer, in particular, must be approved and/or licensed by the Geomembrane Manufacturer. A copy of the approval letter(s) or license(s) must be submitted by the Geosynthetics Installer to the

Project Manager. In addition, prior to confirmation of any contractual agreements, the Geosynthetics Installer must provide the Project Manager with the following written information:

- corporate background and information;
- installation capabilities including:
    - information on installation equipment and personnel;
    - daily anticipated production capabilities;
    - quality control manual for installation of geosynthetic materials, and samples of field seams and certified test results; and
- a list of at least 20 completed facilities, totaling a minimum of 5,000,000 ft^2 (465,000 m^2) for which the Installer has installed a geomembrane of the same material for this project; for each installation, the following information will be provided:
    - name and purpose of facility, its location, and date of installation;
    - name of owner, project manager, DBE, manufacturer, and fabricator (if any);
    - name and qualifications of the supervisor of the Installer's crew;
    - thickness of geomembrane, surface area of the installed geomembrane;
    - type of seaming and/or type of seaming apparatus used; and
    - duration of installation.

All personnel performing seaming operations should be qualified by experience. At least one seamer must have experience seaming a minimum of 2,000,000 ft^2 (184,000 m^2) of geomembrane using the same method of seaming that will be used on this project. The most experienced seamer, the "master seamer", will provide direct supervision, as required, over less experienced seamers. No field seaming will take place without the master seamer being present.

The Geosynthetics Installer will provide the Project Manager with a list of proposed seaming personnel and their professional records (i.e., resumes). This documentation will be reviewed by the Project Manager and the Geosynthetics CQA Consultant. Any proposed seaming personnel deemed insufficiently experienced will not be accepted by the Project Manager or will be invited to pass a seaming test.

The Installer will designate one representative as his Superintendent, who will represent the Installer at all site meetings and be responsible for acting as the Installer's spokesman on site. The Superintendent will be qualified by experience. The Superintendent must have supervised

the installation of a minimum of 2,000,000 ft^2 (185,000 m^2) of geomembrane and 100,000 ft^2 (10,000 m^2) of geotextile. The Superintendent will also exhibit good management skills. The appointment of the Superintendent will be approved by the Project Manager.

3.2.10 Transporter

All personnel responsible for the loading, transport, and unloading of the geosynthetics must be fully aware of the consequences of damage to the geosynthetics, and familiar with handling and transport constraints required by the Manufacturer and/or Fabricator. The Transporter will ensure that all rolls of polyethylene geomembrane are shipped on open trailers (no enclosed vans) for ease and safety of unloading the material.

3.2.11 Soils CQA Consultant

The Soils CQA Consultant shall be a well-established engineering firm incorporated (or otherwise registered) in the United States. The Soils CQA Consultant will be experienced with soils, including high- and low-permeability soils. The Soils CQA Consultant will be experienced in the preparation of CQA documentation including CQA forms, reports, and plans. The Soils CQA Consultant will provide qualified staff for the project, as necessary; such staff may include a Soils CQA Managing Engineer, a Soils CQA Site Manager, and Soils Field Monitors.

The Soils CQA Consultant will provide the following, in writing, to the Project Manager upon request:

- corporate background and information; and
- CQA capabilities:
    - a summary of the firm's experience with soils;
    - a summary of the firm's experience in CQA, including installation CQA of soils; and
    - a summary of CQA documentation and methods used by the firm, including sample CQA forms, reports, and plans prepared by the firm.

The Soils CQA Managing Engineer, if required, must be licensed as a Professional Engineer in the State of Florida. The Soils CQA Managing Engineer must also comply with the experience requirements listed in the previous paragraphs. The Soils CQA Site Manager will be specifically experienced in the installation of soils and will be trained and certified by the Soils CQA Consultant in the duties of a Soils CQA Site Manager. The Soils Field Monitors will be CQA personnel who have been specifically trained in CQA of soils.

In addition, the Soils CQA Consultant will provide the following, in writing, to the Project Manager upon request:

- resumes of personnel to be involved in the project including Soils CQA Managing Engineer, Soils CQA Manager, and Soils CQA Monitors;
- proof of Professional Engineering registration (or ability to be registered) of the engineer to be designated the Soils CQA Managing Engineer; and
- proof of CQA experience of proposed soils CQA personnel.

3.2.12 Geosynthetics CQA Consultant

The Geosynthetics CQA Consultant shall be a well-established engineering firm incorporated (or otherwise registered) in the United States. The Geosynthetics CQA Consultant will be experienced with geosynthetics, including geomembranes, geocomposites, geotextiles, and geonets, GCLs, as well as polyethylene pipe. The Geosynthetics CQA Consultant shall be experienced in the preparation of CQA documentation including CQA forms, reports, and plans.

In addition, the Geosynthetics CQA Consultant shall provide the following, in writing, to the Project Manager upon request:

- corporate background and information; and
- geosynthetics CQA capabilities including:
- a summary of the firm's experience with geosynthetics, particularly including geomembranes, geotextiles, geonets and GCLs;
- a summary of the firm's experience with polyethylene pipe;
- a summary of the firm's experience in geosynthetics CQA, including installation CQA of geomembranes, geotextiles, geonets, pipes; and
- a summary of geosynthetic system CQA documentation and methods used by the firm, including sample Geosynthetics CQA forms, reports, and manuals prepared by the firm.

The Geosynthetics CQA Consultant will provide staff for the project, as necessary. The staff may include a Geosynthetics CQA Managing Engineer, a Geosynthetics CQA Site Manager, and Geosynthetics CQA Field Monitors. The Geosynthetics CQA Managing Engineer must be licensed as a Professional Engineer in the State of Florida. The Geosynthetics CQA Managing Engineer must also comply with the experience requirements listed in the previous paragraph. The Geosynthetics CQA Site Manager will be specifically familiar with the installation of geosynthetics and will be trained and certified by the Geosynthetics CQA Consultant in the

duties of a Geosynthetics CQA Site Manager. Geosynthetics CQA Field Monitors will be CQA personnel who have been specifically trained in CQA of geosynthetics and other applicable geosynthetic liner system components.

The Geosynthetics CQA Consultant will provide the following, in writing, to the Project Manager upon request:

- a list of at least 5 completed facilities totaling a minimum of 1,000,000 ft^2 (100,000 m^2) in area, for which the Geosynthetics CQA Consultant has provided CQA monitoring services for the installation of the corresponding geosynthetic material; for each facility, the following information will be provided:

    - name and purpose of facility, its location, and date of installation,
    - name of owner, and
    - type of geosynthetic, surface area of geosynthetic material installed;

- resumes of personnel to be involved in the project including Geosynthetics CQA Managing Engineer, Geosynthetics CQA Site Manager, and Geosynthetics CQA Field Monitors;

- proof of Professional Engineering registration (or ability to be registered) in the State of Florida of the engineer to be designated the Geosynthetics CQA Managing Engineer; and

- proof of Geosynthetics CQA experience of proposed CQA personnel with emphasis on geomembranes as well as geotextiles.

3.2.13 Soils CQA Laboratory

The Soils CQA Laboratory shall have experience in testing soils, meet all regulatory requirements, and be familiar with ASTM and other applicable standards. The Soils CQA Laboratory will be capable of providing test results in accordance with the specifications.

3.2.14 Geosynthetics CQA Laboratory

The Geosynthetics CQA Laboratory shall have experience in testing geosynthetics and be familiar with ASTM, NSF, GRI, and other applicable test standards. The Geosynthetics CQA Laboratory will be capable of providing test results within 24 hours of receipt of samples and will maintain that capability throughout the duration of geosynthetic material installation.

3.2.15 Certifying Surveyor

The Certifying Surveyor must be a registered Land Surveyor in the State of Florida and employed by a well-established surveying firm incorporated (or otherwise registered) in the United States.

3.3 Duties of Construction Quality Assurance Personnel

3.3.1 Introduction

In this CQA Plan, the roles of the Soils CQA Consultant and Geosynthetics CQA Consultant are described separately, however, one consultant may perform both Soils CQA and Geosynthetics CQA, or two or more separate consultants may be used. The personnel of each CQA Consultant will include:

- the CQA Managing Engineer, who operates from the offices of the CQA Consultant's firm and visits the site periodically;
- the CQA Site Manager, who is located at the site; and
- the CQA Monitors, who are located at the site.

The duties of the CQA Personnel are discussed in the following sections.

3.3.2 CQA Managing Engineer

Each CQA Managing Engineer (i.e., for Soils and for Geosynthetics):

- reviews the liner system designs, plans, and specifications;
- reviews site-specific documentation, including construction documents, proposed layouts, soil sampling and analysis reports, and geosynthetics materials manufacturer's and installer's literature; unless otherwise agreed, such reviews are for familiarization and for evaluation of constructability only and the CQA Managing Engineer will not make comments regarding the feasibility, appropriateness, or comprehensiveness of the design, and the CQA Managing Engineer and his firm will assume no responsibility for any aspect of the design;
- attends the pre-construction and any resolution meeting when requested;

- administers the CQA program (i.e., assigns and manages all CQA personnel, reviews all field reports, and provides engineering review of all CQA related issues);
- provides quality control of the CQA personnel, including site visits;
- reviews changes to the design, plans, and specifications; and
- with assistance from the CQA Site Manager, prepares the final report, including a review of the record drawings.

3.3.3 CQA Site Manager

The CQA Site Manager for soils and/or geosynthetics may be the same, and may also be the CQA Managing Engineer, and:

- acts as the on-site (resident) representative of the CQA Consultant;
- familiarizes the CQA Monitors with the site and the CQA requirements for the project;
- manages the daily activities of the CQA Monitors;
- attends the CQA-related meetings (e.g., resolution, pre-construction, daily, weekly);
- prepares, or oversees the ongoing preparation of the record drawing(s);
- assigns locations for testing and sampling;
- verifies the calibration and condition of on-site CQA equipment;
- reviews the CQA Monitors' daily reports and logs;
- reports to the Project Manager, and logs in his daily report relevant observations reported to him by the CQA Monitors;
- prepares his own daily report;
- oversees the collection and shipping of laboratory test samples;

- reviews and reports results of laboratory testing;
- designates a senior CQA Monitor to act on his behalf whenever he is absent from the site while operations are ongoing;
- reports any unresolved deviations from the CQA Plan to the Project Manager; and
- prepares the final report with the CQA Managing Engineer.

In addition, the Soils CQA Site Manager:

- periodically checks stockpile or borrow pit sources for variability of the soils, and verifies that conformance testing is carried out;
- reviews the work of the DBC's equipment operators, to verify that care is taken to protect other portions of the work; and
- establishes additional test requirements beyond those in the specifications, when necessary.

In addition, the Geosynthetics CQA Site Manager:

- reviews Supplier, Manufacturer, and Installer certifications and documentation and makes appropriate recommendations;
- reviews the Installer's personnel qualifications for conformance with those pre-approved for work on site; and
- notes on-site activities that could result in damage to the geosynthetics.

3.3.4 CQA Monitors

The duties of the CQA Monitors, as assigned by the CQA Site Manager, include monitoring, logging, and/or documenting appropriate operations.

The duties to be performed, and operations to be monitored by the Soils CQA Monitors specifically include:

- observation and documentation of soils delivery, stockpiling, and placement;

- observation and documentation of soils moisture content, and moisture conditioning, if required;
- observation and documentation of unsuitable materials in the excavation area, if required;
- observation and documentation of collection of soil samples for laboratory testing;
- observation and documentation of operations to protect completed areas before the covering materials are placed;
- examination of the soil surface for signs of excessive wetting, desiccation, or other disturbance prior to placement of any liner materials; and
- observation and documentation of specification, rewetting, recompaction, or proof rolling required to repair deteriorated areas.

The operations to be monitored by the Geosynthetics CQA Monitors include:

- observation and documentation of material delivery;
- observation and documentation of material unloading and on-site transport and storage;
- on-site conformance testing to verify geomembrane thickness;
- observation and documentation of geosynthetic placement operations;
- observation and documentation of joining and/or seaming operations;
- observation and documentation of condition of panels as placed;
- sampling for conformance testing by the Geosynthetics CQA Laboratory;
- marking samples for conformance testing; and
- observation and documentation of repair operations.

Specifically, for geomembranes, the seaming operations to be monitored include:

- trial seaming;
- seam preparation;
- seaming;
- nondestructive seam testing;
- sampling for destructive seam testing;
- field tensiometer testing;
- laboratory sample marking; and
- repair operations.

Specifically, for geotextiles, operations to be performed by Geosynthetics CQA Monitors include visual examination by walkover and, for needlepunched non-woven geotextile only, monitoring for Installer's broken needles (if deemed necessary by the CQA Consultant) using a metal detector or other approved system.

In addition to these specific duties, CQA Monitors will take note of on-site activities that could result in damage to the soils, geosynthetic, or other components of the lining system. This is particularly important during placement and compaction of the initial lift of soil on top of the geosynthetic materials. Observations so noted by the CQA Monitors will be reported as soon as possible to the CQA Site Manager.

4.0 SITE AND PROJECT CONTROL

4.1 Project Coordination Meetings

4.1.1 Overview

Meetings of key project personnel are necessary to assure a high degree of quality during installation, and promote clear, open channels of communication. Therefore, Project Coordination Meetings are an essential element in the success of the project. There are three types of Project Coordination Meetings, including: (i) pre-construction meetings; (ii) progress meetings; and (iii) problem or work difficulty meetings. These meetings are described below.

4.1.2 Resolution Meeting

After award of the design build contract to the successful bidder, a Resolution Meeting will be held. This meeting will be attended by the DBT, the CQA Managing Engineer(s), the CQA Site Manager(s) and the Project Manager.

The purpose of the Resolution Meeting is to coordinate construction tasks, anticipate installation problems that might cause difficulties and delays in construction, and, above all, present the CQA Plan to all of the parties involved. It is very important that the criteria stated in the CQA Plan regarding testing, repair, etc., be known and accepted by all parties prior to the installation of geosynthetic materials and construction of the soil components of the landfill liner system The first part of the Resolution Meeting may be devoted to a review of the design drawings and specifications for familiarity. This is different from the peer review of the design, including design calculations, which may have been carried out previously.

The Resolution Meeting shall include the following activities:

- distribute relevant documents to all parties;
- review critical design details of the project;
- review this CQA Plan;
- review the Drawings and project specifications;
- make appropriate modifications to the design criteria, Drawings, and project specifications so that the fulfillment of all design specifications or performance standards can be determined through the implementation of the CQA Plan;

- reach a consensus on the quality control procedures, especially on methods of determining acceptability of the soils and geosynthetic materials comprising the liner system;

- assign the responsibilities of each party;

- establish work area security and safety protocol;

- confirm the methods for documenting observations, reporting, and distributing documents and reports; and

- confirm the lines of authority and communication.

The Project Manager will appoint one of the meeting attendees (typically a representative of the Soils CQA Consultant) to record the discussions and decisions of the meeting. A record of the meeting will be documented by the appointee in the form of meeting minutes, which will be subsequently distributed to the attendees.

4.1.3 Pre-Construction Meeting

A Pre-Construction Meeting will be held at the site prior to installation of the geosynthetic materials and construction of soil components. As a minimum, the meeting will be attended by the Owner, DBT including the Geosynthetics Installer's Superintendent, the CQA Managing Engineer(s), the CQA Site Manager(s), and the Project Manager. The Resolution Meeting and Pre-Construction Meeting may be held at the same time if so arranged between the Owner, DBT, the Geosynthetics Installer's Superintendent, the CQA Managing Engineer(s), the CQA Site Manager(s), and the Project Manager.

Specific items for discussion at the geosynthetics pre-construction meeting include the following:

- appropriate modification or clarification to the CQA Plan;

- review the project specifications;

- review the responsibilities of each party;

- review lines of authority and communication;

- establish work area security and safety protocol;

- review methods for documenting and reporting, and for distributing documents and reports;
- communicate to all parties any change in the project documents;
- review critical design details of the project;
- decide the number of spare seaming units for geomembranes to be maintained on site by the Installer (this number depends on the number of seaming crews and on the type of seaming equipment);
- establish protocols for testing;
- establish protocols for handling deficiencies, repairs, and retesting;
- review the time schedule for all operations;
- establish rules for writing on the geomembrane (i.e., who is authorized to write, what can be written and in which color);
- outline procedures for packaging and storing archive samples;
- review panel layout and numbering systems for panels and seams;
- finalize field destructive seam sample size;
- review seam testing procedures;
- review repair procedures;
- conduct a site walk-around to verify that earthwork construction is proceeding on schedule, and to review material storage locations; and
- establish material stockpile locations.

The meeting will be documented by a person designated at the beginning of the meeting. A record of the meeting will be documented by the appointee in the form of meeting minutes, which will be subsequently distributed to the attendees.

4.1.4 Progress Meetings

Weekly progress meetings will be held during construction between the CQA Site Manager(s), the Geosynthetics Installer's Superintendent, the DBC, the Project Manager, and other concerned parties. This meeting will be used to discuss the current progress of the project, planned activities for the upcoming week, and any new business or revisions to the work. The CQA Site Managers will document any problems, decisions, or questions arising at this meeting in their daily reports. Any matter requiring action raised in this meeting will be reported to the appropriate parties. Minutes of the weekly progress meetings will be recorded by Project Manager or his representative and will be distributed to the appropriate parties.

4.1.5 Problem or Work Deficiency Meeting

A special meeting will be held when and if a problem or deficiency is present or is likely to occur. At a minimum, the meeting will be attended by the affected parties (e.g., DBT(s), the Project Manager, the appropriate CQA Manager(s)). If the problem requires a design modification, the DBE should either be present, consulted prior to, or notified immediately upon conclusion of this meeting. The purpose of the meeting is to define and resolve the problem or work deficiency as follows:

- define and discuss the problem or deficiency;
- review alternative solutions;
- select a suitable solution that is acceptable to all parties; and
- implement an action plan to resolve the problem or deficiency.

The meeting will be documented by the Project Manager and minutes will be transmitted to affected parties.

4.2 Project Control Visits

Periodically, the construction site will be visited by the CQA Managing Engineer and/or the CQA Project Manager (if different from the CQA Managing Engineer) of each of the CQA Consultants. If possible, each such visit should be coordinated with a similar visit by the Engineer. The appropriate regulatory officials should be informed of the dates of the visits.

5.0 DOCUMENTATION

5.1 Introduction

An effective CQA plan depends largely on recognition of all construction activities that should be monitored, and on assigning responsibilities for the monitoring of each activity. This is most effectively accomplished and verified by the documentation of quality assurance activities. Each CQA Consultant will document that the quality assurance requirements have been addressed and satisfied.

Each CQA Site Manager will provide the Project Manager with signed descriptive remarks, data sheets, and logs to verify that the monitoring activities have been carried out. Each CQA Site Manager will also maintain at the job site a complete file of plans and specifications, a CQA plan, checklists, test procedures, daily logs, and other pertinent documents.

5.2 Daily Recordkeeping

5.2.1 Overview

Standard reporting procedures will include preparation of a daily CQA report which, at a minimum, will consist of: (i) field notes, including memoranda of meetings and/or discussions with the DBC, Installer, or Project Manager; (ii) CQA monitoring logs and testing data sheets; and (iii) construction problem and solution summary sheets. This information will be regularly submitted to and reviewed by the Project Manager.

5.2.2 Memorandum of Discussion with Contractor of Installer

Each CQA Consultant will prepare a memorandum (i.e., daily field report) each day, summarizing their discussions with the DBC and/or Installer. At a minimum, the memorandum will include the following information:

- date, project name, location, and other identification;
- name of parties to discussion;
- relevant subject matter or issues;
- activities planned;
- constraints or suggestions;

- schedule; and
- signature of the CQA Managing Engineer and/or CQA Site Manager.

5.2.3 Monitoring Logs and Test Data Sheets

Monitoring logs and test data sheets will be prepared daily. At a minimum, these logs and data sheets will include the following information:

- an identifying sheet number for cross referencing and document control;
- date, project name, location, and other identification;
- data on weather conditions;
- a reduced-scale site plan showing the work areas and test locations;
- descriptions and locations of ongoing construction;
- equipment and personnel in each work area, including subcontractors;
- descriptions and specific locations of areas, or units, of work being tested and/or observed and documented;
- locations where tests and samples were taken;
- a summary of test results;
- calibrations or recalibrations of test equipment, and actions taken as a result of recalibration;
- delivery schedule of off-site materials received, including quality control documentation;
- decisions made regarding acceptance of units of work, and/or corrective actions to be taken in instances of substandard quality; and
- signature of the respective CQA Site Manager(s) and/or the CQA Monitor(s).

In any case, all logs must be completely filled out with no items left blank.

5.2.4 Construction Problem and Solution Data Sheets

Sheets describing special construction situations will be cross-referenced with specific observation logs and testing data sheets, and must include the following information, where available:

- an identifying sheet number for cross-referencing and document control;
- a detailed description of the situation or deficiency;
- the location and probable cause of the situation or deficiency;
- how and when the situation or deficiency was found or located;
- documentation of the response to the situation or deficiency;
- final results of any responses;
- measures taken to prevent a similar situation from occurring in the future; and
- the signature of the CQA Manager and/or CQA Monitor, and signature of the Project Manager indicating concurrence.

The Project Manager will be made aware of any significant recurring non-conformance with specifications. The Project Manager will then determine the cause of the non-conformance and recommend appropriate changes in procedures or specifications. These changes will be submitted to the DBE for approval. When this type of evaluation is made, the results will be documented, and any revision to procedures or specifications will be approved by the Owner and the DBE.

A summary of the supporting data sheets, along with final testing results and the CQA Manager's approval of the work, will be required upon completion of construction.

5.3 Photographic Documentation

Photographs will be taken by the CQA Field Monitors and CQA Site Manager in order to serve as a pictorial record of work progress, problems, and mitigation activities. The basic file will contain electronic copies and stored in a folder on a daily basis in chronological order. These records will be presented to the Project Manager upon completion of the project. Photographic reporting data sheets, where used, will be cross-referenced with observation and testing data sheet(s), and/or construction problem and solution data sheet(s).

5.4 Design and/or Specifications Changes

Design and/or specifications changes may be required during construction. In such cases, the CQA Manager will notify the Project Manager and the DBE.

Design and/or specifications changes will be made only with the written agreement of the Project Manager and the DBE, and will take the form of an addendum to the specifications.

5.5 Progress Reports

Each CQA Consultant will prepare a summary progress report each week, or at time intervals established at the pre-construction meeting. As a minimum, this report will include the following information:

- a unique identifying sheet number for cross-referencing and document control;
- the date, project name, location, and other information;
- a summary of work activities during progress reporting period;
- a summary of construction situations, deficiencies, and/or defects occurring during the progress reporting period;
- photographs and descriptions;
- a summary of test results, failures and retests; and
- the signature of the CQA Site Manager.

5.6 Signatures and Final Report

At the completion of the work, the CQA Consultants will submit to the Project Manager signed and sealed Final Reports. These reports will certify: (i) that the work has been performed in compliance with the drawings and specifications; (ii) physical sampling and testing, except as properly authorized, have been conducted at the appropriate frequencies; and (iii) that the summary document provides the necessary supporting information.

At a minimum, this report will include: (i) summaries of all construction activities; (ii) observation logs and testing data sheets including sample location plans; (iii) construction problems and solutions data sheets; (iv) changes from design and material specifications; (v) record drawings; and (vi) a summary statement sealed and signed by a Professional Engineer licensed in the State of Florida.

The record drawings will include scale drawings depicting the location of the construction and details pertaining to the extent of construction (e.g., depths, plan dimensions, elevations, soil component thicknesses, etc.). All surveying and base maps required for development of the record drawings will be prepared by a qualified land surveyor licensed in the State of Florida. These documents will be prepared and assembled by the appropriate CQA Consultant and included as part of the CQA Final Report.

5.7 Storage of Records

Handwritten data sheet originals, especially those containing signatures, should be stored by the Project Manager or his designee in a safe on-site repository. Other reports may be stored by any standard method which will allow for easy access.

6.0 POLYETHYLENE GEOMEMBRANES

6.1 Introduction

This section discusses the CQA of the high-density polyethylene (HDPE) geomembrane components included in the design of the proposed liner system.

6.2 Design Familiarization

A copy of the Drawings and specifications prepared by the DBE will be given to the Geosynthetics CQA Consultant. The Geosynthetics CQA Consultant will review these items for familiarity. This review should not be considered as peer review of the design, which should have been conducted at an earlier stage of the project. The Geosynthetics CQA Consultant will verify that the specifications include at least all properties listed in Tables 1 and 2, which provide the frequency of testing for the geomembrane.

6.3 Geomembrane Material Conformance

6.3.1 Raw Material

The raw material will be new, first quality polyethylene resin meeting the following specifications given by GRI Test Method GM13:

- Specific Gravity (ASTM D 792 Method A): 0.932g/cc, minimum

Compliance testing will be carried out by the Geomembrane Manufacturer to demonstrate that the product meets this Specification.

Prior to the installation of any geomembrane, the Geomembrane Manufacturer will provide the Project Manager and the Geosynthetics CQA Consultant with the following information:

- the origin (Resin Supplier's name and resin production plant), identification (brand name, number) and production date of the resin;
- a copy of the quality control certificates issued by the Resin Supplier;
- reports on the tests conducted by the Geomembrane Manufacturer to verify the quality of the resin used to manufacture the geomembrane rolls assigned to the project (at a minimum, these tests should include specific gravity (ASTM D792),

percent carbon black (ASTM D 1603) and melt index (ASTM D1238 Condition E 190°C, 2.16 kg)); and

The Geosynthetics CQA Consultant will review these documents and report discrepancies with the above requirements to the Project Manager.

6.3.2 Geomembrane Manufacturing

Prior to the installation, the Geomembrane Manufacturer will provide the Project Manager and the Geosynthetics CQA Consultant with the following:

- a properties sheet including, at a minimum, all specified properties, measured using test methods indicated in the specifications, or equivalent;
- a list of quantities and descriptions of materials other than the base polymer which comprise the geomembrane;
- the sampling procedure and results of testing; and
- a certification that property values given in the properties sheet are minimum average roll values and are guaranteed by the Geomembrane Manufacturer.

The Geosynthetics CQA Consultant will verify that:

- the property values certified by the Geomembrane Manufacturer meet all of the specifications; and
- the measurements of properties by the Geomembrane Manufacturer are properly documented and that the test methods used are acceptable.

In addition, the Geosynthetics CQA Consultant may, at the request of the Owner, undertake a manufacturing plant visit, preferably during the production of the particular geomembrane for this project, in order to evaluate the manufacturer's quality control procedures.

6.3.3 Rolls

The Geomembrane Manufacturer will provide the Project Manager and the Geosynthetics CQA Consultant with a quality control certificate for each roll of geomembrane shipped to the site. The quality control certificate will be signed by a responsible party employed by the Geomembrane Manufacturer, such as the production manager. The quality control certificate will include:

- roll numbers and identification; and

- sampling procedures and results of quality control tests - as a minimum, results will be given for thickness and tensile strength evaluated in accordance with the methods indicated in the specifications or equivalent methods approved by the DBE.

The Geosynthetics CQA Consultant will:

- verify that the quality control certificates have been provided at the specified frequency for all rolls, and that each certificate identifies the rolls related to it; and

- review the quality control certificates and verify that the certified roll properties meet the specifications.

6.3.4 Conformance Testing

At the Geomembrane Manufacturer's plant or upon delivery of the rolls of geomembrane at the site, the Geosynthetics CQA Consultant or a designated personnel will verify that samples are removed at the specified frequency and forwarded to the Geosynthetics CQA Laboratory for testing to verify conformance to both the design specifications and the list of guaranteed properties. The minimum number of tests to be performed and test procedures will be as indicated in Table 1. Where optional procedures are noted in the test method, the requirements of the specifications will prevail.

Samples will be taken across the entire width of the roll and will not include the first linear 3 ft (1 m). Unless otherwise specified, samples will be 3-ft (1-m) long by the roll width. The Geosynthetics CQA Consultant will mark the machine direction on the samples with an arrow. The required minimum sampling frequencies are provided in Table 1.

The Geosynthetics CQA Consultant will examine the results from laboratory conformance testing and will report non-conformance to the Project Manager.

The following procedure will apply whenever a sample fails a conformance test that is conducted by the Geosynthetics CQA Laboratory:

- the Installer will replace the roll of geomembrane that is in nonconformance with the specifications with a roll that meets specifications;

- the Installer will remove conformance samples for testing by the Geosynthetics CQA Laboratory from the closest numerical roll on both sides of the failed roll.

These two samples must both conform to specifications. If either of these samples fail, then every roll of geomembrane on site and every roll delivered subsequently must be tested by the Geosynthetics CQA Laboratory for conformance to the specifications. This additional conformance testing will be at the expense of the Installer.

The Geosynthetics CQA Consultant will document actions taken in conjunction with conformance test failures.

6.4 Delivery

6.4.1 Transportation and Handling

Transportation of the geomembrane is the responsibility of the Geomembrane Manufacturer, Fabricator, Installer, or other party as agreed upon. All handling on site is the responsibility of the Installer.

The Geosynthetics CQA Consultant will verify that:

- handling equipment used on the site is adequate and does not pose any risk of damage to the geomembrane; and

- the Installer's personnel handle the geomembranes with care.

Upon delivery at the site, the Installer and the Geosynthetics CQA Consultant will conduct a surface observation of all rolls for defects and for damage. This examination will be conducted without unrolling rolls unless defects or damages are found or suspected. The Geosynthetics CQA Consultant will indicate to the Project Manager:

- rolls, or portions thereof, that should be rejected and removed from the site because they have severe flaws; and

- rolls or factory panels that include minor repairable flaws.

6.4.2 Storage

The Installer will be responsible for the storage of the geomembrane on site. The Project Manager will provide storage space in a location (or several locations) such that on-site

transportation and handling are optimized if possible. Storage space should be protected from theft, vandalism, passage of vehicles, etc.

The Geosynthetics CQA Consultant will verify that storage of the geomembrane ensures adequate protection against dirt, shock, and other sources of damage.

6.5 Geomembrane Installation

6.5.1 Earthwork

6.5.1.1 Surface Preparation

The DBT will be responsible for preparing the supporting soil according to the specifications. The Geosynthetics CQA Consultant will verify that:

- a qualified land surveyor has verified all lines and grades;
- the Soils CQA Consultant has verified that the supporting soil meets the density and moisture content specifications;
- the surface to be covered with geomembrane has been rolled and compacted so as to be free of irregularities, protrusions, loose soil, and abrupt changes in grade;
- the surface of the supporting soil does not contain stones which may be damaging to the geomembrane; and
- there is no area of the subgrade that has been excessively softened by high water content.

The Installer will certify in writing that the surface on which the geomembrane will be installed is acceptable. The certificate of acceptance will be given by the Installer to the Geosynthetics CQA Consultant prior to commencement of geomembrane installation in the area under consideration.

After the supporting soil has been accepted by the Installer, it will be the Installer's responsibility to indicate to the Geosynthetics CQA Consultant any change in the supporting soil condition that may require repair work. If the Geosynthetics CQA Consultant concurs with the Installer, then the Project Manager will ensure that the supporting soil is repaired.

At any time before and during the geomembrane installation, the Geosynthetics CQA Consultant will indicate to the Project Manager any locations that may not provide adequate support for the geomembrane.

6.5.1.2 Anchorage System

Prior to geomembrane placement, anchor trenches will be excavated by the DBC (unless otherwise specified) to the lines, depths, and widths shown on the drawings. The Geosynthetics CQA Consultant will verify that anchor trenches have been constructed according to drawings.

Slightly rounded corners will be provided in trenches where the geomembrane enters the trench so as to avoid sharp bends in the geomembrane. No loose soil will be allowed to underlie the geomembrane in the trenches.

Backfilling of anchor trenches will be conducted in accordance with Section 6.5.5.

6.5.2 Geomembrane Placement

6.5.2.1 Field Panel Identification

A field panel is the unit area of geomembrane which is to be seamed in the field (i.e., a field panel is a roll or a portion of roll cut in the field).

It will be the responsibility of the Geosynthetics CQA Consultant to ensure that each field panel is given an "identification code" (number or letter-number). This identification code will be agreed upon by the Project Manager, Installer, and Geosynthetics CQA Consultant. This field panel identification code should be as simple and logical as possible. (Note that roll numbers established in the manufacturing plant are usually cumbersome and are not related to location in the field.) It will be the responsibility of the Installer to ensure that each field panel placed is marked with the original roll number. The roll number will be marked at a location agreed upon by the Project Manager, Installer, and Geosynthetics CQA Consultant.

The Geosynthetics CQA Consultant if requested by the Project Manager, will establish a table or chart showing correspondence between roll numbers, factory panels, and field panel identification codes. The field panel identification code will be used for all quality assurance records.

6.5.2.2 Field Panel Placement

Location

The Geosynthetics CQA Consultant will verify that field panels are installed at the location indicated in the proposed panel layout plan, as approved or modified.

Installation Schedule

Field panels will be placed one at a time, and each field panel will be seamed immediately after its placement (in order to minimize the number of unseamed field panels exposed to wind).

It is usually beneficial to "shingle" overlaps in the downward direction to facilitate drainage in the event of precipitation. It is also beneficial to proceed in the direction of prevailing winds. Scheduling decisions must be made during installation, in accordance with varying conditions. In any event, the Installer will be fully responsible for the decision made regarding placement procedures.

The Geosynthetics CQA Consultant will evaluate every change in the schedule proposed by the Installer and advise the Project Manager on the acceptability of that change. The Geosynthetics CQA Consultant and Soils CQA Consultant will verify that the condition of the supporting soil has not changed detrimentally during installation.

The Geosynthetics CQA Consultant will record the identification code, location, and date of installation of each field panel.

Weather Conditions

Geomembrane placement will not proceed at an ambient temperature below 40^0 F (5^0 C) or above 104°F (40°C) or when the wind velocity exceeds 20 miles per hour, unless otherwise authorized by the DBE. Geomembrane placement will not be done during any precipitation, in an area of ponded water, or in the presence of excessive winds.

The Geosynthetics CQA Consultant will verify that the above conditions are fulfilled. Additionally, the Geosynthetics CQA Consultant will verify that the supporting soil has not been damaged by weather conditions.

The Geosynthetics CQA Consultant will inform the Project Manager if the above conditions are not fulfilled.

Method of Placement

The Geosynthetics CQA Consultant will verify that:

- any equipment used does not damage the geomembrane by handling, trafficking, excessive heat, leakage of hydrocarbons, or other means;
- the prepared surface underlying the geomembrane has not deteriorated since previous acceptance, and is still acceptable immediately prior to geomembrane placement;
- any geosynthetic elements immediately underlying the geomembrane are clean and free of debris;
- all personnel working on the geomembrane do not smoke, bring glassware on the geomembrane, wear damaging shoes, or engage in other activities that could damage the geomembrane;
- the method used to unroll the panels does not cause scratches or crimps in the geomembrane and does not damage the supporting soil;
- the method used to place the panels minimizes wrinkles (especially differential wrinkles between adjacent panels);
- adequate temporary loading and/or anchoring (e.g., sand bags, tires), not likely to damage the geomembrane, has been placed to prevent uplift by wind (in case of high winds, continuous loading, e.g., by adjacent sand bags, is recommended along the edges of panels to minimize the risk of wind flow under the panels); and
- direct contact with the geomembrane is minimized (i.e., the geomembrane is protected by geotextiles, extra geomembrane, or other suitable materials, in areas where excessive traffic may be expected).
- the geomembrane is protected from damage in heavily trafficked areas

The Geosynthetics CQA Consultant will inform the Project Manager if the above conditions are not fulfilled.

Damage

The Geosynthetics CQA Consultant will visually observe each panel, after placement and prior to seaming, for damage. The Geosynthetics CQA Consultant will advise the Project Manager which panels, or portions of panels, should be rejected, repaired, or accepted. Damaged panels or portions of damaged panels which have been rejected will be marked and their removal from the work area recorded by the Geosynthetics CQA Consultant. On the landfill base, however, damaged panels or portions of panels may be covered with new panels or portions of panels with the approval of the DBE. If this work is performed, it will be recorded by the Geosynthetics CQA Consultant. Repairs will be made according to procedures described in Section 6.5.4.

As a minimum, the Geosynthetics CQA Consultant will verify:

- the panel is placed in such a manner that it is unlikely to be damaged; and
- any tears, punctures, holes, thin spots, etc. are either marked for repair or the panel is rejected.

6.5.3 Field Seaming

6.5.3.1 Seam Layout

Prior to installation, the Installer will provide the Project Manager and the Geosynthetics CQA Consultant with a seam layout drawing (i.e., a drawing of the facility to be lined showing all expected seams). The Geosynthetics CQA Consultant will review the seam layout drawing and verify that it is consistent with the accepted state of practice and this CQA Plan. No panels may be seamed in the field without the Project Manager's approval. In addition, no panels not specifically shown on the seam layout drawing may be used without the Project Manager's prior approval.

In general, seams should be oriented parallel to the line of maximum slope (i.e., oriented along, not across, the slope). In corners and odd-shaped geometric locations, the number of seams should be minimized. No horizontal seam shall be less than 10 ft (3 m) from the toe of slopes, or areas of potential stress concentrations, unless otherwise authorized by the DBE.

A seam numbering system compatible with the panel numbering system will be agreed upon at the Pre-Construction Meeting.

6.5.3.2 Requirements of Personnel

All personnel performing seaming operations will be qualified by experience. Seaming personnel must have seamed at least 10,000 lineal ft (3,000 m) of polyethylene geomembrane seams using the same type of seaming apparatus to be used on this project. At least one seamer will have experience seaming a minimum of 2,000,000 ft^2 (186,000 m^2) of polyethylene geomembrane using the same type of seaming apparatus to be used on this site-specific geomembrane. The most experienced seamer, the "master seamer", will provide direct supervision over less experienced seamers.

The Installer will provide the Project Manager and the Geosynthetics CQA Consultant with a list of proposed seaming personnel and their experience records. This document will be reviewed by the Project Manager and the Geosynthetics CQA Consultant.

6.5.3.3 Seaming Equipment and Products

Overview

Approved processes for field seaming are extrusion welding and fusion welding. Proposed alternate processes shall be documented and submitted to the Project Manager. Only apparatus which have been specifically approved by make and model shall be used. The Installer will use a pyrometer to ensure that accurate temperatures are being achieved, supply a calibrated field tensiometer and a manual dye press. The Project Manager will submit all documentation to the Geosynthetics CQA Consultant for his concurrence.

Extrusion Process

The extrusion-seaming apparatus will be equipped with gauges giving the relevant temperatures in the apparatus and at nozzle.

The Installer will provide documentation regarding the extrudate to the Project Manager and the Geosynthetics CQA Consultant, and will certify that the extrudate welding rod is compatible with the specifications, and in any event is comprised of the same resin as the geomembrane sheeting.

The Installer shall log apparatus temperatures, extrudate temperatures, and ambient temperatures at appropriate intervals. Ambient temperatures will be measured between 0 to 6 in. (0 to 150 mm) above the geomembrane surface.

The Geosynthetics CQA Consultant will verify that:

- the Installer maintains on-site the number of spare operable seaming apparatus decided at the Resolution Meeting;

- equipment used for seaming is not likely to damage the geomembrane;

- the extruder is purged prior to beginning a seam until all heat-degraded extrudate has been removed from the barrel;

- the electric generator is placed on a smooth base such that no damage occurs to the geomembrane (extra fuel or fuel canisters shall be placed so not to come in direct contact with the geomembrane; i.e., place waste/scrap piece between installed liner and bottom of fuel canister) ;

- a smooth insulating plate or fabric is placed beneath the hot seaming apparatus after usage; and

Fusion Process

The fusion-welding apparatus must be automated vehicular-mounted devices. The fusion-welding apparatus will be equipped with gauges giving the applicable temperatures and pressures.

The Geosynthetics CQA Consultant will log ambient seaming apparatus and geomembrane surface temperatures, as well as seaming apparatus pressures.

The Geosynthetics CQA Consultant will also verify that:

- the Installer maintains on-site the number of spare operable seaming apparatus decided at the Resolution Meeting/Pre-Construction Meeting;

- equipment used for seaming is not likely to damage the geomembrane;

- for cross seams, the edge of the cross seam is ground to a smooth incline (top and bottom) prior to seaming;

- the electric generator is placed on a smooth base such that no damage occurs to the geomembrane (extra fuel or fuel canisters shall be placed so not to come in direct contact with the geomembrane; i.e., place waste/scrap piece between installed liner and bottom of fuel canister);

- a smooth insulating plate or fabric is placed beneath the hot welding apparatus after usage; and

- a movable protective layer may be used directly below each overlap of geomembrane that is to be seamed to prevent build-up of moisture between the sheets.

6.5.3.4 Seam Preparation

The Geosynthetics CQA Consultant will verify that:

- prior to seaming, the seam area is clean and free of moisture dust, dirt, debris of any kind, and foreign material;

- if seam overlap grinding is required, the process is completed according to the Geomembrane Manufacturer's instructions within one hour of the seaming operation, and in a way that does not damage the geomembrane;

- the abrading does not extend more than 0.25 in. (6 mm) on either side of the extrusion seam; and

- seams are aligned with the fewest possible number of wrinkles and "fishmouths".

6.5.3.5 Weather Conditions for Seaming

The normally required weather conditions for seaming are as follows:

- unless authorized in writing by the Project Manager, no seaming will be attempted at an ambient temperature below 40°F (5°C) or above 104°F (40°C);

- between ambient temperatures of 32°F (0°C) and 40°F (5°C), seaming is possible if the geomembrane is preheated by either sun or hot air device, and if there is no excessive cooling resulting from wind;

- above an ambient temperature of 40°F (5°C), no preheating is required;

- in all cases, the geomembrane will be dry and protected from wind; and

- the ambient temperatures will be measured between 0 to 6 in. (0 to 150 mm) above the geomembrane surface.

If the Installer wishes to use methods which may allow seaming at ambient temperatures below 40°F (5°C) or above 104°F (40°C), the Installer must demonstrate and certify that such methods produce seams which are entirely equivalent to seams produced at ambient temperatures above 40°F (5°C) and below 104°F (40°C), and that the overall quality of the geomembrane is not adversely affected. Acceptance of the proposed method is the responsibility of the DBE. In addition, an addendum to the contract between the Owner and the Installer is required which specifically states that the seaming procedure does not cause any physical or chemical modification to the geomembrane that will generate any short or long term damage to the geomembrane. Then, the temperatures in the above quality assurance procedure will be modified accordingly.

The Geosynthetics CQA Consultant will verify that these weather conditions are fulfilled and will advise the Project Manager if they are not. The Project Manager will then decide if the installation will be stopped or postponed.

6.5.3.6 Overlapping and Temporary Bonding

The Geosynthetics CQA Consultant will verify that:

- the panels of geomembrane have a finished overlap of a minimum of 3 in. (75 mm) for extrusion seaming and 5 in. (125 mm) for fusion seaming; and

- the procedure used to temporarily bond adjacent panels together does not damage the geomembrane (in particular, the temperature of hot air at the nozzle of any spot seaming apparatus is controlled such that the geomembrane is not damaged).

The Geosynthetics CQA Consultant will log all appropriate temperatures and conditions, and will log and report to the Project Manager any non-compliance.

6.5.3.7 Trial Seams

Trial seams will be made on fragment pieces of geomembrane to verify that seaming conditions are adequate. Such trial seams will be made at the beginning of each seaming period, and at least once every five hours, for each seaming apparatus used in the seaming period. A trial seam will also be made in the event that the ambient temperature varies more than ± 18°F (10°C) since the last passing trial seam. The ambient temperature will be measured between 0 to 6 in. (0 to 150 mm) above the geomembrane. Also, each seamer will make at least one trial seam each seaming period. Trial seams will be made under the same conditions as actual seams. If any seaming apparatus is turned off for any reason, then a new passing trial seam must be completed for that specific seaming apparatus.

The trial seam sample will be at least 3-ft (1-m) long by 1-ft (0.3-m) wide (after seaming) with the seam centered lengthwise for fusion trial seams and at least 10-ft (3-m) long by 1-ft (0.3-m) wide for extrusion trial seams. Seam overlap will be as indicated in Section 6.5.3.6.

Four specimens, each 1 in. (25 mm) wide, and a minimum of 6 in. (150 mm) apart, will be cut from the trial seam sample by the Installer. Two specimens will be tested in shear and two in peel using a field tensiometer, and they should not fail in the seam. If a specimen fails, the entire operation should be repeated. If the additional specimen fails, the seaming apparatus and seamer will not be accepted and will not be used for seaming until the deficiencies are corrected and two consecutive successful full trial seams are achieved.

The Geosynthetics CQA Consultant will observe the trial seam procedures. The remainder of the successful trial seam sample will be assigned a number and marked accordingly by the Geosynthetics CQA Consultant, who will also log the date, hour, ambient temperature, number of seaming unit, name of seamer, and pass or fail description. The sample itself will be retained in the Owner's archives.

6.5.3.8 General Seaming Procedure

Unless otherwise specified, the general seaming procedure used by the Installer will be as follows:

- For fusion seaming, a movable protective layer of plastic may be placed directly below each overlap of geomembrane that is to be seamed, as directed by the Installer. This is to help prevent any moisture build-up between the sheets to be seamed.

- If required, a firm substrate will be provided by using a flat board or similar hard surface directly under the seam overlap to achieve proper support.

- Fishmouths or wrinkles at the seam will first be eliminated by pulling the geomembranes. If this does not work, overlaps will be cut along the ridge of the wrinkle in order to achieve a flat overlap. The cut fishmouths or wrinkles will be seamed and any portion where the overlap is inadequate will then be patched with an oval or round patch of the same geomembrane extending a minimum of 150 mm (6 in.) beyond the cut in all directions.

- If seaming operations are carried out at night, adequate illumination will be provided and must be approved by the DBE.

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

The Geosynthetics CQA Consultant will verify that the above seaming procedures are followed, and will inform the Project Manager if they are not.

6.5.3.9 Nondestructive Seam Continuity Testing

Concept

The Installer will nondestructively test all field seams over their full length using a vacuum test unit, air pressure test (for double fusion seams only), or other approved method. Vacuum testing and air pressure testing are described in *Vacuum Testing* and the *Air Pressure Testing* of Section 6.5.3.9. The purpose of nondestructive tests is to check the continuity of seams. It does not provide any information on seam strength. Continuity testing will be carried out as the seaming work progresses, not at the completion of all field seaming. Nondestructive testing will not be permitted before sunrise or after sunset unless the Installer demonstrates capabilities to do so.

The Geosynthetics CQA Consultant will:

- observe the continuity testing;
- record location, date, test unit number, name of tester, and outcome of the testing; and
- inform the Installer and Project Manager of any required repairs.

The Installer will complete any required repairs in accordance with Section 6.5.4.

The Geosynthetics CQA Consultant will:

- observe the repair and re-testing of the repair;
- mark on the geomembrane that the repair has been made; and
- document the results.

The following procedures will apply to locations where seams cannot be nondestructively tested.

- Prior to seaming, the seamer will make a new passing trial seam.

- All such seams will be cap-stripped with the same geomembrane.

- If the seam is accessible to testing equipment prior to final installation, then the seam will be nondestructively tested prior to final installation.

- If the seam cannot be tested prior to final installation, then the seaming and cap-stripping operations will be observed by the Geosynthetics CQA Consultant and Installer for uniformity and completeness.

The seam number, date of observation, name of tester, and outcome of the test or observation will be recorded by the Geosynthetics CQA Consultant.

Vacuum Testing

The equipment will be comprised of the following:

- a vacuum box assembly consisting of a rigid housing, a transparent viewing window, a soft neoprene gasket attached to the bottom, port hole or valve assembly, and a vacuum gauge;

- a vacuum tank and pump assembly equipped with a pressure controller and pipe connections;

- a pressure/vacuum hose with fittings and connections;

- an approved applicator; and

- a soapy solution.

The following procedures will be followed:

- if vacuum testing a fusion seam, the flap must be cut off with an approved cutter, prior to testing to expose the seam;

- energize the vacuum pump and reduce the tank pressure to approximately 5 psi (34 kPa) (minimum) gauge;

- with a soapy solution, wet a strip of geomembrane which is 6 in. (150 mm) larger in area than the vacuum box;

- place the box over the wetted area;
- close the bleed valve and open the vacuum valve;
- ensure that a leak tight seal is created;
- for a period of not less than 30 seconds, examine the geomembrane through the viewing window for the presence of soap bubbles;
- if no bubble appears after 30 seconds, close the vacuum valve and open the bleed valve. Before moving the box over the next adjoining area, place a mark (with a marker that will not damage the geomembrane) on the geomembrane at the leading edge of the viewing window, then move the box over the next adjoining area so that the last mark on the geomembrane is at the rear of the viewing window, and repeat the process; and
- all areas where soap bubbles appear will be marked and repaired in accordance with Section 6.5.4.

Air Pressure Testing (For Double Fusion Seam Only)

The following procedures are applicable to those processes which produce a double seam with an enclosed space.

The equipment will be comprised of the following:

- an air pump equipped with a pressure gauge capable of generating and sustaining a pressure between 25 and 30 psi (175 and 210 kPa) and mounted on a cushion to protect the geomembrane;
- a hose with fittings and connections; and
- a sharp hollow needle, or other approved pressure feed device.

The following procedures will be followed:

- seal both ends of the seam to be tested;

- insert the needle or other approved pressure feed device into the tunnel created by the fusion seam;

- insert a protective cushion between the air pump and the geomembrane;

- energize the air pump to a pressure between 25 and 30 psi (175 and 210 kPa), close the valve, and sustain the pressure for not less than 5 minutes;

- if any loss of pressure exceeds 3 psi (23 kPa) or if the pressure does not stabilize, locate the faulty area and repair it in accordance with Section 6.5.4;

- to verify that there is airflow through the entire tunnel, remove the seal at the end of the tunnel away from the air source and listen for the escaping air, and if it is found that there is a blockage in the channel, the entire seam must be vacuum tested as described previously;

- if upon checking for complete airflow it is found that there is a blockage in the tunnel, the seam will be vacuum tested; and

- remove the needle or other approved pressure feed device and seal the hole.

6.5.3.10 Destructive Testing

Concept

Destructive seam tests will be performed at selected locations. The purpose of these tests is to evaluate seam strength. Seam strength testing will be done as the seaming work progresses, not at the completion of all field seaming. The seam testing requirements are summarized in Table 2.

Location and Frequency

The Geosynthetics CQA Consultant will select locations where seam samples will be cut out for laboratory testing. Those locations will be established as follows.

- A minimum frequency of one test location per 500 ft (150 m) of seam length. This minimum frequency is to be determined as an average taken throughout the entire facility. The frequency may be increased to 1000 ft (300 m) if electric leak location testing is performed.

- A maximum frequency will be agreed upon by the Installer, Project Manager and Geosynthetics CQA Consultant at the Resolution and/or Pre-Construction Meeting.

- Test locations will be determined during seaming at the Geosynthetics CQA Consultant's discretion. Selection of such locations may be prompted by suspicion of excess crystallinity, contamination, offset seams, or any other potential cause of imperfect seaming.

The Installer will not be informed in advance of the locations where the seam samples will be taken.

Sampling Procedure

Samples will be cut by the Installer as the seaming progresses in order to have laboratory test results before the geomembrane is covered by another material. The Geosynthetics CQA Consultant will:

- observe sample cutting;

- assign a number to each sample, and mark it accordingly;

- record the sample location on the layout drawing; and

- if necessary, record the reason for taking the sample at this location (e.g., statistical routine, suspicious feature of the geomembrane).

All holes in the geomembrane resulting from destructive seam sampling will be immediately repaired in accordance with repair procedures described in Section 6.5.4.3. The continuity of the new seams in the repaired area will be tested according to Section 6.5.3.9.

Size and Distribution of Samples

At a given sampling location, two types of samples will be taken by the Installer.

First, two specimens for field testing should be taken. Each of these specimens will be 1-in. (25-mm) wide by 12-in. (300-mm) long, with the seam centered parallel to the width. The distance between these two specimens will be 4 ft (1.2 m). If both specimens pass the field test described in *Field Testing* of Section 6.5.3.10, a sample for laboratory testing will be taken.

The sample for laboratory testing will be located between the two specimens for field testing. The destructive sample will be 1-ft (0.3-m) wide by 3.5-ft (1.1-m) long with the seam centered lengthwise. The sample will be cut into three parts and distributed as follows:

- one portion to the Installer, 1 ft by 1 ft (0.3 m by 0.3 m);
- one portion to the Owner for archive storage, 1 ft by 1 ft (0.3 m by 0.3 m); and
- one portion for Geosynthetics CQA Laboratory testing, 1 ft by 1.5 ft (0.3 m by 0.45 m).

Final determination of the sample sizes will be made at the Pre-Construction Meeting.

Field Testing

The two 1-in. (25-mm) wide specimens described above will be tested in the field, by hand and tensiometer, for peel and shear respectively, and should not fail at the seam location. If any field test sample fails to pass, then the procedures outlined in *Procedures for Destructive Test Failures* of Section 6.5.3.10 will be followed.

The Geosynthetics CQA Consultant will witness all field tests and mark all samples and portions with their number. The Geosynthetics CQA Consultant will also log the date and time, ambient temperature, number of seaming unit, name of technician, seaming apparatus temperatures and speeds, pass or fail description, and attach a copy to each sample portion.

Geosynthetics CQA Laboratory Testing

Destructive test samples will be packaged and shipped, if necessary, under the responsibility of the Geosynthetics CQA Consultant in a manner which will not damage the test sample. The Project Manager will verify that packaging and shipping conditions are acceptable. The Project Manager will be responsible for storing the archive samples. This procedure will be fully outlined at the Resolution Meeting. Test samples will be tested by the Geosynthetics CQA Laboratory. The Geosynthetics CQA Laboratory will be selected by the Geosynthetics CQA Consultant with the concurrence of the Project Manager.

Testing will include “Seam Strength” (ASTM D 4437) and “Peel Adhesion” (ASTM D 4437). The minimum acceptable values to be obtained in these tests are those indicated in the specifications. At least 5 specimens will be tested for each test method. Specimens will be selected alternately by test from the samples (i.e., peel, shear, peel, shear).

The Geosynthetics CQA Laboratory will provide test results no more than 24 hours after they receive the samples. The Geosynthetics CQA Manager will review laboratory test results as soon as they become available, and make appropriate verbal recommendations to the Project Manager.

The verbal recommendations will be followed by written recommendations.
Should field tensiometer testing be carried out, the following procedure should be followed: if the test passes, the sample qualifies for testing in the laboratory; if it fails, the seam should be repaired in accordance with *Procedures for Destructive Test Failure* of Section 6.5.3.10.

Installer's Laboratory Testing

The Installer's laboratory test results will be presented to the Project Manager and the Geosynthetics CQA Consultant for comments in writing.

Procedures for Destructive Test Failure

The following procedures will apply whenever a sample fails a destructive test, whether that test is conducted by the Geosynthetics CQA Laboratory, the Installer's laboratory, or by field tensiometer.

The Installer has two options.

- The Installer can reconstruct (e.g., remove the old seam and reseam) the seam between any two passed destructive seam test locations.

- The Installer can trace the seaming path to an intermediate location (at 10 ft (3 m) minimum from the point of the failed test in each direction) and take a small sample for an additional field test at each location. If these additional samples pass tensiometer testing, then full destructive laboratory samples are taken. If these destructive laboratory samples pass the tests, then the seam is reconstructed between these locations by capping. If either sample fails, then the process is repeated to establish the zone in which the seam should be reconstructed.

If a fusion type seam fails destructive testing and the Installer chooses to cap the seam, then the only acceptable capping method is as described in Section 6.5.4.3. Applying topping is not an approved method of capping long lengths of seam.

All acceptable seams must be bounded by two locations from which destructive samples passing laboratory tests have been taken. In cases exceeding 150 ft (50 m) of reconstructed seam, a sample will be taken from the zone in which the seam has been reconstructed. This sample must pass destructive testing or the procedure outlined in this section must be repeated.

The Geosynthetics CQA Consultant will document all actions taken in conjunction with destructive test failures.

6.5.4 Defects and Repairs

6.5.4.1 Identification

All seams and non-seam areas of the geomembrane will be examined by the Geosynthetics CQA Consultant for identification of defects, holes, blisters, undispersed raw materials and any sign of contamination by foreign matter. Because light reflected by the geomembrane helps to detect defects, the surface of the geomembrane will be clean at the time of examination. The geomembrane surface will be broomed or washed by the Installer if the amount of dust or mud inhibits examination.

6.5.4.2 Evaluation

Each suspect location both in seam and non-seam areas will be non-destructively tested using the methods described in Section 6.5.3.9 as appropriate. Each location which fails the non-destructive testing will be marked by the Geosynthetics CQA Consultant and repaired by the Installer. Work will not proceed with any materials which will cover locations which have been repaired until laboratory test results with passing values are available.

6.5.4.3 Repair Procedures

Any portion of the geomembrane exhibiting a flaw, or failing a destructive or nondestructive test, will be repaired. Several procedures exist for the repair of these areas. The final decision as to the appropriate repair procedure will be agreed upon between the Project Manager, Installer, and Geosynthetics CQA Consultant.

The procedures available include:

- patching, used to repair large holes, tears, undispersed raw materials, and contamination by foreign matter;
- grinding and reseaming, used to repair small sections of extruded seams;
- spot seaming, used to repair minor or localized flaws;
- capping, used to repair large lengths of failed seams; and

- topping, used to repair areas of inadequate seams, which have an exposed edge;

In addition, the following provisions will be satisfied:

- surfaces of the geomembrane which are to be repaired will be abraded no more than one hour prior to the repair;
- all surfaces must be clean and dry at the time of the repair;
- all seaming equipment used in repairing procedures must be approved;
- the repair procedures, materials, and techniques will be approved in advance of the specific repair by the Project Manager, Geosynthetic CQA Consultant, and Installer;
- patches or caps will extend at least 6 in. (150 mm) beyond the edge of the defect, and all corners of patches will be rounded with a radius of at least 3 in. (75 mm); and
- the geomembrane below large caps should be appropriately cut to avoid water or gas collection between the two sheets.

6.5.4.4 Verification of Repairs

Each repair will be numbered and logged. Each repair will be non-destructively tested using the methods described in Section 6.5.3.9 as appropriate. Repairs which pass the non-destructive test will be taken as an indication of an adequate repair. Large caps may be of sufficient extent to require destructive testing, at the discretion of the Geosynthetics CQA Consultant. Failed tests will require the repair to be redone and retested until a passing test results. The Geosynthetics CQA Consultant should observe all non-destructive testing of repairs and will record the date of the repair and test outcome.

6.5.4.5 Large Wrinkles

When seaming of the geomembrane is completed (or when seaming of a large area of the geomembrane is completed) and prior to placing overlying materials, the Geosynthetics CQA Consultant will observe the geomembrane wrinkles. The Geosynthetics CQA Consultant will indicate to the Project Manager which wrinkles should be cut and reseamed by the Installer. The seam thus produced will be tested like any other seam.

6.5.5 Backfilling of Anchor Trench

Anchor trenches will be adequately drained, to prevent ponding or otherwise softening of the adjacent soils while the trench is open. Anchor trenches will be backfilled and compacted by the DBC or the Installer on a timely basis. Care will be taken when backfilling the trenches to prevent any damage to the geosynthetics. The Geosynthetics CQA Consultant will observe the backfilling operation and advise the Project Manager of any problems.

6.5.6 Geomembrane System Acceptance

The Installer and the Manufacturers will retain all ownership and responsibility for the geosynthetics until acceptance by the Owner. The geosynthetic will be accepted by the Owner when:

- the installation is finished;
- verification of the adequacy of all seams and repairs, including associated testing, is complete;
- Installer's representative furnishes the Project Manager with certification that the geomembrane was installed in accordance with the Manufacturer's recommendations as well as the plans and specifications;
- all documentation of installation is completed including the Geosynthetics CQA Consultant's final report; and
- a final report, including record drawings, sealed by a Professional Engineer licensed in the Commonwealth of Pennsylvania, have been received by the Project Manager.

The Geosynthetics CQA Consultant will certify that installation has proceeded in accordance with the geosynthetics potion of the CQA Plan for the project except as noted to the Project Manager.

6.5.7 Materials in Contact with the Geomembrane

6.5.7.1 Introduction

The quality assurance procedures indicated in this section are only intended to assure that the installation of these materials does not damage the geomembrane. Additional quality assurance procedures are necessary to assure that systems built with these materials will be constructed in such a way to enable proper performance.

6.5.7.2 Soils

A copy of the Specifications prepared by the DBE for placement of soils will be given to the Geosynthetics CQA Consultant by the Project Manager. The Geosynthetics CQA Consultant will verify that these Specifications are consistent with the state of practice as described below.

- Placement of soils on the geomembrane shall not proceed at an ambient temperature below 40^0 F (5^0C) nor above 104^0F (40^0C) unless otherwise specified.

- Placement of soils above the geomembrane shall be done in such a manner as not to create waves or wrinkles in the geomembrane of a size that could fold over.

- Soils must be free of objects that could cause damage to the geosynthetics.

- A geotextile or other cushion approved by the DBE may be installed between angular aggregate and the geomembrane.

- Equipment used for placing soil will not be driven directly on the geomembrane.

- A minimum thickness of 1 ft (0.3 m) of uncompacted soil is specified between a light dozer (such as a wide pad Caterpillar D-3 or lighter) and the geomembrane.

- A minimum thickness of 3 ft (0.9 m) of soil is specified between rubber-tired vehicles and the geomembrane.

- In heavily trafficked areas such as access ramps, soil thickness should be at least 3 ft (0.9 m).

The Geosynthetics CQA Consultant will:

- measure soil thickness and verify that the required thicknesses are present (or, if applicable, verify that required measurements have been completed by the Soils CQA Consultant, if any);

- verify that final thicknesses are consistent with design; and

- verify that placement of soil is done in such a manner that geomembrane damage is unlikely.

The Geosynthetics CQA Consultant will inform the Project Manager if the above conditions are not fulfilled.

6.5.7.3 Appurtenances

A copy of the specifications prepared by the DBE for appurtenances will be given by the Project Manager to the Geosynthetics CQA Consultant for review.

The Geosynthetics CQA Consultant will verify that:

- installation of the geomembrane in appurtenance areas, and connection of geomembrane to appurtenances have been made according to Specifications;

- extreme care is taken while seaming around appurtenances since neither non-destructive nor destructive testing may be feasible in these areas; and

- the geomembrane has not been visibly damaged while making connections to appurtenances.

The Geosynthetics CQA Consultant will inform the Project Manager if the above conditions are not fulfilled.

7.0 GEOTEXTILES

7.1 Design Familiarization

A copy of the Drawings and specifications prepared by the DBE will be given to the Geosynthetics CQA Consultant. The Geosynthetics CQA Consultant will review these and verify that they are conceptually consistent with the state of practice, and are clear and complete. This review is not considered as a peer review of the design.

7.2 Manufacturing Quality Control

The Geotextile Manufacturer will provide the Project Manager with a list of guaranteed "minimum average roll value" properties as defined by GRI Test Method GT 12, for the type of geotextile to be delivered. The Geotextile Manufacturer will also provide the Project Manager with a written certification signed by a responsible party that the materials actually delivered have "minimum average roll values" properties which meet or exceed all property values guaranteed for that type of geotextile.

The Geosynthetics CQA Consultant will examine the manufacturer certifications to verify that the property values listed on the certifications meet or exceed those specified for the particular type of geotextile. Deviations will be reported to the Project Manager.

7.3 Labeling

The Geotextile Manufacturer will identify all rolls of geotextile with the following:

- manufacturer's name;
- product identification;
- lot number;
- roll number; and
- roll dimensions.

Additionally, if any special handling of the geotextile is required, it will be so marked on the top surface of the geotextile, e.g., "This Side Up".

The Geosynthetics CQA Consultant will examine rolls upon delivery and any deviation from the above requirements will be reported to the Project Manager.

7.4 Shipment and Storage

During shipment and storage, the geotextile will be protected from ultraviolet light exposure, precipitation or other inundation, mud, dirt, dust, puncture, cutting or any other damaging or deleterious conditions. To that effect, geotextile rolls will be shipped and stored in relatively opaque and watertight wrappings.

Geotextiles will not be exposed to precipitation prior to being installed. Wrappings protecting geotextile rolls will be removed less than one hour prior to unrolling the geotextile. After the wrapping has been removed, the geotextiles will not be exposed to sunlight for more than 30 days, unless otherwise specified and guaranteed by the Geotextile Manufacturer.

The Geosynthetics CQA Consultant will observe rolls upon delivery at the site and any deviation from the above requirements will be reported to the Project Manager. Damaged rolls will be rejected and replaced at no cost to the Owner.

7.5 Conformance Testing

7.5.1 Tests

Geotextile samples shall be taken at the site or at the manufacturer's plant and forwarded to the Geosynthetics CQA Laboratory for testing to ensure conformance to both the Specifications and the Manufacturers list of guaranteed properties.

As a minimum, the following tests will be performed on geotextiles:

- mass per unit area;
- grab strength;
- tear strength;
- puncture strength;
- apparent opening size, when the geotextile is used as a filter; and
- permittivity, when the geotextile is used as a filter.

The conformance tests will be performed in accordance with the test methods specified in the specifications. Other conformance tests may be required by the specifications. Testing frequency for the geotextiles is presented in Table 3.

7.5.2 Sampling Procedures

Samples of the manufactured geotextile components may be taken at the plant or at the site. Samples will be taken across the entire width of the roll and will not include the first 3 ft (1 m). Unless otherwise specified, samples will be 3-ft (1-m) long by the roll width. Mark the machine direction on the samples with an arrow.

7.5.3 Test Results

The Geosynthetics CQA Consultant will examine the results from laboratory conformance testing and will report non-conformance to the Project Manager.

7.5.4 Conformance Test Failure

The following procedure will apply whenever a sample fails a conformance test that is conducted by the Geosynthetics CQA Laboratory.

- The Installer will replace the roll of geotextile that is in nonconformance with the specifications with a roll that meets specifications.

- The Installer will remove conformance samples for testing by the Geosynthetics CQA Laboratory from the closest numerical roll on both sides of the failed roll. These two samples must conform to the specifications. If either of these samples fail, the five numerically closest untested rolls on both sides of the failed sample will be tested by the Geosynthetics CQA Laboratory. These ten samples must conform to the specifications. If any of these samples fail, every roll of geotextile on site and every subsequently delivered roll that is from the same supplier must be tested by the Geosynthetics CQA Laboratory for conformance to the specifications. This additional conformance testing will be at the expense of the Installer.

The Geosynthetics CQA Consultant will document actions taken in conjunction with conformance test failures.

7.6 Handling and Placement

The Installer will handle all geotextiles in such a manner as to ensure they are not damaged in any way, and the following will be complied with:

- on slopes, the geotextiles will be securely anchored in the anchor trench and then rolled down the slope in such a manner as to continually keep the geotextile panel in tension;

- in the presence of wind, all geotextiles will be weighted with sandbags or the equivalent; sandbags installed during placement are to remain in place until replaced with protective cover soils;

- geotextiles will be kept continually under tension to minimize the presence of wrinkles in the geotextile;

- geotextiles will be cut using an approved geotextile cutter only; if in place, then special care must be taken to protect other materials from damage which could be caused by the cutting of the geotextiles;

- the Installer will take any necessary precautions to prevent damage to the underlying layers during placement of the geotextile;

- during placement of geotextiles, care will be taken not to entrap in the geotextile stones, excessive dust, or moisture that could damage the geomembrane, generate clogging of drains or filters, or hamper subsequent seaming;

- a visual examination of the geotextile will be carried out over the entire surface, after installation, to ensure that no potentially harmful foreign objects, such as needles, are present; and

- if white colored geotextile is used, then precautions will be taken against “snow blindness” of personnel.

The Geosynthetics CQA Consultant will note any noncompliance and report it to the Project Manager.

7.7 Seams and Overlaps

All geotextiles shall be continuously sewn or thermally bonded (i.e., Geo-welded). Geotextiles will be overlapped at a minimum of 6 in. (150 mm) prior to seaming. No horizontal seams will be allowed on side slopes steeper than 10 percent (i.e. seams will be along, not across, slopes steeper than 10H:1V), except as part of a patch.

The Installer will pay particular attention at seams to ensure that no protective cover material could be inadvertently inserted beneath the geotextile.

Sewing will be done using polymeric thread.

7.8 Repair

Holes or tears in the geotextile will be repaired as follows.

- On slopes steeper than 10 percent, a patch made from the same geotextile will be seamed into place no closer than 1 in. (25 mm) from any edge. Should any tear exceed 10 percent of the width of the roll, that roll will be removed from the slope and replaced.

- On slopes flatter than 10 percent, a patch made from the same geotextile will be spot-seamed in place with a minimum of 2-ft (0.6-m) overlap in all directions.

- Care will be taken to remove soil or other material that may have penetrated the torn geotextile.

The Geosynthetics CQA Consultant will observe the repair of geotextiles, note any non-compliance with the above requirements, and report them to the Project Manager.

7.9 Placement of Soil Materials

The Installer will place soil materials located on top of a geotextile in such a manner as to ensure:

- no damage to the geotextile;

- minimal slippage of the geotextile on underlying layers; and

- no excess tensile stresses in the geotextile.

Unless otherwise specified by the DBE, a minimum thickness of 1 ft (0.3 m) of soil is specified between a low ground-pressure dozer and the geotextile.

Non-compliance will be noted by the Geosynthetics CQA Consultant and reported to the Project Manager.

8.0 GEONETS

8.1 Design Familiarization

A copy of the geonet drawings and specifications prepared by the DBE will be given to the Geosynthetic CQA Consultant. The Geosynthetic CQA Consultant will review these and verify that the geonets can be installed as indicated. This review is not a peer review of the design (including a review of the calculations) which should have been conducted at an earlier stage of the project. The section apply only if geonet is proposed in the project.

8.2 Manufacturing Quality Control

The Geonet Manufacturer will provide the Project Manager and the Geosynthetic CQA Consultant with a list of guaranteed properties for the type of geonet to be supplied. The Geonet Manufacturer will provide the Project Manager and the Geosynthetics CQA Consultant with a written certification signed by a responsible party that the geonets actually delivered have "minimum average roll values" properties which meet or exceed the guaranteed properties.

The Geosynthetic CQA Consultant will examine the manufacturer's certifications to verify that the property values listed on the certifications meet or exceed those specified. Deviations will be reported to the Project Manager.

8.3 Labeling

The Geonet Manufacturer will identify all rolls of geonet with the following:

- manufacturer's name;
- product identification;
- lot number;
- roll number; and
- roll dimensions.

The Geosynthetic CQA Consultant will examine rolls upon delivery and any deviation from the above requirements will be reported to the Project Manager.

8.4 Shipment and Storage

Geonet cleanliness is essential to their performance and geonet rolls should be wrapped in plastic or otherwise protected against dust and dirt during shipping and storage.

The wrapping should be removed less than one hour before placement. The Geosynthetic CQA Consultant will verify that geonets are free of dirt and dust just before installation. The Geosynthetic CQA Consultant will report the outcome of this verification to the Project Manager, and if the geonets are judged dirty or dusty, they will be washed by the Installer prior to installation.

Washing operations will be observed by the Geosynthetic CQA Consultant and improper washing operations will be reported to the Project Manager.

8.5 Conformance Testing

8.5.1 Tests

Geonet samples shall be collected at the site or at the manufacturer's plant and forwarded to Geosynthetic CQA Laboratory for testing, at the frequency indicated in Table 4, to verify conformance to both the design specifications and the list of guaranteed properties.

As a minimum, the following tests will be performed:

- polymer specific gravity;
- carbon black content;
- thickness; and
- hydraulic transmissivity, if required by the DBE.

8.5.2 Sampling Procedures

Samples of the geonet may be taken at the plant or at the site. Samples will be taken across the entire width of the roll and will not include the first 3 ft (1 m). Unless otherwise specified, samples will be 3-ft (1-m) long by the roll width. Mark the machine direction on the samples with an arrow.

Unless otherwise specified, samples will be taken at a rate of one per lot or one per 100,000 ft^2 (10,000 m^2), whichever yields the greater number of samples, as indicated in Table 4.

8.5.3 Test Results

The Geosynthetic CQA Consultant will examine results from laboratory conformance testing and will report any nonconformance to the Project Manager.

8.6 Handling and Placement

The Installer will handle all geonets in such a manner as to ensure the geonets are not damaged in any way, and comply with the following.

- On sideslopes, the geonets will be secured in the anchor trench and then rolled down the slope in such a manner as to continually keep the geonet sheet in tension. If necessary, the geonet will be positioned by hand after being unrolled to minimize wrinkles. Geonets can be placed in the horizontal direction (i.e., across the slope) in some special locations (e.g., at the toe of a slope, if an extra layer of geonet is required, this extra layer of geonet can be placed in the horizontal direction). Such locations will be identified by the DBE on the Drawings. (It should be noted that placement of layers of geonets at 90° angles to each other will result in a partial loss of effective thickness and transmissivity.

- In the presence of wind, all geonets will be weighted with sandbags or the equivalent. Such sandbags will be installed during placement and will remain until replaced with cover material.

- Unless otherwise specified, geonets will not be welded to geomembranes.

- Geonets will only be cut using scissors or hooked blade utility knife.

- The Installer will take any necessary precautions to prevent damage to underlying layers during placement of the geonet.

- During placement of geonets, care will be taken not to entrap in the geonet dirt or excessive dust that could cause clogging of the drainage system, and/or stones that could damage the adjacent geomembrane. If dirt or excessive dust is entrapped in the geonet, it should be hosed clean prior to placement of the next material on top of it. In this regard, care should be taken with the handling of sandbags, to prevent rupture or damage of the sandbag.

- Care should be taken not to leave tools on the geonet.

The Geosynthetic CQA Consultant will note noncompliance and report it to the Project Manager.

8.7 Stacking and Joining

When several layers of geonets are stacked, care should be taken to prevent strands from one layer from penetrating the channels of the next layer, thereby significantly reducing the transmissivity. This cannot happen if stacked geonets are placed in the same direction. A stacked geonet will never be laid in perpendicular directions to the underlying geonet (unless otherwise specified by the DBE). In the corners of side slopes of rectangular landfills, adjacent overlapping geonets are usually perpendicular and special precautions will be taken as discussed below.

Adjacent geonets will be joined according to Drawings and specifications. As a minimum, the following requirements will be met.

- Adjacent rolls will be overlapped by at least 4 in. (100 mm).
- These overlaps will be secured by tying.
- Tying will be achieved by nylon cable ties, plastic fasteners, or polymer braid. Tying devices will be white or yellow for easy observation. Metallic devices are not allowed.
- Tying will be every 5 ft (1.5 m) along the slope, every 2 ft (0.6 m) across the slope, and every 6 in. (150 mm) in the anchor trench and at tie-ins with previously deployed material from a former project. Tying will be every 10 ft (3.0 m) on horizontal surface.
- In the corners of the side slopes of rectangular landfills, where overlaps between perpendicular geonet strips are required, an extra layer of geonet will be unrolled along the slope, on top of the previously installed geonets, from top to bottom of the slope.
- When more than one layer of geonet is installed, joints will be staggered a minimum of 1 ft (0.3 m).

The Geosynthetic CQA Consultant will note noncompliance and report it to the Project Manager.

8.8 Repair

Any holes or tears in the geonet will be repaired by placing a patch extending 2 ft (0.6 m) beyond edges of the hole or tear. The patch will be secured to the original geonet by tying every 6 in. (150 mm). Tying devices will be as indicated in Section 8.7. If the hole or tear width across the roll is more than 50 percent the width of the roll, then the damaged area will be cut out and the two portions of the geonet will be joined as indicated in Section 8.7.

The Geosynthetic CQA Consultant will observe any repair, note any non-compliance with the above requirements and report them to the Project Manager.

8.9 Placement of Soil Materials

Although soil should never be placed in contact with geonets, the Installer will ensure that all soil materials are placed in such a manner as to ensure:

- the geonet and underlying geosynthetic materials are not damaged;
- minimal slippage of the geonet on underlying layers occurs; and
- no excess tensile stresses occur in the geonet.

Noncompliance will be noted by the Geosynthetic CQA Consultant and reported to the Project Manager.

If portions of the geonet are exposed, the Geosynthetic CQA Consultant will periodically place marks on the geonet and the underlying geomembrane and measure the elongation of the geonet during the placement of soil.

9.0 GEOCOMPOSITE

9.1 Design Familiarization

A copy of the Drawings and specifications prepared by the DBE will be given to the Geosynthetics CQA Consultant. The Geosynthetics CQA Consultant will review these and verify that the geocomposite can be installed as indicated. This review is not a peer review of the design. Peer review should have been conducted at an earlier stage of the project.

9.2 Manufacturing Quality Control

The Geocomposite Manufacturer will provide the Project Manager and the Geosynthetic CQA Consultant with a list of guaranteed properties for the type of geonet to be supplied. The Geocomposite Manufacturer will provide the Project Manager and the Geosynthetics CQA Consultant with a written certification signed by a responsible party that the geocomposites actually delivered have "minimum average roll values" properties which meet or exceed the guaranteed properties.

The Geosynthetic CQA Consultant will examine the manufacturer's certifications to verify that the property values listed on the certifications meet or exceed those specified. Any deviations will be reported to the Project Manager.

9.3 Labeling

The Geocomposite Manufacturer will identify all rolls of geocomposite with the following:

- manufacturer's name;
- product identification;
- lot number;
- roll number; and
- roll dimensions.

If any special handling is required, it will be so marked on the geotextile component, e.g., "This Side Up", or "This Side Against Soil To Be Retained". The Geosynthetic CQA Consultant will examine rolls upon delivery and any deviation from the above requirements will be reported to the Project Manager.

9.4 Shipment and Storage

Geocomposite cleanliness is essential to its performance and geocomposite rolls should be wrapped in plastic or otherwise protected against dust and dirt during shipping and storage.

The wrapping should be removed less than one hour before placement. The Geosynthetic CQA Consultant will verify that geocomposites are free of dirt and dust just before installation. The Geosynthetic CQA Consultant will report the outcome of this verification to the Project Manager, and if the geocomposites are judged dirty or dusty, they will be cleared by the Installer prior to installation.

9.5 Conformance Testing

9.5.1 Tests

Geocomposite samples shall be collected at site or at the manufacturer's plant and sent to Geosynthetic CQA Laboratory for testing, at the frequency indicated in 9.5.2, to verify conformance to both the design specifications and Table 5.

9.5.2 Sampling Procedures

Samples of the geonet and geotextile components of the geocomposite shall be taken at the Geocomposite Manufacturer's plant prior to manufacturing the geocomposite. Samples of the manufactured geocomposite components may be taken at the plant or at the site. Samples of the geotextile component will be obtained as in accordance with *Sampling Procedures* of Section 7.5.2. Samples of the geonet component will be obtained in accordance with *Sampling Procedures* of Section 8.5.2. Samples of the geocomposite will be taken across the entire width of the roll and will not include the first 3 ft (1 m). Unless otherwise specified, samples will be 3-ft (1-m) long by the roll width. The Geosynthetic CQA Consultant will mark the machine direction on the samples with an arrow.

Unless otherwise specified, samples will be taken at a rate of one per lot or one per 100,000 ft^2 (10,000 m^2), whichever yields the greater number of samples.

9.5.3 Test Results

The Geosynthetic CQA Consultant will examine the results from laboratory conformance testing and will report nonconformance to the Project Manager.

9.5.4 Conformance Test Failure

The following procedure will apply whenever a sample fails a conformance test that is conducted by the Geosynthetic CQA Laboratory.

- The Manufacturer will replace every roll of geocomposite that is in nonconformance with the specifications with a roll that meets specifications.

- The Installer will remove conformance samples for testing by the Geosynthetic CQA Laboratory from the closest numerical rolls on both sides of the failed roll. These two samples must conform to the specifications. If either of these samples fail, the five numerically closest rolls on the side of the failed sample that is not tested, will be tested by the Geosynthetic CQA Laboratory. These ten samples must conform to the specifications. If any of these samples fail, every roll of geocomposite on site from this lot and every subsequently delivered roll that is from the same lot must be tested by the Geosynthetic CQA Laboratory for conformance to the specifications. This additional conformance testing will be at the expense of the Manufacturer.

- The Geosynthetics CQA Consultant will document actions taken in conjunction with conformance test failures.

9.6 Handling and Placement

The Installer will handle all geocomposites in such a manner as to ensure the geocomposites are not damaged in any way, and the following will be complied with.

- On sideslopes, the geocomposites will be secured in the anchor trench and then rolled down the slope in such a manner as to continually keep the geocomposite sheet in tension. If necessary, the geocomposite will be positioned by hand after being unrolled to minimize wrinkles.

- In the presence of wind, all geocomposites will be weighted with sandbags or the equivalent. Such sandbags will be installed during placement and will remain until replaced with cover material.

- Geocomposites will only be cut using scissors or a hook blade utility knife.

- The Installer will take any necessary precautions to prevent damage to underlying layers during placement of the geocomposite.

- During placement of geocomposites, care will be taken not to entrap in the geocomposite dirt or excessive dust that could cause clogging of the drainage system, and/or stones that could damage the adjacent geomembrane. If dirt or excessive dust is entrapped in the geocomposite, then it should be cleaned prior to placement of the next material on top of it. In this regard, care should be taken with the handling or sandbags, to prevent rupture or damage of the sandbag.

- Care should be taken not to leave tools in the geocomposite.

The Geosynthetic CQA Consultant will note noncompliance and report it to the Project Manager.

9.7 Seams and Overlaps

Adjacent geocomposites will be joined according to Drawings and specifications. As a minimum, the following requirements will be met.

- Adjacent rolls will be overlapped by at least 4 in. (100 mm).

- Each component of the geocomposite will be secured or seamed to the like component at overlaps.

- The geonet composite overlaps will be secured by tying.

- Tying will be achieved by nylon cable ties, plastic fasteners, or polymer braid. Tying devices will be white or yellow for easy observation. Metallic devices are not allowed.

- Tying will be every 5 ft (1.5 m) along the slope, every 2 ft (0.6 m) across the slope, and every 6 in. (150 mm) in the anchor trench and at tie-ins with previously deployed material from a former project. Tying will be every 10 ft (3 m) on horizontal surface.

- The bottom layers of geotextile will be overlapped.

- The top layers of geotextile will be continuously sewn as described in Section 7.7 of the CQA Plan. The geotextiles may be heat-bonded in lieu of sewing at the discretion of the DBE.

The Geosynthetic CQA Consultant will note noncompliance and report it to the Project Manager.

9.8 Repair

Any holes or tears in the geocomposite will be repaired by placing a patch extending 2 ft (0.6 m) beyond edges of the hole or tear. The patch will be secured by tying approved tying devices every 6 in. (150 mm) through the bottom geotextile and the geonet of the patch, and through the top geotextile and geonet components of the geocomposite needing repair. The top geotextile component of the patch will be heat sealed to the top geotextile of the geocomposite needing repair. If the hole or tear width across the roll is more than 50 percent of the width of the roll, the damaged area will be cut out and the two portions of the geocomposite will be joined in accordance with Section 9.7.

The Geosynthetic CQA Consultant will observe each repair, note any non-compliance with the above requirements and report them to the Project Manager.

9.9 Placement of Soil Materials

The Installer will place all soil materials located on top of a geonet composite in such a manner as to ensure:

- the geonet composite and underlying geosynthetic materials are not damaged;
- minimal slippage of the geonet composite on underlying layers occurs; and
- no excess tensile stresses occur in the geocomposite.

Unless otherwise specified by the DBE, a minimum thickness of 1 ft (0.3 m) of soil is specified between a low ground-pressure dozer and the geocomposite.

Noncompliance will be noted by the Geosynthetic CQA Consultant and reported to the Project Manager.

If portions of the geocomposite are exposed, the Geosynthetic CQA Consultant will periodically place marks on the geocomposite and the underlying geomembrane and measure the elongation of the geonet during the placement of soil.

10.0 GEOSYNTHETIC CLAY LINER (GCL)

10.1 Design Familiarization

A copy of the Drawings and specifications prepared by the DBE will be given to the Geosynthetics CQA Consultant. The Geosynthetics CQA Consultant will review these and verify that they are conceptually consistent with the state of practice, and are clear and complete. This review is not considered as a peer review of the design. Installation will be in accordance with the GCL Manufacturer's requirements and recommendations.

10.2 Manufacturing Quality Control

The GCL Manufacturer shall provide the Project Manager with a written certification signed by a responsible party that the materials actually delivered have "minimum average roll values" that meet or exceed all property values guaranteed for that type of GCL.

The Geosynthetics CQA Consultant will examine the manufacturer certifications to verify that the property values listed on the certifications meet or exceed those specified for the particular type of GCL. Deviations will be reported to the Project Manager.

10.3 Labeling

The GCL Manufacturer will identify all rolls of GCL with the following:

- manufacturer's name;
- product identification;
- lot number;
- roll number; and
- roll dimensions.

Additionally, if any special handling of the GCL is required, it will be so marked on the top surface of the GCL, e.g., "This Side Up".

The Geosynthetics CQA Consultant will examine rolls upon delivery and any deviation from the above requirements will be reported to the Project Manager.

10.4 Shipment and Storage

During shipment and storage, the GCL will be protected from ultraviolet light exposure, precipitation or other inundation, mud, dirt, dust, puncture, cutting or any other damaging or deleterious conditions. To that effect, GCL rolls will be shipped and stored in relatively opaque and watertight wrappings.

GCLs will not be exposed to precipitation prior to being installed. Wrappings protecting GCL rolls will be removed less than one hour prior to unrolling the GCL. After the wrapping has been removed, the GCLs will not be exposed to sunlight for more than 30 days, unless otherwise specified and guaranteed by the GCL Manufacturer.

The Geosynthetics CQA Consultant will observe rolls upon delivery at the site and any deviation from the above requirements will be reported to the Project Manager. Damaged rolls will be rejected and replaced at no cost to the Owner.

10.5 Conformance Testing

10.5.1 Tests

The GCL samples shall be removed at site or at the plant and forwarded to the Geosynthetics CQA Laboratory for testing to ensure conformance to both the Specifications and the Manufacturers list of guaranteed properties.

As a minimum, the following tests will be performed on GCLs:

- bentonite mass per unit area;
- flux;
- swell index; and
- weight of upper and lower geosynthetic components.

The conformance tests will be performed in accordance with the test methods specified in the specifications. Other conformance tests may be required by the specifications. Testing frequency for the GCLs is presented in Table 6.

10.5.2 Sampling Procedures

Samples of the GCL may be taken at the plant or at the site. Samples will be taken across the entire width of the roll and will not include the first 3 ft (1 m). Unless otherwise specified, samples will be 3-ft (1-m) long by the roll width. The machine direction shall be marked on the samples with an arrow for lab use.

10.5.3 Test Results

The Geosynthetics CQA Consultant will examine the results from laboratory conformance testing and will report non-conformance to the Project Manager.

10.5.4 Conformance Test Failure

The following procedure will apply whenever a sample fails a conformance test that is conducted by the Geosynthetics CQA Laboratory.

- The Installer will replace the roll of GCL that is in nonconformance with the specifications with a roll that meets specifications.

- The Installer will remove conformance samples for testing by the Geosynthetics CQA Laboratory from the closest numerical roll on both sides of the failed roll. These two samples must conform to the specifications. If either of these samples fail, the five numerically closest untested rolls on both sides of the failed sample will be tested by the Geosynthetics CQA Laboratory. These ten samples must conform to the specifications. If any of these samples fail, every roll of GCL on site and every subsequently delivered roll that is from the same supplier must be tested by the Geosynthetics CQA Laboratory for conformance to the specifications. This additional conformance testing will be at the expense of the Installer.

The Geosynthetics CQA Consultant will document actions taken in conjunction with conformance test failures.

10.6 Handling and Placement

The Installer will handle all GCLs in such a manner as to ensure they are not damaged in any way, and the following will be complied with:

- on slopes, the GCLs will be securely anchored in the anchor trench and then rolled down the slope in such a manner as to continually keep the GCL panel in tension;

- in the presence of wind, all GCLs will be weighted with sandbags or the equivalent; sandbags installed during placement are to remain in place until replaced with protective cover soils;

- GCLs will be kept continually under tension to minimize the presence of wrinkles in the GCL;

- GCLs will be cut using an approved GCL cutter only; if in place, then special care must be taken to protect other materials from damage which could be caused by the cutting of the GCLs;

- the Installer will take any necessary precautions to prevent damage to the underlying layers during placement of the GCL;

- during placement of GCLs, care will be taken not to entrap in the GCL stones, excessive dust, or moisture that could damage the geomembrane, generate clogging of drains or filters, or hamper subsequent seaming;

- a visual examination of the GCL will be carried out over the entire surface, after installation, to ensure that no potentially harmful foreign objects, such as needles, are present; and

- if white colored GCL is used, then precautions will be taken against "snow blindness" of personnel.

- no equipment shall be allowed to move on top of GCL.

The Geosynthetics CQA Consultant will note any noncompliance and report it to the Project Manager.

10.7 Seams and Overlaps

All GCL upper geotextiles will be continuously sewn (i.e., spot sewing and thermal bonding are not allowed) at the discretion of the Geosynthetics CQA Consultant, depending on the roll end details. GCLs will be overlapped at a minimum of 24 in. (600 mm) or greater based on the GCL Manufacturer's recommendations. No horizontal seams will be allowed on side slopes steeper than 10 percent (i.e. seams will be along, not across, slopes steeper than 10H:1V), except as part of a patch.

The Installer will pay particular attention at seams to ensure that no soil material could be inadvertently inserted beneath the GCL.

Sewing, where performed, will be done using polymeric thread.

10.8 Repair

Holes or tears in the GCL will be repaired as follows.

- On slopes steeper than 10 percent, a patch made from the same GCL will be seamed into place no closer than 1 in. (25 mm) from any edge. Should any tear exceed 10 percent of the width of the roll, that roll will be removed from the slope and replaced.

- On slopes flatter than 10 percent, a patch made from the same GCL will be placed with a minimum of 2-ft (0.6-m) overlap in all directions.

- Care will be taken to remove soil or other material that may have penetrated the torn GCL.

The Geosynthetics CQA Consultant will observe the repair of GCLs, note any non-compliance with the above requirements, and report them to the Project Manager.

11.0 PIPE AND FITTINGS

11.1 Material Requirements

11.1.1 Polyethylene Compound

Black PE materials used for the manufacture of polyethylene pipe, tube and fittings shall be PE 4710 high density polyethylene meeting ASTM D3350 cell classification 445574C and shall be listed in the name of the pipe and fitting Manufacturer in PPI (Plastics Pipe Institute) TR-4 with a standard grade HDB rating of 1600 psi at 73°F. The pipe compound will be protected against ultraviolet light and the resin compound will contain no additives, adulterants, extenders, or reworked compounds, but will contain an effective antioxidant.

The resin compound will have a minimum resistance of 125 hours when tested for environmental stress crack in accordance with requirements of ASTM-D1693.

Compliance with the above requirements must be certified by the pipe supplier.

11.1.2 Pipe and Fittings

The pipe and fittings shall have a cell classification of PE 445574C under ASTM D3350 and meet or exceed all the requirements of ASTM D3350. Pipes will be of uniform color and thickness.

The pipe and fittings Manufacturer shall be listed in the name of the pipe in PPI (Plastics Pipe Institute) TR-4 with a standard grade HDB rating of 1600 psi at 73°F. Pipes will be of uniform color and thickness. Each length of pipe will be suitably marked as to its pressure rating for water at 73.4°F (23°C). Pipe will have a Standard Dimension Ratio (SDR) stated on the Drawings.

All pipe fabrications and perforations shall be performed in the factory. Interior beads of all HDPE pipes upon completion of fusion welding shall be removed to achieve a smooth interior surface at the weld location.

Compliance with the above requirements must be certified by the pipe supplier.

11.1.3 Joints

Lengths of pipe will be assembled into suitable installation lengths by the butt-fusion process. Butt-fusion means the butt-joining of the pipe by softening the aligned faces of the pipe ends in a suitable apparatus and pressing them together under controlled pressure. This process will be applied by personnel experienced with the process. Certification will be provided that the person performing this work is qualified by experience and instruction in the procedure. All pipe so joined will be made from the same class and type of raw material made by the same raw material supplier.

Installation lengths will be joined by the use of stub ends. All stub ends for attachment to the polyethylene pipe will be made from the same type and grade of polyethylene, from the same raw material supplier as the pipe, and will be butt-fused to the pipe ends.

The polyethylene stub ends at joints will be backed up by nylon-coated steel flanges conforming to ANSI B16.1 and shaped as necessary to suit the outside dimensions of the pipe. The stub ends will be connected with corrosion resisting bolts and nuts of Type 316 Stainless Steel as specified in ASTM A 726 and ASTM A 307. Flat gaskets of 1/8 in. (3-mm) black reinforced rubber conforming to ANSI B16.21 will be installed between the opposing ends of the stub ends.

In no case will threaded male or female adapters of any plastic material be used for adapting polyethylene pipe to systems, fitting or auxiliary equipment of other materials, or for joining the installation lengths to each other.

Termination to pipes, valves, or fittings made of other material will be by the flanged joints outlined above. The pipe adjacent to these joints and to the joints themselves must be rigidly supported for a distance of one pipe diameter beyond the flange.

The internal beads at joints in the leachate collection pipes and risers should be trimmed to facilitate pipe inspection and cleaning.

11.2 Manufacturer

Prior to the installation of PE pipe, the PE Pipe Manufacturer will provide to the Geosynthetic CQA Consultant:

- a properties' sheet including, at a minimum, all specified properties, measured using test methods indicated in the specifications, or equivalent;

- a list of quantities and descriptions of materials other than the base polymer which comprise the pipe;

- the sampling procedure and results of testing; and

- a certification that property values given in the properties sheet are minimum values and are guaranteed by the PE Pipe Manufacturer.

The Geosynthetic CQA Consultant will verify that:

- the property values certified by the PE Pipe Manufacturer meet all of the specifications; and

- the measurements of properties by the PE Pipe Manufacturer are properly documented and that the test methods used are acceptable.

Prior to shipment, the PE Pipe Manufacturer will provide the Project Manager and the Geosynthetics CQA Consultant with a quality control certificate for each lot/batch of PE pipe provided. The quality control certificate will be signed by a responsible party employed by the PE Pipe Manufacturer, such as the Production Manager. The quality control certificate will include:

- lot/batch numbers and identification; and

- sampling procedures and results of quality control tests.

The Geosynthetics CQA Consultant will:

- verify that the quality control certificates have been provided at the specified frequency for all lots/batches of pipe, and that each certificate identifies the pipe lot/batch related to it; and

- review the quality control certificates and verify that the certified properties meet the specifications.

11.3 Conformance Testing

If deemed necessary by the DBE, the CQA Consultant will select one sample of each size of pipe, at least 3 ft (0.9 m) in length, and forward that sample to the CQA Laboratory for conformance testing. The samples will be taken from sections of pipe which are predetermined to have excess lengths. Samples failing to conform to the original specifications will result in rejection of all pipe of that size.

11.4 Handling and Laying

Care will be taken during transportation of the pipe such that it will not be cut, kinked, or otherwise damaged.

Ropes, fabric, or rubber-protected slings and straps will be used when handling pipes. Chains, cables, or hooks inserted into the pipe ends will not be used. Two slings spread apart will be used for lifting each length of pipe. Pipe or fittings will not be dropped onto rocky or unprepared ground.

Pipes will be stored on level ground, preferably turf or sand, free of sharp objects which could damage the pipe. Stacking of the polyethylene pipe will be limited to a height of four pipes to prevent excessive deformation of pipes under the anticipated temperature conditions. The pipe will be stored on wooden sleepers, spaced suitably and of such width as not to allow deformation of the pipe at the point of contact with the sleeper or between support. The pipes should be stored out of direct sunlight.

The handling of the joined pipe line will be in such a manner that the pipe is not damaged by dragging it over sharp and cutting objects. Slings for handling the pipeline will not be positioned at butt-fused joints. Sections of the pipes with deep cuts and gauges will be removed and the ends of the pipeline rejoined.

Butt-fusion of pipes and fittings will be performed in accordance with the pipe manufacturer's recommendations as to equipment and technique. Depending on site conditions, butt-fusion joining will be performed in or outside of the excavation.

Care will be exercised when lowering pipe into the trench to prevent damage or twisting of the pipe.

When pipes penetrate through the geomembrane, an effective seal will be established in accordance with the details shown on the Drawings.

11.5 Nondestructive Testing of Joints

All solid wall HDPE pipe with thermal butt-fusion type joints must be nondestructively tested. These pipe joints will be tested using the pressure test, as specified in ASTM F1417. Other nondestructive test methods may be used only when:

- the Installer can prove its effectiveness;

- the method is approved by the Pipe Manufacturer; and
- the method is approved by the DBE.

The DBE and the Geosynthetic CQA Consultant will verify the effectiveness and validity of the test method. Nondestructive testing of perforated or slotted pipe is not required.

The Geosynthetic CQA Consultant will report nonconformance of testing methods to the Project Manager.

12.0 SOILS

12.1 Introduction

This section discusses the CQA of the soil components for the construction of proposed liner system.

12.2 Soil Components

12.2.1 Overview

In this section of the Soils CQA Plan, a description of the soil components for the liner system and the procedures that will be used to construct each component is provided. Each of the soil components to be used must first be accepted for use by the DBE based on a review pre-qualification data submitted by the Contractor for each proposed source. The DBE will confirm acceptance of the soil components to the DBC, the Soils CQA Consultant, and the Owner. The DBC will be responsible for performing field and laboratory quality control testing in accordance with the Project Specifications during construction. The Soil CQA Consultant will be responsible for performing field quality assurance testing during construction.

12.2.2 Subgrade

The subgrade will be a 12 in. (300 mm) minimum thick layer of compacted soil above Class I waste in Cells 1 and 2. No thickness requirement for all other areas. The subgrade will consist of relatively homogenous natural soils that are free of debris, foreign objects, excess silt, rocks, plant materials, and organics. The subgrade will have no particles larger than 1 in. (25 mm). The in-place compacted subgrade shall be proof rolled before placemen to the liner.

The DBC will construct the subgrade to the lines, grades, and elevations shown on the Construction Drawings and in accordance with the Project Specifications. The Soils CQA Consultant will confirm that a qualified land surveyor has verified lines and grades prior to and after construction of the subgrade.

The subgrade over materials on top of Cells 1 and 2 will be placed and compacted in one lift. The final thickness of the subgrade will be within 0 to plus 0.2 ft (0.06 m). All subgrade areas will be compacted to the required minimum dry density within the range of moisture contents established by the pre-qualification tests, which will be confirmed based on the results of a test pad constructed in accordance the Project Specifications. The dry density and moisture content of the subgrade measured in the field will be used only as an indicator of the degree of

compaction. Acceptance of the subgrade will be based on the results of compaction requirement which is 95% standard proctor measured per ASTM D698 and visual observation of particle size.

12.2.3 Leachate Collection Gravel

The leachate collection gravel will be placed directly on top of a geocomposite as shown on the Construction Drawings. The gravel will consist of hard, strong, durable, non-carbonate particles that are free of foreign objects, organics, and other deleterious materials. The gravel will have a gradation meeting the requirements of the American Association of State Highway and Transportation Officials (AASHTO) No. 3 coarse aggregate, and will have a hydraulic conductivity of 1×10^{-1} cm/s (1×10^{-3} m/s) or greater. In addition, the gravel will have less than five percent loss of weight, when tested in accordance with ASTM D3042.

Placement of the gravel will be performed using low-ground pressure construction equipment in accordance with the Project Specifications. The equipment will operate only over previously placed gravel or protective cover in accordance with the Project Specifications. No construction equipment will be allowed to operate directly on the geotextile or geocomposite. Hand shovels used to place the gravel will be fitted with a fiberglass reinforced plastic blade. The gravel will be placed within plus or minus 0.1 ft (0.03 m).

12.2.4 Protective Cover

Sand shall be imported clean, excavated, natural sand, free from roots, sharp objects, or organic material, with a maximum size of 1/4 inch or 5% maximum by weight passing the No. 200 Sieve. The sand shall be classified as SW or SP by the Unified Soil Classification System. The sand drainage layer shall have a laboratory permeability not less than 1×10^{-3} cm/sec at 90% relative compaction (ASTM D 698). The material shall not be calcareous or of any other material that would interact with leachate. The sand shall be less than 10 percent loss of weight when tested in accordance with ASTM D3042 (Calcium Carbonate).

12.2.5 Structural Fill

Structural fill will be used to construct berms, cell base fill, and embankments and for other purposes as shown on the Construction Drawings. The structural fill will consist of relatively homogenous natural soils that are free of debris, foreign objects, organics and other deleterious materials. The soil will be classified as SP, GW, GM, GC, SW, SM, SC, ML, or CL in accordance with USCS and will have no particles larger than 4 in. (100 mm).

The structural fill will be placed and compacted in lifts to the lines and grades shown on the Construction Drawings. The compacted thickness of each lift shall be 12 in. (300 mm). Each lift

will be compacted to at least 95 percent of the standard Proctor maximum dry density (ASTM D698).

12.2.6 Low Permeability Soil

Low permeability soil (<1 x 10^{-6} cm/s) will be used in the perimeter anchor trench and as otherwise shown on the Drawings. The material will consist of relatively homogenous, natural soils that are free of debris, foreign objects, excess silt, rock, plant material, and organics. The soil will be classified as MH, ML, CL, or CH in accordance with uses and will have no particles larger than 1 in. (25 mm). The soil will have a plasiticity index between 10 and 40.

The soil will be placed and compacted in lifts to the lines and grades shown on the Drawings. The average compacted thickness lift will be no less than 6 in. (150 mm). Each lift will be compacted to at least 95 percent of the Standard Proctor maximum dry density.

12.3 Soils Testing

12.3.1 Test Methods

The test method that will be used to evaluate the suitability or conformance of soils materials will be carried out in accordance with the procedures cited in the current versions of the ASTM book(s) of standards. The test methods presented in Table 7 are those which will be used for this testing unless the test methods are updated or revised prior to construction. Any revisions to the test methods will be reviewed and approved by the DBE prior to their use.

12.3.2 Soils Testing Requirements

Nuclear moisture/density testing devices will be used to monitor the in-place moisture content and dry density of compacted soils. Other field moisture/density tests (i.e., sand cone and drive cylinder) may be used in cases of uncertainty, or as a means to verify the nuclear gage. Any conflict over the results will be resolved by the DBE.

The minimum frequencies for evaluation of soil materials are presented in Table 8. The actual testing frequency will be determined by the Soils CQA Consultant taking into consideration the natural variability of materials.

The minimum frequencies for soil construction quality assurance testing are presented in Table 9. The actual testing frequency will be determined by the Soils CQA Consultant taking into consideration the natural variability of materials.

Sampling locations will be selected by the Soils CQA Consultant. The location of routine in-place density tests will be determined using a non-biased sampling plan.

A special testing frequency will be used at the discretion of the DBE or the Soils CQA Consultant when visual observations of construction performance indicate a potential problem. Additional testing for suspected areas will be considered when:

- the rollers slip during rolling operations;
- the lift thickness is greater than specified;
- the fill material is at improper and/or variable moisture content;
- fewer than the specified number of roller passes are made;
- soil-clogged rollers are used to compact the material;
- the rollers may not have used optimum ballast;
- the soil materials differ substantially from those specified; or
- the degree of compaction is doubtful.

During construction, the frequency of testing may also be increased in the following situations:

- adverse weather conditions;
- breakdown of equipment;
- at the start and finish of grading;
- if the material fails to meet Project Specifications; or
- the work area is reduced.

12.4 Soils Construction Quality Assurance

12.4.1 Introduction

Soils CQA will be performed on all soil materials used in construction. General CQA procedures are outlined below.

12.4.2 Monitoring

12.4.2.1 General

The Soils CQA Consultant will monitor and document the construction of the soil components of the liner system. In general, monitoring soil construction will include:

- reviewing Contractor submittals and prequalification test results to verify that the proposed materials and construction methods conform to the Project Specifications;
- reviewing Contractor quality control test results;
- inspecting the subgrade and other soil layer surfaces for compliance with the Project Specifications before additional materials are placed;
- monitoring soil placement to verify that no deleterious material is present, no frozen material is placed, and no soil is placed on frozen ground;
- sampling and conformance testing of the materials used in construction and notifying the Contractor of non-complying results;
- documenting that the soil components are constructed using the specified equipment and procedures;
- documenting construction equipment used;
- documenting that each soil component is constructed to the lines and grades and to the specified thickness shown on the Drawings;
- monitoring each soil construction activity to verify that no damage is caused to underlying geosynthetics;

- recording field density and field moisture content measurements and test locations on test logs to verify uniform achievement of specified compaction parameters for each lift;

- monitoring the final thickness of each soil component;

- performing quality assurance testing (as required) to determine the acceptability of the work during construction; and

- monitoring the action of the compaction and heavy hauling equipment on the construction surface (i.e., penetration, pumping, cracking, etc.).

Additional CQA monitoring activities specific to each of the major soils components of the liner system are presented in the following sections.

12.4.2.2 Subgrade

Additional CQA monitoring activities specific to the Subgrade include:

- monitoring compaction activities to verify uniform coverage;

- recording field density and field moisture content measurements and test location on test logs to verify uniform achievement of specified compaction parameters;

- monitoring the compaction procedures to verify that the final surface is compacted, smooth, and uniform;

- obtaining undisturbed samples of the compacted soil for laboratory hydraulic conductivity testing;

- monitoring procedures used to protect the finished surface from desiccation or erosion;

- monitoring procedures used to restore the finished surface damaged by desiccation or erosion so that the geomembrane may be deployed over a smooth surface;

- monitoring construction to verify that completed sections are protected from damage; and

- documenting weather conditions and verifying that construction proceeds only during weather conditions that will not cause completed areas to be damaged, contribute to areas of non-compliance in construction, or impede CQA efforts.

12.4.2.3 Low Permeability Soil

Additional CQA monitoring activities specific to the low permeability soil used in anchor trenches include:

- backfill the anchor trench using low permeability soil when the geomembrane is at its most contracted state; preferably during the morning or an extended period of overcast skies. Take care when backfilling the trenches to prevent any damage to the geosynthetics.

- compact anchor trench in two lifts

- compact by wheel rolling with light, rubber-tired or other light compaction equipment.

- monitoring the compaction procedures to verify that the final surface is compacted, smooth, and uniform;

- monitoring procedures used to protect the finished surface from desiccation or erosion;

12.4.2.4 Protective Cover

Additional CQA monitoring activities specific to the protective cover include:

- monitoring the removal of objects that would damage the underlying geosynthetics; and

- monitoring placement operations to verify the equipment used does not damage the underlying geosynthetics.

12.4.2.5 Leachate Collection Gravel

Additional CQA monitoring activities specific to the leachate collection gravel include:

- monitoring placement to verify that contamination of the gravel with other soils is prevented; and

- monitoring placement and compaction procedures of the gravel to verify that the leachate collection and detection pipes are not damaged.

12.4.2.6 Perforations

Perforations that must be filled will include, but not limited to, the following:

- nuclear density test probe locations;
- sand cone or drive cylinder test locations;
- hydraulic conductivity sampling locations; and
- test pit locations

Hydraulic conductivity samples will be taken such that the sample tube is inserted into the Subgrade normal to the plane of the constructed surface.

Unless otherwise needed in the project specifications, or as directed by the DBE, all perforations of the subgrade layer by probe or sample tube shall be backfilled with a minimum of a ten percent bentonite powder mixture. The mixtures will be compacted in-place with a tamping rod, hammer, or hand tamper, depending on the size of the perforations.

12.4.3 Deficiencies

If a defect is discovered in the earthwork product, the Soils CQA Consultant will immediately evaluate the extent and nature of the defect. If the defect is indicated by an unsatisfactory test result, the Soils CQA Consultant will evaluate the extent of the deficient area by additional tests, observations, a review of records, or other means that the Soils CQA Consultant deems appropriate. If the defect is related to adverse site conditions, such as overly wet soils or surface desiccation, the Soils CQA Consultant will define the limits and nature of the defect.

12.4.3.1 Notification

After determining the extent and nature of a defect, the Soils CQA Consultant will notify the Project Manager (or Owner) and Contractor and schedule appropriate retests when the work deficiency is corrected.

12.4.3.2 Repairs and Retesting

At locations where the field testing indicates densities below the requirements of the Project Specifications, the failing area will be reworked. For the subgrade and low-permeability soil

where the field testing indicates the moisture content is below the requirements, the area will be scarified, moisture-conditioned, and recompacted. Alternatively, at the Soils CQA Consultant and Project Manager's (or Owner's) option, undisturbed samples of in-place material will be obtained and hydraulic conductivity and/or strength tests will be conducted as appropriate. Unless specified otherwise, the density requirements may be waived if the hydraulic conductivity and strength tests reveal acceptable results.

The Contractor will correct the deficiency to the satisfaction of the Soils CQA Consultant. If a project specification criterion cannot be met, or unusual weather conditions hinder work, then the Soils CQA Consultant will develop and present to the DBE and/or Project Manager suggested solutions for approval.

All retests recommended by the Soils CQA Consultant must verify that the defect has been corrected before any additional work is performed by the Contractor in the area of the deficiency. The Soils CQA Consultant will also verify that installation requirements are met and that submittals are provided.

12.5 Soils CQA Records

Records will be kept for soils-related activities will be completed by the Soils CQA Consultant. The information will be recorded as testing is performed in the field or as results are received from the laboratory. The records will be available for review on site, and copies will be issued as part of the Final Report. The relevant forms are briefly described below.

Field and Laboratory Compaction Test Log (ASTM D 698 Method A, B, C, D)

The results of field laboratory compaction tests will be recorded on a Field Laboratory Compaction Test Log. Separate forms will be used for each test method.

Field Sand Cone Density or Drive Cylinder Test Log

The results of the sand cone or drive cylinder in-situ density test on soils, if performed, will be recorded on a Field Sand Cone Density Test Log or a Field Drive Cylinder Density Test Log. The results will be used for comparison or calibration with nuclear density test results, at the discretion of the Soils CQA Consultant.

Summary of Sieve Analysis Test Data

This form will provide a summary of sieve analysis test results for soils.

Summary of Field Density Test

This form will provide a summary of field nuclear density, sand cone, and drive cylinder test results for soils.

Summary of Index Laboratory Test Data

This form will provide a summary of index test results performed in the field laboratory as required for soils.

13.0 SURVEYING

13.1 Introduction

Surveying of lines and grades will be conducted on an ongoing basis during construction of the soil layers, aggregate and placement of the geosynthetics. Certification surveying will be performed to provide documentation for record drawings, to verify quantities of soils and geosynthetics, to verify that the geometric requirements of the placement have been met, and to verify that the full thickness of the various soil and aggregate layers have been met. The DBT will coordinate his work with the work of the Surveyor. The Surveyor may assist the DBE in complying with the required grades. Surveying conducted at the site will be part of the CQA program.

13.2 Survey Control

At least three control monuments will be established for the current phase of the work by the Surveyor at locations convenient for daily tie-in. The vertical and horizontal controls for these control points will be established within normal land surveying standards. The Surveyor will communicate horizontal and vertical data for these control points to the Contractor, for the Contractor's used in laying out and providing ongoing geometric control of the work. The control monuments will be shown on all as-built record drawings.

13.3 Surveying Personnel

Surveying will be performed under the direct supervision of a qualified Land Surveyor, who may also be the Senior Surveyor on site. The survey crew will consist of the Senior Surveyor and as many Surveying Assistants as are required to satisfactorily undertake the work. Surveying personnel will be experienced in the provision of these services, including detailed, accurate documentation.

13.4 Precision and Accuracy

The survey instruments used for this work will be sufficiently precise and accurate to meet the needs of the project. Survey instruments shall be capable of reading to a precision of 0.01 ft (3.1 mm) and with a setting accuracy of 10 seconds. Calibration certificates for survey instruments will be submitted to the Project Manager prior to the initiation of surveying activities.

13.5 Lines and Grades

The following surfaces will be surveyed to determine the lines and grades achieved during construction on a 50-ft by 50-ft (15-m by 15-m) grid of the landfill. The survey shall include the following:

- original ground surface after clearing and grubbing;
- surface prepared for subgrade (including edges, bottom, and limits of pipe trenches and sump);
- surface prepared for protective cover (including edges, bottom, and limits of pipe trenches and sump);
- surface and limits of secondary and primary geomembrane, including locations of seam samples and major repairs;
- leachate collection and structures, including sumps, cleanouts, gravity transmission pipes and forced main pipes;
- access roads, access ramps, drainage channels, and culverts;
- surface and limits of geosynthetics;
- anchor trench;
- alignment and inverts of piping (both inside and outside the landfill);
- profiles and cross sections; and
- items requested by the Owner.

13.6 Frequency and Spacing

Surveying will be performed as soon as possible after completion of a given installation to facilitate progress and avoid delaying the next installation.

The Surveyor will submit to the Soils CQA Consultant, a minimum of 7 days before surveying activities begins, a Survey Plan describing the methods to be used to evaluate conformance of the

work to the geometric requirements of the contract documents. At a minimum, the Survey Plan will be prepared in sufficient detail to demonstrate that:

- all slopes have maximum inclinations as shown on the plans;
- required to have minimum slopes for drainage have at least the required slope; and
- geosynthetics anchor trenches have the required cross-section and are correctly located.

In general, the spacings and locations for survey points should be on a 50-ft by 50-ft (15-m by 15-m) grid and as follows:

- surfaces with slopes less than 10 percent will be surveyed on a square grid not wider than 50-ft (15-m);
- on slopes greater than 10 percent, a square grid not wider than 50-ft (15-m) will be used, but in any case, a line at the crest, midpoint, and toe of the slope will be taken;
- a line of survey points no farther than 50-ft (15-m) apart will be taken along any slope break (this will include the inside edge and outside edge of any terrace on a slope); and
- along linear features, survey sections or points should be at 50-ft (15-m).

The Soils CQA Consultant will review the Survey Plan and will promptly indicate approval or submit a request for additional detail to the Surveyor. Accuracy of the Survey Plan will be the sole responsibility of the Surveyor.

The Surveyor will survey the landfill as described in the Survey Plan. In addition, the Surveyor will provide assurance that uniform grades have been maintained between survey points.

Acceptable tolerance for the horizontal location of breaks in grade will be 0.5 ft (0.15 m).

Deviation from these tolerances will not be permitted unless approved in writing by the DBE.

The tolerances listed above notwithstanding, all permit conditions, regulatory requirements, and thickness requirements must be adhered to. It is the Contractor's sole responsibility to control

the work so that all of the geometric requirements of the project are met. The Contractor will notify the Project Manager 24 hours in advance of the time when any portion of the work will be completed and be ready for surveying by the Surveyor. The Surveyor will immediately notify the Contractor, the Project Manager, and the Soils CQA Consultant of any discrepancy found in the work. It is the Owner's sole prerogative to seek approval for work which is not within the tolerances listed above, but which, in the Owner's sole opinion, may nevertheless meet the intention of the project documents.

13.7 Documentation

Original field survey notes will be retained by the Senior Surveyor. A copy of these notes will be given to the Soils CQA Consultant at the end of each day or surveying task, whichever is shorter.

The Surveyor should produce record plans for the CQA Consultant as the job progresses. The results from the field surveys will be documented on a set of record plans. At the minimum, these plans will show the final elevations of the surfaces on a scale of 1 in. equals 100 ft (30 m), with contour intervals no greater than 2 ft, based on the 50-ft (15-m) grid pattern.

13.8 Certification

Survey as-built drawings will be certified by a registered Land Surveyor and submitted to the Project Manager.

TABLES

TABLE 1

GEOMEMBRANE CONFORMANCE TESTING REQUIREMENTS

Cell 6 and 7 Expansion
Steelfield Road Landfill

TEST NAME	TEST METHOD	MINIMUM FREQUENCY OF TESTING
Specific Gravity	ASTM D792	1 test per 100,000 ft^2 (9,290 m^2)
Thickness	ASTM D 5994	1 test per 100,000 ft^2 (9,290 m^2)
Tensile Strength at Yield	ASTM D638	1 test per 100,000 ft^2 (9,290 m^2)
Tensile Strength at Break	ASTM D638	1 test per 100,000 ft^2 (9,290 m^2)
Elongation at Yield	ASTM D638	1 test per 100,000 ft^2 (9,290 m^2)
Elongation at Break	ASTM D638	1 test per 100,000 ft^2 (9,290 m^2)
Tear Resistance	ASTM D1004, Die C	1 test per 100,000 ft^2 (9,290 m^2)
Puncture Resistance	ASTM D4833	1 test per 100,000 ft^2 (9,290 m^2)
Carbon Black Content	ASTM D1603	1 test per 100,000 ft^2 (9,290 m^2)
Carbon Dispersion	ASTM D5596	1 test per 100,000 ft^2 (9,290 m^2)

TABLE 2

GEOMEMBRANE SEAM TESTING REQUIREMENTS

Cell 6 and 7 Expansion
Steelfield Road Landfill

TEST NAME	TEST METHOD	MINIMUM FREQUENCY OF TESTING
Peel Adhesion	ASTM D4437 (as modified in NSF 54, Appendix A)[1, 3,4]	1 test every 500 ft (150 m)of seam length
Bonded Seam Strength	ASTM D4437 (as modified in NSF 54, Appendix A)[2, 3,4]	1 test every 500 ft (150 m) seam length
Vacuum Testing	---	100 percent of extrusion welded seams
Air Testing	---	100 percent of wedge welded seams

Notes:

1. For peel adhesion, seam separation will not extend more than 10 percent into the seam interface. Testing will be discontinued when the sample has visually yielded.
2. For shear tests, the sheet will yield before failure of the seam.
3. For either test, sample failure will be a Film Tear Bond (FTB) as outlined in NSF 54, Appendix A.
4. Minimum frequency shall be extended up to 1000 feet if electrical leak location survey method is used

TABLE 3

GEOTEXTILE CONFORMANCE TESTING REQUIREMENTS

Cell 6 and 7 Expansion
Steelfield Road Landfill

TEST NAME	TEST METHOD	MINIMUM FREQUENCY OF TESTING
Mass per Unit Area	ASTM D5261	1 test per 100,000 ft^2 (9,290 m^2)
Grab Strength	ASTM D4362	1 test per 100,000 ft^2 (9,290 m^2)
Mullen Burst	ASTM D3786	1 test per 100,000 ft^2 (9,290 m^2)
Puncture Resistance	ASTM D4833	1 test per 100,000 ft^2 (9,290 m^2)
Permeability	ASTM D4491	1 test per 100,000 ft^2 (9,290 m^2)
Apparent Opening Size	ASTM D4751	1 test per 100,000 ft^2 (9,290 m^2)
Trapezoidal Tear Strength	ASTM D4533	1 test per 100,000 ft^2 (9,290 m^2)
Thickness	ASTM D5199	1 test per 100,000 ft^2 (9,290 m^2)

Note: Testing will be carried out at a frequency of one per lot or at listed frequency, whichever is less.

TABLE 4

GEONET CONFORMANCE TESTING REQUIREMENTS

Cell 6 and 7 Expansion
Steelfield Road Landfill

TEST NAME	TEST METHOD	MINIMUM FREQUENCY OF TESTING
Density	ASTM D1505	1 test per 100,000 ft^2 (9,290 m^2)
Thickness	ASTM D5199	1 test per 100,000 ft^2 (9,290 m^2)
Carbon Black Content	ASTM D1603	1 test per 100,000 ft^2 (9,290 m^2)
Tensile Strength	ASTM D1682	1 test per 100,000 ft^2 (9,290 m^2)

Note: Testing will be carried out at a frequency of one per lot or at listed frequency, whichever is less.

TABLE 5

GEOCOMPOSITE CONFORMANCE TESTING REQUIREMENTS

Cell 6 and 7 Expansion
Steelfield Road Landfill

Properties	Qualifiers	Units	Values	Test Method	CQA
Geonet Component					
Resin Polymer Melt Index	maximum	g/10 min.	1.0	ASTM D1238	n/a
Polymer Density (geonet)	Minimum	g/cc	0.94	ASTM D 1505	1/100,000 SF
Carbon Black Content	range	%	2.0 - 3.0	ASTM D 1603	1/100,000 SF
Thickness	minimum	mils	300 +/-15%	ASTM D 5199	1/100,000 SF
Tensile Strength (MD and TD)	minimum	lb/in	100	ASTM D 5035	1/100,000 SF
Geotextile Component					
Polymer Composition	minimum	%	95 polyester or polypropylene	-	n/a
Mass Per Unit Area	minimum	oz/yd^2	6	ASTM D 3776	1/100,000 SF
Apparent Opening Size	maximum	US Sieve	#70	ASTM D 4751	1/100,000 SF
Flow Rate(4)	minimum	$gal/min./ft^2$	110	ASTM D 4491	n/a
Permittivity(4)	minimum	sec^{-1}	1.2	ASTM D 4491	1/500,000 SF
Grab Strength	minimum	lb	160	ASTM D 4632	1/100,000 SF
CBR Puncture Strength	minimum	lb	435	ASTM D 6241	1/100,000 SF
UV Resistance	minimum	% retained	70	ASTM D 4355 (after 300 hours)	n/a
Geocomposite					
Transmissivity@10,000 psf(2)	minimum	m^2/s	2.0×10^{-3}	ASTM D 4716	1/500,000 SF
Ply Adhesion	minimum	lb/in.	1	ASTM F 904(3)	1/100,000 SF

Notes:

(1) All values represent minimum average roll values (i.e., test results for samples collected from any roll in a lot should meet or exceed these values).

(2) Transmissivity @ 100hour: Measure transmissivity using water at 68 DegF with a normal compressive load of 10,000 psf and a hydraulic gradient of 0.02 (ASTM D 4716). Boundary conditions are provided in Section 2.3. Measurements shall be taken 100 hour after application of confining pressure if satisfies the requirements given in Part 2.3A.

(3) Minimum of values measured in machine and cross machine directions with 1 in. clamp on Constant Rate of Extension (CRE) machine.

(4) Either flow rate requirement or permittivity requirement shall be met.

TABLE 6

GCL CONFORMANCE TESTING REQUIREMENTS

Cell 6 and 7 Expansion
Steelfield Road Landfill

TEST NAME	TEST METHOD	MINIMUM FREQUENCY OF TESTING
Bentonite Mass per Unit Area	ASTM D5993	1 test per 100,000 ft^2 (9,290 m^2)
Flux	ASTM D5887	1 test per 100,000 ft^2 (9,290 m^2)
Swell Index	ASTM D5890	1 test per 100,000 ft^2 (9,290 m^2)
Weight of Upper and Lower Geosynthetic Components	ASTM D3776	1 test per 100,000 ft^2 (9,290 m^2)

Note: Testing will be carried out at a frequency of one per lot or at listed frequency, whichever is less.

TABLE 7

TEST PROCEDURES FOR THE EVALUATION OF SOILS

Cell 6 and 7 Expansion
Steelfield Road Landfill

TEST STANDARD	TO DETERMINE	TEST METHOD
Laboratory Test Procedures		
Classification	Classification of Soils	ASTM D2487
Moisture Content	Moisture Content	ASTM D2216
Standard Proctor	Moisture/Density Relationship of Soil (5.5-lb (2.49-kg) rammer and 12-in. (305-mm) drop)	ASTM D698
Atterberg Limits	Liquid Limit and Plastic Limit	ASTM D4318
Hydrometer Analysis	Particle Size Distribution of Fine Fraction of Soils	ASTM D422
Sieve Analysis	Particle Size Distribution of Coarse Fraction of Soils	ASTM D422
Sieve Analyses	Particle Size Distribution of Fine and Coarse Aggregates	ASTM C136
Permeability (Flexible Wall Permeameter)	Permeability of Soils	ASTM D5084
Permeability (Rigid Wall Permeameter)	Permeability of Aggregates	ASTM D2434
Calcium Carbonate Content	Percent of Calcium Carbonate in Soils	ASTM D4373
Field Test Procedures:		
Classification	Classification of Soils	ASTM D2488
Nuclear Densometer	In Situ: Soil Density / Moisture Content	ASTM D2922 / ASTM D3017

TABLE 8

MINIMUM SOILS TESTING FREQUENCIES FOR MATERIALS EVALUATION (CONFORMANCE) BEFORE CONSTRUCTION

Cell 6 and 7 Expansion
Steelfield Road Landfill

TEST	SUBGRADE	LEACHATE COLLECTION GRAVEL	PROTECTIVE COVER	STRUCTURAL FILL
Moisture Content	1 per 2,500 yd^3 (minimum 1 per source)	---	---	1 per 2,500 yd^3 (minimum 1 per source)
Sieve Analysis	1 per 6,500 yd^3 (minimum 1 per source)	1 per 6,500 yd3 (minimum1 per source)	1 per 6,500 yd^3 (minimum 1 per source)	1 per 6,500 yd^3 (minimum 1 per source)
Atterberg Limits	1 per 6,500 yd^3 (minimum 1 per source)	---	---	1 per 6,500 yd^3 (minimum 1 per source)
Standard Proctor	1 per 6,500 yd^3 (minimum 1 per source)	---	---	1 per 6,500 yd^3 (minimum 1 per source)
Hydraulic Conductivity	1 per 13,000 yd^3 (minimum 1 per source)	1 per 10,000 yd^3 (minimum 1 per source)	1 per 10,000 yd^3 (minimum 1 per source)	---

TABLE 9

MINIMUM SOILS TESTING FREQUENCIES DURING CONSTRUCTION

Cell 6 and 7 Expansion
Steelfield Road Landfill

TEST	LOW PERMEABILITY SOIL	SUBGRADE	STRUCTURAL FILL
In-Situ Moisture Content	1 per 100 lineal ft or 1 per acre per lift	1 per acre per lift	1 per acre per lift
In-Situ Density	1 per 100 lineal ft or 1 per acre per lift	1 per acre per lift	1 per acre per lift
Hydraulic Conductivity	1 per source discretionary	-----	-----

Note 1. Hydraulic conductivity of subgrade is not required since GCL is used.

FIELD CQA FORMS

Date _____

Page ____ of ____

TRIAL SEAM REPORT

PROJECT NAME	TECHNICIAN
PROJECT NO.	GEOMEMBRANE TYPE
INSTALLER	

Date/ Time	Ambient Temp.	Seamer Name	Machine Number	Extrusion Welds				Fusion Welds					Peel/Shear Values Lbs/Inch			Pass/ Fail	Comments
				Barrel Temp.		Preheat Temp.		NIP Roller Spacing	Wedge Temp.		Measured Speed Ft/Min.	Speed Setting					
				Set	Actual	Set	Actual		Set	Actual							

Date _____
Page ____ Of ____

GEOMEMBRANE REPAIRS

Repair Number	Date Repaired	Seam Panel or Repair	Location	Defect Type	Size of Repair	Date Tested

DEFECT TYPES

BO = fusion welder burn out
MD = manufacturer damaged
DS-# = fusion weld destruct
T = three panel intersection
FM = fishmouth
WS = welder restart
IO = insufficient overlap
D = installation damage
SI = soil surface irregularity
EXT = liner extension
WR = wrinkle
CR = crease
PT = pressure test cut
DX - #:extrusion weld destruct
VL = vacuum test leak
FS = failed seam length

Date _____

Page ____ of ____

REPORT OF IN-PLACE DENSITY TESTS

PROJECT NAME	TECHNICIAN
PROJECT NO.	ASTM Field Test Method Used
Compaction Requirement	ASTM Laboratory Test Method Used

Test No.	Field Location of Test	Depth or Elevation	Lab Test Results			Field Test Results					Remarks
			Sample No.	Max. Dry Density (pcf)	Opt. Moisture (%)	Wet Density (pcf)	Dry Density (pcf)	Lab Moisture (%)	Compaction (%)	Pass/ Fail	

Date _____

Page ____ of ____

GEOMEMBRANE DESTRUCTIVE SAMPLE LOCATIONS

Destructive Sample Number	Date Sampled	Date Seamed	Seam Number	Location	Welder	Unit Number	Results		
							Third-Party Lab	Installers Lab	Comment

Sheet No. ___ of ___

CQA PANEL INSPECTION FORM

PROJECT NAME	PANEL NO.
PROJECT NO.	GEOMEMBRANE TYPE
CONTRACTOR	INSTALLER
DATE PLACED	SUBGRADE CONDITION
WEATHER CONDITIONS	
DEPLOYMENT METHOD	

SHOW PROJECT NORTH

Panel No.: _____

Date	Defect	Defect Description	Repair Date	Repair Type	Approved By

Panel Length	Technician Comments
Panel Width	
Panel Area	
Roll Number	

DESTRUCTIVE SEAM EVALUATION

PROJECT NAME	**TECHNICIAN**
PROJECT NO.	**GEOMEMBRANE TYPE**
INSTALLER	**DATE SAMPLED**
DESTRUCTIVE SAMPLE NO.	**SEAM NO.**
SEAM TYPE	**DATE TESTED**

PEEL ADHESION					BONDED SHEAR RESISTANCE				
Specimen Number	Load at Yield (lbs/in)	Failure Description	Pass/ Fail	Retest	Specimen Number	Load at Yield (lbs/in)	Failure Description	Pass /Fail	Retest
AVERAGE					AVERAGE				

Checked by: _____	Checked by: _____
Check One: ☐ Seam Qualified ☐ Seam Disqualified	Check One: ☐ Seam Qualified ☐ Seam Disqualified

Sheet No. ___ of ___

CQA SEAM INSPECTION FORM

PROJECT NAME	SEAM NO.
PROJECT NO.	SEAM LENGTH
CONTRACTOR	COMMENTS
INSTALLER	
WELDING TECH.	
TYPE OF WELDER	WELDING UNIT NO.
DATE WELDED	TIME WELDED

QUALITY ASSURANCE SUMMARY

1. VACUUM BOX TEST

Zone	Length	Date Tested	QC Technicians Initials	Number of Defects Found	Date Repairs Made	Date Retested	Date Inspected and Approved

2. AIR PRESSURE TESTING

Start	Start Pressure (psi)	End Time	End Pressure (psi)	Zone	Length (feet)	Pressure Loss (psi)	Approved (Y/N)	If No, Corrective Action	Approved By

LEGEND

	Zone 1 Air Pressure Zone
	Repair Needed
	Repair Completed
	Repair Tested
	Repair Approved
	Destructive Sample
	Cap Strip Repair

3. SEAM REPAIRS (PATCHES)

Repair No.	Defect Type	Date/NDT Type	Approved By

Panel No. ____ Panel No. ____

(*Soils Engineer or Testing Service Letterhead)*

Subgrade Acceptability Certification No.

Project Name:	Regulatory Agency Project No. (if applicable):
Project Owner:	Subgrade Location:
HDR Project No.:	Date:
Owner's Project No. (if applicable)	Time:

I, _____, having examined the subgrade area referenced above, do certify that said area is properly prepared and is in suitable condition for construction to continue.

Comments:

_____

Designated Soils Lab Representative

Date _____

Page ____ of ____

GEOMEMBRANE PANEL PLACEMENT SUMMARY

Date	Time	Panel Number	Roll Number	Length (feet)	Width (feet)	Thickness (Mils)					Comments

Date _____

Page ____ of ____

GEOMEMBRANE SEAMING SUMMARY

Seam Number	Date Seamed	Panel Numbers	Seam Length (Ft)	Seamer Initials	Machine Number	Temp. Setting	Time		Ambient Temp.		Comments
							Start	Finish	Start	Finish	

APPENDIX G
GEOSYNTHETICS AND LCRS REQUIREMENTS

- **Anchor Trench Design;**
- **Geotextile Evaluation;**
- **Leachate Pipe Strength;**
- **Leachate Collection Pipe Capacity**
- **Bottom Liner Veneer Stability;**
- **Geocomposite Layer;**
- **Head Over Perforations in the Leachate Collection Header; and**
- **Pump design**

Geosynthetics and LCRS Calculations

Anchor Trench Design 2
Geotextile Evaluation 2
Leachate Pipe Strength 2
Leachate Collection Pipe Capacity 2
Leachate Collection Pipe Perforation Pattern 2
Veneer Slope Stability of Bottom Liner 2
Geocomposite Drainage Layer 3
Pump Design 3

Attachments

G-1 Anchor Trench Design

G-2 Geotextile Evaluation

G-3 Leachate Collection Pipe Strength Evaluation

G-4 Leachate Collection Pipe Capacity Evaluation

G-5 Head over Perforations in the Leachate Collection Header

G-6 Bottom Liner Veneer Stability

G-7 Geocomposite Drainage Layer

G-8 Pump Design

Anchor Trench Design

Anchor trench design calculations are based on the anchor trench configuration shown in Detail 2 of Sheet 01D-01. The calculations in Attachment G-1 indicates that the base liner will not pullout or tear during construction of protective cover layer or during waste placement.

Geotextile Evaluation

Five basic criteria (retention of adjacent particles, clogging resistance, permeability, survivability, and boundary conditions), were evaluated for the geotextile to demonstrate its sufficiency for the proposed design. Geotextile analysis calculations addressing these criteria including references are presented in Attachment G-2. Accordingly, LINQ GTF400E woven geotextile or similar/better is selected for the separation layer and GSE GEO608002 or similar/better non-woven geotextile is selected for the cushioning layer.

Leachate Pipe Strength

Leachate pipe strengths were confirmed based on anticipated maximum thickness of waste above the pipe. The calculations shown in Attachment G-3 meet the deflection and buckling requirements for the select pipe type.

Leachate Collection Pipe Capacity

The pipe capacity is confirmed based on HELP model peak daily leachate collection rates as shown in Attachment G-4.

Leachate Collection Pipe Perforation Pattern

As shown in Detail 2 of Sheet 01D-03, there are 4 perforations with 5/8-inch diameter per foot of pipe. Leachate perforation design is based on 1-inch of leachate head (i.e. 3.12-inch head over liner) over perforations. As shown in calculation in Attachment G-5, the design perforation pattern is sufficient to convey peak day leachate from HELP Scenario #1 into the sump.

Veneer Slope Stability of Bottom Liner

Veneer slope stability of the sand layer on slope areas was needed to be confirmed during the interim period until waste is placed within the landfill footprint.

The veneer stability of the protective cover was evaluated as shown in Attachment G-6. The calculation considered 12.1 in/hr rainfall intensity per NOAA for 25-year rain event. As shown in the calculation spreadsheet in Attachment G-6, safety factor requirements for veneer stability and drainage layer capacity are not met. Accordingly, the County may have to push the protective cover back onto the slopes after a rain event greater than 0.75 in/hr intensity until the waste elevation is above the elevation of the protective cover. As part of the proposed project,

protective cover for both Cells 6 and 7 footprints will be installed. A rain liner is proposed over Cell 7 and Cell 6B. The rain liner will prevent stormwater infiltrating through the protective cover and will help prevent veneer slope failure during rain events. In addition, the rain liner will prevent mixing of stormwater with leachate. The County will remove the rain liner gradually as fill progresses.

Geocomposite Drainage Layer

In the proposed design a geocomposite leachate collection and detection layer is proposed within the expansion footprint. The worst case maximum drainage length within the geocomposite in the proposed design is 171 feet. The calculation for permeability of the drainage layer is shown in Attachment G-7.

Pump Design

The following section discuses the designs performed to select cell sump pumps and vertical sump drainers at pump stations. Pump network design model outputs are given in Attachment G-8.

Cell Sump Wheeled Sump Drainers

The proposed Cells 6 and 7 are designed to pump out leachate into the pump station (PS #1) through sideslope risers. HELP model peak daily flow rates were used to design leachate collection and the detection sump pumps.

Cell 6 has a total area of 22.6 acres that includes subcells 6A and 6B with 11.7 acres and 10.9 acres, respectively. Initially, Cell 6B will be covered with a rain liner. Accordingly, the peak day leachate generation at Cell 6 is based on Sub-Cell 6A with an intermediate leachate generation rate and Sub-Cell 6B with 10-feet of waste.

Peak day leachate Generated from Sub-Cell 6A	= 891 cf/acre X 11.7 acres
	= 10,425 cf
Peak day leachate generation from Sub-Cell 6B	= 1,774 cf/acre X 10.9 acres
	= 19,337 cf
Peak day total leachate generated from Cell 6	= 29,762 cf
Required pump discharge rate	= 155 gpm
Design flow rate	= 176 gpm

Cell 7 has a total area of 11.3 acres. Peak day leachate generation at Cell 7 is based on 10-feet of waste within the entire Cell 7.

Peak day total leachate generated from Cell 7	= 1,774 cf/acre X 11.3 acres
	= 20,046 cf
Required pump discharge rate	= 104 gpm
Design pump discharge rate	= 116 gpm

Accordingly following pumps are selected for the cell sumps.

- Cell 6 Leachate Collection Sump Pump – EPG WSDPT 46-1
- Cell 7 Leachate Collection Sump Pump – EPG WSDPT 30-1
- Cell 6 and 7 Leachate Detection Sumps Pumps – EPG WSDPT 8-2

PS#1 Pump Design

The leachate collected from existing Cells 3 and 5 will gravity flow into PS#1 located south of the Cell 6 footprint. Leachate generated from Cell 4 will be collected at PS#3 and pumped into PS#1. The proposed Cells 6 and 7 are designed to pump out leachate into the pump station (PS #1) through sideslope risers. It is assumed that leachate generation from Cells 1 and 2 is 20 gpm. Note currently no leachate is generated from Cells 1 and 2. The leachate collected within pump station PS#1 will be pumped into proposed leachate storage tanks for offsite disposal.

The pump at PS #1 design is based on Cell 7 is filled with 10 feet of waste and all other areas are at intermediate leachate generation rates.

Peak day total leachate generated from Cell 7	= 1,774 cf/acre X 11.3 acres
	= 20,046 cf
Leachate generated from Cells 3, 4, 5, and 6	= 891 cf/acre X 31.5 acres
	= 28,256 cf
Total leachate collected at PS #1	= 48,302 cf
	= 251 gpm
Add leachate collected from Cells 1 and 2	= 20 gpm (assumed)

Required pump flow rate at PS #1	= 271 gpm
Design Flow Rate	= 481 gpm

The selected pump is EPG VSDPT 95-2. Note the design flow rate is based on open valve condition. The valve may be partially closed to operate pump around 350 gpm to avoid excessive pump startups. This will also bring the pump operating point toward the center of pump curve.

PS#2 Pump Design

Existing Cells 1 and 2 will gravity flow into PS#2. Currently no leachate is reported from these cells. Accordingly, no pump is designed for this pump station. If a need arises, Bay County will install a temporary pump or dedicated pump in the future.

PS# 3 Pump Design

The Cell 4 leachate collection pipe will gravity flow into PS#3. The Cell 4 leachate detection pipe will gravity flow into manhole MH#1 and ultimately discharge into PS#3 by gravity. The pump at PS#3 is designed based on Cell 4 at intermediate cover elevation from HELP Scenario 3.

Peak day total leachate generated from Cell 4	= 924 cf/acre X 5.71 acres
	= 5276 cf
Required flow rate at PS#3	= 27 gpm
Designed pump flow rate	= 35 gpm

The selected pump is EPG VSDPT 8-2.

Attachment G-1

Anchor Trench Design

Objective:

Determine the maximum anchor trench dimensions to allow geomembrane pullout before yield stress is exceeded. Evaluate cases for runout only and also anchor trench.

Reference:

1. Koerner, G.R. and D. Nareho. "Direct Shear Database of Geosynthetic-to-Geosynthetic and Geosynthetic-to-Soil Interfaces", GRI Report #30, June 2005
2. Koerner, Robert M., "Designing with Geosynthetics," Prentice Hall, Upper Saddle River, New Jersey, 4th edition, pp. 487-491.

Equations:

Without Trench (Runout only):

Figure 5.30 is from Reference 2

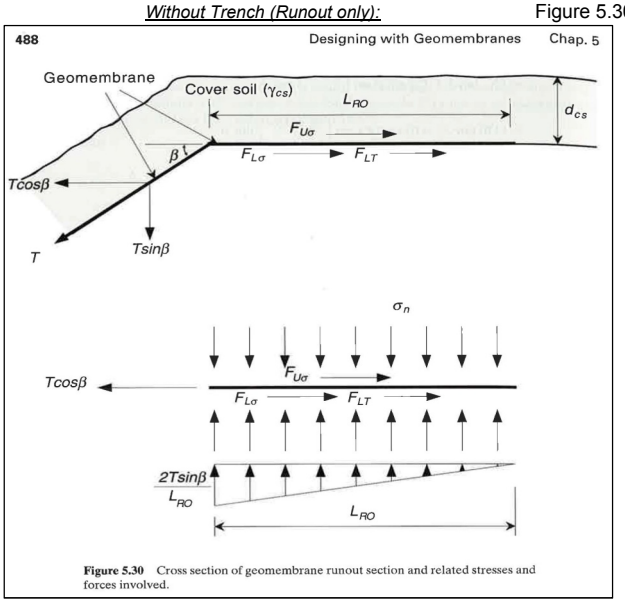


Figure 5.30 Cross section of geomembrane runout section and related stresses and forces involved.

$$L_{RO} = \frac{T_{ALLOW}(\cos\beta - \sin\beta \tan\delta_L)}{\sigma_N(\tan\delta_U + \tan\delta_L)}$$

Where:

T_{ALLOW} = allowable force in geomembrane stress = $\sigma_{ALLOW} t$
σ_{ALLOW} = allowable stress in geomembrane, and
t = thickness of geomembrane;
β = side slope angle;
σ_N = applied normal stress from cover soil;
δ_U = angle of shearing resistance between geomembrane and upper material;
δ_L = angle of shearing resistance between geomembrane and underlying material;
L_{RO} = length of geomembrane runout.

Calculations:

thickness of cover soil =	24	in	(assumed Max.)		12	in per foot
unit weight of cover soil =	120	lb/ft^3	(typical for silty clay)	FS =	1.2	desired FS
σ_N =	240	lb/ft^2		σ_{ULT} =	126	lb/in (from specs - yield)
δ_U =	0.0	B/c the geocomposite/soil moves along with the geomembrane as it deforms		σ_{ULT} =	2,211	psi
				σ_{ALLOW} =	1,842	psi
				t =	0.057	in
δ_L =	23.0	See Attachment A (Ref. 1). Conservatively neglect cohesion at interface. Non-woven side of GCL against textured HDPE.		T_{ALLOW} =	1,260	lb/ft
				β =	3	H:1V
				β =	18.43	degrees

[1]

L_{RO} = 10.1 ft

This represents largest runout w/o anchor trench.

With Runout Length & Trench: Ref. 2

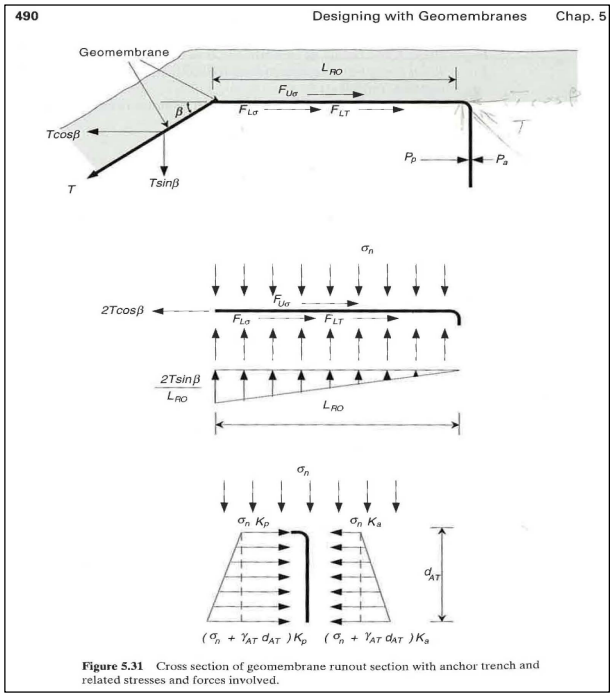


Figure 5.31 Cross section of geomembrane runout section with anchor trench and related stresses and forces involved.

Where:

T_{ALLOW} = allowable force in geomembrane stress = $\sigma_{ALLOW} t$

σ_{ALLOW} = allowable stress in geomembrane, and
t = thickness of geomembrane;
β = side slope angle;
$F_{U\sigma}$ = shear force above geomembrane due to cover soil
$F_{L\sigma}$ = shear force below geomembrane due to cover soil
F_{LT} = shear force below geomembrane due to vertical component of T_{ALLOW}
P_A = active each pressure against the backfill side of the anchor trench
P_P = passive earth pressure against the in-situ side of the anchor trench
γ_{AT} = unit weight of soil in anchor trench
d_{AT} = depth of anchor trench
σ_N = applied normal stress from cover soil;
K_A = coefficient of active earth pressure = $\tan^2(45-\varphi/2)$
K_P = coefficient of passive earth pressure = $\tan^2(45+\varphi/2)$
φ = angle of shearing resistance of respective soil

$T_{ALLOW}\cos\beta = F_{U\sigma} + F_{L\sigma} + F_{LT} - P_A + P_P$

$P_A = (0.5\gamma_{AT} d_{AT} + \sigma_N) K_A d_{AT}$

$P_P = (0.5\gamma_{AT} d_{AT} + \sigma_N) K_P d_{AT}$

$F_{U\sigma} = \sigma_N \tan\delta_U (L_{RO})$

$= 0.0\ L_{RO}$

$F_{L\sigma} = \sigma_N \tan\delta_L (L_{RO})$

$= 101.9\ L_{RO}$

$F_{LT} = T_{ALLOW} \sin\beta \tan\delta_L$

$= 169.1$ lb/ft

γ_{AT} =	120	lb/ft^3
σ_N =	240	lb/ft^2
φ =	30	°, assumed
K_A =	0.3333	
K_P =	3.0000	

TRY L_{RO} = 2 ft

P_A = (60 d_{AT} + 240) x (K_A x d_{AT})

P_P = (60 d_{AT} + 240) x (K_P x d_{AT})

$T_{ALLOW}\cos\beta$ = $F_{U\sigma}$ + $F_{L\sigma}$ + F_{LT} - P_A + P_P

$T_{ALLOW}\cos\beta$ = $F_{U\sigma}$ + $F_{L\sigma}$ + F_{LT} - (60.0 d_{AT} + 240) x (K_A x d_{AT}) + (60.0 d_{AT} + 240) x (K_P x d_{AT})

1,195 = $F_{U\sigma}$ + $F_{L\sigma}$ + F_{LT} - 20.0 d_{AT}^2 - 80 d_{AT} + 180.0 d_{AT}^2 + 720 d_{AT}

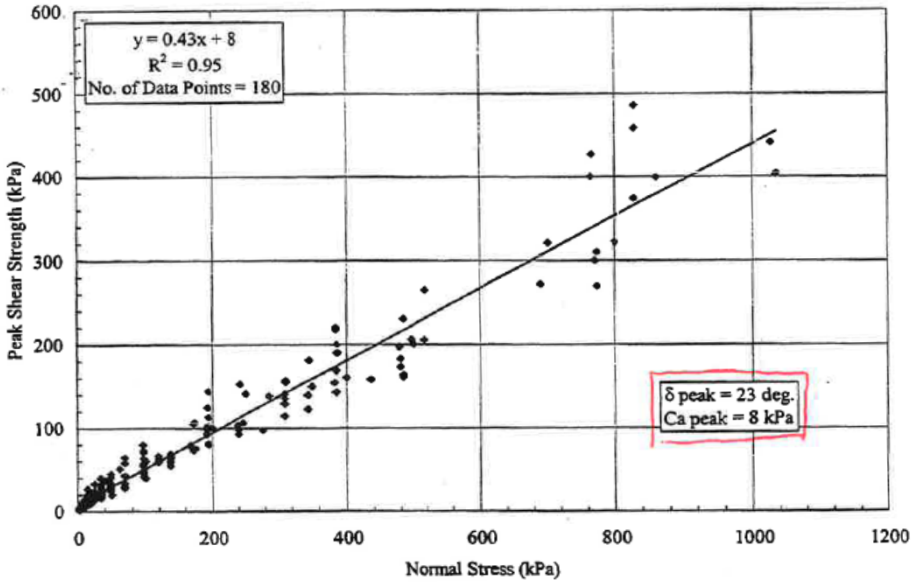
1,195 = 372.9 - 160.0 d_{AT}^2 + 640 d_{AT}

0 = -822.5 - 160.0 d_{AT}^2 + 640 d_{AT}

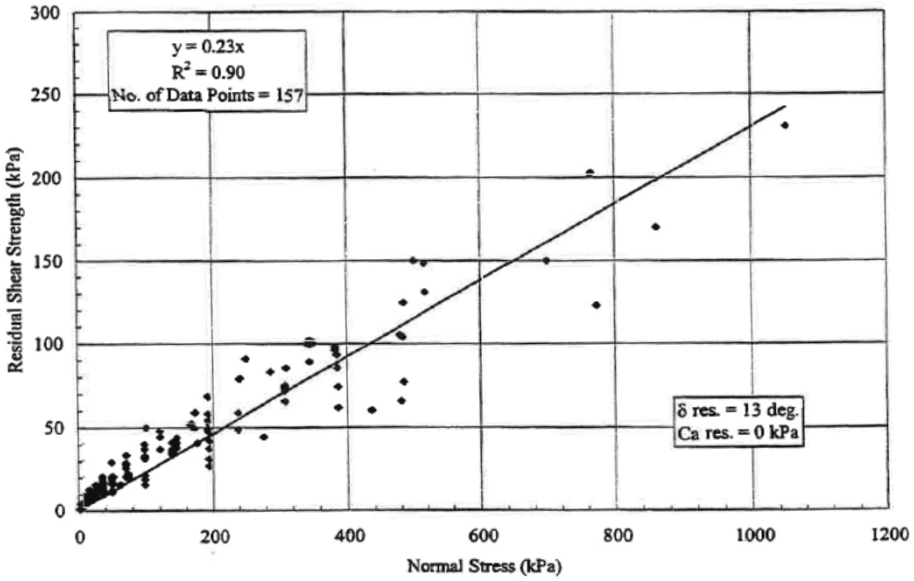
0 = 0 When d_{AT} = 1.023 ft (adjust dAT until right side of equation = 0)

d_{AT} =	**1.0**	**ft**
L_{RO} =	**2**	**ft**

Conservatively use d_{AT} = 2.0 ft and L_{RO} =3.0 ft. Since calcs are based on yield stress, extra FS is included.



Appendix Figure 11a - Peak Shear Strength; Textured HDPE against NW-NP Side of Fabric-Reinforced GCL.



Appendix Figure 11b - Residual Shear Strength; Textured HDPE against NW-NP Side of Fabric-Reinforced GCL.

Attachment G-2

Geotextile Evaluation

Job No.	10150537	No.	

HDR Engineering, Inc.

Project	SFRL Expansion	Computed	KP	Date	2/7/2019
Subject	Liner System Design	Checked	TY	Date	2/23/2019
Task	Geotextile Evaluation				

Problem
Evaluate geotextile for leachate collection system.
Evaluate that the geotextile meet the following filter criteria
- Retention criteria;
- Permeability criteria; and
- Porosity criteria.

In addition serviceability criteria should be met and the boundary requirements dictated by the adjacent drainage material should be met.

References
"Geotextile Filter Design", by Giroud J.P. and Bachus R.C., 1997.(Attachment 1)
"The Influence of Partial Clogging and Pressure on the Behavior of Geotextiles in Drainage Systems", by Palmeira E.M. and Gardoni M.G.
Designing with Geosynthetics, Koerner, R.M., 1998, Fourth Edition, Prentice Hall. (Attachment 3)

Given:

- The liner system components from bottom to top are: compacted subgrade - GCL - 60-mil HDPE scondary liner - Double-sided geocomposite - 60-mil HDPE primary liner - Double-sided geocomposite - 24" protective soil layer

- A representative particle size distribution of the protective cover soil overlying the geotextile is attached based on a sample collect at B-5 during 1986 investigation.
- The minimum hydraulic conductivity of the protective cover soil is 1×10^{-3} cm/s.
- The geocomposite drainage layer is Double-Sided Geocomposite with 6 oz/sy geotextile attached to the geonet. The geotextile properties are attached. (Attachment 5)
- Woven monofilament geotextile separation layer will be placed around the gravel bedding around the leachate collection trench. The selected fabric is LINQ GTF400E or equal as attached. (Attachment 6)

Note: The average overburden load applied on the geosynthetics is 6,000 psf. The calculations were performed based on 20,000 psf loading considering future expanions and is conservative.

Methods:
Retention Criterion (the geotextile's upper limit opening size)
The linear coefficient of uniformity (C'_u) of the protective cover soil.

Sample	d'_{100} (mm)	d'_0 (mm)	$\sqrt{d'_{100}/d'_0}$	$1< C'_u <3$?
B-1	0.707	0.15	2.17	Yes

Where d'_{100} and d'_0 were determined based on Giroud's retentions criterion by drawing a straight line to the central portion of the particle size distribution curve.

When $1<C'_u<3$, the Apparent Opening Size (AOS) should be less than $18xd_{50}/C'_u$ or <18x0.3/2.17 or 2.5 mm.

Therefore, the AOS of the geotextile filter must be less than 2.5 mm.
Example Products
LINQ GTF400E has an AOS of 0.212 mm.
GSE PermaNet has an AOS of 0.210 mm.

Permeability Criterion

$k_g > i_s k_s$

k_g	= geotextile hydraulic conductivity
i_s	= hydraulic gradient in soil next to the geotextile filter = 1.5 (Att. 1)
k_s	= soil hydraulic conductivity = 1×10^{-3} cm/s
k_g	> 1.5×10^{-3} cm/s

LINQ GTF400E has a permeability of 1×10^{-2} cm/s. FS = 6.7
GSE PermaNet has a permeability of 0.27 cm/s, FS = 178

HDR Engineering, Inc.

Project	SFRL Expansion	Computed	KP	Date	2/7/2019
Subject	Liner System Design	Checked	TY	Date	2/23/2019
Task	Geotextile Evaluation				

Porosity Criterion

The minimum requirement for porosity for nonwoven geotextile is 30% and for woven geotextile is 4% (Att. 1).

Check for nonwoven geotextile in the geocomposite:

Porosity of geotextile $= 1 - \frac{\mu}{t_g \rho_f}$

where:

μ = geotextile mass per unit area = 200 g/m^2

$t_{initial}$ = geotextile thickness = 1.78 mm (70 mils)

t_g = initial geotextile thickness divided by the reduction factor = 0.47 mm

$RF_{thickness}$ = thickness reduction factor = 3.81 (Att. 2)

ρ_f = density of polypropylene filaments = 910 kg/m^3

Therefore, the porosity is equal to 0.53 or 53%.

As expected this value is greater than 30% for nonwoven geotextile.

Check for woven geotextile around the leachate collection trench:

Porosity of geotextile $= 1 - \frac{\mu}{t_g \rho_f}$

Where:

μ = geotextile mass per unit area = 190 g/m^2

$t_{initial}$ = geotextile thickness = 0.381 mm (GTF 400E)

t_g = initial geotextile thickness divided by the reduction factor = 0.25 mm

$RF_{thickness}$ = thickness reduction factor for a similar product = 1.5 (assumed for woven geotextiel)

ρ_f = density of polypropylene filaments = 910 kg/m^3

Therefore, the porosity is equal to 0.18 or 18%.

As expected this value is greater than 4% for woven geotextile.

Serviceability Criteria

The geotextile filter must withstand stresses due to construction activities.

Burst Resistance

$$FS = \frac{60.6 \times P_{test}}{P' d_a}$$ (Att. 3)

Where:

P_{test} = Mullen Burst Strength = 450 psi for woven [note that nonwoven geotextile is not proposed above gravel]

P' = Stress on geotextile's surface = 20000 psf = 138 psi

d_a = The average stone diameter in millimeters = 3" (max.) = 76.2 mm

$$FS = \frac{60.6 \times 450}{138 \times 76.2} = 2.59$$

The above reduction factor includes a cumulative reduction factor of 1.5. Therefore, the calculated reduction factor is acceptable.

Grab Strength

$$T_{reqd} = P'(0.33 d_a)^2 (0.52)$$ (Att. 3)

Where:

P' = Stress on geotextile's surface = 20,000 psf = 138 psi

d_a = The average stone diameter = 1.5" (max. for protective cover)

d_a = The average stone diameter = 3" (max. for leachate collection pipe bedding)

T_{reqd} 17.58 lbs (for protective cover)

T_{reqd} 70.33 lbs (for leachate collection trench)

T_{allow} = 250 lbs in the weaker direction for woven and 170 lbs for non-woven

For non-woven

$$FS = \frac{T_{allow}}{T_{reqd}} = \quad 9.67$$

For woven

$$FS = \frac{T_{allow}}{T_{reqd}} = \quad 3.55$$

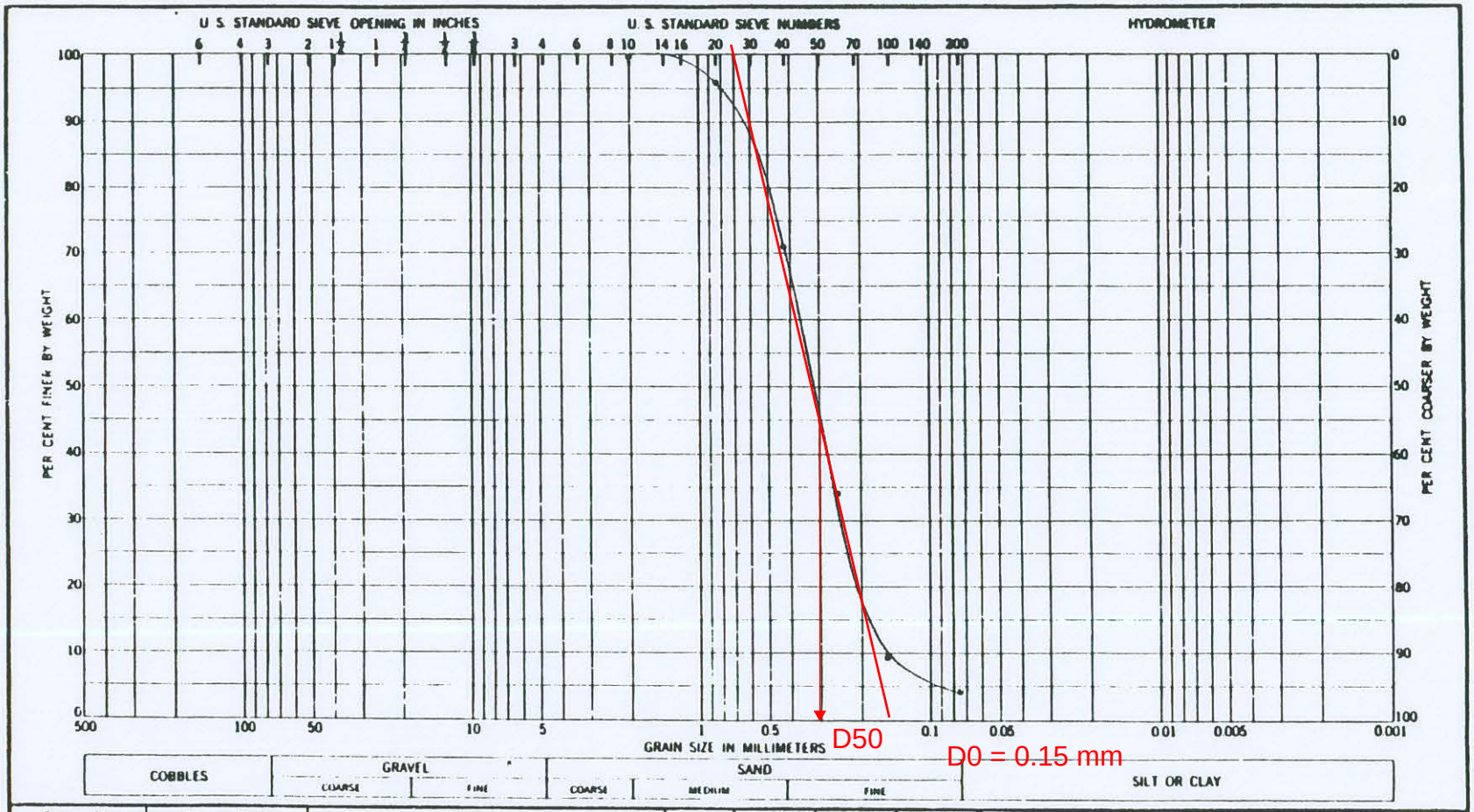
Puncture Resistance

From Fig. 2.32 in Att. 3, required puncture resistance at 957 kPa pressure(20000 psf) is 130 N for geocomposite geotextiles and 500 N for leachate collection trench geotextile. Note that these numbers are based on overall FS OF 2. The puncture strength of selected nonwoven geotextile is equal to 395 N and 530 N for woven geotextile and therefore OK.

Boundary Conditions

Analysis of the specified geotextile was performed with consideration of the actual expected boundary conditions. Conservative load data was used for the determination of ultimate material thickness and hydraulic conductivity. Appropriate reduction factors were accounted for in the analyses. Particle size and other data were based on the materials presented in the project specifications.

D100 = 0.707 mm



Sample No	Elev or Depth	Classification	Nat w %	LL	PL	PI
	25-26.5	Medium to Fine Sand				

GRADATION CURVES

Project Landfill Site off Steelfield Road Job No. F-86-042

Area

Boring No. B-5

Date 7-15-86

Attachment 1
Geotextile Evaluation

LANDFILL
DESIGN CONSTRUCTION

GEOTEXTILE
FILTER
DESIGN

Presented by

J.P. Giroud and R.C. Bachus
GeoSyntec Consultants
One Park Place
621 N.W. 53rd Street, Suite 650
Boca Raton, Florida 33487
Telephone: (561) 995-0900
Telefax: (561) 995-0995

FILTERS IN LANDFILLS

IMPORTANCE OF PROPER INSTALLATION OF GEOTEXTILE FILTER

Geotextile filter should be in good contact with soil.

A filter should prevent migration of particles not stop particles

SLOTTED PIPES

Fines will accumulate on the geotextile filter surrounding the pipe if:

- **aggregate is not clean;**
- **fines pass through the geotextile filter between soil and aggregate.**

The geotextile filter between sand and pipe is not the same as the geotextile filter between soil and gravel.

DESIGN FOR FILTRATION

FILTER CRITERIA FOR GEOTEXTILES

GEOTEXTILE FILTER

- Allow water to pass freely
- Prevent migration of soil particles

SIMILARITY BETWEEN GEOTEXTILE AND GRANULAR FILTER CRITERIA

OVERVIEW OF CRITERIA OF GRANULAR FILTERS

FILTER CRITERIA

- PERMEABILITY CRITERION
- RETENTION CRITERION

MEANING OF PERMEABILITY CRITERION FOR GRANULAR FILTERS

d_{15} (filter) > 4 d_{15} (soil)

d_5 (filter) > 4^2 d_5 (soil)

k proportional to d_5

k (filter) > 16 k (soil)

k (filter) > 10 k (soil)

$$i_g k_g = i_s k_s$$

$$i_g = 1$$

PERMEABILITY CRITERION

$$k_g > i_s k_s$$

$$k_g > FS\ i_s k_s$$

$i_s < 1$	ROADS, EMBANKMENTS
$i_s = 1.5$	LANDFILLS
$i_s = 1.5$	VERTICAL DRAINS
$i_s = 1.5\text{-}2.0$	DAM TOE DRAINS
$i_s = 3\text{-}10$	DAM CLAY CORES
$i_s > 10$	CLAY LINERS (LIQUID IMPOUNDMENT)

$$1 < FS < 10$$

FILTER CRITERIA

- RETENTION CRITERION
- PERMEABILITY CRITERION

MEANING OF RETENTION CRITERION FOR GRANULAR FILTERS

d_{15} (filter) < 4 d_{85} (soil) (1)

$\frac{d_{15}\text{ (filter)}}{4} < d_{85}$ (soil) (2)

filter OPENINGS < d_{85} (soil) (3)

A O S
Apparent Opening Size

GEOTEXTILE APPARENT OPENING SIZE (AOS) can be expressed in

mm	or U.S. Sieve No.
0.075	200
0.100	140
0.150	100
0.210	70
0.300	50

Example: Specify a maximum AOS

AOS (mm) < 0.150 mm

AOS (Sieve No.) > 100

RETENTION CRITERION

$O_{95} < I_R d_{85}$

Simplistic filter criteria may be dangerous.

FILTER CRITERIA

- RETENTION CRITERION
- PERMEABILITY CRITERION
- POROSITY CRITERION

PERCENT OPEN AREA

$$A = \frac{\text{Area of Openings}}{\text{Total Area}}$$

POROSITY

$$n = \frac{\text{Volume of Voids}}{\text{Total Volume}}$$

ROLE OF GEOTEXTILE THICKNESS

RETENTION CRITERION FOR FINE SOILS

RISK OF PIPING

Fine soils with little cohesion (low plasticity soils)

ADDITIONAL PARAMETERS
Soil coefficient of uniformity
Soil density

COEFFICIENT OF UNIFORMITY

$$C_U = \frac{d_{60}}{d_{10}}$$

GIROUD'S RETENTION CRITERION

Density index of the soil (Relative density)		Linear coefficient of uniformity of the soil $1 < C'_u < 3$	Linear coefficient of uniformity of the soil $C'_u > 3$
loose soil	$I_D < 35\%$	$O_{95} < C'_u \, d_{50}$	$O_{95} < \frac{9 \, d_{50}}{C'_u}$
medium dense soil	$35\% < I_D < 65\%$	$O_{95} < 1.5 \, C'_u \, d_{50}$	$O_{95} < \frac{13.5 \, d_{50}}{C'_u}$
dense soil	$I_D > 65\%$	$O_{95} < 2C'_u \, d_{50}$	$O_{95} < \frac{18 \, d_{50}}{C'_u}$

The retention criterion is independent of the amount truncated. Therefore, truncation is not required.

RISK OF PIPING WITH TYPICAL NONWOVENS

25% fines $20 < C_U < 50$

50% fines $10 < C_U < 20$

75% fines $5 < C_U < 10$

RISK OF CLOGGING

Gap graded soils

A filter should prevent migration of particles, not stop particles.

BIOLOGICAL CLOGGING

Geotextile filter criteria are as good as granular filter criteria.

FILTER PERFORMANCE EVALUATION

FILTRATION TESTS

SELECTION OF GEOTEXTILE FILTER FOR GEONET DRAIN

GEOTEXTILE FILTERS

- wovens (monofilament)
- nonwovens (needlepunched, heatbonded)

GEOTEXTILE SELECTION FOR FILTRATION

WOVEN OR NONWOVEN?

As typically indicated by filter criteria:

- MONOFILAMENT WOVEN - typically used with sands
- OTHER WOVENS - usually not suitable
- NONWOVENS - typically used with silts and clays

HEATBONDED OR NEEDLEPUNCHED?

Equivalent regarding filter criteria, but:

- HEATBONDED - recommended with drainage nets
- NEEDLEPUNCHED - often preferred with gravel drains

SURVIVABILITY

GEOTEXTILE FILTER SURVIVABILITY

	(1)	(2)	
GRAB	80	180	lbs
TEAR	25	50	lbs
PUNCTURE	25	80	lbs
BURST	130	290	psi

(1) Less severe conditions
(2) More severe conditions

DESIGN EXAMPLE

GEOTEXTILE FILTER FOR A LANDFILL LEACHATE COLLECTION SYSTEM

Prepared by J.P. Giroud
GeoSyntec Consultants

DEFINITION OF THE DESIGN EXAMPLE

- Type of Structure: Landfill leachate collection system
- Type of Application: Geotextile filter is between the protective cover soil and the drainage medium (sand or geosyntheticsnet)
- Geosynthetic Function: Filtration
- Geosynthetic Properties: Apparent opening size (AOS), permittivity, and porosity

GIVEN DATA

- The cross section of a landfill lining system with a leachate collection system is given in Figure 1.
- The particle size distribution of the protective cover soil overlying the geotextile is given in Figure 2.
- The hydraulic conductivity of the protective cover soil is:

$k_s = 1 \times 10^{-5}$ m/s (1×10^{-3} cm/sec)

- A polyester needlepunched nonwoven geotextile filter is considered. This geotextile has the following properties:
  - Mass per unit area: 0.34 kg/m^2 (10 oz/yd^2)
  - Permittivity (measured under a compressive stress equal to the field overburden stress):

    $\psi_g = 0.3\ s^{-1}$

  - Thickness (measured under a compressive stress equal to the field overburden stress):

    $t_g = 2$ mm

  - Apparent opening size (AOS):

    $O_{95} = 150\ \mu m$ (U.S. Sieve No. 100)

  - Grab strength: 1020 N (230 lbs)
  - Tear strength: 555 N (125 lbs)
  - Puncture strength: 555 N (125 lbs)
  - Burst strength: 2750 kPa (400 psi)

DESIGN

A geotextile filter should meet three geotextile filter criteria:

- retention criterion;
- permeability criterion; and
- porosity criterion.

In addition, survivability criteria should be met and boundary requirements dictated by the adjacent drainage material should be met.

Step 1. Retention Criterion

- Method Number 1

This method uses Giroud's retention criterion as follows:

- Trace a straight line as close as possible to the central portion of the particle size distribution curve of the soil (Figure 3).
- Read the values of d' and d'_{00} at the two extremities of this straight line.
- Calculate the linear coefficient of uniformity of the soil:

$$C'_u = \sqrt{d'_{100} / d'_0} \qquad \text{(Equation 1)}$$

- Use Giroud's retention criterion given in Table 1. To use this criterion, it is necessary to know the linear coefficient of uniformity of the soil (calculated as indicated above) and the density of the soil, which the designer can estimate based on data pertinent to the project.

- Example

- First, determine the linear coefficient of uniformity. According to Figure 3:

  $d'_{0} = 0.007$ mm $\qquad$ $d'_{100} = 17$ mm

  Hence:

  $$C'_u = \sqrt{17 / 0.007} = 49$$

- Then, use Giroud's retention criterion (Table 1).

  Using the linear coefficient of uniformity calculated above and considering that the protective cover soil in a landfill is dense (due to high overburden stress and assuming it has been properly compacted), Table 1 shows that the following criterion should be used:

  $$O_{95} < 18\ d_{50} / C'$$

  where: O_{95} = apparent opening size (AOS) of the filter; d_{50} = soil particle size such that 50% by weight of soil particles are smaller than d_{50}; and C' = linear coefficient of uniformity.

  With the value $C' = 49$ calculated in Step 1, the above equation becomes:

  $$O_{95} < 18\ d_{50} / 49$$

  According to Figure 2, $d_{50} = 0.47$ mm.

  Hence:

  $O_{95} < 0.17$ mm (U.S. Sieve No. 100)

  In other words, the apparent opening size (AOS) of the geotextile filter must be less than 0.17 mm (or the U.S. Sieve number used to express the geotextile filter AOS should be larger than 100). Many available nonwoven geotextiles meet this requirement.

- Method Number 2

If the retention criterion recommended by the Federal Highway Administration (FHWA) is used, the following should be done for Step 1:

- First calculate the coefficient of uniformity of the particle size distribution given in Figure 1:

  $C_u = d_{60} / d_{10}$ (Equation 2)

  $C_u = 1 \text{ mm} / 0.009 \text{ mm} = 111$

- Then use the FHWA retention criterion given in Table 2.
  For $C_u > 8$, the following criterion should be used:

  $O_{95} < d_{85}$ $(B = 1)$

  Hence:

  $O_{95} < 7$ mm (0.275 in.)

In other words, according to this design method, the apparent opening size (AOS) of the geotextile filter must be less than 7 mm (0.275 in.), which is very large.

This geotextile opening size is very large and it is legitimate to fear that the overlying protective cover soil would not be retained if a geotextile filter with such large openings were used. Obtaining excessively large filter openings is a common problem when designing filters for soils with a large coefficient of uniformity. In geotechnical engineering, it is standard practice to eliminate particles coarser than 4.75 mm (U.S. Sieve No. 4) when designing filters. This practice is intended to compensate for the fact that classical filter criteria for granular filters are not applicable to soils with a large coefficient of uniformity. Similarly, the FHWA recommends that only particles smaller than 4.75 mm (U.S. Sieve No. 4) be considered when the FHWA geotextile retention criterion is used for soils having a large value of the coefficient of uniformity,

C_u. If this were done with the particle size distribution curve shown in Figure 2, we would obtain the following new values for d_{100}, d_{85}, etc., as shown in Figure 5:

new $d_{100} = 4.75$ mm = actual d_{80}
new $d_{85} = 1.6$ mm = actual d_{68} (since $80\% \times 85\% = 68\%$)
new $d_{60} = 0.4$ mm = actual d_{48} (since $80\% \times 60\% = 48\%$)
new $d_{10} = 0.005$ mm = actual d_8 (since $80\% \times 10\% = 8\%$)

As a result:

new C_u = new d_{60}/new $d_{10} = 0.4 / 0.005 = 80$

According to Table 2, the FHWA criterion to use in this case is:

$O_{95} < d_{85}$ (using, of course, the new d_{85})

Hence:

$O_{95} < 1.6$ mm (U.S. Sieve No. 10)

In other words, according to this design method, the apparent opening size (AOS) of the geotextile filter must be less than 1.6 mm (or the U.S. Sieve number used to express the geotextile AOS should be no less than 10).

- Selected Method

The filter opening size value of 1.6 mm obtained with the second method, 1.6 mm, is very large and, in our judgment, may lead to soil piping. On the other hand, a filter with 1.6 mm openings is less likely to clog than a filter with 0.17 mm openings, as determined using the first method.

In the case of a filter used for a leachate collection system, clogging of the filter would only delay leachate collection, whereas piping would cause clogging of the

leachate collection drainage layer (here, a geonet), which would severely impair leachate collection.

Therefore, the filter opening size obtained with the first method, 0.17 mm, should be selected.

Step 2. Permeability Criterion

- Method

The criterion established by Giroud is:

$$k_g > i_s k_s \quad \text{(Equation 3)}$$

where: k_g = geotextile hydraulic conductivity; i_s = hydraulic gradient in soil next to the geotextile filter; and k_s = soil hydraulic conductivity.

According to Giroud [1988], typical values of hydraulic gradients are as follows:

- $i_s \leq 1$ for many cases of drainage under roads, embankments, soil layers on slopes, etc., when the main source of liquid is precipitation;
- $i_s = 1.5$ in the case of drainage trenches, vertical drains behind walls, and leachate collection layers in waste disposal landfills;
- $i_s = 1.5$ to 2 for toe drains in earth dams;
- $i_s = 3$ to 10 (or more) in dam clay cores, depending on the core thickness; and
- $i_s = 10$ (or more) in clay liners for liquid impoundments.

A factor of safety of 10 or more is recommended when lack of permeability of the filter could have catastrophic consequences, e.g., dams and soil layers on slopes. As a result, Equation 8 may range from $k_g > k_s$, when $i_s = 1$ and no safety factor is needed, to $k_g > 100\ k_s$ or more in the case if a very thin dam clay core.

Alternatively, the method recommended by the FHWA is as follows:

- For small gradients and stable soil:

$$k_g > k_s \quad \text{(Equation 4)}$$

- For high gradients and erodible soils:

$$k_g > 10\ k_s \quad \text{(Equation 5)}$$

The value of the soil hydraulic conductivity, k_g, to be used in Equations 3, 4, and 5 should be measured under a compressive stress equal to the one expected in the field. In many cases, the geotextile permittivity, ψ_g, is given. The geotextile hydraulic conductivity, k_g, can then be derived as follows:

$$k_g = \psi_g\ t_g \quad \text{(Equation 6)}$$

where: t_g = geotextile thickness under the compressive stress expected in the field.

- Example

The hydraulic conductivity of the considered geotextile is given by Equation 6, using the values of $\psi_g = 0.3\ s^{-1}$ and $t_g = 2 \times 10^{-3}$ m provided in the "Given Data" Section:

$$k_g = 0.3 \times 2 \times 10^{-3} = 6 \times 10^{-4}\ \text{m/s}$$

Then, Equation 3 can be used with $i_s = 1.5$, according to guidance provided after Equation 3, and $k_s = 1 \times 10^{-5}$ m/s provided in the "Given Data" Section:

$$k_g > 1.5 \times 10^{-5} \text{ m/s } (1.5 \times 10^{-3} \text{ cm/s})$$

No factor of safety is necessary since lack of permeability of the filter would not have catastrophic consequences.

The method recommended by the FHWA would give a slightly different result. Since the gradient is small and assuming that the soil is not erodible, Equation 4 applies, hence:

$$k > 1 \times 10^{-5} \text{ m/s } (1 \times 10^{-3} \text{ cm/s})$$

It appears that the considered geotextile filter, with its hydraulic conductivity of 6×10^{-4} m/s, satisfies the above requirements with a factor of safety of 40 (Giroud's criterion) or 40 (FHWA criterion).

Step 3. Porosity Criterion

- Method

To minimize the risk of clogging, the following criteria shall be met:

- Nonwoven geotextile: porosity > 30%
- Woven geotextile: percent open area > 4%

The porosity of a nonwoven geotextile can be calculated using the following equation:

$$n = 1 - \mu/(t_g \rho_f) \qquad \text{(Equation 7)}$$

where: n = geotextile porosity or planar porosity; μ = geotextile mass per unit area; t_g = geotextile thickness; and ρ_f = density of filaments. (Note: The value of n obtained using Equation 7 must be multiplied by 100 to express the porosity of a nonwoven as a percentage or to obtain the percent open area of a woven.)

- Example

In this project, a needlepunched nonwoven geotextile is considered. Most needlepunched nonwoven geotextiles have a porosity of approximately 90%. Therefore, it is expected that the porosity requirement of 30% will easily be met. This is verified below.

The porosity of the considered nonwoven geotextile under the project overburden stress can be calculated using Equation 7, knowing that the density of polyester is 1380 kg/m^3:

$$n = 1 - 0.34/(2 \times 10^{-3} \times 1380)$$

$$n = 0.88 = 88\%$$

As expected, this value is greater than the required 30%.

Step 4. Survivability Requirements

- Method

The geotextile filter must withstand stresses due to construction activities. Survivability requirements that must be met by geotextiles used in drainage applications are given in Table 3.

- Example

The geotextile will be installed over the drainage medium (geonet or granular material) and covered with the protective cover soil. The protective cover soil will be compacted. Therefore the values indicated in the "Class A" column of Table 3 should be selected. The geotextile defined in the "Given Data" Section at the beginning of this design example meets all the above requirements.

Step 5. Boundary Requirements

More and more synthetic drainage materials such as geonets are used. If the geotextile filter is thick, compressible, and compliant, it may partially penetrate into the channels of the synthetic drainage layer, thereby decreasing its hydraulic transmissivity. This effect is particularly marked with needlepunched nonwoven geotextiles in contact with geonets. It is therefore important to conduct hydraulic transmissivity tests of the synthetic drainage layer with the considered geotextile in contact with it.

CONCLUSIONS

The selected geotextile filter must meet the following design and survivability requirements which were determined in this design example.

- Design Requirements

- Apparent opening size:

$O_{95} <$ 0.17 mm (U.S. Sieve No. 100)

- Hydraulic conductivity:

$k_g > 1 \times 10^{-5}$ m/s (1×10^{-3} cm/sec)

- Porosity:

  - nonwovens: porosity > 30%
  - wovens: percent open area > 4%

- Survivability Requirements

  - Grab strength: 800 N (180 lbs)
  - Tear strength: 220 N (50 lbs)
  - Puncture strength: 360 N (80 lbs)
  - Burst strength: 2000 kPa (290 psi)

The geotextile filter considered in the "Given Data" Section at the beginning of this design example meets all the above requirements. In addition, hydraulic transmissivity tests should be conducted on a specimen including the considered synthetic drainage layer and geotextile filter, as well as the adjacent soil, to verify that the synthetic drainage layer has the required hydraulic transmissivity with these boundary conditions. The hydraulic transmissivity test must be conducted under a compressive stress at least equal to the expected field compressive stress.

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Giroud, J.P., "Review of Geotextile Filter Criteria", *Proceedings of the First Indian Geotextiles Conference*, Bombay, Dec 1988, pp. 1-6.

Table 1. *Giroud's Retention Criterion for Geotextile Filters.* [Giroud, 1982]

Density index of the soil (Relative density)		Linear coefficient of uniformity of the soil	
		$1 < C_u' < 3$	$C_u' > 3$
loose soil	$I_D < 35\%$	$O_{95} < C_u' \, d_{50}$	$O_{95} < (9/C_u') \, d_{50}$
medium dense soil	$35\% < I_D < 65\%$	$O_{95} < 1.5 \, C_u' \, d_{50}$	$O_{95} < (13.5/C_u') \, d_{50}$
dense soil	$I_D > 65\%$	$O_{95} < 2 \, C_u' \, d_{50}$	$O_{95} < (18/C_u') \, d_{50}$

Table 2. *FHWA Retention Criterion for Geotextile Filters.* [FHWA, 1985]

1. Less than 50% of the soil particles smaller than 75 micrometers (U.S. Sieve No. 200)

$O_{95} < B\, d_{85}$ (with B depending on C_u)

$C_u < 2$ $B = 1$

$2 < C_u < 4$ $B = 0.5\, C_u$

$4 < C_u < 8$ $B = 8/C_u$

$C_u > 8$ $B = 1$

2. More than 50% of the soil particles smaller than 75 micrometers (U.S. Sieve No. 200)

nonwovens: $O_{95} < 1.8\, d_{85}$

wovens: $O_{95} < d_{85}$

wovens and nonwovens: $O_{95} < 300$ microns (U.S. Sieve No. 50)

Table 3. *Geotextile Filter Survivability Requirements.* [FHWA, 1985]

Property	Class (A)	Class (B)	Test Method
Grab strength	800 N (180 lbs)	360 N (80 lbs)	ASTM D1682
Tear strength	220 N (50 lbs)	110 N (25 lbs)	ASTM D1117
Puncture strength	360 N (80 lbs)	110 N (25 lbs)	ASTM D3787
Burst strength	2000 kPa (290 psi)	900 kPa (130 psi)	ASTM D3786

(A) "Unprotected".

(B) "Protected", i.e., in trench, with rounded gravel; or in contact with concrete slab or geomembrane.

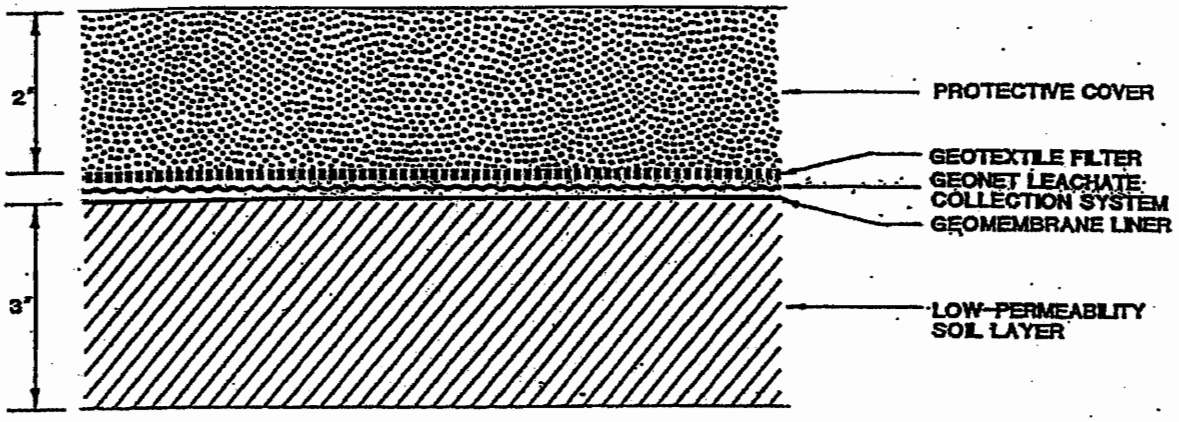

Figure 1. *Cross Section of a Landfill Lining System with a Leachate Collection Layer.*

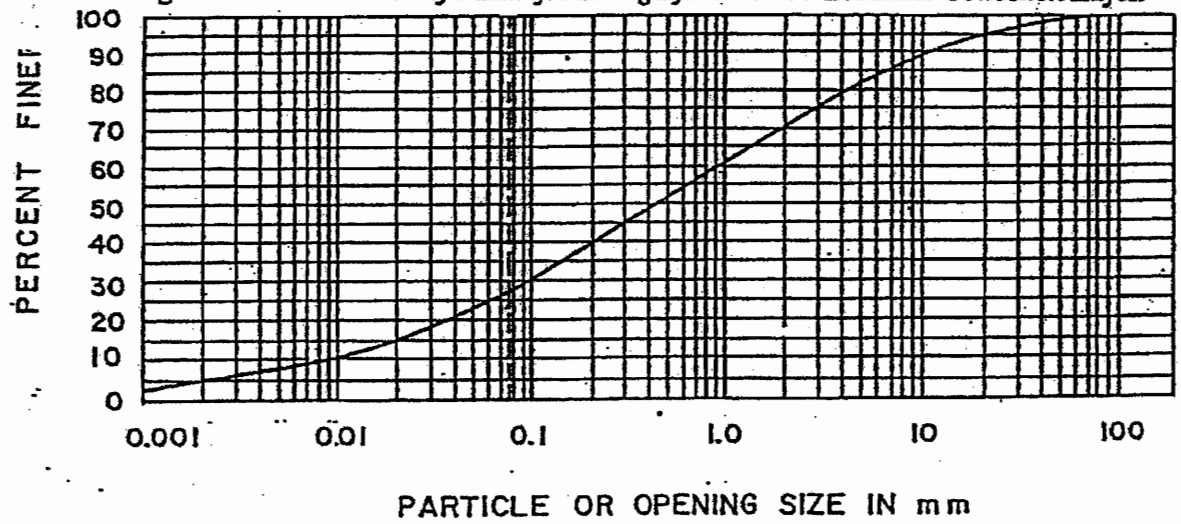

Figure 2. *Particle Size Distribution of the Protective Cover Soil.*

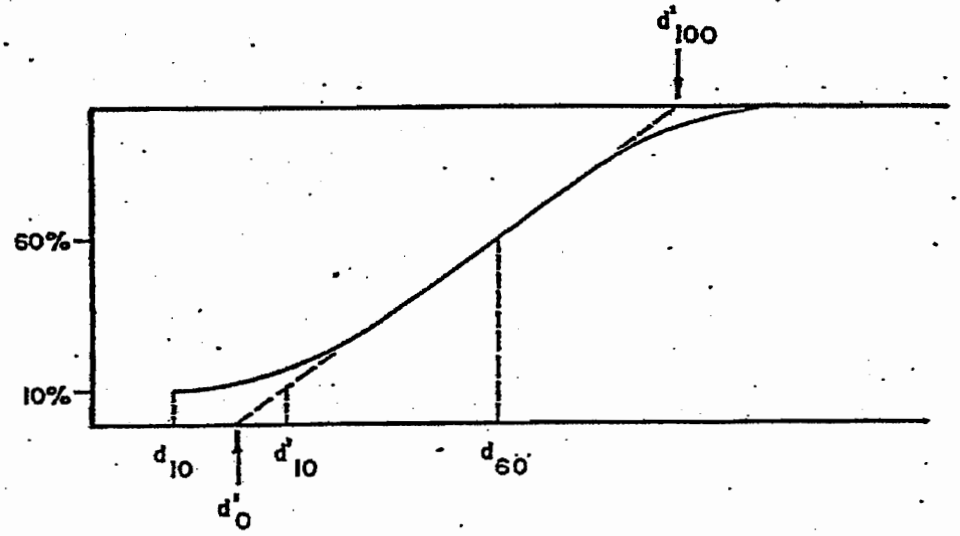

Figure 3. *Determination of the Linear Coefficient of Uniformity.*

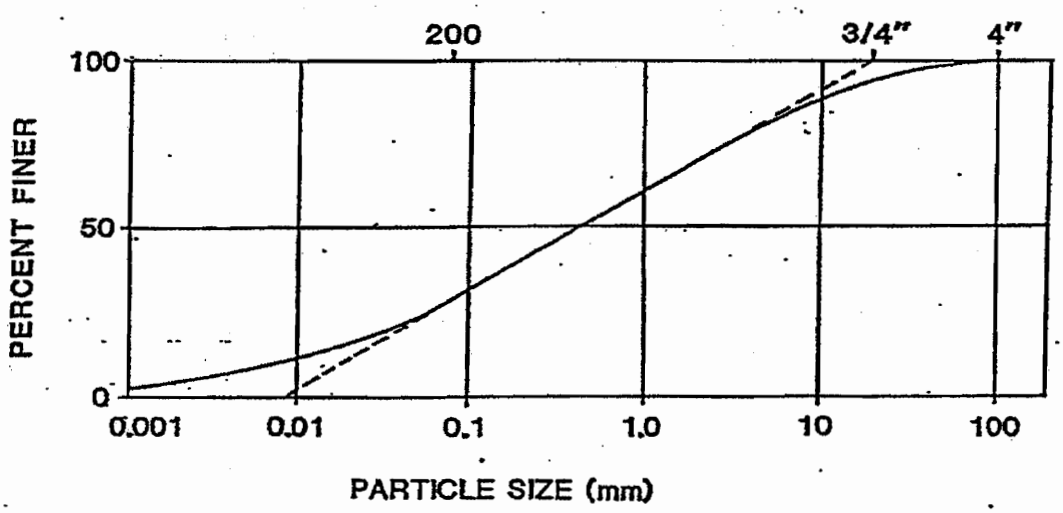

Figure 4. *Determination of the Linear Coefficient of Uniformity for the Considered Soil.*

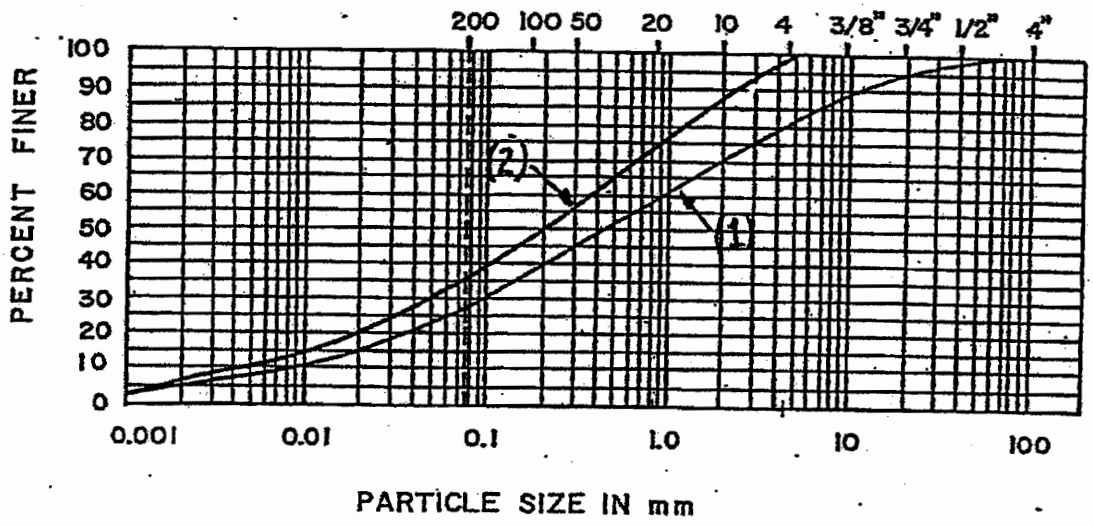

Figure 5. *Truncated Particle Size Distribution Curve.* Curve (1) is the actual curve (as shown in Figure 2) and curve (2) is the truncated curve derived from curve (1) by eliminating particles larger than 4.75 mm (U.S. Sieve No. 4).

Attachment 2
Geotextile Analys

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Technical Paper by E.M. Palmeira and M.G. Gardoni

The Influence of Partial Clogging and Pressure on the Behaviour of Geotextiles in Drainage Systems

ABSTRACT: Nonwoven geotextiles have been used for drainage and filtration in geotechnical engineering works for many years. Concerns related to drainage capacity and clogging potential still remain as factors that restrain a broader use of geotextiles for drainage systems, particularly in major engineering projects. This paper presents the test results of the hydraulic characteristics of partially clogged geotextiles under pressure. Partial clogging can occur during spreading and compaction of soil on geotextiles or throughout the service life of the drainage system. Geotextile specimens, artificially clogged in the laboratory and exhumed from actual field works, were tested to assess their normal and longitudinal permeabilities under different levels of soil impregnation and normal stresses. The results obtained showed that partial clogging significantly influenced the mechanical and hydraulic characteristics of nonwoven geotextiles and that soil impregnation was not necessarily detrimental to the geotextile longitudinal permeability under stress. Comparisons of test and predicted results, confirmed that the expression reported by Giroud in 1996 is a useful tool for the prediction of nonwoven geotextile permeabilities under virgin and soil impregnated conditions. Data on the impregnation levels of geotextile specimens exhumed from actual field works are also presented and discussed.

KEYWORDS: Geotextile, Drainage, Filtration, Clogging, Laboratory testing, Permeability.

AUTHORS: E.M. Palmeira, Associate Professor, and M.G. Gardoni, Ph.D. Student, University of Brasília, Department of Civil and Environmental Engineering, Faculty of Technology, 70910-900 Brasilia, DF, Brazil, Telephone: 55/61-273 7313, Telefax: 55/61-273 4644, E-mail: palmeira@unb.br.

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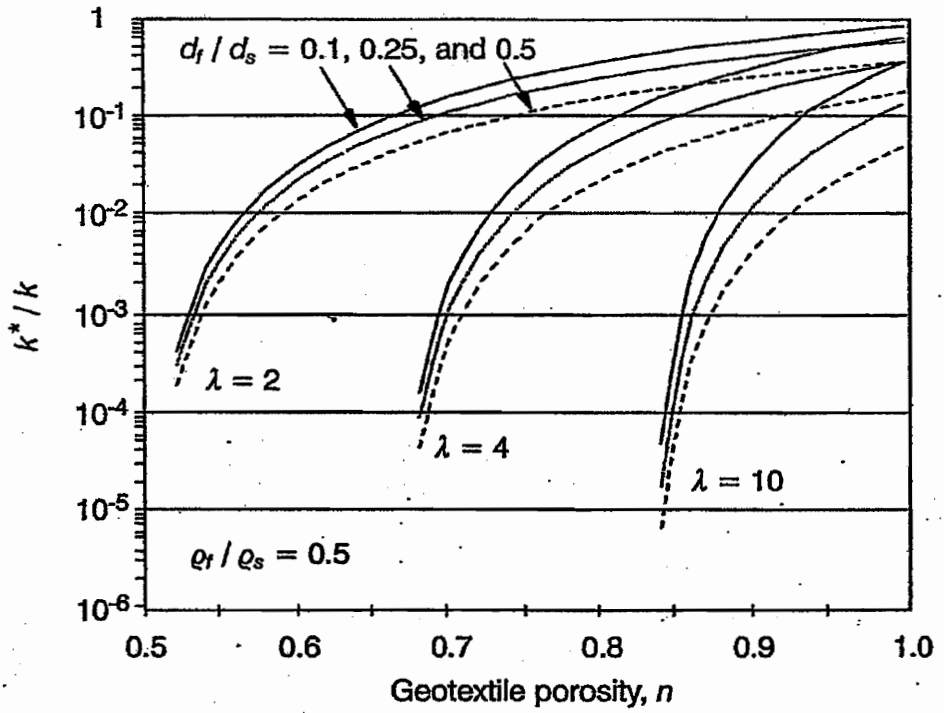


Figure 2. Variation of geotextile permeability with porosity for different λ values (Equation 9).

Note: λ = ratio of soil mass to geotextile fibre mass for partially clogged geotextiles.

3 TESTING PROGRAMME

3.1 Apparatus Used in the Testing Programme

Transmissivity and permittivity tests were performed to evaluate the behaviour of partially clogged geotextile specimens. Both transmissivity and permittivity tests were carried out with the geotextile specimen under normal stresses of up to 2,000 kPa in some cases. The behaviour of the geotextile under these high normal stresses is important in the case of drainage systems for large earth works and waste piles. In addition to the laboratory tests, the partially clogged geotextile specimens were also investigated using an image analyser and scanning microscope.

Figure 3 shows the transmissivity apparatus used in this research programme. The apparatus is similar to that proposed in the standard test method ASTM D 4716. Geosynthetic specimens, 100 mm × 100 mm in size, can be accommodated in the testing cell. The normal stress is applied by a rigid metal plate covering the entire plan area of the specimen. A hydraulic system provides the necessary vertical load, which is measured by a load cell. Rubber seals at the edges of the rigid plate, covering its entire thickness, prevent preferential flow and leakage through gaps or grooves. Distilled water reservoirs at the specimen ends allows the water to flow under a constant hydraulic gradient that can be varied between 0.2 and 3. The variation of the geotextile specimen

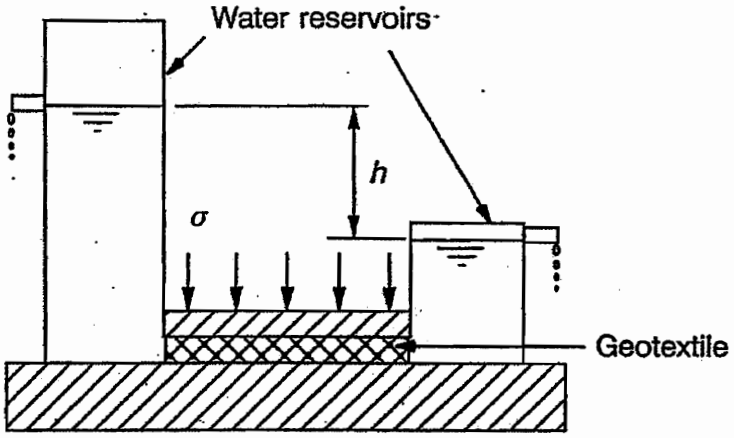


Figure 3. Apparatus for geotextile specimen transmissivity tests.

thickness during the test can also be assessed from the rigid plate vertical displacements measured by displacement transducers. Only tests with a hydraulic gradient equal to 1 are reported in the present paper. Four equally spaced (20 mm) piezometers that measure the water head variations along the geosynthetic length are connected to the base of the specimen. The piezometer measurements were useful in detecting variations of hydraulic properties along the geotextile length (Gardoni and Palmeira 1999).

The permittivity tests were carried out using two different apparatuses. The first (Figure 4a) was used at École Polytechnique, Montréal, Quebec, Canada and was designed according to ASTM D 5493. It comprises a permeameter cell, a de-aired water supply, and a hydraulic system that applies normal stresses on the specimen in the range 20 to 1,000 kPa. The permeameter cell is made of stainless steel with a diameter of 50.8 mm and a height of 280 mm. A 52 mm-diameter geotextile specimen can be accommodated in the cell for testing. A steel piston transfers the vertical load from a hydraulic loading system to the geotextile specimen, which is located between two sets of steel screen meshes for a uniform normal stress distribution. The openings of the screen meshes vary from 1 to 5 mm. Vertical displacements of the upper specimen surface can be measured using a linear variable displacement transducer (LVDT) fixed to the loading piston. Two ports located above and below the geotextile specimen measure the water head loss in the geotextile.

The second permeameter used for the permittivity tests is shown in Figure 4b. A detailed description of this permeameter is reported by Fannin et al. (1996) and Palmeira et al. (1996). In this case, the apparatus comprises a rigid metal cell capable of accommodating soil or geotextile specimens. The specimens consist of geotextile "packs" comprising several individual layers of geotextile. The body of the permeameter is made of anodised aluminium and can accommodate 102 mm-diameter geotextile specimens with heights of up to 125 mm. A pneumatic Bellofram piston transfers vertical load to the specimen. A load cell and a LVDT fixed to the loading piston measure vertical load and compression of the specimen, respectively. Water head losses measured above and below the specimen and flow rate measurements enabled the calculation of

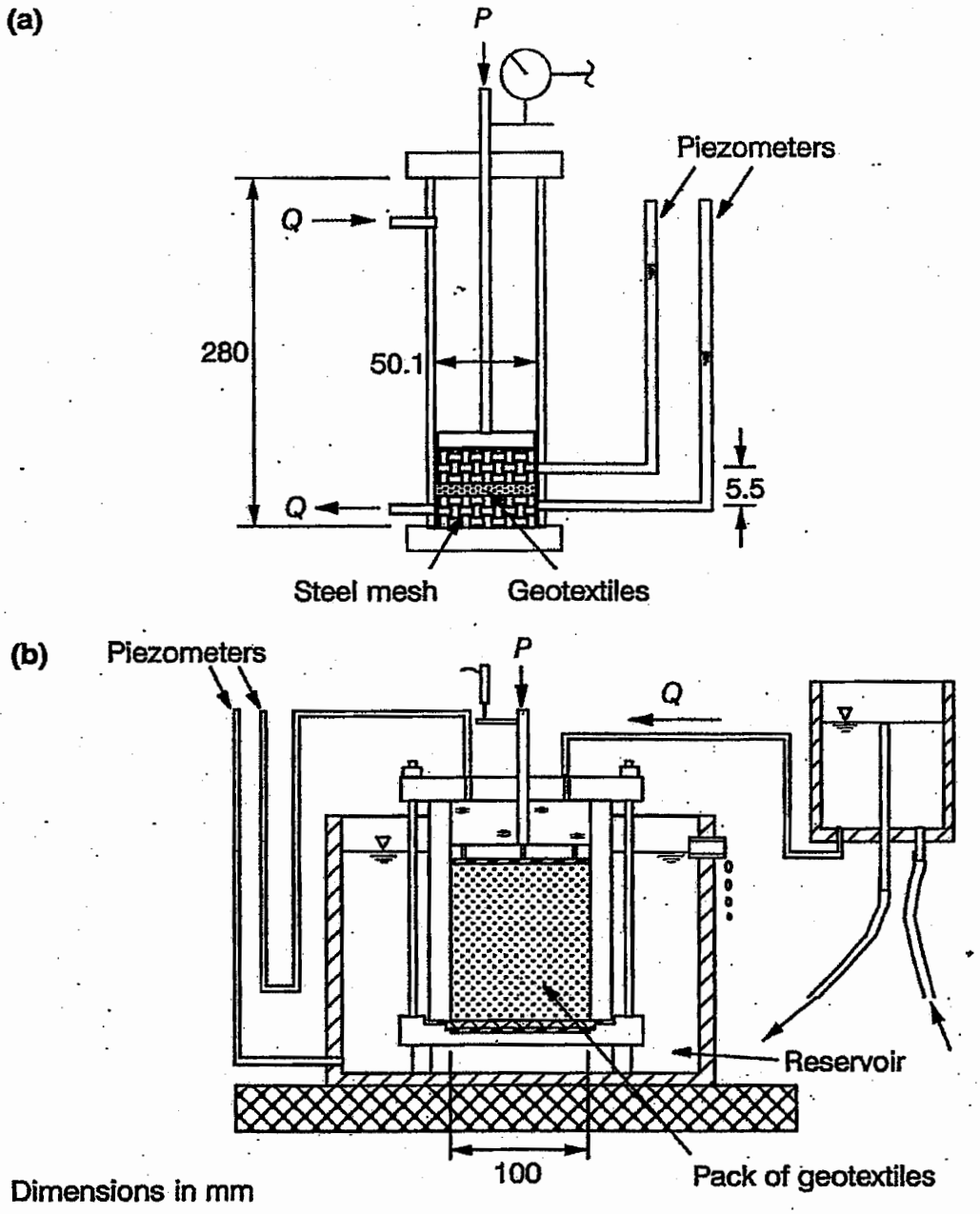


Figure 4. Apparatuses used for geotextile specimen permittivity tests: (a) apparatus for testing individual layers of geotextiles; (b) apparatus for testing packs of geotextile layers.

the average geotextile normal permeability. De-aired water was used in all of the tests performed in the present study.

To assess the dimensions of the soil particles entrapped inside the geotextile layers, investigations were carried out using a Clemex Impak Automatic Image Analizer avail-

Table 2. Characteristics of the soils used.

Soil	Soil type [1]	D_{10} (mm)	D_{50} (mm)	D_{85} (mm)	C_U [2]	w_L (%)	w_P (%)	w_{opt} (%)	G	γ_{dmax} (kN/m^3)
SA	RSQ	0.016	0.11	0.2	9.4	—	—	—	2.73	20.3
SB	Sand	0.053	0.42	0.61	11.5	—	—	—	2.80	—
SC	CS	0.010	0.026	0.20	5.9	48	35	27.2	2.54	14.7
SD	GB1	0.062	0.22	0.36	3.7	—	—	—	2.38	—
SE	GB2	0.055	0.17	0.46	4.6	—	—	—	2.43	—
SF	GB3	0.089	0.11	0.18	1.4	—	—	—	2.43	—
SG	GB4	0.054	0.12	0.14	2.2	—	—	—	2.48	—

Notes: [1] RSQ = residual soil from quartzite collected from the BR-020 Highway geotextile drainage system; Sand = sand collected from the backfill of the Mucambo geotextile-reinforced wall; CS = clayey soil; GB1 = Glass beads 1; GB2 = Glass beads 2; GB3 = Glass beads 3; GB4 = Glass beads 4. [2] C_U = coefficient of uniformity (= D_{60} / D_{10}). D_{85}, D_{50}, and D_{10} = diameter of the soil particle corresponding to 85, 50, and 10% in weight passing, respectively. w_L = liquid limit, w_P = plastic limit, w_{opt} = optimum moisture content (normal Proctor energy), G = specific gravity, γ_{dmax} = maximum dry unit weight (normal Proctor energy).

The geotextile specimens were previously saturated with de-aired water and were then exposed to a vacuum for at least two hours. Installation of the geotextile specimen in the permeameter was performed under total submersion in de-aired water to maintain specimen saturation. The steel screens used in the permittivity tests (Section 3.1) were submitted to the same saturation process by vacuum.

Impregnation of the geotextile specimens with soil particles for the permittivity and transmissivity tests was made under laboratory and field conditions with the use of seven granular materials. The main physical characteristics of the granular materials are summarised in Table 2. Figures 5a and 5b show the particle size distributions of these granular materials; in Figures 5a and 5b, a wide range of particle sizes can be noted. One residual quartzite soil (Soil SA in Table 2), one sand (Soil SB), one clayey soil (Soil SC), and four types of glass beads were used to partially clog the geotextile specimens. The residual quartzite soil is common in Brasilia, Brazil, and is known for having caused severe clogging of granular highway drainage systems in Brasilia (Gardoni and Palmeira 1998). Appropriate quantities of this soil were collected at the contact with a 400 m-long nonwoven geotextile filter in the BR-020 Highway, near the city of Brasilia. Soil SB (sand) was collected from the Mucambo geotextile-reinforced retaining wall structure, built along the Linha Verde Highway, in the state of Bahia, Brazil (Palmeira and Fahel 2000). The clayey soil (Soil SC) comes from a porous clay deposit that covers most of the city of Brasilia. Glass beads SD to SG are industrial-grade glass spheres with particle sizes varying from 0.04 to 1 mm. The spherical form of the glass beads makes it convenient for assessing the accuracy of theoretical expressions developed under the assumption of spherical entrapped soil particles.

It should be noted that, for the finer materials (Soils SA and SC), there are two different particle size distribution curves (Figure 5a), depending on whether or not a deflocculant was used in the sedimentation tests. For these soils, clusters of soil particles, rather than individual particles, were retained in the geotextile filter (Gardoni and Palmeira 1998). The flocculation mechanism, aside from complicating filter design, poses the

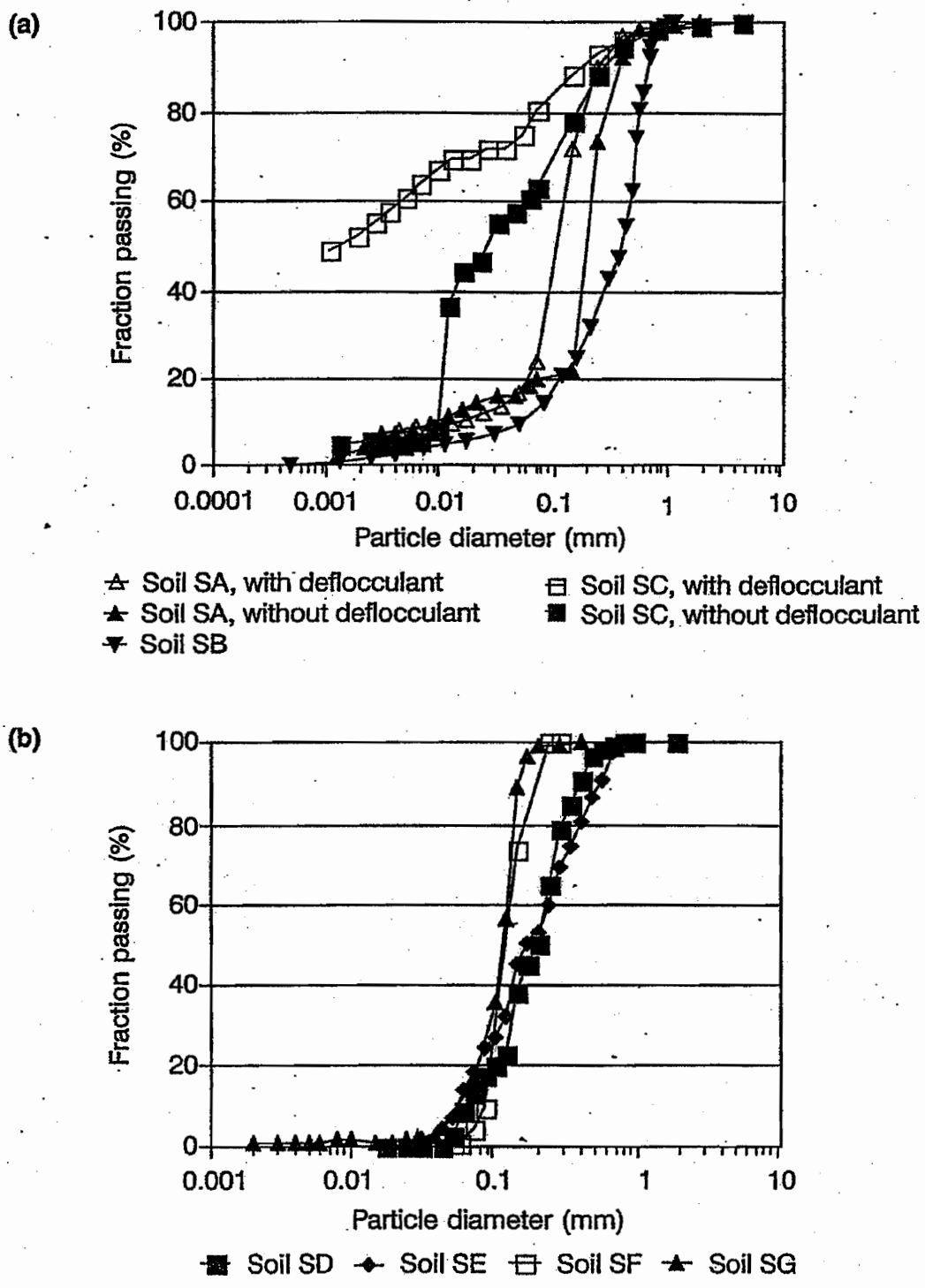


Figure 5. Particle size curves for the soils used in the test programme: (a) Soils SA, SB, and SC; (b) Soils SD, SE, SF, and SG (glass beads).

following question: can these particles be separated from one another by continuous water flow during the lifetime of the filter and then be retained inside the geotextile layer?

In the laboratory, the impregnation of the geotextile specimens with glass beads (Soils SD, SE, and SF in Table 2) was achieved by vibrating a sieve, which contained a geotextile specimen with soil on top of it. The system was vibrated for varying time periods, depending on the level of soil impregnation desired. In the case of the sandy soil, Soil SB, the geotextile specimen used in the test (transmissivity only) was exhumed from the Mucambo reinforced soil structure and tested under the conditions found after exhumation. In addition, two geotextile specimens exhumed from the BR-020 Highway drain were also tested after exhumation.

The geotextile impregnation with the clayey soil (Soil SC) was accomplished using two different procedures. For the first procedure, soil on the geotextile, which was placed at the base of a steel compaction mould used for standard compaction tests, was compacted. The compaction energy used in this case was the Proctor's normal energy and the soil optimum water content was used for compaction. The second procedure simulated geotextile impregnation with Soil SC using field test sections. In each test section, a layer of the geotextile (2 m × 1m) was laid on the ground and the fill material was spread and then compacted over it using a sheep's foot roller. The fill material was compacted under optimum moisture content conditions, using the same compaction energy that was used in the laboratory. After compaction, the geotextile specimens were carefully exhumed from the test sections for laboratory testing.

4 TEST RESULTS

4.1 Permittivity Tests

4.1.1 *Tests with Virgin Specimens*

Figures 6 and 7 show the variation of geotextile thickness, t_{GT}, normal permeability, k_n, and permittivity, ψ, versus normal stress, σ, for some of the geotextiles tested. A marked dependency of t_{GT}, k_n, and ψ on the normal stress can be observed, particularly for values of σ below 50 kPa. Figure 6a shows that the normal stress affects the geotextile thickness for values of up to 800 kPa. For the same normal stress, lighter geotextiles (i.e. low mass per unit area values) have a higher normal permeability value (Figure 6b), and, for products of the same manufacturer, the k_n value drops with increased mass per unit area values. It is also important to note that a significant difference in permeability can be observed for products with the same mass per unit area, but from different manufacturers (e.g. Geotextiles GC and GG in Figure 6b), emphasising the importance of geotextile microstructure.

The comparison of predicted and measured geotextile permeability values is shown in Figure 8. The following relevant values for the fluid (water) were used in the calculations: $\beta = 0.11$, $g = 9.81$ m/s^2, $\varrho_w = 1000$ kg/m^3, and $\eta_w = 0.001$ kg/(ms). Test results obtained by Gourc et al. (1982) for other geotextiles are also presented for comparison. Hydraulic conductivity values, predicted using Equations 2, 4, and 5, are also presented in Figure 8; it can be observed that Equation 2 (Giroud 1996) provided the best fit with the test results.

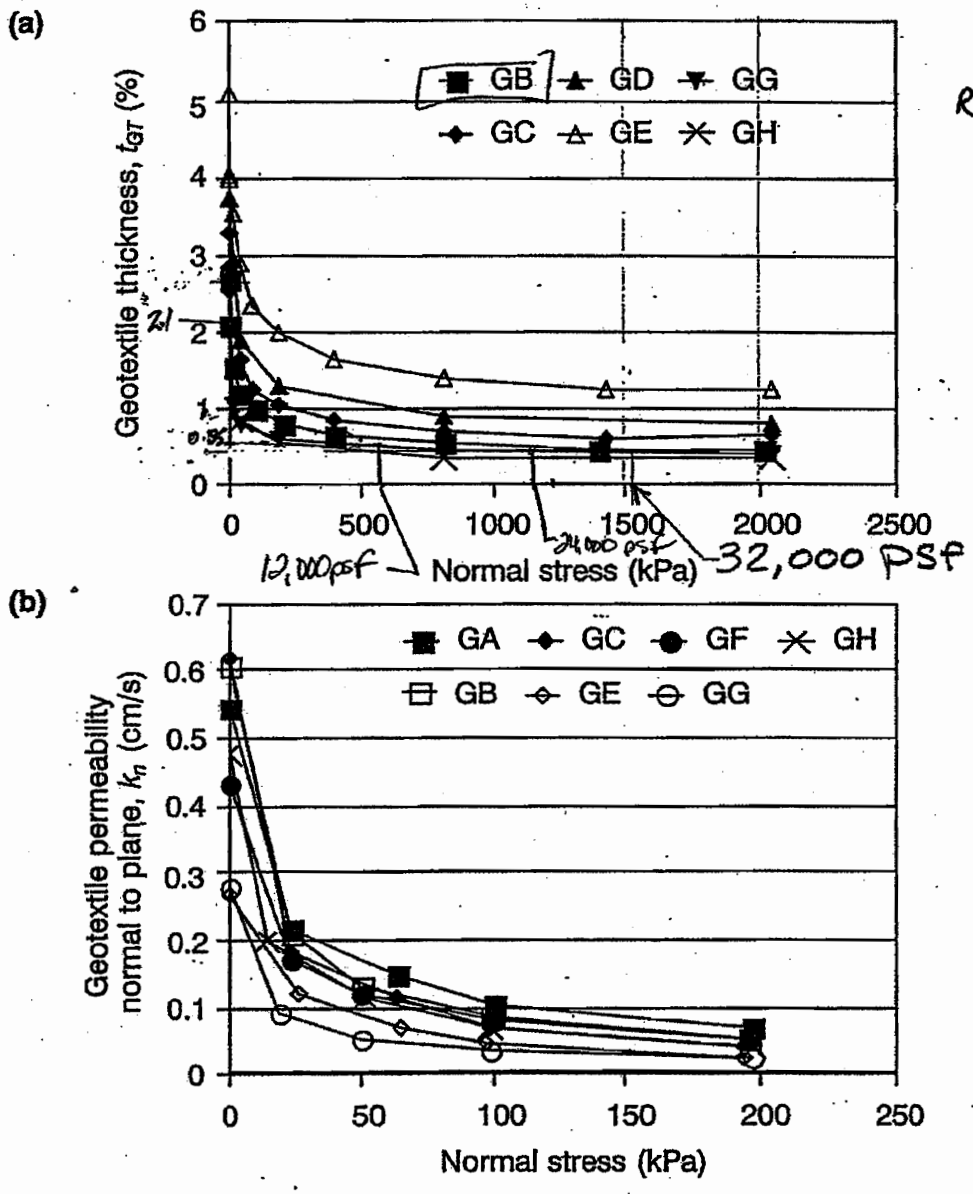


Figure 6. Variation of geotextile normal permeability with normal stress for virgin geotextile specimens: (a) average geotextile thickness versus normal stress; (b) normal permeability versus normal stress.

Figure 9 shows comparisons of test results and values predicted using Equation 2 alone in a nondimensional form. A good agreement between predicted and measured values is confirmed, particularly for geotextile porosities above 0.8 and geotextile mass per unit area values greater than 300 g/m^2.

The variation of the geotextile filtration opening size should be known to assess the accuracy of Equation 3. Figures 10a to 10c show the comparisons of predicted and mea-

Attachment 3
Geotextile
Analysis

Fourth Edition

Designing with Geosynthetics

Robert M. Koerner, Ph.D., P.E.

H. L. Bowman Professor of Civil Engineering,
Drexel University and Director, Geosynthetic
Research Institute

PRENTICE HALL
Upper Saddle River, New Jersey 07458

TABLE 2.12 RECOMMENDED REDUCTION FACTOR VALUES FOR USE IN EQ. (2.25a)

Application	Range of Reduction Factors				
	Soil Clogging and Blinding*	Creep Reduction of Voids	Intrusion into Voids	Chemical Clogging†	Biological Clogging
Retaining wall filters	2.0 to 4.0	1.5 to 2.0	1.0 to 1.2	1.0 to 1.2	1.0 to 1.3
Underdrain filters	5.0 to 10	1.0 to 1.5	1.0 to 1.2	1.2 to 1.5	2.0 to 4.0
Erosion-control filters	2.0 to 10	1.0 to 1.5	1.0 to 1.2	1.0 to 1.2	2.0 to 4.0
Landfill filters	5.0 to 10	1.5 to 2.0	1.0 to 1.2	1.2 to 1.5	5 to 10‡
Gravity drainage	2.0 to 4.0	2.0 to 3.0	1.0 to 1.2	1.2 to 1.5	1.2 to 1.5
Pressure drainage	2.0 to 3.0	2.0 to 3.0	1.0 to 1.2	1.1 to 1.3	1.1 to 1.3

*If stone riprap or concrete blocks cover the surface of the geotextile, use either the upper values or include an additional reduction factor.

†Values can be higher particularly for high alkalinity groundwater.

‡Values can be higher for turbidity and/or for microorganism contents greater than 5000 mg/l.

$$q_{allow} = q_{ult}\left(\frac{1}{\text{IIRF}}\right) \tag{2.25b}$$

where

q_{allow} = allowable flow rate,
q_{ult} = ultimate flow rate,
RF_{SCB} = reduction factor for soil clogging and blinding,
RF_{CR} = reduction factor for creep reduction of void space,
RF_{IN} = reduction factor for adjacent materials intruding into geotextile's void space,
RF_{CC} = reduction factor for chemical clogging,
RF_{BC} = reduction factor for biological clogging, and
IIRF = value of cumulative reduction factors.

As with Eqs. (2.24) for strength reduction, this flow-reduction equation could also have included additional site-specific terms, such as blocking of a portion of the geotextile's surface by riprap or concrete blocks.

2.5 DESIGNING FOR SEPARATION

Application areas for geotextiles used for the separation function were given in Section 1.3.3. There are many specific applications, and it could be said, in a general sense, that geotextiles always serve a separation function. If they do not also serve this function, any other function, including the primary one, will not be served properly. This should not give the impression that the geotextile function of separation always plays a secondary role. Many situations call for separation only, and in such cases the geotextiles serve a significant and worthwhile function.

2.5.1 Overview of Applications

Perhaps the target application that best illustrates the use of geotextiles as separators is their placement between a reasonably firm soil subgrade (beneath) and a stone base course, aggregate, or ballast (above). We say "reasonably firm" because it is assumed that the subgrade deformation is not sufficiently large to mobilize uniformly high tensile stress in the geotextile. (The application of geotextiles in unpaved roads on soft soils with membrane-type reinforcement is treated later in Section 2.6.1.) Thus for a separation function to occur the geotextile has only to be placed on the soil subgrade and then have stone placed, spread, and compacted on top of it. The subsequent deformations are very localized and occur around each individual stone particle. A number of scenarios can be developed showing which geotextile properties are required for a given situation.

2.5.2 Burst Resistance

Consider a geotextile on a soil subgrade with stone of average particle diameter (d_a) placed above it. If the stone is uniformly sized, there will be voids within it that will be available for the geotextile to enter. This entry is caused by the simultaneous action of the traffic loads being transmitted to the stone, through the geotextile, and into the underlying soil. The stressed soil then tries to push the geotextile up into the voids within the stone. The situation is shown schematically in Figure 2.28. Giroud [64] provides a formulation for the required geotextile strength that can be adopted for this application.

$$T_{reqd} = \frac{1}{2} p' d_v [f(\varepsilon)] \tag{2.26}$$

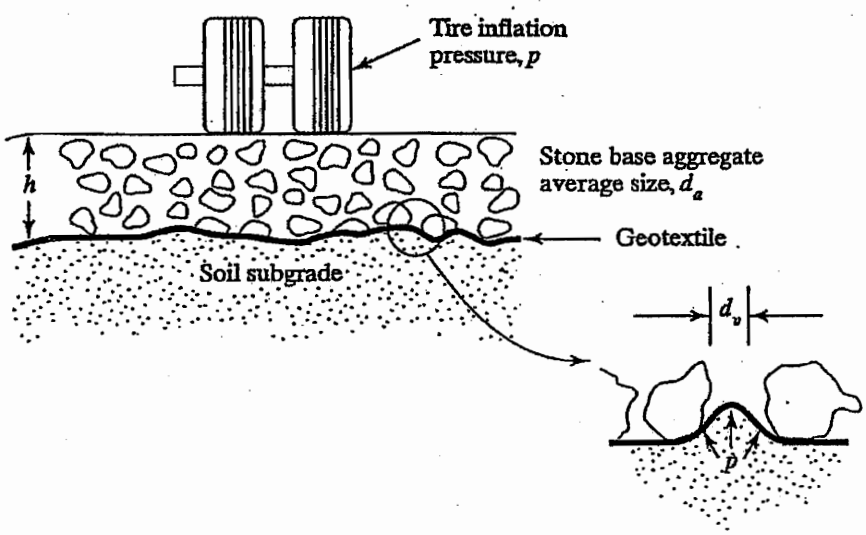


Figure 2.28 Geotextile being forced up into voids of stone base by traffic tire loads.

where

T_{reqd} = required geotextile burst strength;
p' = stress at the geotextile's surface, which is less than or equal to p, the ti inflation pressure at the ground surface;
d_v = maximum void diameter of the stone $\cong 0.33 d_a$;
d_a = the average stone diameter,
$f(\varepsilon)$ = strain function of the deformed geotextile
$= \frac{1}{4}\left(\frac{2y}{b} + \frac{b}{2y}\right)$, in which
b = width of opening (or void), and
y = deformation into the opening (or void).

The field situation is analogous to the ASTM D3786 (Mullen) burst test, which has t geotextile being stressed into a gradually increasing hemispherical shape until it fails radial tension (recall Section 2.3.3). Thus, the adapted form of Eq. (2.26) is:

$$T_{ult} = \frac{1}{2} p_{test} d_{test} [f(\varepsilon)] \qquad (2.2$$

where

T_{ult} = ultimate geotextile strength,
p_{test} = burst test pressure, and
d_{test} = diameter of the burst test device (= 30 mm).

Knowing that $T_{allow} = T_{ult}/(\text{IIRF})$, where IIRF = cumulative reduction factors, we ca formulate an expression for the FS as follows:

$$\text{FS} = \frac{T_{allow}}{T_{reqd}} = \frac{(p_{test} d_{test})}{(\text{IIRF}) p' d_v}$$

For example, if $d_{test} = 30$ mm, $d_v = 0.33\ d_a$, and IIFS = 1.5 (which is not particularly lov since creep is not an issue with this application), then the FS is the following, with d_a in mm.

$$\text{FS} = \frac{p_{test}(30)}{(1.5) p' (0.33 d_a)}$$

$$\text{FS} = \frac{60.6 p_{test}}{p' d_a} \qquad (2.28)$$

Example 2.7

Given a 700 kPa truck tire inflation pressure on a poorly graded stone-base course consisting of 50 mm maximum-size stone, what is the factor of safety using a geotextile with an ultimate burst strength of 2000 kPa and cumulative reduction factors of 1.5?

Solution: Assuming that the tire inflation pressure is not significantly reduced through the thickness of the stone base, we can solve Eq. (2.28) as follows.

$$FS = \frac{60.6(2000)}{700(50)}$$

$$= 3.5$$

Note that with the cumulative reduction factors of 1.5 already included, the resulting factor of safety value is acceptable.

For a range of stone-base particle diameters (d_a), values of tire inflation pressure (p'), and cumulative reduction factors of 1.5, along with a factor of safety of 2.0, we get the design guide in Figure 2.29. Here it can be seen that stone size is quite significant insofar as the required burst-pressure values are concerned. Note also that these are poorly graded aggregates and that the presence of fines will lessen the severity of the design; hence this approach should be considered to be a worst-case design.

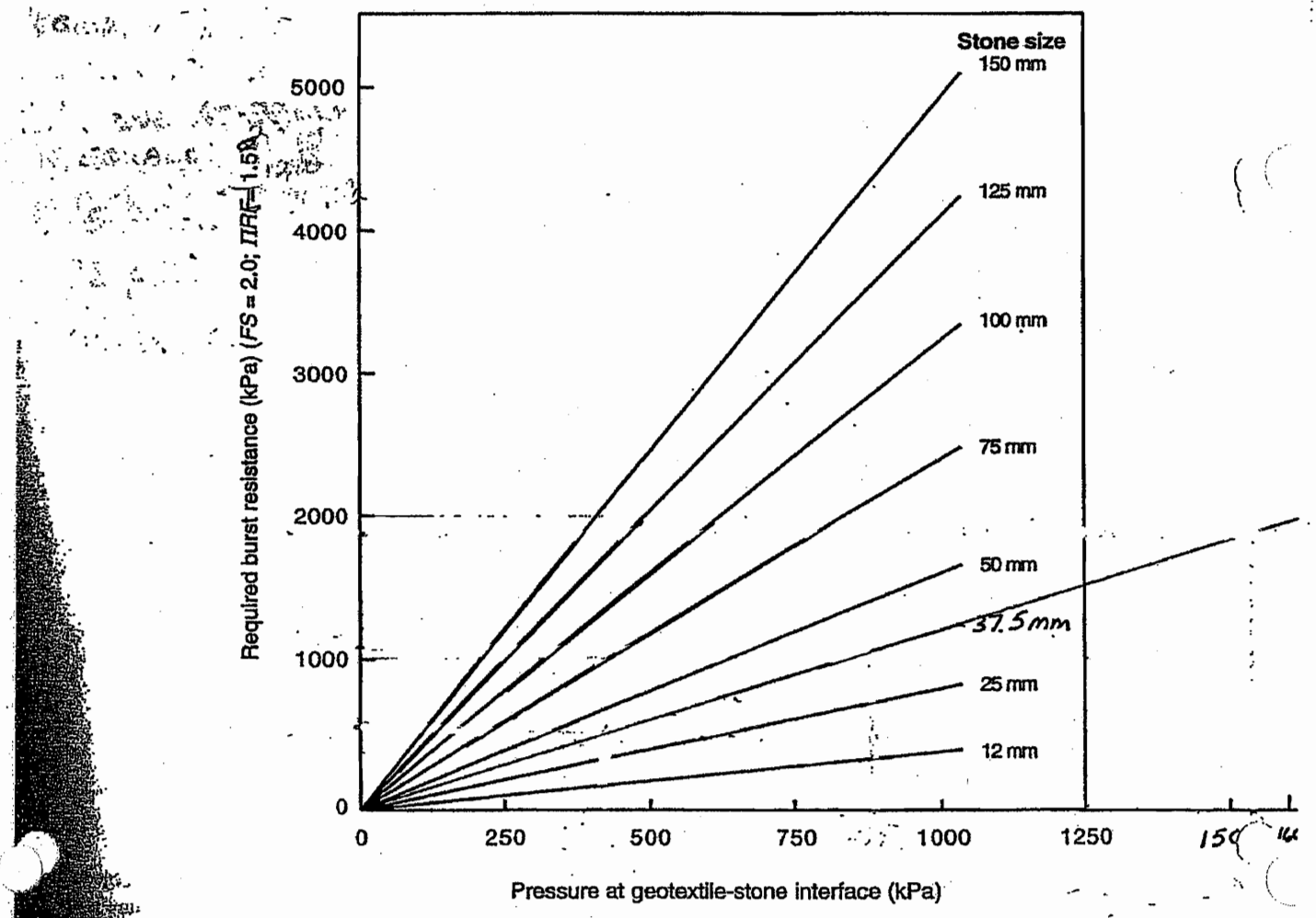


Figure 2.29 Design guide for burst analysis of geotextile used in a separation function based on cumulative reduction factors of 1.5 and a factor of safety of 2.0.

2.5.3 Tensile Strength Requirement

Continuing the discussion of the general problem, there is a process acting on the geotextile simultaneously as its tendency to burst in an out-of-plane mode: tensile stress mobilized by in-plane deformation. This occurs as the geotextile is locked into position by the stone-base aggregate above it and soil subgrade below it. A lateral or in-plane tensile stress in the geotextile is mobilized when an upper piece of aggregate is forced between two lower pieces that lie against the geotextile. The analogy to the grab tensile test can be readily visualized, as illustrated in Figure 2.30. Here we can estimate the maximum strain that the geotextile will undergo as the upper stone wedges itself down to the level of the geotextile. Using the dimensions shown (where S ~ $d/2$ and l_f = deformed geotextile length), the maximum strain with no slippage or stone breakage can be calculated.

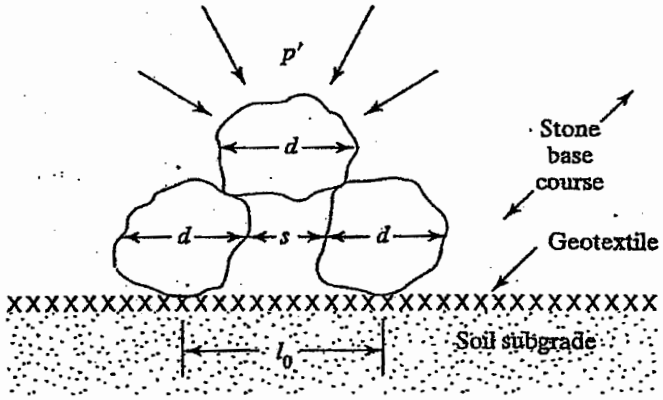
$$\varepsilon = \frac{l_f - l_o}{l_o}(100)$$

$$= \frac{[d + 2(d/2)] - 3(d/2)}{3(d/2)}(100)$$

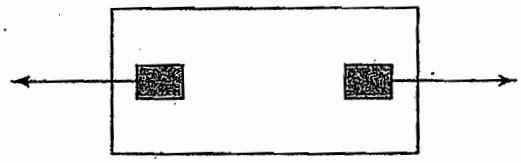
$$= \frac{4(d/2) - 3(d/2)}{3(d/2)}(100)$$

$$= 33\%$$

Note that the preceding assumptions result in a strain that is independent of particle size. Thus the strain in the geotextile could be as high as 33% given the idealized



(a) Actual situation



(b) Analogous grab tension test

Figure 2.30 Geotextile being subjected to tensile stress as surface pressure is applied and stone base attempts to spread laterally.

(upper-bound) assumptions stated above. The tensile force being mobilized is related to the pressure exerted on the stone as follows [64].

$$T_{reqd} = p'(d_v)^2[f(\varepsilon)] \tag{2.29}$$

where

T_{reqd} = required grab tensile force;
p' = applied pressure;
d_v = maximum void diameter $\simeq 0.33\, d_a$, where
d_a = average stone diameter; and
$f(\varepsilon)$ = strain function of the deformed geotextile;
$= \frac{1}{4}\left(\frac{2y}{b} + \frac{b}{2y}\right)$, where
b = width of stone void, and
y = deformation into stone void.

Example 2.8 illustrates the design procedure above.

Example 2.8

Given a 700 kPa truck-tire inflation pressure on a stone-base course consisting of 50 mm maximum-size stone with a geotextile beneath it, calculate **(a)** the required grab tensile stress on the geotextile, and **(b)** the factor of safety for a geotextile whose grab strength at 33% is 500 N with cumulative reduction factors of 2.5 and $f(\varepsilon) = 0.52$.

Solution: (a) Using an empirical relationship that $d_v = 0.33\, d_a$ and $f(\varepsilon) = 0.52$, the required grab tensile strength from Eq. (2.29) is as follows.

$$\begin{aligned} T_{reqd} &= p'(d_v)^2(0.52) \\ &= p'(0.33d_a)^2(0.52) \\ &= 0.057\, p' d_a^2 \\ &= 0.057(700)(1000)(0.0 \\ &= 100 \text{ N} \end{aligned}$$

(b) The factor of safety for a 500 N grab tensile geotextil reduction factors of 2.5 is as follows.

$$\begin{aligned} \text{FS} &= \frac{T_{allow}}{T_{reqd}} \\ &= \frac{500/2.5}{100} \\ &= 2.0 \qquad \text{which is acceptabl..} \end{aligned}$$

2.5.4 Puncture Resistance

The geotextile must survive the installation process. This is not just related to the function of separation; indeed, fabric survivability is critical in all types of applications—without it the best of designs are futile (recall Figure 2.19). In this regard, sharp stones,

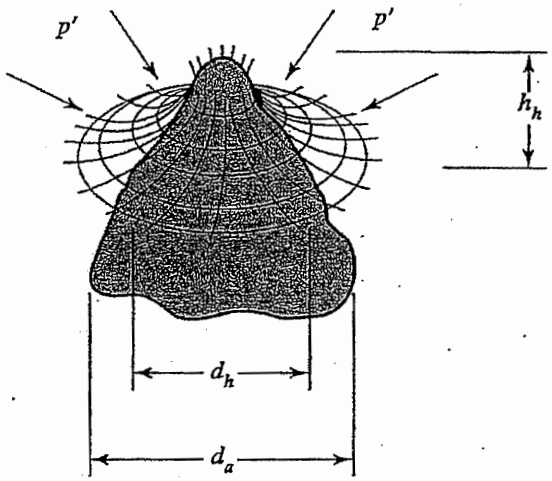


Figure 2.31 Visualization of a stone puncturing a geotextile as pressure is applied from above.

tree stumps, roots, miscellaneous debris, and other items, either on the ground surface beneath the geotextile or placed above it, could puncture through the geotextile after backfilling and traffic loads are imposed. The design method suggested for this situation is shown schematically in Figure 2.31. For these conditions, the vertical force exerted on the geotextile (which is gradually tightening around the protruding object) is as follows:

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

where

F_{reqd} = required vertical force to be resisted;
d_a = average diameter of the puncturing aggregate or sharp object;
p' = pressure exerted on the geotextile (approximately 100% of tire inflation pressure at the ground surface for thin covering thicknesses);
S_1 = protrusion factor = h_h/d_a;
h_h = protrusion height $\leq d_a$;
S_2 = scale factor to adjust the ASTM D4833 puncture test value (which uses an 8.0 mm diameter puncture probe) to the diameter of the actual puncturing object = d_{probe}/d_a;
S_3 = shape factor to adjust the ASTM D4833 flat puncture probe to the actual shape of puncturing object = $1 - A_p/A_c$, (values for A_p/A_c range from 0.8 for rounded sand, to 0.7 for run-of-bank gravel, to 0.4 for crushed rock, to 0.3 for shot rock);
A_p = projected area of puncturing particle;
A_c = area of smallest circumscribed circle around puncturing particle.

Example 2.9

What is the factor of safety against puncture of a geotextile from a 50 mm stone on the ground surface mobilized by a loaded truck with a tire inflation pressure of 550 kPa traveling on the surface of the base course? The geotextile has an ultimate puncture strength of 200 N, according to ASTM D4833.

Solution: Using the full stress on the geotextile of 550 kPa and the values 0.33, 0.15, and 0.6 for the factors S_1, S_2, and S_3, respectively,

$$F_{reqd} = p' d_a^2 S_1 S_2 S_3$$
$$= (550)(1000)(50 \times 0.001)^2(0.33)(0.15)(0.6)$$
$$= 40.8 \text{ N}$$

Assuming that the cumulative reduction factors are 2.0, the factor of safety is as follows:

$$FS = \frac{F_{allow}}{F_{reqd}}$$
$$= \frac{200/2.0}{40.8}$$
$$= 2.4 \quad \text{which is acceptable}$$

Using the following assumptions (which can be modified as desired), a design guide can be developed as shown in Figure 2.32: the geotextile has an angular subgrade

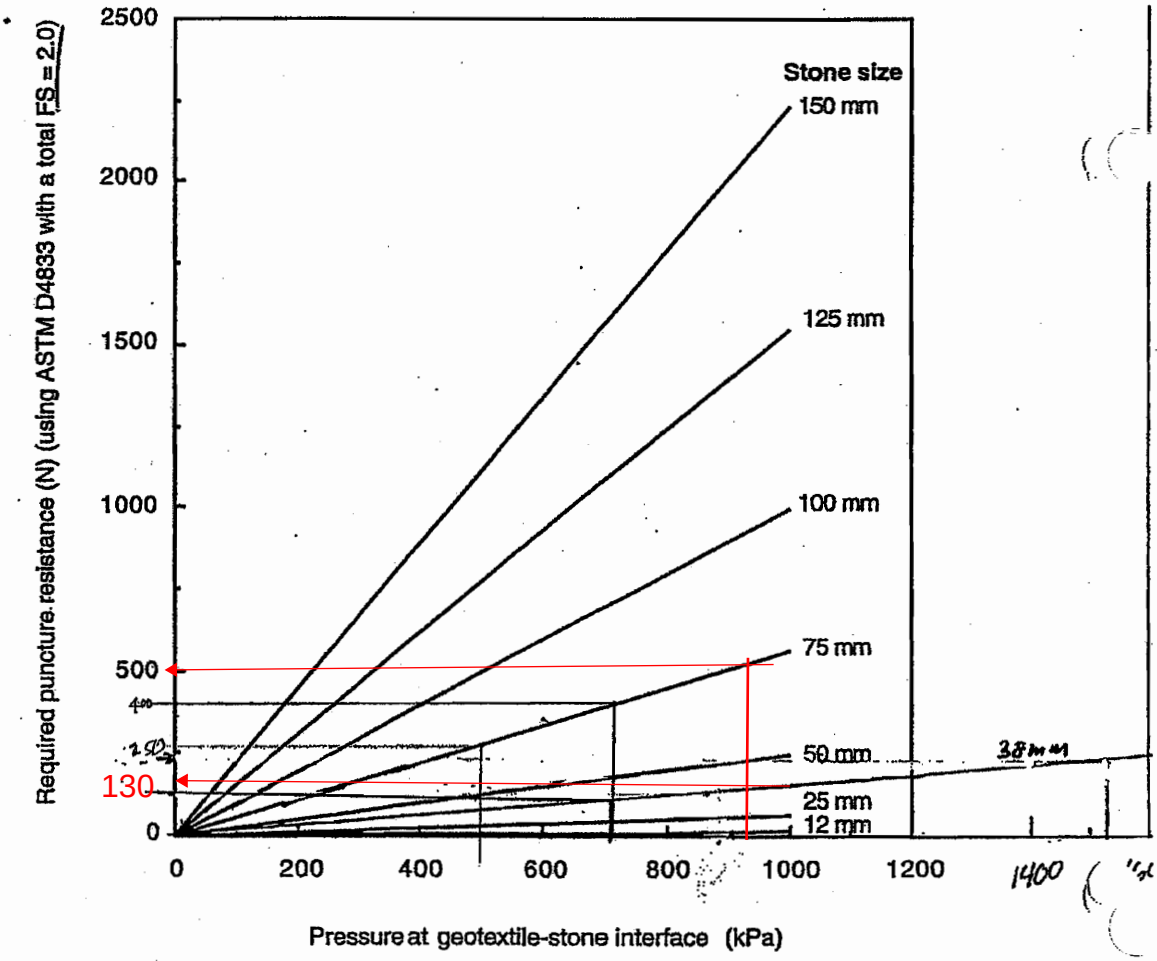


Figure 2.32 Puncture resistance design guide based on cumulative reduction factors of 2.0, a factor of safety of 2.0, and conditions stated in text.

above it such that $S_1 = 0.33$, $S_2 = 0.15$, and $S_3 = 0.5$; the cumulative reduction factors are 2.0; and the factor of safety is also 2.0.

$$F_{reqd} = p'd_a^2(0.33)(0.15)(0.5)$$

$$= 0.0248p'd_a^2$$

$$FS = \frac{F_{ult}/IIRF}{F_{reqd}}$$

$$2.0 = \frac{F_{ult}/2.0}{0.0248p'd_a^2}$$

$$F_{ult} = 0.099p'd_a^2 \qquad \text{which is graphed accordingly.}$$

2.5.5 Impact (Tear) Resistance

As with the puncture requirement just described, the resistance of a geotextile to impact is as much a survivability criterion as it is a separation function. Yet in many applications of separation, the geotextile must resist the impact of various objects. The most obvious one is a rock falling on it, but there are also situations in which construction equipment and materials can cause or contribute to impact damage on geotextiles.

The problem concerns the energy mobilized by a free-falling object of known weight and the height of the drop. Rarely will an object be intentionally impelled onto an exposed geotextile with additional force, so only gravitational energy will be assumed.

To develop a design guide, we assume free-falling stones of specific gravity of 2.60, varying in diameter from 25 to 600 mm and falling from heights of 0.5 to 5 m. Using this data the design curves of Figure 2.33 are developed. The relationship is as follows.

$$E = mgh$$

$$= (V \times \rho)gh$$

$$= [V \times (\rho_w G_s)]gh$$

$$= \left(\frac{\pi(d_a/1000)^3}{6}\right)\left(\frac{1000kg}{m^3}\right)(2.6)(9.81)h$$

$$E = 13.35 \times 10^{-6} d_a^3 h \qquad (2.31)$$

where

E = energy developed (joules),
m = mass of the object (kg),
g = acceleration due to gravity (m/s^2),
h = height of fall (m),

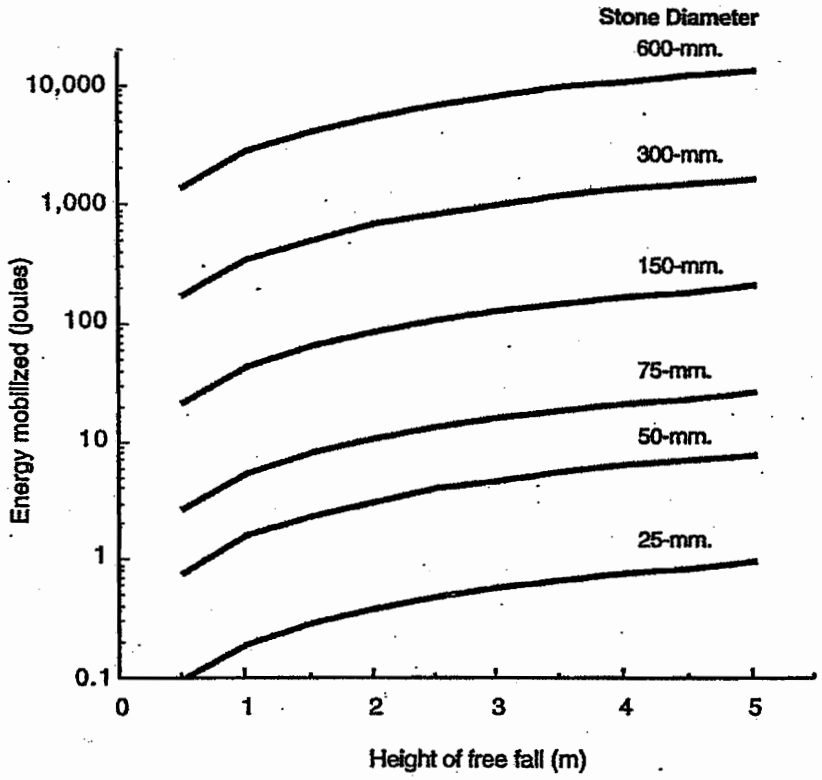


Figure 2.33 Energy mobilization by a free-falling rock on a geotextile with an unyielding support.

V = volume of the object (m^3),
ρ = density of the object (kg/m^3),
ρ_w = density of water (kg/m^3),
G_s = specific gravity of the object (dimensionless), and
d_a = diameter of the object (mm).

Note that these calculated energies are based on the geotextile resting on an unyielding surface, that is, the worst possible condition. As the soil beneath the geotextile deforms, the geotextile can absorb greater amounts of impacting energy. Since this is always the case, the reduction factors of Figure 2.34 are to be used in conjunction with the curves of Figure 2.33. Once the required energy is calculated, it should be compared to the allowable impact strength of the geotextile (e.g., the Elmerdorf tear or other impact test as discussed in Section 2.3.3). Example 2.10 illustrates the procedure.

Example 2.10

What energy is mobilized by a free-falling stone of 300 mm size falling 1.5 m onto a geotextile? The geotextile is supported by a poor subsoil having an unsoaked CBR strength of 4. If the geotextile has an allowable impact strength of 36 J, what is the factor of safety?

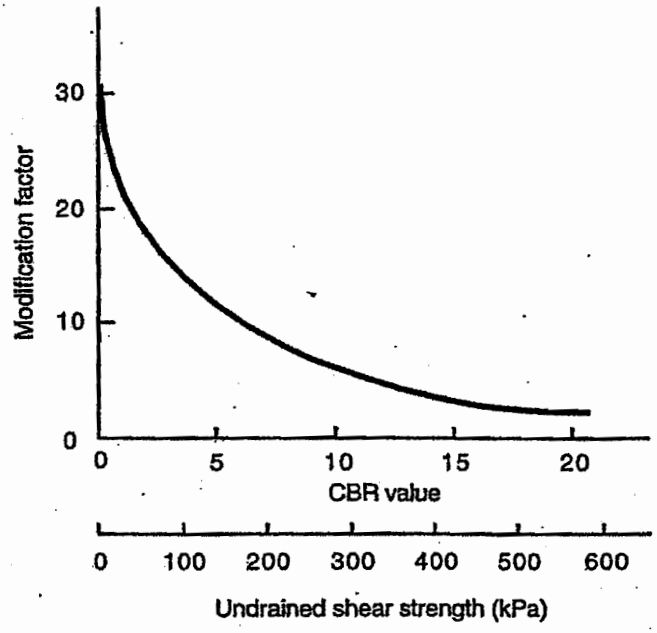


Figure 2.34 Modification factors to b used with energy mobilized by objects falling on geotextiles of varying suppo resistances characterized by their un-soaked CBR values or undrained shea strength.

Solution: Using Eq. (2.31) one calculates the required impact energy

$$E_{max} = 13.35 \times 10^{-6}(d_a^3)(\text{h})$$
$$= 13.35 \times 10^{-6}(300)^3(1.5)$$
$$E_{max} = 540 \text{ J}$$

Note that this value is substantiated by the design chart in Figure 2.33. Of course, other d sign charts can be made for different assumptions.

This value is reduced according to the subgrade conditions of Figure 2.34.

$$E_{reqd} = 540/13$$
$$= 41.5 \text{ J}$$

This results in a global factor of safety calculation as follows.

$$FS = \frac{E_{allow}}{E_{reqd}}$$
$$= \frac{36}{41.5}$$
$$FS = 0.87 \quad \text{which is not acceptable.}$$

Thus holes are likely to be formed when free-falling rocks of this size fall directly on th exposed geotextile. Not included in this analysis is the effect of the contact area of th falling object on the geotextile; for a very rounded rock, the effect is much less severe tha for a sharp, angular one, which could easily cut through the fabric. A more sophisticate design of this nature for puncture of geomembranes, along with geotextile protection, wi be developed in Section 5.6.7.

It should be emphasized that the last two methods of puncture and impact desig refer not only to separation per se, but to the construction survivability of geotextiles i

general (recall Table 2.2a). In all cases these considerations should be examined, for they are critical in many situations.

2.5.6 Summary

Separation, the *most underrated of all geotextile functions,* was addressed in this section. I say underrated because every use of geotextiles carries with it the separation function, yet rarely is separation designed on its own merit. Hopefully, the designs in this section will allow the engineer to determine quantitatively which geotextile is suitable for a specific situation.

Last, and in a sense most important, is the economic justification for the use of geotextiles in the separation function. It lies in the greater use and service lifetime of the system with geotextiles than without. When a geotextile separator is used in roadway cross sections, geotextiles could well double or triple the lifetime; however, field data for such quantification is sparse and greater efforts should be taken to provide test sites for this. Figure 2.35 is the photograph of a 40 m long driveway test plot, which was subdivided into four elongated quadrants, two with geotextiles and two without. Further, the two geotextiles were different and placed diagonally across from one another. After nine years of service, no cracks have surfaced on the paving and the test is continuing with the objective of providing lifetime data with and without geotextiles, and on which is the preferred type of geotextile. A database of like projects is being developed; see [65].

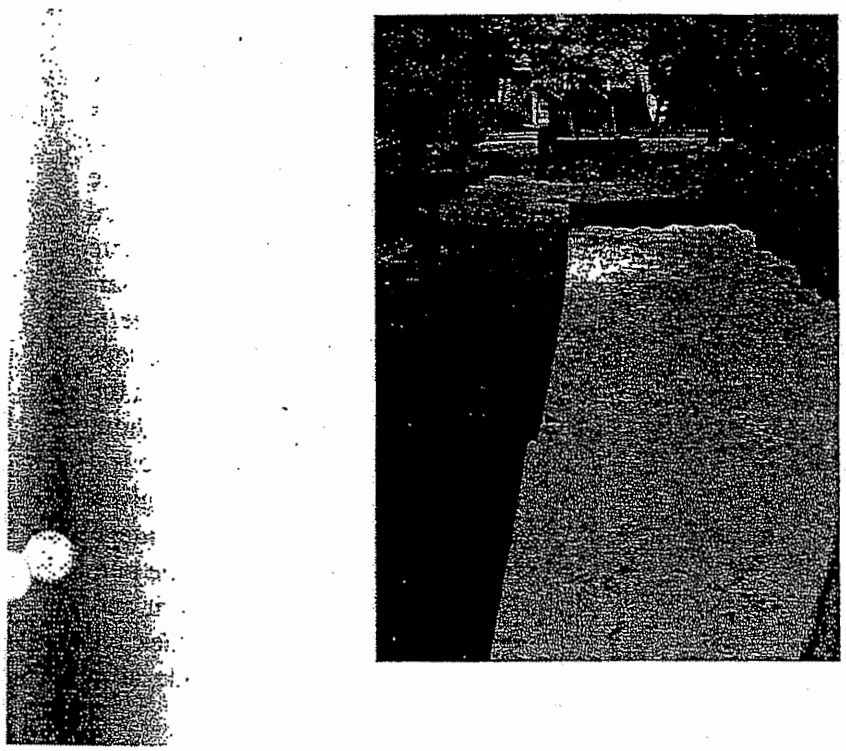


Figure 2.35 Different separation geotextiles being used to determine pavement (driveway) lifetime contrasted to sections with no geotextile.

Attachment 5
Geotextile Evaluation

GSE®

GSE STANDARD PRODUCTS

Product Data Sheet

GSE Nonwoven Geotextile

GSE Nonwoven Geotextiles is a family of polypropylene, staple fiber, nonwoven, needlepunched geotextiles. Manufactured using an advanced manufacturing and quality system, these products are the most uniform and consistent nonwoven, needlepunched geotextile currently available in the industry. GSE combines a fiber selection and approval system with in-line quality control and a state-of-the-art laboratory to ensure that every roll shipped meets customer specifications. The company has performed extensive performance testing to evaluate suitability of its nonwovens for various applications. GSE Nonwoven Geotextiles are available in a range of weights to meet your specific project needs. *These product specifications meet or exceed GRI GT12, GRI GT13 and AASHTO M288.*

Product Specifications

TESTED PROPERTY	TEST METHOD	FREQUENCY	NW4	NW6	NW8	NW10	NW12	NW16
Product Code			GEO 0408002	GEO 0608002	GEO 0808002	GEO 1008002	GEO 1208002	GEO 1608002
AASHTO M288 Class			3	2	1	>1	>>1	>>>1
Mass per Unit Area, oz/yd² (g/m²)	ASTM D 5261	90,000 ft²	4 (135)	6 (200)	8 (270)	10 (335)	12 (405)	16 (540)
Thickness	ASTM D 5199	1/90,000 ft²	45 mil	70 mil	80 mil	100 mil	110 mil	155 mil
Grab Tensile Strength, lb (N)	ASTM D 4632	90,000 ft²	120 (530)	170 (755)	220 (975)	260 (1,155)	320 (1,420)	390 (1,735)
Grab Elongation, %	ASTM D 4632	90,000 ft²	50	50	50	50	50	50
Puncture Strength, lb (N)	ASTM D 4833	90,000 ft²	60 (265)	90 (395)	120 (525)	165 (725)	190 (835)	240 (1,055)
Trapezoidal Tear Strength, lb (N)	ASTM D 4533	90,000 ft²	50 (220)	70 (310)	95 (420)	100 (445)	125 (555)	150 (665)
Apparent Opening Size, Sieve No. (mm)	ASTM D 4751	540,000 ft²	70 (0.212)	70 (0.212)	80 (0.180)	100 (0.150)	100 (0.150)	100 (0.150)
Permittivity, sec⁻¹	ASTM D 4491	540,000 ft²	1.50	1.50	1.50	1.20	0.80	0.70
Permeability, cm/sec	ASTM D 4491	540,000 ft²	0.22	0.30	0.30	0.30	0.29	0.27
Water Flow Rate, gpm/ft² (l/min/m²)	ASTM D 4491	540,000 ft²	120 (4,885)	110 (4,480)	110 (4,480)	85 (3,460)	60 (2,440)	50 (2,035)
UV Resistance (% retained after 500 hours)	ASTM D 4355	per formulation	70	70	70	70	70	70
Roll Length[1], ft (m)			600 (182)	600 (182)	600 (182)	300 (91)	300 (91)	300 (91)
Roll Width[1], ft (m)			15 (4.6)	15 (4.6)	15 (4.6)	15 (4.6)	15 (4.6)	15 (4.6)
Roll Area, ft² (m²)			9,000 (836)	9,000 (836)	9,000 (836)	4,500 (418)	4,500 (418)	4,500 (418)

NOTES:

- The property values listed are in weaker principal direction. All values listed are Minimum Average Roll Values (MARV) except apparent opening size in mm and UV resistance. Apparent opening size (mm) is a Maximum Average Roll Value. UV is a typical value.
- [1]Roll lengths and widths have a tolerance of ±1%.

DS037 NW R03/15/06

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South America	GSE Lining Technology Chile S.A.	Santiago, Chile		56 2 595 4200	Fax: 56 2 595 4290
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Europe & Africa	GSE Lining Technology GmbH	Hamburg, Germany		49 40 767420	Fax: 49 40 7674234
Middle East	GSE Lining Technology-Egypt	The 6th of October City, Egypt		202 2 828 8888	Fax: 202 2 828 8889

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Attachment D
Geotextile Evaluation

Thrace-LINQ™

MEMBER THRACE GROUP

Woven Polypropylene

			Silt Fence			Woven Slit Film						
Properties MARV[2]	**ASTM TEST**	**UNIT**	**GTF 160SF**	**GTF 170SF**	**GTF 180SF/190SF**	**GTF 100**	**GTF 150**	**GTF 200S**	**GTF 200**	**GTF 250**	**GTF 270**	**GTF 300**
MECHANICAL PROPERTIES												
Grab Tensile Strength	D4632	lbs (N)	80 (356)	90 (400)	123/101 (550/450)	150/100 (668/445)	150 (668)	180 (801)	200 (890)	250 (1110)	270 (1201)	315 (1401)
Elongation	D4632	%	15	15	15	20/15	20/15	15	15	15	20/15	15
Trapezoid Tear	D4533	lbs (N)	25 (111)	30 (133)	50 (222)	40 (178)	60 (267)	70 (312)	75 (334)	90 (400)	100 (445)	120 (534)
Puncture	D4833	lbs (N)	N/A	N/A	60 (267)	60 (267)	70 (312)	80 (356)	100 (445)	100 (445)	120 (534)	150 (667)
CBR Puncture	D6241	lbs (N)	N/A	N/A	250 (1112)	350 (1558)	475 (2114)	550 (2448)	700 (3115)	900 (4005)	925 (4116)	1000 (4450)
Mullen Burst	D3786	psi (kPa)	N/A	N/A	280 (1930)	N/A	N/A	305 (2103)	400 (2757)	500 (3447)	600 (4137)	630 (4343)
ENDURANCE PROPERTIES												
UV Stability	D4355	% @ 500 hrs	80	80	80	80	80	80	80	80	80	80
HYDRAULIC PROPERTIES												
Permittivity	D4491	sec^{-1}	0.050	0.050	0.150	0.080	0.100	0.080	0.080	0.050	0.050	0.050
Water Flow Rate	D4491	gpm/ft² (l/min/m²)	N/A	N/A	10 (407)	5.6 (228)	7.4 (302)	6.0 (244)	6.0 (244)	4.0 (163)	4.0 (163)	4.0 (163)
Permeability	D4491	(cm/sec)	0.010[T]	0.010[T]	0.010[T]	0.010[T]	0.010[T]	0.010[T]	0.010[T]	0.005[T]	0.005[T]	0.005
Apparent Opening Size (AOS[3])	D4751	US Sieve Number (mm)	30 (0.600)	30 (0.600)	30 (0.600)	30 (0.600)	30 (0.600)	40 (0.425)	40 (0.425)	40 (0.425)	40 (0.425)	50 (0.300)
PHYSICAL PROPERTIES												
Weight	D5261	oz/yd² (g/m²)	1.8[T] (60[T])	2.2[T] (75[T])	3.2[T] (109[T])	2.4[T] (82[T])	3.3[T] (113[T])	4.5[T] (152[T])	5.0[T] (170[T])	5.5[T] (187[T])	5.9[T] (200[T])	6.8[T] (231[T])
Thickness	D5199	mils (mm)	N/A	N/A	16[T] (0.406[T])	N/A	N/A	18[T] (0.457[T])	20[T] (0.508[T])	21[T] (0.533[T])	26[T] (0.660[T])	28[T] (0.711[T])
PACKAGING[4,5]			**GTF 160SF**	**GTF 170SF**	**GTF 180SF/190SF**	**GTF 100**	**GTF 150**	**GTF 200S**	**GTF 200**	**GTF 250**	**GTF 270**	**GTF 300**
Roll Width		ft (m)	3 (0.914)	3 (0.914)	3 (0.914)	12.5/15/17.5 (3.81/4.57/5.33)	12.5/15/17.5 (3.81/4.57/5.33)	12.5/15/17.5 (3.81/4.57/5.33)	12.5/15/17.5 (3.81/4.57/5.33)	12.5/15/17.5 (3.81/4.57/5.33)	12.5/15/17.5 (3.81/4.57/5.33)	12.5/15/17.5 (3.81/4.57/5.33)
Roll Area		yd² (m²)	1000 (836)	1000 (836)	1000 (836)	600/800 (501/669)	600/800 (501/669)	600/800 (501/669)	600/800 (501/669)	600/800 (501/669)	600/800 (501/669)	600/800 (501/669)
Roll Weight		lbs (kg)	117 (53)	141 (64)	204 (92.6)	101/135 (46/61)	139/185 (63/84)	190/253 (86/115)	205/273 (93/139)	230/307 (104/124)	247/330 (112/150)	270/360 (123/163)

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2 - Minimum Average Roll Value. All values are MARV unless otherwise noted.
3 - AOS values are reported as Maximum Average Roll Value.
4 - Other products and roll sizes may be available.
5 - Roll lengths, areas and weights may vary.
T - Typical roll properties.

Authorized Distributor:

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Thrace-LINQ™

MEMBER THRACE GROUP

.oven Polypropylene

			Medium Strength								Monofilament		
Properties MARV[2]	ASTM TEST	UNIT	GTF 350	GTF 400	GTF 500	GTF 700	GTF 370	GTF 375N	GTF 565	GTF 570	GTF 400EO	GTF 400E	GTF 404
MECHANICAL PROPERTIES													
Grab Tensile Strength	D4632	lbs (N)	350 (1557)	400 (1780)	600/550 (2670/2450)	700 (3120)	400/250 (1780/1113)	425/350 (1891/1558)	550 (2448)	475/440 (2114/1958)	365/200 (1624/890)	370/250 (1647/1113)	400/315 (1780/1402)
Wide Width Tensile	D4595	lbs/in (kN/m)	250 (44)	333 (58)	400 (70)	517 (91)	270/225 (47.3/39.4)	270/225 (47.3/39.4)	350/400 (61.3/70.1)	400/400 (70.1/70.1)	200/140 (35/24)	225/145 (40/26)	250/230 (43.8/40.3)
Elongation	D4632	%	20/15	20/15	20/15	20/15	15/6	21/21	15/6	12/6	24/10	16/15	15/15
Trapezoid Tear	D4533	lbs (N)	120 (534)	150 (668)	230/200 (1024/890)	300/250 (1335/1112)	180/110 (757/490)	145/125 (645/556)	150/200 (668/890)	180/180 (801/801)	115/75 (512/334)	100/60 (445/267)	150/165 (668/704)
Puncture	D4833	lbs (N)	150 (668)	180 (801)	230 (1024)	300 (1335)	180 (801)	150 (668)	200 (890)	195 (868)	90 (401)	120 (534)	150 (668)
CBR Puncture	D6241	lbs (N)	1000 (4450)	1200 (5340)	1400 (6230)	1700 (7565)	N/A	N/A	N/A	N/A	675 (3004)	950 (4228)	1150 (5118)
Mullen Burst	D3786	psi (kPa)	N/A	N/A	N/A	N/A	800 (5515)	650 (4481)	1200 (8273)	1200 (8273)	450 (3102)	450 (3102)	800 (5506)
ENDURANCE PROPERTIES													
UV Stability	D4355	% @ 500 hrs	80	80	80	80	70	90	70	70	90	90	90
HYDRAULIC PROPERTIES													
...rmittivity	D4491	sec^{-1}	0.270	0.210	0.145	0.060	0.520	0.960	0.025	0.400	2.100	0.280	0.900
...ater Flow Rate	D4491	gpm/ft² (l/min/m²)	20 (815)	15.7 (640)	15 (432)	5.1 (208)	40 (1630)	70 (2852)	2 (82)	30 (1222)	145 (5908)	18 (733)	70 (2852)
Permeability	D4491	cm/sec	0.034	0.033	0.023	0.030	0.050	0.046	0.001	0.050	0.140	0.010	0.068
Apparent Opening Size (AOS[3])	D4751	US Sieve Number (mm)	35 (0.500)	40 (0.425)	80 (0.180)	80 (0.180)	30 (0.600)	40 (0.425)	40 (0.425)	30 (0.600)	40 (0.425)	70 (0.212)	40 (0.425)
Percent Open Area CW02215		%	N/A	N/A	N/A	N/A	N/A	6	N/A	N/A	10	4 to 6	1
PHYSICAL PROPERTIES													
Weight	D5261	oz/yd² (g/m²)	6.9[T] (234[T])	9.1[T] (309[T])	12.1[T] (410[T])	14.75[T] (500[T])	8.5[T] (288[T])	7.4[T] (251[T])	13.7[T] (465[T])	14.5[T] (492[T])	5.6[T] (190[T])	5.6[T] (190[T])	8.0[T] (271[T])
Thickness	D5199	mils (mm)	46[T] (1.168[T])	N/A N/A	42[T] (1.07[T])	N/A N/A	38[T] (0.965[T])	20[T] (0.508[T])	45[T] (1.143[T])	63[T] (1.160[T])	28[T] (0.711[T])	15[T] (0.381[T])	35[T] (0.889[T])
PACKAGING[4,5]			GTF 350	GTF 400	GTF 500	GTF 700	GTF 370	GTF 375N	GTF 565	GTF 570	GTF 400EO	GTF 400E	GTF 404
Roll Width		ft (m)	15/17.1 (4.57/5.21)	15/17.1 (4.57/5.21)	15/17.1 (4.57/5.21)	15/17.1 (4.57/5.21)	15 (4.57)	12.5 (3.81)	15 (4.57)	15 (4.57)	12.5 (3.81)	12 (3.66)	15 (4.57)
Roll Area		yd² (m²)	500/622 (418/520)	500/622 (418/520)	500/622 (418/520)	500/622 (418/520)	500 (418)	417 (349)	500 (418)	500 (418)	417 (349)	400 (334)	500 (418)
Roll Weight		lbs (kg)	231/287 (105/130)	304/379 (138/172)	398/490 (181/222)	481/598 (218/272)	265 (120)	210 (95)	445 (202)	475 (215)	170 (77)	155 (70)	265 (120)

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T - Typical roll properties.

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Thrace-LINQ™

MEMBER THRACE GROUP

Needlepunched Nonwoven

Civil Nonwoven

Properties MARV[2]	ASTM TEST	UNIT	120EX	125EX	130EX	140EX	145EX	150EX	160EX	180EX	225EX	245EX	250EX	275EX	350EX
MECHANICAL PROPERTIES															
Grab Tensile Strength	D4632	lbs	80	90	105	120	140	165	180	205	215	250	270	300	380
		(N)	(356)	(401)	(467)	(534)	(623)	(734)	(801)	(912)	(957)	(1112)	(1202)	(1335)	(1691)
Elongation	D4632	%	50	50	50	50	50	50	50	50	50	50	50	50	50
Trapezoid Tear	D4533	lbs	35	40	45	50	60	65	75	80	85	100	100	115	145
		(N)	(156)	(178)	(200)	(223)	(267)	(289)	(334)	(356)	(378)	(445)	(445)	(512)	(645)
Puncture	D4833	lbs	45	55	65	70	85	90	105	120	125	155	155	175	240
		(N)	(200)	(245)	(289)	(312)	(378)	(401)	(467)	(534)	(556)	(690)	(690)	(779)	(1068)
CBR Puncture	D6241	lbs	220	265	300	340	360	400	475	535	600	700	725	830	1050
		(N)	(979)	(1179)	(1335)	(1513)	(1602)	(1780)	(2114)	(2380)	(2670)	(3115)	(3226)	(3694)	(4673)
Mullen Burst	D3786	psi	160	185	210	230	280	310	330	350	360	460	460	580	750
		(kPa)	(1103)	(1275)	(1448)	(1586)	(1930)	(2137)	(2275)	(2413)	(2482)	(3171)	(3171)	(3999)	(5171)
ENDURANCE PROPERTIES															
UV Stability	D4355	% @ 500 hrs	70	70	70	70	70	70	70	70	70	70	70	70	70
HYDRAULIC PROPERTIES															
Permittivity	D4491	sec^{-1}	2.2	2.1	2.0	1.8	1.5	1.5	1.5	1.5	1.3	1.2	1.2	0.9	0.7
Water Flow Rate	D4491	gpm/ft^2	160	155	145	135	115	110	110	110	95	85	85	65	50
		($l/min/m^2$)	(6519)	(6315)	(5908)	(5501)	(4886)	(4482)	(4482)	(4482)	(3871)	(3463)	(3463)	(2648)	(2037)
Permeability	D4491	cm/sec	0.22	0.22	0.22	0.22	0.22	0.25	0.25	0.30	0.30	0.25	0.25	0.20	0.20
Apparent Opening Size (AOS[3])	D4751	US Sieve Number	70	70	70	70	70	80	100	100	100	100	100	100	100
		(mm)	(0.212)	(0.212)	(0.212)	(0.212)	(0.212)	(0.180)	(0.150)	(0.150)	(0.150)	(0.150)	(0.150)	(0.150)	(0.150)
PHYSICAL PROPERTIES															
Weight	D5261	oz/yd^2	3.2[T]	3.5[T]	4.0[T]	4.5[T]	5.5[T]	6.0[T]	7.0[T]	8.0[T]	9.0[T]	10.0[T]	10.5[T]	12.0[T]	16.0[T]
		(g/m^2)	(109[T])	(119[T])	(136[T])	(153[T])	(186[T])	(204[T])	(237[T])	(271[T])	(305[T])	(339[T])	(356[T])	(407[T])	(543[T])
Thickness	D5199	mils	45[T]	50[T]	55[T]	60[T]	70[T]	75[T]	85[T]	90[T]	95[T]	100[T]	100[T]	120[T]	150[T]
		(mm)	(1.143[T])	(1.27[T])	(1.397[T])	(1.524[T])	(1.778[T])	(1.905[T])	(2.159[T])	(2.286[T])	(2.413[T])	(2.54[T])	(2.54[T])	(3.048[T])	(3.81[T])
PACKAGING[4,5]			120EX	125EX	130EX	140EX	145EX	150EX	160EX	180EX	225EX	245EX	250EX	275EX	350EX
Roll Width		ft	12.5/15	12.5/15	12.5/15	12.5/15	12.5/15	12.5/15	12.5/15	12.5/15	12.5/15	12.5/15	12.5/15	12.5/15	12.5/15
		(m)	(3.81/4.57)	(3.81/4.57)	(3.81/4.57)	(3.81/4.57)	(3.81/4.57)	(3.81/4.57)	(3.81/4.57)	(3.81/4.57)	(3.81/4.57)	(3.81/4.57)	(3.81/4.57)	(3.81/4.57)	(3.81/4.57)
Roll Area		yd^2	500	500	500	500	500	500	500	500	500	500	500	500	500
		(m^2)	(418)	(418)	(418)	(418)	(418)	(418)	(418)	(418)	(418)	(418)	(418)	(418)	(418)
Roll Weight		lbs	125	135	150	170	208	230	260	290	315	350	365	410	530
		(kg)	(57)	(61)	(68)	(77)	(94)	(104)	(118)	(131)	(143)	(159)	(166)	(186)	(240)

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Thrace-LINQ™

MEMBER THRACE GROUP

Needlepunched Nonwoven

Paving Fabric

Properties MARV[2]	ASTM TEST	UNIT	AOL	AOM	AOH	AOE
MECHANICAL PROPERTIES						
Grab Tensile Strength	D4632	lbs (N)	90 (401)	105 (467)	120 (534)	150 (668)
Elongation	D4632	%	50	50	50	50
Trapezoid Tear	D4533	lbs (N)	40 (178)	45 (200)	50 (223)	55 (245)
Puncture	D4833	lbs (N)	55 (245)	65 (289)	70 (312)	80 (356)
CBR Puncture	D6241	lbs (N)	N/A	N/A	N/A	N/A
Mullen Burst	D3786	psi (kPa)	185 (1275)	210 (1448)	230 (1586)	290 (1999)
ENDURANCE PROPERTIES						
UV Stability	D4355	% @ 500 hrs	70	70	70	70
ASPHALT RETENTION AND MELTING POINT						
Asphalt Retention	D6140	gal/yd² (l/m²)	0.25 (1.13)	0.25 (1.13)	0.25 (1.13)	0.30 (1.36)
Melting Point	D276	F (C)	302 (150)	302 (150)	302 (150)	302 (150)
PHYSICAL PROPERTIES						
Weight	D5261	oz/yd² (g/m²)	3.5 (119)	4.1 (140)	4.6 (156)	6 (203)
Thickness	D5199	mils (mm)	50[T] (1.27[T])	55[T] (1.397[T])	60[T] (1.524[T])	75[T] (1.905[T])
PACKAGING[4,5]			AOL	AOM	AOH	AOE
Roll Width		ft (m)	10.5/12.5 (3.20/3.81)	10.5/12.5 (3.20/3.81)	10.5/12.5 (3.20/3.81)	10.5/12.5 (3.20/3.81)
Roll Area		yd² (m²)	420/500 (351/418)	420/500 (351/418)	420/500 (351/418)	420/500 (351/418)
Roll Weight		lbs (kg)	125/145 (57/66)	143/167 (65/76)	159/186 (72/84)	207/243 (94/110)

Environmental Nonwoven

Properties MARV[2]	ASTM TEST	UNIT	600EX	800EX	1000EX	1200EX	1600EX
MECHANICAL PROPERTIES							
Grab Tensile Strength	D4632	lbs (N)	165 (734)	205 (912)	270 (1202)	310 (1380)	425 (1891)
Elongation	D4632	%	50	50	50	50	50
Trapezoid Tear	D4533	lbs (N)	65 (289)	90 (401)	100 (445)	120 (534)	145 (645)
Puncture	D4833	lbs (N)	95 (423)	135 (601)	160 (712)	180 (801)	240 (1068)
CBR Puncture	D6241	lbs (N)	425 (1891)	535 (2381)	725 (3225)	830 (3694)	1200 (5340)
Mullen Burst	D3786	psi (kPa)	315 (2172)	420 (2895)	500 (3447)	650 (4481)	800 (5515)
ENDURANCE PROPERTIES							
UV Stability	D4355	% @ 500 hrs	70	70	70	70	70
HYDRAULIC PROPERTIES							
Permittivity	D4491	sec^{-1}	1.5	1.4	1.2	0.9	0.5
Water Flow Rate	D4491	gpm/ft² (l/min/m²)	110 (4482)	100 (4075)	85 (3463)	65 (2648)	40 (1630)
Permeability	D4491	cm/sec	0.25	0.25	0.25	0.20	0.20
Apparent Opening Size (AOS[3])	D4751	US Sieve Number (mm)	80 (0.180)	100 (0.150)	100 (0.150)	100 (0.150)	100 (0.150)
PHYSICAL PROPERTIES							
Weight	D5261	oz/yd² (g/m²)	6 (204)	8 (271)	10 (339)	12 (407)	16 (543)
Thickness	D5199	mils (mm)	75[T] (1.905[T])	95[T] (2.413[T])	110[T] (2.794[T])	120[T] (3.048[T])	175[T] (4.445[T])
PACKAGING[4,5]			600EX	800EX	1000EX	1200EX	1600EX
Roll Width		ft (m)	12.5/15 (3.81/4.57)	12.5/15 (3.81/4.57)	12.5/15 (3.81/4.57)	12.5/15 (3.81/4.57)	12.5/15 (3.81/4.57)
Roll Area		yd² (m²)	500 (418)	500 (418)	500 (418)	500 (418)	500 (418)
Roll Weight		lbs (kg)	250 (114)	320 (145)	400 (182)	480 (218)	572 (260)

1 - Contact your Thrace-LINQ distributor or representative for technical assistance and for products meeting AASHTO M288 specifications. Additional information can also be found at www.thracelinq.com. Methods and values may change without notice.
2 - Minimum Average Roll Value. All values are MARV unless otherwise noted.
3 - AOS values are reported as Maximum Average Roll Value.
4 - Other products and roll sizes may be available.
5 - Roll lengths, areas and weights may vary.
T - Typical roll properties.

2550 West Fifth North St. • Summerville, SC 29483 • 800.445.4675 • 843.873.5800 • Fax: 843.875.8276 • www.thracelinq.com • geo@thracelinq.com

Sales Office:
Engineered Synthetic Products, Inc.
Phone (770) 564-1857
Fax (770) 564-1818
www.espgeosynthetics.com

Geotextile Product Description Sheet

SKAPS W200

SKAPS woven geotextile fabrics are woven polypropylene materials offering optimum performance when used in stabilization applications. Produced from first quality raw materials, they provide the perfect balance of strength and separation in styles capable of functioning exceptionally well in a wide range of performance requirements. Unless indicated below, all listed properties are Minimum Average Roll Values:

PROPERTY	TEST METHOD	UNIT	M.A.R.V. (Minimum Average Roll Value)
Weight	ASTM D 5261	oz/yd^2 (g/m^2)	4.0 (136)
Grab Tensile	ASTM D 4632	lbs (kN)	200 (.889)
Grab Elongation	ASTM D 4632	%	15
Trapezoid Tear Strength	ASTM D 4533	lbs (kN)	75 (.333)
Puncture Resistance	ASTM D 4833	lbs (kN)	90 (.400)
Mullen Burst Strength	ASTM D 3786	psi (kPa)	400 (2756)
Permittivity*	ASTM D 4491	sec^{-1}	.05
Water Flow*	ASTM D 4491	gpm/ft^2 (l/min/m^2)	5 (203)
AOS*	ASTM D 4751	US Sieve (mm)	50 (.300)
UV Resistance	ASTM D 4355	%/hrs	70/500

* At the time of manufacturing. Handling, storage, and shipping may change these properties.

PACKAGING	
Roll Dimensions (W x L) – ft	12.5 x 432 / 17.5 x 309
Square Yards Per Roll	600
Estimated Roll Weight - lbs	180

This information is provided for reference purposes only and is not intended as a warranty or guarantee. SKAPS assumes no liability in connection with the use of this information.

SKAPS Industries,
316 S. Holland Dr., Pendergrass, GA 30567,
Phone: (706)-693-3440, Fax (706)-693-3450,
www.skaps.com

Made in U.S.A.

able at the Clemex Technology Incorporation (Longueil, Quebec, Canada). The apparatus consists of a Nikon microscope with a Sony RGB photographic camera and a computer for data acquisition. Additional information on this apparatus and the measuring technique can be found in publications by Clemex (1999), Forget and Goldman (1998), and Gardoni (2000).

3.2 Materials and Methodology Employed

Eight types of needle-punched nonwoven geotextiles used worldwide were employed in the test series and will be referred to hereafter as Geotextiles GA to GH (Table 1). Geotextiles GA to GE are made of continuous polyester monofilaments (Manufacturer 1). Geotextile GF (Manufacturer 2) is also a nonwoven geotextile made of polyester. Geotextiles GG and GH are made of polypropylene (Manufacturers 3 and 4). Table 1 summarizes the main characteristics of the geotextiles. The mass per unit area of the specimens varies between 130 and 600 g/m^2 and their filtration opening sizes between 60 and 500 µm. The diameter of the geotextile fibres was measured by microscopy.

Geotextile specimens for each product were randomly chosen by mapping a layer of the product and choosing the specimens using a table of random numbers. A statistical technique associating the number of specimens tested to an allowable measuring error was employed to establish the minimum number of specimens to be tested (Gardoni 2000). For each normal stress, seven specimens of each geotextile were tested.

Table 1. Characteristics of the geotextiles tested.

Geotextile	Manufacturer	Polymer	ϱ_f (kg/m^3) (1)	M_A (g/m^2) (2)	t_{GT} (mm) (3)	n (4)	k_n (cm/s) (5)	ψ (s^{-1}) (6)	O_s (µm) (7)	d_f (µm) (8)
GA	1	Polyester	1,380	140	2.2	0.94	0.40	2.1	140	27
GB	1	Polyester	1,380	200	2.3	0.93	0.40	1.9	130	27
GC	1	Polyester	1,380	300	3.4	0.93	0.40	1.5	110	27
GD	1	Polyester	1,380	400	3.7	0.92	0.40	1.1	90	27
GE	1	Polyester	1,380	600	4.6	0.90	0.40	0.9	60	27
GF	2	Polyester	1,380	222	2.3	0.92	——	——	60	26
GG	3	Polypropylene	910	300	2.8	0.88	——	1.5	150	28
GH	4	Polypropylene	910	130	1.4	0.90	0.40	2.9	200–500	37

Notes: (1) ϱ_f = density of the fibres. (2) M_A = mass per unit area (ASTM D 3776), nominal values from manufacturers' catalogues. (3) t_{GT} = geotextile thickness under 1 kPa normal stress. (4) n = geotextile porosity under 1 kPa normal stress, calculated as $n = 1 - M_A/(\varrho_f\, t_{GT})$. (5) k_n = geotextile permeability normal to its plane (AFNOR NF G 38016 standards for Geotextiles GA to GE and ASTM D 4491 for Geotextiles GF to GH). (6) ψ = geotextile permittivity (AFNOR NF G 38016 for Geotextiles GA to GE and ASTM D 4491 for Geotextiles GF to GH) (CFG 1986). (7) O_s = opening size equal to filtration opening size (AFNOR NF G 38017) for Geotextiles GA to GE and equal to apparent opening size (ASTM D 4751) for Geotextiles GF to GH (CFG 1986). (8) d_f = diameter of the geotextile fibres obtained from microscopic measurements.

Attachment G-3

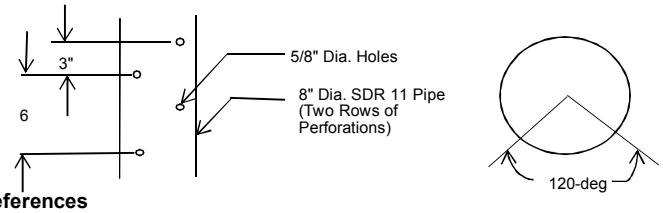
Leachate Collection Pipe Strength Evaluation

HDR Engineering, Inc.

Project	SFRL Expansion	Computed	KP	Date	2/7/2019
Subject	Leachate Collection System	Checked	TY	Date	2/26/2019
Task	Leachate Pipe Structural Strength Analysis				

Problem

Evaluate structural integrity of the leachate collection pipe (8" HDPE SDR 11).



References

PolyPipe, Inc. "Design and Engineering Guide for Polyethlene Piping", September 2008, http://www.polypipeinc.com/engineeringguide.asp (Attachment 1)
U.S. EPA, "Lining of Waste Containment and other Impoundment Facilities", September 1988, EPA/600/2-88/052 (Attachment 2)
Chevron Phillips Chemical Company, LP "Polyethylene Piping Systems Manual" (Attachment 3)

Calculations

Given

0.625	in	perforation diameter
2		perforation around diameter of pipe
4		perforations per foot
658	ft	length of pipe run
6.963	in	inside diameter of 8-inch SDR 11 HDPE pipe
8.625	in	outer diameter of 8-inch SDR 11 HDPE pipe
11		dimension ratio of pipe

Loading Conditions

The highest design loading on the leachate collection pipe will be stress from overlying waste and soils.

	Full Build-out	
	Layer Thickness (ft)	Specific Weight (lb/ft^3)
Top Soil	0.5	120
Vegetative subsoil	1.5	120
Waste & Daily Cover*	187	90
Drainage Sand	2	120

**see Attachment H-2D*

Prismatic Pressure Load	**17,265**	**psf**
Prismatic Pressure Load	**119.9**	**psi**
Average Density of Soil Cover and Waste	**90.63**	**lb/cf**

To account for the supporting system of the trench backfill material, use the modified Marston formula. The load must be increased to account for inlet holes drilled into the pipe as depicted above.

$$W = \frac{C_d \cdot \rho \cdot B_d \cdot D}{144} \quad \text{(Att. 1)} \qquad W_{design} = \frac{12}{12 - l_p} * W \quad \text{(Att. 2)}$$

where,

Value	Variable	
2.55	C_d	Trench Coefficient (dimensionless)
2	B_d	trench width at top of pipe (feet)
8.625	D	outside diameter (in)
90.63	ρ	soil density (lb/ft^3), average density of waste and soil cover material shown in above table
2.50	l_p	cumulative length of perforations per foot of pipe (in)
28	W	**loading per unit length of pipe (lbs/in)**
35	W_{design}	**design loading per unit length of pipe (lbs/in)**

Deflection Calculation using Modified Spangler Formula

$$\Delta x = \frac{D_L \cdot K \cdot W}{[\frac{2E}{3(DR-1)^3}] + 0.061E'} \quad \text{(Att. 1)}$$

HDR Engineering, Inc.

Project	SFRL Expansion		Computed	KP	Date 2/7/2019
Subject	Leachate Collection System		Checked	TY	Date 2/26/2019
Task	Leachate Pipe Structural Strength Analysis				

where,

Value	Variable	
1.0	D_L	deflection lag factor (dimensionless)
0.1	K	bedding constant (dimensionless)
35	W	load per unit length of pipe (lbs/in)
30,000	E	modulus of elasticity of the pipe material (psi)
700	E'	modulus of soil reactions (lbs/in^2)
11	DR	dimension ratio (dimensionless)
8.625	D	outside diameter (in)
0.06	Δx	horizontal deflection or change in diameter (in)
0.65%		**percent deflection, deflection/outside diameter**
2.7%		manufacturer's maximum allowable design limit for deflection (Att. 3)

Wall Buckling Calculation

$$P_{cb} = \frac{1}{SF} \cdot \left(\frac{2.67 \cdot R_w \cdot B \cdot E_s \cdot E}{DR^3}\right)^{1/2} \qquad R_w = 1 - (0.33 \cdot \frac{H_w}{H}) \qquad B = \frac{1}{1 + 4 \cdot e^{-0.065H}} \qquad \text{(Att. 1)}$$

Value	Variable	
700	E_s	soil modulus
30,000	E	pipe modulus of elasticity (psi)
11	DR	dimension ratio (dimensionless)
0.28	H_w	height of water table above pipe (ft)
191	H	height of soil/waste above pipe (ft)
1.000	B	empirical coefficient of elastic support (dimensionless)
0.9995	R_w	water buoyancy factor (dimensionless)
205	$\mathbf{P_{cb}}$	**critical buckling stress (psi)**
1.7		**factor of safety (critical buckling stress/prismatic loading)**

Therefore, 8-inch SDR 11 is structurally suitable for use as the leachate collection piping.

Attachment 1
Leachate Pipe
Structural Strength
Analysis

FOREWORD

The Design and Engineering Guide for Polyethylene Piping, hereinafter referred to as "Guide," defines engineering requirements deemed necessary for the safe design and construction of **PolyPipe®** polyethylene piping systems. While safety is a basic consideration in this Guide, the user should be aware of other safety measures to consider in the safe installation or operation of the piping system. This Guide contains prohibitions in some areas where practice can be unintentionally easily extended into an unsafe condition, and in other areas, "flags" of caution are offered.

The Guide is intended to provide basic principles in areas of concern when designing a piping system. However, it does not replace the need for competent engineering evaluation and judgment.

The Guide contains basic reference data and formulas necessary for design of a **PolyPipe®** piping system. These formulas are recognized industry standard adaptations and, in most cases, modified for ease of application to **PolyPipe®** systems. It is intended to state these in terms of basic design principles to the maximum extent possible, supplemented where necessary, with examples to obtain uniform and accurate interpretation.

PolyPipe® also has available a CD Rom to aide in calculations of formulas presented in the Sections of this Guide. To request a CD Rom or to download the PolyCalc program, please visit our website at www.polypipeinc.com.

If additional technical assistance is needed, please contact the **PolyPipe®** Technical Services Department at (800) 433-5632.

A comprehensive, industry consensus design guide for the proper use of polyethylene pipe is available from the Plastics Pipe Institute (PPI). The design guide is available, free of charge, via download from the PPI website at www.plasticpipe.org.

PIPE DATA

EARTHLOADING

PolyPipe®, due to its flexibility, will deflect when it is buried. The degree of deflection will depend upon the soil conditions, burial conditions, trench width, and the depth of burial. The degree of deflection of the pipe is limited by the soil around its periphery, especially in the lateral direction. When the soil compacts around the pipe, there is a supportive effect from the soil itself, and as compaction occurs, there is soil friction and cohesion over the pipe that reduces the direct load on the pipe.

PolyPipe®, as do other flexible conduits, depends on the surrounding soil for support, and has to be considered as one component in a pipe/soil system. The presence of the soil arch and the support derived from the lateral movement limitations are highly beneficial to the efficiency of the system. Therefore, the flexibility of **PolyPipe®** is the major reason for these advantages. As has been stated, the durability of polyethylene is the reason for its resistance to high levels of mechanical abuse, and this is no less true for buried systems where forced deflections may occur due to subsidence, washout and settlement.

External loading analysis must be conducted to determine the application's feasibility. There are two loading calculations necessary when designing or engineering below ground applications of **PolyPipe®.** These calculations are ring deflection and wall buckling. Wall crushing, calculated using the allowable compressive strength of the PE material, is usually not critical when using solid wall **PolyPipe®,** as ring deflection and wall buckling are predominant parameters.

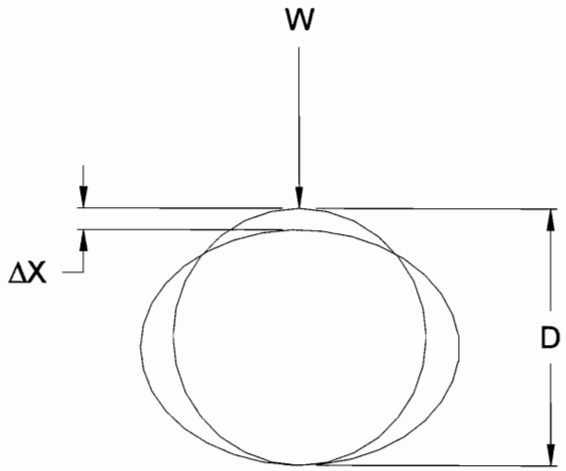
RING DEFLECTION

PolyPipe®, when buried in loose soil conditions, will exhibit the tendency to deflect, called ring deflection. Listed below are the recommended maximum allowable design limits for ring deflection of **PolyPipe®** for the different available Dimension Ratios (DR).

Table C-1
Design Limits for Ring Deflection

DR	Safe Deflection, % of Diameter
32.5	8.0
26	7.0
21	6.0
17	5.0

Figure C-1



PolyPipe®, due to its inherent physical properties of flexibility, resilience and toughness can withstand significant deflection without failure. It can be flattened without causing a fracture of the pipe wall. However, this condition is unacceptable as far as service is concerned. A deflection of 15% would be acceptable for a butt fused polyethylene system, although a reduction in flow would be noted. It would also be difficult to utilize conventional cleaning equipment with this severity of deflection. Ring deflection resulting in hydraulic flow area reductions should be taken into account when engineering the flow characteristics. Refer to Table C-2 for the percentage of area reduction based on percent of ring deflection.

Table C-2
AREA REDUCTION DUE TO RING DEFLECTION

Ring Deflection, %	Area Reduction, %
2	0.04
4	0.16
5	0.25
6	0.36
8	0.64
10	1.00
12	1.44
14	1.96
15	2.25
16	2.56

In calculating the soil load placed on a buried pipe, the designer must be able to calculate to some degree of accuracy the type and condition of the backfill material. Saturated clay would be more difficult to place and adequately compact than would coarse granular material that would not stick together. It is important in the pipe/soil system that the backfill material utilized for haunching and initial backfill (see Installation, Section F, for explanation of terminology) be granular and non-cohesive, free of debris, organic matter, frozen earth and rocks larger than 1½ inch in diameter. This material can be described as Class I or II of ASTM D2321 "Angular ¼ to 1½ inch Graded Stone, Slag, Cinders, Crushed Shells and Stone or Sands and Gravel Containing Small Percentages of Fines, Generally Granular and Non-Cohesive, Wet or Dry." This material can easily be worked into the pipe haunch, and compacted in approximately 4-6 inch lifts.

To determine the ring deflection of externally loaded **PolyPipe®**, you must first determine the earthload in pounds per linear inch of pipe by use of the following modified Marston formula[5]:

$$W = \frac{C_d \cdot \rho \cdot B_d \cdot D}{144} \qquad (17)$$

Where W = Earthload per unit length of pipe, lbs/in
C_d = Trench Coefficient, (dimensionless) (See Figure C-2)
ρ = Soil density, lbs/ft^3
D = Outside diameter, inches
B_d = Trench width at top of pipe, feet

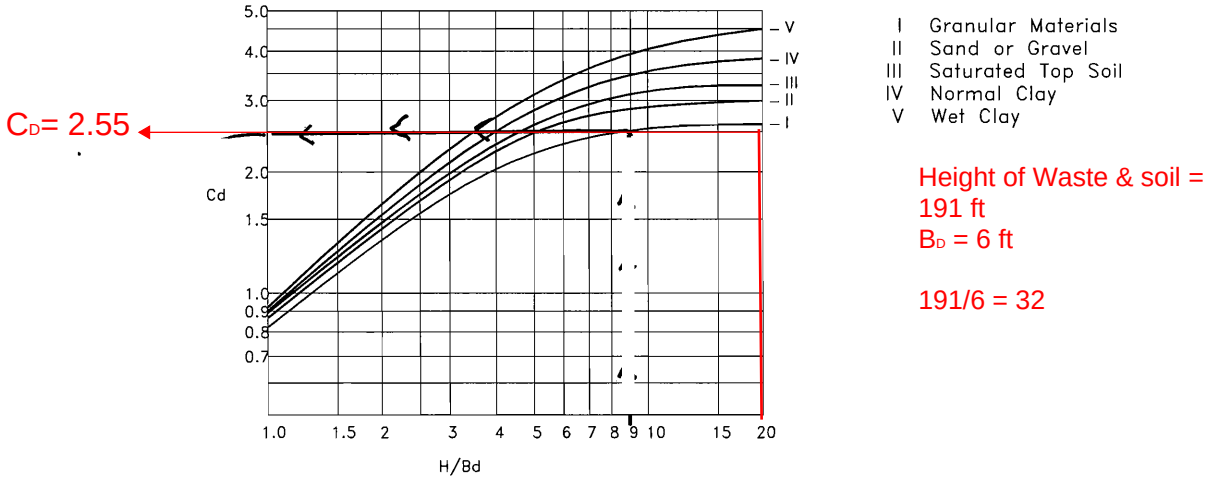
[5] Moser, A.P. Buried Pipe Design. 2nd Edition. New York: McGraw-Hill, 2001.

Table C-3
CLASSIFICATION OF BACKFILL MATERIAL
PER ASTM D2321*

Class	Comments
Class I - Angular graded stone, ¼" to 1½", including a number of fill materials that have regional significance such as coral, slag, cinders, crushed stone, crushed gravel and crushed shells.	100 - 200 pounds per cubic foot. Pipe sizes less than 10" should limit maximum particle size to ½" to ¾" for ease of placement.
Class II - Coarse sands and gravel with maximum particle size of 1½", including variously graded sands and gravel containing small percentages of fines, generally granular and non-cohesive, wet or dry.	110 - 130 pounds per cubic foot. Pipe sizes less than 10" should limit maximum particle size to ½" to ¾" inch for ease of placement.
Class III - Fine sand and clay gravel, including fine sands, sand-clay mixtures, and gravel-clay mixtures.	140 - 150 pounds per cubic foot.
Class IV - Silt, silty clays, and clays, including inorganic clays and silts of medium to high plasticity and liquid limits.	150 - 180 pounds per cubic foot.
Class V - Includes organic soils as well as soils containing frozen earth, debris, rocks larger than 1½" in diameter, and other foreign materials.	Not recommended for backfill except in the final backfill zone.

* For further classification of soils the designer may want to review ASTM D2487, "Standard Test Method for Classification of Soil for Engineering Purposes."

Figure C-2
TRENCH COEFFICIENT, C_d
DEPENDENT ON SOIL TYPE AND DITCH CONFIGURATION



In general practice, the trench width can be kept to a minimum of six inches per side greater than the pipe diameter itself. Although this may seem narrow in comparison to trenching of conventional materials, it must be noted that **PolyPipe®** can be pre-assembled above ground and later placed into the trench. The trench width should be maintained as narrow as possible as the soil loading on the pipe is a relationship of the trench width.

The linear deflection of the pipe can be calculated from the following modified Spangler equation[6]:

$$\Delta x = \frac{D_l \cdot K \cdot W}{\left(\frac{2E}{3(DR-1)^3}\right) + 0.061E'} \qquad (18)$$

Where
Δx = Horizontal deflection or change in diameter, inches
D_l = Deflection lag factor, **PolyPipe®** recommends 1.0 (dimensionless)
K = Bedding constant, **PolyPipe®** recommends 0.1 (dimensionless)
W = Earthload, lbs/inch (See Equation (17))
E = Modulus of elasticity of pipe, 30,000 psi
E' = Soil modulus, psi
DR = Dimension ratio, (dimensionless)

** For further values of K see reference.*

The percent deflection can be calculated by use of the following formula[6]:

$$d = \frac{\Delta x}{D} \cdot 100 \qquad (19)$$

Where
d = Percent deflection, %
Δx = Horizontal deflection, inches (See Equation (*18*))
D = Outside diameter, inches

Table C-4
TYPICAL SOIL MODULUS VALUES (PSI)

Type of Soil	Depth of Cover		Standard AASHTO relative compaction			
	ft	m	85%	90%	95%	100%
Fine-grained soils with less than 25% sand content (CL, ML, CL-ML)	0-5	0-1.5	500	700	1000	1500
	5-10	1.5-3.1	600	1000	1400	2000
	10-15	3.0-4.6	700	1200	1600	2300
	15-20	4.6-6.1	800	1300	1800	2600
Coarse-grained soils with fines (SM., SC)	0-5	0-1.5	600	1000	1200	1900
	5-10	1.5-3.0	900	1400	1800	2700
	10-15	3.0-4.6	1000	1500	2100	3200
	15-20	4.6-6.1	1100	1600	2400	3700
Coarse-grained soils with little or no fines (SP, SW, GP, GW)	0-5	0-1.5	700	1000	1600	2500
	5-10	1.5-3.0	1000	1500	2200	3300
	10-15	3.0-4.6	1050	1600	2400	3600
	15-20	4.6-6.1	1100	1700	2500	3800

$E' = 700$ psi

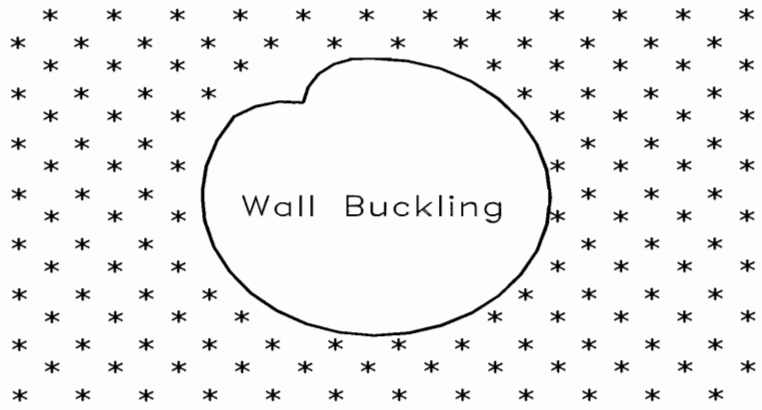
[6] Plastics Pipe Institute. Underground Installation of Polyethylene Pipe, 1996.

Values of modulus of soil reaction, E' (psi) based on depth of cover, type of soil, and relative compaction. Soil type symbols are from the United Classifications System. Source: Hartley, James D. and Duncan, James M., "E' and its Variation with Depth," Journal of Transportation, Division of ASCE, Sept. 1987.

WALL BUCKLING

PolyPipe®, when buried in dense soil conditions and subjected to excessive external loading, will exhibit the tendency of wall buckling. As seen in Figure C-3, wall buckling is a longitudinal wrinkle that usually occurs between the 10:00 and 2:00 positions. Wall buckling should become a design consideration when the total vertical load exceeds the critical buckling stress of **PolyPipe®.**

Figure C-3



Vertical loading can be determined by the summation of the calculated dead load (load resulting from backfill overburden and static surface loads) and live load (loads resulting from cars, trucks, trains, etc.).

BACKFILL LOAD[1]

$$P_b = \frac{\rho_{soil} \cdot H}{144} \qquad (20)$$

Where P_b = Backfill load, psi
ρ_{soil} = Backfill density, lbs/ft^3
H = Height of backfill above pipe, feet

SURFACE LOAD

Surface loads are those forces exerted by permanent structures in close proximity to buried **PolyPipe®.** These loads can be buildings, storage tanks, or other structures of significant weight that could add to the backfill loading. The force exerted on **PolyPipe®** by structural surface loads can be approximated by use of the following Boussinesq[17] formulation:

$$P_s = \frac{3Lz^3}{144 \cdot 2\pi R^5} \qquad (21)$$

Where P_s = Surface load on pipe, psi
L = Static surface load, lbs.
z = Vertical distance from top of pipe to surface load level, feet
R = Straight line distance from the top of pipe to surface load, feet

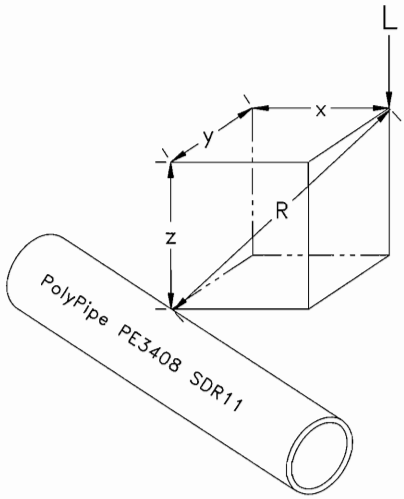
Where,

[1] Nayyar, Mohinder L. Ed. Piping Handbook. 6th Edition. New York: McGraw-Hill, Inc., 1992.
[17] Chen, W. F., Liew, Richard L. Y. The Civil Engineering Handbook. New York: CRC Press, 2003. 2nd Edition.

$$R = \sqrt{x^2 + y^2 + z^2} \qquad (22)$$

Where x = Horizontal distance from surface load, feet (Refer to Figure C-4)
y = Horizontal distance from surface load, feet (Refer to Figure C-4)
z = Vertical distance from top of pipe to surface load level, feet (Refer to Figure C-4)

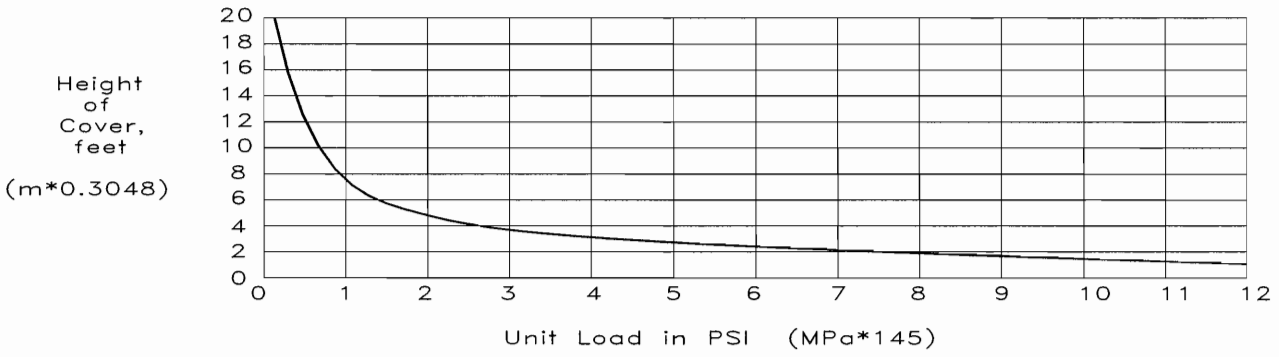
Figure C-4
RESULTANT SURFACE LOAD



LIVE LOAD

Live loading can be determined by extracting the load from Figure C-5 for H20 highway loading or from Figure C-6 for Cooper E-80 loading or by estimating, using available analytical techniques.

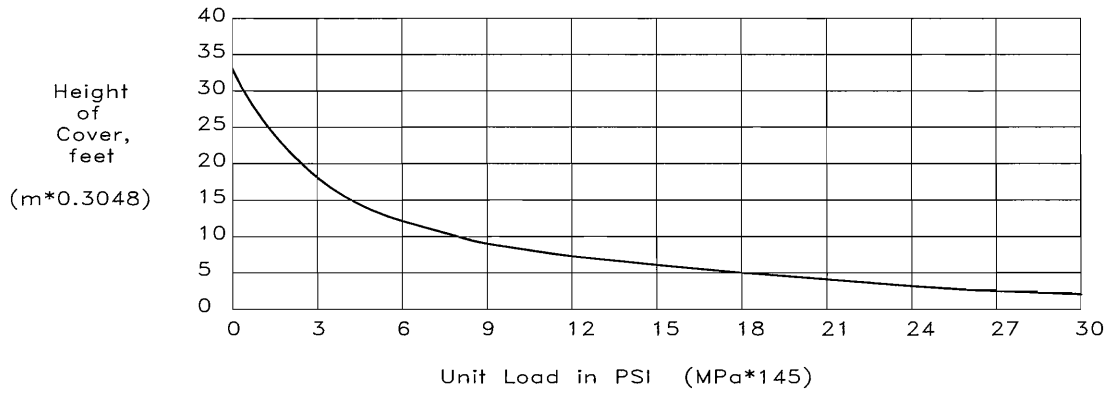
Figure C-5
H20 HIGHWAY LOADING



Note: *The H20 live load assumes two 16,000 lb. loads applied to two 18" x 20" areas, one located over the point in question, and the other located at a distance of 72" away. In this manner, a truckload of 20 tons is simulated.*

Source: American Iron and Steel Institute, Washington, DC

Figure C-6
COOPER E-80



Note: *The Cooper E-80 live load assumes 80,000 pounds applied to three 2' x 6' areas on 5' centers, such as might be encountered through live loading from a locomotive with three 80,000 pounds axle loads.*

Source: American Iron and Steel Institute, Washington, DC

TOTAL EXTERNAL LOADING

Total Load = Live Load + Backfill Load + Surface Load

$$P_t = P_l + P_b + P_s \tag{23}$$

Once the external loading on buried **PolyPipe®** has been determined, it will be necessary to calculate the critical buckling stress for contained **PolyPipe®** to determine if the pipe can withstand the external loading. The external loading capacity, or critical buckling stress, can be determined by the use of the following Von Mises formula:

$$P_{cb} = \frac{1}{SF} \cdot \left(\frac{2.67 \cdot R_w \cdot B \cdot E_s \cdot E}{DR^3} \right)^{1/2} \tag{24}$$

Where P_{cb} = Critical buckling stress, psi
SF = Safety factor, **PolyPipe®** recommends SF=2
R_w = Water buoyancy factor, (dimensionless)
B = Empirical Coefficient of Elastic Support, (dimensionless)
E_s = Soil modulus, (See Table C-4)
E = Pipe modulus of elasticity, psi
DR = Dimension Ratio

Where,

$$R_w = 1 - \left(0.33 \cdot \frac{H_w}{H} \right) \tag{25}$$

H_w = Height of water table above pipe, feet
H = Height of soil cover above pipe, feet

Note: *H_w must be less than H*

and,

$$B = \frac{1}{1 + 4 \cdot e^{-0.065 \cdot H}} \qquad (26)$$

Where e = 2.718
H = Height of soil cover above pipe, feet

If the total external loading, Equation (23), is less than the critical buckling stress ($P_t < P_{cb}$), then the application should be considered safe. However, if this is not the case ($P_t > P_{cb}$), then the required parameters can be determined for a safe application from the following variations of the above equation:

$$DR = \left(\frac{2.67 \cdot R_w \cdot B \cdot E_s \cdot E}{SF^2 \cdot P_{cb}^{\ 2}} \right) \qquad (27)$$

or

$$E_s = \frac{P_{cb}^{\ 2} \cdot SF^2 \cdot DR^3}{2.67 \cdot R_w \cdot B \cdot E} \qquad (28)$$

NOTICE:
The data contained herein is a guide to the use of **PolyPipe®** polyethylene pipe and fittings and is believed to be accurate and reliable. However, general data does not adequately cover specific applications, and its suitability in particular applications should be independently verified. In all cases, the user should assume that additional safety measures might be required in the safe installation or operation of the project. Due to the wide variation in service conditions, quality of installation, etc., no warranty or guarantee, expressed or implied, is given in conjunction with the use of this material.

excerpt from U.S. EPA's "Lining of Waste Containment and Other Impoundment Facilities"
EPA/600/2-88/052
Sept 1988

Attachment 2
Leachate Pipe
Structural
Strength
Analysis

where

W = force per unit length of pipe, and

B_c = outside diameter of pipe.

I.2.2.2 Positive Projecting Condition (Figure I-4)--

This condition is assumed to exist whenever the top of the pipe is at or above the level of the bottom of the unit. In this case, the load on the pipe can be assumed to be equal to the weight of a prism of overlying waste fill with a width B_c and height H_f plus the weight of a similar prism of gravel backfill above the pipe; because the pressure due to the gravel backfill typically will be small compared to the pressure due to the waste fill, the vertical pressure on the top of the pipe can be assumed to be equal to the unit weight of the waste fill multiplied by the distance from top of the waste fill to top of pipe, thus:

$$\sigma_v = (\omega_f)(H_f).$$

I.2.2.3 Perforated Pipe--

Perforations will reduce the effective length of pipe available to carry loads and resist deflection. The effect of perforations can be taken into account by using an increased load per nominal unit length of the pipe. If l_p equals the cumulative length in inches of perforations per foot of pipe, the increased vertical stress to be used equals:

$$(\sigma_v)_{design} = \frac{12}{12-l_p} \times (\sigma_v)_{actual}.$$

I.2.3 Deflection

A well-accepted formula for calculating flexible pipe deflection under earth loading is that developed by Spangler. This equation, also known as the Iowa formula, is presented together with suggested values for its various constants in the 1970 edition of the American Society of Civil Engineers (ASCE) Manual of Practice, No. 37, Chapter 9, Section E, Subsection 1, and is as follows:

$$\Delta y = D_e \frac{KWr^3}{EI + 0.061\, E'r^3}$$

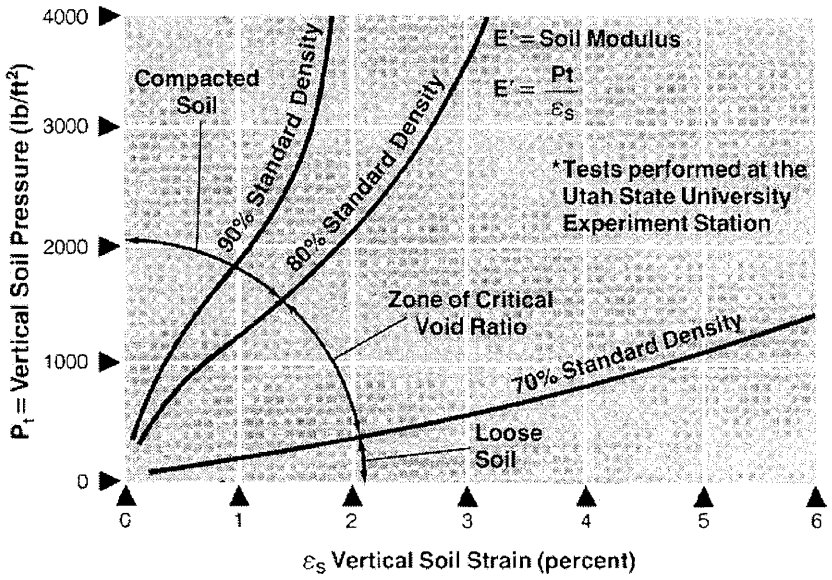
where,

Δy = horizontal and vertical deflection of the pipe (in.),

D_e = a factor, generally taken at a conservative value of 1.5, compensating for the lag or time dependent behavior of the soil/pipe systems (dimensionless),

Attachment 3
Leachate Pipe
Structural Strength
Analysis

FIGURE 7: PLOT OF VERTICAL STRESS-STRAIN DATA FOR TYPCIAL TRENCH BACKFILL (EXCEPT CLAY) FROM ACTUAL TESTS



Example:
Find: E' @ 2000 psf and 80% density
Formula: $E' = P_T/\varepsilon_S$
Calculations: E' = 2000 psf / (0.018 * 144) = 771 psi

Note: The curves shown on this chart are sample curves for a granular soil. If other types of soil are used for backfill such as clay or clay loam, curves should be developed from laboratory test data for the material used. Soil pressures greater than 4000 psf may be extrapolated with the slope of the curve or curves can be generated by testing at higher soil pressures. Probable error of curves is about half the distance between adjacent lines.

Design by Ring Deflection Ring deflection, by definition, is the ratio of the vertical change in diameter to the pipe's original diameter. It is often expressed as a percentage.

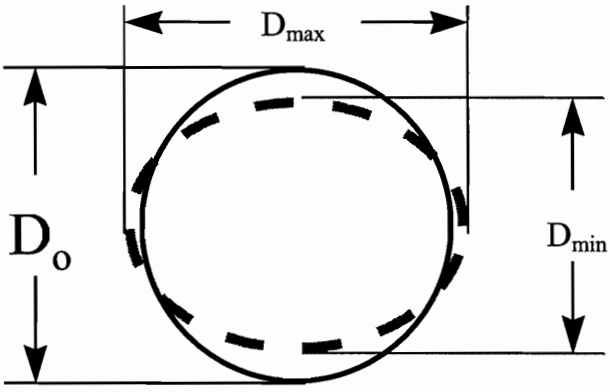
Driscopipe HDPE pipe is designed to be "flexible". This assumes the pipe will deflect the same as the vertical compression of the soil around it. Design by ring deflection matches the ability of the pipe to accommodate, without structural distress, the vertical compression of the surrounding soil. Design by ring deflection calculates the vertical soil strain and compares it to the allowable ring deflection of the pipe.

TABLE 15: ALLOWABLE RING DEFLECTION OF DRISCOPIPE® POLYETHYLENE PIPE BASED UPON DR

DR	Allowable Ring Deflection
32.5	8.1%
26	6.5%
21	5.2%
19	4.7%
17	4.2%
15.5	3.9%
13.5	3.4%
11	2.7%

The allowable ring deflection of polyethylene pipe is limited to create no more than 1 to 1.5% tangential strain in the outer surface of the pipe wall. As the wall of a pipe becomes thicker (a "lower" DR value), the distance from the neutral axis to the outer surface increases. As a result, less deflection is required to create the allowable tangential strain. Deflection of the pipe-soil system is controlled by proper specification of the backfill compaction.

FIGURE 8: CALCULATING RING DEFLECTION



$$\%RingDeflection = \left(1 - \frac{D_{min}}{D_o}\right) \times 100\%$$

The percentage ring deflection based upon strain for a given DR pipe can be calculated as follows:

$$\frac{\Delta Y}{D} = (0.25)(\varepsilon)(SDR)$$

Attachment G-4

Leachate Collection Pipe Capacity Evaluation

Job No.	10150537	No.	

HDR Engineering, Inc.

Project	SFRL Cells 6 and 7 Expansion	Computed	KP	Date	2/7/2019
Subject	Leachate Collection System	Checked	TY	Date	
Task	Leachate Pipe Capacity				

Problem
Evaluate the flow capacity of the leachate collection pipe.

Calculations
Given:

0.625	in	perforation diameter
2		perforation around diameter of pipe
25	%	reduction in perforation diameter due to clogging
0.469	in	revised perforation diameter
4		perforations per foot
85	ft	length of pipe run (The header between Cells 6A and 6B)
6.963	in	inside diameter of 8-inch SDR 11 HDPE pipe
1,774	cf/ac/day	peak leachate collection layer flow (see Attachment B-3; HELP Model Scenario #1)
14.8	ac	contributing area of flow conservatively assuming entire Cell 6 footprint is filled with 10 foot of MSW
26,255	**cf/day**	**peak leachate collection layer flow**

Determine the Capacity through the Slotted Perforations

$$Q_{perf} = C_d \times A \times (2gh)^{0.5}$$ using Orifice Flow Equation

where:

Qperf = flow through perforations (cfs)
A = cross sectional area through one perforation
C_d = orifice coefficient = 0.6 (see Attachment 1)
g = acceleration due to gravity = 32.2 ft/s^2
h = head of water above the orifice = 1 in = 0.0833 ft

A	0.172	in^2
	0.00119	ft^2
Q	1.66E-03	cfs per perforation
	6.64E-03	cfs per foot of pipe
	0.56	cfs per entire length of pipe
	48,761	**cfd day per entire length of pipe**

Determine the Capacity of the Pipe

$$Q_p = \frac{1.49}{n} AR^{\frac{2}{3}} S^{\frac{1}{2}}$$ Manning's Equation

where:

Qp = pipe capacity (cfs)
n = Manning's roughness coefficient (dimensionless)
A = cross section area (sq. feet)
R = hydraulic radius = area/wetted perimeter (ft)
S = slope of pipe (ft/ft)

n	0.012		HDPE pipe
A	38.06	in^2	3.14 *(inner diameter^2)/4
	0.264	ft^2	
R	0.145	ft	assume full flow
S	0.0050		slope of pipe post-settlement
Q_p	0.64	ft^3 / sec	
	55,353	**ft^3 / day**	

Therefore, the limiting factor is orifice flow. Accordingly, the calculated factor of safety is 1.9 (flow capacity through perforations/peak day leachate collection) which means a 8-inch slotted pipe is sufficient.

Attachment G-5

Head Over Perforations in the Leachate Collection Header

Prepared By : KP **Date: 2-27-2019**

Checked By : TY **Date: 2-28-2019**

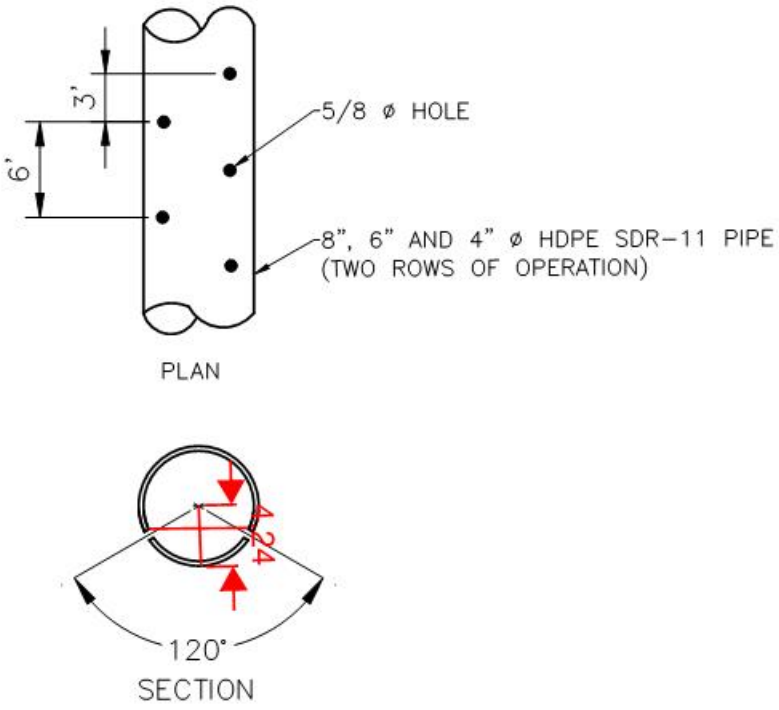
Head Over Perforations in the Leachate Collection Header

Longest flow path for the Header = 171 ft

Peak impingement rate (Attachment B-3 HELP Model Scenario 1) = 0.49 in/day

Therefore, total maximum flow per foot length of the pipe =(0.49/12)*x1x171x7.48 gal/ft/day

= 52 gal/ft/day = 0.04 gpm/ft



Proposed Perforation Pattern

As shown in the above perforation detail, approximate head above the perforations when the leachate head is 4.24-inches is 2.12 inches. Use 1-inch head over perforations conservatively.

At 1-inch head over perforations, flow per orifice (ref Pipe Capacity Calcs) = 0.00166 cfs

= 0.74 gpm

In the select perforation pattern, 4 perforation per foot.

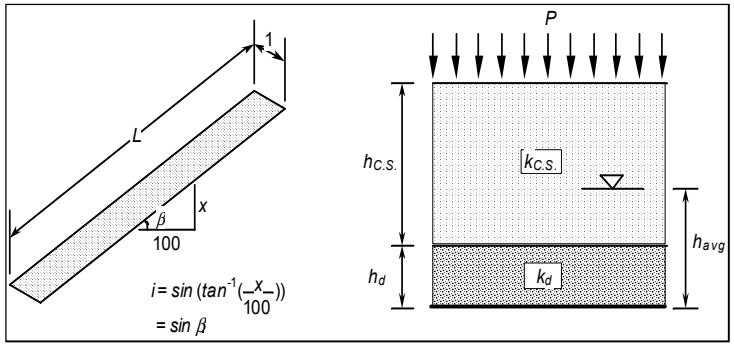
Available orifice flow capacity per foot = 2.98 gpm/ft > 0.04 gpm/ft

FS = 74; OK

Attachment G-6

Bottom Liner Veneer Stability

Stability of Protective Cover During a 25-year Rainfall Event



L =	25.9	m	i = *0.3132*		
β =	18.2	°	$L(\cos\beta)$ = *24.60*	m	
			x = *8.11*	m	
$h_{C.S.}$ =	608	mm	$h_{C.S.}$ = *0.6*	m	
h_d or t_{GS} =	6.9	mm	h_d or t_{GS} = *0.01*	m	
			$h_{C.S.} + h_d$ = *0.61*	m	
$k_{C.S.}$ =	1.0E-03	cm/s	$k_{C.S.}$ = *1.0E-05*	m/s	
k_d or k_{GS} =	5.7E+00	cm/s	k_d or k_{GS} = *5.7E-02*	m/s	
P =	307.34	mm/hr	P (RC) = *30.7*	mm/hr	
RC =	0.1		*Actual runoff* = *271.34*	mm/hr	
			PERC = *36.00*	mm/hr	
			$FLUX_{actual}$ = *0.886*	m^3/hr	
			$FLUX_{allow}$ = *0.441*	m^3/hr	***DLC = 0.4977***

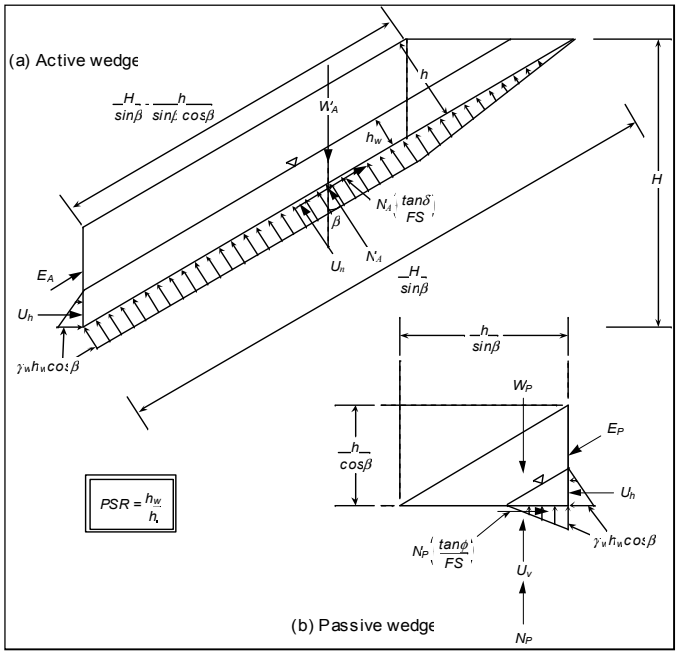
DLC =	***0.498***
PSR	***1.000***
FS	***0.675***

* Note: if only one soil layer above GM, treat it as the drainage layer.

q = *2.5E-04* m^3/sec

h_{avg} = *39.47* m

PSR = 1.000



Calculation of FS

Active Wedge:

W_A =	*298.16*	*kN*
U_n =	*142.453*	*kN*
U_h =	*1.85434*	*kN*
N_A =	*140.129*	*kN*

Passive Wedge:

W_P =	*12.3936*	*kN*
U_V =	*5.62349*	*kN*

$$FS = \frac{-b + \sqrt{b^2 - 4ac}}{2a}$$

where a = *88.9*, b = *-75.9*, c = *10.8*

FS= 0.675

thickness of cover soil = h = *0.61* m

length of slope measured along the geomembrane = L = *26* m

soil slope angle beneath the geomembrane = β = *18.2* ° = *0.32* (rad.)

vertical height of the slope measured from the toe = H = *8.1* m

parallel submergence ratio = PSR = *1.00*

depth of the water surface measured from the geomembrane = h_w = *0.61* m

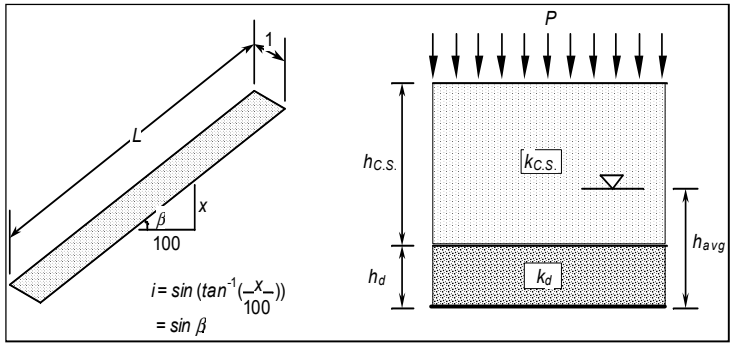
dry unit weight of the protective cover = γ_{dry} =	15.6	kN/m^3	
saturated unit weight of protective cover = $\gamma_{sat'd}$ =	19.5	kN/m^3	
unit weight of water = γ_w =	9.81	kN/m^3	
friction angle of protective cover = ϕ =	30.0	°	= *0.52* (rad.)
interface friction angle between geotextile and geomembrane = δ =	23.0	°	= *0.40* (rad.)

Note: numbers in boxes are input values

numbers in Italics are calculated values

Constructed by Te-Yang Soong

Stability of Protective Cover During a Nominal Rainfall Event



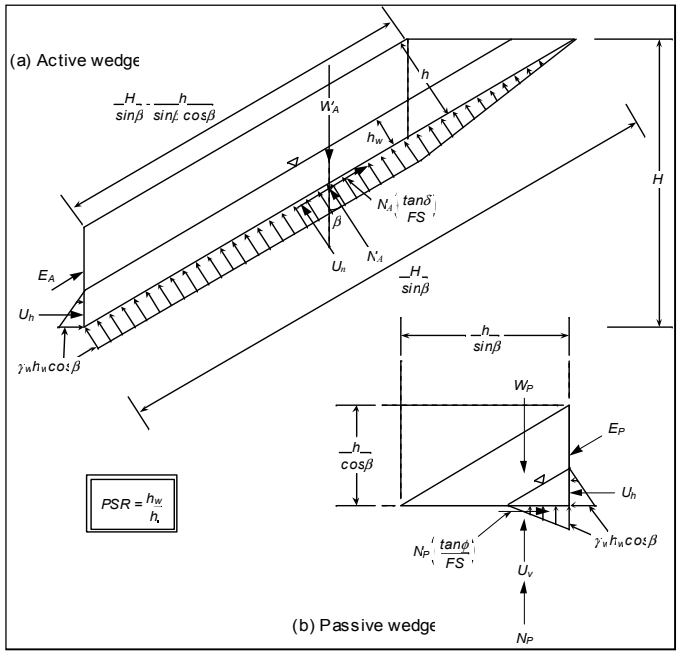
L =	25.9	m	i =	0.3132	
β =	18.2	°	$L(\cos\beta)$ =	24.60	m
			x =	8.11	m
$h_{C.S.}$ =	608	mm	$h_{C.S.}$ =	0.6	m
h_d or t_{GS} =	6.9	mm	h_d or t_{GS} =	0.01	m
			$h_{C.S.} + h_d$ =	0.61	m
$k_{C.S.}$ =	1.0E-03	cm/s	$k_{C.S.}$ =	1.0E-05	m/s
k_d or k_{GS} =	5.7E+00	cm/s	k_d or k_{GS} =	5.7E-02	m/s
P =	19.05	mm/hr	$P(RC)$ =	1.9	mm/hr
RC =	0.1		Actual runoff =	1.91	mm/hr
			PERC =	17.15	mm/hr
			$FLUX_{actual}$ =	0.422	m^3/hr
			$FLUX_{allow}$ =	0.441	m^3/hr
			q =	1.2E-04	m^3/sec
			h_{avg} =	0.01	m

DLC =	**1.045**
PSR	**0.011**
FS	**1.347**

DLC = 1.0449

PSR = 0.011

* Note: if only one soil layer above GM, treat it as the drainage layer.



Calculation of FS

Active Wedge:

W_A =	239.191	kN
U_n =	1.58308	kN
U_h =	0.00021	kN
N_A =	225.576	kN

Passive Wedge:

W_P =	9.9152	kN
U_V =	0.00064	kN

$$FS = \frac{-b + \sqrt{b^2 - 4ac}}{2a}$$

where a = 71.1

b = -108.7

c = 17.3

FS= 1.347

thickness of cover soil = h = 0.61 m

length of slope measured along the geomembrane = L = 26 m

soil slope angle beneath the geomembrane = β = 18.2 ° = 0.32 (rad.)

vertical height of the slope measured from the toe = H = 8.1 m

parallel submergence ratio = PSR = 0.01

depth of the water surface measured from the geomembrane = h_w = 0.01 m

dry unit weight of the protective cover = γ_{dry} =	15.6	kN/m³	
saturated unit weight of protective cover = $\gamma_{sat'd}$ =	19.5	kN/m³	
unit weight of water = γ_w =	9.81	kN/m³	
friction angle of protective cover = ϕ =	30.0	°	= 0.52 (rad.)
interface friction angle between geotextile and geomembrane = δ =	23.0	°	= 0.40 (rad.)

Note: numbers in boxes are input values

numbers in Italics are calculated values

Constructed by Te-Yang Soong

Attachment G-7

Geocomposite Drainage Layer

HDR Engineering, Inc.

Job No.				No.	
Project	Steelfield Road Landfill Expansion	Computed	K. Perera	Date	2/4/2019
Task	Hydraulic Conductivity and Drainage Layer Capacity	Checked	TY	Date	2/27/2019

Problem

Determine the hydraulic conductivity and drainage capacity of the geocomposite drainage layer in the bottom liner system.

Transmissivity Calculations

Landfill section from bottom to top: GCL-60-mil HDPE-Double-sided Geocomposite- 60-mil HDPE - Double-sided Geocomposite - MSW

Determine 100-hour transmissivity (Θ_{100}).

Use GSE 300-mil Permanet UL or equivalent (see attached cut sheet with product specifications). The allowable transmissivity is determined based on GRI-GC-8 as below.

$$\Theta_{allow} = \frac{\Theta_{100}}{RF_{CR} * RF_{CC} * RF_{BC}}$$

$$\Theta_{allow} = \Theta_{req} \cdot FS_D$$

$$\Theta_{req} = k \cdot b$$

where,

Θ_{allow}	= minimum allowable transmissivity of geocomposite (cm^2/s)
Θ_{req}	= required transmissivity for site (cm^2/s)
FS_D	= overall factor of safety for drainage
Θ_{100}	= transmissivity after 100 hrs. under expected load (cm^2/s)
RF_{CR}	= reduction factor for creep deformation
RF_{CC}	= reduction factor for chemical clogging
RF_{BC}	= reduction factor for biological clogging
k	= saturated hydraulic conductivity (cm/s)
b	= thickness of geonet (cm)

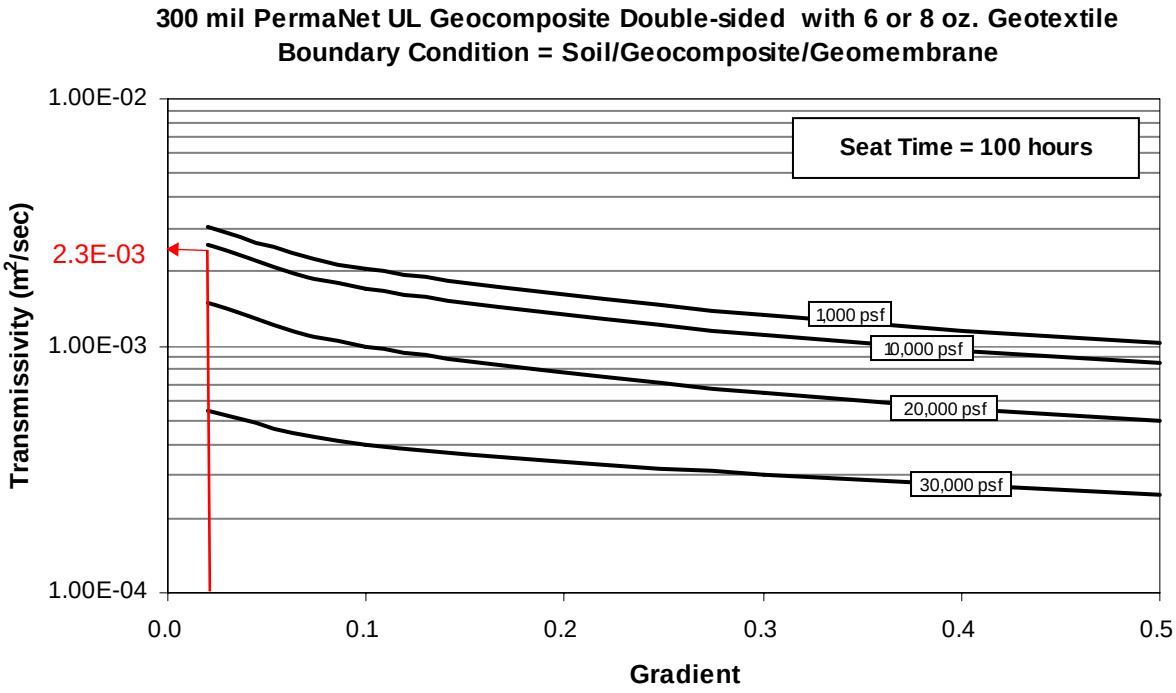
Information based on product data for GSE Permanet 300mil.

Maximum Height of MSW	60.00	ft
Density of MSW, Daily Cover and Intermediate Cover	90	pcf
Combined Final Cover and Protective Cover Thickness	4.00	ft
Density of Cover Soil and Protective Cover	120.00	pcf
Overburden Pressure (with cover soil)	5880.00	psf
Overburden Pressure (without cover soil)	5640.00	psf

Parameters	Geocomposite (300-mil) @ 10,000 psf (1)
Slope	0.02
FS_D (2)	2.0
RF_{CR} (2)	1.10
RF_{CC} (2)	2.00
RF_{BC} (2)	1.2
Θ_{100} (m^2/s)	0.0023
Θ_{allow} (cm^2/s)	8.71
Θ_{req} (cm^2/s)	4.36
Manufactured Thickness (mils)	300
Manufactured Thickness (cm)	0.76
Design Hydraulic Conductivity (cm/s)	5.7
Retained Final Thickness (mils)	270.0

Notes:

1. psf = pounds per square foot.
2. Reduction factors based on HDR experience and information provided by GSE.



A-17 Performance Transmissivity of PermaNet UL Geocomposite under Soil

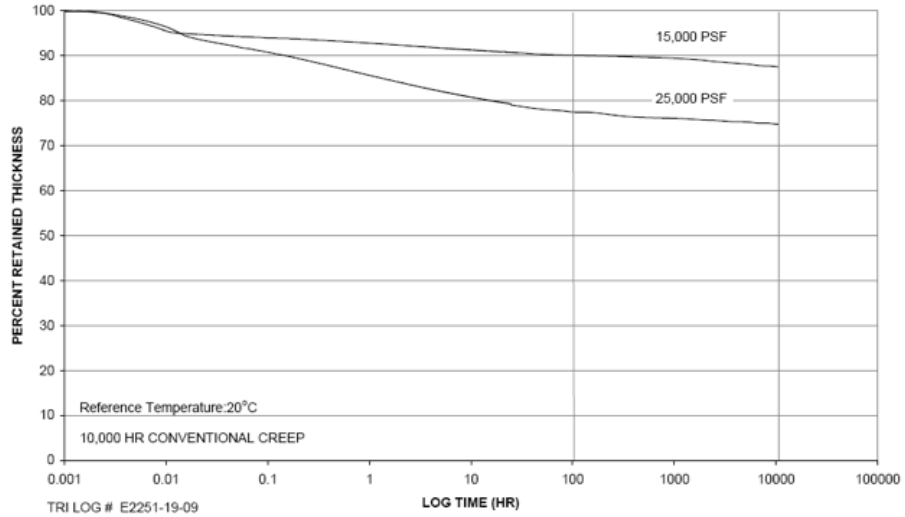


Figure 6 Creep Curves for PermaNet HL Geonet.

Table 6 Creep Reduction Factors for GSE PermaNet HL Geonet from 100 hours to 50 Years.

Stress (psf)	Creep Reduction Factor
15,000	1.12
25,000	1.16

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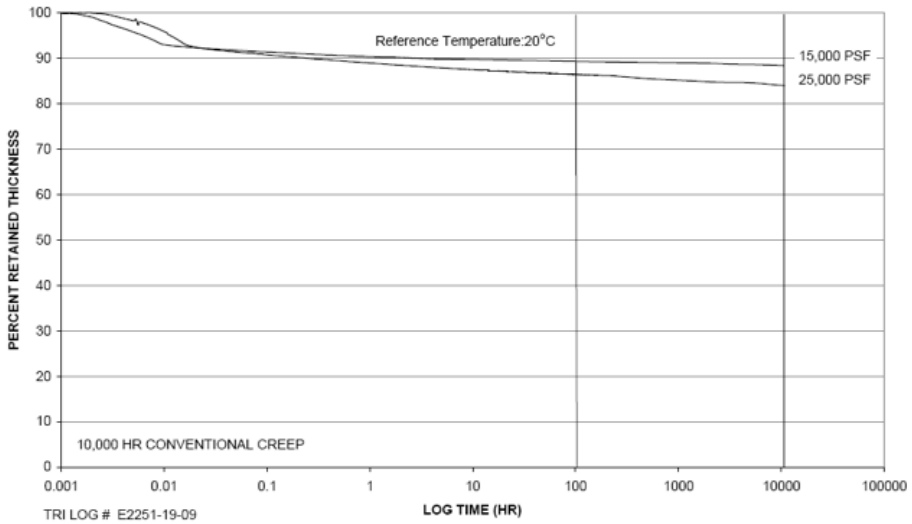

Figure 7 Creep Curves for PermaNet UL Geonet.

Table 7 Creep Reduction Factors for PermaNet UL from 100 hours to 50 Years.

Stress (psf)	Creep Reduction Factor
15,000	1.10
25,000	1.14

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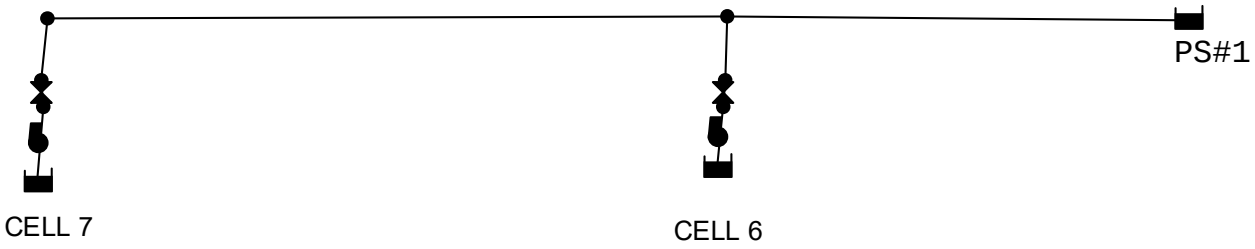
Attachment G-8

Pump Design

Schematic Diagram for Cells 6 and 7 Pump Design



Both Cells 6 and 7 Pumps are Working at the Same Time Option


```
******************************************************************
*                          E P A N E T                           *
*                  Hydraulic and Water Quality                   *
*                  Analysis for Pipe Networks                    *
*                        Version 2.0                             *
******************************************************************

Input File: pipenetwork.net


Link - Node Table:
------------------------------------------------------------------
Link            Start           End                 Length  Diameter
ID              Node            Node                    ft        in
------------------------------------------------------------------
PI1             JU6             JU2                     57         3
PI2             JU2             JU4                   1204         6
PI3             JU7             JU4                     83         3
PI4             JU4             RE3                    230         6
PU1             RE2             JU1                    #N/A      #N/A Pump
PU2             RE1             JU3                    #N/A      #N/A Pump
1               JU1             JU6                    #N/A         3 Valve
2               JU3             JU7                    #N/A         3 Valve

Energy Usage:
------------------------------------------------------------------
                  Usage   Avg.     Kw-hr      Avg.      Peak      Cost
Pump            Factor Effic.     /Mgal        Kw        Kw      /day
------------------------------------------------------------------
PU1             100.00  75.00    147.86      1.03      1.03      0.00
PU2             100.00  75.00    174.11      1.84      1.84      0.00
------------------------------------------------------------------
                                           Demand Charge:      0.00
                                           Total Cost:         0.00

Node Results:
------------------------------------------------------------------
Node                Demand      Head  Pressure   Quality
ID                     GPM        ft       psi
------------------------------------------------------------------
JU1                   0.00     81.30      7.50      0.00
JU2                   0.00     77.55      5.87      0.00
JU3                   0.00     86.56      6.31      0.00
JU4                   0.00     74.84      1.23      0.00
JU6                   0.00     81.30      7.50      0.00
JU7                   0.00     86.56      6.31      0.00
RE1                -175.92     45.00      0.00      0.00 Reservoir
RE2                -116.45     46.00      0.00      0.00 Reservoir
RE3                 292.37     72.00      0.00      0.00 Reservoir
```

♀


```
Link Results:
------------------------------------------------------------------
Link                Flow  VelocityUnit Headloss    Status
ID                   GPM       fps    ft/Kft
------------------------------------------------------------------
PI1                116.45      5.29     65.78      Open
PI2                116.45      1.32      2.25      Open
PI3                175.92      7.98    141.22      Open
```

PI4	292.37	3.32	12.36	Open
PU1	116.45	0.00	-35.30	Open Pump
PU2	175.92	0.00	-41.56	Open Pump
1	116.45	5.29	0.00	Open Valve
2	175.92	7.98	0.00	Open Valve

Only Cell 6 Pump Working Option

pipenetwork-2-cell 7 closed.rpt


```
******************************************************************************
*                            E P A N E T                                     *
*                     Hydraulic and Water Quality                            *
*                     Analysis for Pipe Networks                             *
*                          Version 2.0                                       *
******************************************************************************

Input File: pipenetwork.net


Link - Node Table:
------------------------------------------------------------------------------
Link            Start           End                  Length  Diameter
ID              Node            Node                     ft        in
------------------------------------------------------------------------------
PI1             JU6             JU2                      57         3
PI2             JU2             JU4                    1204         6
PI3             JU7             JU4                      83         3
PI4             JU4             RE3                     230         6
PU1             RE2             JU1                    #N/A      #N/A Pump
PU2             RE1             JU3                    #N/A      #N/A Pump
1               JU1             JU6                    #N/A         3 Valve
2               JU3             JU7                    #N/A         3 Valve

Energy Usage:
------------------------------------------------------------------------------
                  Usage   Avg.     Kw-hr      Avg.      Peak      Cost
Pump             Factor Effic.     /Mgal        Kw        Kw      /day
------------------------------------------------------------------------------
PU1                0.00   0.00      0.00      0.00      0.00      0.00
PU2              100.00  75.00    170.86      1.87      1.87      0.00
------------------------------------------------------------------------------
                                                Demand Charge:      0.00
                                                Total Cost:         0.00

Node Results:
------------------------------------------------------------------------------
Node                Demand       Head  Pressure   Quality
ID                     GPM         ft       psi
------------------------------------------------------------------------------
JU1                    0.00      73.19      3.98      0.00
JU2                    0.00      73.19      3.98      0.00
JU3                    0.00      85.79      5.97      0.00
JU4                    0.00      73.19      0.52      0.00
JU6                    0.00      73.19      3.98      0.00
JU7                    0.00      85.79      5.97      0.00
RE1                 -182.90      45.00      0.00      0.00 Reservoir
RE2                    0.00      46.00      0.00      0.00 Reservoir
RE3                  182.90      72.00      0.00      0.00 Reservoir
```

♀


```
Link Results:
------------------------------------------------------------------------------
Link                 Flow  VelocityUnit Headloss    Status
ID                    GPM       fps    ft/Kft
------------------------------------------------------------------------------
PI1                    0.00      0.00      0.00      Open
PI2                    0.00      0.00      0.00      Open
PI3                  182.90      8.30    151.76      Open
```

PI4	182.90	2.08	5.19	Open	
PU1	0.00	0.00	0.00	Closed	Pump
PU2	182.90	0.00	-40.79	Open	Pump
1	0.00	0.00	0.00	Open	Valve
2	182.90	8.30	0.00	Open	Valve

Checking for Detection Pump Operation


```
******************************************************************
*                          E P A N E T                           *
*                  Hydraulic and Water Quality                   *
*                  Analysis for Pipe Networks                    *
*                        Version 2.0                             *
******************************************************************

Input File: pipenetwork.net


Link - Node Table:
------------------------------------------------------------------
Link            Start           End                  Length  Diameter
ID              Node            Node                     ft        in
------------------------------------------------------------------
PI1             JU6             JU2                      57         3
PI2             JU2             JU4                    1204         6
PI3             JU7             JU4                      83         3
PI4             JU4             RE3                     230         6
PU1             RE2             JU1                    #N/A      #N/A Pump
PU2             RE1             JU3                    #N/A      #N/A Pump
1               JU1             JU6                    #N/A         3 Valve
2               JU3             JU7                    #N/A         3 Valve

Energy Usage:
------------------------------------------------------------------
                  Usage   Avg.     Kw-hr      Avg.      Peak      Cost
Pump            Factor Effic.     /Mgal        Kw        Kw      /day
------------------------------------------------------------------
PU1             100.00  75.00    121.96      0.36      0.36      0.00
PU2             100.00  75.00    172.08      1.86      1.86      0.00
------------------------------------------------------------------
                                           Demand Charge:      0.00
                                           Total Cost:         0.00

Node Results:
------------------------------------------------------------------
Node                Demand      Head  Pressure   Quality
ID                     GPM        ft       psi
------------------------------------------------------------------
JU1                   0.00     75.12      4.82      0.00
JU2                   0.00     74.36      4.49      0.00
JU3                   0.00     86.08      6.10      0.00
JU4                   0.00     73.81      0.79      0.00
JU6                   0.00     75.12      4.82      0.00
JU7                   0.00     86.08      6.10      0.00
RE1                -180.29     45.00      0.00      0.00 Reservoir
RE2                 -49.06     46.00      0.00      0.00 Reservoir
RE3                 229.35     72.00      0.00      0.00 Reservoir

♀
Page 2
Link Results:
------------------------------------------------------------------
Link                 Flow  VelocityUnit Headloss    Status
ID                    GPM       fps    ft/Kft
------------------------------------------------------------------
PI1                 49.06      2.23     13.27      Open
PI2                 49.06      0.56      0.45      Open
PI3                180.29      8.18    147.78      Open
```

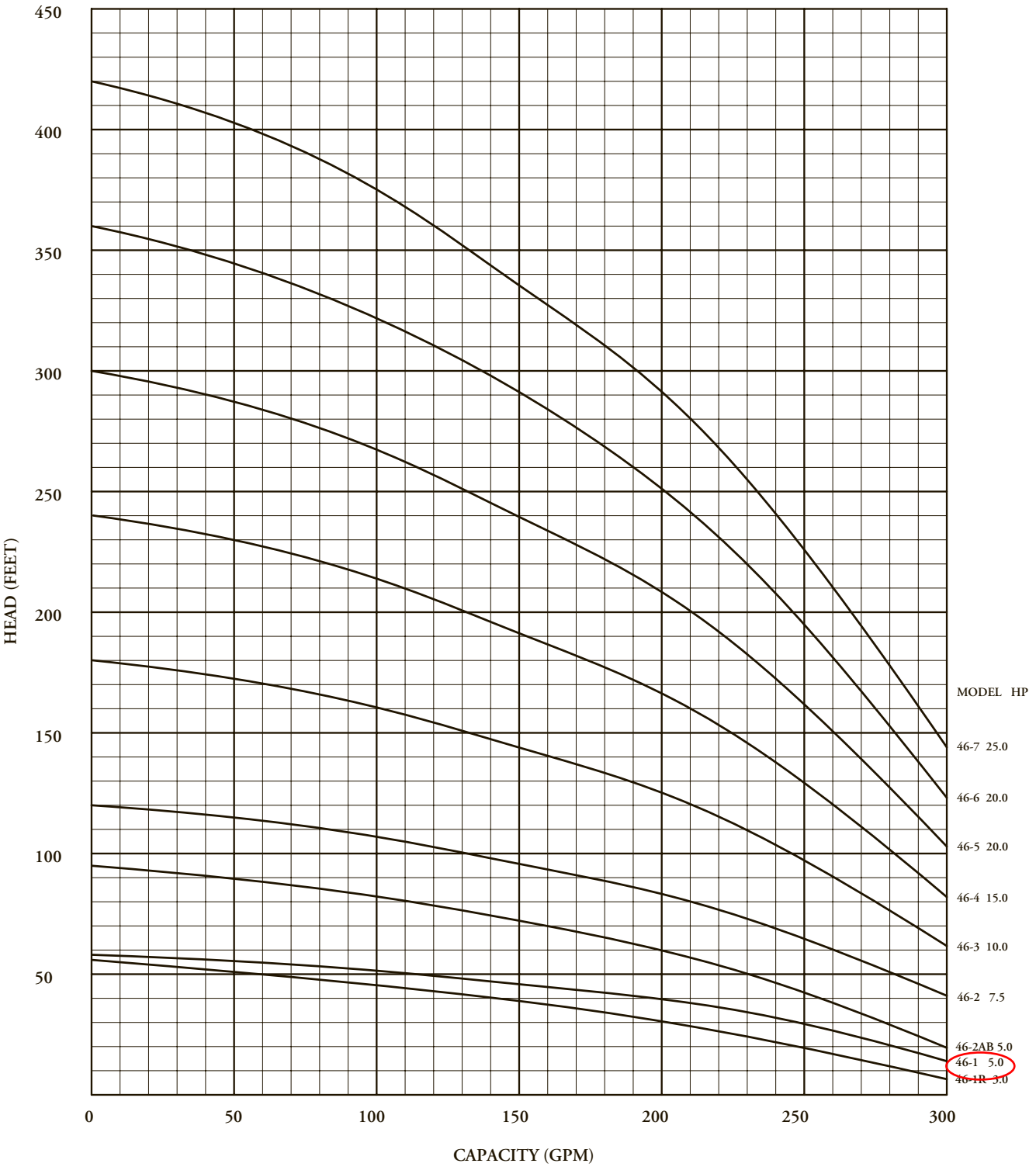
PI4	229.35	2.60	7.89	Open
PU1	49.06	0.00	-29.12	Open Pump
PU2	180.29	0.00	-41.08	Open Pump
1	49.06	2.23	0.00	Open Valve
2	180.29	8.18	0.00	Open Valve

SERIES 46 SurePump™

Flow Range 75-300 GPM

60 Hz

Next
Back



DATA SUBJECT TO CHANGE WITHOUT NOTICE

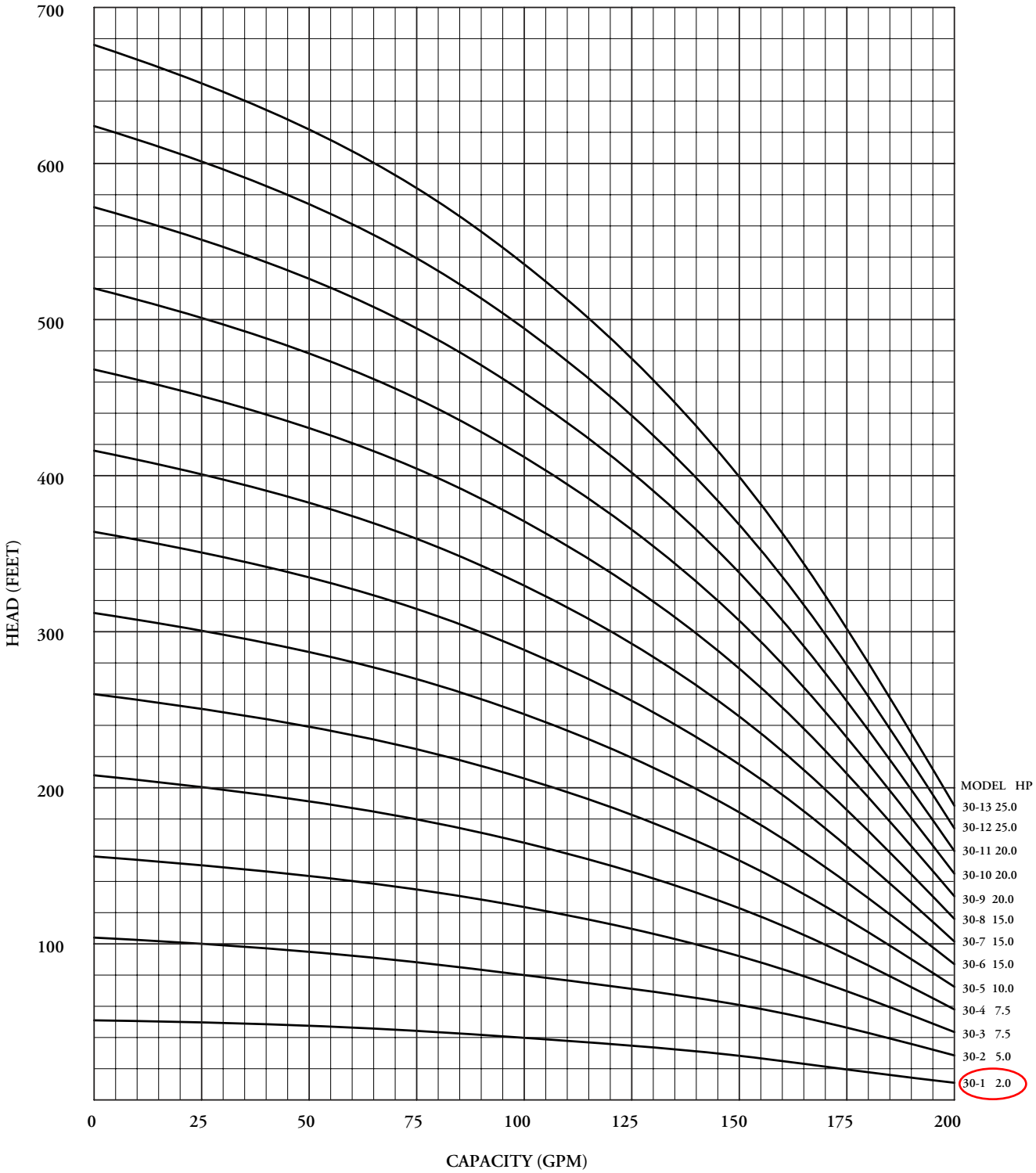
© 2003 EPG Companies Inc.
SurePump is a Reg. TM of EPG Companies Inc.

SERIES 30 SurePump™

Flow Range 50-200 GPM

60 Hz

Next
Back



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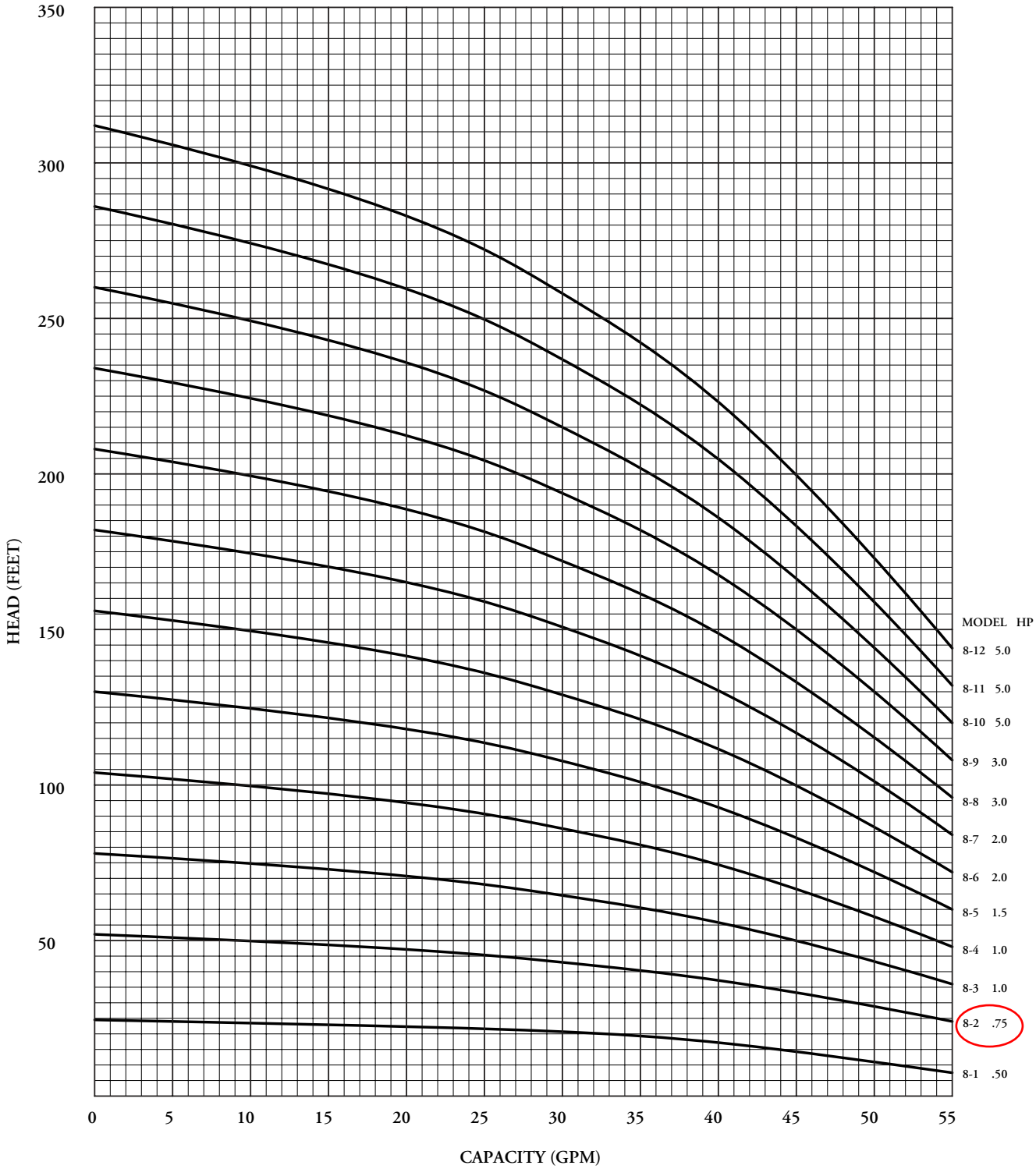
SurePump is a Reg. TM of EPG Companies Inc.

SERIES 8 SurePump™

Flow Range 20-50 GPM

60 Hz

Next
Back



DATA SUBJECT TO CHANGE WITHOUT NOTICE

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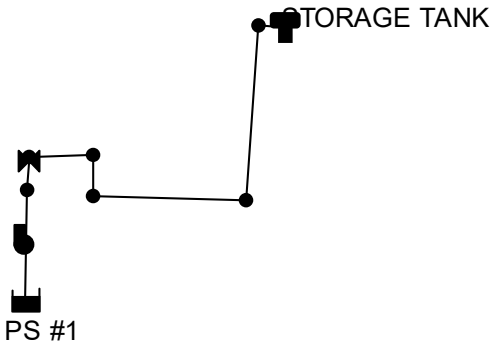
Schematic for PS#1 Operation

Day 1



```
******************************************************************
*                          E P A N E T                           *
*                   Hydraulic and Water Quality                  *
*                   Analysis for Pipe Networks                   *
*                         Version 2.0                            *
******************************************************************

Input File: ps-1-network.net


Link - Node Table:
------------------------------------------------------------------
Link            Start           End                  Length  Diameter
ID              Node            Node                     ft        in
------------------------------------------------------------------
PI1             JU2             JU3                       5         6
PI2             JU3             JU4                     5.3         6
PI3             JU4             JU5                      50         6
PI4             JU5             JU6                   79.18         6
PI5             JU6             TA1                      20         6
PU1             RE1             JU1                    #N/A      #N/A Pump
1               JU1             JU2                    #N/A         3 Valve

Energy Usage:
------------------------------------------------------------------
                   Usage   Avg.     Kw-hr      Avg.      Peak      Cost
Pump              Factor Effic.     /Mgal        Kw        Kw      /day
------------------------------------------------------------------
PU1               100.00  75.00    352.47     10.18     10.18      0.00
------------------------------------------------------------------
                                              Demand Charge:      0.00
                                              Total Cost:         0.00

Node Results:
------------------------------------------------------------------
Node                Demand      Head  Pressure   Quality
ID                     GPM        ft       psi
------------------------------------------------------------------
JU1                    0.00    104.04     13.80      0.00
JU2                    0.00    104.04     13.32      0.00
JU3                    0.00    103.89     13.25      0.00
JU4                    0.00    103.72     15.48      0.00
JU5                    0.00    102.17     14.80      0.00
JU6                    0.00     99.70      0.27      0.00
RE1                 -481.35     19.90      0.00      0.00 Reservoir
TA1                  481.35     99.08      0.00      0.00 Tank


♀
Page 2
Link Results:
------------------------------------------------------------------
Link                Flow  VelocityUnit Headloss    Status
ID                   GPM       fps    ft/Kft
------------------------------------------------------------------
PI1                481.35      5.46     31.13      Open
PI2                481.35      5.46     31.13      Open
PI3                481.35      5.46     31.13      Open
PI4                481.35      5.46     31.13      Open
PI5                481.35      5.46     31.13      Open
PU1                481.35      0.00    -84.14      Open Pump
```

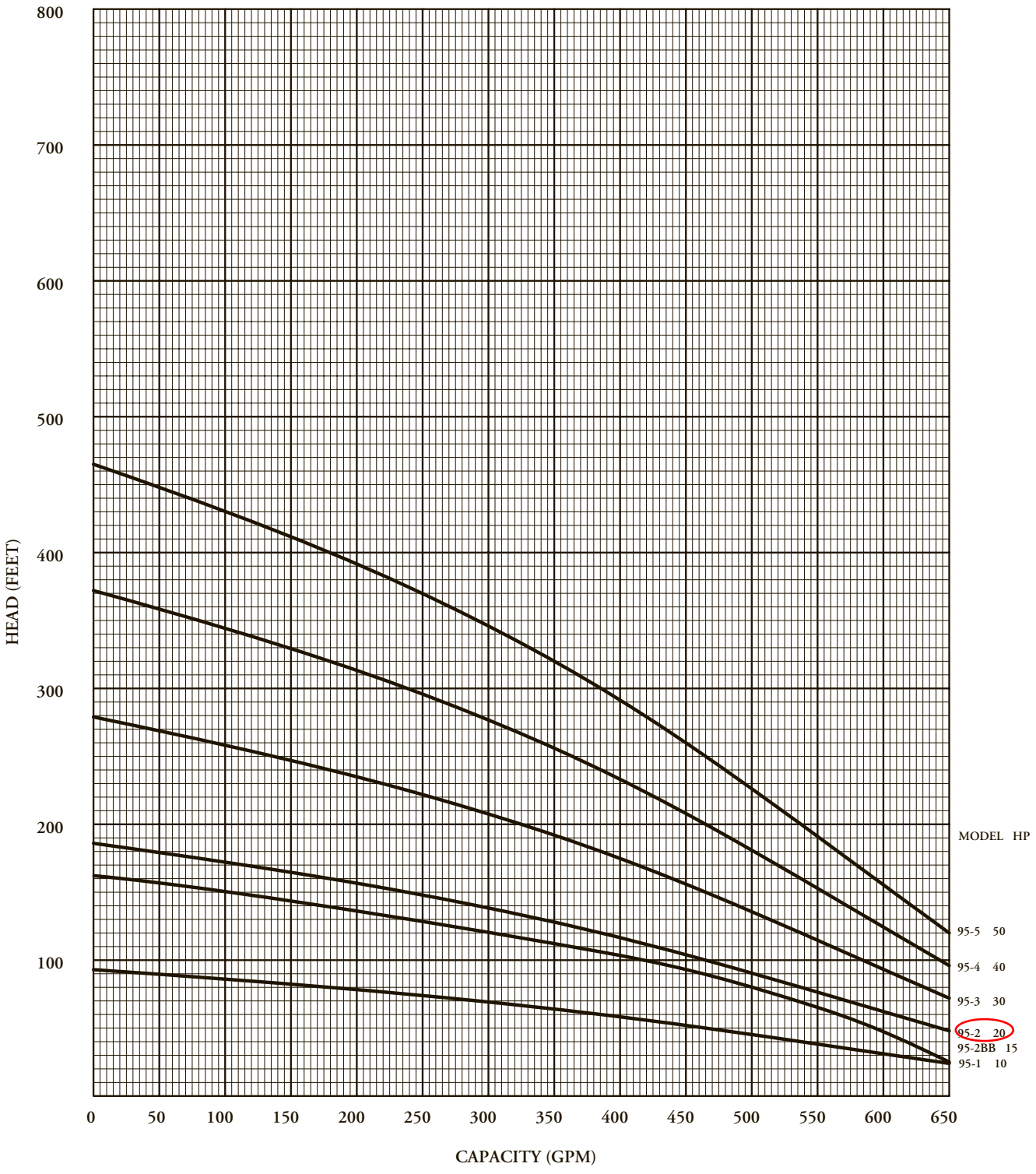
1	481.35	21.85	0.00	Open Valve

SERIES 95 SurePump™

Flow Range 95-680 GPM

60 Hz

Next
Back



DATA SUBJECT TO CHANGE WITHOUT NOTICE

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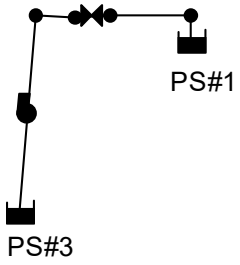
Schematic for PS#3 Operation



```
**********************************************************************
*                            E P A N E T                             *
*                    Hydraulic and Water Quality                     *
*                    Analysis for Pipe Networks                      *
*                          Version 2.0                               *
**********************************************************************

Input File: ps-3-network.net


Link - Node Table:
----------------------------------------------------------------------
Link            Start           End                  Length  Diameter
ID              Node            Node                     ft        in
----------------------------------------------------------------------
PI1             JU1             JU2                       2         3
PI2             JU3             JU4                       8         3
PI3             JU4             RE2                       4         3
PU1             RE1             JU1                     #N/A      #N/A Pump
1               JU2             JU3                     #N/A         3 Valve

Energy Usage:
----------------------------------------------------------------------
                  Usage   Avg.     Kw-hr      Avg.      Peak      Cost
Pump            Factor Effic.     /Mgal        Kw        Kw      /day
----------------------------------------------------------------------
PU1             100.00  75.00    166.72      0.35      0.35      0.00
----------------------------------------------------------------------
                                            Demand Charge:      0.00
                                            Total Cost:         0.00

Node Results:
----------------------------------------------------------------------
Node                Demand       Head   Pressure   Quality
ID                     GPM         ft        psi
----------------------------------------------------------------------
JU1                    0.00      72.10     -0.61      0.00
JU2                    0.00      72.09     -0.61      0.00
JU3                    0.00      72.09     -0.61      0.00
JU4                    0.00      72.03     -0.64      0.00
RE1                  -35.28      32.30      0.00      0.00 Reservoir
RE2                   35.28      72.00      0.00      0.00 Reservoir


Page 2
Link Results:
----------------------------------------------------------------------
Link                 Flow  VelocityUnit Headloss    Status
ID                    GPM       fps    ft/Kft
----------------------------------------------------------------------
PI1                  35.28      1.60      7.20      Open
PI2                  35.28      1.60      7.20      Open
PI3                  35.28      1.60      7.20      Open
PU1                  35.28      0.00    -39.80      Open Pump
1                    35.28      1.60      0.00      Open Valve
```

EPG VSDPT 8-2

APPENDIX H
STORAGE TANK CALCULATIONS

Leachate Storage Tanks and Surface Impoundments

Leachate Impoundment Calculation 2

Attachments

H-1 Fill Sequence-1

H-2 HELP Model Scenario #5

Leachate Impoundment Calculation

The estimation of leachate storage requirements at SFRL should be confirmed due to proposed landfill expansion footprint. The estimation is based on the assumption that Bay County closes the entire landfill footprint in a single phase upon completion of filling to its maximum permitted capacity. The leachate storage requirements are estimated based on following criteria:

- The existing cells and Cell 6 footprint is filled to elevation 220 NAVD (HELP Scenario #5). This elevation is the FDEP permitted operational height increase.
- Cell 7 is filled with 10-foot waste (HELP Scenario #1, in Appendix B)

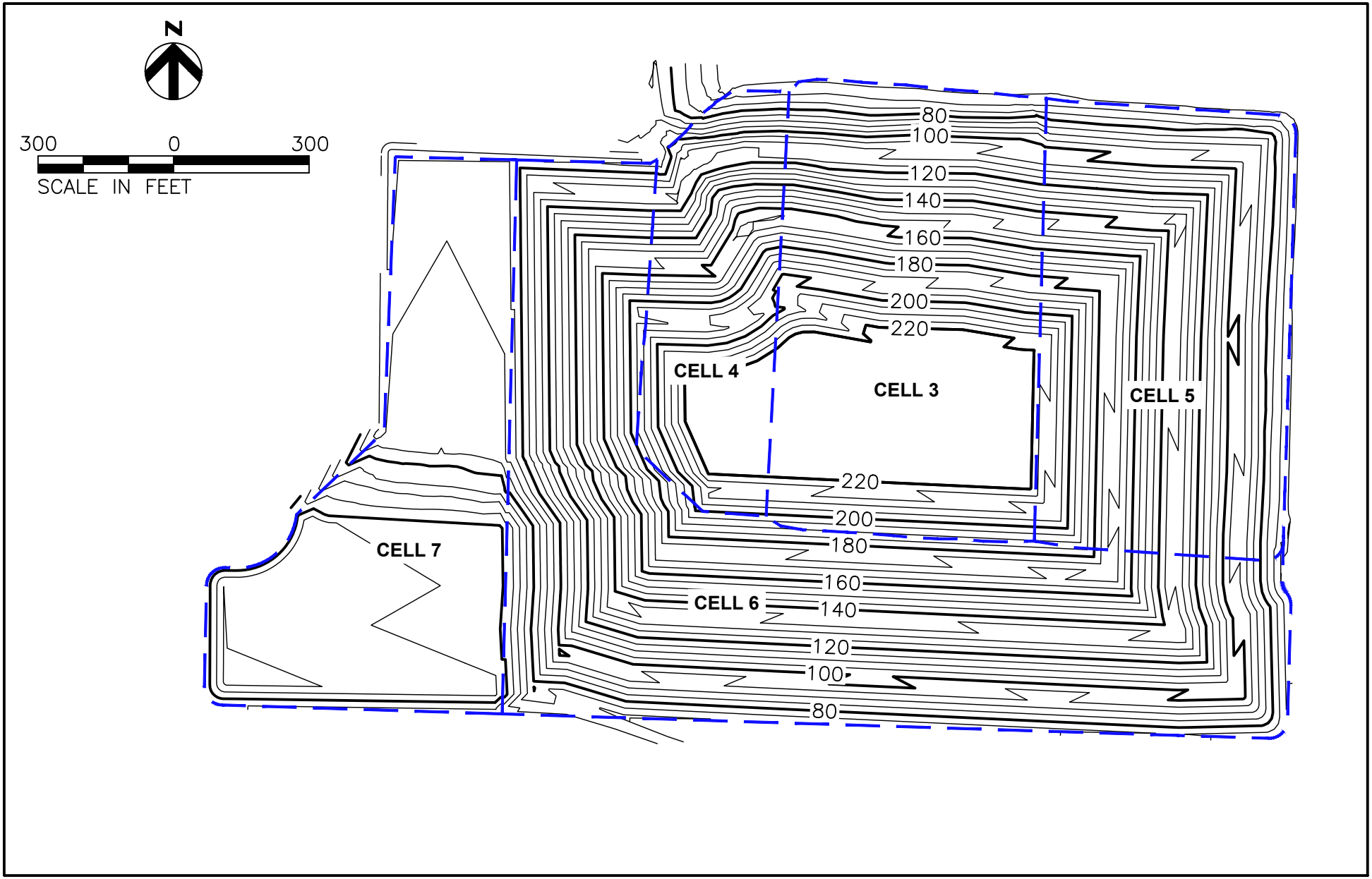
AutoCAD Civil 3-D is used to estimate waste capacity within Cells 1 to 6 based on intermediate fill elevations as shown in the Figure (Refer to Attachment H-1) by comparing Fill Sequence-1 intermediate fill elevation. The calculated waste capacity is estimated to be 7,055,991 cy. This waste volume is occupied within 53.84 acres. Accordingly average height of waste is 79.25 Feet while considering 2 foot protective cover. HELP Scenario #5 (Refer to Attachment H-2) is used to estimate leachate generation within entire SFRL footprint except Cell 7 which is filled with 10-ft depth of waste. The HELP model results are attached based on a 5 year simulation.

Description	Area (Acres)	Leachate Generation (cf/acre)
Average Annual Leachate Generation from Cells 1-6	53.9	107,002
Average Daily Leachate Generation from Cells 1-6	53.9	293
Average Annual Leachate Generation from Cell 7 (10-feet of waste)	11.5	117,278
Average Daily Leachate Generation from Cell 7	11.5	335
Total average leachate generation (cf/day)	19,646	
Total average leachate generation (gal/day)	146,953	
Accordingly required 3-day storage	440,859 gal	

The existing above ground storage tank has a capacity of 100,000 gals within 120,000 gal secondary containment area. However, this thank is partially corroded and require abandonment. Accordingly, 3x150,000 gal new leachate tanks are required to manage leachate generated when all cells are in operation. This will provide approximately 3 day storage considering a long weekend.

Attachment H-1

Fill Sequence-1



PROJECT TITLE
STEELFIELD ROAD LANDFILL
CONCEPTUAL DESIGN BUILD BID - BAY COUNTY, FLORIDA

SHEET TITLE
PHASE 1 FILL PLAN

PROJECT NUMBER
10150537

PROJECT MANAGER
M. ROBERTS, P.E.

DATE
FEB. 2019

REFERENCE SHEET

REFERENCE DOCUMENT

EXHIBIT NUMBER
FIGURE 1

Attachment H-2

HELP Model Scenario #5

```
 ******************************************************************************
 ******************************************************************************
 **                                                                          **
 **                                                                          **
 **               HYDROLOGIC EVALUATION OF LANDFILL PERFORMANCE               **
 **                                                                          **
 **               HELP Version 3.95 D          (10 August 2012)             **
 **                                developed at                              **
 **         Institute of Soil Science, University of Hamburg, Germany        **
 **                                  based on                                **
 **               US HELP MODEL VERSION 3.07  (1 NOVEMBER 1997)              **
 **                   DEVELOPED BY ENVIRONMENTAL LABORATORY                  **
 **                     USAE WATERWAYS EXPERIMENT STATION                    **
 **                FOR USEPA RISK REDUCTION ENGINEERING LABORATORY           **
 **                                                                          **
 **                                                                          **
 ******************************************************************************
 ******************************************************************************


 TIME:  15.02     DATE:   7.02.2019


 PRECIPITATION DATA FILE:      C:\HDR_Projects\steelfield\Engineering
Design\leachate Storage\precipitation.d4
 TEMPERATURE DATA FILE:        C:\HDR_Projects\steelfield\Engineering
Design\leachate Storage\temperature.d7
 SOLAR RADIATION DATA FILE:    C:\HDR_Projects\steelfield\Engineering
Design\leachate Storage\solar radiation.d13
 EVAPOTRANSPIRATION DATA F. 1: C:\HDR_Projects\steelfield\Engineering
Design\leachate Storage\evapotranspiration.d11
 SOIL AND DESIGN DATA FILE  1: C:\HDR_Projects\steelfield\Engineering
Design\leachate Storage\soil and design data-existing landfill.d10
 OUTPUT DATA FILE:             C:\HDR_Projects\steelfield\Engineering
Design\leachate Storage\HELP Model Sequence-1.out
 YEARLY OUTPUT DATA FILE:      C:\HDR_Projects\steelfield\Engineering
Design\leachate Storage\HELP Model Sequence-1.YR

 COLUMNS OF YEARLY OUTPUT DATA FILE:

      1    DATE OF ULTIMO (yyyy1231, years 2101 to 2200 from weather generator)
      2    PRECIPITATION (INCH)
      3    RUNOFF (INCH)
      4    POTENTIAL EVAPOTRANSPIRATION (INCH)
      5    ACTUAL EVAPOTRANSPIRATION (INCH)
      6    DRAIN #1: LATERAL DRAINAGE FROM LAYER  4 (INCH)
      7    LEAK  #1: PERCOLATION/LEAKAGE THROUGH LAYER  5 (INCH)
      8    DRAIN #2: LATERAL DRAINAGE FROM LAYER  6 (INCH)
      9    LEAK  #2: PERCOLATION/LEAKAGE THROUGH LAYER  7 (INCH)
     10    LEAK  #3: PERCOLATION/LEAKAGE THROUGH LAYER  8 (INCH)
     11    CHANGE IN TOTAL WATER STORAGE (INCH)
     12    CHANGE IN SOIL WATER STORAGE (INCH)
     13    CHANGE IN INTERCEPTION WATER STORAGE (INCH)
     14    CHANGE IN SNOW WATER STORAGE (INCH)
     15    ANNUAL WATER BUDGET BALANCE (INCH)


 ******************************************************************************

      TITLE:  Senario 5: Intermediate Cover-Sequence-1

 ******************************************************************************
```

WEATHER DATA SOURCES

NOTE: PRECIPITATION DATA WAS SYNTHETICALLY GENERATED USING
COEFFICIENTS FOR TALLAHASSEE FLORIDA

NORMAL MEAN MONTHLY PRECIPITATION (INCHES)

JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
4.66	5.00	5.60	4.13	5.16	6.55
8.75	7.30	6.45	3.10	3.31	4.58

NOTE: TEMPERATURE DATA WAS SYNTHETICALLY GENERATED USING
COEFFICIENTS FOR TALLAHASSEE FLORIDA

NORMAL MEAN MONTHLY TEMPERATURE (DEGREES FAHRENHEIT)

JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
51.60	53.60	60.20	67.10	74.00	79.50
81.20	81.10	78.30	68.40	58.80	53.00

NOTE: SOLAR RADIATION DATA WAS SYNTHETICALLY GENERATED USING
COEFFICIENTS FOR TALLAHASSEE FLORIDA
AND STATION LATITUDE = 30.23 DEGREES

****

LAYER DATA 1

VALID FOR 5 YEARS

NOTE: INITIAL MOISTURE CONTENT OF THE LAYERS AND SNOW WATER WERE
COMPUTED AS NEARLY STEADY-STATE VALUES BY THE PROGRAM.

LAYER 1

TYPE 1 - VERTICAL PERCOLATION LAYER
MATERIAL TEXTURE NUMBER 7

THICKNESS	=	12.00	INCHES
POROSITY	=	0.4730	VOL/VOL
FIELD CAPACITY	=	0.2220	VOL/VOL
WILTING POINT	=	0.1040	VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.3020	VOL/VOL
EFFECTIVE SAT. HYD. CONDUCT.	=	0.5200E-03	CM/SEC

NOTE: SATURATED HYDRAULIC CONDUCTIVITY IS MULTIPLIED BY 3.00
FOR ROOT CHANNELS IN TOP HALF OF EVAPORATIVE ZONE.

LAYER 2

TYPE 1 - VERTICAL PERCOLATION LAYER
MATERIAL TEXTURE NUMBER 18

THICKNESS	=	951.00	INCHES
POROSITY	=	0.6710	VOL/VOL
FIELD CAPACITY	=	0.2920	VOL/VOL
WILTING POINT	=	0.0770	VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.3012	VOL/VOL
EFFECTIVE SAT. HYD. CONDUCT.	=	0.1000E-02	CM/SEC

LAYER 3

TYPE 1 - VERTICAL PERCOLATION LAYER
MATERIAL TEXTURE NUMBER 0

THICKNESS	=	24.00	INCHES
POROSITY	=	0.5010	VOL/VOL
FIELD CAPACITY	=	0.2840	VOL/VOL
WILTING POINT	=	0.1350	VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.3291	VOL/VOL
EFFECTIVE SAT. HYD. CONDUCT.	=	0.1000E-03	CM/SEC

LAYER 4

TYPE 2 - LATERAL DRAINAGE LAYER
MATERIAL TEXTURE NUMBER 0

THICKNESS	=	0.30	INCHES
POROSITY	=	0.8500	VOL/VOL
FIELD CAPACITY	=	0.0100	VOL/VOL
WILTING POINT	=	0.0050	VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.0315	VOL/VOL
EFFECTIVE SAT. HYD. CONDUCT.	=	5.200	CM/SEC
SLOPE	=	2.00	PERCENT
DRAINAGE LENGTH	=	165.0	FEET

LAYER 5

TYPE 4 - FLEXIBLE MEMBRANE LINER
MATERIAL TEXTURE NUMBER 35

THICKNESS	=	0.06	INCHES
EFFECTIVE SAT. HYD. CONDUCT.	=	0.2000E-12	CM/SEC
FML PINHOLE DENSITY	=	2.00	HOLES/ACRE
FML INSTALLATION DEFECTS	=	2.00	HOLES/ACRE
FML PLACEMENT QUALITY	=	3 - GOOD	

LAYER 6

HELP Model Sequence-1

TYPE 2 - LATERAL DRAINAGE LAYER
MATERIAL TEXTURE NUMBER 0

THICKNESS	=	0.30	INCHES
POROSITY	=	0.8500	VOL/VOL
FIELD CAPACITY	=	0.0100	VOL/VOL
WILTING POINT	=	0.0050	VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.0151	VOL/VOL
EFFECTIVE SAT. HYD. CONDUCT.	=	5.200	CM/SEC
SLOPE	=	2.00	PERCENT
DRAINAGE LENGTH	=	165.0	FEET

LAYER 7

TYPE 4 - FLEXIBLE MEMBRANE LINER
MATERIAL TEXTURE NUMBER 35

THICKNESS	=	0.06	INCHES
EFFECTIVE SAT. HYD. CONDUCT.	=	0.2000E-12	CM/SEC
FML PINHOLE DENSITY	=	2.00	HOLES/ACRE
FML INSTALLATION DEFECTS	=	2.00	HOLES/ACRE
FML PLACEMENT QUALITY	=	3 - GOOD	

LAYER 8

TYPE 1 - VERTICAL PERCOLATION LAYER
MATERIAL TEXTURE NUMBER 17

THICKNESS	=	0.25	INCHES
POROSITY	=	0.7500	VOL/VOL
FIELD CAPACITY	=	0.7470	VOL/VOL
WILTING POINT	=	0.4000	VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.7470	VOL/VOL
EFFECTIVE SAT. HYD. CONDUCT.	=	0.3000E-08	CM/SEC

****

GENERAL DESIGN AND EVAPORATIVE ZONE DATA 1

VALID FOR 5 YEARS

NOTE: SCS RUNOFF CURVE NUMBER WAS COMPUTED FROM DEFAULT SOIL DATA BASE USING SOIL TEXTURE # 4 WITH A FAIR STAND OF GRASS, A SURFACE SLOPE OF 5.% AND A SLOPE LENGTH OF 100. FEET.

SCS RUNOFF CURVE NUMBER	=	63.03	
FRACTION OF AREA ALLOWING RUNOFF	=	100.0	PERCENT
AREA PROJECTED ON HORIZONTAL PLANE	=	1.000	ACRES
EVAPORATIVE ZONE DEPTH	=	6.0	INCHES
INITIAL WATER IN EVAPORATIVE ZONE	=	1.861	INCHES
UPPER LIMIT OF EVAPORATIVE STORAGE	=	2.838	INCHES
FIELD CAPACITY OF EVAPORATIVE ZONE	=	1.332	INCHES
LOWER LIMIT OF EVAPORATIVE STORAGE	=	0.624	INCHES
SOIL EVAPORATION ZONE DEPTH	=	6.000	INCHES
INITIAL SNOW WATER	=	0.000	INCHES

INITIAL INTERCEPTION WATER	=	0.002	INCHES
INITIAL WATER IN LAYER MATERIALS	=	298.161	INCHES
TOTAL INITIAL WATER	=	298.163	INCHES
TOTAL SUBSURFACE INFLOW	=	0.00	INCHES/YEAR

****

EVAPOTRANSPIRATION DATA 1

VALID FOR 5 YEARS

NOTE: EVAPOTRANSPIRATION DATA WAS OBTAINED FROM
TALLAHASSEE FLORIDA

STATION LATITUDE	=	30.23	DEGREES
MAXIMUM LEAF AREA INDEX	=	2.00	
START OF GROWING SEASON (JULIAN DATE)	=	36	
END OF GROWING SEASON (JULIAN DATE)	=	350	
EVAPORATIVE ZONE DEPTH	=	6.0	INCHES
AVERAGE ANNUAL WIND SPEED	=	6.60	MPH
AVERAGE 1ST QUARTER RELATIVE HUMIDITY	=	73.0	%
AVERAGE 2ND QUARTER RELATIVE HUMIDITY	=	73.0	%
AVERAGE 3RD QUARTER RELATIVE HUMIDITY	=	80.0	%
AVERAGE 4TH QUARTER RELATIVE HUMIDITY	=	77.0	%

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ANNUAL TOTALS FOR YEAR 1

	INCHES	CU. FEET	PERCENT
PRECIPITATION	48.59	176381.703	100.00
RUNOFF	0.037	136.088	0.08
POTENTIAL EVAPOTRANSPIRATION	54.031	196131.859	
ACTUAL EVAPOTRANSPIRATION	26.933	97767.664	55.43
DRAINAGE COLLECTED FROM LAYER 4	18.7959	68229.039	38.68
PERC./LEAKAGE THROUGH LAYER 5	2.813903	10214.468	5.79
AVG. HEAD ON TOP OF LAYER 5	0.0145		
DRAINAGE COLLECTED FROM LAYER 6	2.8139	10214.481	5.79
PERC./LEAKAGE THROUGH LAYER 7	0.000003	0.009	0.00
AVG. HEAD ON TOP OF LAYER 7	0.0022		
PERC./LEAKAGE THROUGH LAYER 8	0.000003	0.009	0.00
CHANGE IN WATER STORAGE	0.009	34.188	0.02

SOIL WATER AT START OF YEAR	298.351	1083013.500	
SOIL WATER AT END OF YEAR	298.360	1083045.625	
INTERCEPTION WATER AT START OF YEAR	0.002	5.764	
INTERCEPTION WATER AT END OF YEAR	0.002	7.822	
SNOW WATER AT START OF YEAR	0.000	0.000	0.00
SNOW WATER AT END OF YEAR	0.000	0.000	0.00
ANNUAL WATER BUDGET BALANCE	0.0001	0.224	0.00

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ANNUAL TOTALS FOR YEAR 2

	INCHES	CU. FEET	PERCENT
PRECIPITATION	80.32	291561.531	100.00
RUNOFF	0.151	549.010	0.19
POTENTIAL EVAPOTRANSPIRATION	53.954	195854.750	
ACTUAL EVAPOTRANSPIRATION	33.261	120736.617	41.41
DRAINAGE COLLECTED FROM LAYER 4	31.8504	115616.875	39.65
PERC./LEAKAGE THROUGH LAYER 5	3.769539	13683.427	4.69
AVG. HEAD ON TOP OF LAYER 5	0.0244		
DRAINAGE COLLECTED FROM LAYER 6	3.7690	13681.437	4.69
PERC./LEAKAGE THROUGH LAYER 7	0.000003	0.010	0.00
AVG. HEAD ON TOP OF LAYER 7	0.0029		
PERC./LEAKAGE THROUGH LAYER 8	0.000003	0.010	0.00
CHANGE IN WATER STORAGE	11.289	40977.746	14.05
SOIL WATER AT START OF YEAR	298.360	1083045.625	
SOIL WATER AT END OF YEAR	309.650	1124031.250	
INTERCEPTION WATER AT START OF YEAR	0.002	7.822	
INTERCEPTION WATER AT END OF YEAR	0.000	0.000	
SNOW WATER AT START OF YEAR	0.000	0.000	0.00
SNOW WATER AT END OF YEAR	0.000	0.000	0.00

ANNUAL WATER BUDGET BALANCE	-0.0001	-0.198	0.00

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ANNUAL TOTALS FOR YEAR 3

	INCHES	CU. FEET	PERCENT
PRECIPITATION	69.92	253809.578	100.00
RUNOFF	0.237	859.103	0.34
POTENTIAL EVAPOTRANSPIRATION	54.644	198359.312	
ACTUAL EVAPOTRANSPIRATION	30.333	110107.438	43.38
DRAINAGE COLLECTED FROM LAYER 4	35.4843	128808.086	50.75
PERC./LEAKAGE THROUGH LAYER 5	4.036486	14652.442	5.77
AVG. HEAD ON TOP OF LAYER 5	0.0272		
DRAINAGE COLLECTED FROM LAYER 6	4.0364	14652.044	5.77
PERC./LEAKAGE THROUGH LAYER 7	0.000003	0.010	0.00
AVG. HEAD ON TOP OF LAYER 7	0.0031		
PERC./LEAKAGE THROUGH LAYER 8	0.000003	0.010	0.00
CHANGE IN WATER STORAGE	-0.170	-617.038	-0.24
SOIL WATER AT START OF YEAR	309.650	1124031.250	
SOIL WATER AT END OF YEAR	309.480	1123414.250	
INTERCEPTION WATER AT START OF YEAR	0.000	0.000	
INTERCEPTION WATER AT END OF YEAR	0.000	0.000	
SNOW WATER AT START OF YEAR	0.000	0.000	0.00
SNOW WATER AT END OF YEAR	0.000	0.000	0.00
ANNUAL WATER BUDGET BALANCE	0.0000	-0.038	0.00

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ANNUAL TOTALS FOR YEAR 4

	INCHES	CU. FEET	PERCENT

PRECIPITATION	58.15	211084.484	100.00
RUNOFF	0.156	567.721	0.27
POTENTIAL EVAPOTRANSPIRATION	54.811	198963.734	
ACTUAL EVAPOTRANSPIRATION	31.955	115997.656	54.95
DRAINAGE COLLECTED FROM LAYER 4	34.2354	124274.602	58.87
PERC./LEAKAGE THROUGH LAYER 5	4.026515	14616.251	6.92
AVG. HEAD ON TOP OF LAYER 5	0.0262		
DRAINAGE COLLECTED FROM LAYER 6	4.0261	14614.874	6.92
PERC./LEAKAGE THROUGH LAYER 7	0.000003	0.010	0.00
AVG. HEAD ON TOP OF LAYER 7	0.0031		
PERC./LEAKAGE THROUGH LAYER 8	0.000003	0.010	0.00
CHANGE IN WATER STORAGE	-12.223	-44370.348	-21.02
SOIL WATER AT START OF YEAR	309.480	1123414.250	
SOIL WATER AT END OF YEAR	297.257	1079043.875	
INTERCEPTION WATER AT START OF YEAR	0.000	0.000	
INTERCEPTION WATER AT END OF YEAR	0.000	0.000	
SNOW WATER AT START OF YEAR	0.000	0.000	0.00
SNOW WATER AT END OF YEAR	0.000	0.000	0.00
ANNUAL WATER BUDGET BALANCE	0.0000	-0.047	0.00

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ANNUAL TOTALS FOR YEAR 5

	INCHES	CU. FEET	PERCENT
PRECIPITATION	70.21	254862.266	100.00
RUNOFF	0.124	451.829	0.18
POTENTIAL EVAPOTRANSPIRATION	53.436	193973.516	
ACTUAL EVAPOTRANSPIRATION	33.082	120086.977	47.12
DRAINAGE COLLECTED FROM LAYER 4	27.0204	98084.000	38.49
PERC./LEAKAGE THROUGH LAYER 5	3.476597	12620.047	4.95

AVG. HEAD ON TOP OF LAYER 5	0.0207		
DRAINAGE COLLECTED FROM LAYER 6	3.4762	12618.473	4.95
PERC./LEAKAGE THROUGH LAYER 7	0.000003	0.010	0.00
AVG. HEAD ON TOP OF LAYER 7	0.0027		
PERC./LEAKAGE THROUGH LAYER 8	0.000003	0.010	0.00
CHANGE IN WATER STORAGE	6.507	23621.033	9.27
SOIL WATER AT START OF YEAR	297.257	1079043.875	
SOIL WATER AT END OF YEAR	303.764	1102664.875	
INTERCEPTION WATER AT START OF YEAR	0.000	0.000	
INTERCEPTION WATER AT END OF YEAR	0.000	0.000	
SNOW WATER AT START OF YEAR	0.000	0.000	0.00
SNOW WATER AT END OF YEAR	0.000	0.000	0.00
ANNUAL WATER BUDGET BALANCE	0.0000	-0.074	0.00

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FINAL WATER STORAGE AT END OF YEAR 5

LAYER	(INCHES)	(VOL/VOL)
1	2.8013	0.2334
2	291.4174	0.3064
3	9.1340	0.3806
4	0.0293	0.0987
5	0.0000	0.0000
6	0.0059	0.0200
7	0.0000	0.0000
8	0.1867	0.7470

TOTAL WATER IN LAYERS	303.575
SNOW WATER	0.000
INTERCEPTION WATER	0.000
TOTAL FINAL WATER	303.575

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PEAK DAILY VALUES FOR YEARS 1 THROUGH 5

	(INCHES)	(CU. FT.)
PRECIPITATION	4.34	15754.200
RUNOFF	0.232	841.2319
DRAINAGE COLLECTED FROM LAYER 4	0.21099	765.89105
PERCOLATION/LEAKAGE THROUGH LAYER 5	0.017589	63.84763
AVERAGE HEAD ON TOP OF LAYER 5	0.059	
MAXIMUM HEAD ON TOP OF LAYER 5	0.116	
LOCATION OF MAXIMUM HEAD IN LAYER 4 (DISTANCE FROM DRAIN)	2.7 FEET	
DRAINAGE COLLECTED FROM LAYER 6	0.01730	62.80660
PERCOLATION/LEAKAGE THROUGH LAYER 7	0.000000	0.00003
AVERAGE HEAD ON TOP OF LAYER 7	0.005	
MAXIMUM HEAD ON TOP OF LAYER 7	0.012	
LOCATION OF MAXIMUM HEAD IN LAYER 6 (DISTANCE FROM DRAIN)	0.0 FEET	
PERCOLATION/LEAKAGE THROUGH LAYER 8	0.000000	0.00003
SNOW WATER	0.31	1126.4432
MAXIMUM VEG. SOIL WATER (VOL/VOL)	0.3647	
MINIMUM VEG. SOIL WATER (VOL/VOL)	0.1040	

*** Maximum heads are computed using McEnroe's equations. ***

Reference: Maximum Saturated Depth over Landfill Liner
by Bruce M. McEnroe, University of Kansas
ASCE Journal of Environmental Engineering
Vol. 119, No. 2, March 1993, pp. 262-270.

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AVERAGE MONTHLY VALUES IN INCHES FOR YEARS 1 THROUGH 5

HELP Model Sequence-1

	JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
PRECIPITATION						
TOTALS	4.32	5.01	4.43	3.72	5.07	8.32
	9.44	8.08	7.36	2.45	1.84	5.40
STD. DEVIATIONS	2.35	2.10	2.66	4.55	4.30	3.23
	4.08	2.93	2.62	1.77	0.49	3.13
RUNOFF						
TOTALS	0.003	0.000	0.000	0.009	0.007	0.000
	0.032	0.046	0.039	0.000	0.000	0.005
STD. DEVIATIONS	0.007	0.000	0.000	0.020	0.015	0.000
	0.069	0.104	0.048	0.000	0.000	0.011
POTENTIAL EVAPOTRANSPIRATION						
TOTALS	2.376	2.703	3.829	5.296	6.636	6.855
	6.955	6.056	4.828	3.839	2.805	1.997
STD. DEVIATIONS	0.133	0.137	0.201	0.415	0.431	0.212
	0.293	0.381	0.266	0.225	0.149	0.159
ACTUAL EVAPOTRANSPIRATION						
TOTALS	1.448	1.914	2.327	2.016	2.942	4.250
	5.511	4.001	3.300	1.366	0.900	1.137
STD. DEVIATIONS	0.225	0.203	0.646	1.460	1.103	0.842
	0.514	0.610	0.189	0.442	0.410	0.042
LATERAL DRAINAGE COLLECTED FROM LAYER 4						
TOTALS	1.4151	2.3342	2.2189	2.7283	2.8471	2.0534
	1.4466	2.4778	2.3028	3.4339	3.0475	3.1717
STD. DEVIATIONS	0.7911	0.4886	1.2202	0.4623	0.9548	1.2084
	0.8207	1.2366	1.1144	1.2852	0.5854	0.8981
PERCOLATION/LEAKAGE THROUGH LAYER 5						
TOTALS	0.2196	0.2885	0.2817	0.3289	0.3423	0.2671
	0.2135	0.3021	0.2843	0.3750	0.3532	0.3683
STD. DEVIATIONS	0.0802	0.0248	0.1043	0.0312	0.0724	0.1059
	0.0774	0.0904	0.0903	0.0946	0.0412	0.0629
LATERAL DRAINAGE COLLECTED FROM LAYER 6						
TOTALS	0.2199	0.2880	0.2816	0.3282	0.3426	0.2681
	0.2136	0.3011	0.2841	0.3747	0.3532	0.3691
STD. DEVIATIONS	0.0799	0.0236	0.1044	0.0318	0.0726	0.1059
	0.0772	0.0896	0.0906	0.0945	0.0416	0.0626
PERCOLATION/LEAKAGE THROUGH LAYER 7						
TOTALS	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000

STD. DEVIATIONS	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000

PERCOLATION/LEAKAGE THROUGH LAYER 8

TOTALS	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
STD. DEVIATIONS	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000

AVERAGES OF MONTHLY AVERAGED DAILY HEADS (INCHES)

DAILY AVERAGE HEAD ON TOP OF LAYER 5

AVERAGES	0.0128	0.0232	0.0200	0.0255	0.0257	0.0192
	0.0131	0.0224	0.0215	0.0310	0.0284	0.0286
STD. DEVIATIONS	0.0071	0.0048	0.0110	0.0043	0.0086	0.0113
	0.0074	0.0112	0.0104	0.0116	0.0055	0.0081

DAILY AVERAGE HEAD ON TOP OF LAYER 7

AVERAGES	0.0020	0.0029	0.0025	0.0031	0.0031	0.0025
	0.0019	0.0027	0.0027	0.0034	0.0033	0.0033
STD. DEVIATIONS	0.0007	0.0002	0.0009	0.0003	0.0007	0.0010
	0.0007	0.0008	0.0008	0.0009	0.0004	0.0006

AVERAGE ANNUAL TOTALS & (STD. DEVIATIONS) FOR YEARS 1 THROUGH 5

	INCHES		CU. FEET	PERCENT
PRECIPITATION	65.44	(12.261)	237539.9	100.00
RUNOFF	0.141	(0.0716)	512.75	0.216
POTENTIAL EVAPOTRANSPIRATION	54.175	(0.5573)	196656.62	
ACTUAL EVAPOTRANSPIRATION	31.113	(2.6114)	112939.27	47.545
LATERAL DRAINAGE COLLECTED FROM LAYER 4	29.47728	(6.79126)	107002.523	45.04612
PERCOLATION/LEAKAGE THROUGH LAYER 5	3.62461	(0.50777)	13157.327	5.53900
AVERAGE HEAD ON TOP OF LAYER 5	0.023	(0.005)		
LATERAL DRAINAGE COLLECTED FROM LAYER 6	3.62431	(0.50766)	13156.263	5.53855

PERCOLATION/LEAKAGE THROUGH LAYER 7	0.00000	(0.00000)	0.010	0.00000
AVERAGE HEAD ON TOP OF LAYER 7	0.003	(0.000)		
PERCOLATION/LEAKAGE THROUGH LAYER 8	0.00000	(0.00000)	0.010	0.00000
CHANGE IN WATER STORAGE	1.082	(8.8509)	3929.12	1.654

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APPENDIX I
REQUIREMENTS OF PERIMETER CHANNEL AND CULVERT CAPACITY

Job No.	10150537	Calc No.	

Computation

Project	Steelfield Road Landfill - Cells 6 and 7 Expansion	Computed	K. Perera
System	Drainage Calculations	Date	2/18/2019
Component	Perimeter Ditch and Culvert Design	Reviewed	M. Roberts
Task		Date	2/19/2019

Purpose

Determine the appropriate dimensions for the perimeter ditches and culverts.

Methodology

Using the Rational Method procedures in the FDOT Drainage Manual and Manning's Equation for flow in open channels the dimensions for the perimeter ditches is determined. The equations are used in an iterative process where the assumed depth of channel flow is modified until it is equal to the calculated depth of channel flow. The design ditch is a 10-foot bottom width grass lined channel.

Assumptions

1) The perimeter ditches are to be designed for the 25-yr storm.

2) Use procedures in the FDOT Drainage Manual.

3) The time of concentration (Tc) for each subarea is assumed to be 8 minutes. This is the most conservative value using the FDOT Drainage Manual procedures.

Calculation

Surface Flow Equations and Known Variables

Rational Method → Q = CiA (design storm 25-yr)

Q (25-yr) = Flow Rate, cfs

C_1 = Runoff Coefficient → Paved Road = 0.85 *(Attachment I-1A)*

Adjust by 1.1 for 25-yr return period

C_1 = 0.94 *(Attachment I-1B)*

C_2 = Runoff Coefficient → Flat, Sandy Soils, Grass = 0.15 *(Attachment I-1A)*

Adjust by 1.1 for 25-yr return period *(Attachment I-1B)*

C_2 = 0.17

C_3 = Runoff Coefficient → Steep Slopes, Sandy Soils, Bare Earth = 0.60 *(Attachment I-1A)*

Adjust by 1.1 for 25-yr return period

C_3 = 0.66 *(Attachment I-1B)*

i = rainfall intensity for 25-yr storm = 8.5 in/hr *(Attachment I-2)*

A = Drainage Area

Open Channel Equations and Known Variables

Manning's Equation → $Q = \frac{1.49}{n} A R_h^{2/3} S^{1/2}$

Q (25-yr) = Flow Rate, cfs

n = Roughness Coefficient → maintained grass ditches = 0.060 *(Attachment I-3)*

Decrease 30% for flows > 0.7 ft

n = 0.042

A = Area of Flow

R_h = Hydraulic Radius

S = Slope of Channel

Job No.	10150537	Calc No.	

Computation

Project	Steelfield Road Landfill - Cells 6 and 7 Expansion	Computed	K. Perera
System	Drainage Calculations	Date	2/18/2019
Component	Perimeter Ditch and Culvert Design	Reviewed	M. Roberts
Task		Date	2/19/2019

Perimeter Ditch and Culvert

Location of Upper End	Segment Length (ft)	Rational Method				Manning's Equation									
		Landfill Drainage Area (ac)	Perimeter Drainage Area (ac)	Paved Road Area (ac)	Total Flow (cfs)	Assumed Depth of Flow (ft)	Calculated Depth of Flow (ft)	Flow Line El. Up (ft)	Flow Line El. Low (ft)	Slope (%)	Velocity (fps)	Bottom Width (ft)	Left Slope (H:1)	Right Slope (H:1)	Rh (ft)	Area of Flow (sf)
HP	4646	53.60	5.25	3.07	332.46	3.56	3.54	75.8	50.0	0.56	4.55	10	3	3	2.3	73.6
Culvert		11.97	0.95	0.51	72.54											

Job No.	10150537	Calc No.	

Computation

Project	Steelfield Road Landfill - Cells 6 and 7 Expansion	Computed	K. Perera
System	Drainage Calculations	Date	2/18/2019
Component	Attachment I-1A: Runoff Coefficients	Reviewed	M. Roberts
Task		Date	2/19/2019

Source: FDOT Drainage Handbook Hydrology

Hydrology Handbook
January 2004

Table T-4
Runoff Coefficients For A Design Storm Return Period Of 10 Years Or Less[a]

Slope	Land Use	Sandy Soils Min.	Sandy Soils Max.		Clay Soils Min.	Clay Soils Max.
				Use		
Flat (0-2%)	Woodlands	0.10	0.15	0.15	0.15	0.20
	Pasture, grass, and farmland [b]	0.15	0.20		0.20	0.25
	Bare Earth	0.30	0.50		0.50	0.60
	Rooftops and pavement	0.95	0.95	Use	0.95	0.95
	Pervious pavements [c]	0.75	0.95	0.85	0.90	0.95
	SFR: 1/2-acre lots and larger	0.30	0.35		0.35	0.45
	Smaller lots	0.35	0.45		0.40	0.50
	Duplexes	0.35	0.45		0.40	0.50
	MFR: Apartments, townhouses, and condominiums	0.45	0.60		0.50	0.70
	Commercial and Industrial	0.50	0.95		0.50	0.95
Rolling (2-7%)	Woodlands	0.15	0.20		0.20	0.25
	Pasture, grass, and farmland [b]	0.20	0.25		0.25	0.30
	Bare Earth	0.40	0.60		0.60	0.70
	Rooftops and pavement	0.95	0.95		0.95	0.95
	Pervious pavements [c]	0.80	0.95		0.90	0.95
	SFR: 1/2-acre lots and larger	0.35	0.50		0.40	0.55
	Smaller lots	0.40	0.55		0.45	0.60
	Duplexes	0.40	0.55		0.45	0.60
	MFR: Apartments, townhouses, and condominiums	0.50	0.70		0.60	0.80
	Commercial and Industrial	0.50	0.95		0.50	0.95
Steep (7%+)	Woodlands	0.20	0.25		0.25	0.30
	Pasture, grass, and farmland [b]	0.25	0.35	Use	0.30	0.40
	Bare Earth	0.50	0.70	0.60	0.70	0.80
	Rooftops and pavement	0.95	0.95		0.95	0.95
	Pervious pavements [c]	0.85	0.95		0.90	0.95
	SFR: 1/2-acre lots and larger	0.40	0.55		0.50	0.65
	Smaller lots	0.45	0.60		0.55	0.70
	Duplexes	0.45	0.60		0.55	0.70
	MFR: Apartments, townhouses, and condominiums	0.60	0.75		0.65	0.85
	Commercial and Industrial	0.60	0.95		0.65	0.95

[a] Weighted coefficient based on percentage of impervious surfaces and green areas must be selected for each site.

[b] Coefficients assume good ground cover and conservation treatment.

[c] Depends on depth and degree of permeability of underlying strata.

Note: SFR = Single Family Residential
MFR = Multi-Family Residential

Job No.	10150537	Calc No.	

Computation

Project	Steelfield Road Landfill - Cells 6 and 7 Expansion	Computed	K. Perera
System	Drainage Calculations	Date	2/18/2019
Component	Attachment I-1A: Runoff Coefficients	Reviewed	M. Roberts
Task		Date	2/19/2019

Attachment I-1B Source: FDOT Drainage Handbook Hydrology

Hydrology Handbook
January 2004

Table T-5
Design Storm Frequency Factors For Pervious Area Runoff Coefficients *

Return Period (years)	Design Storm Frequency Factor, X_T
2 to 10	1.0
25	1.1
50	1.2
100	1.25

Reference: Wright-McLaughlin Engineers (1969).

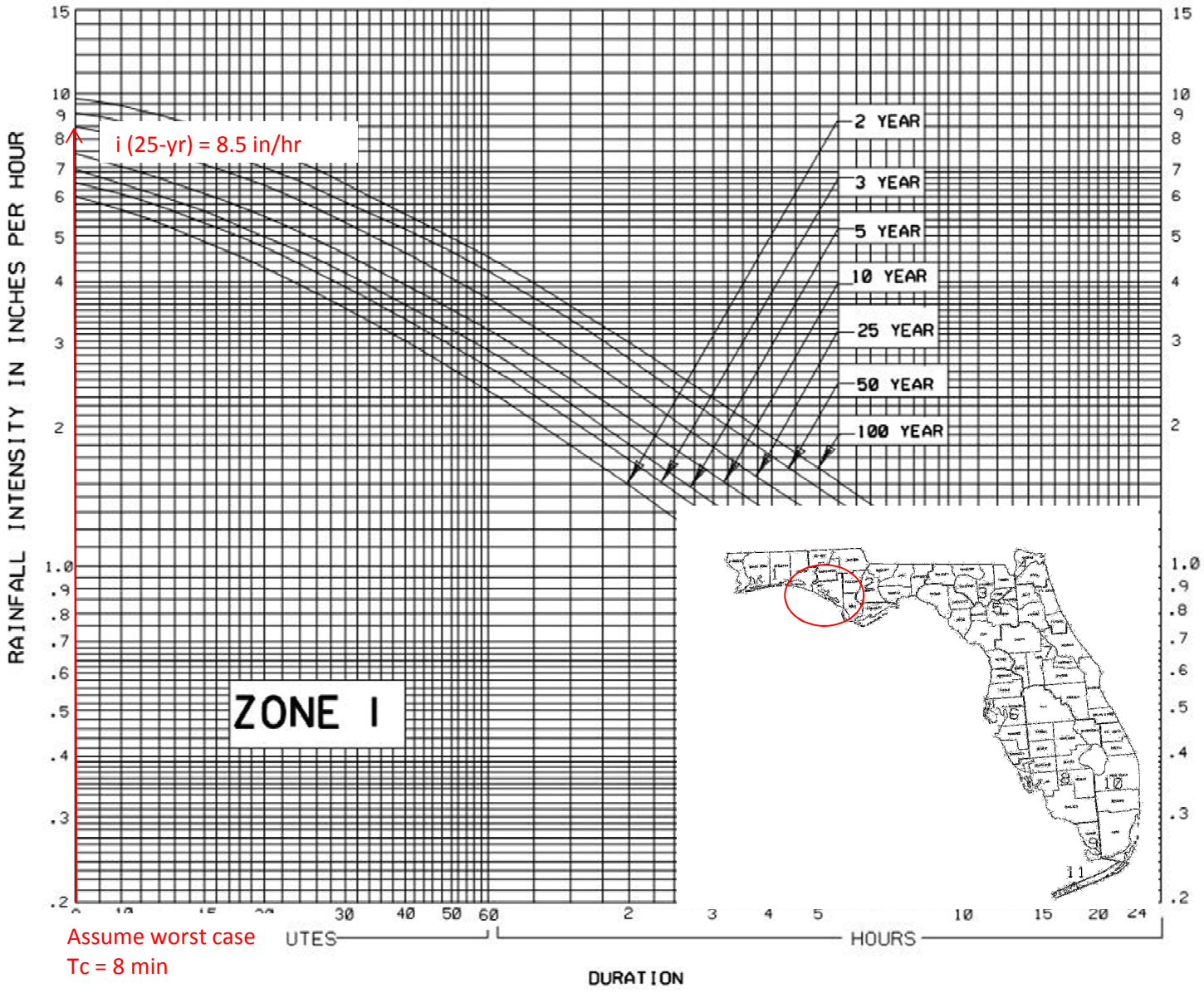
* DUE TO THE INCREASE IN THE DURATION TIME THAT THE PEAK OR NEAR PEAK DISCHARGE RATE IS RELEASED FROM STORMWATER MANAGEMENT SYSTEMS, THE USE OF THESE SHORT DURATION PEAK RATE DISCHARGE ADJUSTMENT FACTORS IS NOT APPROPRIATE FOR FLOOD ROUTING COMPUTATIONS.

Job No.	10150537	Calc No.	

Computation

Project	Steelfield Road Landfill - Cells 6 and 7 Expansion	Computed	K. Perera
System	Drainage Calculations	Date	2/18/2019
Component	Attachment I-2: Rainfall Intensity	Reviewed	M. Roberts
Task		Date	2/19/2019

Source: FDOT Drainage Handbook Hydrology



Assume worst case
Tc = 8 min

Job No.	10150537	Calc No.	

Computation

Project	Steelfield Road Landfill - Cells 6 and 7 Expansion	Computed	K. Perera
System	Drainage Calculations	Date	2/18/2019
Component	Attachment I-3: Roughness Coefficients for Channels	Reviewed	M. Roberts
Task		Date	2/19/2019

Hydrology Handbook
January 2004

Table T-3
Recommended Manning's n Values for Artificial Channels with Various Linings

Channel Lining	Lining Description	Design Manning's n Value
Bare Earth or Vegetative Linings		
Bare earth, fairly uniform	Clean, recently completed	0.022
Bare earth, fairly uniform	Short grass and some weeds	0.028
Dragline excavated	No vegetation	0.030
Dragline excavated	Light brush	0.040
Channels not maintained	Dense weeds to flow depth	0.100
Channels not maintained	Clear bottom, brush sides	0.080
Maintained grass or sodded ditches	Good stand, well maintained 2" - 6"	0.060*
Maintained grass or sodded ditches	Fair stand, length 12" - 24"	0.200*
Rigid Linings		
Concrete paved	Broomed**	0.016
Concrete paved	"Roughened" - standard	0.020
Concrete paved	Gunite	0.020
Concrete paved	Over rubble	0.023
Asphalt concrete	Smooth	0.013
Asphalt concrete	Rough	0.016

* Decrease 30% for flows > 0.7' (maximum flow depth 1.5').

** Because this is not the standard finish, it must be specified.

Job No.	10150537	Calc No.	

Computation

Project	Steelfield Road Landfill - Cells 6 and 7 Expansion	Computed	K. Perera
System	Drainage Calculations	Date	2/18/2019
Component	Attachment I-4: Recommended Maximum Velocities	Reviewed	M. Roberts
Task		Date	2/19/2019

Table 7-1
MAXIMUM SHEAR STRESS VALUES
AND ALLOWABLE VELOCITIES FOR DIFFERENT SOILS

Soil Type	Shear Stress (lb/ft^2)	Allowable Velocity for a Flow Depth of About 3 ft (ft/sec)
Silt or Fine Sand	0.027	1.50
Sandy Loam	0.037	1.75
Silt Loam	0.048	2.00
Firm Loam	0.075	2.50
Stiff Clay	0.260	3.75
Hardpans	0.670	6.00

Reference: University of Florida (1972)

Table 7-2
RECOMMENDED MAXIMUM VELOCITIES FOR VARIOUS LINING TYPES

Lining Type	Maximum Velocity (ft/sec)
Grass with Mulch	Bare Soil (see Table 17-1)
Sod	(4)
Lapped Sod	5.5
Geotextile Grid	4-8*
Flexible	Design in Accordance with HEC-15
Rigid	10**

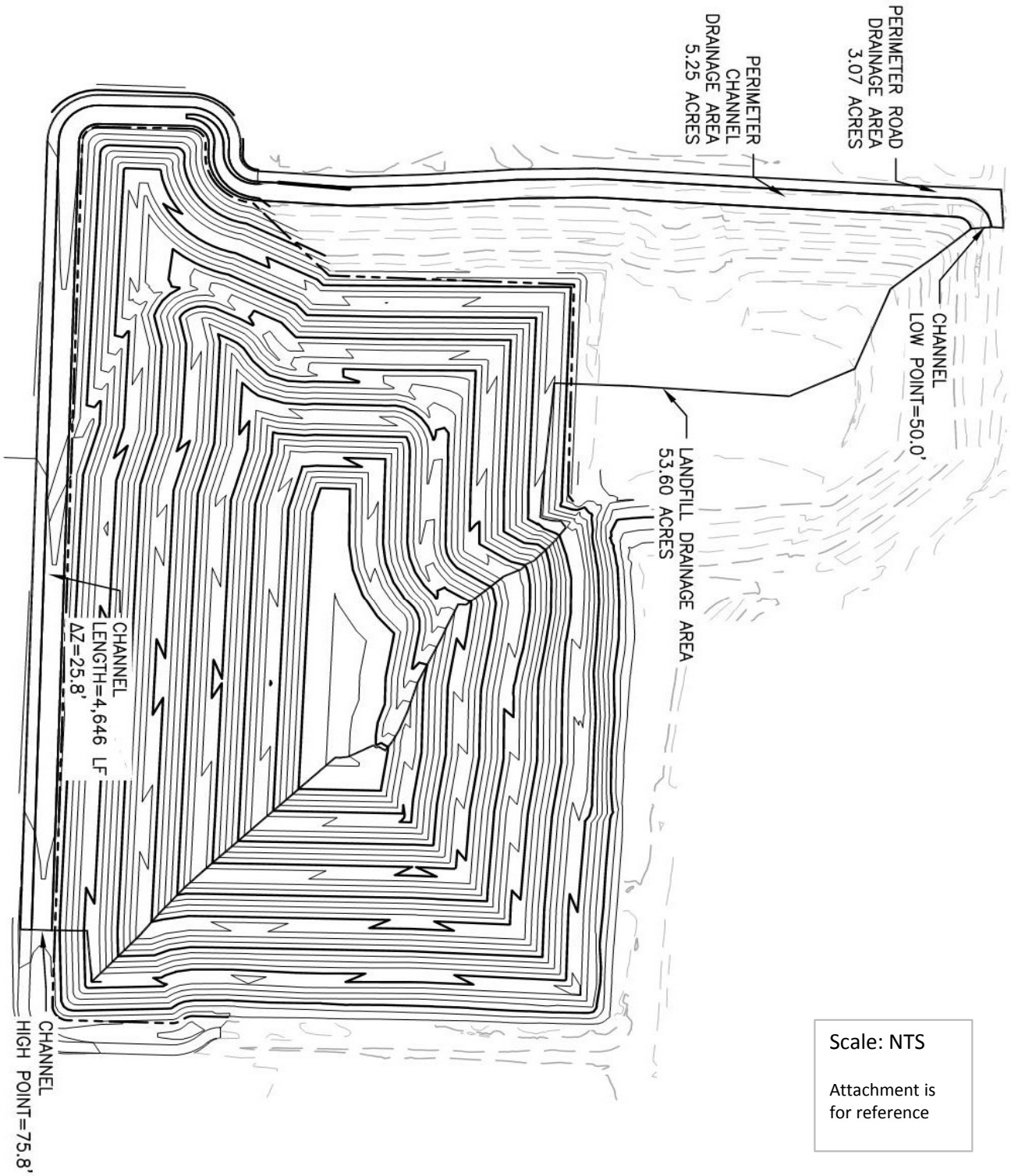
*Varies with grid.

**Higher velocities are acceptable if energy dissipation is accomplished.

Job No.	10150537	Calc No.	

Computation

Project	Steelfield Road Landfill - Cells 6 and 7 Expansion	Computed	K. Perera
System	Drainage Calculations	Date	2/18/2019
Component	Attachment I-5: Drainage Limits	Reviewed	M. Roberts
Task		Date	2/19/2019



Scale: NTS

Attachment is for reference

Culvert Report

Hydraflow Express Extension for Autodesk® AutoCAD® Civil 3D® by Autodesk, Inc. Wednesday, Feb 20 2019

Worst Case Culvert Design

Invert Elev Dn (ft)	=	66.95
Pipe Length (ft)	=	120.00
Slope (%)	=	0.78
Invert Elev Up (ft)	=	67.88
Rise (in)	=	19.0
Shape	=	Elliptical
Span (in)	=	30.0
No. Barrels	=	3
n-Value	=	0.012
Culvert Type	=	Elliptical Inlet Face (E)
Culvert Entrance	=	Tapered inlet-beveled edges (E)
Coeff. K,M,c,Y,k	=	0.536, 0.622, 0.0368, 0.83, 0.2

Embankment

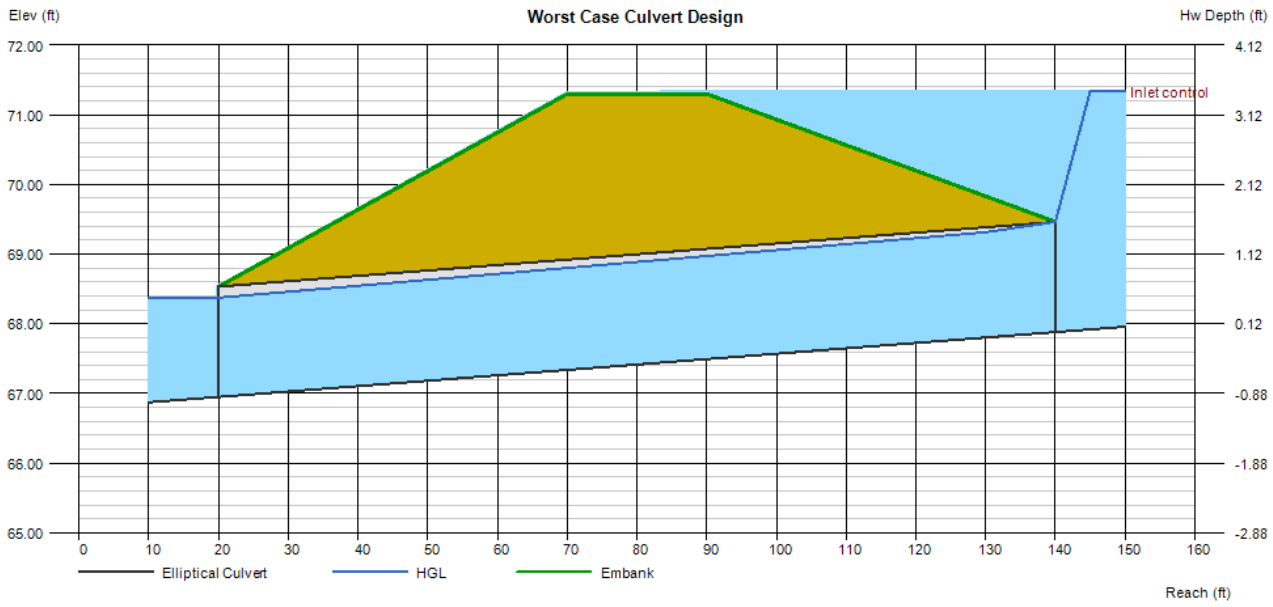
Top Elevation (ft)	=	71.30
Top Width (ft)	=	20.00
Crest Width (ft)	=	50.00

Calculations

Qmin (cfs)	=	50.00
Qmax (cfs)	=	72.50
Tailwater Elev (ft)	=	(dc+D)/2

Highlighted

Qtotal (cfs)	=	72.50
Qpipe (cfs)	=	71.36
Qovertop (cfs)	=	1.14
Veloc Dn (ft/s)	=	7.97
Veloc Up (ft/s)	=	7.77
HGL Dn (ft)	=	68.38
HGL Up (ft)	=	69.40
Hw Elev (ft)	=	71.34
Hw/D (ft)	=	2.19
Flow Regime	=	Inlet Control



APPENDIX J
CLOSURE AND LONG-TERM CARE

Bay County Steelfield Road Landfill

Closure & Long-Term Care Plan

Facility ID Number: NWD/03/0000016
Permit No.: ~~0007875-005-SO & 0007876-006-SF~~0007875-009-SO-01 & 0127100-005-SF-T3

Prepared for:

Bay County General Services Department
11411 Landfill Road
West Bay, FL 32413

Revised by:

HDR, Inc.
~~200 W Forsyth St, Suite 800~~76 S. Laura St, Suite 1600
Jacksonville, Florida 32202
904-598-8900

Revised on: ~~June 2014~~March 2019

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LANDFILL CLOSURE PLAN (62-701.600, FAC)

1.0 Closure Permit Requirements (62-701.600(2), FAC)

Bay County will submit an application to the FDEP for final closure of the landfill at least 90 days before the date when wastes will no longer be accepted. The application shall be on Form 62-701.900(1). The application shall include a Closure Plan.

The Closure Plan will be consist of the following:

1. closure design plan
2. closure operation plan
3. plan for long-term care (LTC)
4. demonstration that proof of financial assurance for long-term care will be provided

2.0 Closure Design Plan (62-701.600(3), FAC)

A closure design plan shall be submitted with the application for final closure pursuant to Chapter 62-701.600, FAC. A preliminary closure design plan is presented below as part of this Closure Plan, and consists of overall engineering plans and a report on closing procedures that apply to the final closing of solid waste disposal units during the operation of the landfill, the final closing of the landfill, and the monitoring and maintenance during the long-term care period.

The anticipated components of the closure cap and their associated thicknesses are described below from top to bottom.

- The vegetative cover will consist of a commercially grown sod.
- The final cover soil layer will consist of twenty four (24) inches of soil capable of supporting the local species of sod grasses. The uppermost soil may include on-site soils amended with composted yard waste mulch fines.
- The drainage layer above the low permeability cap will consist of a geocomposite drainage layer with ~~1 x 10^{-3}~~6.3 cm/sec. or greater average hydraulic conductivity.
- The low permeability layer will consist of a 40 mil textured LLDPE liner. The LLDPE liner will significantly reduce the amount of infiltration passing through the topsoil and cover soil layers and into the waste material. The reduction in the amount of infiltration through the waste will ultimately reduce the amount of leachate generated.
- The bedding layer will consist of a 12 inches existing soil cover material.

This Closure Design Plan includes the following information:

a. A Plan Sheet Showing Phases of Site Closure

An original Construction Phasing Plan for the Steelfield Road Landfill was

provided as Sheets 6 to 7 of the Construction & Operation Permit Application (August 1986). The phasing was slightly modified because of the Cell name designation detailed in Major Permit Modification (May 1997). In general, the closure site phasing will closely follow construction phasing. Cells 1 and 2 have already been closed. ~~Once Cells 3, 4 and 5 reaches the final elevation, they will be closed as per the closure plans (C-701 to C-703) provided in Cell 5 Construction Application (November 2010).~~ Part of Cells 6 and 7 will be constructed on top of closed Cells 1 and 2. Bay County anticipates to begin filling Cell 6A once existing Cells 3, 4 and 5 have reached permitted elevations. During filling of Cells 6A, stormwater from Cells 6B and 7 will be collected over the rain liner and pumped out of the cell into the perimeter channel. Filling of Cell 6B will begin once the Cell 6A reaches to an elevation that could be reasonably accessed using internal temporary roads as determined by Bay County operations staff. The rain liner will be removed gradually as filling progresses. The last stage of filling will include Cell 7 and remaining airspace within Cells 3 to 6 to bring final grades as shown in Conceptual Final Grading Plan on Sheet 00C-05.

b. *Drawings Showing Existing Topography and Proposed Final Grades*

The existing topography and proposed final grades were detailed in the Plans provided in ~~Cell 5 Construction Application (November 2010)~~the drawing package prepared by HDR as part of Cells 6 and 7 bidding**.** ~~Final grades for remaining Cells have been detailed in the original Construction & Operation Permit Application (August 1986). These will be updated at the time of closure.~~ Bay County will determine the sequence of next cell construction and fill grades before reaching the final grades proposed as part of this application.

c. *Provisions to Close Units When They Reach Approved Design Dimensions*

The landfill cells will be closed as soon as they reach approved design dimensions and elevations. The Steelfield Road Landfill calculates the remaining capacity annually for the waste disposal cells to estimate if the landfill is reaching its design capacity and dimensions.

d. *Proposed Final Elevations Before Settlement*

The final top of waste elevations before settlement will be 3 feet below the elevations shown on the final grading plans for the landfill referenced in Section 2(~~b~~a).

e. *Final Side Slope Design*

The side slopes shall not be steeper than 3 horizontal to 1 vertical (3H: 1V). Drainage diversion ~~beams~~ terraces~~or benches~~ will be provided every ~~20~~ 40 vertical feet on the side slope.

f. Final Cover Installation Plans

1. *Construction Quality Assurance Plan*

   Upon completion of a final closure design a Construction Quality Assurance (CQA) Plan for installing and testing the final cover during construction will be developed which meets the requirements of Subsections 62-701.400(7) and (8), F.A.C. The final cover shall be installed and tested in accordance with a construction quality assurance plan which meets the requirements of subsections 62-701.400(7) and (8), F.A.C.

2. *Schedule For Installing Final Cover*

   The final cover installation will begin within 180 days after the final receipt of waste.

3. *Description of Drought-Resistant Species To Be Used in Vegetative Cover*

   The surface of the final cover system will be vegetated by sodding (and seeding where allowable) to minimize erosion.

4. *Top Gradient Design to Maximize Runoff and Control Erosion*

   The top slopes of the cover will be not less than two percent. Based on the current design top deck is graded at 5% slope.

5. *Provisions For Cover Material To Be Used For Final Cover Maintenance*

   The site will maintain a stockpile of soil material to be used for final cover maintenance.

g. Final Cover Installation Plans

The cross section of the final cover system on the top and side slopes of the closure areas consists of, from top to bottom:

- commercially grown sod
- 24-inches of soil with top 6-inches able to support vegetative growth
- geocomposite drainage layer
- 40-mil ~~of~~ LLDPE liner
- 12-inches of soil cover material

~~Plans~~ Drawings showing details on the final cover ~~were~~ are included in ~~closure plans (C-701 to C-703) provided in Cell 5 Construction Application (November 2010)~~the drawings package.

1. *Protective Soil Layer Design*

   The final cover design includes a 2-foot thick protective soil layer capable of sustaining vegetation growth from on-site or off-site sources.

2. *Barrier Soil Layer Design*

   No barrier soil layer is proposed in this cap design.

3. *Erosion Control Vegetation*

   The surface of the final cover system will be vegetated by sodding to minimize erosion.

4. *Geomembrane Barrier Layer Design*

   A component of the final cover system is a geomembrane placed under the drainage layer to reduce infiltration of precipitation through the final cover system into the waste. The barrier layer will consist of 40-mil thick LLDPE geomembrane.

5. *Geosynthetic Clay Liner Design*

   The final cover design will not include a Geosynthetic Clay Liner (GCL). This subsection is not applicable.

6. *Stability Analysis of Cover System and Disposed Waste*

   Stability analysis of the final cover system is ~~of was~~ performed as ~~part of Cell 5 Construction Application (November 2010~~shown in~~)~~ – Appendix ~~C~~B of the engineering report. The stability analysis will be reviewed and updated before closure of the landfill.

h. *Proposed Method of Stormwater Control*

Stormwater will be controlled by a series of HDPE downcomers from the top terrace of the landfill to the bottom. The top of the landfill will be sloped at approximately ~~2~~ 5 percent from the center to the edge where the slope changes to 3:1. ~~At the change of slope a small diversion berm will be installed around the perimeter of the top so that stormwater is conveyed to downcomers via the drop inlets at change of the slope.~~ Terrace(s) will ~~also~~ be installed to control and convey stromwater runoff to downcommer inlets as shown in the drawing package. ~~For Cells 3 to 5, final closure one terrace is proposed as indicated on closure plans (C-701 to C-703) provided in Cell 5 Construction Application (November 2010).~~ Other typical details related to stromwater control are provided in the same closure plans. At the base of the stormwater pipes, Florida

Department of Transportation energy dissipation structures will be installed. Perimeter channels convey the stormwater to the strormwater pond.

i. *Proposed Method of Access Control*

Access to the closed landfill shall be restricted to prevent unauthorized use. Restricted access will be maintained after closure and will not be modified.

j. *Description of Proposed or Existing Gas Management System*

Passive gas vents will be installed to release gas within the landfill and perimeter gas probe monitoring will be performed on quarterly basis unless otherwise directed by the FDEP. The monitoring locations will be sampled for concentrations of combustible gases, and the results are reported to the Department. Combustible gases are determined as a percent of the lower explosive limit and calibrated to methane. If the results of monitoring show that combustible gas concentrations exceed the lower explosive limits then following actions will be taken:

- Immediately take all necessary steps to ensure protection of human health and notify the Department; and
- Within 7 days of detection, submit to the Department for approval a gas remediation plan for the gas releases. The plan shall describe the nature and extent of the problem and the proposed remedy. The remedy shall be completed within 60 days of detection unless otherwise approved by the Department.

3.0 Closure Operation Plan (62-701.600(4), FAC)

The Closure Operation Plan is included below and follows the FDEP application form outline. This will be updated at the time of the closure.

a. *Description of Actions Which Will Be Taken To Close The Landfill*

The intermediate cover will be stripped of vegetation and surface grading will be performed as needed. Additional cover soil may be added which will act as a stable base grade for the cap system. The final cover system will be installed from bottom to top with:

- 12-inches of soil cover
- 40-mil LLDPE geomembrane
- geocomposite drainage layer
- 24-inches of cover soil with top 6-inches able to support vegetation
- vegetative cover consisting of a commercially grown sod

The final cover or cap system components will be constructed in accordance with the final design plans and specifications, which will detail the requirements for each material. A Construction Quality Assurance (CQA) Plan will be developed as indicated in Section 2(f) of this Closure and LTC Plan. An independent inspector will be used to inspect the installation and to document the construction so that a certification of the cap installation can be made.

b. Time Schedule For Completion of Closing and Long-Term Care

A Closure Permit Application will be submitted to FDEP at least 90 days before closing date. Signs will be posted at the site two months before landfill closure.

c. Proposed Method For Demonstrating Financial Assurance for Long-Term Care

Bay County plans to use an escrow account as the financial assurance mechanism for the LTC period.

d. Operation of the Water Quality Monitoring Plan

The water quality monitoring will be performed in accordance with the WQM plan provided in facility's operations permit.

e. Development and Implementation of Gas Management System

Landfill gas management system consists of passive vents and perimeter gas monitoring probes. The gas monitoring probes are monitored as outlined in the Operations Permit for the facility.

4.0 Certification of Closure Construction Completion (62-701.600(6), FAC)

Certification of closure construction will be performed in accordance with 62-701.600(6), FAC. At end of the construction, a closure certification report be produced summarizing all the QA/QC activities during the construction of the cap. Any substantial deviations of the plans will be noted in the report. The FDEP Construction Certification form will be completed and signed and sealed by a Professional Engineer in accordance with FDEP requirements.

a. Survey Monuments

There are existing benchmarks on the Steelfield Road Landfill property. Additional survey monumentation will be added to define the limits of waste as required by 62-701.600(6)(a), FAC. These waste limit markers will be included in the final survey.

b. *Final Survey Report*

A final survey report signed by a professional surveyor will be prepared as part of the closure construction certification and submitted with the closure certification report. This survey report will include an topographic survey included with the record plans and documents in accordance with 62-701.600(6)(b), FAC.

5.0 Declaration to the Public (62-701.600(7), FAC)

Once closure construction has been completed, the County will file a declaration to the public in the deed records in the office of the County clerk. The declaration will include a legal description of the landfill property and a site plan specifying the area actually filled with solid waste. The declaration will also include a notice that any future owner or user of the site must consult with the Department prior to planning or initiating any activity involving the disturbance of the landfill cover, monitoring system or other control structures. A certified copy of the declaration will be filed with the Department.

6.0 Official Date of Closing (62-701.600(8), FAC)

The Department will evaluate the closure certification documents and within 30 days of its determination that closure complies with the closure plan and the closure requirements, the Department will acknowledge by letter to the County that closing of the facility has been completed. The letter will be the official date of landfill closing for purposes of determining the long-term care period. If only a portion of the landfill has been closed, the long-term care period will begin upon the closing of the entire landfill, unless the portion that has been closed can be monitored and maintained separately from the rest of the landfill.

7.0 Temporary Closure (62-701.600(9), FAC)

If temporary closure is ever desired, justification for and detailed description of procedures to be followed for temporary closure will be submitted pursuant to Chapter 62-701.600(7), FAC. Currently there is no plan for temporary closure of the Steelfield Road Landfill. However, if it appears that temporary closure will be considered, Bay County will prepare a justification and a detailed description of procedures to be followed for temporary closure at least 90 days prior to the temporary closure.

OTHER CLOSURE PROCEDURES (FAC 62-701.610)

8.0 Use of Closed Solid Waste Disposal Facilities (62-701.610(1), FAC)

There is no current proposed final use of this property other than as a closed landfill.

9.0 Relocation of Waste (62-701.610(2), FAC)

Relocation of waste is associated with the landfill mining activities currently performed at the landfill. A detailed description on the mining activities is provided in the Minor Permit Modification for Proposed Mining Activities (July 2012).

In addition, in 2019 Bay County requested to increase SFRL operational height to accommodate emergency solid waste debris generated from Hhurricane Michael. The proposed height increase included steepening the sideslopes to increase airspace within the lined footprint. During closure if prevailing waste grades are found to be higher than the closure grades presented in drawing 00C-04 of this application, Bay County will relocate excess waste within the other areas of the landfill.

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LONG TERM CARE (62-701.620, FAC)

Long-term care will be provided for the landfill pursuant to Chapter 62-701.620, FAC. Bay County shall continue to monitor and maintain the facility in accordance with the approved closure plan for 30 years from the date of closing. The surface water management system, passive gas control system, and vegetative cover will be maintained during the long-term care period. In addition, the leachate collection and disposal system will be operated during the long-term care period. At a minimum, monthly inspections of the closed landfill facilities will be conducted.

10.0 Maintaining the Gas Venting and Monitoring System (62-701.620(5), FAC)

Bay County will maintain the landfill gas venting and monitoring system throughout the entire LTC period. If the waste stabilizes in less than 30 years and the landfill gas production is reduced so that there is no significant production of combustible gases or objectionable odors, Bay County may apply to FDEP for a reduction in the gas monitoring frequency.

During the LTC period, the County shall inspect the site in accordance with the frequency outlined in the landfill's LTC Permit. The inspection shall direct attention to:

- Final cover or cap system;
- Differential settlement;
- Stormwater pond operations;
- Stormwater pond berms, site vegetative cover, drainage, and erosion control; and
- Evidence of any unauthorized use of the site.

Repairs to restore the site to the intended condition will be made after any deficiency is noticed. A record of the inspections and findings shall be kept in the supervisor's office.

Groundwater, surfacewater and methane gas monitoring will be continued based on the landfill's closure and LTC permit.

11.0 Stabilization Report (62-701.620(6), FAC)

Every five years after issuance of a permit for long-term care, Bay County will submit a report to FDEP that addresses stabilization of the landfill. The submittal will include the technical report required in 62-701.510(9)(b), FAC, and also address subsidence, barrier layer effectiveness, storm water management, and gas production and management. The submittal will also address leachate collection and removal system effectiveness.

12.0 Right of Access (FAC 62-701.620(7))

The landfill will maintain the right of access to the site to carry out long-term care requirements. Bay County will retain the right of entry to the landfill property for the long-term care period, after termination of solid waste operations, for inspection, monitoring and maintenance of the site.

13.0 Replacement of Monitoring Devices (FAC 62-701.620(8))

If a monitoring well or other device required by FDEP permit is damaged or fails to operate for any reason, Bay County will notify FDEP in writing upon discovery of this condition. All inoperative devices will be replaced with functioning devices within 60 days of discovery in accordance with 62-701.620(8), FAC.

14.0 Completion of Long-Term Care (FAC 62-701.620(9))

Following completion of the long-term care period for each solid waste management unit, Bay County will notify the FDEP that a certification, signed and sealed by a professional engineer, verifying that long-term care has been completed in accordance with the closure plan has been placed in the operating record.