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WINYAH GENERATING STATION

Run-on and Run-off Control System Plan for New Class 3 CCR Landfill Area 1

40 CFR Part 257
Operating Criteria
§257.81(c)

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1. INTRODUCTION

The United States Environmental Protection Agency (EPA) promulgated regulations (40 CFR Part 257) regarding coal combustion residuals (CCRs). The CCR rule was published in the Federal Register on April 17, 2015 and became effective on October 19, 2015. The Class Three CCR Landfill is subject to the CCR Rule as a new landfill as defined in 40 CFR §257.53. A requirement of the CCR rule is to prepare a written run-on and run-off control system plan (§257.81(c)) for new CCR landfills. This initial plan must be placed in the facility operating record no later than the date of initial receipt of CCR in the CCR unit as required by §257.81(c)(3)(ii).

This document serves as certification that the run-on and run-off control system plan for the new CCR landfill Area 1 at Winyah Generating Station in Georgetown, South Carolina meets the requirements of §257.81. The design is documented in the Winyah Generating Station Class Three Landfill Permit Application approved by the South Carolina Department of Health and Environmental Control (DHEC) on 15 September 2017 (Permit #LF3-00042). The run-on and run-off control system meets the South Carolina solid waste management regulation R.61-107.19 as certified by the design engineer-of-record, Scott M. Graves, P.E., Geosyntec Consultants, Inc. The South Carolina Department of Health and Environmental Control issued a permit to construct on September 15, 2017 with an effective date of September 30, 2017. Construction Quality Assurance was documented in a report by Terracon Consultants, Inc. dated October 12, 2018.

2. DISCUSSION

Title 40 CFR §257.81(c)(1) requires that the run-on and run-off control system plan must document how the run-on and run-off control systems have been designed and constructed to meet the applicable requirements of this section. Each plan must be supported by appropriate engineering calculations.

The applicable requirements for the run-on and run-off control system plan are listed below, with a description of how the systems are designed and constructed to satisfy each requirement. Supporting engineering calculations are included in the Appendices.

257.81(a) states *the owner or operator of an existing or new CCR landfill or any lateral expansion of a CCR landfill must design, construct, operate, and maintain:*

> *(1) A run-on control system to prevent flow onto the active portion of the CCR unit during the peak discharge from a 24-hour, 25-year storm*

The new Class Three CCR Landfill Area 1 is located within the footprint of the former Unit 2 Slurry Pond. The Class Three landfill is surrounded by a perimeter berm that prevents run-on from flowing onto the active portion of the landfill. Run-on controls will be used to prevent stormwater flow onto the active portion of the landfill. Storm water run-on will be directed away from the active working area using temporary diversion berms positioned if necessary around/up-gradient from the active working area, to prevent flow onto the active portion of the landfill during the peak discharge from a 24-hour, 25-year storm. Calculations for sizing of the temporary diversion berms, depending on the up-gradient area contributing stormwater run-on, are presented in Appendix B. All facilities are designed to handle the peak discharge from a 24-hour, 25-year storm event.

(2) A run-off control system from the active portion of the CCR unit to collect and control at least the water volume resulting from a 24-hour, 25-year storm

Run-off controls will be used at active portions of the landfill to collect and control at least the water volume resulting from a 25-year, 24-hour storm. The new Class Three CCR Landfill Area 1 includes a leachate collection system as well as a decant structure (chimney drain) located within the active portion of the landfill. The decant structure is designed to intercept stormwater runoff from the active face of the landfill and convey it directly to the NPDES permitted waste water pond via gravity piping and pumping. The purpose of the decant structure is to minimize leachate generation and to collect and control at least the water volume (and peak flow rate) resulting from a 24-hour, 25-year storm. The design documentation for the decant structure and leachate collection system in included in Appendix A.

257.81(b) states *run-off from the active portion of the CCR unit must be handled in accordance with the surface water requirements under §257.3-3*

Storm water run-off from active areas (that has come in contact with waste) will be managed as contact water. Contact water will be removed to prevent stagnant ponding of water and to minimize it from otherwise infiltrating through the waste to become leachate. This will be accomplished using pumps to remove contact water, as well as using decant structures to filter and convey contact water out of the cell. In both cases (pumps and decant structures), the contact water will be conveyed through piping that is independent from the rest of the leachate management system. Calculations for sizing of the decant structure to effectively manage the volume of water resulting from the design storm are included in Appendix A. All run-off generated from the active portion of the new Class Three CCR Landfill Area 1 is conveyed to an on-site NPDES permitted waste water pond. The wastewater pond will contain the volume resulting from a 24-hour, 25-year storm. The effluent from the wastewater pond is ultimately discharged from the site through permitted NPDES (SC0022471) outfall 002. All run-off from the active portion of the new Class Three Landfill Area 1 is designed to be handled in accordance with the surface water requirements under §257.3-3. The State permit includes plans for a dedicated landfill leachate and contact water storage pond to be constructed in the next phase to satisfy future Steam Effluent Limitation Guidelines. Design documentation is included in Appendix A.

This document satisfies the requirements of §257.81(c) by providing a run-on and run-off control system plan that documents how the run-on and run-off control system for the new Class Three CCR landfill Area 1 at the Winyah Generating Station has been designed and constructed and will be operated and maintained to meet the applicable requirements of this section and state regulations, including supporting engineering calculations.

3. CONCLUSIONS

This document presents the run-on and run-off control system plan for the new Class Three CCR landfill Area 1 at Winyah Generating Station in Georgetown, SC. The run-on and run-off control system plan is in accordance with the requirements of Title 40 CFR §257.81 for new CCR landfills.

4. CERTIFICATION

Certification for Run-on and Run-off Control System Plan

Federal CCR Rule: 40 CFR §257.81

CCR Unit: WGS Class Three Landfill Area 1 - New CCR Landfill

I, the undersigned Professional Engineer registered in good standing in the State of South Carolina, do hereby certify under penalty of law that I have personally examined and am familiar with the information submitted in this demonstration, and that, based on my inquiry of the individuals responsible for obtaining the information, I believe that the submitted information is true, accurate, and complete. I am aware that there are significant penalties for submitting false information, including the possibility of fine and imprisonment. I certify, for the above-referenced CCR Unit, that the run-on and run-off control system plan contained herein is in accordance with the requirements of Title 40 CFR §257.81.

SOUTH CAROLINA REGISTERED PROFESSIONAL ENGINEER No. 18485 DENISE A. BUNTE-BISNETT

Seal and Signature: Denise A. Bunte-Bisnett

Printed Name: Denise A. Bunte-Bisnett

P.E. License Number: 18485 State of South Carolina

APPENDIX A

Calculations for Decant and Leachate Systems

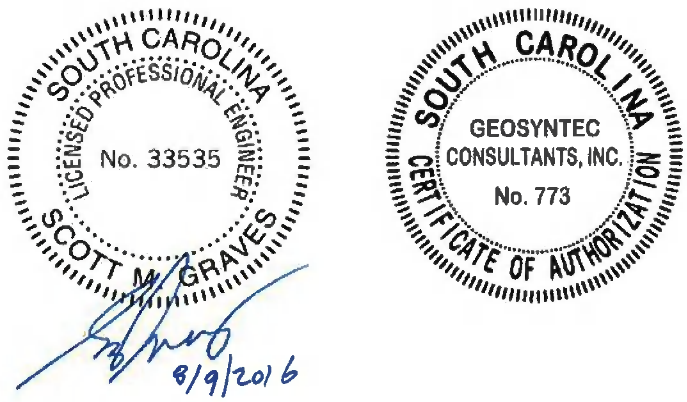
APPENDIX E-1

LEACHATE GENERATION RATES AND HEAD ON LINER (HELP MODELING)

Written by:	V. Krishnan	Date:	06/10/16	Reviewed by:	M. Christman	Date:	07/01/16
Client:	Santee Cooper	Project:	Winyah Generating Station	Project No.:	GSC5242	Task No.:	01BT

APPENDIX E-1

LEACHATE GENERATION RATES AND HEAD ON LINER (HELP MODELING)



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CALCULATION PAGES E-1-1
THROUGH E-1-65

PURPOSE

The purpose of this calculation package is to estimate the quantity of leachate to be generated by the proposed Class Three Landfill (composed of Landfill Area 1 and Landfill Area 2) at the Winyah Generating Station (WGS). The calculations are used to:

- estimate the annual flow rate from the leachate collection system drainage layer during filling and following facility closure;
- estimate contact water generation rate due to runoff over exposed waste (i.e., the active area);
- estimate the maximum leachate head on the liner system; and
- estimate the quantity of leachate that may percolate through the liner system.

METHOD OF ANALYSIS

The leachate flow rates and maximum leachate head on the liner system are calculated using the Hydrologic Evaluation of Landfill Performance (*HELP*) computer model, Version 3.07, developed by the U.S. Environmental Protection Agency (USEPA). *HELP* simulates hydrologic processes for a landfill by performing daily, sequential water balance analyses using a quasi-two-dimensional, deterministic approach (Schroeder et al., 1994a, 1994b).

The hydrologic processes considered in the *HELP* model include precipitation, surface-water evaporation, runoff, infiltration, plant transpiration, soil water evaporation, soil water storage, vertical drainage (saturated and unsaturated), lateral drainage (saturated), vertical drainage (saturated) through compacted clay liners and geosynthetic clay liners (GCLs), and leakage through geomembranes.

The volumetric flow rate from the leachate collection system for a given plan area of liner system is calculated as:

$$Q = q_d \times A_d \qquad \text{Eqn. (1)}$$

where:

Q = volumetric flow rate of leachate from a given plan area of liner system;

q_d = unitized flow rate from the leachate collection system drainage layer per the *HELP* model output (see Attachment E-1.A); and

A_d = plan area of liner system drained by the leachate collection system drainage layer.

The contact water generation rate due to runoff from exposed waste (i.e., active area) is calculated as:

$$Q = q_r \times A_e \qquad \text{Eqn. (2)}$$

where:

Q = volumetric generation rate of contact water from a given cell;

q_r = unitized runoff from the cell per the *HELP* model output (see Attachment E-1.A); and

A_e = plan area of exposed waste (active area).

ANALYSIS CASES AND SCENARIOS

The proposed Class 3 Landfill consists of a total of eight cells, each with two subcells (e.g., Cell 1 consists of Subcell 1E and Subcell 1W). The leachate generation rate, contact water generation rate, and maximum leachate head on the liner system are calculated for four typical operational conditions (A to D) for the two landfill areas. These operational conditions are:

- Case A – initial conditions: 10-ft of waste over the liner system;
- Case B – intermediate condition: 30-ft of waste over the liner system;
- Case C-1 – final grades in Landfill Area 1 before installing final cover: average of 52 ft of waste over the liner system;

- Case C-2 – final grades in Landfill Area 2 before installing final cover: average of 88 ft of waste over the liner system;
- Case D-1 – final grades in Landfill Area 1 after installing final cover: final cover system overlying an average of 52 ft of waste over the liner system; and
- Case D-2 – final grades in Landfill Area 2 after installing final cover: final cover system overlying an average of 88 ft of waste over the liner system.

The total leachate generation in the cells is calculated by adding the volumetric flow rate of leachate from each cell, which depends on the operational condition and the area of the cell. The progress of the operational condition of the cells, i.e., from initial waste placement to final cover installation, is evaluated for each cell over the course of the life of the facility, and the volumetric leachate generation at each stage is calculated by multiplying the unitized leachate generation rate per the *HELP* model with the plan area of the cell, as shown in Equation (1).

Two types of composite liner systems options are proposed: (i) "Option 1", a standard (prescriptive) composite liner system; and (ii) "Option 2", an alternative composite liner system. The standard composite liner system consists of the following components, from top to bottom:

- 2-ft thick protective cover/drainage layer (sand or other permeable material, such as bottom ash) with a minimum hydraulic conductivity (k_{min}) of 1×10^{-4} cm/s;
- geocomposite drainage layer (geotextile filters bonded to both sides of a geonet drainage core);
- 60-mil thick textured high-density polyethylene (HDPE) geomembrane (flexible membrane liner); and
- 2-ft thick compacted clay layer with a maximum hydraulic conductivity (k_{max}) of 1×10^{-7} cm/s).

The alternative composite liner system consists of the following components, from top to bottom:

- 2-ft thick protective cover/drainage layer (sand or other permeable material, such as bottom ash) with $k_{min} = 1 \times 10^{-4}$ cm/s;
- geocomposite drainage layer (geotextile filters bonded to both sides of a geonet drainage core);
- 60-mil thick textured HDPE geomembrane (flexible membrane liner);

- needlepunched reinforced GCL; and
- 12-in. thick compacted clay layer with $k_{max} = 1 \times 10^{-5}$ cm/s.

Two types of final cover systems options are proposed: (i) "Option 1", a standard (prescriptive) final cover system; and (ii) "Option 2", an alternative final cover system. The standard final cover system consists of the following components, from top to bottom:

- 2-ft thick layer of soil capable of supporting native vegetation, consisting of an upper 6-in. thick topsoil layer and a lower 18-in. thick protective cover soil layer;
- geocomposite drainage layer (geotextile filters bonded to both sides of a geonet drainage core);
- 20-mil (minimum) thick textured linear low-density polyethylene (LLDPE) geomembrane; and
- 18-in. thick compacted soil layer with $k_{max} = 1 \times 10^{-5}$ cm/s.

The alternative final cover system is similar to the standard final cover system except that a needlepunched reinforced GCL is used in place of the 18-in. thick compacted soil layer. Both final cover system design options significantly limit percolation of rainfall through the final cover system and, therefore, limit leachate generation. Because the *HELP* analyses for the final cover system operational condition were found to be similar (i.e., leachate generation is negligible) for both final cover system design options, only the analyses for the alternative final cover system option are presented in this calculation package.

PARAMETERS USED IN ANALYSIS

The *HELP* model requires the input of daily weather data, vegetation data, soils data, and landfill design data. The input data are described in this section and summarized on the *HELP* model output presented in Attachment E-1.A.

Weather Data

Thirty years of synthetic weather data are generated for the Class 3 Landfill using climate data for Charleston, South Carolina. The peak daily rainfall from the synthetically generated precipitation record is manually increased to assess the impact of the 25-year, 24-hour storm event. The 25-year, 24-hour storm depth of 8.3 in. is selected based on information from the

NOAA Atlas 14 Point Precipitation Frequency Estimates (NOAA Hydrometeorological Design Studies Center, retrieved 06-May-2016), as shown in Table E-1.1.

Evapotranspiration Data

Vegetation is assumed only on the final cover system (Cases D-1 and D-2). The final cover system is assumed to have fair vegetation with a maximum leaf area index of 2.0. The default evaporative zone depth of 22 in. is selected for the thickness of the erosion layer above the composite barrier. For initial and intermediate conditions, no vegetation is assumed, and the default evaporative zone depth of 10 in. is selected for the purpose of evaluating evaporation.

Materials Data

Topsoil/Protective Cover Soil Layers for Final Cover System

The topsoil and protective cover soils layers of the final cover system are modeled as a vertical percolation layer with *HELP* material texture 10 (representative of sandy material with a hydraulic conductivity (k) of 1.2×10^{-4} cm/s).

Geocomposite Drainage Layer for Final Cover System

The geocomposite drainage layer is modeled as a lateral drainage layer with *HELP* material texture 20 (representative of 0.2-in. thick geonet drainage layer) and having a slope of 33 percent and a slope length of 100 feet. The design hydraulic conductivity (k) of the drainage layer is calculated by varying the parameter *k* until the peak daily average head on the geomembrane cover is maintained within the drainage layer (i.e., equal to or less than the thickness of the geocomposite = 0.2 in.).

Geomembrane Cover for Final Cover System

The geomembrane cover is modeled as a flexible membrane liner with *HELP* material texture 36 (representative of low density polyethylene (LDPE) geomembrane), installation condition = good, pinhole defect frequency = 2 per acre, and installation defect frequency = 2 per acre. This hole frequency is an assumption for design purposes only and does not imply the expected or allowable hole density.

GCL for Final Cover System

The GCL of the alternative final cover system is modeled as a barrier soil liner with *HELP* material texture 17 (representative of 0.25-in. thick bentonite mat with $k = 3 \times 10^{-9}$ cm/s).

Interim Cover

The waste placed in the landfill will be coal combustion residuals such as fly ash, bottom ash, etc. The hydraulic conductivity of interim cover, to the extent that it is placed, is expected to be similar to the coal combustion residual materials; therefore the interim cover is not separately modeled in *HELP*. As filling of waste progresses in the landfill cells, it is anticipated that interim cover will be placed on the sideslopes. The acreage of the exposed waste (i.e., the active area) is considered for estimating the volume of the contact water generated.

Waste

The waste layer is modeled as a vertical percolation layer with *HELP* material texture 30 (representative of fly ash from coal-fired power plants with $k = 5 \times 10^{-5}$ cm/s).

Protective Cover/Drainage Layer for Liner System

The protective cover layer is modeled as a vertical percolation layer with *HELP* material texture 10 (representative of sandy material with $k = 1.2 \times 10^{-4}$ cm/s).

Geocomposite Drainage Layer for Liner System

The leachate collection system includes a geocomposite drainage layer designed to collect and laterally convey percolating leachate to the leachate collection corridor. The geocomposite drainage layer of the liner system is modeled as a lateral drainage layer with *HELP* material texture 20 (representative of 0.2-in. thick geonet drainage layer) and having a slope of 2.25 percent and a length of 325 ft. The design hydraulic conductivity (k) of the drainage layer is calculated for each operational condition by varying the parameter *k* until the peak daily average head on the geomembrane liner is maintained within the drainage layer (i.e., equal to or less than the thickness of the geocomposite = 0.2 in.).

Geomembrane Liner for Liner System

The geomembrane liner is modeled as a flexible membrane liner with *HELP* material texture 35 (representative of HDPE geomembrane), installation condition = good, pinhole defect frequency = 2 per acre, and installation defect frequency = 2 per acre. This hole density is an assumption for design purposes only and does not imply the expected or allowable hole density.

GCL for Liner System

The GCL of the alternative liner system is modeled as a barrier soil liner with *HELP* default material texture 17 (representative of 0.25-in. thick bentonite mat with $k = 3 \times 10^{-9}$ cm/s). The compacted clay liner component of the alternative composite liner system is not modeled in *HELP* due to the inability of *HELP* to model two vertically contiguous barrier soil layers. The absence of the compacted clay liner component in the model is expected to yield conservative results (i.e., higher calculated rates of leachate percolation through the liner system) relative to a model that includes the clay liner component.

Compacted Clay Liner for Liner System

The compacted clay liner for the standard liner system is modeled as a barrier soil liner with *HELP* material texture 16 (representative of heavily compacted clay soil with $k = 1 \times 10^{-7}$ cm/s).

Landfill Design Data

The design data required by the *HELP* model consists of: (i) the slope and slope length of the surface of the top layer; (ii) vegetation and soil surface condition of the top layer; and (iii) the percentage of landfill area allowing runoff. These design data are summarized in Table E-1.2.

RESULTS OF ANALYSIS

The results of the *HELP* model analysis are summarized below. The output files are presented in Attachment E-1.A.

Design Hydraulic Conductivity and Transmissivity

The design hydraulic conductivity of the geosynthetic drainage layers for the leachate collection system and final cover system is evaluated for each operational condition as the minimum hydraulic conductivity for which the head on the liner system computed by *HELP* is less than the thickness of the geosynthetic drainage layer. The design hydraulic conductivity, design transmissivity, and calculated head on the liner for the leachate collection system and final cover system are summarized for the various operational conditions in Table E-1.3 and Table E-1.4, respectively. The design of the geocomposite drainage layer for the leachate collection system and final cover system is included in Appendix E-2 and Appendix G, respectively.

Leachate Collection Rates

The calculated leachate generation rates are shown in Table E-1.5. The leachate generation rates shown correspond to the lateral drainage collected from the leachate collection layer. Per the results of the *HELP* model, Case A has the highest leachate generation with annual average leachate generation of 75 gpad (gallons/acre/day [gpad]), peak daily leachate generation of 261 gpad, and peak monthly average leachate generation of 86 gpad. Cases D-1 and D-2 have the lowest leachate generation: leachate generation is essentially negligible, with annual average leachate generation of 0.06 gallons/acre/year. Thus annual average leachate, peak daily, and peak monthly average leachate generation rates for Cases D-1 and D-2 are shown as values of "0" in Table E-1.5.

Contact Water Generation

The calculated runoff rates are shown in Table E-1.6, and the active areas of the cells for the various operational conditions are listed in Table E-1.7. The contact water generation is generally higher during the initial stage of waste placement (Case A) because this stage comprises the largest active area for each cell. As waste filling progresses, the active area of each cell is reduced, resulting in smaller volumes of contact water generated. After construction of the final cover system, contact water is not expected to be generated in a cell.

Leachate Collection Volumes

The total volume of leachate generated is calculated for the various stages in the operational condition of the landfill cells, which are summarized in Table E-1.8. The volume of leachate generated is reported in terms of: (i) annual average flow (cubic feet [CF]), shown in Table E-1.9; (ii) peak daily flow (CF), shown in Table E-1.10; and (iii) peak monthly average flow (CF), i.e., the maximum of the set of monthly average flows reported for 12 months of the year, shown in Table E-1.11. The calculated maximum leachate is generated for the condition where final cover is constructed in Landfill Area 1 (corresponds to operational condition D-1 in Cells 1 to 3), Cells 4 and 5 have reached intermediate grades (corresponds to operational condition C-1), and Cells 6 and 7 have approximately 10 feet of waste (corresponds to operational condition A). The annual average leachate flow during this stage is estimated to be approximately 156,000 CF, the peak daily flow is estimated to be 1,300 CF, and the peak monthly average flow is estimated to be 15,400 CF.

Contact Water Volumes

Similar to leachate volumes, the volume of contact water generated is reported in terms of: (i) annual average flow (CF), shown in Table E-1.12; (ii) peak daily flow (CF), shown in Table E-1.13; and (iii) peak monthly average flow (CF), shown in Table E-1.14. The maximum contact water volume is generated during the condition where final cover is constructed in Landfill Area 1 (corresponds to operational condition D-1 in Cells 1 to 3) and Cells 4 and 5 have approximately 10 feet of waste (corresponds to operational condition A). The calculated annual average contact water generation during this stage is approximately 2,280,000 CF, the calculated peak daily flow is approximately 755,000 CF, and the calculated peak monthly average flow is 376,000 CF.

Leachate Percolation through Liner System

The calculated leachate percolation rates through the standard composite liner system and alternative composite liner system are very low. For Case A (initial condition with 10 ft of waste), a comparison between the standard composite liner system and alternative composite liner system was performed (Table E-1.15), keeping all other *HELP* model parameters equal. For the standard composite liner system, an average of 0.195 CF/acre/year of leachate is calculated to percolate through the liner system. For the alternative composite liner system, an average of 0.025 CF/acre/year of leachate is calculated to percolate through the liner system. The calculated leachate percolation rate for the proposed alternative liner system is lower than the standard liner system.

Leachate and Contact Water Collection Over the Life of the Facility

The total leachate and contact water generated for the different stages of operational condition of the cells (Table E-1.8) and presented in Tables E-1.9 to E-1.14 is summarized in Table E-1.16. The maximum volume is generated during the condition where final cover is constructed in Landfill Area 1 (corresponds to operational condition D-1 in Cells 1 to 3) and Cells 4 and 5 have approximately 10 feet of waste (corresponds to operational condition A). The calculated annual average volume during this stage (Stage 4) is approximately 2,408,000 CF, the calculated peak daily flow is approximately 756,000 CF, and the calculated peak monthly average flow is 388,000 CF.

SUMMARY AND CONCLUSIONS

The *HELP* model is used to estimate the leachate volume collected and conveyed in the geosynthetic drainage layer, the contact water generation rate, the maximum head on the liner system, and the quantity of leachate that may percolate through the liner system. It is also used to calculate the design in-plane hydraulic conductivity and transmissivity of the geosynthetic drainage layer. Representative parameters for various design and operational conditions that characterize the site over time are used in the model.

Results from the *HELP* model show that maximum peak daily leachate collection rate during the operational phase of the landfill is approximately 1,300 CF, while the annual average leachate generation rate is 156,000 CF. The calculated maximum peak daily generation of contact water is 755,000 CF while the annual average generation rate is 2,280,000 CF. For all operational cases, the calculated head of leachate on the liner is less than the thickness of the geocomposite drainage layer. The calculated leachate percolation rates through both the standard composite liner system and alternative composite liner system are very low.

REFERENCES

NOAA Hydrometeorological Design Studies Center. *Precipitation Frequency Data Server.* Retrieved May 06, 2016, from NOAA Atlas 14 Pont Precipitation Frequency Estimates (South Carolina): http://hdsc.nws.noaa.gov/hdsc/pfds/pfds_map_cont.html.

Schroeder, P.R., Lloyd, C.M., Zappi, P.A., and Aziz, N. M., "*The Hydrologic Evaluation of Landfill Performance (HELP) Model, User's Guide for Version 3*," U.S. Environmental Protection Agency, Office of Research and Development Washington, D.C., Report No. EPA/600/R094/168a, 1994a.

Schroeder, P.R., Dozier, T.S., Zappi, P.A., McEnroe, B.M., Sjostrom, J.W., and Peyton, R.L., "*The Hydrologic Evaluation of Landfill Performance (HELP) Model, Engineering Documentation for Version 3*," U.S. Environmental Protection Agency, Office of Research and Development, Washington, D.C., Report No. EPA/600/R-94/168b, 1994b, 116 p.

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TABLE E-1.1 – POINT PRECIPITATION FREQUENCY ESTIMATE FOR THE SITE

PF tabular

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

Duration	Average recurrence interval (years)									
	1	2	5	10	25	50	100	200	500	1000
5-min	0.502 (0.468–0.543)	0.586 (0.545–0.636)	0.668 (0.619–0.722)	0.761 (0.703–0.823)	0.857 (0.769–0.927)	0.942 (0.863–1.02)	1.02 (0.929–1.10)	1.10 (0.993–1.19)	1.19 (1.07–1.29)	1.28 (1.14–1.39)
10-min	0.803 (0.748–0.868)	0.938 (0.872–1.02)	1.07 (0.992–1.16)	1.22 (1.13–1.32)	1.37 (1.26–1.48)	1.50 (1.38–1.62)	1.62 (1.48–1.75)	1.74 (1.57–1.88)	1.89 (1.69–2.04)	2.02 (1.79–2.19)
15-min	1.00 (0.935–1.09)	1.18 (1.10–1.28)	1.35 (1.25–1.46)	1.54 (1.42–1.67)	1.73 (1.59–1.87)	1.90 (1.74–2.05)	2.05 (1.87–2.22)	2.19 (1.99–2.37)	2.37 (2.13–2.57)	2.53 (2.25–2.75)
30-min	1.38 (1.28–1.49)	1.63 (1.51–1.77)	1.92 (1.78–2.08)	2.23 (2.06–2.41)	2.57 (2.36–2.77)	2.86 (2.62–3.09)	3.14 (2.86–3.39)	3.42 (2.09–3.69)	3.78 (3.38–4.09)	4.10 (3.64–4.45)
60-min	1.72 (1.60–1.85)	2.04 (1.90–2.22)	2.46 (2.29–2.67)	2.90 (2.68–3.14)	3.42 (3.14–3.69)	3.88 (3.55–4.19)	4.32 (3.94–4.67)	4.79 (4.34–5.18)	5.42 (4.86–5.87)	5.98 (5.32–6.50)
2-hr	2.08 (1.92–2.24)	2.49 (2.30–2.69)	3.07 (2.83–3.31)	3.66 (3.37–3.96)	4.37 (4.00–4.72)	5.01 (4.56–5.40)	5.63 (5.10–6.07)	6.27 (5.64–6.76)	7.10 (6.33–7.68)	7.85 (6.93–8.51)
3-hr	2.23 (2.06–2.42)	2.66 (2.45–2.91)	3.30 (3.03–3.59)	3.97 (3.64–4.33)	4.79 (4.37–5.21)	5.55 (5.02–6.03)	6.31 (5.66–6.84)	7.10 (6.33–7.71)	8.19 (7.20–8.91)	9.18 (7.98–10.0)
6-hr	2.66 (2.44–2.91)	3.19 (2.91–3.49)	3.94 (3.60–4.32)	4.75 (4.33–5.20)	5.76 (5.21–6.30)	6.69 (6.01–7.31)	7.61 (6.79–8.32)	8.62 (7.62–9.41)	9.97 (8.71–10.9)	11.2 (9.69–12.3)
12-hr	3.11 (2.84–3.44)	3.72 (3.39–4.11)	4.63 (4.21–5.12)	5.62 (5.10–6.20)	6.85 (6.17–7.54)	8.00 (7.15–8.78)	9.17 (8.12–10.0)	10.4 (9.15–11.4)	12.2 (10.5–13.4)	13.8 (11.8–15.2)
24-hr	3.64 (3.33–3.99)	4.42 (4.06–4.85)	5.71 (5.22–6.26)	6.77 (6.17–7.41)	8.28 (7.50–9.05)	9.53 (8.59–10.4)	10.9 (9.74–11.9)	12.3 (11.0–13.5)	14.4 (12.7–15.8)	16.1 (14.1–17.7)
2-day	4.30 (3.94–4.70)	5.21 (4.78–5.70)	6.66 (6.10–7.27)	7.85 (7.18–8.57)	9.56 (8.68–10.4)	11.0 (9.90–12.0)	12.5 (11.2–13.6)	14.1 (12.6–15.4)	16.4 (14.5–18.1)	18.3 (16.0–20.2)
3-day	4.55 (4.18–4.98)	5.50 (5.06–6.03)	7.00 (6.41–7.65)	8.21 (7.50–8.97)	9.93 (9.03–10.8)	11.3 (10.3–12.4)	12.9 (11.6–14.1)	14.5 (12.9–15.8)	16.7 (14.8–18.4)	18.6 (16.4–20.5)
4-day	4.81 (4.42–5.27)	5.80 (5.33–6.36)	7.33 (6.72–8.02)	8.57 (7.83–9.37)	10.3 (9.37–11.3)	11.7 (10.6–12.8)	13.2 (11.9–14.5)	14.8 (13.3–16.2)	17.1 (15.2–18.8)	18.9 (16.7–20.9)
7-day	5.61 (5.19–6.10)	6.76 (6.25–7.35)	8.44 (7.78–9.16)	9.78 (9.00–10.6)	11.6 (10.7–12.6)	13.1 (12.0–14.2)	14.7 (13.3–16.0)	16.3 (14.8–17.8)	18.6 (16.7–20.3)	20.5 (18.3–22.4)
10-day	6.36 (5.91–6.87)	7.63 (7.08–8.23)	9.36 (8.68–10.1)	10.7 (9.92–11.5)	12.5 (11.6–13.5)	14.0 (12.9–15.1)	15.5 (14.2–16.7)	17.0 (15.5–18.4)	19.1 (17.3–20.7)	20.9 (18.8–22.6)
20-day	8.60 (8.05–9.20)	10.2 (9.59–11.0)	12.4 (11.6–13.2)	14.0 (13.1–15.0)	16.3 (15.2–17.4)	18.0 (16.7–19.3)	19.8 (18.3–21.2)	21.7 (20.0–23.2)	24.2 (22.1–26.0)	26.2 (23.8–28.2)
30-day	10.6 (9.98–11.2)	12.6 (11.8–13.3)	14.9 (14.0–15.8)	16.7 (15.7–17.7)	19.0 (17.8–20.1)	20.8 (19.5–22.1)	22.6 (21.1–24.0)	24.4 (22.7–26.0)	26.9 (24.8–28.6)	28.8 (26.5–30.7)
45-day	13.4 (12.7–14.2)	15.8 (15.0–16.8)	18.5 (17.5–19.6)	20.5 (19.3–21.7)	23.1 (21.8–24.5)	25.1 (23.6–26.6)	27.1 (25.4–28.7)	29.0 (27.1–30.9)	31.6 (29.4–33.7)	33.6 (31.1–35.9)
60-day	16.0 (15.1–16.9)	18.8 (17.8–19.9)	21.8 (20.6–23.0)	24.1 (22.7–25.4)	26.9 (25.3–28.4)	29.1 (27.3–30.7)	31.2 (29.3–33.0)	33.3 (31.1–35.3)	36.0 (33.6–38.3)	38.1 (35.4–40.6)

[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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TABLE E-1.2 – LANDFILL DESIGN DATA FOR HELP MODEL

Case	Percent of Area Allowing Runoff	Ground Conditions	Surface Slope (%)	Surface Slope Length (ft)	Surface Material Description	SCS Runoff Curve Number*	Evaporative Zone Depth	Maximum Leaf Area Index (LAI)
A	90	Bare	1	500	Fly Ash from Coal Plant	96.7	10	0
B	100	Bare	1	400	Fly Ash from Coal Plant	96.7	10	0
C-1	100	Bare	33.3	100	Fly Ash from Coal Plant	97.2	10	0
C-2	100	Bare	33.3	100	Fly Ash from Coal Plant	97.2	10	0
D-1	100	Fair	33.3	100	Sandy Clay	87.4	22	2
D-2	100	Fair	33.3	100	Sandy Clay	87.4	22	2

* calculated by *HELP* based on input

TABLE E-1.3 – DESIGN HYDRAULIC CONDUCTIVITY AND TRANSMISSIVITY FOR LEACHATE COLLECTION SYSTEM DRAINAGE LAYER

Case	Operational Condition	h_{waste} (ft)	$h_{soil\ cover}$ (ft)	k_{design} (cm/s)	$t_{geocomposite}$ (in.)	Design Transmissivity (m^2/s)	Peak Daily Average Head on liner (in.)
A	Initial Condition	10	0	0.12	0.2	6.2E-06	0.200
B	Intermediate Condition	30	0	4.5E-02	0.2	2.3E-06	0.190
C-1	Final Grades in Landfill Area 1 - No Cover	52	0	2.8E-02	0.2	1.4E-06	0.190
C-2	Final Grades in Landfill Area 2 - No Cover	88	0	2.4E-02	0.2	1.2E-06	0.194
D-1	Final Grades in Landfill Area 1 - With Cover	52	2	2.8E-03	0.2	1.4E-07	0.011
D-2	Final Grades in Landfill Area 2 - With Cover	88	2	2.4E-03	0.2	1.2E-07	0.012

Note: For Cases D-1 and D-2, head on liner is less than the thickness of geocomposite because, per HELP computation, leachate generation substantially reduces after construction of final cover system.

TABLE E-1.4 – DESIGN HYDRAULIC CONDUCTIVITY AND TRANSMISSIVITY FOR FINAL COVER SYSTEM DRAINAGE LAYER

Case	Operational Condition	h_{waste} (ft)	$h_{soil\ cover}$ (ft)	k_{design} (cm/s)	$t_{geocomposite}$ (in.)	Design Transmissivity (m^2/s)	Peak Daily Average Head on liner (in.)
D-1	Final Grades in Landfill Area 1 - With Cover	52	2	8.1E-01	0.2	4.1E-05	0.180
D-2	Final Grades in Landfill Area 2 - With Cover	88	2	8.1E-01	0.2	4.1E-05	0.180

TABLE E-1.5 – LEACHATE GENERATION RATES

Case	Average Annual Rates		Peak Daily Rates		Peak Monthly Average Rates		
	(in./ac./yr)	(gpad)	(in./ac./day)	(gpad)	Month	(in./ac./mo.)	(gpad)
A	1.01	75.4	9.6E-03	260.7	1	9.8E-02	85.8
B	0.48	35.9	3.3E-03	89.9	5	4.9E-02	43.1
C-1	0.37	27.3	2.1E-03	56.8	5	3.8E-02	33.3
C-2	0.33	24.7	1.8E-03	49.7	5	3.4E-02	30.1
D-1	0.00	0.0	0.00	0.0	-	0.00	0.0
D-2	0.00	0.0	0.00	0.0	-	0.00	0.0

Written by:	V. Krishnan	Date:	06/10/16	Reviewed by:	M. Christman	Date:	07/01/16
Client:	Santee Cooper	Project:	Winyah Generating Station	Project No.:	GSC5242	Task No.:	01BT

TABLE E-1.6 – RUNOFF RATES

Case	Average Annual Rates		Peak Daily Rates		Peak Monthly Average Rates		
	(in./ac./yr)	(gpad)	(in./ac./day)	(gpad)	Month	(in./ac./mo.)	(gpad)
A	20.3	1,509	6.7	182,232	8	3.3	2,923
B	22.6	1,680	7.5	203,630	8	3.7	3,261
C-1	24.9	1,851	7.6	207,703	8	4.1	3,548
C-2	24.9	1,851	7.6	207,703	8	4.1	3,548
D-1	5.1	380	5.4	145,438	8	1.0	833
D-2	5.1	380	5.4	145,438	8	1.0	833

TABLE E-1.7 – ACTIVE AREA OF CELLS DURING STAGES OF WASTE FILLING (ACRES)

Cell	Stages of Filling of Waste					
	A	B	C-1	C-2	D-1	D-2
Cell 1	9.3	6.5	3.8	-	0.0	-
Cell 2	8.6	6.1	3.5	-	0.0	-
Cell 3	9.2	6.5	3.8	-	0.0	-
Cell 4	15.2	11.6	0.0	0.0	-	0.0
Cell 5	15.8	12.1	0.0	0.0	-	0.0
Cell 6	15.9	13.8	-	0.0	-	0.0
Cell 7	12.8	11.2	-	0.0	-	0.0
Cell 8	10.2	9.1	-	0.0	-	0.0

Written by: **V. Krishnan** Date: **06/10/16** Reviewed by: **M. Christman** Date: **07/01/16**

Client: **Santee Cooper** Project: **Winyah Generating Station** Project No.: **GSC5242** Task No.: **01BT**

TABLE E-1.8 – WASTE FILLING AND PARTIAL CLOSURE SEQUENCE

Cell	Cell Area (acre)	Stage 1	Stage 2	Stage 3	Stage 4	Stage 5	Stage 6	Stage 7	Stage 8	Stage 9	Stage 10	Stage 11
Cell 1	10.7	A	B	C-1	D-1	D-1	D-1	D-1	D-1	D-1	D-1	D-1
Cell 2	9.9	A	B	C-1	D-1	D-1	D-1	D-1	D-1	D-1	D-1	D-1
Cell 3	10.6	A	B	C-1	D-1	D-1	D-1	D-1	D-1	D-1	D-1	D-1
Cell 4	16.6				A	B	C-1	C-1	C-2	C-2	C-2	D-2
Cell 5	17.3				A	B	C-1	C-1	C-2	C-2	C-2	D-2
Cell 6	16.6						A	B	C-2	C-2	C-2	D-2
Cell 7	13.4						A	B	C-2	C-2	C-2	D-2
Cell 8	11.4								A	B	C-2	D-2

TABLE E-1.9 – ANNUAL AVERAGE LEACHATE GENERATION (CF)

Cell	Cell Area (acre)	Stage 1	Stage 2	Stage 3	Stage 4	Stage 5	Stage 6	Stage 7	Stage 8	Stage 9	Stage 10	Stage 11
Cell 1	10.7	39,522	18,791	14,307	0	0	0	0	0	0	0	0
Cell 2	9.9	36,542	17,374	13,228	0	0	0	0	0	0	0	0
Cell 3	10.6	39,117	18,599	14,160	0	0	0	0	0	0	0	0
Cell 4	16.6	-	-	-	60,976	28,992	22,073	22,073	20,010	20,010	20,010	0
Cell 5	17.3	-	-	-	63,589	30,234	23,019	23,019	20,867	20,867	20,867	0
Cell 6	16.6	-	-	-	-	-	61,197	29,097	20,082	20,082	20,082	0
Cell 7	13.4	-	-	-	-	-	49,421	23,498	16,218	16,218	16,218	0
Cell 8	11.4	-	-	-	-	-	-	-	41,841	19,894	13,730	0
Total		115,181	54,764	41,695	124,565	59,226	155,710	97,686	119,018	97,071	90,908	0

Written by: **V. Krishnan** Date: **06/10/16** Reviewed by: **M. Christman** Date: **07/01/16**

Client: **Santee Cooper** Project: **Winyah Generating Station** Project No.: **GSC5242** Task No.: **01BT**

TABLE E-1.10 – PEAK DAILY LEACHATE GENERATION (CF)

Cell	Cell Area (acre)	Stage 1	Stage 2	Stage 3	Stage 4	Stage 5	Stage 6	Stage 7	Stage 8	Stage 9	Stage 10	Stage 11
Cell 1	10.7	374	129	81	0	0	0	0	0	0	0	0
Cell 2	9.9	346	119	75	0	0	0	0	0	0	0	0
Cell 3	10.6	370	128	81	0	0	0	0	0	0	0	0
Cell 4	16.6	-	-	-	577	199	126	126	110	110	110	0.0
Cell 5	17.3	-	-	-	602	208	131	131	115	115	115	0.0
Cell 6	16.6	-	-	-	-	-	580	200	110	110	110	0.0
Cell 7	13.4	-	-	-	-	-	468	161	89	89	89	0.0
Cell 8	11.4	-	-	-	-	-	-	-	396	137	76	0.0
Total		1,091	376	237	1,180	407	1,304	618	821	561	500	0

TABLE E-1.11 – PEAK MONTHLY AVERAGE LEACHATE GENERATION (CF)

Cell	Cell Area (acre)	Stage 1	Stage 2	Stage 3	Stage 4	Stage 5	Stage 6	Stage 7	Stage 8	Stage 9	Stage 10	Stage 11
Cell 1	10.7	3,817	1,918	1,481	0	0	0	0	0	0	0	0
Cell 2	9.9	3,529	1,773	1,370	0	0	0	0	0	0	0	0
Cell 3	10.6	3,778	1,898	1,466	0	0	0	0	0	0	0	0
Cell 4	16.6	-	-	-	5,889	2,959	2,286	2,286	2,069	2,069	2,069	0
Cell 5	17.3	-	-	-	6,141	3,086	2,384	2,384	2,158	2,158	2,158	0
Cell 6	16.6	-	-	-	-	-	5,910	2,970	2,077	2,077	2,077	0
Cell 7	13.4	-	-	-	-	-	4,773	2,399	1,677	1,677	1,677	0
Cell 8	11.4	-	-	-	-	-	-	-	4,041	2,031	1,420	0
Total		11,123	5,590	4,318	12,030	6,045	15,352	10,038	12,021	10,011	9,400	0

Written by: V. Krishnan Date: 06/10/16 Reviewed by: M. Christman Date: 07/01/16

Client: Santee Cooper Project: Winyah Generating Station Project No.: GSC5242 Task No.: 01BT

TABLE E-1.12 – ANNUAL AVERAGE CONTACT WATER GENERATION (CF)

Cell	Cell Area (acre)	Stage 1	Stage 2	Stage 3	Stage 4	Stage 5	Stage 6	Stage 7	Stage 8	Stage 9	Stage 10	Stage 11
Cell 1	10.7	683,497	535,982	343,222	0	0	0	0	0	0	0	0
Cell 2	9.9	631,940	495,825	317,932	0	0	0	0	0	0	0	0
Cell 3	10.6	676,868	531,065	339,609	0	0	0	0	0	0	0	0
Cell 4	16.6	-	-	-	1,118,048	952,311	0	0	0	0	0	0
Cell 5	17.3	-	-	-	1,165,186	993,289	0	0	0	0	0	0
Cell 6	16.6	-	-	-	-	-	1,168,868	1,134,250	0	0	0	0
Cell 7	13.4	-	-	-	-	-	943,491	915,432	0	0	0	0
Cell 8	11.4	-	-	-	-	-	-	-	753,688	745,458	0	0
Total		1,992,306	1,562,873	1,000,764	2,283,234	1,945,600	2,112,359	2,049,682	753,688	745,458	0	0

TABLE E-1.13 – PEAK DAILY CONTACT WATER GENERATION (CF)

Cell	Cell Area (acre)	Stage 1	Stage 2	Stage 3	Stage 4	Stage 5	Stage 6	Stage 7	Stage 8	Stage 9	Stage 10	Stage 11
Cell 1	10.7	226,069	178,028	105,510	0	0	0	0	0	0	0	0
Cell 2	9.9	209,017	164,689	97,736	0	0	0	0	0	0	0	0
Cell 3	10.6	223,877	176,394	104,400	0	0	0	0	0	0	0	0
Cell 4	16.6	-	-	-	369,799	316,312	0	0	0	0	0	0
Cell 5	17.3	-	-	-	385,390	329,923	0	0	0	0	0	0
Cell 6	16.6	-	-	-	-	-	386,608	376,744	0	0	0	0
Cell 7	13.4	-	-	-	-	-	312,064	304,063	0	0	0	0
Cell 8	11.4	-	-	-	-	-	-	-	249,285	247,606	0	0
Total		658,963	519,112	307,646	755,189	646,235	698,671	680,806	249,285	247,606	0	0

Written by:	V. Krishnan	Date:	06/10/16	Reviewed by:	M. Christman	Date:	07/01/16
Client:	Santee Cooper	Project:	Winyah Generating Station	Project No.:	GSC5242	Task No.:	01BT

TABLE E-1.14 – PEAK MONTHLY AVERAGE CONTACT WATER GENERATION (CF)

Cell	Cell Area (acre)	Stage 1	Stage 2	Stage 3	Stage 4	Stage 5	Stage 6	Stage 7	Stage 8	Stage 9	Stage 10	Stage 11
Cell 1	10.7	112,412	88,385	55,879	0	0	0	0	0	0	0	0
Cell 2	9.9	103,932	81,763	51,762	0	0	0	0	0	0	0	0
Cell 3	10.6	111,321	87,574	55,291	0	0	0	0	0	0	0	0
Cell 4	16.6	-	-	-	183,880	157,038	0	0	0	0	0	0
Cell 5	17.3	-	-	-	191,633	163,796	0	0	0	0	0	0
Cell 6	16.6	-	-	-	-	-	192,238	187,041	0	0	0	0
Cell 7	13.4	-	-	-	-	-	155,172	150,957	0	0	0	0
Cell 8	11.4	-	-	-	-	-	-	-	123,956	122,928	0	0
Total		327,665	257,721	162,933	375,513	320,834	347,410	337,997	123,956	122,928	0	0

Written by:	V. Krishnan	Date:	06/10/16	Reviewed by:	M. Christman	Date:	07/01/16
Client:	Santee Cooper	Project:	Winyah Generating Station	Project No.:	GSC5242	Task No.:	01BT

TABLE E-1.15 – COMPARISON OF ANNUAL AVERAGE PERCOLATION THROUGH LINER (CF)

HELP Model	Typer of Liner	Thickness (in.)	Permeability (cm/s)	Average Annual Percolation Through Liner (CF/acre)	Monthly Average Percolation Through Liner (CF/acre)
Case A	Geosynthetic Clay Liner	0.25	3×10^{-9}	0.025	2.08E-03
Case A - Std. Liner	Compacted Clay Liner	24	1×10^{-7}	0.195	1.63E-02

TABLE E-1.16 – SUMMARY OF TOTAL LEACHATE AND CONTACT WATER GENERATED DURING FILLING AND POST-CLOSURE (CF)

Stage	Annual Average	Peak Daily	Peak Monthly Average
Stage 1	2,107,487	660,054	338,788
Stage 2	1,617,637	519,488	263,311
Stage 3	1,042,459	307,883	167,250
Stage 4	2,407,799	756,368	387,542
Stage 5	2,004,826	646,642	326,879
Stage 6	2,268,069	699,976	362,762
Stage 7	2,147,369	681,424	348,035
Stage 8	872,706	250,106	135,977
Stage 9	842,529	248,167	132,939
Stage 10	90,908	500	9,400
Stage 11	0	0	0

APPENDIX E-1.A

HELP MODEL COMPUTER PROGRAM OUTPUT FILES

1. Case A – initial conditions: 10-ft of waste over the liner system
2. Case A-Std. – initial conditions: 10-ft of waste over the standard composite liner system
3. Case B – intermediate condition: 30-ft of waste over the liner system
4. Case C-1 – final grades in Landfill Area 1 before installing final cover: average of 52 ft of waste over the liner system
5. Case C-2 – final grades in Landfill Area 2 before installing final cover: average of 88 ft of waste over the liner system
6. Case D-1 – final grades in Landfill Area 1 after installing final cover: final cover system overlying an average of 52 ft of waste over the liner system
7. Case D-2 – final grades in Landfill Area 2 after installing final cover: final cover system overlying an average of 88 ft of waste over the liner system

Case A

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**                                                                          **
**                                                                          **
**              HYDROLOGIC EVALUATION OF LANDFILL PERFORMANCE               **
**                HELP MODEL VERSION 3.07  (1 NOVEMBER 1997)                **
**                 DEVELOPED BY ENVIRONMENTAL LABORATORY                    **
**                   USAE WATERWAYS EXPERIMENT STATION                      **
**             FOR USEPA RISK REDUCTION ENGINEERING LABORATORY              **
**                                                                          **
**                                                                          **
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PRECIPITATION DATA FILE:     C:\W14\DATA4.D4
TEMPERATURE DATA FILE:       C:\W14\DATA7.D7
SOLAR RADIATION DATA FILE:   C:\W14\DATA13.D13
EVAPOTRANSPIRATION DATA:     C:\W14\DATA11.D11
SOIL AND DESIGN DATA FILE:   C:\W14\DATA10.D10
OUTPUT DATA FILE:            C:\W14\OUT.OUT


TIME:  19:30     DATE:   6/ 6/2016


******************************************************************************

     TITLE:  Case A (Initial Condition)

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     NOTE:  INITIAL MOISTURE CONTENT OF THE LAYERS AND SNOW WATER WERE
              COMPUTED AS NEARLY STEADY-STATE VALUES BY THE PROGRAM.


                                   LAYER  1
                                   --------

                         TYPE 1 - VERTICAL PERCOLATION LAYER
                             MATERIAL TEXTURE NUMBER  30
           THICKNESS                     =    120.00    INCHES
           POROSITY                      =      0.5410 VOL/VOL
           FIELD CAPACITY                =      0.1870 VOL/VOL
           WILTING POINT                 =      0.0470 VOL/VOL
           INITIAL SOIL WATER CONTENT    =      0.2090 VOL/VOL
           EFFECTIVE SAT. HYD. COND.     =  0.499999987000E-04 CM/SEC


                                   LAYER  2
                                   --------

                         TYPE 1 - VERTICAL PERCOLATION LAYER
                             MATERIAL TEXTURE NUMBER  10
           THICKNESS                     =     24.00    INCHES
           POROSITY                      =      0.3980 VOL/VOL
```

Case A

```
            FIELD CAPACITY             =       0.2440 VOL/VOL
            WILTING POINT              =       0.1360 VOL/VOL
            INITIAL SOIL WATER CONTENT =       0.2440 VOL/VOL
            EFFECTIVE SAT. HYD. COND.  =  0.119999997000E-03 CM/SEC


                              LAYER  3
                              --------

                    TYPE 2 - LATERAL DRAINAGE LAYER
                      MATERIAL TEXTURE NUMBER   0
            THICKNESS                  =       0.20   INCHES
            POROSITY                   =       0.8500 VOL/VOL
            FIELD CAPACITY             =       0.0100 VOL/VOL
            WILTING POINT              =       0.0050 VOL/VOL
            INITIAL SOIL WATER CONTENT =       0.0509 VOL/VOL
            EFFECTIVE SAT. HYD. COND.  =  0.122500002000     CM/SEC
            SLOPE                      =       2.25   PERCENT
            DRAINAGE LENGTH            =     325.0    FEET


                              LAYER  4
                              --------

                    TYPE 4 - FLEXIBLE MEMBRANE LINER
                      MATERIAL TEXTURE NUMBER  35
            THICKNESS                  =       0.06   INCHES
            POROSITY                   =       0.0000 VOL/VOL
            FIELD CAPACITY             =       0.0000 VOL/VOL
            WILTING POINT              =       0.0000 VOL/VOL
            INITIAL SOIL WATER CONTENT =       0.0000 VOL/VOL
            EFFECTIVE SAT. HYD. COND.  =  0.199999996000E-12 CM/SEC
            FML PINHOLE DENSITY        =       2.00   HOLES/ACRE
            FML INSTALLATION DEFECTS   =       2.00   HOLES/ACRE
            FML PLACEMENT QUALITY      =  3 - GOOD


                              LAYER  5
                              --------

                      TYPE 3 - BARRIER SOIL LINER
                      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 =       0.7500 VOL/VOL
            EFFECTIVE SAT. HYD. COND.  =  0.300000003000E-08 CM/SEC


                  GENERAL DESIGN AND EVAPORATIVE ZONE DATA
                  ----------------------------------------

          NOTE:  SCS RUNOFF CURVE NUMBER WAS COMPUTED FROM DEFAULT
                   SOIL DATA BASE USING SOIL TEXTURE #30 WITH BARE
                   GROUND CONDITIONS, A SURFACE SLOPE OF  1.% AND
                   A SLOPE LENGTH OF  500. FEET.
```

Case A

SCS RUNOFF CURVE NUMBER	=	96.70	
FRACTION OF AREA ALLOWING RUNOFF	=	90.0	PERCENT
AREA PROJECTED ON HORIZONTAL PLANE	=	1.000	ACRES
EVAPORATIVE ZONE DEPTH	=	10.0	INCHES
INITIAL WATER IN EVAPORATIVE ZONE	=	3.371	INCHES
UPPER LIMIT OF EVAPORATIVE STORAGE	=	5.410	INCHES
LOWER LIMIT OF EVAPORATIVE STORAGE	=	0.470	INCHES
INITIAL SNOW WATER	=	0.000	INCHES
INITIAL WATER IN LAYER MATERIALS	=	31.132	INCHES
TOTAL INITIAL WATER	=	31.132	INCHES
TOTAL SUBSURFACE INFLOW	=	0.00	INCHES/YEAR

EVAPOTRANSPIRATION AND WEATHER DATA

NOTE: EVAPOTRANSPIRATION DATA WAS OBTAINED FROM
CHARLESTON SOUTH CAROLINA

STATION LATITUDE	=	32.90 DEGREES
MAXIMUM LEAF AREA INDEX	=	0.00
START OF GROWING SEASON (JULIAN DATE)	=	59
END OF GROWING SEASON (JULIAN DATE)	=	336
EVAPORATIVE ZONE DEPTH	=	10.0 INCHES
AVERAGE ANNUAL WIND SPEED	=	8.70 MPH
AVERAGE 1ST QUARTER RELATIVE HUMIDITY	=	70.00 %
AVERAGE 2ND QUARTER RELATIVE HUMIDITY	=	74.00 %
AVERAGE 3RD QUARTER RELATIVE HUMIDITY	=	80.00 %
AVERAGE 4TH QUARTER RELATIVE HUMIDITY	=	75.00 %

NOTE: PRECIPITATION DATA WAS SYNTHETICALLY GENERATED USING
COEFFICIENTS FOR CHARLESTON SOUTH CAROLINA

NORMAL MEAN MONTHLY PRECIPITATION (INCHES)

JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
3.33	3.37	4.38	2.58	4.41	6.54
7.33	6.50	4.94	2.92	2.18	3.11

NOTE: TEMPERATURE DATA WAS SYNTHETICALLY GENERATED USING
COEFFICIENTS FOR CHARLESTON SOUTH CAROLINA

NORMAL MEAN MONTHLY TEMPERATURE (DEGREES FAHRENHEIT)

JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
47.90	49.80	56.70	64.30	72.20	77.60
80.50	80.00	75.70	65.80	56.70	50.00

NOTE: SOLAR RADIATION DATA WAS SYNTHETICALLY GENERATED USING
COEFFICIENTS FOR CHARLESTON SOUTH CAROLINA
AND STATION LATITUDE = 32.90 DEGREES

Case A

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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	2.45	3.63	4.70	2.73	4.18	5.96
	7.34	7.26	4.83	2.68	2.00	2.89
STD. DEVIATIONS	1.74	1.59	2.10	1.69	2.34	3.23
	3.68	3.06	2.66	2.46	1.05	1.52
RUNOFF						
TOTALS	0.734	1.196	1.773	0.857	1.731	2.762
	3.229	3.337	2.053	1.115	0.562	0.941
STD. DEVIATIONS	0.911	0.842	1.176	0.755	1.495	2.074
	2.461	1.862	1.609	1.350	0.542	0.803
EVAPOTRANSPIRATION						
TOTALS	1.936	2.046	3.059	2.156	2.270	3.099
	3.718	3.711	2.814	1.808	1.296	1.428
STD. DEVIATIONS	0.440	0.431	0.713	0.944	0.982	1.178
	1.305	1.160	0.893	0.788	0.643	0.373
LATERAL DRAINAGE COLLECTED FROM LAYER 3						
TOTALS	0.0979	0.0867	0.0897	0.0867	0.0853	0.0809
	0.0786	0.0693	0.0695	0.0849	0.0887	0.0956
STD. DEVIATIONS	0.0619	0.0519	0.0551	0.0529	0.0541	0.0546
	0.0557	0.0508	0.0464	0.0576	0.0648	0.0661
PERCOLATION/LEAKAGE THROUGH LAYER 5						
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)

	JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
DAILY AVERAGE HEAD ON TOP OF LAYER 4						
AVERAGES	0.0657	0.0639	0.0602	0.0601	0.0573	0.0561
	0.0528	0.0465	0.0482	0.0570	0.0615	0.0642
STD. DEVIATIONS	0.0416	0.0384	0.0370	0.0367	0.0363	0.0379
	0.0374	0.0341	0.0322	0.0387	0.0449	0.0444

****

****

Case A

AVERAGE ANNUAL TOTALS & (STD. DEVIATIONS) FOR YEARS 1 THROUGH 30

	INCHES		CU. FEET	PERCENT
PRECIPITATION	50.67	(8.914)	183938.1	100.00
RUNOFF	20.290	(5.4753)	73651.98	40.042
EVAPOTRANSPIRATION	29.341	(3.2211)	106509.63	57.905
LATERAL DRAINAGE COLLECTED FROM LAYER 3	1.01375	(0.51502)	3679.909	2.00062
PERCOLATION/LEAKAGE THROUGH LAYER 5	0.00001	(0.00000)	0.025	0.00001
AVERAGE HEAD ON TOP OF LAYER 4	0.058	(0.029)		
CHANGE IN WATER STORAGE	0.027	(1.1871)	96.61	0.053

****

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PEAK DAILY VALUES FOR YEARS 1 THROUGH 30

	(INCHES)	(CU. FT.)
PRECIPITATION	8.28	30056.398
RUNOFF	6.711	24362.1406
DRAINAGE COLLECTED FROM LAYER 3	0.00960	34.83021
PERCOLATION/LEAKAGE THROUGH LAYER 5	0.000000	0.00021
AVERAGE HEAD ON TOP OF LAYER 4	0.200	
MAXIMUM HEAD ON TOP OF LAYER 4	0.391	
LOCATION OF MAXIMUM HEAD IN LAYER 3 (DISTANCE FROM DRAIN)	6.5 FEET	
SNOW WATER	1.03	3735.4602
MAXIMUM VEG. SOIL WATER (VOL/VOL)	0.3906	
MINIMUM VEG. SOIL WATER (VOL/VOL)	0.0470	

*** 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.

****

Case A

****

FINAL WATER STORAGE AT END OF YEAR 30

LAYER	(INCHES)	(VOL/VOL)
1	25.8536	0.2154
2	5.8560	0.2440
3	0.0332	0.1660
4	0.0000	0.0000
5	0.1875	0.7500
SNOW WATER	0.000	

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Case A_Std.

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**                                                                          **
**                                                                          **
**              HYDROLOGIC EVALUATION OF LANDFILL PERFORMANCE               **
**                HELP MODEL VERSION 3.07  (1 NOVEMBER 1997)                **
**                 DEVELOPED BY ENVIRONMENTAL LABORATORY                    **
**                   USAE WATERWAYS EXPERIMENT STATION                      **
**             FOR USEPA RISK REDUCTION ENGINEERING LABORATORY              **
**                                                                          **
**                                                                          **
******************************************************************************
******************************************************************************


PRECIPITATION DATA FILE:     C:\W15\DATA4.D4
TEMPERATURE DATA FILE:       C:\W15\DATA7.D7
SOLAR RADIATION DATA FILE:   C:\W15\DATA13.D13
EVAPOTRANSPIRATION DATA:     C:\W15\DATA11.D11
SOIL AND DESIGN DATA FILE:   C:\W15\DATA10.D10
OUTPUT DATA FILE:            C:\W15\OUT.OUT


TIME:  19:13     DATE:   6/ 7/2016


******************************************************************************

     TITLE:  Case A (Initial Condition with Standard Composite Liner)

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     NOTE:  INITIAL MOISTURE CONTENT OF THE LAYERS AND SNOW WATER WERE
              COMPUTED AS NEARLY STEADY-STATE VALUES BY THE PROGRAM.


                                   LAYER  1
                                   --------

                    TYPE 1 - VERTICAL PERCOLATION LAYER
                        MATERIAL TEXTURE NUMBER  30
           THICKNESS                 =    120.00   INCHES
           POROSITY                  =      0.5410 VOL/VOL
           FIELD CAPACITY            =      0.1870 VOL/VOL
           WILTING POINT             =      0.0470 VOL/VOL
           INITIAL SOIL WATER CONTENT =     0.2090 VOL/VOL
           EFFECTIVE SAT. HYD. COND.  =  0.499999987000E-04 CM/SEC


                                   LAYER  2
                                   --------

                    TYPE 1 - VERTICAL PERCOLATION LAYER
                        MATERIAL TEXTURE NUMBER  10
           THICKNESS                 =     24.00   INCHES
           POROSITY                  =      0.3980 VOL/VOL
```

Case A_Std.

```
          FIELD CAPACITY            =     0.2440 VOL/VOL
          WILTING POINT             =     0.1360 VOL/VOL
          INITIAL SOIL WATER CONTENT =    0.2440 VOL/VOL
          EFFECTIVE SAT. HYD. COND. = 0.119999997000E-03 CM/SEC


                            LAYER  3
                            --------

                  TYPE 2 - LATERAL DRAINAGE LAYER
                   MATERIAL TEXTURE NUMBER   0
          THICKNESS                 =     0.20   INCHES
          POROSITY                  =     0.8500 VOL/VOL
          FIELD CAPACITY            =     0.0100 VOL/VOL
          WILTING POINT             =     0.0050 VOL/VOL
          INITIAL SOIL WATER CONTENT =    0.0509 VOL/VOL
          EFFECTIVE SAT. HYD. COND. = 0.122500002000     CM/SEC
          SLOPE                     =     2.25   PERCENT
          DRAINAGE LENGTH           =   325.0    FEET


                            LAYER  4
                            --------

                  TYPE 4 - FLEXIBLE MEMBRANE LINER
                   MATERIAL TEXTURE NUMBER  35
          THICKNESS                 =     0.06   INCHES
          POROSITY                  =     0.0000 VOL/VOL
          FIELD CAPACITY            =     0.0000 VOL/VOL
          WILTING POINT             =     0.0000 VOL/VOL
          INITIAL SOIL WATER CONTENT =    0.0000 VOL/VOL
          EFFECTIVE SAT. HYD. COND. = 0.199999996000E-12 CM/SEC
          FML PINHOLE DENSITY       =     2.00   HOLES/ACRE
          FML INSTALLATION DEFECTS  =     2.00   HOLES/ACRE
          FML PLACEMENT QUALITY     = 3 - GOOD


                            LAYER  5
                            --------

                   TYPE 3 - BARRIER SOIL LINER
                   MATERIAL TEXTURE NUMBER  16
          THICKNESS                 =    24.00   INCHES
          POROSITY                  =     0.4270 VOL/VOL
          FIELD CAPACITY            =     0.4180 VOL/VOL
          WILTING POINT             =     0.3670 VOL/VOL
          INITIAL SOIL WATER CONTENT =    0.4270 VOL/VOL
          EFFECTIVE SAT. HYD. COND. = 0.100000001000E-06 CM/SEC


              GENERAL DESIGN AND EVAPORATIVE ZONE DATA
              ----------------------------------------

        NOTE:  SCS RUNOFF CURVE NUMBER WAS COMPUTED FROM DEFAULT
               SOIL DATA BASE USING SOIL TEXTURE #30 WITH BARE
               GROUND CONDITIONS, A SURFACE SLOPE OF  1.% AND
               A SLOPE LENGTH OF  500. FEET.
```

Case A_Std.

```
SCS RUNOFF CURVE NUMBER              =    96.70
FRACTION OF AREA ALLOWING RUNOFF     =    90.0    PERCENT
AREA PROJECTED ON HORIZONTAL PLANE   =     1.000  ACRES
EVAPORATIVE ZONE DEPTH               =    10.0    INCHES
INITIAL WATER IN EVAPORATIVE ZONE    =     3.371  INCHES
UPPER LIMIT OF EVAPORATIVE STORAGE   =     5.410  INCHES
LOWER LIMIT OF EVAPORATIVE STORAGE   =     0.470  INCHES
INITIAL SNOW WATER                   =     0.000  INCHES
INITIAL WATER IN LAYER MATERIALS     =    41.192  INCHES
TOTAL INITIAL WATER                  =    41.192  INCHES
TOTAL SUBSURFACE INFLOW              =     0.00   INCHES/YEAR
```

EVAPOTRANSPIRATION AND WEATHER DATA

NOTE: EVAPOTRANSPIRATION DATA WAS OBTAINED FROM
CHARLESTON SOUTH CAROLINA

```
STATION LATITUDE                          =  32.90 DEGREES
MAXIMUM LEAF AREA INDEX                   =   0.00
START OF GROWING SEASON (JULIAN DATE)     =     59
END OF GROWING SEASON (JULIAN DATE)       =    336
EVAPORATIVE ZONE DEPTH                    =  10.0  INCHES
AVERAGE ANNUAL WIND SPEED                 =   8.70 MPH
AVERAGE 1ST QUARTER RELATIVE HUMIDITY     =  70.00 %
AVERAGE 2ND QUARTER RELATIVE HUMIDITY     =  74.00 %
AVERAGE 3RD QUARTER RELATIVE HUMIDITY     =  80.00 %
AVERAGE 4TH QUARTER RELATIVE HUMIDITY     =  75.00 %
```

NOTE: PRECIPITATION DATA WAS SYNTHETICALLY GENERATED USING
COEFFICIENTS FOR CHARLESTON SOUTH CAROLINA

NORMAL MEAN MONTHLY PRECIPITATION (INCHES)

JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
3.33	3.37	4.38	2.58	4.41	6.54
7.33	6.50	4.94	2.92	2.18	3.11

NOTE: TEMPERATURE DATA WAS SYNTHETICALLY GENERATED USING
COEFFICIENTS FOR CHARLESTON SOUTH CAROLINA

NORMAL MEAN MONTHLY TEMPERATURE (DEGREES FAHRENHEIT)

JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
47.90	49.80	56.70	64.30	72.20	77.60
80.50	80.00	75.70	65.80	56.70	50.00

NOTE: SOLAR RADIATION DATA WAS SYNTHETICALLY GENERATED USING
COEFFICIENTS FOR CHARLESTON SOUTH CAROLINA
AND STATION LATITUDE = 32.90 DEGREES

Case A_Std.

****

AVERAGE MONTHLY VALUES IN INCHES FOR YEARS 1 THROUGH 30

	JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
PRECIPITATION						
TOTALS	2.45	3.63	4.70	2.73	4.18	5.96
	7.34	7.26	4.83	2.68	2.00	2.89
STD. DEVIATIONS	1.74	1.59	2.10	1.69	2.34	3.23
	3.68	3.06	2.66	2.46	1.05	1.52
RUNOFF						
TOTALS	0.734	1.196	1.773	0.857	1.731	2.762
	3.229	3.337	2.053	1.115	0.562	0.941
STD. DEVIATIONS	0.911	0.842	1.176	0.755	1.495	2.074
	2.461	1.862	1.609	1.350	0.542	0.803
EVAPOTRANSPIRATION						
TOTALS	1.936	2.046	3.059	2.156	2.270	3.099
	3.718	3.711	2.814	1.808	1.296	1.428
STD. DEVIATIONS	0.440	0.431	0.713	0.944	0.982	1.178
	1.305	1.160	0.893	0.788	0.643	0.373
LATERAL DRAINAGE COLLECTED FROM LAYER 3						
TOTALS	0.0979	0.0867	0.0897	0.0867	0.0853	0.0809
	0.0786	0.0693	0.0695	0.0849	0.0887	0.0956
STD. DEVIATIONS	0.0619	0.0519	0.0551	0.0529	0.0541	0.0546
	0.0557	0.0508	0.0464	0.0576	0.0648	0.0661
PERCOLATION/LEAKAGE THROUGH LAYER 5						
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)

	JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
DAILY AVERAGE HEAD ON TOP OF LAYER 4						
AVERAGES	0.0657	0.0639	0.0602	0.0601	0.0573	0.0561
	0.0528	0.0465	0.0482	0.0570	0.0615	0.0642
STD. DEVIATIONS	0.0416	0.0384	0.0370	0.0367	0.0363	0.0379
	0.0374	0.0341	0.0322	0.0387	0.0449	0.0444

****

****

Case A_Std.

AVERAGE ANNUAL TOTALS & (STD. DEVIATIONS) FOR YEARS 1 THROUGH 30

	INCHES		CU. FEET	PERCENT
PRECIPITATION	50.67	(8.914)	183938.1	100.00
RUNOFF	20.290	(5.4753)	73651.98	40.042
EVAPOTRANSPIRATION	29.341	(3.2211)	106509.63	57.905
LATERAL DRAINAGE COLLECTED FROM LAYER 3	1.01370	(0.51500)	3679.740	2.00053
PERCOLATION/LEAKAGE THROUGH LAYER 5	0.00005	(0.00002)	0.195	0.00011
AVERAGE HEAD ON TOP OF LAYER 4	0.058	(0.029)		
CHANGE IN WATER STORAGE	0.027	(1.1871)	96.61	0.053

****

****

PEAK DAILY VALUES FOR YEARS 1 THROUGH 30

	(INCHES)	(CU. FT.)
PRECIPITATION	8.28	30056.398
RUNOFF	6.711	24362.1406
DRAINAGE COLLECTED FROM LAYER 3	0.00959	34.82882
PERCOLATION/LEAKAGE THROUGH LAYER 5	0.000000	0.00165
AVERAGE HEAD ON TOP OF LAYER 4	0.200	
MAXIMUM HEAD ON TOP OF LAYER 4	0.391	
LOCATION OF MAXIMUM HEAD IN LAYER 3 (DISTANCE FROM DRAIN)	6.4 FEET	
SNOW WATER	1.03	3735.4602
MAXIMUM VEG. SOIL WATER (VOL/VOL)	0.3906	
MINIMUM VEG. SOIL WATER (VOL/VOL)	0.0470	

*** 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.

****

Case A_Std.

****

FINAL WATER STORAGE AT END OF YEAR 30

LAYER	(INCHES)	(VOL/VOL)
1	25.8536	0.2154
2	5.8560	0.2440
3	0.0332	0.1660
4	0.0000	0.0000
5	10.2480	0.4270
SNOW WATER	0.000	

****
****

Case B

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**                                                                          **
**                                                                          **
**             HYDROLOGIC EVALUATION OF LANDFILL PERFORMANCE                **
**               HELP MODEL VERSION 3.07  (1 NOVEMBER 1997)                 **
**               DEVELOPED BY ENVIRONMENTAL LABORATORY                      **
**                 USAE WATERWAYS EXPERIMENT STATION                        **
**           FOR USEPA RISK REDUCTION ENGINEERING LABORATORY                **
**                                                                          **
**                                                                          **
******************************************************************************
******************************************************************************


PRECIPITATION DATA FILE:     C:\W24\DATA4.D4
TEMPERATURE DATA FILE:       C:\W24\DATA7.D7
SOLAR RADIATION DATA FILE:   C:\W24\DATA13.D13
EVAPOTRANSPIRATION DATA:     C:\W24\DATA11.D11
SOIL AND DESIGN DATA FILE:   C:\W24\DATA10.D10
OUTPUT DATA FILE:            C:\W24\OUT.OUT


TIME:  12:30     DATE:   6/ 7/2016


******************************************************************************

     TITLE:  Case B (Intermediate Condition)

******************************************************************************


     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  30
          THICKNESS                  =   360.00   INCHES
          POROSITY                   =     0.5410 VOL/VOL
          FIELD CAPACITY             =     0.1870 VOL/VOL
          WILTING POINT              =     0.0470 VOL/VOL
          INITIAL SOIL WATER CONTENT =     0.1931 VOL/VOL
          EFFECTIVE SAT. HYD. COND.  = 0.499999987000E-04 CM/SEC


                                  LAYER  2
                                  --------

                    TYPE 1 - VERTICAL PERCOLATION LAYER
                        MATERIAL TEXTURE NUMBER  10
          THICKNESS                  =    24.00   INCHES
          POROSITY                   =     0.3980 VOL/VOL
```

Case B

```
FIELD CAPACITY             =      0.2440 VOL/VOL
WILTING POINT              =      0.1360 VOL/VOL
INITIAL SOIL WATER CONTENT =      0.2440 VOL/VOL
EFFECTIVE SAT. HYD. COND.  =  0.119999997000E-03 CM/SEC


                      LAYER  3
                      --------

           TYPE 2 - LATERAL DRAINAGE LAYER
             MATERIAL TEXTURE NUMBER   0
THICKNESS                  =      0.20   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. COND.  =  0.445000008000E-01 CM/SEC
SLOPE                      =      2.25   PERCENT
DRAINAGE LENGTH            =    325.0    FEET


                      LAYER  4
                      --------

           TYPE 4 - FLEXIBLE MEMBRANE LINER
             MATERIAL TEXTURE NUMBER  35
THICKNESS                  =      0.06   INCHES
POROSITY                   =      0.0000 VOL/VOL
FIELD CAPACITY             =      0.0000 VOL/VOL
WILTING POINT              =      0.0000 VOL/VOL
INITIAL SOIL WATER CONTENT =      0.0000 VOL/VOL
EFFECTIVE SAT. HYD. COND.  =  0.199999996000E-12 CM/SEC
FML PINHOLE DENSITY        =      2.00   HOLES/ACRE
FML INSTALLATION DEFECTS   =      2.00   HOLES/ACRE
FML PLACEMENT QUALITY      =  3 - GOOD


                      LAYER  5
                      --------

             TYPE 3 - BARRIER SOIL LINER
             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 =      0.7500 VOL/VOL
EFFECTIVE SAT. HYD. COND.  =  0.300000003000E-08 CM/SEC


         GENERAL DESIGN AND EVAPORATIVE ZONE DATA
         ----------------------------------------

NOTE:  SCS RUNOFF CURVE NUMBER WAS COMPUTED FROM DEFAULT
         SOIL DATA BASE USING SOIL TEXTURE #30 WITH BARE
         GROUND CONDITIONS, A SURFACE SLOPE OF  1.% AND
         A SLOPE LENGTH OF  400. FEET.
```

Case B

SCS RUNOFF CURVE NUMBER	=	96.70	
FRACTION OF AREA ALLOWING RUNOFF	=	100.0	PERCENT
AREA PROJECTED ON HORIZONTAL PLANE	=	1.000	ACRES
EVAPORATIVE ZONE DEPTH	=	10.0	INCHES
INITIAL WATER IN EVAPORATIVE ZONE	=	3.268	INCHES
UPPER LIMIT OF EVAPORATIVE STORAGE	=	5.410	INCHES
LOWER LIMIT OF EVAPORATIVE STORAGE	=	0.470	INCHES
INITIAL SNOW WATER	=	0.000	INCHES
INITIAL WATER IN LAYER MATERIALS	=	75.579	INCHES
TOTAL INITIAL WATER	=	75.579	INCHES
TOTAL SUBSURFACE INFLOW	=	0.00	INCHES/YEAR

EVAPOTRANSPIRATION AND WEATHER DATA

NOTE: EVAPOTRANSPIRATION DATA WAS OBTAINED FROM
CHARLESTON SOUTH CAROLINA

STATION LATITUDE	=	32.90 DEGREES
MAXIMUM LEAF AREA INDEX	=	0.00
START OF GROWING SEASON (JULIAN DATE)	=	59
END OF GROWING SEASON (JULIAN DATE)	=	336
EVAPORATIVE ZONE DEPTH	=	10.0 INCHES
AVERAGE ANNUAL WIND SPEED	=	8.70 MPH
AVERAGE 1ST QUARTER RELATIVE HUMIDITY	=	70.00 %
AVERAGE 2ND QUARTER RELATIVE HUMIDITY	=	74.00 %
AVERAGE 3RD QUARTER RELATIVE HUMIDITY	=	80.00 %
AVERAGE 4TH QUARTER RELATIVE HUMIDITY	=	75.00 %

NOTE: PRECIPITATION DATA WAS SYNTHETICALLY GENERATED USING
COEFFICIENTS FOR CHARLESTON SOUTH CAROLINA

NORMAL MEAN MONTHLY PRECIPITATION (INCHES)

JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
3.33	3.37	4.38	2.58	4.41	6.54
7.33	6.50	4.94	2.92	2.18	3.11

NOTE: TEMPERATURE DATA WAS SYNTHETICALLY GENERATED USING
COEFFICIENTS FOR CHARLESTON SOUTH CAROLINA

NORMAL MEAN MONTHLY TEMPERATURE (DEGREES FAHRENHEIT)

JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
47.90	49.80	56.70	64.30	72.20	77.60
80.50	80.00	75.70	65.80	56.70	50.00

NOTE: SOLAR RADIATION DATA WAS SYNTHETICALLY GENERATED USING
COEFFICIENTS FOR CHARLESTON SOUTH CAROLINA
AND STATION LATITUDE = 32.90 DEGREES

Case B

****

AVERAGE MONTHLY VALUES IN INCHES FOR YEARS 1 THROUGH 30

	JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
PRECIPITATION						
TOTALS	2.45	3.63	4.70	2.73	4.18	5.96
	7.34	7.26	4.83	2.68	2.00	2.89
STD. DEVIATIONS	1.74	1.59	2.10	1.69	2.34	3.23
	3.68	3.06	2.66	2.46	1.05	1.52
RUNOFF						
TOTALS	0.812	1.329	1.962	0.957	1.913	3.063
	3.615	3.723	2.285	1.241	0.627	1.051
STD. DEVIATIONS	1.001	0.929	1.290	0.840	1.641	2.282
	2.727	2.049	1.774	1.491	0.602	0.892
EVAPOTRANSPIRATION						
TOTALS	1.871	1.970	2.964	1.986	2.166	2.821
	3.374	3.376	2.643	1.655	1.233	1.425
STD. DEVIATIONS	0.462	0.456	0.707	0.863	0.929	1.079
	1.153	1.063	0.877	0.748	0.627	0.372
LATERAL DRAINAGE COLLECTED FROM LAYER 3						
TOTALS	0.0431	0.0414	0.0475	0.0472	0.0492	0.0434
	0.0387	0.0329	0.0299	0.0327	0.0354	0.0406
STD. DEVIATIONS	0.0255	0.0209	0.0224	0.0228	0.0249	0.0245
	0.0240	0.0217	0.0199	0.0205	0.0224	0.0250
PERCOLATION/LEAKAGE THROUGH LAYER 5						
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)

	JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
DAILY AVERAGE HEAD ON TOP OF LAYER 4						
AVERAGES	0.0796	0.0840	0.0878	0.0902	0.0909	0.0829
	0.0715	0.0608	0.0572	0.0604	0.0676	0.0749
STD. DEVIATIONS	0.0471	0.0424	0.0413	0.0435	0.0460	0.0468
	0.0443	0.0401	0.0381	0.0380	0.0427	0.0461

****

****

Case B

AVERAGE ANNUAL TOTALS & (STD. DEVIATIONS) FOR YEARS 1 THROUGH 30

	INCHES		CU. FEET	PERCENT
PRECIPITATION	50.67	(8.914)	183938.1	100.00
RUNOFF	22.577	(6.0448)	81955.90	44.556
EVAPOTRANSPIRATION	27.485	(2.9890)	99770.14	54.241
LATERAL DRAINAGE COLLECTED FROM LAYER 3	0.48200	(0.20368)	1749.662	0.95122
PERCOLATION/LEAKAGE THROUGH LAYER 5	0.00001	(0.00000)	0.032	0.00002
AVERAGE HEAD ON TOP OF LAYER 4	0.076	(0.032)		
CHANGE IN WATER STORAGE	0.127	(0.8450)	462.46	0.251

****

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PEAK DAILY VALUES FOR YEARS 1 THROUGH 30

	(INCHES)	(CU. FT.)
PRECIPITATION	8.28	30056.398
RUNOFF	7.499	27222.4531
DRAINAGE COLLECTED FROM LAYER 3	0.00331	12.01820
PERCOLATION/LEAKAGE THROUGH LAYER 5	0.000000	0.00020
AVERAGE HEAD ON TOP OF LAYER 4	0.190	
MAXIMUM HEAD ON TOP OF LAYER 4	0.372	
LOCATION OF MAXIMUM HEAD IN LAYER 3 (DISTANCE FROM DRAIN)	6.1 FEET	
SNOW WATER	1.03	3735.4602

MAXIMUM VEG. SOIL WATER (VOL/VOL)	0.3774
MINIMUM VEG. SOIL WATER (VOL/VOL)	0.0583

*** 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.

****

Case B

FINAL WATER STORAGE AT END OF YEAR 30

LAYER	(INCHES)	(VOL/VOL)
1	73.3006	0.2036
2	5.8560	0.2440
3	0.0574	0.2870
4	0.0000	0.0000
5	0.1875	0.7500
SNOW WATER	0.000	

Case C-1

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**                                                                          **
**                                                                          **
**              HYDROLOGIC EVALUATION OF LANDFILL PERFORMANCE               **
**                HELP MODEL VERSION 3.07  (1 NOVEMBER 1997)                **
**                DEVELOPED BY ENVIRONMENTAL LABORATORY                     **
**                  USAE WATERWAYS EXPERIMENT STATION                       **
**             FOR USEPA RISK REDUCTION ENGINEERING LABORATORY              **
**                                                                          **
**                                                                          **
******************************************************************************
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PRECIPITATION DATA FILE:     C:\W34\DATA4.D4
TEMPERATURE DATA FILE:       C:\W34\DATA7.D7
SOLAR RADIATION DATA FILE:   C:\W34\DATA13.D13
EVAPOTRANSPIRATION DATA:     C:\W34\DATA11.D11
SOIL AND DESIGN DATA FILE:   C:\W34\DATA10.D10
OUTPUT DATA FILE:            C:\W34\OUT.OUT


TIME:  12:36     DATE:   6/ 7/2016


******************************************************************************

     TITLE:  Case C-1 (Final Grades without Cover in Landfill Area 1)

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     NOTE:  INITIAL MOISTURE CONTENT OF THE LAYERS AND SNOW WATER WERE
              COMPUTED AS NEARLY STEADY-STATE VALUES BY THE PROGRAM.


                                   LAYER  1
                                   --------

                      TYPE 1 - VERTICAL PERCOLATION LAYER
                          MATERIAL TEXTURE NUMBER  30
          THICKNESS                     =   624.00   INCHES
          POROSITY                      =     0.5410 VOL/VOL
          FIELD CAPACITY                =     0.1870 VOL/VOL
          WILTING POINT                 =     0.0470 VOL/VOL
          INITIAL SOIL WATER CONTENT    =     0.1906 VOL/VOL
          EFFECTIVE SAT. HYD. COND.     = 0.499999987000E-04 CM/SEC


                                   LAYER  2
                                   --------

                      TYPE 1 - VERTICAL PERCOLATION LAYER
                          MATERIAL TEXTURE NUMBER  10
          THICKNESS                     =    24.00   INCHES
          POROSITY                      =     0.3980 VOL/VOL
```

Case C-1

```
FIELD CAPACITY              =      0.2440 VOL/VOL
WILTING POINT               =      0.1360 VOL/VOL
INITIAL SOIL WATER CONTENT  =      0.2440 VOL/VOL
EFFECTIVE SAT. HYD. COND.   =  0.119999997000E-03 CM/SEC


                    LAYER  3
                    --------

          TYPE 2 - LATERAL DRAINAGE LAYER
            MATERIAL TEXTURE NUMBER   0
THICKNESS                   =      0.20   INCHES
POROSITY                    =      0.8500 VOL/VOL
FIELD CAPACITY              =      0.0100 VOL/VOL
WILTING POINT               =      0.0050 VOL/VOL
INITIAL SOIL WATER CONTENT  =      0.0452 VOL/VOL
EFFECTIVE SAT. HYD. COND.   =  0.280000009000E-01 CM/SEC
SLOPE                       =      2.25   PERCENT
DRAINAGE LENGTH             =    325.0    FEET


                    LAYER  4
                    --------

          TYPE 4 - FLEXIBLE MEMBRANE LINER
            MATERIAL TEXTURE NUMBER  35
THICKNESS                   =      0.06   INCHES
POROSITY                    =      0.0000 VOL/VOL
FIELD CAPACITY              =      0.0000 VOL/VOL
WILTING POINT               =      0.0000 VOL/VOL
INITIAL SOIL WATER CONTENT  =      0.0000 VOL/VOL
EFFECTIVE SAT. HYD. COND.   =  0.199999996000E-12 CM/SEC
FML PINHOLE DENSITY         =      2.00   HOLES/ACRE
FML INSTALLATION DEFECTS    =      2.00   HOLES/ACRE
FML PLACEMENT QUALITY       =  3 - GOOD


                    LAYER  5
                    --------

            TYPE 3 - BARRIER SOIL LINER
            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  =      0.7500 VOL/VOL
EFFECTIVE SAT. HYD. COND.   =  0.300000003000E-08 CM/SEC


        GENERAL DESIGN AND EVAPORATIVE ZONE DATA
        ----------------------------------------

NOTE:  SCS RUNOFF CURVE NUMBER WAS COMPUTED FROM DEFAULT
         SOIL DATA BASE USING SOIL TEXTURE #30 WITH BARE
         GROUND CONDITIONS, A SURFACE SLOPE OF 33.% AND
         A SLOPE LENGTH OF  100. FEET.
```

Case C-1

```
SCS RUNOFF CURVE NUMBER                  =    97.20
FRACTION OF AREA ALLOWING RUNOFF         =   100.0    PERCENT
AREA PROJECTED ON HORIZONTAL PLANE       =     1.000  ACRES
EVAPORATIVE ZONE DEPTH                   =    10.0    INCHES
INITIAL WATER IN EVAPORATIVE ZONE        =     3.184  INCHES
UPPER LIMIT OF EVAPORATIVE STORAGE       =     5.410  INCHES
LOWER LIMIT OF EVAPORATIVE STORAGE       =     0.470  INCHES
INITIAL SNOW WATER                       =     0.000  INCHES
INITIAL WATER IN LAYER MATERIALS         =   124.981  INCHES
TOTAL INITIAL WATER                      =   124.981  INCHES
TOTAL SUBSURFACE INFLOW                  =     0.00   INCHES/YEAR
```

EVAPOTRANSPIRATION AND WEATHER DATA

NOTE: EVAPOTRANSPIRATION DATA WAS OBTAINED FROM
CHARLESTON SOUTH CAROLINA

```
STATION LATITUDE                         =  32.90 DEGREES
MAXIMUM LEAF AREA INDEX                  =   0.00
START OF GROWING SEASON (JULIAN DATE)    =     59
END OF GROWING SEASON (JULIAN DATE)      =    336
EVAPORATIVE ZONE DEPTH                   =  10.0  INCHES
AVERAGE ANNUAL WIND SPEED                =   8.70 MPH
AVERAGE 1ST QUARTER RELATIVE HUMIDITY    =  70.00 %
AVERAGE 2ND QUARTER RELATIVE HUMIDITY    =  74.00 %
AVERAGE 3RD QUARTER RELATIVE HUMIDITY    =  80.00 %
AVERAGE 4TH QUARTER RELATIVE HUMIDITY    =  75.00 %
```

NOTE: PRECIPITATION DATA WAS SYNTHETICALLY GENERATED USING
COEFFICIENTS FOR CHARLESTON SOUTH CAROLINA

NORMAL MEAN MONTHLY PRECIPITATION (INCHES)

JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
3.33	3.37	4.38	2.58	4.41	6.54
7.33	6.50	4.94	2.92	2.18	3.11

NOTE: TEMPERATURE DATA WAS SYNTHETICALLY GENERATED USING
COEFFICIENTS FOR CHARLESTON SOUTH CAROLINA

NORMAL MEAN MONTHLY TEMPERATURE (DEGREES FAHRENHEIT)

JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
47.90	49.80	56.70	64.30	72.20	77.60
80.50	80.00	75.70	65.80	56.70	50.00

NOTE: SOLAR RADIATION DATA WAS SYNTHETICALLY GENERATED USING
COEFFICIENTS FOR CHARLESTON SOUTH CAROLINA
AND STATION LATITUDE = 32.90 DEGREES

Case C-1

****

AVERAGE MONTHLY VALUES IN INCHES FOR YEARS 1 THROUGH 30

	JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
PRECIPITATION						
TOTALS	2.45	3.63	4.70	2.73	4.18	5.96
	7.34	7.26	4.83	2.68	2.00	2.89
STD. DEVIATIONS	1.74	1.59	2.10	1.69	2.34	3.23
	3.68	3.06	2.66	2.46	1.05	1.52
RUNOFF						
TOTALS	0.922	1.495	2.172	1.093	2.119	3.314
	3.942	4.051	2.508	1.360	0.720	1.184
STD. DEVIATIONS	1.069	0.987	1.370	0.917	1.723	2.383
	2.825	2.162	1.867	1.605	0.642	0.959
EVAPOTRANSPIRATION						
TOTALS	1.755	1.884	2.760	1.798	1.955	2.621
	3.088	3.071	2.445	1.502	1.136	1.354
STD. DEVIATIONS	0.451	0.469	0.736	0.781	0.831	0.958
	1.035	0.937	0.827	0.667	0.594	0.387
LATERAL DRAINAGE COLLECTED FROM LAYER 3						
TOTALS	0.0290	0.0289	0.0362	0.0372	0.0380	0.0345
	0.0311	0.0276	0.0251	0.0257	0.0257	0.0279
STD. DEVIATIONS	0.0165	0.0136	0.0145	0.0154	0.0175	0.0168
	0.0157	0.0152	0.0147	0.0154	0.0150	0.0162
PERCOLATION/LEAKAGE THROUGH LAYER 5						
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)

	JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
DAILY AVERAGE HEAD ON TOP OF LAYER 4						
AVERAGES	0.0853	0.0932	0.1062	0.1128	0.1115	0.1046
	0.0914	0.0811	0.0763	0.0755	0.0781	0.0820
STD. DEVIATIONS	0.0484	0.0436	0.0426	0.0466	0.0513	0.0510
	0.0462	0.0447	0.0446	0.0453	0.0456	0.0475

****

****

Case C-1

AVERAGE ANNUAL TOTALS & (STD. DEVIATIONS) FOR YEARS 1 THROUGH 30

	INCHES		CU. FEET	PERCENT
PRECIPITATION	50.67	(8.914)	183938.1	100.00
RUNOFF	24.882	(6.3226)	90320.00	49.103
EVAPOTRANSPIRATION	25.369	(2.8056)	92090.56	50.066
LATERAL DRAINAGE COLLECTED FROM LAYER 3	0.36697	(0.14081)	1332.105	0.72421
PERCOLATION/LEAKAGE THROUGH LAYER 5	0.00001	(0.00000)	0.037	0.00002
AVERAGE HEAD ON TOP OF LAYER 4	0.092	(0.035)		
CHANGE IN WATER STORAGE	0.054	(0.6756)	195.46	0.106

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PEAK DAILY VALUES FOR YEARS 1 THROUGH 30

	(INCHES)	(CU. FT.)
PRECIPITATION	8.28	30056.398
RUNOFF	7.649	27765.3066
DRAINAGE COLLECTED FROM LAYER 3	0.00209	7.59205
PERCOLATION/LEAKAGE THROUGH LAYER 5	0.000000	0.00020
AVERAGE HEAD ON TOP OF LAYER 4	0.190	
MAXIMUM HEAD ON TOP OF LAYER 4	0.373	
LOCATION OF MAXIMUM HEAD IN LAYER 3 (DISTANCE FROM DRAIN)	6.2 FEET	
SNOW WATER	1.03	3735.4602
MAXIMUM VEG. SOIL WATER (VOL/VOL)	0.3684	
MINIMUM VEG. SOIL WATER (VOL/VOL)	0.0520	

*** 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.

****

Case C-1

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FINAL WATER STORAGE AT END OF YEAR 30

LAYER	(INCHES)	(VOL/VOL)
1	120.4760	0.1931
2	5.8560	0.2440
3	0.0764	0.3820
4	0.0000	0.0000
5	0.1875	0.7500
SNOW WATER	0.000	

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Case C-2

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**                                                                          **
**                                                                          **
**              HYDROLOGIC EVALUATION OF LANDFILL PERFORMANCE               **
**                HELP MODEL VERSION 3.07  (1 NOVEMBER 1997)                **
**                DEVELOPED BY ENVIRONMENTAL LABORATORY                     **
**                  USAE WATERWAYS EXPERIMENT STATION                       **
**              FOR USEPA RISK REDUCTION ENGINEERING LABORATORY             **
**                                                                          **
**                                                                          **
******************************************************************************
******************************************************************************


PRECIPITATION DATA FILE:     C:\W44\DATA4.D4
TEMPERATURE DATA FILE:       C:\W44\DATA7.D7
SOLAR RADIATION DATA FILE:   C:\W44\DATA13.D13
EVAPOTRANSPIRATION DATA:     C:\W44\DATA11.D11
SOIL AND DESIGN DATA FILE:   C:\W44\DATA10.D10
OUTPUT DATA FILE:            C:\W44\OUT.OUT


TIME:  12:39     DATE:   6/ 7/2016


******************************************************************************

     TITLE:  Case C-2 (Final Grades without Cover in Landfill Area 2)

******************************************************************************


     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  30
           THICKNESS                 =   1056.00   INCHES
           POROSITY                  =      0.5410 VOL/VOL
           FIELD CAPACITY            =      0.1870 VOL/VOL
           WILTING POINT             =      0.0470 VOL/VOL
           INITIAL SOIL WATER CONTENT =     0.1891 VOL/VOL
           EFFECTIVE SAT. HYD. COND. =  0.499999987000E-04 CM/SEC


                                  LAYER  2
                                  --------

                      TYPE 1 - VERTICAL PERCOLATION LAYER
                          MATERIAL TEXTURE NUMBER  10
           THICKNESS                 =     24.00   INCHES
           POROSITY                  =      0.3980 VOL/VOL
```

Case C-2

```
FIELD CAPACITY              =      0.2440 VOL/VOL
WILTING POINT               =      0.1360 VOL/VOL
INITIAL SOIL WATER CONTENT  =      0.2440 VOL/VOL
EFFECTIVE SAT. HYD. COND.   =  0.119999997000E-03 CM/SEC


                  LAYER  3
                  --------

        TYPE 2 - LATERAL DRAINAGE LAYER
          MATERIAL TEXTURE NUMBER   0
THICKNESS                   =      0.20   INCHES
POROSITY                    =      0.8500 VOL/VOL
FIELD CAPACITY              =      0.0100 VOL/VOL
WILTING POINT               =      0.0050 VOL/VOL
INITIAL SOIL WATER CONTENT  =      0.0524 VOL/VOL
EFFECTIVE SAT. HYD. COND.   =  0.240000002000E-01 CM/SEC
SLOPE                       =      2.25   PERCENT
DRAINAGE LENGTH             =    325.0    FEET


                  LAYER  4
                  --------

        TYPE 4 - FLEXIBLE MEMBRANE LINER
          MATERIAL TEXTURE NUMBER  35
THICKNESS                   =      0.06   INCHES
POROSITY                    =      0.0000 VOL/VOL
FIELD CAPACITY              =      0.0000 VOL/VOL
WILTING POINT               =      0.0000 VOL/VOL
INITIAL SOIL WATER CONTENT  =      0.0000 VOL/VOL
EFFECTIVE SAT. HYD. COND.   =  0.199999996000E-12 CM/SEC
FML PINHOLE DENSITY         =      2.00   HOLES/ACRE
FML INSTALLATION DEFECTS    =      2.00   HOLES/ACRE
FML PLACEMENT QUALITY       =  3 - GOOD


                  LAYER  5
                  --------

          TYPE 3 - BARRIER SOIL LINER
          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  =      0.7500 VOL/VOL
EFFECTIVE SAT. HYD. COND.   =  0.300000003000E-08 CM/SEC


      GENERAL DESIGN AND EVAPORATIVE ZONE DATA
      ----------------------------------------

NOTE:  SCS RUNOFF CURVE NUMBER WAS COMPUTED FROM DEFAULT
         SOIL DATA BASE USING SOIL TEXTURE #30 WITH BARE
         GROUND CONDITIONS, A SURFACE SLOPE OF 33.% AND
         A SLOPE LENGTH OF  100. FEET.
```

Case C-2

SCS RUNOFF CURVE NUMBER	=	97.20	
FRACTION OF AREA ALLOWING RUNOFF	=	100.0	PERCENT
AREA PROJECTED ON HORIZONTAL PLANE	=	1.000	ACRES
EVAPORATIVE ZONE DEPTH	=	10.0	INCHES
INITIAL WATER IN EVAPORATIVE ZONE	=	3.184	INCHES
UPPER LIMIT OF EVAPORATIVE STORAGE	=	5.410	INCHES
LOWER LIMIT OF EVAPORATIVE STORAGE	=	0.470	INCHES
INITIAL SNOW WATER	=	0.000	INCHES
INITIAL WATER IN LAYER MATERIALS	=	205.766	INCHES
TOTAL INITIAL WATER	=	205.766	INCHES
TOTAL SUBSURFACE INFLOW	=	0.00	INCHES/YEAR

EVAPOTRANSPIRATION AND WEATHER DATA

NOTE: EVAPOTRANSPIRATION DATA WAS OBTAINED FROM
CHARLESTON SOUTH CAROLINA

STATION LATITUDE	=	32.90 DEGREES
MAXIMUM LEAF AREA INDEX	=	0.00
START OF GROWING SEASON (JULIAN DATE)	=	59
END OF GROWING SEASON (JULIAN DATE)	=	336
EVAPORATIVE ZONE DEPTH	=	10.0 INCHES
AVERAGE ANNUAL WIND SPEED	=	8.70 MPH
AVERAGE 1ST QUARTER RELATIVE HUMIDITY	=	70.00 %
AVERAGE 2ND QUARTER RELATIVE HUMIDITY	=	74.00 %
AVERAGE 3RD QUARTER RELATIVE HUMIDITY	=	80.00 %
AVERAGE 4TH QUARTER RELATIVE HUMIDITY	=	75.00 %

NOTE: PRECIPITATION DATA WAS SYNTHETICALLY GENERATED USING
COEFFICIENTS FOR CHARLESTON SOUTH CAROLINA

NORMAL MEAN MONTHLY PRECIPITATION (INCHES)

JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
3.33	3.37	4.38	2.58	4.41	6.54
7.33	6.50	4.94	2.92	2.18	3.11

NOTE: TEMPERATURE DATA WAS SYNTHETICALLY GENERATED USING
COEFFICIENTS FOR CHARLESTON SOUTH CAROLINA

NORMAL MEAN MONTHLY TEMPERATURE (DEGREES FAHRENHEIT)

JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
47.90	49.80	56.70	64.30	72.20	77.60
80.50	80.00	75.70	65.80	56.70	50.00

NOTE: SOLAR RADIATION DATA WAS SYNTHETICALLY GENERATED USING
COEFFICIENTS FOR CHARLESTON SOUTH CAROLINA
AND STATION LATITUDE = 32.90 DEGREES

Case C-2

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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	2.45	3.63	4.70	2.73	4.18	5.96
	7.34	7.26	4.83	2.68	2.00	2.89
STD. DEVIATIONS	1.74	1.59	2.10	1.69	2.34	3.23
	3.68	3.06	2.66	2.46	1.05	1.52
RUNOFF						
TOTALS	0.922	1.495	2.172	1.093	2.119	3.314
	3.942	4.051	2.508	1.360	0.720	1.184
STD. DEVIATIONS	1.069	0.987	1.370	0.917	1.723	2.383
	2.825	2.162	1.867	1.605	0.642	0.959
EVAPOTRANSPIRATION						
TOTALS	1.755	1.884	2.760	1.798	1.955	2.621
	3.088	3.071	2.445	1.502	1.136	1.354
STD. DEVIATIONS	0.451	0.469	0.736	0.781	0.831	0.958
	1.035	0.937	0.827	0.667	0.594	0.387
LATERAL DRAINAGE COLLECTED FROM LAYER 3						
TOTALS	0.0262	0.0259	0.0321	0.0332	0.0344	0.0313
	0.0286	0.0254	0.0232	0.0235	0.0234	0.0253
STD. DEVIATIONS	0.0145	0.0120	0.0130	0.0138	0.0157	0.0152
	0.0144	0.0135	0.0132	0.0139	0.0135	0.0143
PERCOLATION/LEAKAGE THROUGH LAYER 5						
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	0.0898	0.0975	0.1101	0.1177	0.1178	0.1107
	0.0981	0.0872	0.0821	0.0806	0.0828	0.0868
STD. DEVIATIONS	0.0497	0.0448	0.0446	0.0488	0.0537	0.0537
	0.0493	0.0464	0.0466	0.0476	0.0476	0.0492

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Case C-2

AVERAGE ANNUAL TOTALS & (STD. DEVIATIONS) FOR YEARS 1 THROUGH 30

	INCHES		CU. FEET	PERCENT
PRECIPITATION	50.67	(8.914)	183938.1	100.00
RUNOFF	24.882	(6.3226)	90320.00	49.103
EVAPOTRANSPIRATION	25.369	(2.8056)	92090.56	50.066
LATERAL DRAINAGE COLLECTED FROM LAYER 3	0.33267	(0.12894)	1207.595	0.65652
PERCOLATION/LEAKAGE THROUGH LAYER 5	0.00001	(0.00000)	0.039	0.00002
AVERAGE HEAD ON TOP OF LAYER 4	0.097	(0.037)		
CHANGE IN WATER STORAGE	0.088	(0.6687)	319.97	0.174

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PEAK DAILY VALUES FOR YEARS 1 THROUGH 30

	(INCHES)	(CU. FT.)
PRECIPITATION	8.28	30056.398
RUNOFF	7.649	27765.3066
DRAINAGE COLLECTED FROM LAYER 3	0.00183	6.64375
PERCOLATION/LEAKAGE THROUGH LAYER 5	0.000000	0.00020
AVERAGE HEAD ON TOP OF LAYER 4	0.194	
MAXIMUM HEAD ON TOP OF LAYER 4	0.381	
LOCATION OF MAXIMUM HEAD IN LAYER 3 (DISTANCE FROM DRAIN)	6.2 FEET	
SNOW WATER	1.03	3735.4602
MAXIMUM VEG. SOIL WATER (VOL/VOL)	0.3684	
MINIMUM VEG. SOIL WATER (VOL/VOL)	0.0520	

*** 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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Case C-2

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FINAL WATER STORAGE AT END OF YEAR 30

LAYER	(INCHES)	(VOL/VOL)
1	202.2842	0.1916
2	5.8560	0.2440
3	0.0826	0.4128
4	0.0000	0.0000
5	0.1875	0.7500
SNOW WATER	0.000	

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Case D-1

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**                                                                          **
**                                                                          **
**              HYDROLOGIC EVALUATION OF LANDFILL PERFORMANCE               **
**                HELP MODEL VERSION 3.07  (1 NOVEMBER 1997)                **
**                 DEVELOPED BY ENVIRONMENTAL LABORATORY                    **
**                   USAE WATERWAYS EXPERIMENT STATION                      **
**             FOR USEPA RISK REDUCTION ENGINEERING LABORATORY              **
**                                                                          **
**                                                                          **
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PRECIPITATION DATA FILE:     C:\W55\DATA4.D4
TEMPERATURE DATA FILE:       C:\W55\DATA7.D7
SOLAR RADIATION DATA FILE:   C:\W55\DATA13.D13
EVAPOTRANSPIRATION DATA:     C:\W55\DATA11.D11
SOIL AND DESIGN DATA FILE:   C:\W55\DATA10.D10
OUTPUT DATA FILE:            C:\W55\OUT.OUT


TIME:  17:20     DATE:   6/ 7/2016


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     TITLE:  Case D-1 (Final Grades with Cover in Landfill Area 1)

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     NOTE:  INITIAL MOISTURE CONTENT OF THE LAYERS AND SNOW WATER WERE
              COMPUTED AS NEARLY STEADY-STATE VALUES BY THE PROGRAM.


                                   LAYER  1
                                   --------

                     TYPE 1 - VERTICAL PERCOLATION LAYER
                         MATERIAL TEXTURE NUMBER  10
          THICKNESS                   =    24.00   INCHES
          POROSITY                    =     0.3980 VOL/VOL
          FIELD CAPACITY              =     0.2440 VOL/VOL
          WILTING POINT               =     0.1360 VOL/VOL
          INITIAL SOIL WATER CONTENT  =     0.3210 VOL/VOL
          EFFECTIVE SAT. HYD. COND.   =  0.119999997000E-03 CM/SEC
        NOTE:  SATURATED HYDRAULIC CONDUCTIVITY IS MULTIPLIED BY  3.00
                 FOR ROOT CHANNELS IN TOP HALF OF EVAPORATIVE ZONE.


                                   LAYER  2
                                   --------

                       TYPE 2 - LATERAL DRAINAGE LAYER
                         MATERIAL TEXTURE NUMBER   0
```

Case D-1

```
THICKNESS                    =      0.20   INCHES
POROSITY                     =      0.8500 VOL/VOL
FIELD CAPACITY               =      0.0100 VOL/VOL
WILTING POINT                =      0.0050 VOL/VOL
INITIAL SOIL WATER CONTENT   =      0.0206 VOL/VOL
EFFECTIVE SAT. HYD. COND.    =  0.810000002000     CM/SEC
SLOPE                        =     33.00   PERCENT
DRAINAGE LENGTH              =    100.0    FEET


                    LAYER  3
                    --------

         TYPE 4 - FLEXIBLE MEMBRANE LINER
           MATERIAL TEXTURE NUMBER  36
THICKNESS                    =      0.02   INCHES
POROSITY                     =      0.0000 VOL/VOL
FIELD CAPACITY               =      0.0000 VOL/VOL
WILTING POINT                =      0.0000 VOL/VOL
INITIAL SOIL WATER CONTENT   =      0.0000 VOL/VOL
EFFECTIVE SAT. HYD. COND.    =  0.399999993000E-12 CM/SEC
FML PINHOLE DENSITY          =      2.00   HOLES/ACRE
FML INSTALLATION DEFECTS     =      2.00   HOLES/ACRE
FML PLACEMENT QUALITY        =  3 - GOOD


                    LAYER  4
                    --------

           TYPE 3 - BARRIER SOIL LINER
           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   =      0.7500 VOL/VOL
EFFECTIVE SAT. HYD. COND.    =  0.300000003000E-08 CM/SEC


                    LAYER  5
                    --------

        TYPE 1 - VERTICAL PERCOLATION LAYER
           MATERIAL TEXTURE NUMBER  30
THICKNESS                    =    624.00   INCHES
POROSITY                     =      0.5410 VOL/VOL
FIELD CAPACITY               =      0.1870 VOL/VOL
WILTING POINT                =      0.0470 VOL/VOL
INITIAL SOIL WATER CONTENT   =      0.1870 VOL/VOL
EFFECTIVE SAT. HYD. COND.    =  0.499999987000E-04 CM/SEC


                    LAYER  6
                    --------

        TYPE 1 - VERTICAL PERCOLATION LAYER
           MATERIAL TEXTURE NUMBER  10
THICKNESS                    =     24.00   INCHES
POROSITY                     =      0.3980 VOL/VOL
```

Case D-1

```
FIELD CAPACITY             =       0.2440 VOL/VOL
WILTING POINT              =       0.1360 VOL/VOL
INITIAL SOIL WATER CONTENT =       0.2440 VOL/VOL
EFFECTIVE SAT. HYD. COND.  =  0.119999997000E-03 CM/SEC


                      LAYER  7
                      --------

             TYPE 2 - LATERAL DRAINAGE LAYER
               MATERIAL TEXTURE NUMBER   0
THICKNESS                  =       0.20   INCHES
POROSITY                   =       0.8500 VOL/VOL
FIELD CAPACITY             =       0.0100 VOL/VOL
WILTING POINT              =       0.0050 VOL/VOL
INITIAL SOIL WATER CONTENT =       0.0101 VOL/VOL
EFFECTIVE SAT. HYD. COND.  =  0.279999990000E-02 CM/SEC
SLOPE                      =       2.25   PERCENT
DRAINAGE LENGTH            =     325.0    FEET


                      LAYER  8
                      --------

             TYPE 4 - FLEXIBLE MEMBRANE LINER
               MATERIAL TEXTURE NUMBER  35
THICKNESS                  =       0.06   INCHES
POROSITY                   =       0.0000 VOL/VOL
FIELD CAPACITY             =       0.0000 VOL/VOL
WILTING POINT              =       0.0000 VOL/VOL
INITIAL SOIL WATER CONTENT =       0.0000 VOL/VOL
EFFECTIVE SAT. HYD. COND.  =  0.199999996000E-12 CM/SEC
FML PINHOLE DENSITY        =       2.00   HOLES/ACRE
FML INSTALLATION DEFECTS   =       2.00   HOLES/ACRE
FML PLACEMENT QUALITY      =  3 - GOOD


                      LAYER  9
                      --------

               TYPE 3 - BARRIER SOIL LINER
               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 =       0.7500 VOL/VOL
EFFECTIVE SAT. HYD. COND.  =  0.300000003000E-08 CM/SEC


        GENERAL DESIGN AND EVAPORATIVE ZONE DATA
        ----------------------------------------

NOTE:  SCS RUNOFF CURVE NUMBER WAS COMPUTED FROM DEFAULT
         SOIL DATA BASE USING SOIL TEXTURE #10 WITH A
         FAIR STAND OF GRASS, A SURFACE SLOPE OF 33.%
         AND A SLOPE LENGTH OF  100. FEET.
```

Case D-1

SCS RUNOFF CURVE NUMBER	=	87.40	
FRACTION OF AREA ALLOWING RUNOFF	=	100.0	PERCENT
AREA PROJECTED ON HORIZONTAL PLANE	=	1.000	ACRES
EVAPORATIVE ZONE DEPTH	=	22.0	INCHES
INITIAL WATER IN EVAPORATIVE ZONE	=	7.119	INCHES
UPPER LIMIT OF EVAPORATIVE STORAGE	=	8.756	INCHES
LOWER LIMIT OF EVAPORATIVE STORAGE	=	2.992	INCHES
INITIAL SNOW WATER	=	0.000	INCHES
INITIAL WATER IN LAYER MATERIALS	=	130.629	INCHES
TOTAL INITIAL WATER	=	130.629	INCHES
TOTAL SUBSURFACE INFLOW	=	0.00	INCHES/YEAR

EVAPOTRANSPIRATION AND WEATHER DATA

NOTE: EVAPOTRANSPIRATION DATA WAS OBTAINED FROM
CHARLESTON SOUTH CAROLINA

STATION LATITUDE	=	32.90 DEGREES
MAXIMUM LEAF AREA INDEX	=	2.00
START OF GROWING SEASON (JULIAN DATE)	=	59
END OF GROWING SEASON (JULIAN DATE)	=	336
EVAPORATIVE ZONE DEPTH	=	22.0 INCHES
AVERAGE ANNUAL WIND SPEED	=	8.70 MPH
AVERAGE 1ST QUARTER RELATIVE HUMIDITY	=	70.00 %
AVERAGE 2ND QUARTER RELATIVE HUMIDITY	=	74.00 %
AVERAGE 3RD QUARTER RELATIVE HUMIDITY	=	80.00 %
AVERAGE 4TH QUARTER RELATIVE HUMIDITY	=	75.00 %

NOTE: PRECIPITATION DATA WAS SYNTHETICALLY GENERATED USING
COEFFICIENTS FOR CHARLESTON SOUTH CAROLINA

NORMAL MEAN MONTHLY PRECIPITATION (INCHES)

JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
3.33	3.37	4.38	2.58	4.41	6.54
7.33	6.50	4.94	2.92	2.18	3.11

NOTE: TEMPERATURE DATA WAS SYNTHETICALLY GENERATED USING
COEFFICIENTS FOR CHARLESTON SOUTH CAROLINA

NORMAL MEAN MONTHLY TEMPERATURE (DEGREES FAHRENHEIT)

JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
47.90	49.80	56.70	64.30	72.20	77.60
80.50	80.00	75.70	65.80	56.70	50.00

NOTE: SOLAR RADIATION DATA WAS SYNTHETICALLY GENERATED USING
COEFFICIENTS FOR CHARLESTON SOUTH CAROLINA
AND STATION LATITUDE = 32.90 DEGREES

Case D-1

****

AVERAGE MONTHLY VALUES IN INCHES FOR YEARS 1 THROUGH 30

	JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
PRECIPITATION						
TOTALS	2.45	3.63	4.70	2.73	4.18	5.96
	7.34	7.26	4.83	2.68	2.00	2.89
STD. DEVIATIONS	1.74	1.59	2.10	1.69	2.34	3.23
	3.68	3.06	2.66	2.46	1.05	1.52
RUNOFF						
TOTALS	0.123	0.218	0.409	0.119	0.440	0.771
	0.951	0.924	0.519	0.353	0.085	0.195
STD. DEVIATIONS	0.284	0.295	0.482	0.206	0.707	0.957
	1.415	0.745	0.696	0.653	0.160	0.274
EVAPOTRANSPIRATION						
TOTALS	1.757	2.155	3.438	3.334	3.791	4.648
	5.202	5.160	3.675	1.756	0.924	1.106
STD. DEVIATIONS	0.203	0.213	0.398	1.040	1.393	1.476
	1.454	1.284	0.982	0.468	0.205	0.146
LATERAL DRAINAGE COLLECTED FROM LAYER 2						
TOTALS	1.1543	1.0329	1.3108	0.3456	0.1904	0.3401
	0.5421	0.8758	0.7025	0.6956	0.4772	0.9620
STD. DEVIATIONS	1.1227	1.0476	1.1269	0.4264	0.5528	0.7117
	0.9716	1.3356	0.9540	1.2217	0.8212	0.9679
PERCOLATION/LEAKAGE THROUGH LAYER 4						
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
LATERAL DRAINAGE COLLECTED FROM 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 9						
TOTALS	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
STD. DEVIATIONS	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000

AVERAGES OF MONTHLY AVERAGED DAILY HEADS (INCHES)

Case D-1

DAILY AVERAGE HEAD ON TOP OF LAYER 3

AVERAGES	0.0027	0.0027	0.0031	0.0008	0.0004	0.0008
	0.0014	0.0021	0.0017	0.0017	0.0012	0.0023
STD. DEVIATIONS	0.0027	0.0027	0.0027	0.0010	0.0013	0.0017
	0.0028	0.0032	0.0023	0.0031	0.0020	0.0023

DAILY AVERAGE HEAD ON TOP OF LAYER 8

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

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AVERAGE ANNUAL TOTALS & (STD. DEVIATIONS) FOR YEARS 1 THROUGH 30

	INCHES	CU. FEET	PERCENT
PRECIPITATION	50.67 (8.914)	183938.1	100.00
RUNOFF	5.107 (2.7796)	18537.97	10.078
EVAPOTRANSPIRATION	36.946 (3.0505)	134114.22	72.913
LATERAL DRAINAGE COLLECTED FROM LAYER 2	8.62921 (4.42874)	31324.049	17.02966
PERCOLATION/LEAKAGE THROUGH LAYER 4	0.00000 (0.00000)	0.009	0.00000
AVERAGE HEAD ON TOP OF LAYER 3	0.002 (0.001)		
LATERAL DRAINAGE COLLECTED FROM LAYER 7	0.00000 (0.00000)	0.004	0.00000
PERCOLATION/LEAKAGE THROUGH LAYER 9	0.00000 (0.00000)	0.008	0.00000
AVERAGE HEAD ON TOP OF LAYER 8	0.000 (0.000)		
CHANGE IN WATER STORAGE	-0.011 (0.9322)	-38.13	-0.021

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PEAK DAILY VALUES FOR YEARS 1 THROUGH 30

	(INCHES)	(CU. FT.)
PRECIPITATION	8.28	30056.398

Case D-1

RUNOFF	5.356	19443.6875
DRAINAGE COLLECTED FROM LAYER 2	0.90545	3286.78467
PERCOLATION/LEAKAGE THROUGH LAYER 4	0.000000	0.00057
AVERAGE HEAD ON TOP OF LAYER 3	0.180	
MAXIMUM HEAD ON TOP OF LAYER 3	0.128	
LOCATION OF MAXIMUM HEAD IN LAYER 2 (DISTANCE FROM DRAIN)	0.0 FEET	
DRAINAGE COLLECTED FROM LAYER 7	0.00000	0.00014
PERCOLATION/LEAKAGE THROUGH LAYER 9	0.000000	0.00002
AVERAGE HEAD ON TOP OF LAYER 8	0.000	
MAXIMUM HEAD ON TOP OF LAYER 8	0.006	
LOCATION OF MAXIMUM HEAD IN LAYER 7 (DISTANCE FROM DRAIN)	0.0 FEET	
SNOW WATER	1.03	3735.4602

MAXIMUM VEG. SOIL WATER (VOL/VOL) 0.3463

MINIMUM VEG. SOIL WATER (VOL/VOL) 0.1360

*** Maximum heads are computed using McEnroe's equations. ***

Reference: Maximum Saturated Depth over Landfill Liner
by Bruce M. McEnroe, University of Kansas
ASCE Journal of Environmental Engineering
Vol. 119, No. 2, March 1993, pp. 262-270.

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FINAL WATER STORAGE AT END OF YEAR 30

LAYER	(INCHES)	(VOL/VOL)
1	7.3740	0.3073
2	0.0189	0.0943
3	0.0000	0.0000
4	0.1875	0.7500
5	116.6880	0.1870
6	5.8560	0.2440
7	0.0020	0.0100
8	0.0000	0.0000

Case D-1

9	0.1875	0.7500
SNOW WATER	0.000	

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Case D-2

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**              HYDROLOGIC EVALUATION OF LANDFILL PERFORMANCE               **
**                HELP MODEL VERSION 3.07  (1 NOVEMBER 1997)                **
**                  DEVELOPED BY ENVIRONMENTAL LABORATORY                   **
**                    USAE WATERWAYS EXPERIMENT STATION                     **
**              FOR USEPA RISK REDUCTION ENGINEERING LABORATORY             **
**                                                                          **
**                                                                          **
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PRECIPITATION DATA FILE:      C:\W64\DATA4.D4
TEMPERATURE DATA FILE:        C:\W64\DATA7.D7
SOLAR RADIATION DATA FILE:    C:\W64\DATA13.D13
EVAPOTRANSPIRATION DATA:      C:\W64\DATA11.D11
SOIL AND DESIGN DATA FILE:    C:\W64\DATA10.D10
OUTPUT DATA FILE:             C:\W64\OUT.OUT


TIME:  17:30     DATE:   6/ 7/2016


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     TITLE:  Case D-2 (Final Grades with Cover in Landfill Area 2)

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     NOTE:  INITIAL MOISTURE CONTENT OF THE LAYERS AND SNOW WATER WERE
              COMPUTED AS NEARLY STEADY-STATE VALUES BY THE PROGRAM.


                                  LAYER  1
                                  --------

                     TYPE 1 - VERTICAL PERCOLATION LAYER
                         MATERIAL TEXTURE NUMBER  10
          THICKNESS                   =     24.00   INCHES
          POROSITY                    =      0.3980 VOL/VOL
          FIELD CAPACITY              =      0.2440 VOL/VOL
          WILTING POINT               =      0.1360 VOL/VOL
          INITIAL SOIL WATER CONTENT  =      0.3210 VOL/VOL
          EFFECTIVE SAT. HYD. COND.   =  0.119999997000E-03 CM/SEC
        NOTE:  SATURATED HYDRAULIC CONDUCTIVITY IS MULTIPLIED BY  3.00
                FOR ROOT CHANNELS IN TOP HALF OF EVAPORATIVE ZONE.


                                  LAYER  2
                                  --------

                       TYPE 2 - LATERAL DRAINAGE LAYER
                         MATERIAL TEXTURE NUMBER   0
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Case D-2

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THICKNESS                  =      0.20   INCHES
POROSITY                   =      0.8500 VOL/VOL
FIELD CAPACITY             =      0.0100 VOL/VOL
WILTING POINT              =      0.0050 VOL/VOL
INITIAL SOIL WATER CONTENT =      0.0206 VOL/VOL
EFFECTIVE SAT. HYD. COND.  =  0.810000002000     CM/SEC
SLOPE                      =     33.00   PERCENT
DRAINAGE LENGTH            =    100.0    FEET


                    LAYER  3
                    --------

          TYPE 4 - FLEXIBLE MEMBRANE LINER
            MATERIAL TEXTURE NUMBER  36
THICKNESS                  =      0.02   INCHES
POROSITY                   =      0.0000 VOL/VOL
FIELD CAPACITY             =      0.0000 VOL/VOL
WILTING POINT              =      0.0000 VOL/VOL
INITIAL SOIL WATER CONTENT =      0.0000 VOL/VOL
EFFECTIVE SAT. HYD. COND.  =  0.399999993000E-12 CM/SEC
FML PINHOLE DENSITY        =      2.00   HOLES/ACRE
FML INSTALLATION DEFECTS   =      2.00   HOLES/ACRE
FML PLACEMENT QUALITY      =  3 - GOOD


                    LAYER  4
                    --------

            TYPE 3 - BARRIER SOIL LINER
            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 =      0.7500 VOL/VOL
EFFECTIVE SAT. HYD. COND.  =  0.300000003000E-08 CM/SEC


                    LAYER  5
                    --------

        TYPE 1 - VERTICAL PERCOLATION LAYER
            MATERIAL TEXTURE NUMBER  30
THICKNESS                  =   1056.00   INCHES
POROSITY                   =      0.5410 VOL/VOL
FIELD CAPACITY             =      0.1870 VOL/VOL
WILTING POINT              =      0.0470 VOL/VOL
INITIAL SOIL WATER CONTENT =      0.1870 VOL/VOL
EFFECTIVE SAT. HYD. COND.  =  0.499999987000E-04 CM/SEC


                    LAYER  6
                    --------

        TYPE 1 - VERTICAL PERCOLATION LAYER
            MATERIAL TEXTURE NUMBER  10
THICKNESS                  =     24.00   INCHES
POROSITY                   =      0.3980 VOL/VOL
```

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```
FIELD CAPACITY              =       0.2440 VOL/VOL
WILTING POINT               =       0.1360 VOL/VOL
INITIAL SOIL WATER CONTENT  =       0.2440 VOL/VOL
EFFECTIVE SAT. HYD. COND.   =  0.119999997000E-03 CM/SEC


                        LAYER  7
                        --------

              TYPE 2 - LATERAL DRAINAGE LAYER
                MATERIAL TEXTURE NUMBER   0
THICKNESS                   =       0.20    INCHES
POROSITY                    =       0.8500 VOL/VOL
FIELD CAPACITY              =       0.0100 VOL/VOL
WILTING POINT               =       0.0050 VOL/VOL
INITIAL SOIL WATER CONTENT  =       0.0103 VOL/VOL
EFFECTIVE SAT. HYD. COND.   =  0.240000011000E-02 CM/SEC
SLOPE                       =       2.25    PERCENT
DRAINAGE LENGTH             =     325.0     FEET


                        LAYER  8
                        --------

              TYPE 4 - FLEXIBLE MEMBRANE LINER
                MATERIAL TEXTURE NUMBER  35
THICKNESS                   =       0.06    INCHES
POROSITY                    =       0.0000 VOL/VOL
FIELD CAPACITY              =       0.0000 VOL/VOL
WILTING POINT               =       0.0000 VOL/VOL
INITIAL SOIL WATER CONTENT  =       0.0000 VOL/VOL
EFFECTIVE SAT. HYD. COND.   =  0.199999996000E-12 CM/SEC
FML PINHOLE DENSITY         =       2.00    HOLES/ACRE
FML INSTALLATION DEFECTS    =       2.00    HOLES/ACRE
FML PLACEMENT QUALITY       =  3 - GOOD


                        LAYER  9
                        --------

                TYPE 3 - BARRIER SOIL LINER
                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  =       0.7500 VOL/VOL
EFFECTIVE SAT. HYD. COND.   =  0.300000003000E-08 CM/SEC


            GENERAL DESIGN AND EVAPORATIVE ZONE DATA
            ----------------------------------------

NOTE:  SCS RUNOFF CURVE NUMBER WAS COMPUTED FROM DEFAULT
         SOIL DATA BASE USING SOIL TEXTURE #10 WITH A
         FAIR STAND OF GRASS, A SURFACE SLOPE OF 33.%
         AND A SLOPE LENGTH OF  100. FEET.
```

Case D-2

SCS RUNOFF CURVE NUMBER	=	87.40	
FRACTION OF AREA ALLOWING RUNOFF	=	100.0	PERCENT
AREA PROJECTED ON HORIZONTAL PLANE	=	1.000	ACRES
EVAPORATIVE ZONE DEPTH	=	22.0	INCHES
INITIAL WATER IN EVAPORATIVE ZONE	=	7.119	INCHES
UPPER LIMIT OF EVAPORATIVE STORAGE	=	8.756	INCHES
LOWER LIMIT OF EVAPORATIVE STORAGE	=	2.992	INCHES
INITIAL SNOW WATER	=	0.000	INCHES
INITIAL WATER IN LAYER MATERIALS	=	211.413	INCHES
TOTAL INITIAL WATER	=	211.413	INCHES
TOTAL SUBSURFACE INFLOW	=	0.00	INCHES/YEAR

EVAPOTRANSPIRATION AND WEATHER DATA

NOTE: EVAPOTRANSPIRATION DATA WAS OBTAINED FROM
CHARLESTON SOUTH CAROLINA

STATION LATITUDE	=	32.90 DEGREES
MAXIMUM LEAF AREA INDEX	=	2.00
START OF GROWING SEASON (JULIAN DATE)	=	59
END OF GROWING SEASON (JULIAN DATE)	=	336
EVAPORATIVE ZONE DEPTH	=	22.0 INCHES
AVERAGE ANNUAL WIND SPEED	=	8.70 MPH
AVERAGE 1ST QUARTER RELATIVE HUMIDITY	=	70.00 %
AVERAGE 2ND QUARTER RELATIVE HUMIDITY	=	74.00 %
AVERAGE 3RD QUARTER RELATIVE HUMIDITY	=	80.00 %
AVERAGE 4TH QUARTER RELATIVE HUMIDITY	=	75.00 %

NOTE: PRECIPITATION DATA WAS SYNTHETICALLY GENERATED USING
COEFFICIENTS FOR CHARLESTON SOUTH CAROLINA

NORMAL MEAN MONTHLY PRECIPITATION (INCHES)

JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
3.33	3.37	4.38	2.58	4.41	6.54
7.33	6.50	4.94	2.92	2.18	3.11

NOTE: TEMPERATURE DATA WAS SYNTHETICALLY GENERATED USING
COEFFICIENTS FOR CHARLESTON SOUTH CAROLINA

NORMAL MEAN MONTHLY TEMPERATURE (DEGREES FAHRENHEIT)

JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
47.90	49.80	56.70	64.30	72.20	77.60
80.50	80.00	75.70	65.80	56.70	50.00

NOTE: SOLAR RADIATION DATA WAS SYNTHETICALLY GENERATED USING
COEFFICIENTS FOR CHARLESTON SOUTH CAROLINA
AND STATION LATITUDE = 32.90 DEGREES

Case D-2

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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	2.45	3.63	4.70	2.73	4.18	5.96
	7.34	7.26	4.83	2.68	2.00	2.89
STD. DEVIATIONS	1.74	1.59	2.10	1.69	2.34	3.23
	3.68	3.06	2.66	2.46	1.05	1.52
RUNOFF						
TOTALS	0.123	0.218	0.409	0.119	0.440	0.771
	0.951	0.924	0.519	0.353	0.085	0.195
STD. DEVIATIONS	0.284	0.295	0.482	0.206	0.707	0.957
	1.415	0.745	0.696	0.653	0.160	0.274
EVAPOTRANSPIRATION						
TOTALS	1.757	2.155	3.438	3.334	3.791	4.648
	5.202	5.160	3.675	1.756	0.924	1.106
STD. DEVIATIONS	0.203	0.213	0.398	1.040	1.393	1.476
	1.454	1.284	0.982	0.468	0.205	0.146
LATERAL DRAINAGE COLLECTED FROM LAYER 2						
TOTALS	1.1543	1.0329	1.3108	0.3456	0.1904	0.3401
	0.5421	0.8758	0.7025	0.6956	0.4772	0.9620
STD. DEVIATIONS	1.1227	1.0476	1.1269	0.4264	0.5528	0.7117
	0.9716	1.3356	0.9540	1.2217	0.8212	0.9679
PERCOLATION/LEAKAGE THROUGH LAYER 4						
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
LATERAL DRAINAGE COLLECTED FROM 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 9						
TOTALS	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
STD. DEVIATIONS	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000

AVERAGES OF MONTHLY AVERAGED DAILY HEADS (INCHES)

Case D-2

DAILY AVERAGE HEAD ON TOP OF LAYER 3

AVERAGES	0.0027	0.0027	0.0031	0.0008	0.0004	0.0008
	0.0014	0.0021	0.0017	0.0017	0.0012	0.0023
STD. DEVIATIONS	0.0027	0.0027	0.0027	0.0010	0.0013	0.0017
	0.0028	0.0032	0.0023	0.0031	0.0020	0.0023

DAILY AVERAGE HEAD ON TOP OF LAYER 8

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

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AVERAGE ANNUAL TOTALS & (STD. DEVIATIONS) FOR YEARS 1 THROUGH 30

	INCHES	CU. FEET	PERCENT
PRECIPITATION	50.67 (8.914)	183938.1	100.00
RUNOFF	5.107 (2.7796)	18537.97	10.078
EVAPOTRANSPIRATION	36.946 (3.0505)	134114.22	72.913
LATERAL DRAINAGE COLLECTED FROM LAYER 2	8.62921 (4.42874)	31324.049	17.02966
PERCOLATION/LEAKAGE THROUGH LAYER 4	0.00000 (0.00000)	0.009	0.00000
AVERAGE HEAD ON TOP OF LAYER 3	0.002 (0.001)		
LATERAL DRAINAGE COLLECTED FROM LAYER 7	0.00000 (0.00000)	0.008	0.00000
PERCOLATION/LEAKAGE THROUGH LAYER 9	0.00000 (0.00000)	0.008	0.00000
AVERAGE HEAD ON TOP OF LAYER 8	0.000 (0.000)		
CHANGE IN WATER STORAGE	-0.011 (0.9322)	-38.14	-0.021

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PEAK DAILY VALUES FOR YEARS 1 THROUGH 30

	(INCHES)	(CU. FT.)
PRECIPITATION	8.28	30056.398

Case D-2

RUNOFF	5.356	19443.6875
DRAINAGE COLLECTED FROM LAYER 2	0.90545	3286.78467
PERCOLATION/LEAKAGE THROUGH LAYER 4	0.000000	0.00057
AVERAGE HEAD ON TOP OF LAYER 3	0.180	
MAXIMUM HEAD ON TOP OF LAYER 3	0.128	
LOCATION OF MAXIMUM HEAD IN LAYER 2 (DISTANCE FROM DRAIN)	0.0 FEET	
DRAINAGE COLLECTED FROM LAYER 7	0.00000	0.00025
PERCOLATION/LEAKAGE THROUGH LAYER 9	0.000000	0.00002
AVERAGE HEAD ON TOP OF LAYER 8	0.000	
MAXIMUM HEAD ON TOP OF LAYER 8	0.011	
LOCATION OF MAXIMUM HEAD IN LAYER 7 (DISTANCE FROM DRAIN)	0.0 FEET	
SNOW WATER	1.03	3735.4602
MAXIMUM VEG. SOIL WATER (VOL/VOL)	0.3463	
MINIMUM VEG. SOIL WATER (VOL/VOL)	0.1360	

*** Maximum heads are computed using McEnroe's equations. ***

Reference: Maximum Saturated Depth over Landfill Liner
by Bruce M. McEnroe, University of Kansas
ASCE Journal of Environmental Engineering
Vol. 119, No. 2, March 1993, pp. 262-270.

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FINAL WATER STORAGE AT END OF YEAR 30

LAYER	(INCHES)	(VOL/VOL)
1	7.3740	0.3073
2	0.0189	0.0943
3	0.0000	0.0000
4	0.1875	0.7500
5	197.4719	0.1870
6	5.8560	0.2440
7	0.0020	0.0100
8	0.0000	0.0000

Case D-2

9	0.1875	0.7500
SNOW WATER	0.000	

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APPENDIX E-2

LEACHATE COLLECTION GEOSYNTHETIC DRAINAGE LAYER DESIGN

APPENDIX E-2

LEACHATE COLLECTION GEOSYNTHETIC DRAINAGE LAYER DESIGN

SOUTH CAROLINA LICENSED PROFESSIONAL ENGINEER No. 33535 SCOTT M. GRAVES 8/9/2016

SOUTH CAROLINA GEOSYNTEC CONSULTANTS, INC. No. 773 CERTIFICATE OF AUTHORIZATION

SEALED FOR
CALCULATION PAGES E-2-1
THROUGH E-2-16

PURPOSE

The purpose of this calculation package is to present the design of the geosynthetic drainage layer of the leachate collection system (LCS) for the proposed Class Three Landfill at the Winyah Generating Station (WGS). The geocomposite drainage layer is composed of a high density polyethylene (HDPE) geonet core with a needlepunched non-woven geotextile bonded to its top and bottom surfaces (i.e., a double-sided geocomposite). The geocomposite drainage layer will be located between a 2-ft thick overlying protective cover/drainage layer (sand or other permeable material, such as bottom ash) and a 60-mil thick HDPE geomembrane.

The design criteria evaluated include: (i) filtration capability and specifications for the geotextile component of the geocomposite drainage layer; (ii) survivability specifications for the geotextile component; and (iii) hydraulic capacities of the geosynthetic drainage layer and testing conditions for verifying that the required capacities are achieved.

METHOD OF ANALYSIS

Geotextile Filtration

The filtration characteristics of the geotextile component of the geocomposite layer are evaluated using a retention criterion, a permeability criterion, a porosity criterion, and a thickness criterion based on methods proposed by Holtz et al. (1998) and Giroud (2010). These criteria are summarized in Table E-2.1.

Table E-2.1. Filtration Criteria for Geotextile Components (adapted from Holtz et al., 1998; and Giroud, 2010)

1. Retention Criterion

1.1 Soils with less than 50% particles < 0.075 mm (US Sieve No. 200)

Density index of the soil (Relative density)		Linear coefficient of uniformity of the soil	
		$1 < C'_u < 3$	$C'_u > 3$
loose soil	$I_D \leq 35\%$	$O_{95} \leq (C'_u)^{0.3}\ d'_{85}$	$O_{95} < \frac{9}{(C'_u)^{1.7}}\ d'_{85}$
medium dense soil	$35\% < I_D < 65\%$	$O_{95} \leq 1.5\ (C'_u)^{0.3}\ d'_{85}$	$O_{95} < \frac{13.5}{(C'_u)^{1.7}}\ d'_{85}$
dense soil	$I_D \geq 65\%$	$O_{95} \leq 2\ (C'_u)^{0.3}\ d'_{85}$	$O_{95} < \frac{18}{(C'_u)^{1.7}}\ d'_{85}$

1.2 Soils with more than 50% particles < 0.075 mm (US Sieve No. 200)

$O_{95} \leq 210\ \mu m$ (US Sieve No. 70)

2 Permeability Criterion

$k_{geotextile} \geq \max\ (i_{soil}\ k_{soil}\ ,\ k_{soil})$

3. Porosity Criterion

Nonwoven geotextiles: $n_g \geq 55\%$

4. Thickness Criterion

Nonwoven geotextiles: $N_{constrictions} \geq 25$

Notes:
- O_{95} is the apparent opening size (AOS) of the geotextile
- C'_u = linear coefficient of uniformity = $\sqrt{d'_{100}/d'_0}$

  where d'_{100} and d'_0 is the top and bottom extremities, respectively, of a line drawn through the soil particle-size distribution curve and tangent at d_{50}.
- d'_{85} is the "linear particle size" for which 85% of particles are finer by weight, derived from the straight line drawn through the soil particle-size distribution curve.
- I_D = relative density or density index = $(e_{max} - e)/(e_{max} - e_{min})$, where e = soil void ratio; e_{min} = soil minimum void ratio, and e_{max} = soil maximum void ratio.
- $k_{geotextile}$ = geotextile hydraulic conductivity; k_{soil} = soil hydraulic conductivity; i_{soil} = hydraulic gradient in the soil next to the geotextile,

- porosity, n_g (dimensionless) is calculated as follows: $n_g = 1 - \mu_g/(\rho_g\, t_g)$, where: μ_g = geotextile mass per unit area, ρ_g = polymer density, and t_g = geotextile thickness
- Number of constrictions ($N_{constrictions}$) is calculated as follows: ($N_{constrictions}$) $= \mu_g / [\rho_g d_f \sqrt{1 - n_g}]$, where: μ_g = geotextile mass per unit area; ρ_g = polymer density; d_f = geotextile fiber diameter; and n_g = geotextile porosity.

Geotextile Survivability

Survivability requirements (grab, tear, and puncture strengths) are considered so that the geotextile component of the geocomposite will have adequate resistance to stresses applied to the geotextile during construction (i.e., when concentrated stresses should be the highest), using the method presented in GRI-GT13 (2012). The procedure involves two steps: (i) establish the required degree of survivability as a function of subgrade conditions, type of construction equipment operation above the geotextile, and lift thickness using Table E-2.2; and (ii) establish the recommended minimum values of certain mechanical strength properties (i.e., grab strength, puncture resistance, and trapezoidal tear strength) using Table E-2.3. The survivability requirements are then compared to characteristics of geotextile products on the current market to check that products are available to meet the calculated minimum strengths.

Drainage Layer Hydraulic Capacity

The drainage layer hydraulic capacity design evaluation is performed using the design-by-function concept presented by Koerner (2005) and based on Darcy's equation (flow rate = hydraulic conductivity × hydraulic gradient × cross-sectional area of flow) for hydraulic flow in porous, saturated media. The approach herein then follows the design methodologies presented in Giroud et al. (2000) and GRI-GC8 (2013).

The design method involves the following steps:

Step 1) Calculate the required (design) transmissivity (θ_{req}) based on results of leachate generation calculations using the USEPA Hydrologic Evaluation of Landfill Performance (*HELP*) model.

Step 2) Apply a global factor of safety (FS) to find the allowable flow rate and corresponding "Long-Term In-Soil" (LTIS) transmissivity (θ_{LTIS}).

Step 3) Apply partial reduction factors (RFs) for creep, chemical clogging, and biological clogging to account for the long-term decrease in flow capacity behavior, and calculate the baseline flow rate and corresponding baseline transmissivity (θ_{100}).

Step 4) Determine the critical operational case for θ_{100} by comparing required θ_{100} to typical θ_{100} for biplanar geocomposites at various loading conditions.

Step 5) Identify GRI-GC8 test conditions to measure θ_{100}. The resulting θ_{100} from Step 4 is a product specification for the baseline laboratory test transmissivity that should be achieved if tested in accordance with GRI-GC8, Part 6 (2013). Therefore, it is necessary to identify test conditions which simulate site-specific loading conditions and boundary conditions.

Step 6) Calculate the index transmissivity that corresponds to the baseline transmissivity from previous steps. Geocomposite manufacturers typically provide product index transmissivities based on laboratory tests in which the drainage layer is sandwiched between two steel plates as opposed to site specific boundary conditions. The index transmissivity is determined by applying a reduction factor to θ_{100} to account for geotextile/soil intrusion.

Written by: **V. Krishnan** Date: **06/10/16** Reviewed by: **M. Christman** Date: **07/28/16**

Client: **Santee Cooper** Project: **Winyah Generating Station** Project No.: **GSC5242** Task No.: **01BT**

Table E-2.2. Required Degree of Survivability as a Function of Subgrade Conditions, Construction Equipment, and Lift Thickness (GRI-GT13)*

Subgrade Conditions	Low ground-pressure equipment (≤ 3.6 psi)	Medium ground-pressure equipment (> 3.6 psi, ≤ 7.3 psi)	High ground-pressure equipment (> 7.3 psi)
Subgrade has been cleared of all obstacles except grass, leaves, and fine wood debris. Surface is smooth and level so that any shallow depressions and humps do not exceed 18 in. in depth or height. All larger depressions are filled. Alternatively, a smooth working table may be placed.	Low	Moderate	High
Subgrade has been cleared of obstacles larger than small to moderate-sized tree limbs and rocks. Tree trunks and stumps should be removed or covered with a partial working table. Depressions and humps should not exceed 18 in. in depth or height. Larger depressions should be filled.	Moderate	High	Very High
Minimal site preparation is required. Trees may be felled, delimbed, and left in place. Stumps should be cut to project not more than ± 6 in. above subgrade. Fabric may be draped directly over the tree trunks, stumps, large depressions and humps, holes, stream channels, and large boulders. Items should be removed only if placing the fabric and cover material over them will distort the finished road surface.	High	Very High	Not Recommended

* Recommendations are for 6 to 12 in. initial lift thickness. For other initial lift thicknesses:

12 to 18 in.: reduce survivability requirement one level;
18 to 24 in.: reduce survivability requirement two levels;
> 24 in.: reduce survivability requirement three levels

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Table E-2.3. GRI-GT13 Geotextile Strength Property Requirements

			Geotextile Classification (1)					
			Class 1 (high)		Class 2 (moderate)		Class 3 (low)	
Tests	Test Methods	Units	Elongation < 50%	Elongation ≥ 50%	Elongation < 50%	Elongation ≥ 50%	Elongation < 50%	Elongation ≥ 50%
Grab Tensile Strength	ASTM D 4632	lb	315	203	248	158	180	113
Trapezoid Tear Strength	ASTM D 4533	lb	112	79	90	56	68	41
CBR Puncture Strength	ASTM D 6241	lb	630	440	500	320	380	230
Permittivity	ASTM D 4491	sec^{-1}	0.02	0.02	0.02	0.02	0.02	0.02
Apparent Opening Size	ASTM D 4751	in.	0.024	0.024	0.024	0.024	0.024	0.024
Ultraviolet stability (2)	ASTM D 7238	% Ret. @ 500 hrs	50	50	50	50	50	50

Notes: (1) All values are minimum average roll values (MARV) except AOS, which is a maximum average roll value (MaxARV) and UV stability which is a minimum average value.
(2) Evaluation to be on 2-in. strip tensile specimens after 500 hours exposure.

FILTRATION EVALUATION RESULTS

Geotextile Retention

The geotextile must have openings that are small enough to retain fine-grained soil particles so that they do not enter the leachate collection drainage layer, which could result in clogging or flow capacity reduction of the drainage layer. Therefore, the apparent opening size (AOS, hereafter referred to as O_{95}) of the geotextile must be less than a maximum value.

The geocomposite drainage layer in the leachate collection system will be overlain by a protective cover consisting of sand or other permeable material, such as bottom ash. The Unified Soil Classifications specified for the protective cover are SW, SP, SW-SM, SW-SC, SP-SM, and SP-SC. The maximum fines content (particles passing the U.S. No. 200 sieve) specified for the protective cover is 10%. The protective cover will be spread over the liner system geosynthetics in one lift with a medium ground-pressure bulldozer. It is anticipated that protective cover placed with this technique will have a medium dense relative density.

The O_{95} is calculated depending on the type of soil used for the protective cover as follows:

Written by:	V. Krishnan	Date:	06/10/16	Reviewed by:	M. Christman	Date:	07/28/16
Client:	Santee Cooper	Project:	Winyah Generating Station	Project No.:	GSC5242	Task No.:	01BT

Bottom ash is typically a well graded material with a relatively high coefficient of uniformity (e.g., $C_u = 17$ [$C'_u = 10$] in Kim et al., 2005). Applying the retention criterion in Table E-2.1 to the gradation of the bottom ash presented by Kim et al. (2005), the calculated O_{95} is approximately:

$$O_{95} < \frac{13.5}{(C'_u)^{1.7}} d'_{85}$$

$$O_{95} < \frac{13.5}{(10)^{1.7}} (6 \text{ mm}) = 1.62 \text{ mm} = 1620 \text{ µm (U.S. Sieve No. 10-12)}$$

If, for example, a SP soil having $C'_u = 1.5$ and $d'_{85} = 0.15$ mm (U.S. Sieve No. 100) is used as protective cover, then by applying the criterion in Table E-2.1, O_{95} is calculated as:

$$O_{95} \leq 1.5\,(C'_u)^{0.3}\, d'_{85}$$

$$O_{95} \leq 1.5\,(1.5)^{0.3}\,(0.15 \text{ mm}) = 0.25 \text{ mm} = 250 \text{ µm (U.S. Sieve No. 60)}$$

Considering a more well-graded protective cover, for example, a SW-SC soil having $C'_u = 10$ and $d'_{85} = 2.4$ mm (US Sieve No. 8) and $d_{10} = 0.074$ mm (U.S. Sieve No. 200), O_{95} is calculated as:

$$O_{95} < \frac{13.5}{(C'_u)^{1.7}} d'_{85}$$

$$O_{95} < \frac{13.5}{(10)^{1.7}} (2.4 \text{ mm}) = 0.65 \text{ mm} = 650 \text{ µm (U.S. Sieve No. 25-30)}$$

The range of geotextile mass per unit areas anticipated for use as a filtration layer or drainage layer component are 6 to 16 oz/yd^2 (200 to 540 g/m^2). Typical O_{95} values for 6 to 16 oz/yd^2 geotextiles on the current market range from 90 to 850 µm (U.S. Sieve No. 20-170) (IFAI, 2015); thus, products are available that can meet this specification.

Geotextile Permeability

The minimum hydraulic conductivity specified for the protective cover placed over the geocomposite drainage layer is 1×10^{-4} cm/s. Based on the Unified Soil Classifications specified for the protective cover, it is anticipated that the hydraulic conductivity of the protective cover will fall within the range of 1×10^{-4} to 1×10^{-2} cm/s. The geotextile must have openings that are large enough to allow leachate to pass through the retained soil/geotextile interface without significant flow impedance. Thus, the hydraulic conductivity or permeability of the geotextile must be greater than a minimum required value. The hydraulic gradient in the protective cover is assumed to be <10 based on typical values in Giroud (2010). A hydraulic gradient of 10 will be used in the calculations.

Applying the permeability criterion of Table E-2.1, the calculated hydraulic conductivity of the geotextile, $k_{geotextile}$, is:

$$k_{geotextile} \geq \max(i_{soil}\, k_{soil},\, k_{soil})$$
$$\geq 10 \times (1 \times 10^{-2}\ \text{cm/s}) = 0.1\ \text{cm/s}$$

This requirement is achievable by most geotextiles. Note that some manufacturers report the permeability property as "permittivity" (Ψ), which is defined as Ψ=k/t. Based on the range of geotextile mass per unit areas and thicknesses anticipated for the project (6 to 16 oz/yd^2 (200 to 540 g/m^2) and 1.3 to 5.7 mm, respectively), typical $k_{geotextile}$ values (calculated from typical permittivities and thicknesses) for needlepunched non-woven geotextiles are 0.2 to 0.4 cm/s.

Geotextile Porosity

The geotextile filter must have enough openings so that blocking some of them will not significantly clog the geotextile and inhibit flow into the geonet. Thus, the porosity of the geotextile must be greater than a minimum value. As shown in Table E-2.1, for non-woven geotextiles, the geotextile porosity n_g is required to be:

$$n_g > 55\%$$

The porosity criterion requirements apply for the geotextile component of the geocomposite drainage layer. Geotextile porosity is not a property that is directly measured or reported by manufacturers, however it can be calculated as indicated in Table E-2.1 (i.e., $n_g = 1 - \mu_g/(\rho_g\, t_g)$). Typical resulting n_g values for non-woven geotextiles are 50 to 95%. Based on the geotextile density of polypropylene or polyethylene and the range of mass per unit areas and thicknesses anticipated for the project (6 to 16 oz/yd^2 (200 to 540 g/m^2) and 1.3 to 5.7 mm, respectively), the calculated n_g values range from approximately 80% to 90%, which is well in excess of the minimum porosity required to prevent clogging.

Geotextile Thickness

For non-woven geotextiles, such as those proposed for the leachate collection geocomposite drainage layer, the geotextile filter must be thick enough to have a sufficient number of constrictions. From Table E-2.1, the number of constrictions, $N_{constrictions}$, needs to be at least 25.

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The number of constrictions in non-woven geotextiles is a function of mass per unit area, porosity, polymer density, and geotextile fiber diameter:

$$N_{constrictions} = \mu_g / (\rho_g d_f \sqrt{1 - n_g})$$

Based on data for non-woven needlepunched geotextiles presented by Palmeira and Gardoni (2000) and Faure et al. (2006), as well as data compiled by Geosyntec from manufacturers, most non-woven needlepunched geotextiles that have at least 25 constrictions have a minimum thickness of 2.3 mm (90 mils). Therefore, a minimum thickness of 90 mils will be specified for the geotextile filter.

SURVIVABILITY EVALUATION RESULTS

Survivability refers to the ability of the geotextile to withstand the stresses during installation and handling in the field. The degree of survivability is first evaluated using Table E-2.2 with the anticipated installation conditions. The following conditions are conservatively assumed to apply: (i) smooth and level subgrade condition; (ii) initial lift thickness of protective cover placed above geotextile is 24 in.; and (iii) maximum equipment ground pressure of 5 psi (i.e., medium ground-pressure equipment is used). Using Table E-2.2, a "moderate" degree of survivability is used.

In the second step, the minimum required values for the mechanical properties of the geotextile are established from Table E-2.3 based on the "moderate" or "Class 2" survivability requirement. The chart provides minimum required values for two ranges of geotextile extensibility. Values were selected for the more extensible range because this range is applicable to non-woven materials that are required for the geotextile. These survivability requirements, which are outlined in Table E-2.3, apply for the geotextile component of the geocomposite drainage layer.

HYDRAULIC CAPACITY EVALUATION

Step 1) Calculate Required (Design) Transmissivity, θ_{req}

As presented in Appendix E-1, the *HELP* model was used to calculate the required (design) in-plane hydraulic conductivity and equivalent transmissivity of the geocomposite drainage layer of the LCS. The required transmissivity is based on maintaining the peak daily average head on the liner less than or equal to the approximate thickness of the drainage layer. The required (design) transmissivity, θ_{req}, was calculated for each operational condition, and the results are repeated below.

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Case	Operational Condition	Average Waste Thickness (ft)	Soil Cover Thickness (ft)	Design Hydraulic Conductivity (cm/s)	Leachate Drainage Layer Thickness (in.)	Design Transmissivity (m^2/s)
A	Initial Condition	10	0	0.12	0.2	6.2E-06
B	Intermediate Condition	30	0	4.5E-02	0.2	2.3E-06
C-1	Final Grades in Landfill Area 1 - No Cover	52	0	2.8E-02	0.2	1.4E-06
C-2	Final Grades in Landfill Area 2 - No Cover	88	0	2.4E-02	0.2	1.2E-06
D-1	Final Grades in Landfill Area 1 - With Cover	52	2	2.8E-03	0.2	1.4E-07
D-2	Final Grades in Landfill Area 2 - With Cover	88	2	2.4E-03	0.2	1.2E-07

Step 2) Calculate Allowable "Long Term In Soil" Transmissivity, θ_{LTIS}

The allowable "Long Term In Soil" transmissivity, θ_{LTIS} is calculated by applying a factor of safety to increase the minimum required transmissivity. A factor of safety (FS) of 2 was assumed for the LCS geocomposite drainage layer in the analysis.

$$\theta_{LTIS} = \theta_{req} * FS \qquad \text{(Eqn. 1)}$$

The θ_{LTIS} was calculated for each operational condition, as shown below.

Case	θ_{req} (m^2/s)	θ_{LTIS} (m^2/s)
A	6.2E-06	1.2E-05
B	2.3E-06	4.5E-06
C-1	1.4E-06	2.8E-06
C-2	1.2E-06	2.4E-06
D-1	1.4E-07	2.8E-07
D-2	1.2E-07	2.4E-07

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Step 3) Calculate Baseline Geocomposite Transmissivity, θ_{100}

Factors which account for additional long-term transmissivity reduction due to intrusion, creep, chemical clogging, and biological clogging were applied to determine the minimum baseline product transmissivity, θ_{100}, for laboratory testing results as shown in Eqns. 2 and 3.

$$\theta_{LTIS} = \frac{\theta_{100}}{RF_{CR}RF_{CC}RF_{BC}} \quad \text{(Eqn. 2)}$$

Where RF_{CR} = reduction factor for creep, RF_{CC} = reduction factor for chemical clogging and/or precipitation of chemicals, and RF_{BC} = reduction factor for biological clogging.

Creep is the long-term reduction in thickness of the drainage layer under a sustained compressive stress. For landfill leachate collection systems, Koerner (2005) recommends that reduction factors for creep range from 1.4 to 2.0. For the final cover condition (Cases D-1 and D-2), the reduction factor for creep is assumed to be 2.0. The reduction factor for creep is assumed as 1.0 for Cases A and B, and as 1.5 for Cases C-1 and C-2.

GRI (2013) provides guidance for clogging reduction factors for landfill leachate collection systems. Chemical and biological clogging is expected to increase over time as leachate passes through the geocomposite. Thus, the reduction factors for clogging are assumed to increase from initial operational conditions through final cover conditions. GRI (2013) recommends a chemical clogging reduction factor between 1.5 and 2.0 and a biological clogging reduction factor between 1.1 and 1.3 at final conditions. Based on recommendations by GRI, the chemical clogging reduction factors are assumed to increase from 1.0 to 2.0 from initial through final cover conditions. For biological clogging, the assumed reduction factors increase from 1 to 1.2 from initial through final cover conditions.

Rearranging Eqn. 2 and substituting θ_{LTIS} and the reduction factors above, we obtain the following equation:

$$\theta_{100} = \theta_{LTIS}\, RF_{CR}\, RF_{CC}\, RF_{BC} \quad \text{(Eqn. 3)}$$

The θ_{100} was calculated for each operational condition, as shown below.

Cases	θ_{LTIS} (m^2/s)	RF_{CR}	RF_{CC}	RF_{BC}	RF_{total}	θ_{100} (m^2/s)
A	1.2E-05	1.0	1.0	1.0	1.0	1.2E-05
B	4.5E-06	1.0	1.0	1.0	1.0	4.5E-06
C-1	2.8E-06	1.5	1.5	1.1	2.5	7.0E-06
C-2	2.4E-06	1.5	1.7	1.1	2.8	6.8E-06

Cases	θ_{LTIS} (m^2/s)	RF_{CR}	RF_{CC}	RF_{BC}	RF_{total}	θ_{100} (m^2/s)
D-1	2.8E-07	2.0	2.0	1.2	4.8	1.4E-06
D-2	2.4E-07	2.0	2.0	1.2	4.8	1.2E-06

Step 4) Calculate the Critical Operational Case for θ_{100}

Geosyntec contacted SKAPS Industries to obtain typical θ_{100} data for a common biplanar geocomposite on the market. The data correspond to the product, TN 270-2-8, a geocomposite with non-woven geotextile on both sides of the geonet. This does not constitute specification or endorsement of this product; it is merely intended to compare the required transmissivities to a commercially available product to check reasonableness of the design and availability of products. The TN 270-2-8 geocomposite transmissivity was measured at a gradient of 0.1 while sandwiched between sand and a geomembrane for a seating time of 100 hours under four different normal stresses.

To compare the required θ_{100} to the typical θ_{100} on the market, the normal stress on the geocomposite drainage layer expected for each operational condition must be calculated. The stress can be determined from the thickness of fill to be placed above the drainage layer as follows:

$$p = \gamma_{waste} \times h_{waste} + \gamma_{cover} \times h_{cover} \qquad \text{(Eqn. 4)}$$

where: p represents the normal stress; γ represents the density of the waste or the protective cover soil; h_{waste} represents the maximum thickness of the waste for the given operational condition; and h_{cover} represents the thickness of the protective cover. The calculated stress for each operational condition is tabulated below.

Cases	h_{waste} (ft)	Average γ_{waste} (pcf)	h_{cover} (ft)	γ_{cover} (pcf)	Stress (psf)	θ_{100} (m^2/s)
A	10	100	0	120	1,000	1.2E-05
B	30	100	0	120	3,000	4.5E-06
C-1	91	100	0	120	9,100	7.0E-06
C-2	210	100	0	120	21,000	6.8E-06
D-1	91	100	2	120	9,340	1.4E-06
D-2	210	100	2	120	21,240	1.2E-06

The required (minimum) θ_{100} is plotted versus the calculated stress in Figure E-2.1. The expected θ_{100} data for a typical biplanar geocomposite (SKAPS TN 270-2-8) is shown for reference. As shown in Figure E-2.1, the required θ_{100} for all operational cases are less than θ_{100}

for a typical biplanar geocomposite at corresponding stress conditions. Therefore, the geocomposite should provide adequate hydraulic capacity for operational conditions. By inspection of Figure E-2.1, the most critical operational condition for the geocomposite drainage layer is Case C-2. The critical condition occurs where the difference between required θ_{100} and measured θ_{100} is the least. The required θ_{100} is 6.8×10^{-6} m^2/s and the applied stress is approximately 21,000 psf.

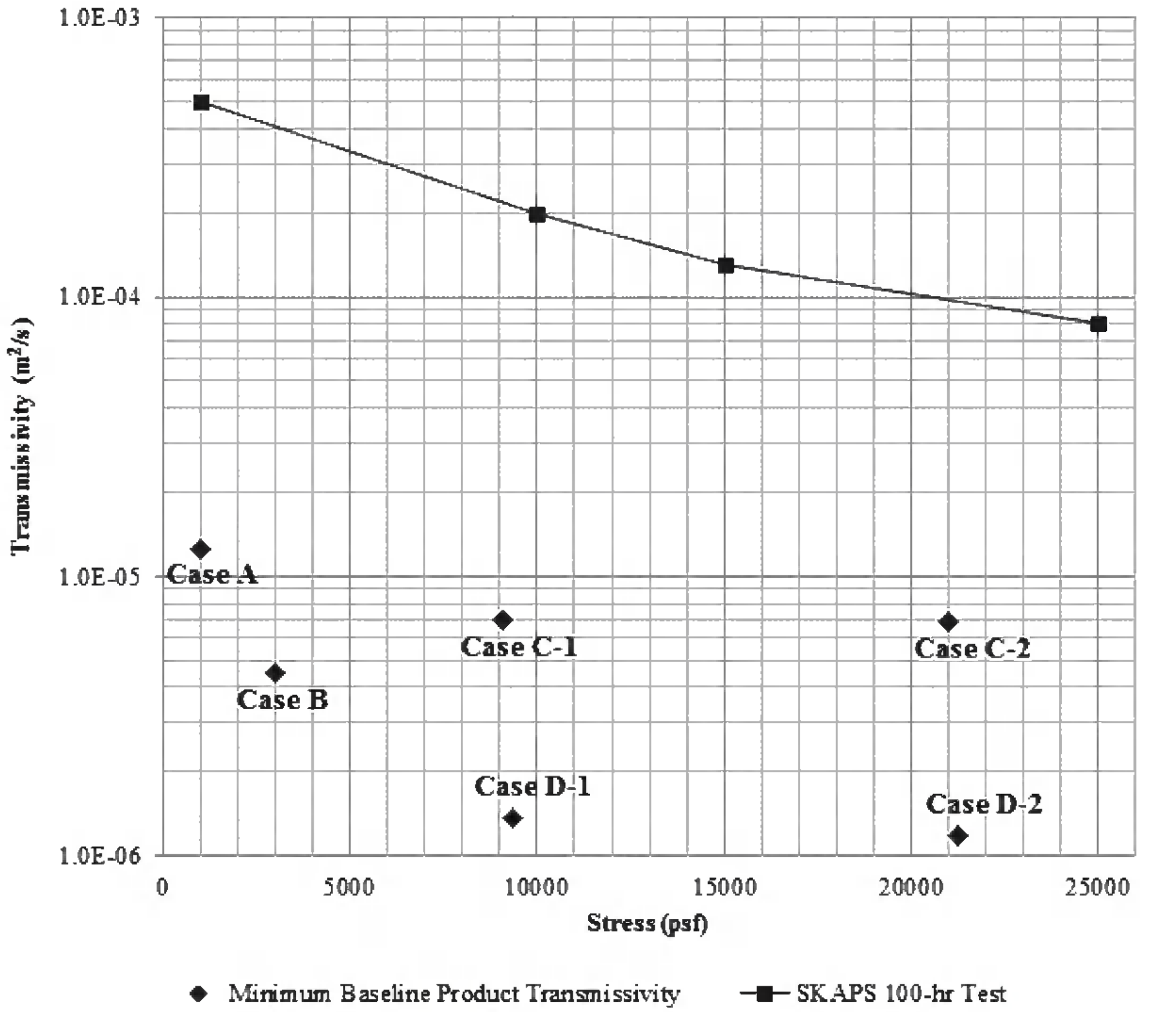

Figure E-2.1. Comparison of Required θ_{100} to Typical θ_{100} Test Results at Various Normal Stresses. Note: The typical product information shown does not constitute an endorsement of these products, nor does this require the use of any specific manufacturer or product. This information is presented for comparison purposes only.

Step 5) Identify Site-Specific Conditions for Evaluating θ_{100}

The testing conditions to be used in evaluating θ_{100} using GRI Standard GC8, Part 6 are: (i) the testing configuration (i.e., stratum configuration); (ii) the applied stress; and (iii) the hydraulic gradient. These conditions are specified below:

- The recommended testing configuration for transmissivity testing of the LCS drainage layer should consist of a 60-mil HDPE geomembrane on one side of the geocomposite specimen (to simulate site-specific liner design) and sand on the other side of the geocomposite specimen (to simulate the protective cover of sand or bottom ash).
- The stress to be applied in testing the leachate drainage layer should be equivalent to the stress at the most critical condition found in Step 4. As noted in Step 4, the most critical operational condition for the geocomposite occurs in Case C-2. Therefore, the stress on the leachate drainage layer geocomposite material to be used in determining θ_{100} is 21,000 psf.
- The geocomposite drainage layer slopes at about 2.25% on the cell floor towards the leachate corridor. Therefore, the hydraulic gradient to be used in determining θ_{100} for the geocomposite is 0.02.

Step 6) Determine Index Transmissivity, θ_{INDEX}, Based on θ_{100}

While the θ_{100} given above is suitable for use as a specification if desired, it is usually more convenient to report the transmissivity between two steel plates for a short duration test since manufacturers of geocomposite drainage materials often present the hydraulic capacities of their product in this manner. These transmissivities are usually higher than those that would be obtained using the site specific boundary conditions of soil on one side and a geomembrane on the other side. This is because the short duration test does not completely account for the time-delayed intrusion of the geotextile into the transmissive core resulting from the deformation of the geotextile under sustained loading. Additionally, the steel plate boundary condition of the short duration test will not account for a reduction in transmissivity due to particle migration into the transmissive core.

To compare the specified θ_{100} of the leachate drainage layer with index values reported by the manufacturer, factors can be applied to account for the reduction of the transmissivity that may be experienced due to intrusion and particulate clogging when testing the drainage layer with boundary materials other than steel plates. The index transmissivity, θ_{INDEX}, which accounts for intrusion and particulate clogging, can be determined as shown in Eqn. 5:

$$\theta_{INDEX} = \theta_{100} * RF_{INT}\, RF_{PC} \quad \text{(Eqn. 5)}$$

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Koerner (2005) recommends using an intrusion reduction factor (RF_{INT}) between 1.5 and 2.0. The intrusion factor of 1.5 is assumed for initial conditions and 2.0 for final conditions. The geotextile is expected to adequately retain particulates to avoid potential clogging of the transmissive core; however, a particulate clogging reduction factor (RF_{PC}) of 1.1 is applied. The index transmissivity, θ_{INDEX}, for the geocomposite at the critical condition specified in Step 4 (Case A) is found to be:

$$\theta_{INDEX} = 6.8\times10^{-6}\ (m^2/s) \times 2.0 \times 1.1$$
$$\theta_{INDEX} = 1.5\times10^{-5}\ m^2/s$$

CONCLUSIONS

Based on the evaluations herein, the following specification is recommended for the leachate collection system drainage layer geocomposite.

- Filtration of Geotextile Components:
  - Apparent Opening Size, $O_{95} \leq 210$ μm (U.S. Sieve No. 70)
  - Geotextile Water Permeability, $k_{geotextile} \geq 0.1$ cm/s for geotextile component of the geocomposite overlain by the protective cover
  - Geotextile Thickness, $t_{geotextile} \geq 90$ mils for geotextile component of the geocomposite overlain by the protective cover
- Survivability (Mechanical) Properties of Geotextile Components:
  - Grab Strength = 158 lbs
  - Trapezoid Tear Strength = 56 lbs
  - CBR Puncture Strength = 320 lbs
- Hydraulic Capacity (Transmissivity) of Geocomposite Drainage Layer
  - $\theta_{INDEX} = 1.5\times10^{-5}\ m^2/s$ (when tested between two steel plates with an applied stress of 21,000 psf at a gradient of 0.02) based on the site-specific design calculations

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REFERENCES

Faure, Y.H, Baudoin, P., and Pierson, O., "A Contribution for Predicting Geotextile Clogging During Filtration of Suspended Solids", *Geotextiles and Geomembranes*, 24(1), 2006, pp. 11-20.

Geosynthetic Research Institute (GRI), "Standard Guide for Determination of the Allowable Flow Rate of a Drainage Geocomposite", *GRI Standard GC8,* 2013.

Geosynthetic Research Institute (GRI), "Standard Specification for Test Methods and Properties for Geotextiles Used as Separation Between Subgrade Soil and Aggregate", *GRI Standard GT-13*, 2012.

Giroud, J.P., "Development of Criteria for Geotextiles and Granular Filters", *Proceedings of the 9th International Conference on Geosynthetics*, Guaruja, Brazil, Vol. 1, 2010, pp. 45-64.

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

Holtz, R.D., Christopher, B.R., and Berg, R.R., "Geosynthetic Design and Construction Guidelines Participant Notebook, National Highway Institute (NHI) Course No. 13213", U.S. Department of Transportation, Federal Highway Administration, Rep. No. FHWA-HI-95-038, 1998.

Industrial Fabrics Association International (IFAI), *Geosynthetics Specifier's Guide 2015*, Roseville, MN, January 2015.

Kim, B., Prezzi, M., Salgado, R., "Geotechnical Properties of Fly and Bottom Ash Mixtures for Use in Highway Embankments", *Journal of Geotechnical and Geoenvironmental Engineering*, July 2005, pp. 914-924.

Koerner, R.M., "*Designing with Geosynthetics*", Fifth Edition, Prentice Hall, 2005, 767 p.

Palmeira, E.M. and Gardoni, M.G., "The Influence of Partial Clogging and Pressure on the Behavior of Geotextiles in Drainage Systems", *Geosynthetics International*, Vol. 7, Nos. 4-6, 2000, pp. 403-431.

APPENDIX E-3

LEACHATE PIPE HYDRAULIC DESIGN

Written by:	V. Krishnan	Date:	06/10/16	Reviewed by:	M. Christman	Date:	07/01/16
Client:	Santee Cooper	Project:	Winyah Generating Station	Project No.:	GSC5242	Task No.:	01BT

APPENDIX E-3

LEACHATE PIPE HYDRAULIC DESIGN

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SOUTH CAROLINA GEOSYNTEC CONSULTANTS, INC. No. 773 CERTIFICATE OF AUTHORIZATION

SEALED FOR
CALCULATION PAGES E-3-1
THROUGH E-3-3

PURPOSE

The purpose of this analysis is to evaluate the flow capacity of the leachate collection corridors for the proposed Class Three Landfill at the Winyah Generating Station (WGS). Each subcell of the landfill has a leachate collection corridor which collects leachate from the leachate drainage layer and conveys it to a leachate collection sump. Within each subcell, the leachate collection corridor is centrally located and slopes at 1% (min) towards the sump. The leachate collection corridor consists of a perforated high-density polyethylene (HDPE) standard dimension ratio (SDR)-11 pipe embedded within a granular drainage media.

METHOD OF ANALYSIS

The leachate collection pipe flow capacity should be greater than the leachate flow entering the pipe. The pipe flow capacity is calculated using Manning's equation (Chow, 1959) as follows:

$$Q_p = \frac{1.486 R_h^{0.66} i_p^{0.5} A_p}{n} \qquad \text{(Eqn. 1)}$$

where:

Q_p = pipe flow capacity, cfs;

R_h = hydraulic radius, ft (i.e., ratio of the flow area to the perimeter of the wetted area, $\frac{B_i}{4}$, where B_i is pipe inner diameter, ft);

i_p = hydraulic gradient (i.e., slope of the pipe);

A_p = cross-sectional area of the pipe, ft^2; and

n = Manning's roughness coefficient.

For a pipe with a circular cross section that is flowing full, Manning's equation assumes steady uniform fully turbulent conditions. The maximum flow rate of leachate entering the leachate collection pipe was calculated using peak daily leachate generation rates (see Table E-1.5 in Appendix E-1). The calculated peak daily generation rate for the most critical condition, Case A (initial condition with 10 feet of waste), is 261 gallons per acre per day (gpad). Subcell 5N is the largest proposed subcell, having a lined area of 9.4 acres, and is therefore assumed to generate the highest peak daily flow rate. The maximum daily flow rate from Subcell 5N is compared to the capacity of the leachate corridor collector pipe to check that the calculated collector pipe flow capacity is greater than the calculated maximum flow rate.

CALCULATIONS

The proposed collector pipe for the leachate collection corridor is a perforated 6 in. diameter HDPE SDR-11 pipe. The flow capacity of the 6 in. diameter pipe is calculated using Eqn. 1, where:

n = Manning's roughness coefficient = 0.009 (from Chow, 1959)

i_p = hydraulic gradient = 1 percent (slope along drainage corridor)

B_i = 5.42 in. / 12 in./ft = 0.4517 ft

$$R_h = \text{hydraulic radius} = \frac{B_i}{4} = \frac{0.4517\ \text{ft}}{4} = 0.113\ \text{ft}$$

$$A_p = \text{cross-sectional area of the pipe} = \frac{\pi B_i^2}{4} = \frac{\pi\,(0.4517\ \text{ft})^2}{4} = 0.16\ \text{ft}^2$$

Based on the parameters above, the flow capacity of the 6 in. diameter pipe is calculated as follows:

$$Q_p = \frac{1.486 R_h^{0.66} i_p^{0.5} A_p}{n}$$

$$Q_p = \frac{1.486(0.113)^{0.66}(0.01)^{0.5}(0.16)}{(0.009)}$$

Q_p = 0.618 cfs = 399,400 gallons per day (gpd)

The peak daily flow rate from Subcell 5N is calculated to be 2,450 gpd (i.e., 9.4 acres × 261 gpad).

Comparison between the calculated peak daily flow rate of leachate into the leachate collection corridor and the calculated pipe flow capacity indicates that the leachate collection corridor possesses sufficient capacity to convey the peak daily leachate generated at the facility.

SUMMARY AND CONCLUSIONS

The highest peak daily leachate collection rate from both the floor and the sideslopes by a single leachate collection corridor is 2,450 gpd. The proposed 6-in. diameter collection pipe with a calculated flow capacity of 399,400 gpd has adequate hydraulic capacity to convey the peak daily leachate generated.

For reference, it is also noted that using the same approach, the calculated flow capacity of the pipe with a slope that is flattened (e.g., due to any foundation settlement) from 1% to 0.1% is approximately 128,000 gpd. Settlement calculations presented elsewhere in the Engineering Report reveal a predicted post-settlement slope that has not grade reversals, and is steeper than 0.1%. The hydraulic capacity of the leachate collection pipe after the predicted settlement is far in excess of the predicted leachate flow rates, indicating that it can perform adequately to convey flows, even after settlement.

REFERENCES

Chow, V.T., *Open Channel-Hydraulics,* McGraw-Hill, 1959.

APPENDIX E-4

LEACHATE SUMP AND TRANSMISSION SYSTEM DESIGN

Written by:	V. Krishnan	Date:	06/17/16	Reviewed by:	M. Christman	Date:	07/08/16
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APPENDIX E-4

LEACHATE SUMP AND TRANSMISSION SYSTEM DESIGN

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CALCULATION PAGES E-4-1
THROUGH E-4-40

PURPOSE

The purpose of this calculation package is to evaluate the design of the leachate collection sumps and leachate transmission system (LTS) forcemain for the proposed Class Three Landfill (composed of Landfill Area 1, with Cells 1 to 3, and Landfill Area 2, with Cells 4 to 8) at the Winyah Generating Station (WGS). The calculations are used to:

- estimate the storage capacity of the leachate collection sumps;
- estimate the required cycle time of the submersible pump to remove leachate from the sump; and
- develop design requirements for the submersible sump pumps and the LTS forcemain.

METHOD OF ANALYSIS

Leachate Storage Capacity of Sumps

The proposed sump has the shape of an inverted truncated pyramid with a square base. The volume of a truncated pyramid is calculated as follows:

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$$V = \frac{1}{3}\left(A_1 + A_2 + \sqrt{A_1 A_2}\right)H \quad \text{(Eqn. 1)}$$

where:

V = Volume of truncated pyramid;

A_1 = Area of base;

A_2 = Area of top; and

H = Height of truncated pyramid.

The volume of the solid particles of the granular drainage material reduces the volume available for leachate storage. The effective leachate storage capacity of sump is calculated as:

$$V_s = V \times n \quad \text{(Eqn. 2)}$$

where:

V_s = Effective leachate storage capacity of sump; and

n = Porosity of granular drainage material.

The effective leachate storage capacity of the sump is calculated as the storage volume available between the turn-on level and turn-off level of the submersible pump. The turn-off level of the submersible pump is typically above the base of the sump, and the volume between the pump turn-off level and the base of the sump will remain saturated and therefore, will not contribute to the effective storage capacity of the sump.

Cycle Time of Submersible Pump

The pump-on duration is equal to the amount of time required by the pump to reduce the leachate level from the pump turn-on level to the pump turn-off level. The pump-on duration is calculated as:

$$t_1 = \frac{V_s}{(Q_{pump} - Q_{in})} \quad \text{(Eqn. 3)}$$

where:

t_1 = Pump-on duration;

Q_{pump} = Pump flow rate; and

Q_{in} = Flow rate of leachate into the sump.

The pump-off duration is equal to the amount of time it takes for the sump to fill up from the pump turn-off level to the pump turn-on level. The pump-off duration is:

$$t_2 = \frac{V_s}{Q_{in}} \qquad \text{(Eqn. 4)}$$

where:

t_2 = Pump-off duration.

Sump Pump and LTS Forcemain Design

The sump pump and LTS forcemain design was evaluated by modeling the hydraulic networks for the three representative cases (Cases 1, 2, and 3) described below using EPANET Pipe Network Modeling Software, version 2.0 (U.S. Environmental Protection Agency, 2000). The model considers sumps in the landfill cells, submersible pumps, riser pipes, and the LTS forcemain network. EPANET assumes that all pipes in the network are flowing full at all times.

Leachate pumped from the sumps will be conveyed to various locations at the site by the LTS forcemain. The three representative cases considered are as follows:

- Case 1: During initial filling operations in Landfill Area 1, leachate from cells in Landfill Area 1 will be conveyed and discharged to the Cooling Pond Intake Canal.
- Case 2: After Landfill Area 1 reaches final grades and filling starts in Landfill Area 2, leachate from cells in Landfill Areas 1 and 2 will be conveyed to the Temporary Leachate/ Contact Water Storage Pond.
- Case 3: During latter stages of filling Landfill Area 2 and post-closure, leachate from cells in Landfill Areas 1 and 2 will be conveyed to the Permanent Leachate/ Contact Water Storage Pond.

LTS network diagrams for Cases 1, 2, and 3 are shown in Figure E-4.1, Figure E-4.2, and Figure E-4.3, respectively. The proposed LTS forcemain pipe is a 4-in. diameter SDR 17 high-density polyethylene (HDPE) pipe. The sump pumps will connect to the forcemain via a 3-in. diameter SDR 17 HDPE pipe.

Since the sump pumps will be automatically controlled based on leachate levels in the sump, some of the sump pumps may be on at the same time. To be conservative, the analysis assumes steady-state conditions with all of the sump pumps on at the same time.

The Hazen-Williams equation was used to approximate the energy loss in the piping system. The Hazen-Williams roughness coefficient for HDPE pipe was conservatively assumed as 130 (U.S. Environmental Protection Agency, 2000). The frictional head losses in the system were calculated the summation of losses for the fittings, valves, and pipe lengths between the sumps and the LTS forcemain outlet. The loss coefficient for the 3-in. diameter riser pipe was assumed to be 30 to account for the various fittings and valves. The loss coefficient for the 4-in. diameter forcemain piping was assumed to be zero because relatively few fittings and valves are proposed and frictional head loss due to pipe length will be more significant.

The calculated maximum hydraulic pressure in the pipe network was used to calculate the factor of safety against pipe rupture. The factor of safety against hydraulic rupture of the forcemain pipe is calculated as:

$$FS_{hr} = \frac{P_{rating}}{P_{max}} \qquad \text{(Eqn. 5)}$$

where:

FS_{hr} = Factor of safety against hydraulic rupture;

P_{rating} = Pressure rating of the LTS forcemain pipe material; and

P_{max} = maximum hydraulic pressure in the LTS forcemain.

The pressure rating of the SDR 17 HDPE riser and forcemain pipes was determined as 125 psi based on guidance provided in the Handbook of PE Pipe (Plastics Pipe Institute, retreived 07-July-2016). This incorporates the effects of temperature of the fluid conveyed, effects of chemicals in the fluid conveyed on HDPE, and surge and water hammer effects.

CALCULATIONS

Total Volume of Sump

The proposed leachate collection sump is 4.0 ft deep, has base dimensions of 20 ft × 20 ft, top dimensions of 44 ft × 44 ft, and a sideslope of 3H:1V, as shown in Figure E-4.4. The submersible pump turn-off level is typically 18 in. above the base of the sump, so the assumed operating depth of the sump is 2.5 feet.

From Figure E4-4, the operating parameters are:

$$A_1 = 29 \text{ ft} \times 29 \text{ ft} = 841 \text{ ft}^2;$$

$$A_2 = 44 \text{ ft} \times 44 \text{ ft} = 1{,}936 \text{ ft}^2; \text{ and}$$

$$H = 2.5 \text{ ft}$$

Therefore, the total operating volume of the sump is:

$$V = \frac{1}{3}\left(A_1 + A_2 + \sqrt{A_1 A_2}\right)H$$

$$V = (0.333)(841 \text{ ft}^2 + 1{,}936 \text{ ft}^2 + [(841 \text{ ft}^2)(1{,}936 \text{ ft}^2)]^{0.5})(2.5 \text{ ft})$$

$$\underline{V = 3{,}378 \text{ ft}^3}$$

Effective Volume of Sump

The proposed sump is filled with granular drainage media with a porosity assumed to be approximately 0.3. Therefore, the effective volume of the sump is:

$$V_s = V \times n$$

$$V_s = (3{,}378 \text{ ft}^3)(0.3)$$

$$\underline{V_s = 1{,}013 \text{ ft}^3}$$

$$V_s = (1{,}013 \text{ ft}^3)(7.48 \text{ gal/ft}^3)$$

$$\underline{V_s = 7{,}580 \text{ gallons}}$$

Pump-on/Pump-off Duration

The peak daily flow rate of leachate into the sump (Q_{in}) was calculated using leachate generation rates obtained from the Hydrologic Evaluation of Landfill Performance (*HELP*) model (see Appendix E-1). The peak daily flow rate for the most critical condition is 261 gallons per acre per day (gpad). The sump of Subcell 5N serves an area of 9.4 acres. Therefore, the calculated peak daily flow rate to the Subcell 5N sump is 2,450 gallons per day (gpd) (1.7 gallons per minute (gpm)).

For comparison purposes, the average annual flow rate of leachate into the Subcell 5N sump (Q_{in}) was also calculated. The average annual flow rate for the most critical condition, Case A, is 75.4 gpad. The calculated average annual leachate flow rate into the sump of Subcell 5N is 709 gpd (0.5 gpm).

A submersible leachate sump pump operating at a rate (Q_{pump}) of approximately 25 gpm was assumed. Therefore, for the peak daily leachate flow, the pump-on duration is:

$$t_1 = V_s/(Q_{pump} - Q_{in})$$

$$t_1 = 7{,}580 \text{ gal}/(25 \text{ gpm} - 1.7 \text{ gpm})$$

$$t_1 = 325 \text{ min } (5.4 \text{ hr})$$

and the pump-off duration is:

$$t_2 = V_s/Q_{in}$$

$$t_2 = 7{,}580 \text{ gal}/1.7 \text{ gpm}$$

$$t_2 = 4{,}459 \text{ min } (74.3 \text{ hr})$$

With a pump-on duration of 5.4 hr and a pump-off duration of 74.3 hr, a full on and off pump cycle is 79.7 hr (~3.3 days). Most pump manufacturers recommend that the sump pump cycle time be more than 15 min, so a cycle time of 79.7 hr is an acceptable cycle time for the peak daily flow condition.

For the average annual case, the pump-on duration is:

$$t_1 = V_s/(Q_{pump} - Q_{in})$$

$$t_1 = 7{,}580 \text{ gal}/(25 \text{ gpm} - 0.5 \text{ gpm})$$

$$t_1 = 309 \text{ min } (5.2 \text{ hr})$$

and the pump-off duration is:

$$t_2 = V_s/Q_{in}$$

$$t_2 = 7{,}580 \text{ gal}/0.5 \text{ gpm}$$

$$t_2 = 15{,}160 \text{ min } (252.7 \text{ hr or } 10.5 \text{ days})$$

With a pump-on duration of 5.2 hr and a pump-off duration of 252.7 hr, a full on and off pump cycle is 257.9 hr (~ 10.7 days). Since most pump manufacturers recommend that sump pump cycle times be more than 15 min, a cycle time of 258 hours is an acceptable cycle time for the average daily condition.

Sump Pump and LTS Forcemain Design

A summary table of input parameters, and total head and pressure computed by the EPANET model at network nodes for Case 1 is shown in Table E-4.1. The calculated hydraulic heads at the network nodes varied from 20 ft to 55 ft, and the maximum computed hydraulic pressure at the nodes was 14.6 psi. A summary table of input parameters and computed flow in the network links for Case 1 is shown in Table E-4.2; the computed flow varied from 40 gpm to 252 gpm.

A summary table of input parameters, and total head and pressure computed by the EPANET model at network nodes for Case 2 is shown in Table E-4.3. The calculated hydraulic heads at the network nodes varied from 25 ft to 97 ft, and the maximum computed hydraulic pressure at the nodes was 32.7 psi. A summary table of input parameters and computed flow in the network links for Case 2 is shown in Table E-4.4; the computed flow varied from 16 gpm to 268 gpm.

A summary table of input parameters, and total head and pressure computed by the EPANET model at network nodes for Case 3 is shown in Table E-4.5. The calculated hydraulic heads at the network nodes varied from 24 ft to 103 ft, and the maximum computed hydraulic pressure at the nodes was 35.5 psi. A summary table of input parameters and computed flow in the network links for Case 3 is shown in Table E-4.6; the computed flow varied from 8 gpm to 329 gpm. Note that a higher hydraulic head for the Subcell 5S sump pump in Case 3 is required to accommodate the increased static head associated with the sideslope riser pipe extension proposed during filling of Cell 8. The submersible pump curve assumed for the EPANET models is shown in Figure E-4.5, and the pump curve for Subcell 5S (designed to accommodate a higher head) is shown in Figure E-4.6.

During the EPANET model simulation for Case 2 and Case 3, negative pressures were calculated for some nodes, as shown in Table E-4.3 and E-4.5. The calculated negative pressure indicates that gravitational forces in the downgradient reach of pipe are sufficient to convey full pipe flow for the given pipe diameter. Because the proposed forcemain will be open to the atmosphere at the discharge point, some portions of the system may flow under open-channel conditions rather than pressured flow as the model assumes.

Based on the maximum nodal pressures computed for Cases 1, 2, and 3, the factor of safety against hydraulic rupture is shown in Table E-4.7. The lowest computed factor of safety was 3.5 (for Case 3). Therefore, the factor of safety is considered adequate for all cases (greater than 3.0 for all cases). The EPANET output files and results for Cases 1, 2, and 3 are provided in Appendix E-4.A.

CONCLUSIONS

Calculations for the leachate storage capacity of sump and submersible pump cycle duration indicate that, for a given (assumed) submersible sump pump of 25 gpm, the proposed leachate sump has adequate storage capacity to provide acceptable pump cycle times considering peak and average annual operation rates.

Based on the results above, typical leachate pumps with capacities on the order of 25 gallons per minute that provide a hydraulic head on the order of 60 ft are suitable for managing leachate at the site. One exception is that for Subcell 5S during the Case 3 condition where a leachate pump with a hydraulic head on the order of 120 ft is needed.

This calculation does not require a specific size or capacity of the sump pump. It merely recommends a typical size based on the anticipated flow rates and good practice for pump operation. Other pump capacities can result in adequate performance, and may be selected by facility operations based on actual leachate generation rates and consistent with the concepts presented in this calculation.

Calculations for the factor of safety against hydraulic rupture of the LTS forcemain indicate that the forcemain has adequate capacity to withstand the calculated peak hydraulic pressures in the forcemain network.

REFERENCES

Plastics Pipe Institute. *Handbook of PE Pipe - Second Edition.* Retrieved July 7, 2016, from Plastics Pipe Institute: https://plasticpipe.org/publications/pe-handbook.html

U.S. Environmental Protection Agency. (2000). *EPANET 2 User's Manual.* Cincinnati, OH: U.S. Environmental Protection Agency.

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TABLE E-4.1. EPANET RESULTS FOR NETWORK NODES IN CASE 1

	Elevation	Base Demand	Head	Pressure
Node ID	ft	GPM	ft	psi
Junc 1	21.9	0	51.9	13.0
Junc 2	22.2	0	43.8	9.4
Junc 3	21.9	0	54.3	14.1
Junc 4	23.7	0	46.8	10.0
Junc 5	21.4	0	55.0	14.6
Junc 6	23.6	0	47.5	10.4
Junc 7	36.0	0	50.0	6.1
Junc 8	36.0	0	41.7	2.5
Junc 9	36.0	0	52.5	7.2
Junc 10	36.0	0	44.7	3.8
Junc 11	36.0	0	53.2	7.5
Junc 12	36.0	0	45.5	4.1
Junc 14	36.0	0	37.8	0.8
Resvr 13	20.0	na	20.0	0.0
Resvr 15	27.6	na	27.6	0.0
Resvr 16	25.4	na	25.4	0.0
Resvr 17	27.7	na	27.7	0.0
Resvr 18	26.0	na	26.0	0.0
Resvr 19	26.2	na	26.2	0.0
Resvr 20	25.9	na	25.9	0.0

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TABLE E-4.2. EPANET RESULTS FOR NETWORK LINKS IN CASE 1

Link ID	Length ft	Diameter in.	Hazen-Williams Roughness Coefficient	Flow GPM	Velocity fps
Pipe 1	44.5	3	130	41.1	1.9
Pipe 2	43.7	3	130	44.1	2.0
Pipe 3	44.7	3	130	40.2	1.8
Pipe 4	42.4	3	130	43.6	2.0
Pipe 5	46.1	3	130	39.7	1.8
Pipe 6	41.1	3	130	43.3	2.0
Pipe 7	514	4	130	79.9	2.0
Pipe 8	516	4	130	87.0	2.2
Pipe 9	516	4	130	39.7	1.0
Pipe 10	515	4	130	43.3	1.1
Pipe 11	1139	4	130	121.0	3.1
Pipe 12	312	4	130	131.1	3.4
Pipe 13	427	4	130	252.1	6.4
Pump 14	na	na	na	41.1	0.0
Pump 15	na	na	na	44.1	0.0
Pump 16	na	na	na	40.2	0.0
Pump 17	na	na	na	43.6	0.0
Pump 18	na	na	na	39.7	0.0
Pump 19	na	na	na	43.3	0.0

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TABLE E-4.3. EPANET RESULTS FOR NETWORK NODES IN CASE 2

	Elevation	Base Demand	Head	Pressure
Node ID	ft	GPM	ft	psi
Junc 1	21.9	0	96.2	32.2
Junc 2	22.2	0	94.6	31.4
Junc 3	21.9	0	96.7	32.4
Junc 4	23.7	0	95.3	31.1
Junc 5	21.4	0	96.8	32.7
Junc 6	23.6	0	95.5	31.1
Junc 7	36.0	0	95.8	25.9
Junc 8	36.0	0	94.2	25.2
Junc 9	36.0	0	96.3	26.1
Junc 10	36.0	0	94.9	25.5
Junc 11	36.0	0	96.5	26.2
Junc 12	36.0	0	95.1	25.6
Junc 14	36.0	0	93.4	24.9
Junc 29	21.0	0	87.1	28.6
Junc 30	20.9	0	38.6	7.7
Junc 31	21.0	0	92.0	30.8
Junc 32	23.1	0	28.8	2.5
Junc 33	21.2	0	94.2	31.6
Junc 34	22.3	0	97.1	32.4
Junc 35	21.2	0	95.4	32.2
Junc 36	21.4	0	96.9	32.7

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TABLE E-4.3. EPANET RESULTS FOR NETWORK NODES IN CASE 2 (CONTD.)

	Elevation	Base Demand	Head	Pressure
Node ID	ft	GPM	ft	psi
Junc 37	40.0	0	86.4	20.1
Junc 38	40.0	0	36.2	-1.7
Junc 39	40.0	0	91.5	22.3
Junc 40	34.5	0	26.2	-3.6
Junc 41	40.0	0	93.8	23.3
Junc 42	40.0	0	96.7	24.6
Junc 43	40.0	0	95.1	23.9
Junc 44	40.0	0	96.5	24.5
Junc 47	40.0	0	88.4	21.0
Resvr 15	27.6	na	27.6	0.0
Resvr 16	25.4	na	25.4	0.0
Resvr 17	27.7	na	27.7	0.0
Resvr 18	26.0	na	26.0	0.0
Resvr 19	26.2	na	26.2	0.0
Resvr 20	25.9	na	25.9	0.0
Resvr 21	25.0	na	25.0	0.0
Resvr 22	24.9	na	24.9	0.0
Resvr 23	25.0	na	25.0	0.0
Resvr 24	27.1	na	27.1	0.0
Resvr 25	25.2	na	25.2	0.0
Resvr 26	26.3	na	26.3	0.0
Resvr 27	25.2	na	25.2	0.0
Resvr 28	25.4	na	25.4	0.0
Resvr 46	26.0	na	26.0	0.0

TABLE E-4.4. EPANET RESULTS FOR NETWORK LINKS IN CASE 2

Link ID	Length ft	Diameter in.	Hazen-Williams Roughness Coefficient	Flow GPM	Velocity fps
Pipe 1	44.5	3	130	17.5	0.8
Pipe 2	43.7	3	130	19.0	0.9
Pipe 3	44.7	3	130	17.1	0.8
Pipe 4	42.4	3	130	19.6	0.9
Pipe 5	46.1	3	130	16.4	0.8
Pipe 6	41.1	3	130	19.5	0.9
Pipe 7	514	4	130	33.5	0.9
Pipe 8	516	4	130	39.1	1.0
Pipe 9	516	4	130	16.4	0.4
Pipe 10	515	4	130	19.5	0.5
Pipe 11	1139	4	130	51.0	1.3
Pipe 12	312	4	130	58.1	1.5
Pipe 21	60.1	3	130	23.7	1.1
Pipe 22	60.4	3	130	45.5	2.1
Pipe 23	60	3	130	20.2	0.9
Pipe 24	36.1	3	130	49.7	2.3
Pipe 25	59.5	3	130	18.5	0.8
Pipe 26	55.8	3	130	17.0	0.8
Pipe 27	59.4	3	130	17.5	0.8
Pipe 28	58.7	3	130	16.4	0.7
Pipe 37	587	4	130	50.9	1.3

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TABLE E-4.4. EPANET RESULTS FOR NETWORK LINKS IN CASE 2 (CONTD.)

	Length	Diameter	Hazen-Williams	Flow	Velocity
Link ID	ft	in.	Roughness Coefficient	GPM	fps
Pipe 38	586	4	130	69.5	1.8
Pipe 40	1522	4	130	222.3	5.7
Pipe 41	596	4	130	17.0	0.4
Pipe 46	514	4	130	89.6	2.3
Pipe 47	74	4	130	198.7	5.1
Pipe 48	563	4	130	109.1	2.8
Pipe 13	1506	4	130	33.4	0.9
Pipe 39	76	4	130	49.7	1.3
Pipe 42	218	4	130	267.9	6.8
Pump 14	na	na	na	17.5	0.0
Pump 15	na	na	na	19.0	0.0
Pump 16	na	na	na	17.1	0.0
Pump 17	na	na	na	19.6	0.0
Pump 18	na	na	na	16.4	0.0
Pump 19	na	na	na	19.5	0.0
Pump 29	na	na	na	23.7	0.0
Pump 30	na	na	na	45.5	0.0
Pump 31	na	na	na	20.2	0.0
Pump 32	na	na	na	49.7	0.0
Pump 33	na	na	na	18.5	0.0
Pump 34	na	na	na	17.0	0.0
Pump 35	na	na	na	17.5	0.0
Pump 36	na	na	na	16.4	0.0

TABLE E-4.5. EPANET RESULTS FOR NETWORK NODES IN CASE 3

	Elevation	Base Demand	Head	Pressure
Node ID	ft	GPM	ft	psi
Junc 1	21.9	0	103.1	35.2
Junc 2	22.2	0	102.7	34.9
Junc 3	21.9	0	103.2	35.3
Junc 4	23.7	0	103.0	34.4
Junc 5	21.4	0	103.2	35.5
Junc 6	23.6	0	103.1	34.4
Junc 7	36.0	0	103.0	29.0
Junc 8	36.0	0	102.5	28.8
Junc 9	36.0	0	103.1	29.1
Junc 10	36.0	0	102.8	29.0
Junc 11	36.0	0	103.2	29.1
Junc 12	36.0	0	102.9	29.0
Junc 14	36.0	0	102.2	28.7
Junc 29	21.0	0	100.7	34.5
Junc 30	20.9	0	101.4	34.9
Junc 31	21.0	0	97.3	33.0
Junc 32	23.1	0	43.4	8.8
Junc 33	21.2	0	92.2	30.8
Junc 34	22.3	0	45.8	10.2
Junc 35	21.2	0	85.2	27.7
Junc 36	21.4	0	60.4	16.9
Junc 37	40.0	0	100.6	26.2
Junc 38	40.0	0	101.2	26.5
Junc 39	40.0	0	97.0	24.7
Junc 40	125.0	0	40.9	-36.4

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TABLE E-4.5. EPANET RESULTS FOR NETWORK NODES IN CASE 3 (CONTD.)

	Elevation	Base Demand	Head	Pressure
Node ID	ft	GPM	ft	psi
Junc 41	40.0	0	91.7	22.4
Junc 42	40.0	0	43.6	1.6
Junc 43	40.0	0	84.5	19.3
Junc 44	40.0	0	58.8	8.1
Junc 47	40.0	0	100.5	26.2
Junc 13	22.8	0	41.7	8.2
Junc 45	40.0	0	39.4	-0.3
Junc 48	21.6	0	101.5	34.6
Junc 49	40.0	0	101.3	26.6
Junc 52	40.0	0	36.9	-1.3
Resvr 15	27.6	na	27.6	0.0
Resvr 16	25.4	na	25.4	0.0
Resvr 17	27.7	na	27.7	0.0
Resvr 18	26.0	na	26.0	0.0
Resvr 19	26.2	na	26.2	0.0
Resvr 20	25.9	na	25.9	0.0
Resvr 21	25.0	na	25.0	0.0
Resvr 22	24.9	na	24.9	0.0
Resvr 23	25.0	na	25.0	0.0
Resvr 24	27.1	na	27.1	0.0
Resvr 25	25.2	na	25.2	0.0
Resvr 26	26.3	na	26.3	0.0
Resvr 27	25.2	na	25.2	0.0
Resvr 28	25.4	na	25.4	0.0
Resvr 46	24.0	na	24.0	0.0
Resvr 50	25.6	na	25.6	0.0
Resvr 51	26.8	na	26.8	0.0

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TABLE E-4.6. EPANET RESULTS FOR NETWORK LINKS IN CASE 3

	Length	Diameter	Hazen-Williams	Flow	Velocity
Link ID	ft	in.	Roughness Coefficient	GPM	fps
Pipe 1	44.5	3	130	9.4	0.4
Pipe 2	43.7	3	130	10.5	0.5
Pipe 3	44.7	3	130	9.3	0.4
Pipe 4	42.4	3	130	12.1	0.6
Pipe 5	46.1	3	130	8.3	0.4
Pipe 6	41.1	3	130	12.0	0.5
Pipe 7	514	4	130	17.5	0.5
Pipe 8	516	4	130	24.1	0.6
Pipe 9	516	4	130	8.3	0.2
Pipe 10	515	4	130	12.0	0.3
Pipe 11	1139	4	130	26.9	0.7
Pipe 12	312	4	130	34.6	0.9
Pipe 21	60.1	3	130	11.6	0.5
Pipe 22	60.4	3	130	10.5	0.5
Pipe 23	60	3	130	15.6	0.7
Pipe 24	36.1	3	130	47.4	2.2
Pipe 25	59.5	3	130	20.2	0.9
Pipe 26	55.8	3	130	43.5	2.0
Pipe 27	59.4	3	130	25.0	1.1
Pipe 28	58.7	3	130	37.5	1.7
Pipe 37	587	4	130	130.6	3.3
Pipe 38	586	4	130	110.5	2.8
Pipe 40	1522	4	130	21.8	0.6
Pipe 41	596	4	130	193.2	4.9
Pipe 46	514	4	130	94.9	2.4

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TABLE E-4.6. EPANET RESULTS FOR NETWORK LINKS IN CASE 3 (CONTD.)

Link ID	Length ft	Diameter in.	Hazen-Williams Roughness Coefficient	Flow GPM	Velocity fps
Pipe 47	74	4	130	33.4	0.9
Pipe 48	563	4	130	61.5	1.6
Pipe 13	1506	4	130	155.6	4.0
Pipe 43	55	3	130	45.1	2.1
Pipe 44	58	3	130	11.3	0.5
Pipe 45	423	4	130	11.3	0.3
Pipe 49	377	4	130	92.5	2.4
Pipe 50	181	4	130	236.7	6.0
Pipe 51	189	4	130	329.1	8.4
Pipe 52	838	4	130	47.4	1.2
Pump 14	na	na	na	9.4	0.0
Pump 15	na	na	na	10.5	0.0
Pump 16	na	na	na	9.3	0.0
Pump 17	na	na	na	12.1	0.0
Pump 18	na	na	na	8.3	0.0
Pump 19	na	na	na	12.0	0.0
Pump 29	na	na	na	11.6	0.0
Pump 30	na	na	na	10.5	0.0
Pump 31	na	na	na	15.6	0.0
Pump 32	na	na	na	47.4	0.0
Pump 33	na	na	na	20.2	0.0
Pump 34	na	na	na	43.5	0.0
Pump 35	na	na	na	25.0	0.0
Pump 36	na	na	na	37.5	0.0
Pump 20	na	na	na	11.3	0.0
Pump 42	na	na	na	45.1	0.0

Written by: V. Krishnan Date: 06/17/16 Reviewed by: M. Christman Date: 07/08/16

Client: Santee Cooper Project: Winyah Generating Station Project No.: GSC5242 Task No.: 01BT

TABLE E-4.7. FACTOR OF SAFETY AGAINST HYDRAULIC RUPTURE

Case No.	Pipe Pressure Rating (psi)	Max. Pipe Pressure (psi)	FS_{hr}
1	125	14.6	8.6
2	125	32.7	3.8
3	125	35.5	3.5

Written by:	V. Krishnan	Date:	06/17/16	Reviewed by:	M. Christman	Date:	07/08/16
Client:	Santee Cooper	Project:	Winyah Generating Station	Project No.:	GSC5242	Task No.:	01BT

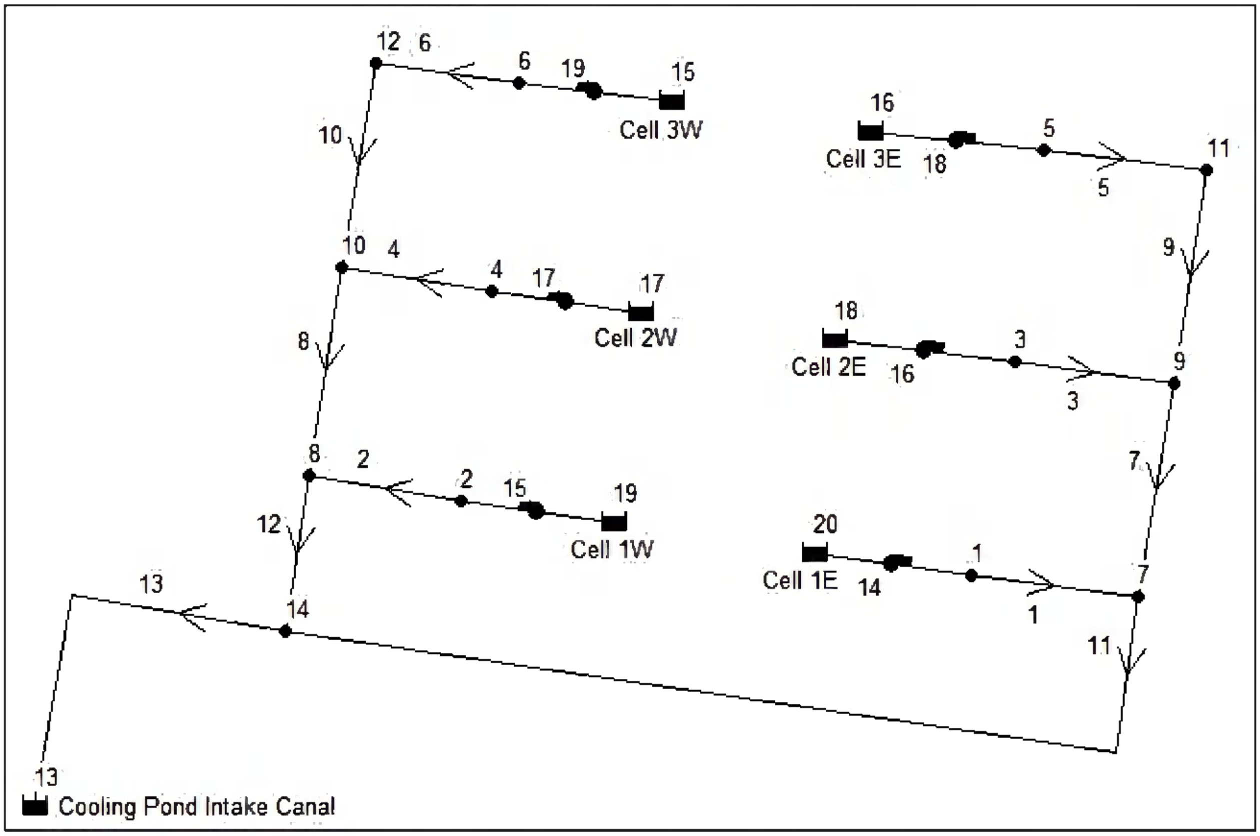


Figure E-4.1. Network Diagram for Case 1 (Node IDs, Link IDs, and Cells are shown)

Written by:	V. Krishnan	Date:	06/17/16	Reviewed by:	M. Christman	Date:	07/08/16
Client:	Santee Cooper	Project:	Winyah Generating Station	Project No.:	GSC5242	Task No.:	01BT

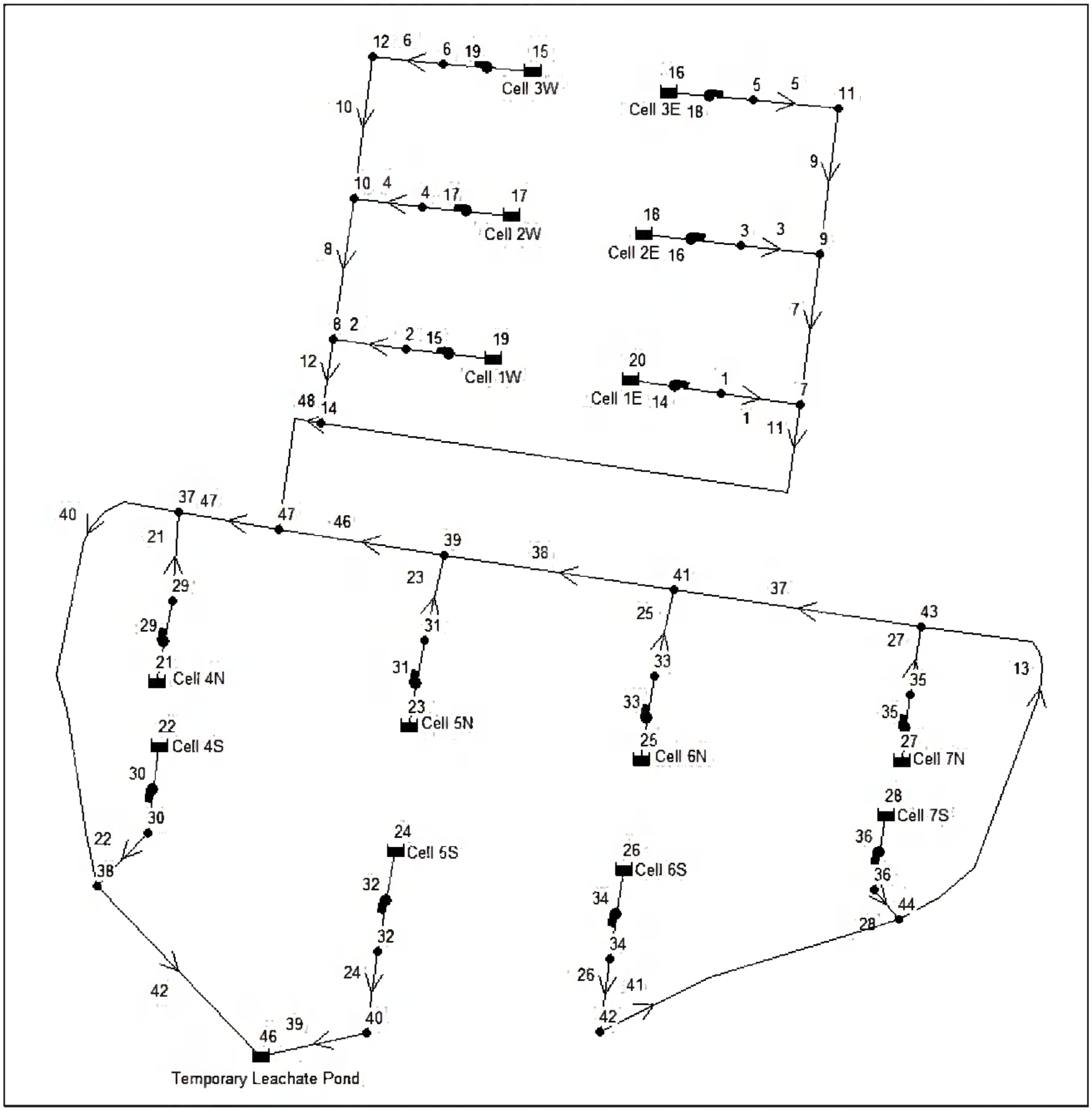


Figure E-4.2. Network Diagram for Case 2 (Node IDs, Link IDs, and Cells are shown)

Written by: **V. Krishnan** Date: **06/17/16** Reviewed by: **M. Christman** Date: **07/08/16**

Client: **Santee Cooper** Project: **Winyah Generating Station** Project No.: **GSC5242** Task No.: **01BT**

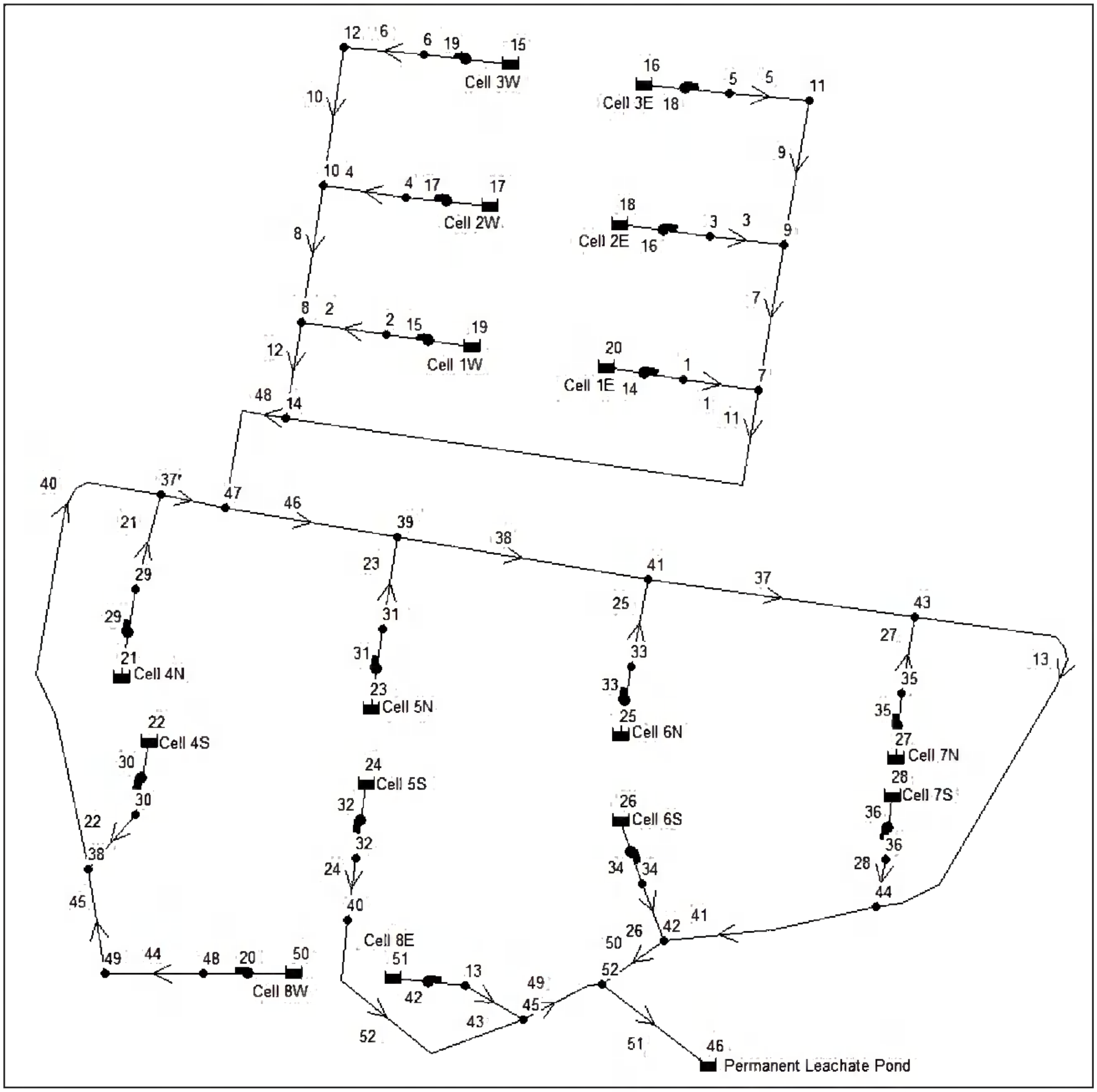
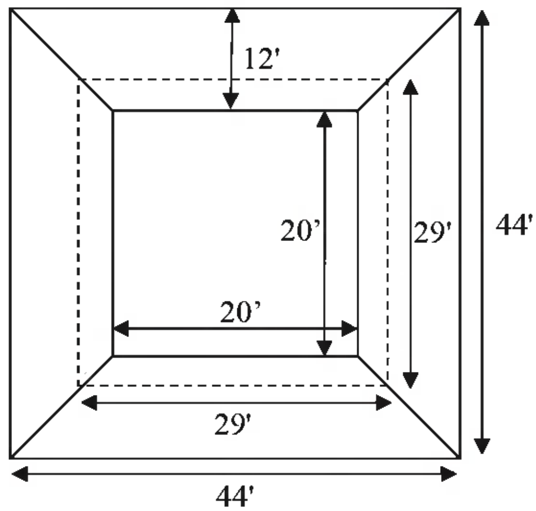
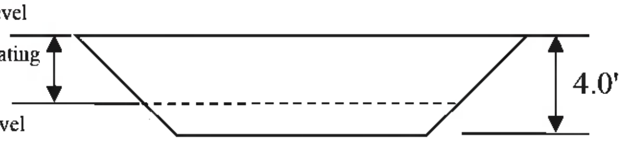


Figure E-4.3. Network Diagram for Case 3 (Node IDs, Link IDs, and Cells are shown)

Written by:	V. Krishnan	Date:	06/17/16	Reviewed by:	M. Christman	Date:	07/08/16
Client:	Santee Cooper	Project:	Winyah Generating Station	Project No.:	GSC5242	Task No.:	01BT



Plan View



Elevation View

Figure E-4.4. Proposed Leachate Collection Sump Configuration

Written by:	V. Krishnan	Date:	06/17/16	Reviewed by:	M. Christman	Date:	07/08/16
Client:	Santee Cooper	Project:	Winyah Generating Station	Project No.:	GSC5242	Task No.:	01BT

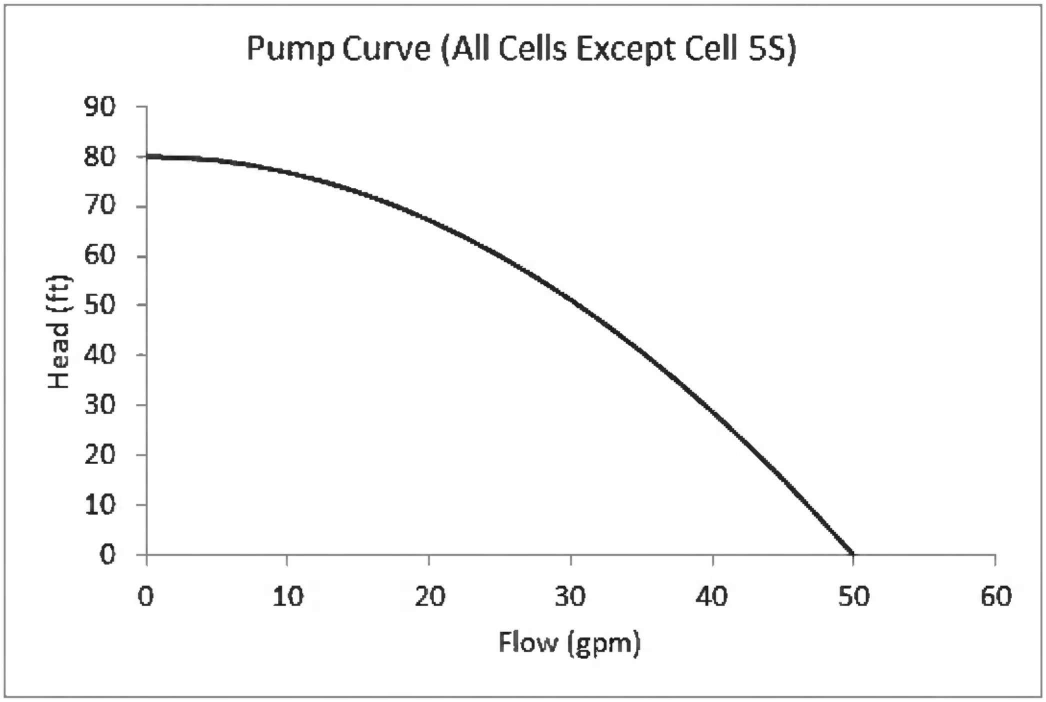


Figure E-4.5. EPANET Pump Curve for All Cells Except Subcell 5S

Written by:	V. Krishnan	Date:	06/17/16	Reviewed by:	M. Christman	Date:	07/08/16
Client:	Santee Cooper	Project:	Winyah Generating Station	Project No.:	GSC5242	Task No.:	01BT

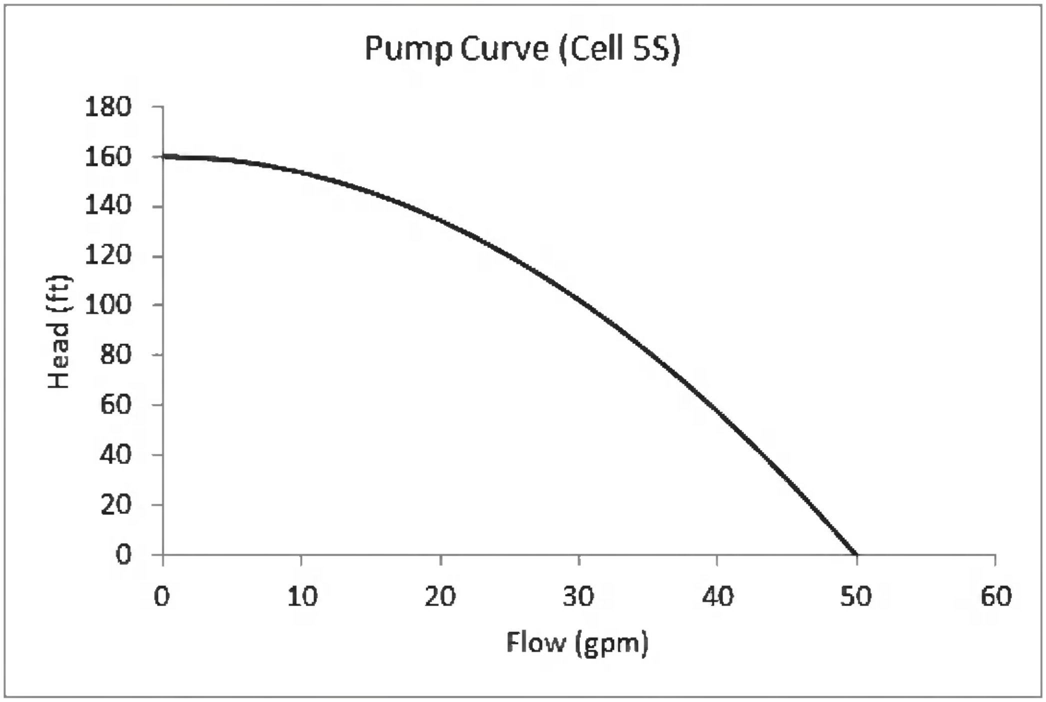


Figure E-4.6. EPANET Pump Curve for Subcell 5S

Written by:	**V. Krishnan**	Date:	**06/17/16**	Reviewed by:	**M. Christman**	Date:	**07/08/16**
Client:	**Santee Cooper**	Project:	**Winyah Generating Station**	Project No.:	**GSC5242**	Task No.:	**01BT**

APPENDIX E-4.A

EPANET COMPUTER PROGRAM OUTPUT FILES

Case 1

7/8/2016 9:43:55 AM

```
**********************************************************************
*                             E P A N E T                            *
*                     Hydraulic and Water Quality                    *
*                     Analysis for Pipe Networks                     *
*                             Version 2.0                            *
**********************************************************************
```

Input File: Pump Design_Case 1.net

Link - Node Table:

Link ID	Start Node	End Node	Length ft	Diameter in	
1	1	7	44.5	4	
2	2	8	43.7	4	
3	3	9	44.7	4	
4	4	10	42.4	4	
5	5	11	46.1	4	
6	6	12	41.1	4	
7	9	7	514	4	
8	10	8	516	4	
9	11	9	516	4	
10	12	10	515	4	
11	7	14	1139	4	
12	8	14	312	4	
13	14	13	427	4	
14	20	1	#N/A	#N/A	Pump
15	19	2	#N/A	#N/A	Pump
16	18	3	#N/A	#N/A	Pump
17	17	4	#N/A	#N/A	Pump
18	16	5	#N/A	#N/A	Pump
19	15	6	#N/A	#N/A	Pump

Energy Usage:

Pump	Usage Factor	Avg. Effic.	Kw-hr /Mgal	Avg. Kw	Peak Kw	Cost /day
14	100.00	75.00	103.62	0.26	0.26	0.00
15	100.00	75.00	67.10	0.18	0.18	0.00
16	100.00	75.00	114.31	0.28	0.28	0.00
17	100.00	75.00	73.75	0.20	0.20	0.00
18	100.00	75.00	119.57	0.29	0.29	0.00
19	100.00	75.00	77.30	0.20	0.20	0.00
					Demand Charge:	0.00
					Total Cost:	0.00

♀

Node Results at 0:00 Hrs:

Node ID	Demand GPM	Head ft	Pressure psi	Quality
1	0.00	50.65	12.45	0.00
2	0.00	42.20	8.67	0.00
3	0.00	53.24	13.60	0.00
4	0.00	45.26	9.36	0.00
5	0.00	53.95	14.10	0.00
6	0.00	46.09	9.73	0.00
7	0.00	50.58	6.32	0.00

Case 1

8	0.00	42.12	2.65	0.00	
9	0.00	53.18	7.44	0.00	
10	0.00	45.19	3.98	0.00	
11	0.00	53.89	7.75	0.00	
12	0.00	46.03	4.34	0.00	
14	0.00	38.16	0.94	0.00	
13	254.97	20.00	0.00	0.00	Reservoir
15	-43.86	27.64	0.00	0.00	Reservoir
16	-40.10	25.41	0.00	0.00	Reservoir
17	-44.16	27.65	0.00	0.00	Reservoir
18	-40.59	25.95	0.00	0.00	Reservoir
19	-44.71	26.18	0.00	0.00	Reservoir
20	-41.56	25.91	0.00	0.00	Reservoir

Link Results at 0:00 Hrs:

Link ID	Flow GPM	VelocityUnit fps	Headloss ft/Kft	Status	
1	41.56	1.06	1.48	Open	
2	44.71	1.14	1.69	Open	
3	40.59	1.04	1.41	Open	
4	44.16	1.13	1.65	Open	
5	40.10	1.02	1.38	Open	
6	43.86	1.12	1.63	Open	
7	80.69	2.06	5.05	Open	
8	88.01	2.25	5.93	Open	
9	40.10	1.02	1.38	Open	
10	43.86	1.12	1.63	Open	
11	122.24	3.12	10.90	Open	
12	132.73	3.39	12.70	Open	
13	254.97	6.51	42.54	Open	
14	41.56	0.00	-24.74	Open	Pump
15	44.71	0.00	-16.02	Open	Pump
16	40.59	0.00	-27.29	Open	Pump
17	44.16	0.00	-17.61	Open	Pump
18	40.10	0.00	-28.54	Open	Pump
19	43.86	0.00	-18.45	Open	Pump

♀

Node Results at 1:00 Hrs:

Node ID	Demand GPM	Head ft	Pressure psi	Quality	
1	0.00	50.65	12.45	0.00	
2	0.00	42.20	8.67	0.00	
3	0.00	53.24	13.60	0.00	
4	0.00	45.26	9.36	0.00	
5	0.00	53.95	14.10	0.00	
6	0.00	46.09	9.73	0.00	
7	0.00	50.58	6.32	0.00	
8	0.00	42.12	2.65	0.00	
9	0.00	53.18	7.44	0.00	
10	0.00	45.19	3.98	0.00	
11	0.00	53.89	7.75	0.00	
12	0.00	46.03	4.34	0.00	
14	0.00	38.16	0.94	0.00	
13	254.97	20.00	0.00	0.00	Reservoir
15	-43.86	27.64	0.00	0.00	Reservoir
16	-40.10	25.41	0.00	0.00	Reservoir
17	-44.16	27.65	0.00	0.00	Reservoir
18	-40.59	25.95	0.00	0.00	Reservoir
19	-44.71	26.18	0.00	0.00	Reservoir
20	-41.56	25.91	0.00	0.00	Reservoir

Case 1

Link Results at 1:00 Hrs:

Link ID	Flow GPM	VelocityUnit fps	Headloss ft/Kft	Status
1	41.56	1.06	1.48	Open
2	44.71	1.14	1.69	Open
3	40.59	1.04	1.41	Open
4	44.16	1.13	1.65	Open
5	40.10	1.02	1.38	Open
6	43.86	1.12	1.63	Open
7	80.69	2.06	5.05	Open
8	88.01	2.25	5.93	Open
9	40.10	1.02	1.38	Open
10	43.86	1.12	1.63	Open
11	122.24	3.12	10.90	Open
12	132.73	3.39	12.70	Open
13	254.97	6.51	42.54	Open
14	41.56	0.00	-24.74	Open Pump
15	44.71	0.00	-16.02	Open Pump
16	40.59	0.00	-27.29	Open Pump
17	44.16	0.00	-17.61	Open Pump
18	40.10	0.00	-28.54	Open Pump
19	43.86	0.00	-18.45	Open Pump

Case 2

7/10/2016 10:36:34 PM

```
**********************************************************************
*                           E P A N E T                              *
*                   Hydraulic and Water Quality                      *
*                   Analysis for Pipe Networks                       *
*                           Version 2.0                              *
**********************************************************************
```

Input File: Pump Design_Case 2.net

Link - Node Table:

Link ID	Start Node	End Node	Length ft	Diameter in	
1	1	7	44.5	3	
2	2	8	43.7	3	
3	3	9	44.7	3	
4	4	10	42.4	3	
5	5	11	46.1	3	
6	6	12	41.1	3	
7	9	7	514	4	
8	10	8	516	4	
9	11	9	516	4	
10	12	10	515	4	
11	7	14	1139	4	
12	8	14	312	4	
21	29	37	60.1	3	
22	30	38	60.4	3	
23	31	39	60.0	3	
24	32	40	36.1	3	
25	33	41	59.5	3	
26	34	42	55.8	3	
27	35	43	59.4	3	
28	36	44	58.7	3	
37	43	41	587	4	
38	41	39	586	4	
40	37	38	1522	4	
41	42	44	596	4	
46	39	47	514	4	
47	47	37	74	4	
48	14	47	563	4	
13	44	43	1506	4	
39	40	46	76	4	
42	38	46	218	4	
14	20	1	#N/A	#N/A	Pump
15	19	2	#N/A	#N/A	Pump
16	18	3	#N/A	#N/A	Pump
17	17	4	#N/A	#N/A	Pump
18	16	5	#N/A	#N/A	Pump
19	15	6	#N/A	#N/A	Pump
29	21	29	#N/A	#N/A	Pump

Link - Node Table: (continued)

Link ID	Start Node	End Node	Length ft	Diameter in	
30	22	30	#N/A	#N/A	Pump
31	23	31	#N/A	#N/A	Pump
32	24	32	#N/A	#N/A	Pump
33	25	33	#N/A	#N/A	Pump
34	26	34	#N/A	#N/A	Pump

Case 2

35	27	35	#N/A	#N/A	Pump
36	28	36	#N/A	#N/A	Pump

Energy Usage:

Pump	Usage Factor	Avg. Effic.	Kw-hr /Mgal	Avg. Kw	Peak Kw	Cost /day
14	100.00	75.00	294.25	0.31	0.31	0.00
15	100.00	75.00	286.69	0.33	0.33	0.00
16	100.00	75.00	296.15	0.30	0.30	0.00
17	100.00	75.00	283.49	0.33	0.33	0.00
18	100.00	75.00	298.89	0.29	0.29	0.00
19	100.00	75.00	284.28	0.33	0.33	0.00
29	100.00	75.00	259.99	0.37	0.37	0.00
30	100.00	75.00	57.30	0.16	0.16	0.00
31	100.00	75.00	280.63	0.34	0.34	0.00
32	100.00	75.00	7.38	0.02	0.02	0.00
33	100.00	75.00	289.08	0.32	0.32	0.00
34	100.00	75.00	296.18	0.30	0.30	0.00
35	100.00	75.00	294.03	0.31	0.31	0.00
36	100.00	75.00	299.14	0.29	0.29	0.00
				Demand Charge:		0.00
				Total Cost:		0.00

Node Results at 0:00 Hrs:

Node ID	Demand GPM	Head ft	Pressure psi	Quality
1	0.00	96.16	32.17	0.00
2	0.00	94.62	31.39	0.00
3	0.00	96.65	32.41	0.00
4	0.00	95.33	31.06	0.00
5	0.00	96.76	32.65	0.00
6	0.00	95.50	31.14	0.00
7	0.00	95.81	25.92	0.00
8	0.00	94.21	25.22	0.00
9	0.00	96.32	26.14	0.00
10	0.00	94.89	25.52	0.00
11	0.00	96.46	26.20	0.00

♀

Node Results at 0:00 Hrs: (continued)

Node ID	Demand GPM	Head ft	Pressure psi	Quality
12	0.00	95.08	25.60	0.00
14	0.00	93.35	24.85	0.00
29	0.00	87.07	28.63	0.00
30	0.00	38.58	7.66	0.00
31	0.00	92.02	30.76	0.00
32	0.00	28.83	2.50	0.00
33	0.00	94.19	31.64	0.00
34	0.00	97.05	32.37	0.00
35	0.00	95.41	32.15	0.00
36	0.00	96.85	32.68	0.00
37	0.00	86.40	20.11	0.00
38	0.00	36.16	-1.66	0.00
39	0.00	91.54	22.33	0.00
40	0.00	26.16	-3.62	0.00
41	0.00	93.78	23.30	0.00
42	0.00	96.70	24.57	0.00
43	0.00	95.05	23.85	0.00

Case 2

44	0.00	96.53	24.50	0.00	
47	0.00	88.39	20.97	0.00	
15	-19.47	27.64	0.00	0.00	Reservoir
16	-16.44	25.41	0.00	0.00	Reservoir
17	-19.62	27.65	0.00	0.00	Reservoir
18	-17.05	25.95	0.00	0.00	Reservoir
19	-19.01	26.18	0.00	0.00	Reservoir
20	-17.46	25.91	0.00	0.00	Reservoir
21	-23.67	25.00	0.00	0.00	Reservoir
22	-45.53	24.90	0.00	0.00	Reservoir
23	-20.16	25.03	0.00	0.00	Reservoir
24	-49.72	27.07	0.00	0.00	Reservoir
25	-18.53	25.18	0.00	0.00	Reservoir
26	-17.04	26.34	0.00	0.00	Reservoir
27	-17.51	25.22	0.00	0.00	Reservoir
28	-16.38	25.44	0.00	0.00	Reservoir
46	317.59	26.00	0.00	0.00	Reservoir

Link Results at 0:00 Hrs:

Link ID	Flow GPM	VelocityUnit fps	Headloss ft/Kft	Status
1	17.46	0.79	7.78	Open
2	19.01	0.86	9.34	Open
3	17.05	0.77	7.39	Open
4	19.62	0.89	10.21	Open
5	16.44	0.75	6.70	Open
6	19.47	0.88	10.33	Open
7	33.49	0.85	0.99	Open

Link Results at 0:00 Hrs: (continued)

Link ID	Flow GPM	VelocityUnit fps	Headloss ft/Kft	Status
8	39.10	1.00	1.32	Open
9	16.44	0.42	0.27	Open
10	19.47	0.50	0.36	Open
11	50.95	1.30	2.16	Open
12	58.11	1.48	2.75	Open
21	23.67	1.07	11.06	Open
22	45.53	2.07	40.03	Open
23	20.16	0.92	8.07	Open
24	49.72	2.26	74.09	Open
25	18.53	0.84	6.88	Open
26	17.04	0.77	6.15	Open
27	17.51	0.79	6.16	Open
28	16.38	0.74	5.46	Open
37	50.93	1.30	2.15	Open
38	69.46	1.77	3.83	Open
40	222.34	5.68	33.01	Open
41	17.04	0.44	0.28	Open
46	89.62	2.29	6.14	Open
47	198.67	5.07	26.80	Open
48	109.05	2.78	8.82	Open
13	33.42	0.85	0.99	Open
39	49.72	1.27	2.06	Open
42	267.87	6.84	46.61	Open
14	17.46	0.00	-70.25	Open Pump
15	19.01	0.00	-68.44	Open Pump
16	17.05	0.00	-70.70	Open Pump
17	19.62	0.00	-67.68	Open Pump
18	16.44	0.00	-71.35	Open Pump
19	19.47	0.00	-67.86	Open Pump

Case 2

29	23.67	0.00	-62.07	Open Pump
30	45.53	0.00	-13.68	Open Pump
31	20.16	0.00	-66.99	Open Pump
32	49.72	0.00	-1.76	Open Pump
33	18.53	0.00	-69.01	Open Pump
34	17.04	0.00	-70.71	Open Pump
35	17.51	0.00	-70.19	Open Pump
36	16.38	0.00	-71.41	Open Pump

♀

Node Results at 1:00 Hrs:

Node ID	Demand GPM	Head ft	Pressure psi	Quality	
1	0.00	96.16	32.17	0.00	
2	0.00	94.62	31.39	0.00	
3	0.00	96.65	32.41	0.00	
4	0.00	95.33	31.06	0.00	
5	0.00	96.76	32.65	0.00	
6	0.00	95.50	31.14	0.00	
7	0.00	95.81	25.92	0.00	
8	0.00	94.21	25.22	0.00	
9	0.00	96.32	26.14	0.00	
10	0.00	94.89	25.52	0.00	
11	0.00	96.46	26.20	0.00	
12	0.00	95.08	25.60	0.00	
14	0.00	93.35	24.85	0.00	
29	0.00	87.07	28.63	0.00	
30	0.00	38.58	7.66	0.00	
31	0.00	92.02	30.76	0.00	
32	0.00	28.83	2.50	0.00	
33	0.00	94.19	31.64	0.00	
34	0.00	97.05	32.37	0.00	
35	0.00	95.41	32.15	0.00	
36	0.00	96.85	32.68	0.00	
37	0.00	86.40	20.11	0.00	
38	0.00	36.16	-1.66	0.00	
39	0.00	91.54	22.33	0.00	
40	0.00	26.16	-3.62	0.00	
41	0.00	93.78	23.30	0.00	
42	0.00	96.70	24.57	0.00	
43	0.00	95.05	23.85	0.00	
44	0.00	96.53	24.50	0.00	
47	0.00	88.39	20.97	0.00	
15	-19.47	27.64	0.00	0.00	Reservoir
16	-16.44	25.41	0.00	0.00	Reservoir
17	-19.62	27.65	0.00	0.00	Reservoir
18	-17.05	25.95	0.00	0.00	Reservoir
19	-19.01	26.18	0.00	0.00	Reservoir
20	-17.46	25.91	0.00	0.00	Reservoir
21	-23.67	25.00	0.00	0.00	Reservoir
22	-45.53	24.90	0.00	0.00	Reservoir
23	-20.16	25.03	0.00	0.00	Reservoir
24	-49.72	27.07	0.00	0.00	Reservoir
25	-18.53	25.18	0.00	0.00	Reservoir
26	-17.04	26.34	0.00	0.00	Reservoir
27	-17.51	25.22	0.00	0.00	Reservoir
28	-16.38	25.44	0.00	0.00	Reservoir
46	317.59	26.00	0.00	0.00	Reservoir

♀

Link Results at 1:00 Hrs:

Case 2

Link ID	Flow GPM	Velocity fps	Unit Headloss ft/Kft	Status
1	17.46	0.79	7.78	Open
2	19.01	0.86	9.34	Open
3	17.05	0.77	7.39	Open
4	19.62	0.89	10.21	Open
5	16.44	0.75	6.70	Open
6	19.47	0.88	10.33	Open
7	33.49	0.85	0.99	Open
8	39.10	1.00	1.32	Open
9	16.44	0.42	0.27	Open
10	19.47	0.50	0.36	Open
11	50.95	1.30	2.16	Open
12	58.11	1.48	2.75	Open
21	23.67	1.07	11.06	Open
22	45.53	2.07	40.03	Open
23	20.16	0.92	8.07	Open
24	49.72	2.26	74.09	Open
25	18.53	0.84	6.88	Open
26	17.04	0.77	6.15	Open
27	17.51	0.79	6.16	Open
28	16.38	0.74	5.46	Open
37	50.93	1.30	2.15	Open
38	69.46	1.77	3.83	Open
40	222.34	5.68	33.01	Open
41	17.04	0.44	0.28	Open
46	89.62	2.29	6.14	Open
47	198.67	5.07	26.80	Open
48	109.05	2.78	8.82	Open
13	33.42	0.85	0.99	Open
39	49.72	1.27	2.06	Open
42	267.87	6.84	46.61	Open
14	17.46	0.00	-70.25	Open Pump
15	19.01	0.00	-68.44	Open Pump
16	17.05	0.00	-70.70	Open Pump
17	19.62	0.00	-67.68	Open Pump
18	16.44	0.00	-71.35	Open Pump
19	19.47	0.00	-67.86	Open Pump
29	23.67	0.00	-62.07	Open Pump
30	45.53	0.00	-13.68	Open Pump
31	20.16	0.00	-66.99	Open Pump
32	49.72	0.00	-1.76	Open Pump
33	18.53	0.00	-69.01	Open Pump
34	17.04	0.00	-70.71	Open Pump
35	17.51	0.00	-70.19	Open Pump
36	16.38	0.00	-71.41	Open Pump

Case 3

7/10/2016 8:33:15 PM

```
**********************************************************************
*                             E P A N E T                            *
*                     Hydraulic and Water Quality                    *
*                     Analysis for Pipe Networks                     *
*                            Version 2.0                             *
**********************************************************************
```

Input File: Pump Design_Case 3.net

Link - Node Table:

Link ID	Start Node	End Node	Length ft	Diameter in	
1	1	7	44.5	3	
2	2	8	43.7	3	
3	3	9	44.7	3	
4	4	10	42.4	3	
5	5	11	46.1	3	
6	6	12	41.1	3	
7	9	7	514	4	
8	10	8	516	4	
9	11	9	516	4	
10	12	10	515	4	
11	7	14	1139	4	
12	8	14	312	4	
21	29	37	60.1	3	
22	30	38	60.4	3	
23	31	39	60.0	3	
24	32	40	36.1	3	
25	33	41	59.5	3	
26	34	42	55.8	3	
27	35	43	59.4	3	
28	36	44	58.7	3	
37	41	43	587	4	
38	39	41	586	4	
40	38	37	1522	4	
41	44	42	596	4	
46	47	39	514	4	
47	37	47	74	4	
48	14	47	563	4	
13	43	44	1506	4	
43	13	45	55	3	
44	48	49	58	3	
45	49	38	423	4	
49	45	52	377	4	
50	42	52	181	4	
51	52	46	189	4	
52	40	45	838	4	
14	20	1	#N/A	#N/A	Pump
15	19	2	#N/A	#N/A	Pump

Link - Node Table: (continued)

Link ID	Start Node	End Node	Length ft	Diameter in	
16	18	3	#N/A	#N/A	Pump
17	17	4	#N/A	#N/A	Pump
18	16	5	#N/A	#N/A	Pump
19	15	6	#N/A	#N/A	Pump
29	21	29	#N/A	#N/A	Pump

Case 3

30	22	30	#N/A	#N/A	Pump
31	23	31	#N/A	#N/A	Pump
32	24	32	#N/A	#N/A	Pump
33	25	33	#N/A	#N/A	Pump
34	26	34	#N/A	#N/A	Pump
35	27	35	#N/A	#N/A	Pump
36	28	36	#N/A	#N/A	Pump
20	50	48	#N/A	#N/A	Pump
42	51	13	#N/A	#N/A	Pump

Energy Usage:

Pump	Usage Factor	Avg. Effic.	Kw-hr /Mgal	Avg. Kw	Peak Kw	Cost /day
14	100.00	75.00	323.18	0.18	0.18	0.00
15	100.00	75.00	320.36	0.20	0.20	0.00
16	100.00	75.00	323.63	0.18	0.18	0.00
17	100.00	75.00	315.53	0.23	0.23	0.00
18	100.00	75.00	325.97	0.16	0.16	0.00
19	100.00	75.00	315.87	0.23	0.23	0.00
29	100.00	75.00	317.18	0.22	0.22	0.00
30	100.00	75.00	320.34	0.20	0.20	0.00
31	100.00	75.00	302.60	0.28	0.28	0.00
32	100.00	75.00	68.24	0.19	0.19	0.00
33	100.00	75.00	280.60	0.34	0.34	0.00
34	100.00	75.00	81.46	0.21	0.21	0.00
35	100.00	75.00	251.20	0.38	0.38	0.00
36	100.00	75.00	146.48	0.33	0.33	0.00
20	100.00	75.00	317.93	0.22	0.22	0.00
42	100.00	75.00	62.57	0.17	0.17	0.00
				Demand Charge:		0.00
				Total Cost:		0.00

♀

Node Results at 0:00 Hrs:

Node ID	Demand GPM	Head ft	Pressure psi	Quality
1	0.00	103.06	35.16	0.00
2	0.00	102.66	34.87	0.00
3	0.00	103.21	35.25	0.00
4	0.00	102.98	34.37	0.00
5	0.00	103.23	35.45	0.00
6	0.00	103.05	34.41	0.00
7	0.00	102.96	29.01	0.00
8	0.00	102.53	28.83	0.00
9	0.00	103.11	29.08	0.00
10	0.00	102.81	28.95	0.00
11	0.00	103.15	29.10	0.00
12	0.00	102.89	28.98	0.00
14	0.00	102.20	28.69	0.00
29	0.00	100.72	34.54	0.00
30	0.00	101.37	34.87	0.00
31	0.00	97.27	33.03	0.00
32	0.00	43.36	8.79	0.00
33	0.00	92.17	30.76	0.00
34	0.00	45.79	10.16	0.00
35	0.00	85.19	27.72	0.00
36	0.00	60.41	16.89	0.00
37	0.00	100.56	26.24	0.00
38	0.00	101.24	26.53	0.00
39	0.00	96.98	24.69	0.00

Case 3

40	0.00	40.93	-36.43	0.00	
41	0.00	91.68	22.39	0.00	
42	0.00	43.61	1.56	0.00	
43	0.00	84.45	19.26	0.00	
44	0.00	58.77	8.13	0.00	
47	0.00	100.48	26.21	0.00	
13	0.00	41.69	8.21	0.00	
45	0.00	39.35	-0.28	0.00	
48	0.00	101.45	34.62	0.00	
49	0.00	101.30	26.56	0.00	
52	0.00	36.90	-1.34	0.00	
15	-11.98	27.64	0.00	0.00	Reservoir
16	-8.26	25.41	0.00	0.00	Reservoir
17	-12.09	27.65	0.00	0.00	Reservoir
18	-9.25	25.95	0.00	0.00	Reservoir
19	-10.49	26.18	0.00	0.00	Reservoir
20	-9.44	25.91	0.00	0.00	Reservoir
21	-11.57	25.00	0.00	0.00	Reservoir
22	-10.50	24.90	0.00	0.00	Reservoir
23	-15.57	25.03	0.00	0.00	Reservoir
24	-47.39	27.07	0.00	0.00	Reservoir
25	-20.17	25.18	0.00	0.00	Reservoir
26	-43.50	26.34	0.00	0.00	Reservoir

Node Results at 0:00 Hrs: (continued)

Node ID	Demand GPM	Head ft	Pressure psi	Quality	
27	-25.02	25.22	0.00	0.00	Reservoir
28	-37.51	25.44	0.00	0.00	Reservoir
46	329.13	24.00	0.00	0.00	Reservoir
50	-11.32	25.55	0.00	0.00	Reservoir
51	-45.09	26.75	0.00	0.00	Reservoir

Link Results at 0:00 Hrs:

Link ID	Flow GPM	Velocity fps	Unit Headloss ft/Kft	Status
1	9.44	0.43	2.31	Open
2	10.49	0.48	2.89	Open
3	9.25	0.42	2.21	Open
4	12.09	0.55	3.92	Open
5	8.26	0.37	1.72	Open
6	11.98	0.54	3.95	Open
7	17.51	0.45	0.30	Open
8	24.07	0.61	0.54	Open
9	8.26	0.21	0.07	Open
10	11.98	0.31	0.15	Open
11	26.94	0.69	0.66	Open
12	34.56	0.88	1.05	Open
21	11.57	0.52	2.70	Open
22	10.50	0.48	2.22	Open
23	15.57	0.71	4.85	Open
24	47.39	2.15	67.34	Open
25	20.17	0.92	8.13	Open
26	43.50	1.97	39.07	Open
27	25.02	1.14	12.46	Open
28	37.51	1.70	27.97	Open
37	130.62	3.33	12.33	Open
38	110.46	2.82	9.04	Open
40	21.82	0.56	0.45	Open
41	193.16	4.93	25.44	Open
46	94.89	2.42	6.82	Open

Case 3

47	33.38	0.85	0.99	Open
48	61.50	1.57	3.05	Open
13	155.64	3.97	17.05	Open
43	45.09	2.05	42.45	Open
44	11.32	0.51	2.66	Open
45	11.32	0.29	0.13	Open
49	92.48	2.36	6.50	Open
50	236.66	6.04	37.05	Open
51	329.13	8.40	68.26	Open
52	47.39	1.21	1.88	Open
14	9.44	0.00	-77.15	Open Pump

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Link Results at 0:00 Hrs: (continued)

Link ID	Flow GPM	VelocityUnit fps	Headloss ft/Kft	Status
15	10.49	0.00	-76.48	Open Pump
16	9.25	0.00	-77.26	Open Pump
17	12.09	0.00	-75.33	Open Pump
18	8.26	0.00	-77.82	Open Pump
19	11.98	0.00	-75.41	Open Pump
29	11.57	0.00	-75.72	Open Pump
30	10.50	0.00	-76.47	Open Pump
31	15.57	0.00	-72.24	Open Pump
32	47.39	0.00	-16.29	Open Pump
33	20.17	0.00	-66.99	Open Pump
34	43.50	0.00	-19.45	Open Pump
35	25.02	0.00	-59.97	Open Pump
36	37.51	0.00	-34.97	Open Pump
20	11.32	0.00	-75.90	Open Pump
42	45.09	0.00	-14.94	Open Pump

Node Results at 1:00 Hrs:

Node ID	Demand GPM	Head ft	Pressure psi	Quality
1	0.00	103.06	35.16	0.00
2	0.00	102.66	34.87	0.00
3	0.00	103.21	35.25	0.00
4	0.00	102.98	34.37	0.00
5	0.00	103.23	35.45	0.00
6	0.00	103.05	34.41	0.00
7	0.00	102.96	29.01	0.00
8	0.00	102.53	28.83	0.00
9	0.00	103.11	29.08	0.00
10	0.00	102.81	28.95	0.00
11	0.00	103.15	29.10	0.00
12	0.00	102.89	28.98	0.00
14	0.00	102.20	28.69	0.00
29	0.00	100.72	34.54	0.00
30	0.00	101.37	34.87	0.00
31	0.00	97.27	33.03	0.00
32	0.00	43.36	8.79	0.00
33	0.00	92.17	30.76	0.00
34	0.00	45.79	10.16	0.00
35	0.00	85.19	27.72	0.00
36	0.00	60.41	16.89	0.00
37	0.00	100.56	26.24	0.00
38	0.00	101.24	26.53	0.00
39	0.00	96.98	24.69	0.00
40	0.00	40.93	-36.43	0.00
41	0.00	91.68	22.39	0.00

Case 3

Node Results at 1:00 Hrs: (continued)

Node ID	Demand GPM	Head ft	Pressure psi	Quality
42	0.00	43.61	1.56	0.00
43	0.00	84.45	19.26	0.00
44	0.00	58.77	8.13	0.00
47	0.00	100.48	26.21	0.00
13	0.00	41.69	8.21	0.00
45	0.00	39.35	-0.28	0.00
48	0.00	101.45	34.62	0.00
49	0.00	101.30	26.56	0.00
52	0.00	36.90	-1.34	0.00
15	-11.98	27.64	0.00	0.00 Reservoir
16	-8.26	25.41	0.00	0.00 Reservoir
17	-12.09	27.65	0.00	0.00 Reservoir
18	-9.25	25.95	0.00	0.00 Reservoir
19	-10.49	26.18	0.00	0.00 Reservoir
20	-9.44	25.91	0.00	0.00 Reservoir
21	-11.57	25.00	0.00	0.00 Reservoir
22	-10.50	24.90	0.00	0.00 Reservoir
23	-15.57	25.03	0.00	0.00 Reservoir
24	-47.39	27.07	0.00	0.00 Reservoir
25	-20.17	25.18	0.00	0.00 Reservoir
26	-43.50	26.34	0.00	0.00 Reservoir
27	-25.02	25.22	0.00	0.00 Reservoir
28	-37.51	25.44	0.00	0.00 Reservoir
46	329.13	24.00	0.00	0.00 Reservoir
50	-11.32	25.55	0.00	0.00 Reservoir
51	-45.09	26.75	0.00	0.00 Reservoir

Link Results at 1:00 Hrs:

Link ID	Flow GPM	VelocityUnit fps	Headloss ft/Kft	Status
1	9.44	0.43	2.31	Open
2	10.49	0.48	2.89	Open
3	9.25	0.42	2.21	Open
4	12.09	0.55	3.92	Open
5	8.26	0.37	1.72	Open
6	11.98	0.54	3.95	Open
7	17.51	0.45	0.30	Open
8	24.07	0.61	0.54	Open
9	8.26	0.21	0.07	Open
10	11.98	0.31	0.15	Open
11	26.94	0.69	0.66	Open
12	34.56	0.88	1.05	Open
21	11.57	0.52	2.70	Open
22	10.50	0.48	2.22	Open
23	15.57	0.71	4.85	Open

Link Results at 1:00 Hrs: (continued)

Link ID	Flow GPM	VelocityUnit fps	Headloss ft/Kft	Status
24	47.39	2.15	67.34	Open
25	20.17	0.92	8.13	Open
26	43.50	1.97	39.07	Open
27	25.02	1.14	12.46	Open
28	37.51	1.70	27.97	Open

Case 3

37	130.62	3.33	12.33	Open
38	110.46	2.82	9.04	Open
40	21.82	0.56	0.45	Open
41	193.16	4.93	25.44	Open
46	94.89	2.42	6.82	Open
47	33.38	0.85	0.99	Open
48	61.50	1.57	3.05	Open
13	155.64	3.97	17.05	Open
43	45.09	2.05	42.45	Open
44	11.32	0.51	2.66	Open
45	11.32	0.29	0.13	Open
49	92.48	2.36	6.50	Open
50	236.66	6.04	37.05	Open
51	329.13	8.40	68.26	Open
52	47.39	1.21	1.88	Open
14	9.44	0.00	-77.15	Open Pump
15	10.49	0.00	-76.48	Open Pump
16	9.25	0.00	-77.26	Open Pump
17	12.09	0.00	-75.33	Open Pump
18	8.26	0.00	-77.82	Open Pump
19	11.98	0.00	-75.41	Open Pump
29	11.57	0.00	-75.72	Open Pump
30	10.50	0.00	-76.47	Open Pump
31	15.57	0.00	-72.24	Open Pump
32	47.39	0.00	-16.29	Open Pump
33	20.17	0.00	-66.99	Open Pump
34	43.50	0.00	-19.45	Open Pump
35	25.02	0.00	-59.97	Open Pump
36	37.51	0.00	-34.97	Open Pump
20	11.32	0.00	-75.90	Open Pump
42	45.09	0.00	-14.94	Open Pump

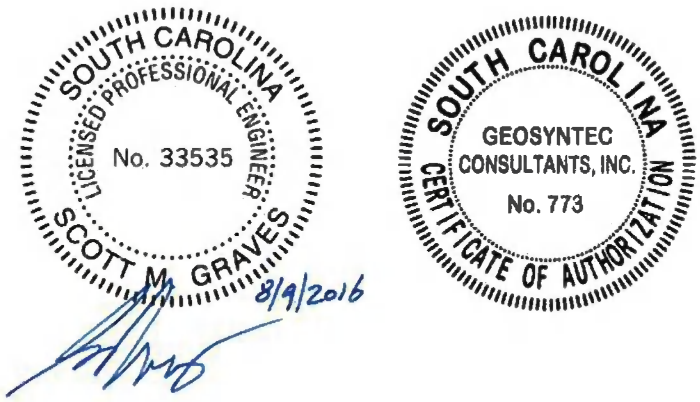
APPENDIX E-5

ACTIVE AREA RUNOFF CONTROL AND LEACHATE/CONTACT WATER STORAGE POND DESIGN

Written by:	V. Krishnan	Date:	06/29/16	Reviewed by:	M. Christman	Date:	07/08/16
Client:	Santee Cooper	Project:	Winyah Generating Station	Project No.:	GSC5242	Task No.:	01BT

APPENDIX E-5

ACTIVE AREA RUNOFF CONTROL AND LEACHATE/CONTACT WATER STORAGE POND DESIGN



SEALED FOR
CALCULATION PAGES E-5-1
THROUGH E-5-61

PURPOSE

The purpose of this calculation package is to evaluate the active area runoff control and leachate/contact water storage pond design for the proposed Class Three Landfill (composed of Landfill Area 1 and Landfill Area 2) at the Winyah Generating Station (WGS). The calculations are used to:

- estimate contact water generation due to the 25-yr, 24-hr and 100-yr, 24-hr rainfall events;
- design the contact water decant structures used to convey contact water from the active areas; and
- design the leachate/contact water storage ponds.

METHOD OF ANALYSIS

Contact water, or stormwater runoff that has been in contact with exposed waste in the active cells, is proposed to be managed with a decant structure. The decant structure will consist of a perforated vertical concrete riser pipe and will be surrounded by an attenuation basin. The attenuation basin will be a depressed area around the decant structure intended to temporarily

Written by:	V. Krishnan	Date:	06/29/16	Reviewed by:	M. Christman	Date:	07/08/16
Client:	Santee Cooper	Project:	Winyah Generating Station	Project No.:	GSC5242	Task No.:	01BT

detain the contact water. The entire active area, including the attenuation basin, will be graded to drain towards the decant structure.

The decant structure will convey contact water through a horizontal high-density polyethylene (HDPE) pipe to the Cooling Pond Intake Canal, the Temporary Leachate/Contact Water Storage Pond, or the Permanent Leachate/Contact Water Storage Pond. The Cooling Pond Intake Canal is proposed to store and treat contact water from Landfill Area 1. The Temporary Leachate/Contact Water Storage Pond (Temporary Pond) is proposed to be operational during the initial phases of Landfill Area 2. The Permanent Leachate/Contact Water Storage Pond (Permanent Pond) is proposed to be operational during the latter phases of Landfill Area 2 and post-closure.

The contact water drainage area, attenuation basin, decant structure, and leachate storage ponds were modeled using HydroCAD Stormwater Modeling Software Version 10 (HydroCAD Software Solutions, LLC, 2011). The attenuation basin and the decant structure were sized using the model with the objective of draining contact water generated from the 25-year, 24-hour rainfall event within approximately 3 days. In addition, the model was used to evaluate the decant structure and water levels in the active area considering runoff from a 100-year, 24-hour storm event.

The storage capacities of the Temporary and Permanent Ponds were evaluated in HydroCAD assuming that the ponds store the peak monthly average volume of leachate, contact water, and direct rainfall at the beginning of the model simulation run, and receive additional flow during the simulation due to direct runoff and contact water generated from the design rainfall event. Values for peak monthly average leachate, contact water, and rainfall were obtained using the Hydrologic Evaluation of Landfill Performance (*HELP*) model (see Appendix E-1).

DESIGN PARAMETERS

<u>Rainfall Event</u>

The rainfall depth corresponding to the 25-year, 24-hour storm and 100-year, 24-hour storm were selected based on information from the NOAA Atlas 14 Point Precipitation Frequency Estimates (NOAA Hydrometeorological Design Studies Center, retrieved 06-May-2016). The 25-year, 24-hour rainfall depth is 8.3 in., and the 100-year, 24-hour rainfall depth is 10.9 in., as shown in Table E-5.1. The rainfall intensity used in the analysis was the SCS Type III distribution.

Contact Water Catchment Area

The largest area of exposed waste is expected to occur during the initial stages of waste placement in Cell 4 and Cell 5 of Landfill Area 2, and encompasses an area of approximately 31 acres. As the landfill is filled above grade, cover soil will be placed on the fill sideslopes and the active area will decrease. For the purposes of design, the largest active area (31 acres) is used in this calculation package. The surface condition of the catchment area was assumed to be bare (without vegetation) and graded to drain towards the decant structure. The runoff curve number of the surface material was assumed to be 97 based on *HELP* model output files (see Appendix E-1, Case A). The time of concentration for contact water to drain from the catchment area to the attenuation basin of the decant structure was assumed to be 10 minutes.

Contact Water Attenuation Basin and Outlet Structures

The attenuation basin is a depressed area surrounding the decant structure that attenuates the contact water inflow by temporarily detaining it while it flows into the decant structure. The attenuation basin is proposed to have a depth of 4 ft and sideslopes graded at approximately at 5H:1V. The decant structure will be extended vertically upward as the landfill is filled. Since the active area and corresponding contact water generation will decrease as the landfill is filled, the size of the attenuation basin required to manage contact water also decreases. For the purposes of this calculation package, the attenuation basin is sized assuming the largest active area and the corresponding approximate waste elevations. For calculation purposes, the invert elevation of the attenuation basin was set at a hypothetical elevation of 51 ft-MSL, which would result in a crest elevation of 55 ft-MSL (i.e., 4-ft depth). A 2-ft high containment berm will be maintained around the active area, which would be at elevation 57 ft-MSL for this hypothetical case. Note that these elevations are not specifications, and the actual decant structure elevation will vary as development occurs, and it is raised during filling of additional lifts of waste. The elevations are intended to be generic – used to set up and run the model for sizing and calculation of flows The HydroCAD model was used to calculate the required attenuation basin volume and corresponding area so that the 25-yr, 24-hr rainfall event does not overtop the decant structure.

The decant structure consists of a vertical 36-in. diameter perforated reinforced concrete pipe. The pipe is proposed to be installed in 4 ft long segments. The pipe perforations are 2-in. in diameter, spaced at 30-degree intervals around the pipe (i.e., 12 perforations per row), and staggered vertically at 6-in. spacing for a total of 8 rows. The decant structure drains into an 18-in. diameter HDPE pipe, having a longitudinal slope of 0.5 percent and approximately 1,300 ft long, which discharges to the Intake Canal, Temporary Pond, or Permanent Pond.

Leachate and Contact Water Storage Ponds

The Temporary Pond will be located in the future Subcell 8W footprint. The pond has a proposed depth of approximately 14 ft (i.e., approximately between 27 ft-MSL and 41 ft-MSL) and a proposed storage capacity of approximately 2.56 million ft^3. The plan area of the pond is approximately 6.2 acres. Figure E-5.1 shows the profile of cumulative storage volume of the pond with elevation.

The Permanent Pond will be located west of and adjacent to Landfill Area 2. The pond has a proposed depth of approximately 16 ft (i.e., approximately between 24 ft-MSL and 40 ft-MSL and a proposed storage capacity of approximately 2.16 million ft^3. The plan area of the pond is approximately 4.1 acres. Figure E-5.2 shows the profile of cumulative storage volume of the pond with elevation.

The initial water surface elevation in the leachate and contact water storage ponds is assumed to be the elevation corresponding to volume occupied by the peak monthly average volume of leachate, contact water, and direct rainfall calculated using the HELP model (see Appendix E-1). The initial volume was calculated to be 552,000 ft^3 for the Temporary Pond and 496,300 ft^3 for the Permanent Pond. Based on the stage-storage profile of the ponds, the elevation corresponding to this storage volume is 32.6 ft-MSL for the Temporary Pond and 28.9 ft-MSL for the Permanent Pond, as shown in Figure E-5.1 and Figure E-5.2.

The Temporary Pond and Permanent Pond are proposed to be equipped with a submersible pump to convey leachate and contact water to a separate on-site facility for additional treatment prior to discharge. A pump with a 300 gpm flow rate at a head of 20 ft was assumed in the HydroCAD model. For the Temporary Pond, the pump-on elevation was assumed at 33 ft-MSL and the pump-off elevation was assumed at 32 ft-MSL. For the Permanent Pond, the pump-on elevation was assumed at 29 ft-MSL and the pump-off elevation was assumed at 28 ft-MSL.

RESULTS

Attenuation Basin and Decant Structure

Based on the HydroCAD model output, the calculated peak water surface elevation in the contact water attenuation basin for the hypothetical case was 55.00 ft-MSL for the 25-yr, 24-hr storm and 55.15 ft-MSL for the 100-yr, 24-hr storm. Therefore, calculations indicate that the peak water surface elevation in the attenuation basin does not exceed the top of the decant structure for the 25-yr, 24-hr rainfall event, and the peak water surface elevation for the 100-yr, 24-hr rainfall event does not overtop the containment berm. As discussed, the actual elevation of the decant structure will vary, and these are hypothetical elevations established in order to run the model.

The area of the bottom of the attenuation basin required to discharge the contact water from the 25-yr, 24-hr rainfall event without overtopping the decant structure is 2.8 acres, i.e., approximately 9 percent of the total active area of 31 acres. As the overall landfill elevation increases, the active area would decrease and therefore, the minimum required volume of the attenuation basin would also decrease. For smaller active areas, the bottom of the attenuation basin will be sized in the same proportion, i.e., 9 percent, of the total active open area. Table E-5.2 summarizes the area and volume required to size the bottom of the attenuation basin for typical active areas.

The calculated peak inflow from the active area is 218 cfs for the 25-yr storm and 287 cfs for the 100-yr storm, and the calculated peak outflow from the decant structure is 14.2 cfs for the 25-yr storm and 16.5 cfs for the 100-yr storm, as shown in Table E-5.3. The inflow and outflow hydrographs for the 25-yr storm are shown in Figure E-5.3, and the hydrographs for the 100-yr storm are shown in Figure E-5.4. As shown in the hydrographs, the inflow during the 25-yr, 24-hr storm and 100-yr, 24-hr storm is discharged by the decant structure within approximately three days.

Leachate and Contact Water Storage Ponds

The calculated peak water surface elevation in the Temporary Pond and Permanent Pond is shown in Table E-5.4. As shown in Table E-5.4, the calculated peak water surface elevations do not exceed the top elevation of the temporary and permanent ponds for 25-yr, 24-hr storm or the 100-yr, 24-hr storm. Therefore, the ponds are designed to have adequate capacity to store the contact water generated by the design storm events (25-yr, 24-hr storm and 100-yr, 24-hr storm), in addition to storing the peak monthly average discharge of leachate and contact water from the active area. Following the design storm events, the submersible pumps in the Temporary and Permanent Leachate Ponds will pump the contact water to the on-site wastewater treatment facility until the contact water elevations reach the pump turn-off level, which approximately equals the contact water elevations at the beginning of the simulation for each pond.

CONCLUSIONS

Based on calculation herein, the decant structure is designed to drain the 25-yr, 24-hr storm within approximately three days without overtopping. In addition, the decant structure is

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designed to drain the 100-yr, 24-hr storm within approximately three days without overtopping the containment berm. The Temporary Pond and Permanent Pond are designed to have adequate capacity to manage the contact water and direct rainfall generated by the design storm events, in addition to storing the peak monthly average discharge of leachate and contact water from the active area, and peak monthly average rainfall over the ponds.

REFERENCES

HydroCAD Software Solutions, LLC. (2011). *HydroCAD Stormwater Modeling System Version 10 Owner's Manual.* Chocorua, NH: HydroCAD Software Solutions, LLC.

NOAA Hydrometeorological Design Studies Center. *Precipitation Frequency Data Server.* Retrieved May 06, 2016, from NOAA Atlas 14 Pont Precipitation Frequency Estimates (South Carolina): http://hdsc.nws.noaa.gov/hdsc/pfds/pfds_map_cont.html

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TABLE E-5.1 – POINT PRECIPITATION FREQUENCY ESTIMATE FOR THE SITE

PF tabular

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

Duration	Average recurrence interval (years)									
	1	2	5	10	25	50	100	200	500	1000
5-min	0.502 (0.468–0.543)	0.586 (0.545–0.636)	0.668 (0.619–0.722)	0.761 (0.703–0.823)	0.857 (0.789–0.927)	0.942 (0.863–1.02)	1.02 (0.929–1.10)	1.10 (0.993–1.19)	1.19 (1.07–1.29)	1.28 (1.14–1.39)
10-min	0.803 (0.748–0.868)	0.938 (0.872–1.02)	1.07 (0.992–1.16)	1.22 (1.13–1.32)	1.37 (1.26–1.48)	1.50 (1.38–1.62)	1.62 (1.48–1.75)	1.74 (1.57–1.88)	1.89 (1.69–2.04)	2.02 (1.79–2.19)
15-min	1.00 (0.935–1.09)	1.18 (1.10–1.28)	1.35 (1.25–1.46)	1.54 (1.42–1.67)	1.73 (1.59–1.87)	1.90 (1.74–2.05)	2.05 (1.87–2.22)	2.19 (1.99–2.37)	2.37 (2.13–2.57)	2.53 (2.25–2.75)
30-min	1.38 (1.28–1.49)	1.63 (1.51–1.77)	1.92 (1.78–2.08)	2.23 (2.06–2.41)	2.57 (2.36–2.77)	2.86 (2.62–3.09)	3.14 (2.86–3.39)	3.42 (3.09–3.69)	3.78 (3.38–4.09)	4.10 (3.64–4.45)
60-min	1.72 (1.60–1.85)	2.04 (1.90–2.22)	2.46 (2.29–2.67)	2.90 (2.68–3.14)	3.42 (3.14–3.69)	3.88 (3.55–4.19)	4.32 (3.94–4.67)	4.79 (4.34–5.18)	5.42 (4.86–5.87)	5.98 (5.32–6.50)
2-hr	2.08 (1.92–2.24)	2.49 (2.30–2.69)	3.07 (2.83–3.31)	3.66 (3.37–3.96)	4.37 (4.00–4.72)	5.01 (4.56–5.40)	5.63 (5.10–6.07)	6.27 (5.64–6.76)	7.10 (6.33–7.68)	7.85 (6.93–8.51)
3-hr	2.23 (2.06–2.42)	2.66 (2.45–2.91)	3.30 (3.03–3.59)	3.97 (3.64–4.33)	4.79 (4.37–5.21)	5.55 (5.02–6.03)	6.31 (5.66–6.84)	7.10 (6.33–7.71)	8.19 (7.20–8.91)	9.18 (7.98–10.0)
6-hr	2.66 (2.44–2.91)	3.19 (2.91–3.49)	3.94 (3.60–4.32)	4.75 (4.33–5.20)	5.76 (5.21–6.30)	6.69 (6.01–7.31)	7.61 (6.79–8.32)	8.62 (7.62–9.41)	9.97 (8.71–10.9)	11.2 (9.69–12.3)
12-hr	3.11 (2.84–3.44)	3.72 (3.39–4.11)	4.63 (4.21–5.12)	5.62 (5.10–6.20)	6.85 (6.17–7.54)	8.00 (7.15–8.78)	9.17 (8.12–10.0)	10.4 (9.15–11.4)	12.2 (10.5–13.4)	13.8 (11.8–15.2)
24-hr	3.64 (3.33–3.99)	4.42 (4.06–4.85)	5.71 (5.22–6.26)	6.77 (6.17–7.41)	8.28 (7.50–9.05)	9.53 (8.59–10.4)	10.9 (9.74–11.9)	12.3 (11.0–13.5)	14.4 (12.7–15.8)	16.1 (14.1–17.7)
2-day	4.30 (3.94–4.70)	5.21 (4.78–5.70)	6.66 (6.10–7.27)	7.85 (7.18–8.57)	9.56 (8.68–10.4)	11.0 (9.90–12.0)	12.5 (11.2–13.6)	14.1 (12.6–15.4)	16.4 (14.5–18.1)	18.3 (16.0–20.2)
3-day	4.55 (4.18–4.98)	5.50 (5.06–6.03)	7.00 (6.41–7.65)	8.21 (7.50–8.97)	9.93 (9.03–10.8)	11.3 (10.3–12.4)	12.9 (11.6–14.1)	14.5 (12.9–15.8)	16.7 (14.8–18.4)	18.6 (16.4–20.5)
4-day	4.81 (4.42–5.27)	5.80 (5.33–6.36)	7.33 (6.72–8.02)	8.57 (7.83–9.37)	10.3 (9.37–11.3)	11.7 (10.6–12.8)	13.2 (11.9–14.5)	14.8 (13.3–16.2)	17.1 (15.2–18.8)	18.9 (16.7–20.9)
7-day	5.61 (5.19–6.10)	6.76 (6.25–7.35)	8.44 (7.78–9.16)	9.78 (9.00–10.6)	11.6 (10.7–12.6)	13.1 (12.0–14.2)	14.7 (13.3–16.0)	16.3 (14.8–17.8)	18.6 (16.7–20.3)	20.5 (18.3–22.4)
10-day	6.36 (5.91–6.87)	7.63 (7.08–8.23)	9.36 (8.66–10.1)	10.7 (9.92–11.5)	12.5 (11.6–13.5)	14.0 (12.9–15.1)	15.5 (14.2–16.7)	17.0 (15.5–18.4)	19.1 (17.3–20.7)	20.9 (18.8–22.6)
20-day	8.60 (8.05–9.20)	10.2 (9.59–11.0)	12.4 (11.6–13.2)	14.0 (13.1–15.0)	16.3 (15.2–17.4)	18.0 (16.7–19.3)	19.8 (18.3–21.2)	21.7 (20.0–23.2)	24.2 (22.1–26.0)	26.2 (23.8–28.2)
30-day	10.6 (9.98–11.2)	12.6 (11.8–13.3)	14.9 (14.0–15.8)	16.7 (15.7–17.7)	19.0 (17.8–20.1)	20.8 (19.5–22.1)	22.6 (21.1–24.0)	24.4 (22.7–26.0)	26.9 (24.8–28.6)	28.8 (26.5–30.7)
45-day	13.4 (12.7–14.2)	15.8 (15.0–16.8)	18.5 (17.5–19.6)	20.5 (19.3–21.7)	23.1 (21.8–24.5)	25.1 (23.6–26.6)	27.1 (25.4–28.7)	29.0 (27.1–30.9)	31.6 (29.4–33.7)	33.6 (31.1–35.9)
60-day	16.0 (15.1–16.9)	18.8 (17.8–19.9)	21.8 (20.6–23.0)	24.1 (22.7–25.4)	26.9 (25.3–28.4)	29.1 (27.3–30.7)	31.2 (29.3–33.0)	33.3 (31.1–35.3)	36.0 (33.6–38.3)	38.1 (35.4–40.6)

[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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TABLE E-5.2 – SIZING OF ATTENUATION BASIN

Active Area (acres)	Area of Bottom of Attenuation Basin (acres)	Area of Top of Attenuation Basin (acres)	Approx. Basin Volume (acre-ft)
31	2.79	3.34	12.3
25	2.25	2.75	10.0
20	1.80	2.25	8.1
15	1.35	1.74	6.2
10	0.90	1.22	4.2
5	0.45	0.69	2.3

NOTE: The volume of the basin is computed assuming the basin is 4 ft deep.

TABLE E-5.3 – SUMMARY OF HYDROCAD MODEL OUTPUT FOR DECANT STRUCTURE

HydroCAD Output Parameter	25-yr, 24-hr storm	100-yr, 24-hr storm
Peak Inflow (cfs)	218	287
Peak Outflow (cfs)	14.2	16.5
Peak Water Surface Elevation (ft-MSL)	55.00	55.15

NOTE: Elevations in Table E-5.3 are hypothetical for the purposes of calculating flows, and are based on a bottom of decant structure intake at 51 ft-MSL (i.e., 4-ft deep basin area) for this hypothetical case. The actual elevation of the decant structures will vary as development and filling occurs and as it is raised with additional waste lifts.

TABLE E-5.4 – SUMMARY OF HYDROCAD MODEL OUTPUT FOR LEACHATE/CONTACT WATER PONDS

Pond	Rainfall Event	Peak Water Surface Elevation (ft-MSL)	Volume Stored in Pond (ft^3)
Temporary Leachate Pond	25-yr, 24-hr Rainfall	36.69	1,462,024
	100-yr, 24-hr Rainfall	38.06	1,801,184
Permanent Leachate Pond	25-yr, 24-hr Rainfall	35.04	1,341,498
	100-yr, 24-hr Rainfall	37.07	1,660,229

NOTE: Bottom of the temporary leachate pond is at 27 ft-MSL and bottom of the permanent leachate pond is at 24 ft-MSL.

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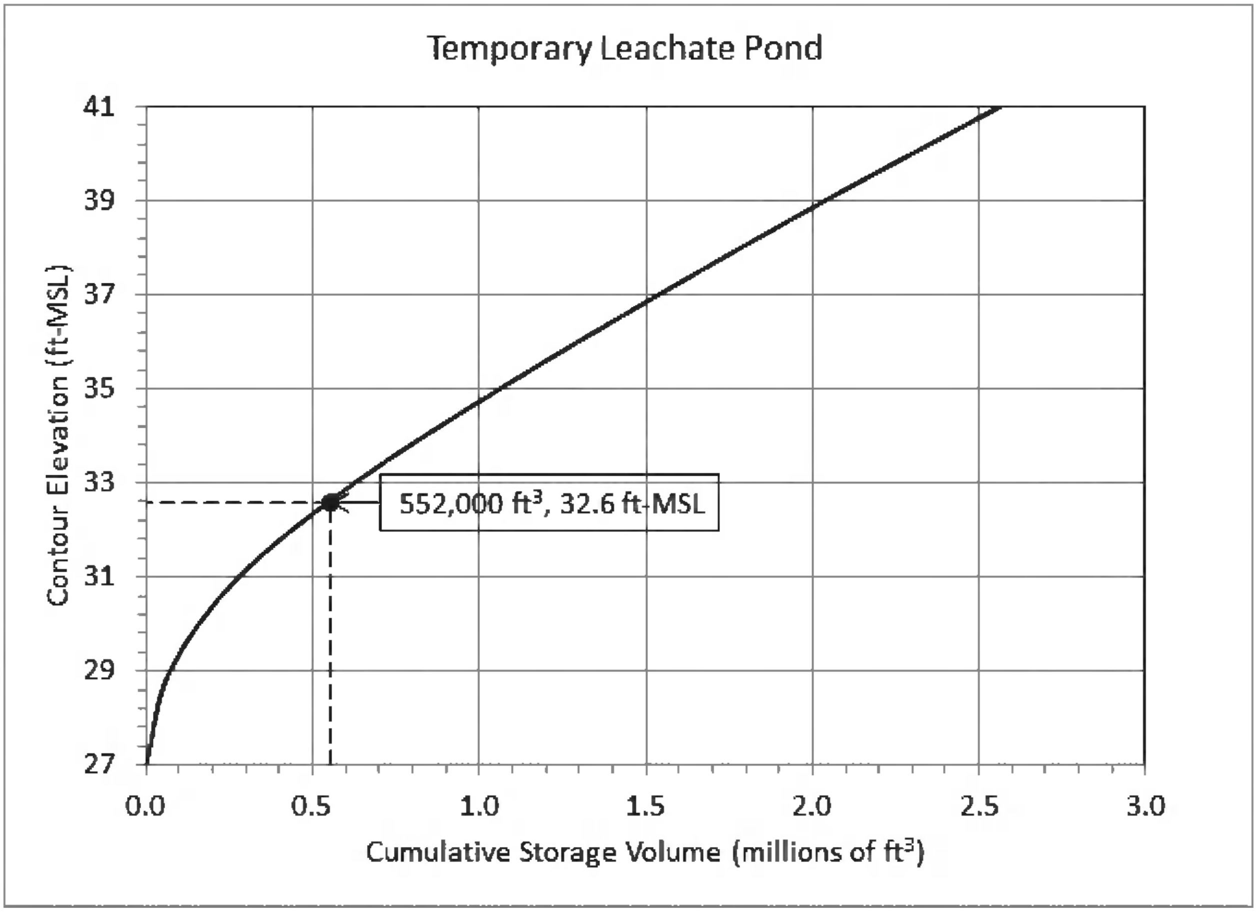

Figure E-5.1. Cumulative Storage Volume of Temporary Pond (Initial Pond Elevation Shown)

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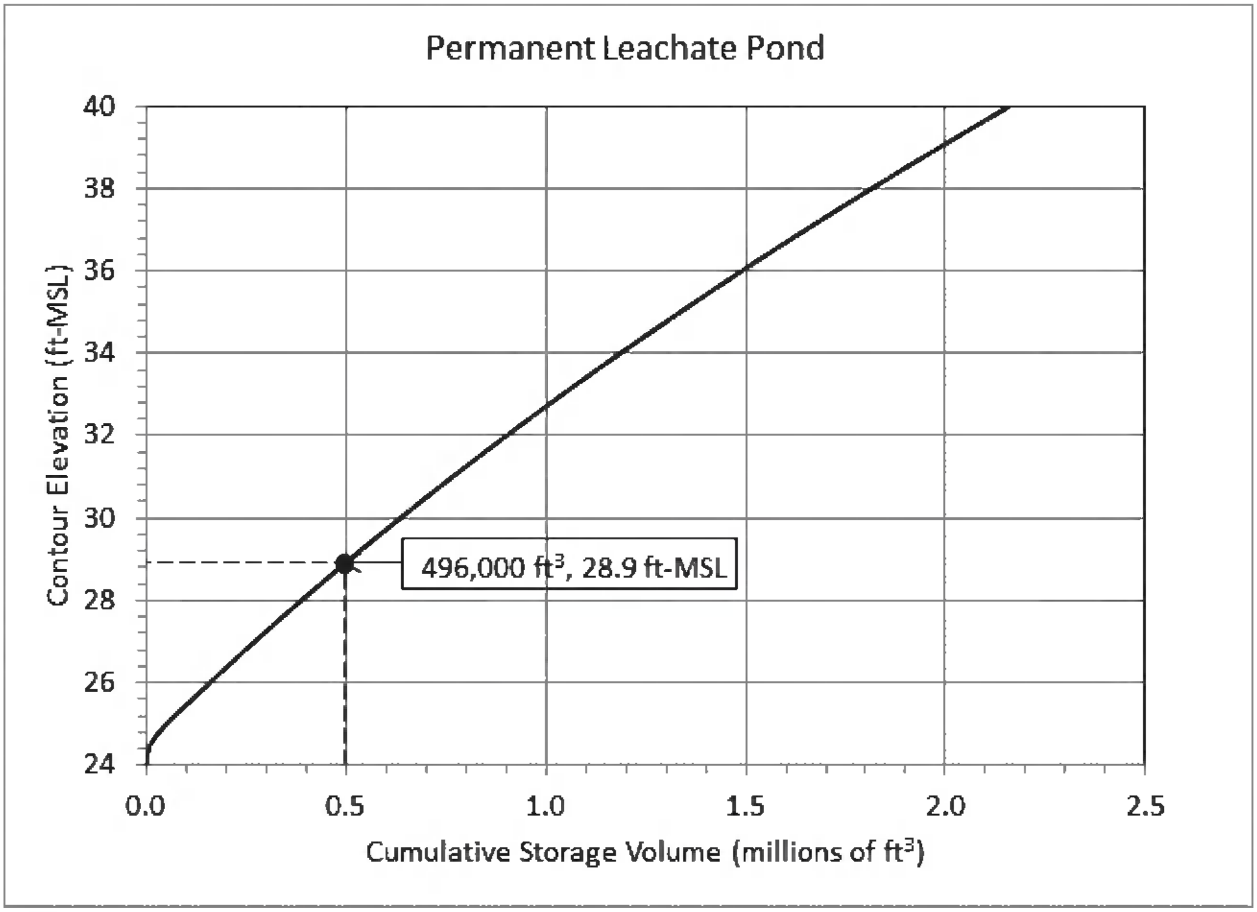

Figure E-5.2. Cumulative Storage Volume of Permanent Pond (Initial Pond Elevation Shown)

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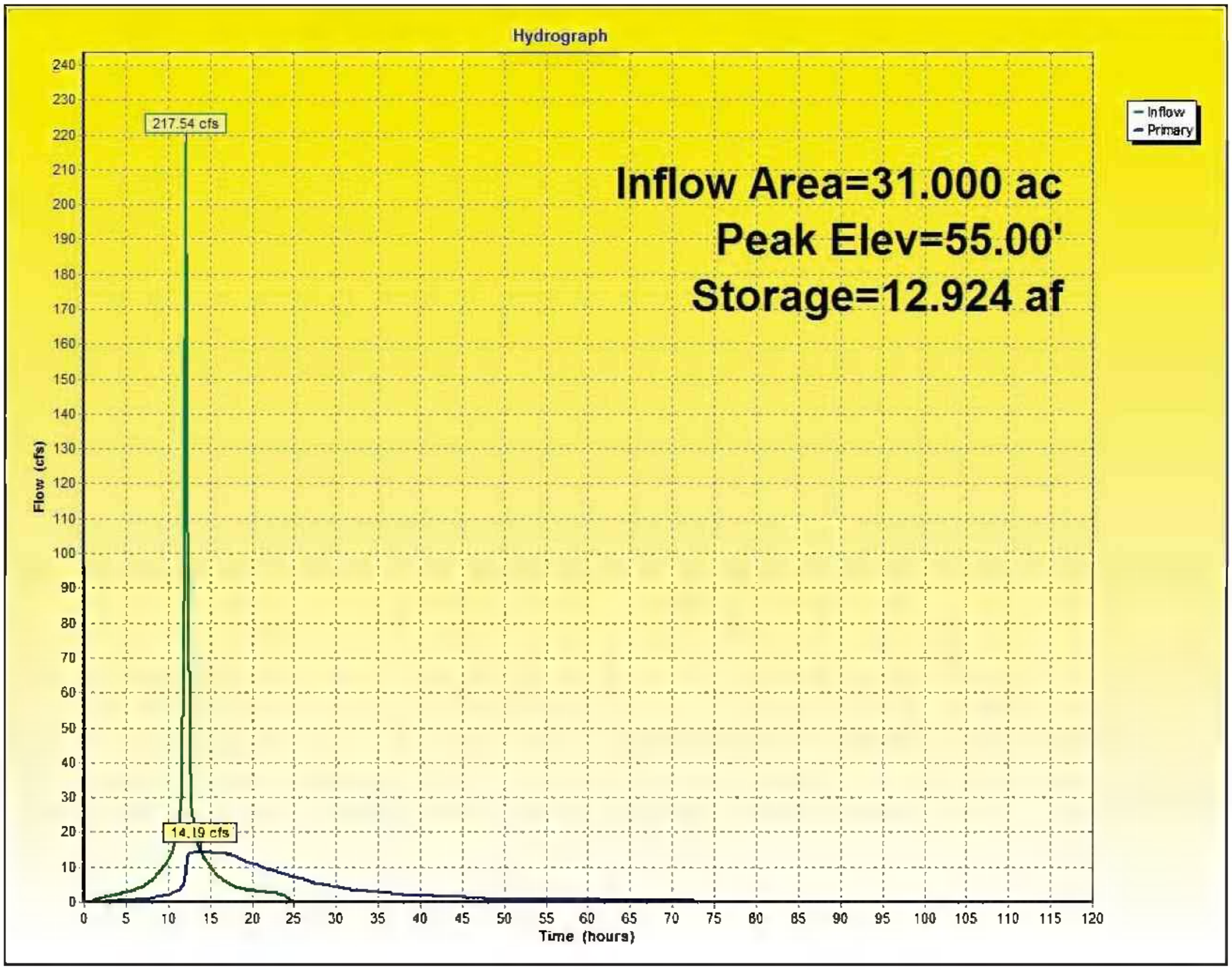

Figure E-5.3. Inflow and Outflow Hydrograph for 25-yr, 24-hr Rainfall Event

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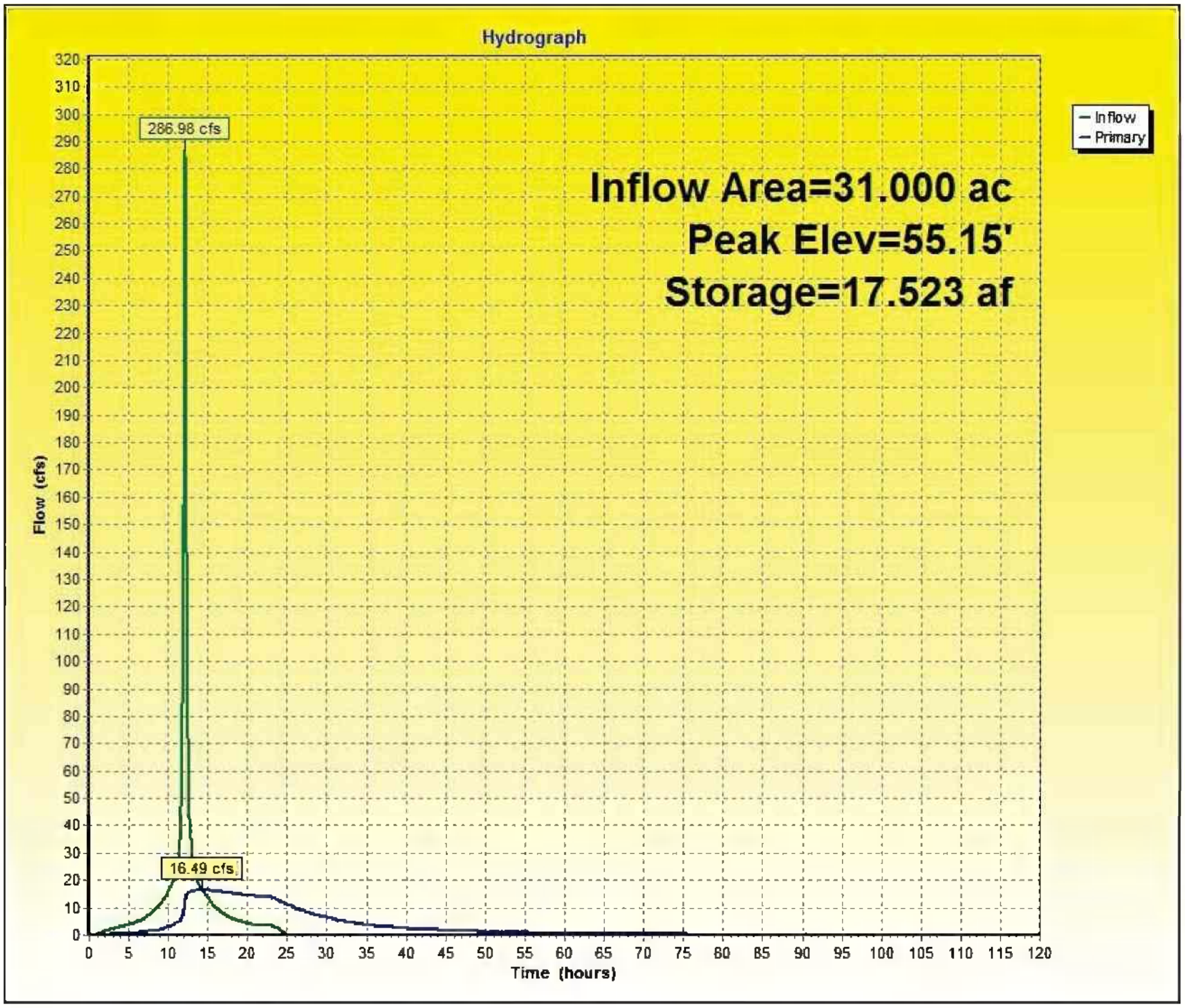

Figure E-5.4. Inflow and Outflow Hydrograph for 100-yr, 24-hr Rainfall Event

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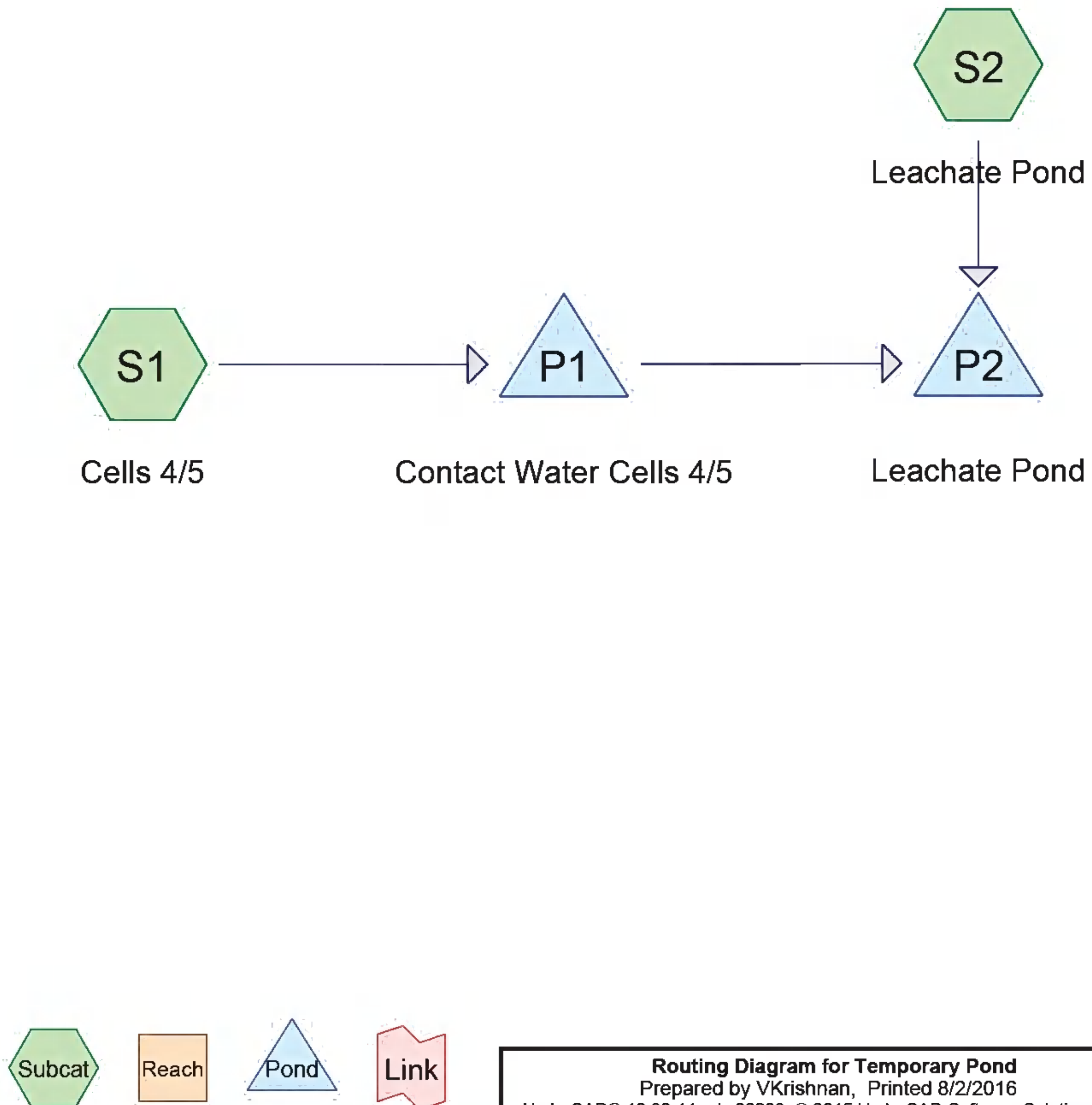
APPENDIX E-5.A

HYDROCAD REPORTS

TEMPORARY LEACHATE POND



Routing Diagram for Temporary Pond
Prepared by VKrishnan, Printed 8/2/2016
HydroCAD® 10.00-14 s/n 02233 © 2015 HydroCAD Software Solutions LLC

Area Listing (all nodes)

Area (acres)	CN	Description (subcatchment-numbers)
31.000	97	(S1)
6.173	100	(S2)
37.173	**97**	**TOTAL AREA**

Soil Listing (all nodes)

Area (acres)	Soil Group	Subcatchment Numbers
0.000	HSG A	
0.000	HSG B	
0.000	HSG C	
0.000	HSG D	
37.173	Other	S1, S2
37.173		**TOTAL AREA**

Ground Covers (all nodes)

HSG-A (acres)	HSG-B (acres)	HSG-C (acres)	HSG-D (acres)	Other (acres)	Total (acres)	Ground Cover	Subcatchment Numbers
0.000	0.000	0.000	0.000	37.173	37.173		S1, S2
0.000	**0.000**	**0.000**	**0.000**	**37.173**	**37.173**	**TOTAL AREA**	

Pipe Listing (all nodes)

Line#	Node Number	In-Invert (feet)	Out-Invert (feet)	Length (feet)	Slope (ft/ft)	n	Diam/Width (inches)	Height (inches)	Inside-Fill (inches)
1	P1	36.00	29.50	1,300.0	0.0050	0.009	18.0	0.0	0.0

Time span=0.00-120.00 hrs, dt=0.05 hrs, 2401 points
Runoff by SCS TR-20 method, UH=SCS, Weighted-CN
Reach routing by Dyn-Stor-Ind method - Pond routing by Dyn-Stor-Ind method

Subcatchment S1: Cells 4/5 Runoff Area=31.000 ac 0.00% Impervious Runoff Depth=7.92"
Tc=10.0 min CN=97 Runoff=217.54 cfs 20.460 af

Subcatchment S2: Leachate Pond Runoff Area=268,880 sf 100.00% Impervious Runoff Depth=8.28"
Tc=0.0 min CN=100 Runoff=57.55 cfs 4.259 af

Pond P1: Contact Water Cells 4/5 Peak Elev=55.00' Storage=12.924 af Inflow=217.54 cfs 20.460 af
Outflow=14.19 cfs 20.460 af

Pond P2: Leachate Pond Peak Elev=36.69' Storage=1,462,024 cf Inflow=64.22 cfs 24.719 af
Outflow=0.78 cfs 7.004 af

Total Runoff Area = 37.173 ac Runoff Volume = 24.719 af Average Runoff Depth = 7.98"
83.39% Pervious = 31.000 ac 16.61% Impervious = 6.173 ac

Summary for Subcatchment S1: Cells 4/5

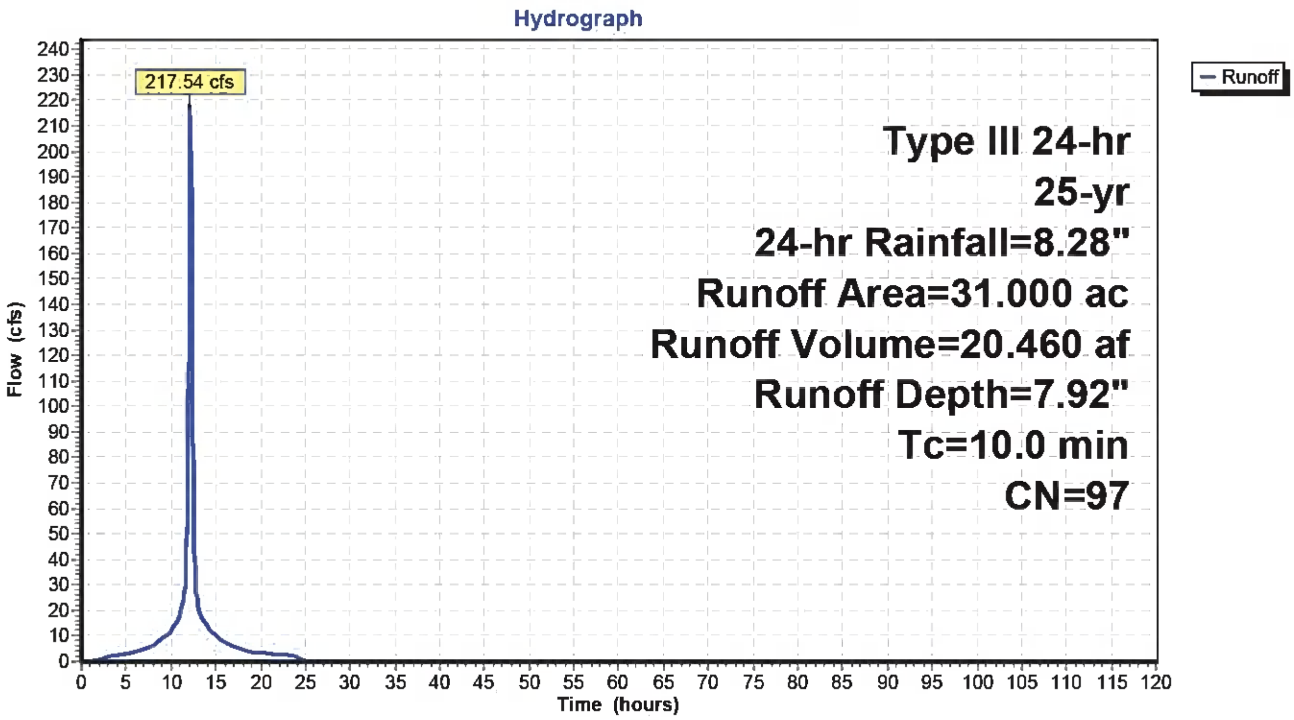
Runoff = 217.54 cfs @ 12.14 hrs, Volume= 20.460 af, Depth= 7.92"

Runoff by SCS TR-20 method, UH=SCS, Weighted-CN, Time Span= 0.00-120.00 hrs, dt= 0.05 hrs
Type III 24-hr 25-yr, 24-hr Rainfall=8.28"

	Area (ac)	CN	Description
*	31.000	97	
	31.000		100.00% Pervious Area

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
10.0					**Direct Entry,**

Subcatchment S1: Cells 4/5



Summary for Subcatchment S2: Leachate Pond

[46] Hint: Tc=0 (Instant runoff peak depends on dt)

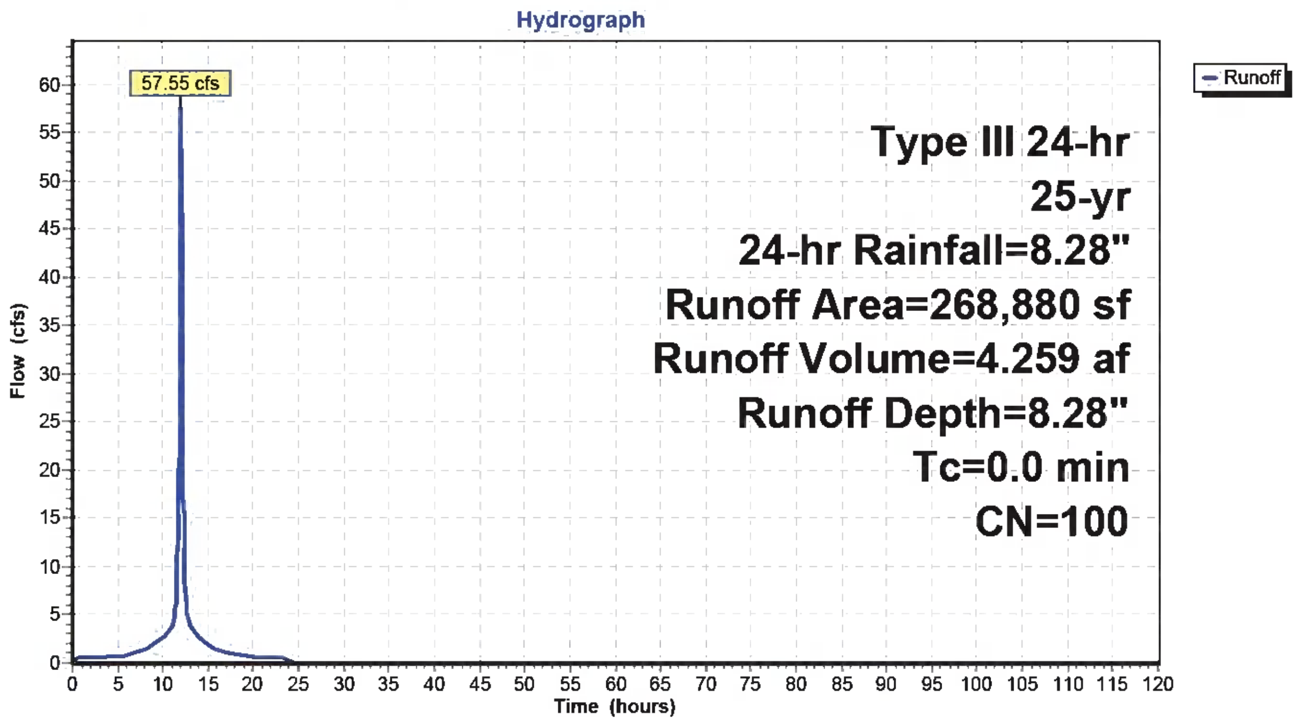
Runoff = 57.55 cfs @ 12.00 hrs, Volume= 4.259 af, Depth= 8.28"

Runoff by SCS TR-20 method, UH=SCS, Weighted-CN, Time Span= 0.00-120.00 hrs, dt= 0.05 hrs
Type III 24-hr 25-yr, 24-hr Rainfall=8.28"

	Area (sf)	CN	Description
*	268,880	100	
	268,880		100.00% Impervious Area

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
0.0					**Direct Entry,**

Subcatchment S2: Leachate Pond



Summary for Pond P1: Contact Water Cells 4/5

Inflow Area = 31.000 ac, 0.00% Impervious, Inflow Depth = 7.92" for 25-yr, 24-hr event
Inflow = 217.54 cfs @ 12.14 hrs, Volume= 20.460 af
Outflow = 14.19 cfs @ 13.91 hrs, Volume= 20.460 af, Atten= 93%, Lag= 106.2 min
Primary = 14.19 cfs @ 13.91 hrs, Volume= 20.460 af

Routing by Dyn-Stor-Ind method, Time Span= 0.00-120.00 hrs, dt= 0.05 hrs
Peak Elev= 55.00' @ 13.91 hrs Surf.Area= 29.989 ac Storage= 12.924 af

Plug-Flow detention time= 687.1 min calculated for 20.452 af (100% of inflow)
Center-of-Mass det. time= 688.1 min (1,438.4 - 750.3)

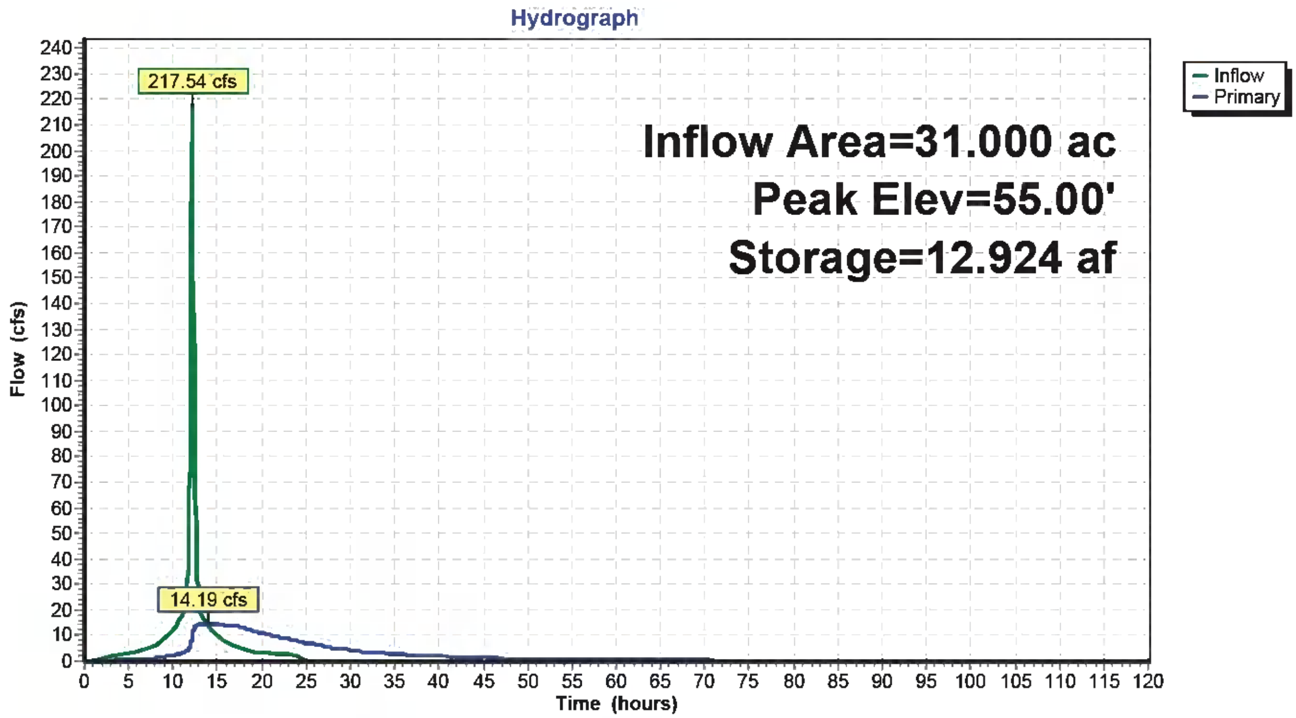
Volume	Invert	Avail.Storage	Storage Description
#1	51.00'	74.999 af	**Custom Stage Data (Conic)**Listed below (Recalc)

Elevation (feet)	Surf.Area (acres)	Inc.Store (acre-feet)	Cum.Store (acre-feet)	Wet.Area (acres)
51.00	0.001	0.000	0.000	0.001
51.20	2.790	0.190	0.190	2.790
54.90	3.340	11.325	11.515	3.351
55.00	31.000	1.484	12.999	31.011
57.00	31.000	62.000	74.999	31.200

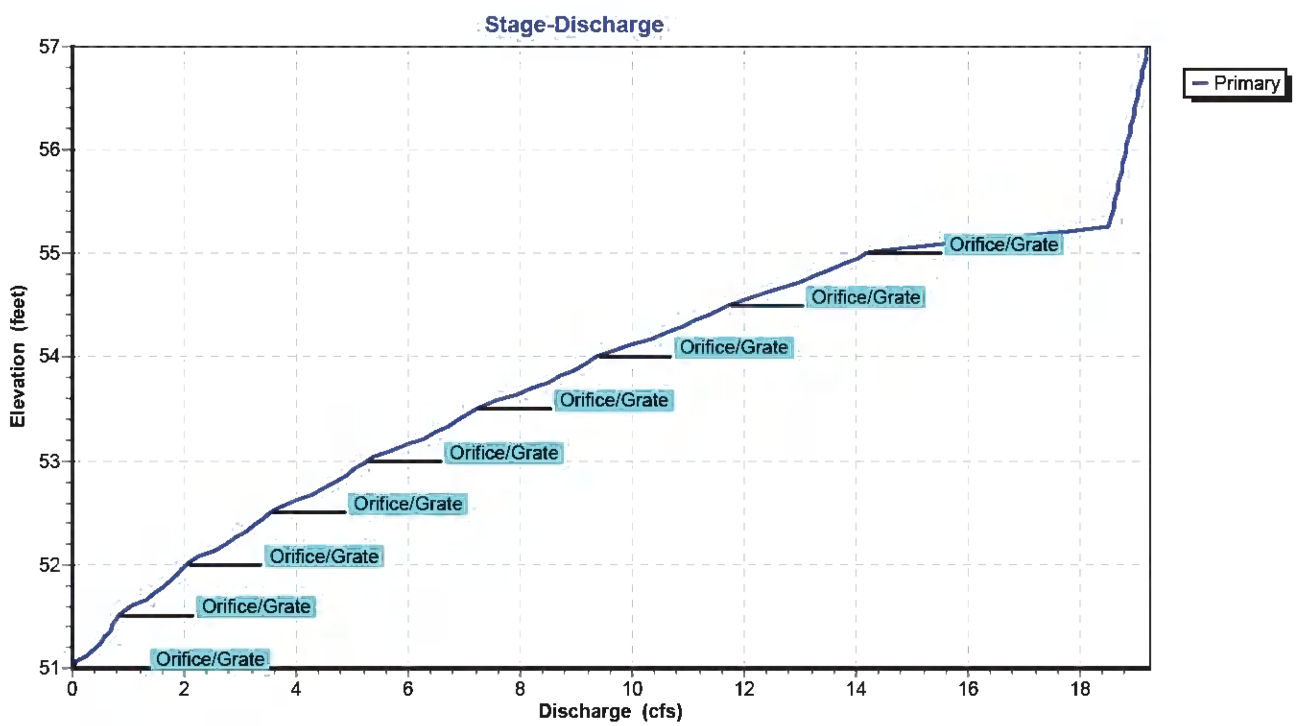
Device	Routing	Invert	Outlet Devices
#1	Primary	36.00'	**18.0" Round Culvert** L= 1,300.0' CPP, projecting, no headwall, Ke= 0.900 Inlet / Outlet Invert= 36.00' / 29.50' S= 0.0050 '/' Cc= 0.900 n= 0.009, Flow Area= 1.77 sf
#2	Device 1	51.00'	**2.0" Vert. Orifice/Grate X 12.00** C= 0.600
#3	Device 1	51.50'	**2.0" Vert. Orifice/Grate X 12.00** C= 0.600
#4	Device 1	52.00'	**2.0" Vert. Orifice/Grate X 12.00** C= 0.600
#5	Device 1	52.50'	**2.0" Vert. Orifice/Grate X 12.00** C= 0.600
#6	Device 1	53.00'	**2.0" Vert. Orifice/Grate X 12.00** C= 0.600
#7	Device 1	53.50'	**2.0" Vert. Orifice/Grate X 12.00** C= 0.600
#8	Device 1	54.00'	**2.0" Vert. Orifice/Grate X 12.00** C= 0.600
#9	Device 1	54.50'	**2.0" Vert. Orifice/Grate X 12.00** C= 0.600
#10	Device 1	55.00'	**36.0" Horiz. Orifice/Grate** C= 0.600 Limited to weir flow at low heads

Primary OutFlow Max=14.19 cfs @ 13.91 hrs HW=55.00' TW=33.94' (Dynamic Tailwater)
└─1=**Culvert** (Passes 14.19 cfs of 17.85 cfs potential flow)
  ├─2=**Orifice/Grate** (Orifice Controls 2.49 cfs @ 9.53 fps)
  ├─3=**Orifice/Grate** (Orifice Controls 2.33 cfs @ 8.90 fps)
  ├─4=**Orifice/Grate** (Orifice Controls 2.15 cfs @ 8.22 fps)
  ├─5=**Orifice/Grate** (Orifice Controls 1.96 cfs @ 7.48 fps)
  ├─6=**Orifice/Grate** (Orifice Controls 1.74 cfs @ 6.66 fps)
  ├─7=**Orifice/Grate** (Orifice Controls 1.50 cfs @ 5.73 fps)
  ├─8=**Orifice/Grate** (Orifice Controls 1.21 cfs @ 4.60 fps)
  ├─9=**Orifice/Grate** (Orifice Controls 0.81 cfs @ 3.10 fps)
  └─10=**Orifice/Grate** (Controls 0.00 cfs)

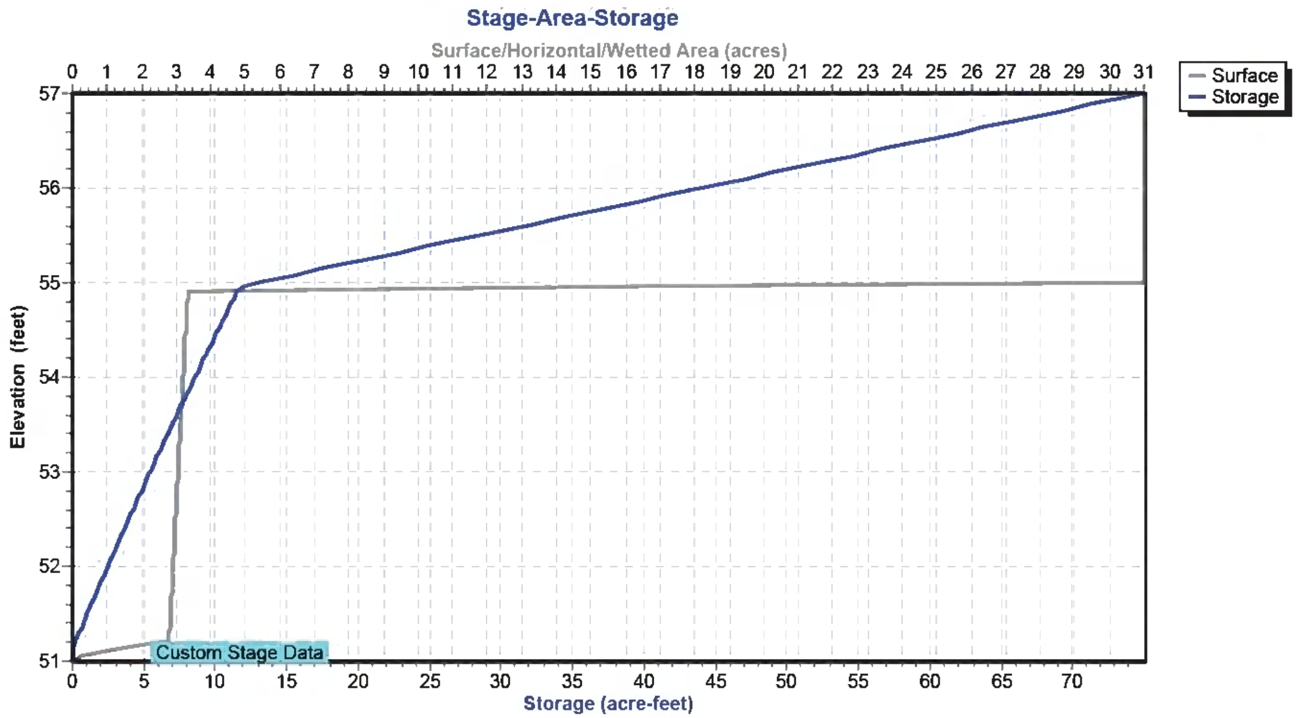
Pond P1: Contact Water Cells 4/5



Pond P1: Contact Water Cells 4/5



Pond P1: Contact Water Cells 4/5



Summary for Pond P2: Leachate Pond

Inflow Area = 37.173 ac, 16.61% Impervious, Inflow Depth = 7.98" for 25-yr, 24-hr event
Inflow = 64.22 cfs @ 12.00 hrs, Volume= 24.719 af
Outflow = 0.78 cfs @ 52.27 hrs, Volume= 7.004 af, Atten= 99%, Lag= 2,416.3 min
Primary = 0.78 cfs @ 52.27 hrs, Volume= 7.004 af

Routing by Dyn-Stor-Ind method, Time Span= 0.00-120.00 hrs, dt= 0.05 hrs
Starting Elev= 32.57' Surf.Area= 190,842 sf Storage= 540,966 cf
Peak Elev= 36.69' @ 52.27 hrs Surf.Area= 242,272 sf Storage= 1,462,024 cf (921,058 cf above start)

Plug-Flow detention time= (not calculated: initial storage exceeds outflow)
Center-of-Mass det. time= 2,627.2 min (3,941.8 - 1,314.6)

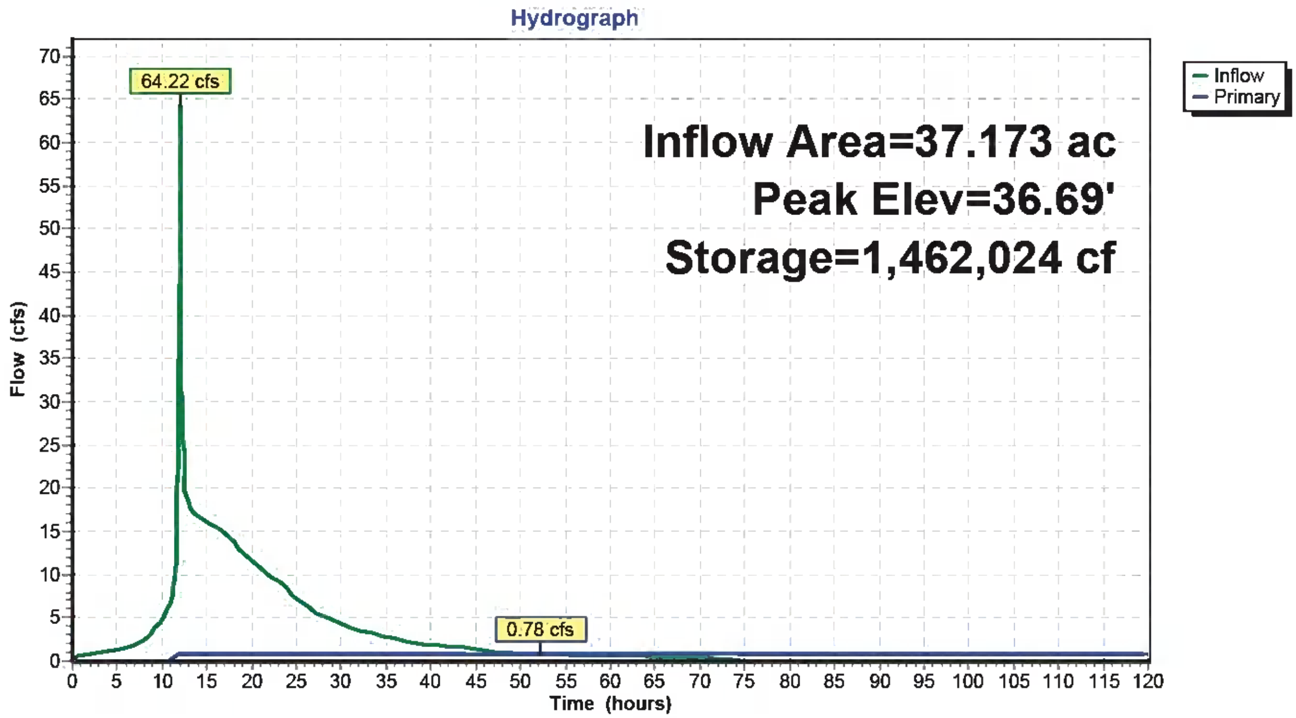
Volume	Invert	Avail.Storage	Storage Description
#1	27.00'	2,564,304 cf	**Custom Stage Data (Conic)**Listed below (Recalc)

Elevation (feet)	Surf.Area (sq-ft)	Inc.Store (cubic-feet)	Cum.Store (cubic-feet)	Wet.Area (sq-ft)
27.00	12,042	0	0	12,042
29.00	66,802	71,471	71,471	66,818
31.00	144,923	206,745	278,216	144,971
33.00	204,519	347,736	625,952	204,641
35.00	232,106	436,334	1,062,286	232,425
37.00	244,194	476,249	1,538,535	244,999
39.00	256,397	500,541	2,039,076	257,707
41.00	268,880	525,228	2,564,304	270,708

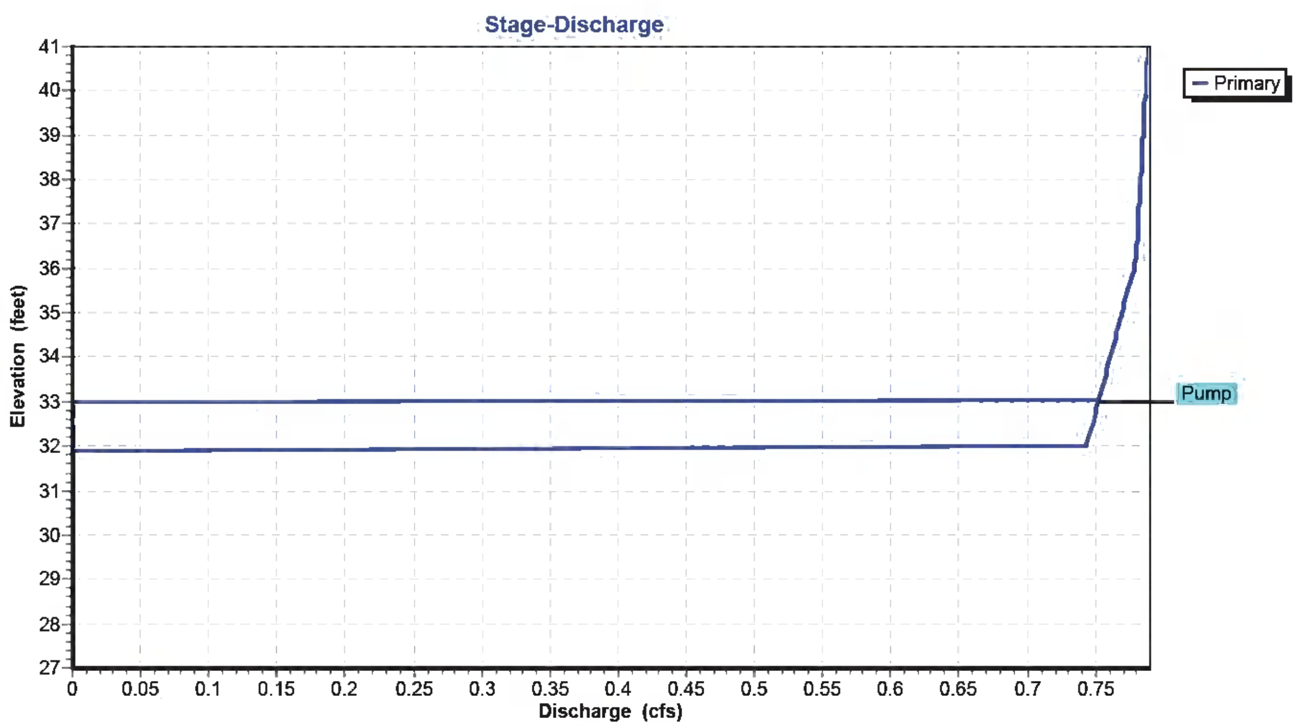
Device	Routing	Invert	Outlet Devices
#1	Primary	33.00'	**Pump** Discharges@41.00' Turns Off@32.00' 6.0" Diam. x 300.0' Long Discharge, Hazen-Williams C= 130 Flow (gpm)= 40.0 100.0 200.0 250.0 300.0 350.0 355.0 Head (feet)= 47.00 40.00 30.00 26.00 20.00 8.00 2.00 -Loss (feet)= 0.06 0.31 1.13 1.71 2.39 3.18 3.27 =Lift (feet)= 46.94 39.69 28.87 24.29 17.61 4.82 -1.27

Primary OutFlow Max=0.78 cfs @ 52.27 hrs HW=36.69' (Free Discharge)
↑—**1=Pump** (Pump Controls 0.78 cfs)

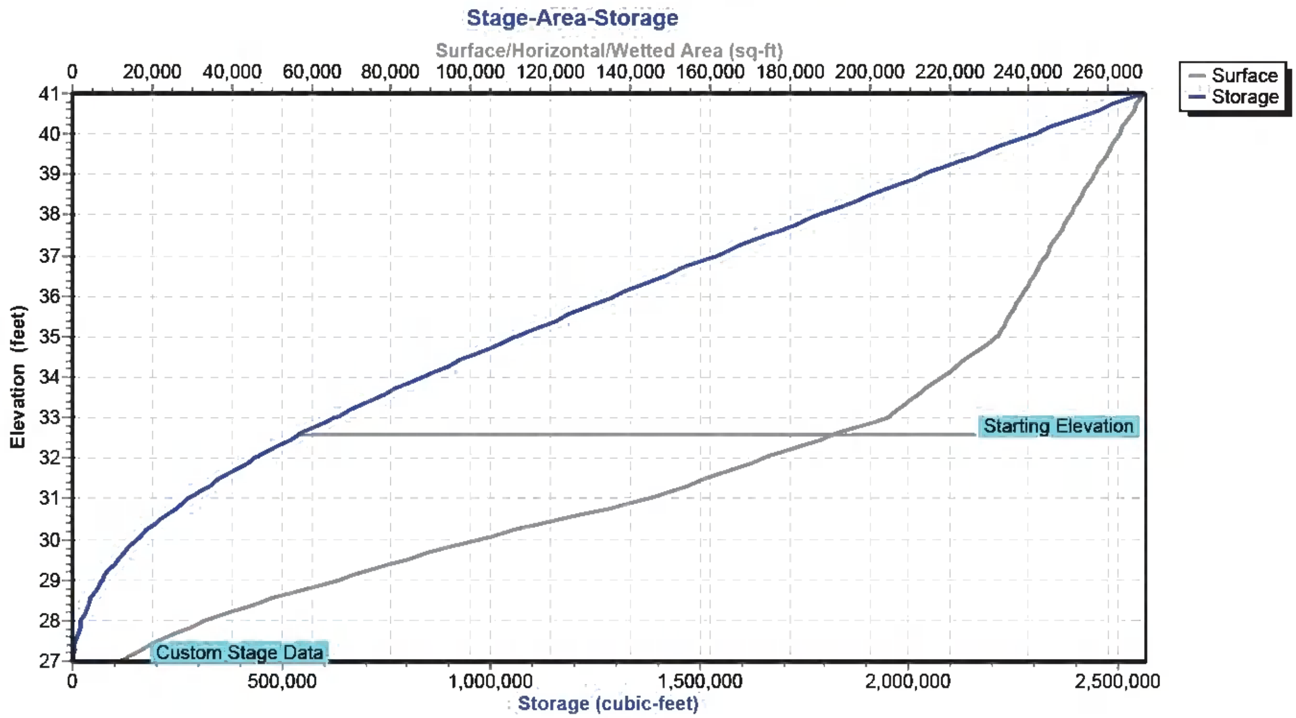
Pond P2: Leachate Pond



Pond P2: Leachate Pond



Pond P2: Leachate Pond



Time span=0.00-120.00 hrs, dt=0.05 hrs, 2401 points
Runoff by SCS TR-20 method, UH=SCS, Weighted-CN
Reach routing by Dyn-Stor-Ind method - Pond routing by Dyn-Stor-Ind method

Subcatchment S1: Cells 4/5 Runoff Area=31.000 ac 0.00% Impervious Runoff Depth=10.54"
Tc=10.0 min CN=97 Runoff=286.98 cfs 27.222 af

Subcatchment S2: Leachate Pond Runoff Area=268,880 sf 100.00% Impervious Runoff Depth=10.90"
Tc=0.0 min CN=100 Runoff=75.76 cfs 5.607 af

Pond P1: Contact Water Cells 4/5 Peak Elev=55.15' Storage=17.523 af Inflow=286.98 cfs 27.222 af
Outflow=16.49 cfs 27.222 af

Pond P2: Leachate Pond Peak Elev=38.06' Storage=1,801,184 cf Inflow=85.23 cfs 32.829 af
Outflow=0.78 cfs 7.091 af

Total Runoff Area = 37.173 ac Runoff Volume = 32.829 af Average Runoff Depth = 10.60"
83.39% Pervious = 31.000 ac 16.61% Impervious = 6.173 ac

Summary for Subcatchment S1: Cells 4/5

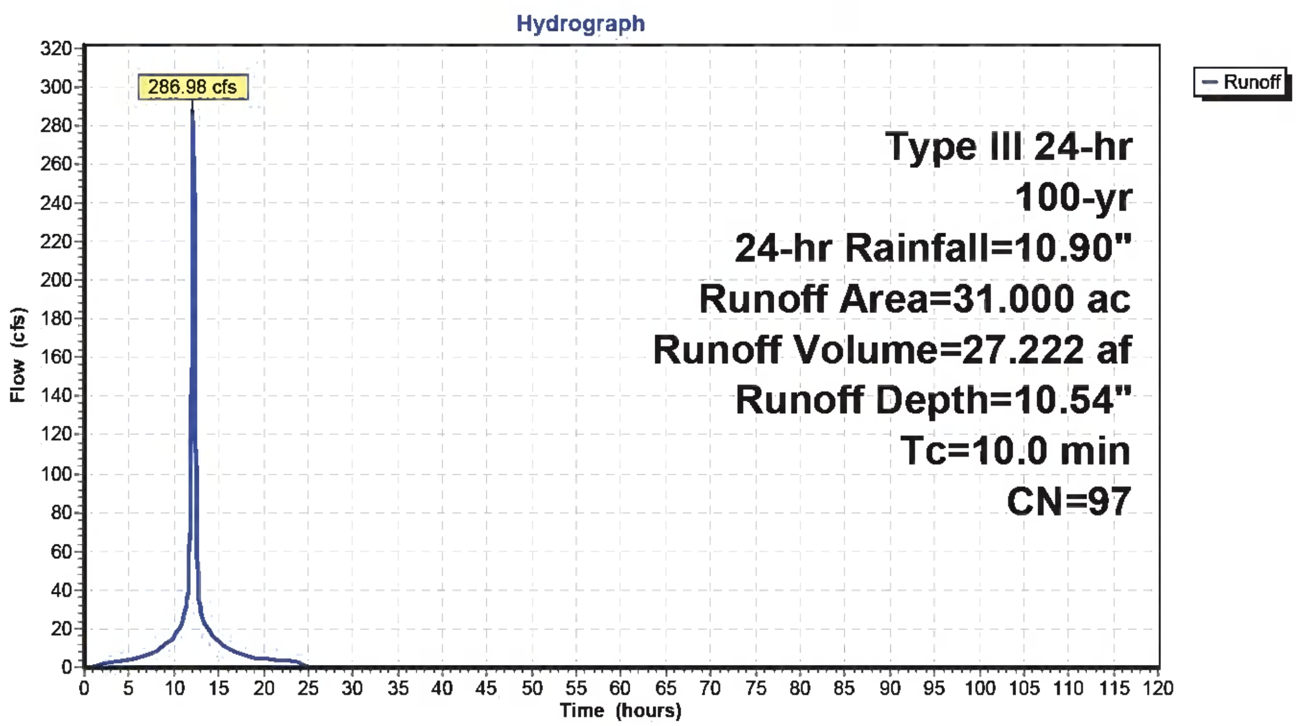
Runoff = 286.98 cfs @ 12.14 hrs, Volume= 27.222 af, Depth=10.54"

Runoff by SCS TR-20 method, UH=SCS, Weighted-CN, Time Span= 0.00-120.00 hrs, dt= 0.05 hrs
Type III 24-hr 100-yr, 24-hr Rainfall=10.90"

	Area (ac)	CN	Description
*	31.000	97	
	31.000		100.00% Pervious Area

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
10.0					**Direct Entry,**

Subcatchment S1: Cells 4/5



Summary for Subcatchment S2: Leachate Pond

[46] Hint: Tc=0 (Instant runoff peak depends on dt)

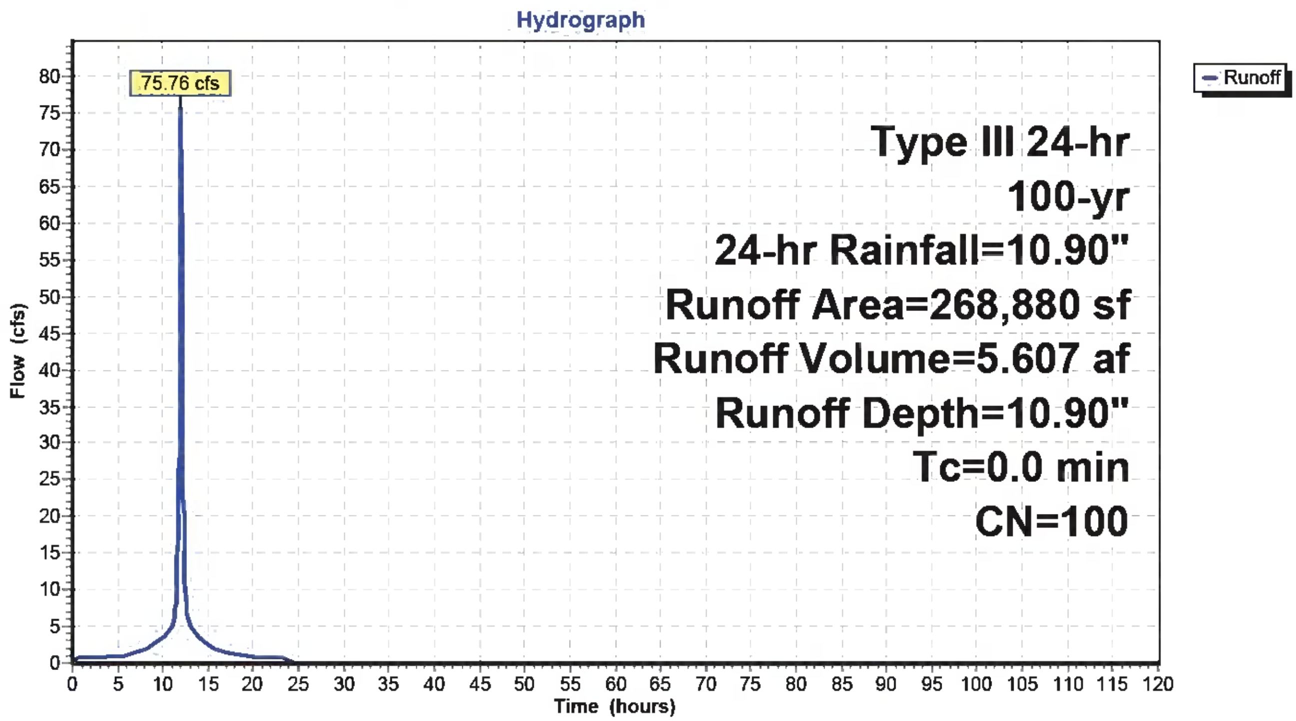
Runoff = 75.76 cfs @ 12.00 hrs, Volume= 5.607 af, Depth=10.90"

Runoff by SCS TR-20 method, UH=SCS, Weighted-CN, Time Span= 0.00-120.00 hrs, dt= 0.05 hrs
Type III 24-hr 100-yr, 24-hr Rainfall=10.90"

	Area (sf)	CN	Description
*	268,880	100	
	268,880		100.00% Impervious Area

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
0.0					**Direct Entry,**

Subcatchment S2: Leachate Pond



Summary for Pond P1: Contact Water Cells 4/5

Inflow Area = 31.000 ac, 0.00% Impervious, Inflow Depth = 10.54" for 100-yr, 24-hr event
Inflow = 286.98 cfs @ 12.14 hrs, Volume= 27.222 af
Outflow = 16.49 cfs @ 14.22 hrs, Volume= 27.222 af, Atten= 94%, Lag= 124.7 min
Primary = 16.49 cfs @ 14.22 hrs, Volume= 27.222 af

Routing by Dyn-Stor-Ind method, Time Span= 0.00-120.00 hrs, dt= 0.05 hrs
Peak Elev= 55.15' @ 14.22 hrs Surf.Area= 31.000 ac Storage= 17.523 af

Plug-Flow detention time= 719.7 min calculated for 27.210 af (100% of inflow)
Center-of-Mass det. time= 720.8 min (1,467.1 - 746.3)

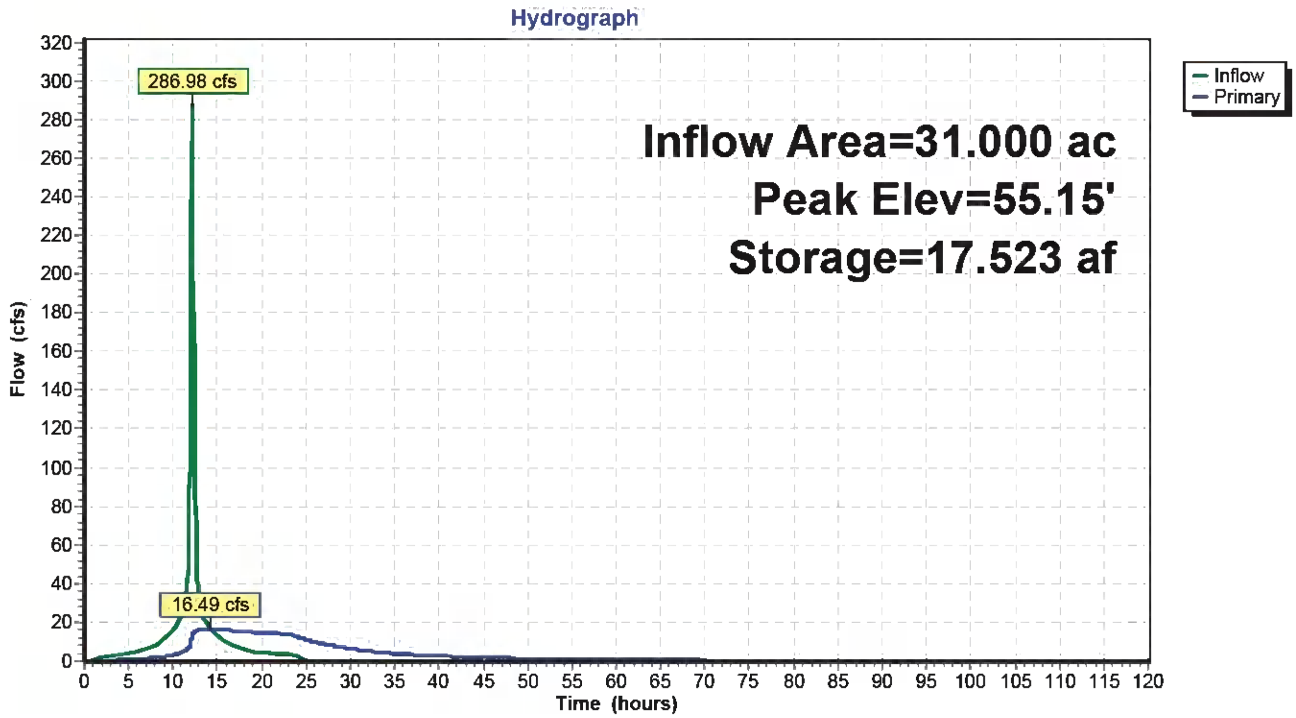
Volume	Invert	Avail.Storage	Storage Description
#1	51.00'	74.999 af	**Custom Stage Data (Conic)**Listed below (Recalc)

Elevation (feet)	Surf.Area (acres)	Inc.Store (acre-feet)	Cum.Store (acre-feet)	Wet.Area (acres)
51.00	0.001	0.000	0.000	0.001
51.20	2.790	0.190	0.190	2.790
54.90	3.340	11.325	11.515	3.351
55.00	31.000	1.484	12.999	31.011
57.00	31.000	62.000	74.999	31.200

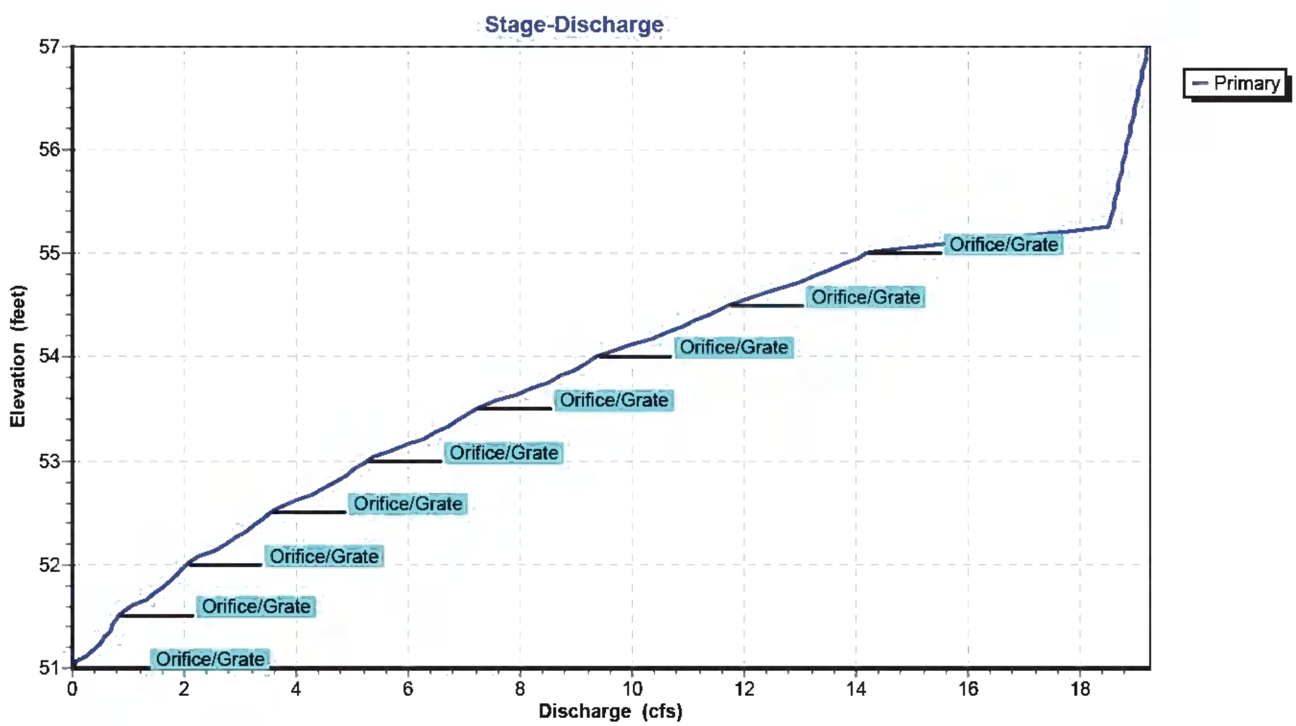
Device	Routing	Invert	Outlet Devices
#1	Primary	36.00'	**18.0" Round Culvert** L= 1,300.0' CPP, projecting, no headwall, Ke= 0.900 Inlet / Outlet Invert= 36.00' / 29.50' S= 0.0050 '/' Cc= 0.900 n= 0.009, Flow Area= 1.77 sf
#2	Device 1	51.00'	**2.0" Vert. Orifice/Grate X 12.00** C= 0.600
#3	Device 1	51.50'	**2.0" Vert. Orifice/Grate X 12.00** C= 0.600
#4	Device 1	52.00'	**2.0" Vert. Orifice/Grate X 12.00** C= 0.600
#5	Device 1	52.50'	**2.0" Vert. Orifice/Grate X 12.00** C= 0.600
#6	Device 1	53.00'	**2.0" Vert. Orifice/Grate X 12.00** C= 0.600
#7	Device 1	53.50'	**2.0" Vert. Orifice/Grate X 12.00** C= 0.600
#8	Device 1	54.00'	**2.0" Vert. Orifice/Grate X 12.00** C= 0.600
#9	Device 1	54.50'	**2.0" Vert. Orifice/Grate X 12.00** C= 0.600
#10	Device 1	55.00'	**36.0" Horiz. Orifice/Grate** C= 0.600 Limited to weir flow at low heads

Primary OutFlow Max=16.49 cfs @ 14.22 hrs HW=55.15' TW=34.40' (Dynamic Tailwater)
- **1=Culvert** (Passes 16.49 cfs of 17.72 cfs potential flow)
  - **2=Orifice/Grate** (Orifice Controls 2.54 cfs @ 9.71 fps)
  - **3=Orifice/Grate** (Orifice Controls 2.38 cfs @ 9.09 fps)
  - **4=Orifice/Grate** (Orifice Controls 2.21 cfs @ 8.43 fps)
  - **5=Orifice/Grate** (Orifice Controls 2.02 cfs @ 7.71 fps)
  - **6=Orifice/Grate** (Orifice Controls 1.81 cfs @ 6.92 fps)
  - **7=Orifice/Grate** (Orifice Controls 1.58 cfs @ 6.02 fps)
  - **8=Orifice/Grate** (Orifice Controls 1.30 cfs @ 4.96 fps)
  - **9=Orifice/Grate** (Orifice Controls 0.95 cfs @ 3.61 fps)
  - **10=Orifice/Grate** (Weir Controls 1.72 cfs @ 1.25 fps)

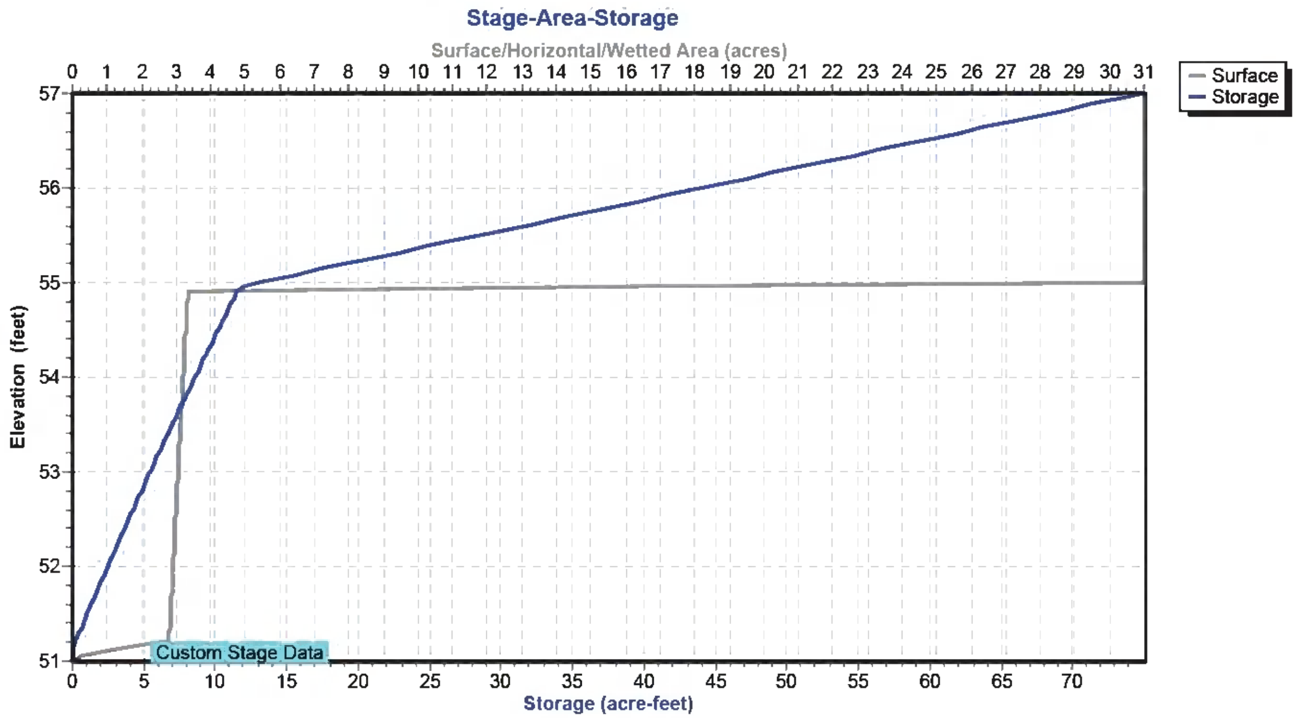
Pond P1: Contact Water Cells 4/5



Pond P1: Contact Water Cells 4/5



Pond P1: Contact Water Cells 4/5



Summary for Pond P2: Leachate Pond

Inflow Area = 37.173 ac, 16.61% Impervious, Inflow Depth = 10.60" for 100-yr, 24-hr event
Inflow = 85.23 cfs @ 12.00 hrs, Volume= 32.829 af
Outflow = 0.78 cfs @ 56.04 hrs, Volume= 7.091 af, Atten= 99%, Lag= 2,642.4 min
Primary = 0.78 cfs @ 56.04 hrs, Volume= 7.091 af

Routing by Dyn-Stor-Ind method, Time Span= 0.00-120.00 hrs, dt= 0.05 hrs
Starting Elev= 32.57' Surf.Area= 190,842 sf Storage= 540,966 cf
Peak Elev= 38.06' @ 56.04 hrs Surf.Area= 250,634 sf Storage= 1,801,184 cf (1,260,218 cf above start)

Plug-Flow detention time= (not calculated: initial storage exceeds outflow)
Center-of-Mass det. time= 2,573.0 min (3,912.4 - 1,339.5)

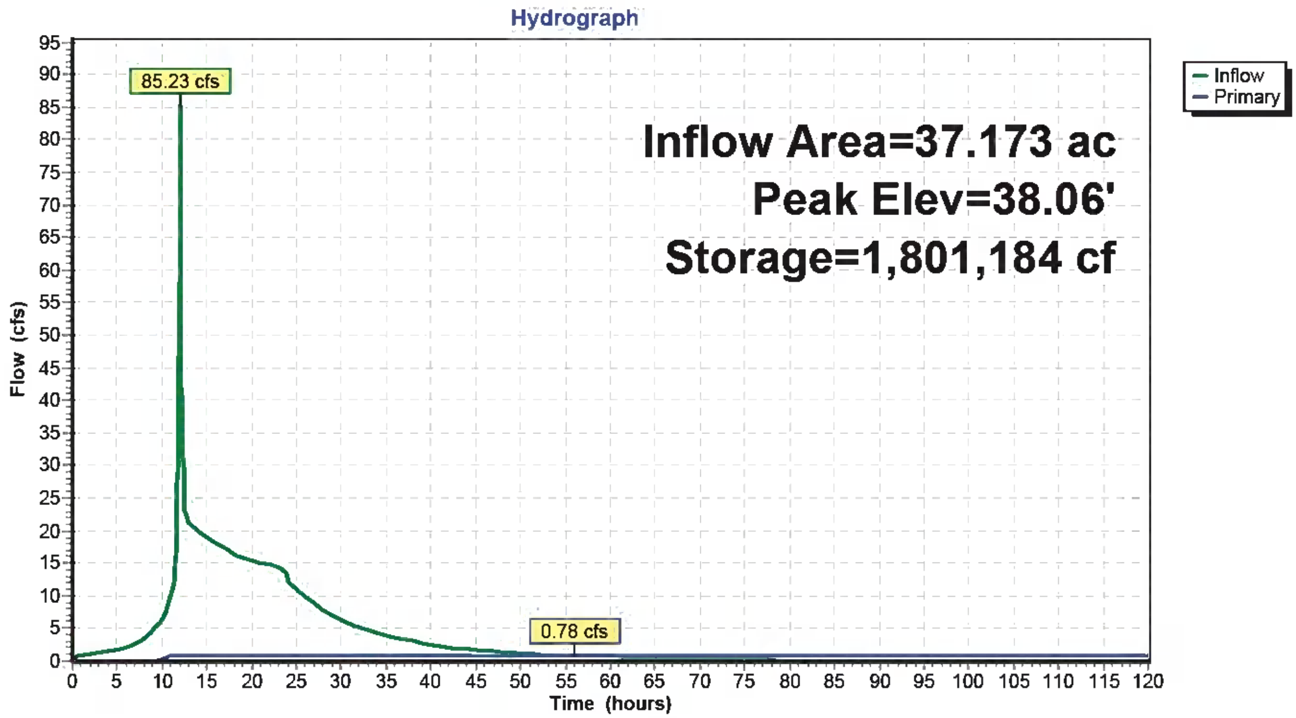
Volume	Invert	Avail.Storage	Storage Description
#1	27.00'	2,564,304 cf	**Custom Stage Data (Conic)** Listed below (Recalc)

Elevation (feet)	Surf.Area (sq-ft)	Inc.Store (cubic-feet)	Cum.Store (cubic-feet)	Wet.Area (sq-ft)
27.00	12,042	0	0	12,042
29.00	66,802	71,471	71,471	66,818
31.00	144,923	206,745	278,216	144,971
33.00	204,519	347,736	625,952	204,641
35.00	232,106	436,334	1,062,286	232,425
37.00	244,194	476,249	1,538,535	244,999
39.00	256,397	500,541	2,039,076	257,707
41.00	268,880	525,228	2,564,304	270,708

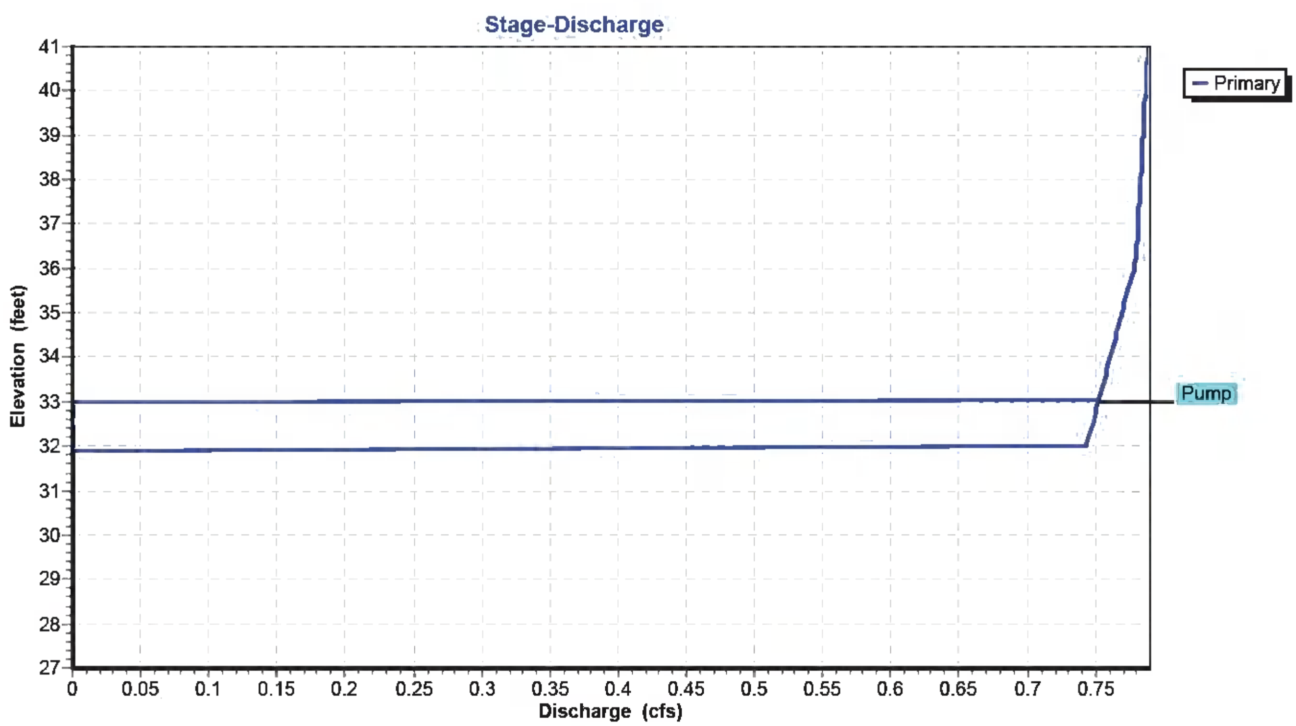
Device	Routing	Invert	Outlet Devices
#1	Primary	33.00'	**Pump** Discharges@41.00' Turns Off@32.00' 6.0" Diam. x 300.0' Long Discharge, Hazen-Williams C= 130 Flow (gpm)= 40.0 100.0 200.0 250.0 300.0 350.0 355.0 Head (feet)= 47.00 40.00 30.00 26.00 20.00 8.00 2.00 -Loss (feet)= 0.06 0.31 1.13 1.71 2.39 3.18 3.27 =Lift (feet)= 46.94 39.69 28.87 24.29 17.61 4.82 -1.27

Primary OutFlow Max=0.78 cfs @ 56.04 hrs HW=38.06' (Free Discharge)
↑—**1=Pump** (Pump Controls 0.78 cfs)

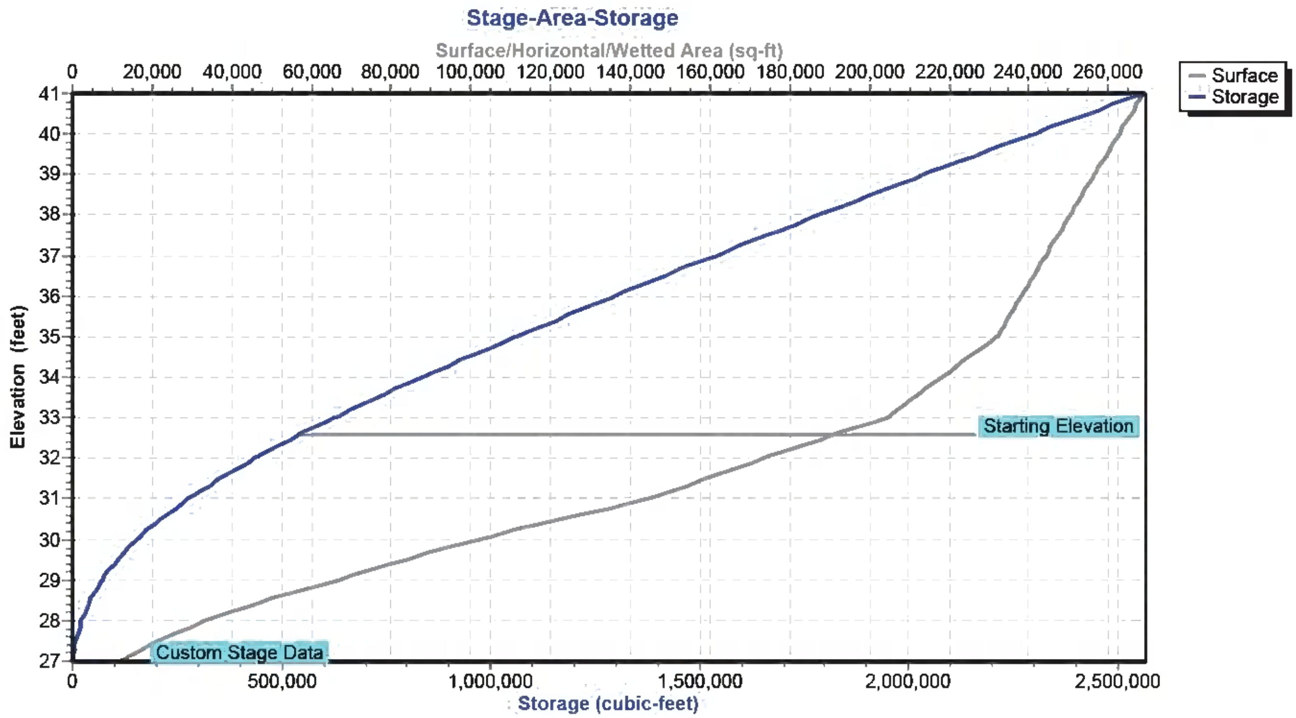
Pond P2: Leachate Pond



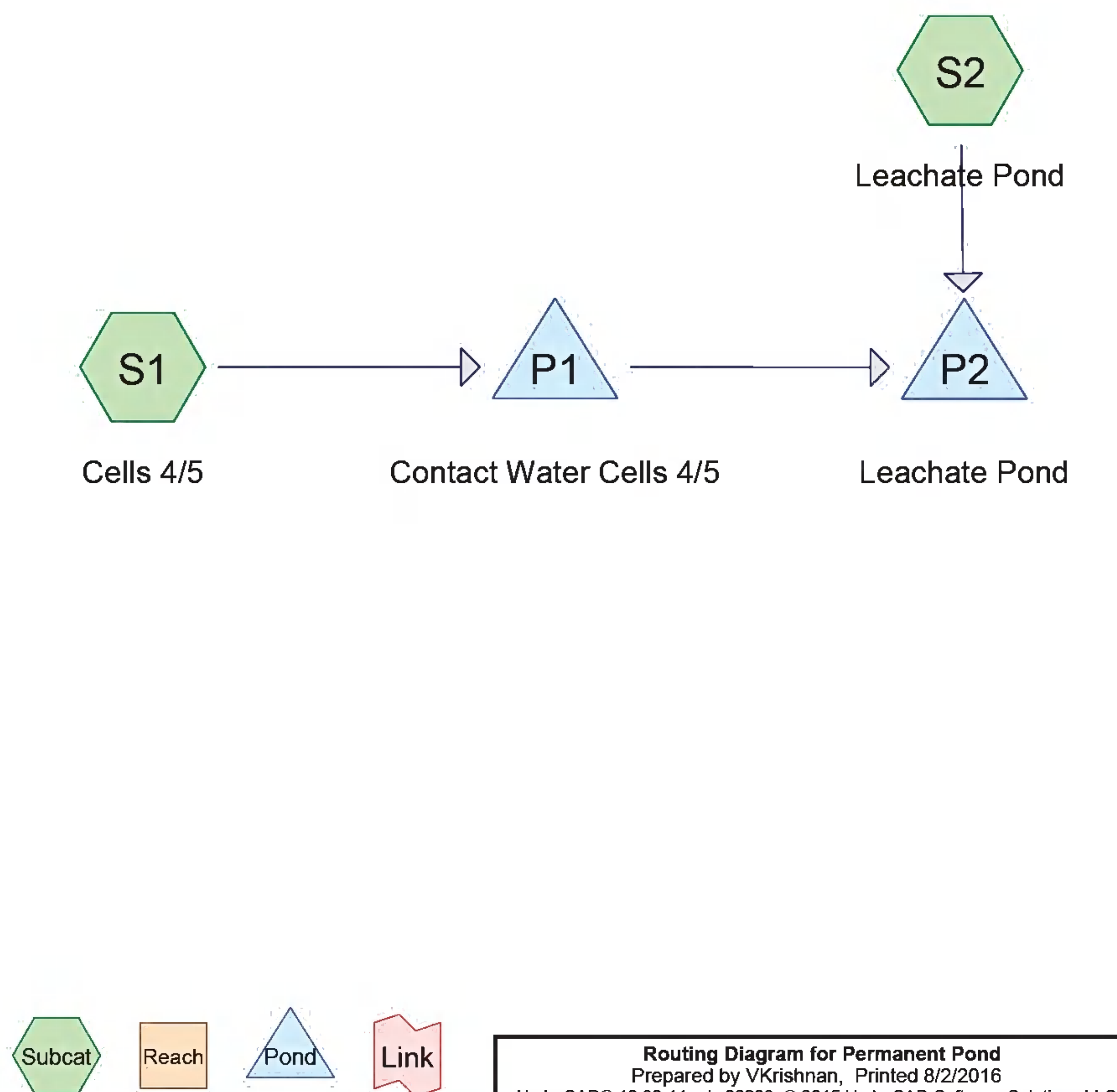
Pond P2: Leachate Pond



Pond P2: Leachate Pond



PERMANENT LEACHATE POND



Routing Diagram for Permanent Pond
Prepared by VKrishnan, Printed 8/2/2016
HydroCAD® 10.00-14 s/n 02233 © 2015 HydroCAD Software Solutions LLC

Area Listing (all nodes)

Area (acres)	CN	Description (subcatchment-numbers)
31.000	97	(S1)
4.080	100	(S2)
35.080	**97**	**TOTAL AREA**

Soil Listing (all nodes)

Area (acres)	Soil Group	Subcatchment Numbers
0.000	HSG A	
0.000	HSG B	
0.000	HSG C	
0.000	HSG D	
35.080	Other	S1, S2
35.080		**TOTAL AREA**

Ground Covers (all nodes)

HSG-A (acres)	HSG-B (acres)	HSG-C (acres)	HSG-D (acres)	Other (acres)	Total (acres)	Ground Cover	Subcatchment Numbers
0.000	0.000	0.000	0.000	35.080	35.080		S1, S2
0.000	**0.000**	**0.000**	**0.000**	**35.080**	**35.080**	**TOTAL AREA**	

Pipe Listing (all nodes)

Line#	Node Number	In-Invert (feet)	Out-Invert (feet)	Length (feet)	Slope (ft/ft)	n	Diam/Width (inches)	Height (inches)	Inside-Fill (inches)
1	P1	36.00	29.50	1,300.0	0.0050	0.009	18.0	0.0	0.0

Time span=0.00-120.00 hrs, dt=0.05 hrs, 2401 points
Runoff by SCS TR-20 method, UH=SCS, Weighted-CN
Reach routing by Dyn-Stor-Ind method - Pond routing by Dyn-Stor-Ind method

Subcatchment S1: Cells 4/5 Runoff Area=31.000 ac 0.00% Impervious Runoff Depth=7.92"
Tc=10.0 min CN=97 Runoff=217.54 cfs 20.460 af

Subcatchment S2: Leachate Pond Runoff Area=177,726 sf 100.00% Impervious Runoff Depth=8.28"
Tc=0.0 min CN=100 Runoff=38.04 cfs 2.815 af

Pond P1: Contact Water Cells 4/5 Peak Elev=55.00' Storage=12.924 af Inflow=217.54 cfs 20.460 af
Outflow=14.19 cfs 20.460 af

Pond P2: Leachate Pond Peak Elev=35.04' Storage=1,341,498 cf Inflow=44.71 cfs 23.275 af
Outflow=0.77 cfs 7.213 af

Total Runoff Area = 35.080 ac Runoff Volume = 23.275 af Average Runoff Depth = 7.96"
88.37% Pervious = 31.000 ac 11.63% Impervious = 4.080 ac

Summary for Subcatchment S1: Cells 4/5

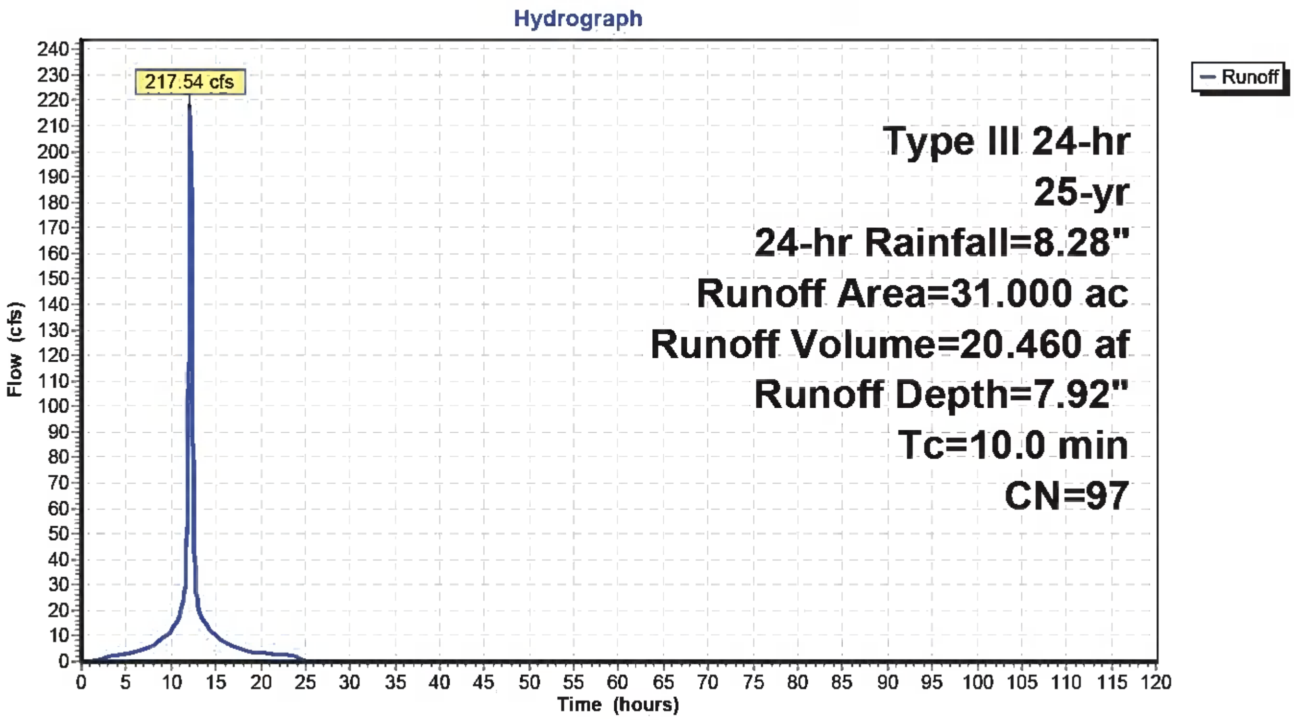
Runoff = 217.54 cfs @ 12.14 hrs, Volume= 20.460 af, Depth= 7.92"

Runoff by SCS TR-20 method, UH=SCS, Weighted-CN, Time Span= 0.00-120.00 hrs, dt= 0.05 hrs
Type III 24-hr 25-yr, 24-hr Rainfall=8.28"

	Area (ac)	CN	Description
*	31.000	97	
	31.000		100.00% Pervious Area

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
10.0					**Direct Entry,**

Subcatchment S1: Cells 4/5



Summary for Subcatchment S2: Leachate Pond

[46] Hint: Tc=0 (Instant runoff peak depends on dt)

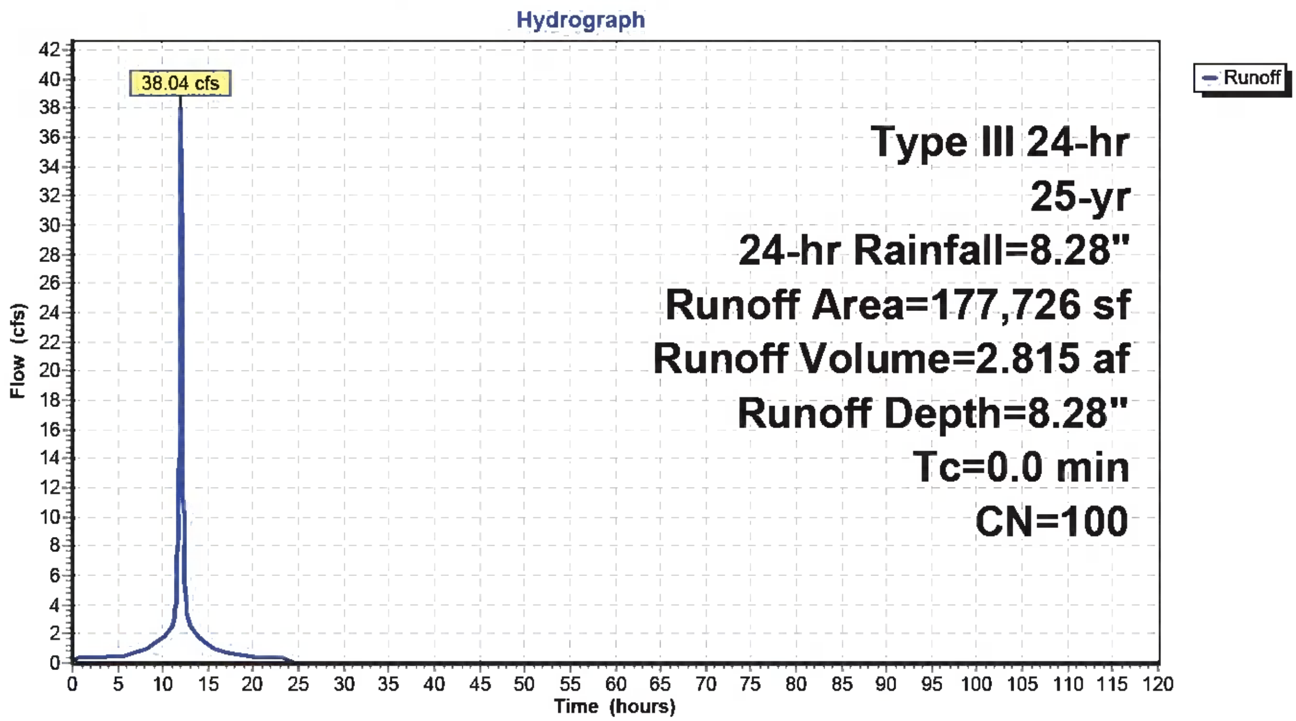
Runoff = 38.04 cfs @ 12.00 hrs, Volume= 2.815 af, Depth= 8.28"

Runoff by SCS TR-20 method, UH=SCS, Weighted-CN, Time Span= 0.00-120.00 hrs, dt= 0.05 hrs
Type III 24-hr 25-yr, 24-hr Rainfall=8.28"

	Area (sf)	CN	Description
*	177,726	100	
	177,726		100.00% Impervious Area

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
0.0					**Direct Entry,**

Subcatchment S2: Leachate Pond



Summary for Pond P1: Contact Water Cells 4/5

Inflow Area = 31.000 ac, 0.00% Impervious, Inflow Depth = 7.92" for 25-yr, 24-hr event
Inflow = 217.54 cfs @ 12.14 hrs, Volume= 20.460 af
Outflow = 14.19 cfs @ 13.91 hrs, Volume= 20.460 af, Atten= 93%, Lag= 106.2 min
Primary = 14.19 cfs @ 13.91 hrs, Volume= 20.460 af

Routing by Dyn-Stor-Ind method, Time Span= 0.00-120.00 hrs, dt= 0.05 hrs
Peak Elev= 55.00' @ 13.91 hrs Surf.Area= 29.989 ac Storage= 12.924 af

Plug-Flow detention time= 687.1 min calculated for 20.452 af (100% of inflow)
Center-of-Mass det. time= 688.1 min (1,438.4 - 750.3)

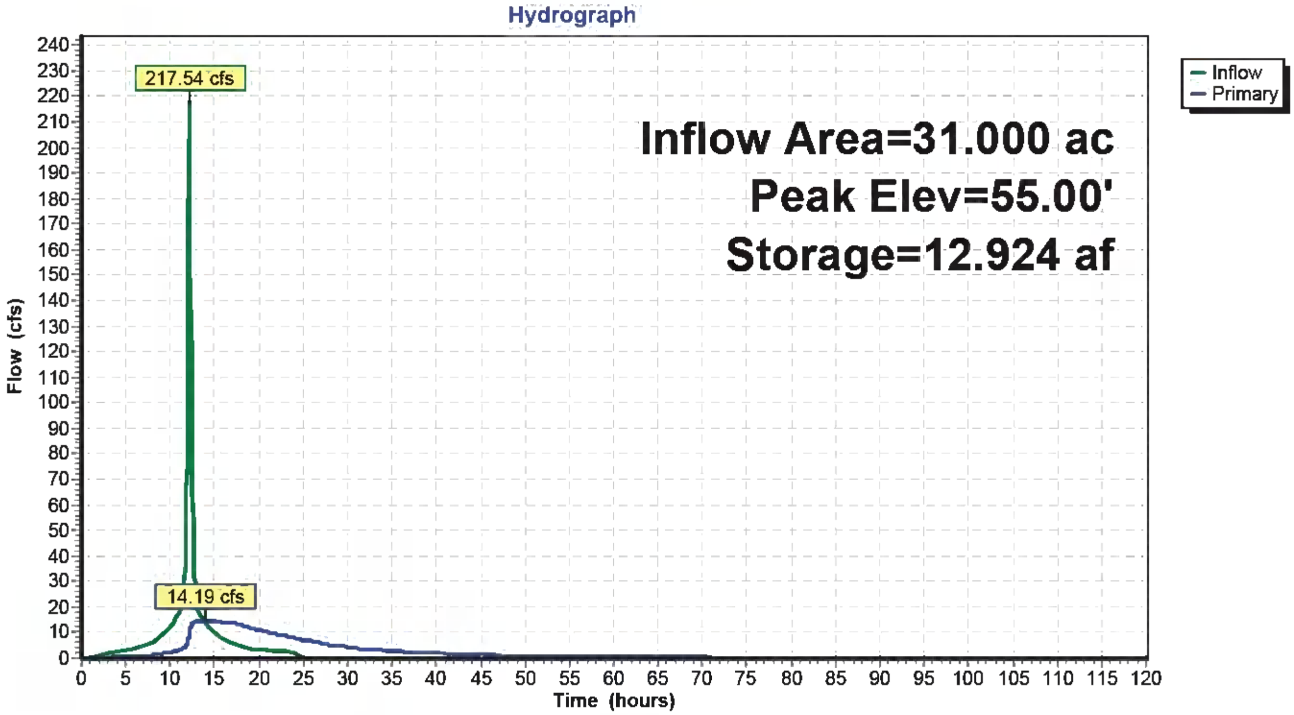
Volume	Invert	Avail.Storage	Storage Description
#1	51.00'	74.999 af	**Custom Stage Data (Conic)**Listed below (Recalc)

Elevation (feet)	Surf.Area (acres)	Inc.Store (acre-feet)	Cum.Store (acre-feet)	Wet.Area (acres)
51.00	0.001	0.000	0.000	0.001
51.20	2.790	0.190	0.190	2.790
54.90	3.340	11.325	11.515	3.351
55.00	31.000	1.484	12.999	31.011
57.00	31.000	62.000	74.999	31.200

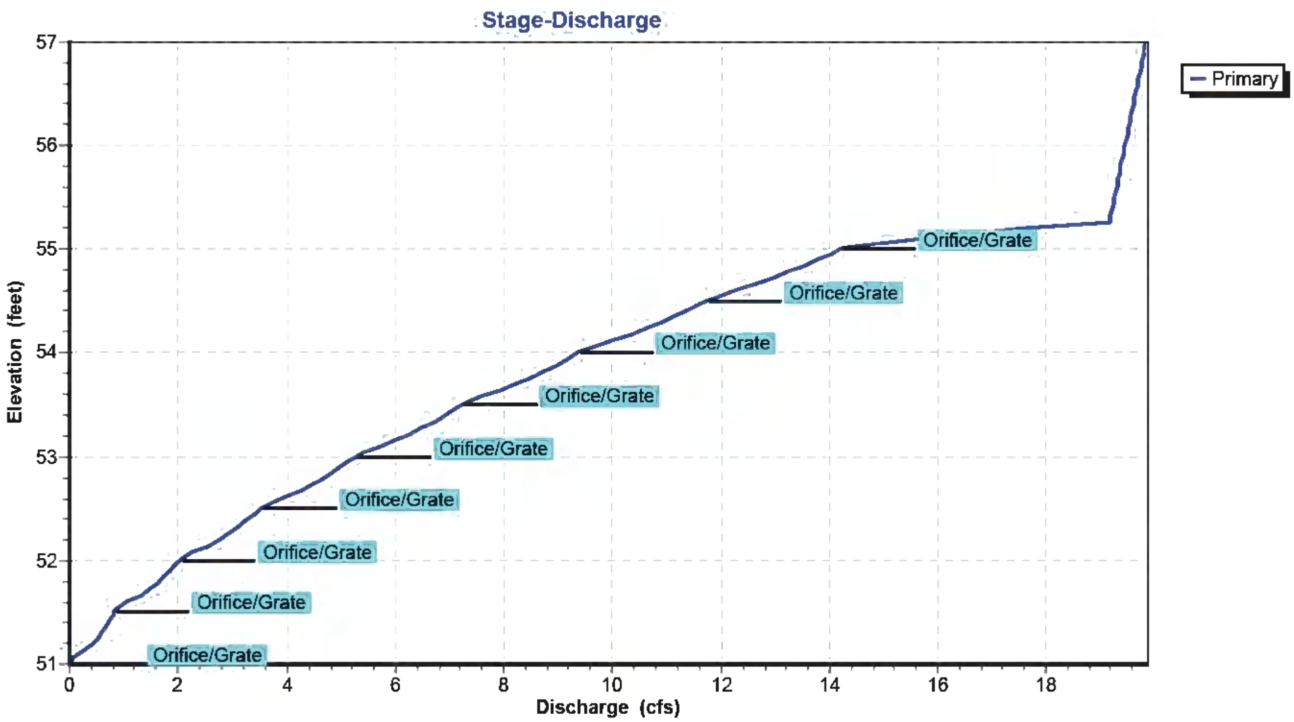
Device	Routing	Invert	Outlet Devices
#1	Primary	36.00'	**18.0" Round Culvert** L= 1,300.0' CPP, projecting, no headwall, Ke= 0.900 Inlet / Outlet Invert= 36.00' / 29.50' S= 0.0050 '/' Cc= 0.900 n= 0.009, Flow Area= 1.77 sf
#2	Device 1	51.00'	**2.0" Vert. Orifice/Grate X 12.00** C= 0.600
#3	Device 1	51.50'	**2.0" Vert. Orifice/Grate X 12.00** C= 0.600
#4	Device 1	52.00'	**2.0" Vert. Orifice/Grate X 12.00** C= 0.600
#5	Device 1	52.50'	**2.0" Vert. Orifice/Grate X 12.00** C= 0.600
#6	Device 1	53.00'	**2.0" Vert. Orifice/Grate X 12.00** C= 0.600
#7	Device 1	53.50'	**2.0" Vert. Orifice/Grate X 12.00** C= 0.600
#8	Device 1	54.00'	**2.0" Vert. Orifice/Grate X 12.00** C= 0.600
#9	Device 1	54.50'	**2.0" Vert. Orifice/Grate X 12.00** C= 0.600
#10	Device 1	55.00'	**36.0" Horiz. Orifice/Grate** C= 0.600 Limited to weir flow at low heads

Primary OutFlow Max=14.19 cfs @ 13.91 hrs HW=55.00' TW=30.60' (Dynamic Tailwater)
└─**1=Culvert** (Passes 14.19 cfs of 19.05 cfs potential flow)
  ├─**2=Orifice/Grate** (Orifice Controls 2.49 cfs @ 9.53 fps)
  ├─**3=Orifice/Grate** (Orifice Controls 2.33 cfs @ 8.90 fps)
  ├─**4=Orifice/Grate** (Orifice Controls 2.15 cfs @ 8.22 fps)
  ├─**5=Orifice/Grate** (Orifice Controls 1.96 cfs @ 7.48 fps)
  ├─**6=Orifice/Grate** (Orifice Controls 1.74 cfs @ 6.66 fps)
  ├─**7=Orifice/Grate** (Orifice Controls 1.50 cfs @ 5.73 fps)
  ├─**8=Orifice/Grate** (Orifice Controls 1.21 cfs @ 4.60 fps)
  ├─**9=Orifice/Grate** (Orifice Controls 0.81 cfs @ 3.10 fps)
  └─**10=Orifice/Grate** (Controls 0.00 cfs)

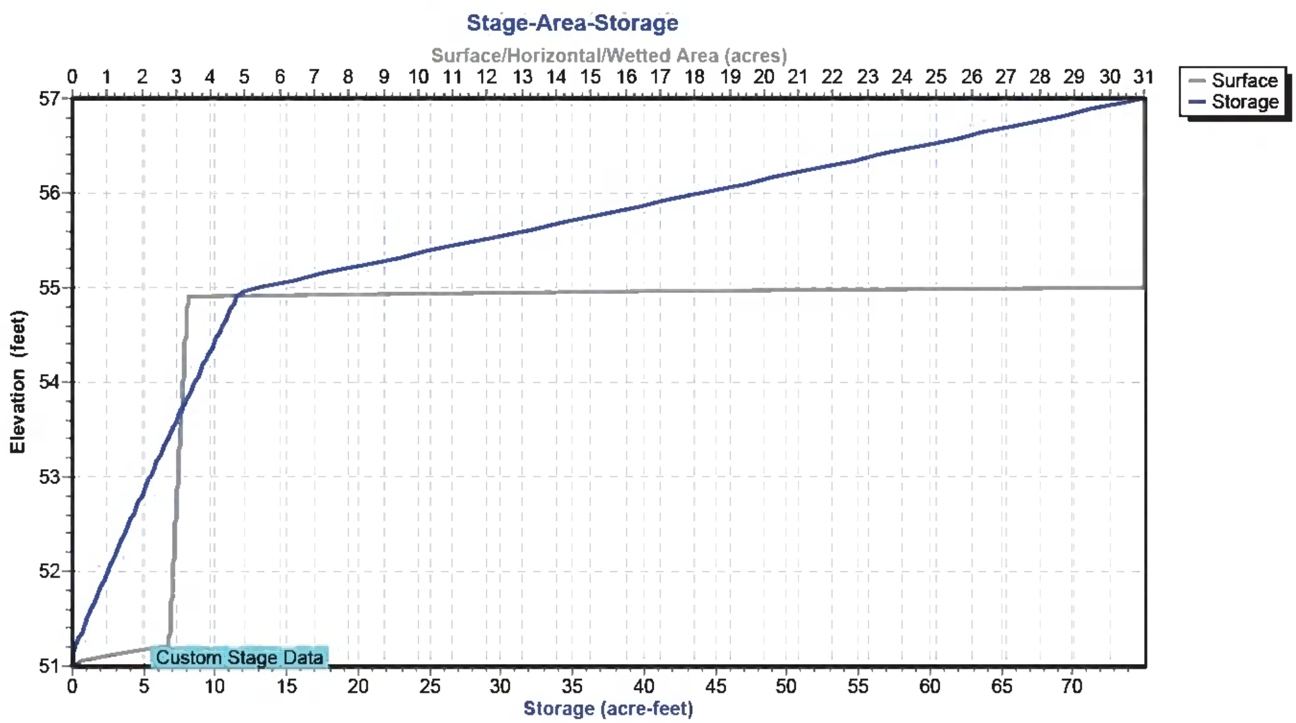
Pond P1: Contact Water Cells 4/5



Pond P1: Contact Water Cells 4/5



Pond P1: Contact Water Cells 4/5



Summary for Pond P2: Leachate Pond

Inflow Area = 35.080 ac, 11.63% Impervious, Inflow Depth = 7.96" for 25-yr, 24-hr event
Inflow = 44.71 cfs @ 12.00 hrs, Volume= 23.275 af
Outflow = 0.77 cfs @ 52.74 hrs, Volume= 7.213 af, Atten= 98%, Lag= 2,444.4 min
Primary = 0.77 cfs @ 52.74 hrs, Volume= 7.213 af

Routing by Dyn-Stor-Ind method, Time Span= 0.00-120.00 hrs, dt= 0.05 hrs
Starting Elev= 28.89' Surf.Area= 123,068 sf Storage= 495,790 cf
Peak Elev= 35.04' @ 52.74 hrs Surf.Area= 152,290 sf Storage= 1,341,498 cf (845,709 cf above start)

Plug-Flow detention time= (not calculated: initial storage exceeds outflow)
Center-of-Mass det. time= 2,429.3 min (3,780.8 - 1,351.5)

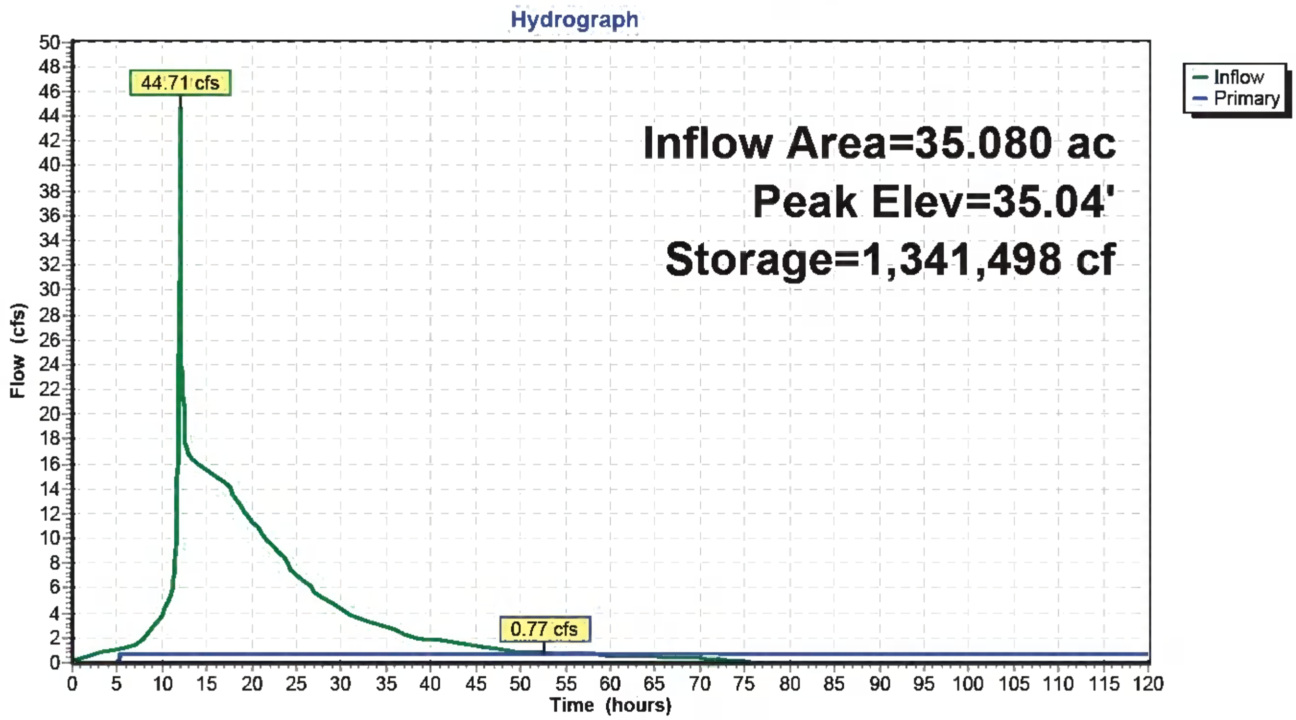
Volume	Invert	Avail.Storage	Storage Description
#1	24.00'	2,158,865 cf	**Custom Stage Data (Conic)** Listed below (Recalc)

Elevation (feet)	Surf.Area (sq-ft)	Inc.Store (cubic-feet)	Cum.Store (cubic-feet)	Wet.Area (sq-ft)
24.00	341	0	0	341
24.50	57,719	10,416	10,416	57,719
25.00	105,931	40,307	50,723	105,934
25.50	108,076	53,501	104,224	108,156
26.00	110,238	54,578	158,802	110,396
26.50	112,416	55,663	214,464	112,653
27.00	114,612	56,756	271,221	114,929
27.50	116,826	57,859	329,079	117,223
28.00	119,056	58,970	388,049	119,535
28.50	121,303	60,089	448,138	121,864
29.00	123,568	61,217	509,355	124,213
29.50	125,850	62,354	571,708	126,579
30.00	128,149	63,499	635,207	128,963
30.50	130,465	64,653	699,860	131,365
31.00	132,798	65,815	765,675	133,785
31.50	135,148	66,986	832,660	136,223
32.00	137,516	68,165	900,825	138,680
32.50	139,900	69,353	970,178	141,154
33.00	142,302	70,550	1,040,728	143,646
33.50	144,721	71,755	1,112,483	146,157
34.00	147,157	72,969	1,185,452	148,685
34.50	149,610	74,191	1,259,643	151,231
35.00	152,080	75,422	1,335,064	153,795
35.50	154,568	76,661	1,411,725	156,378
36.00	157,072	77,909	1,489,635	158,978
36.50	159,594	79,166	1,568,800	161,597
37.00	162,133	80,431	1,649,231	164,234
37.50	164,689	81,705	1,730,936	166,888
38.00	167,262	82,987	1,813,923	169,561
38.50	169,853	84,278	1,898,201	172,252
39.00	172,460	85,577	1,983,778	174,960
39.50	175,085	86,885	2,070,663	177,687
40.00	177,726	88,202	2,158,865	180,431

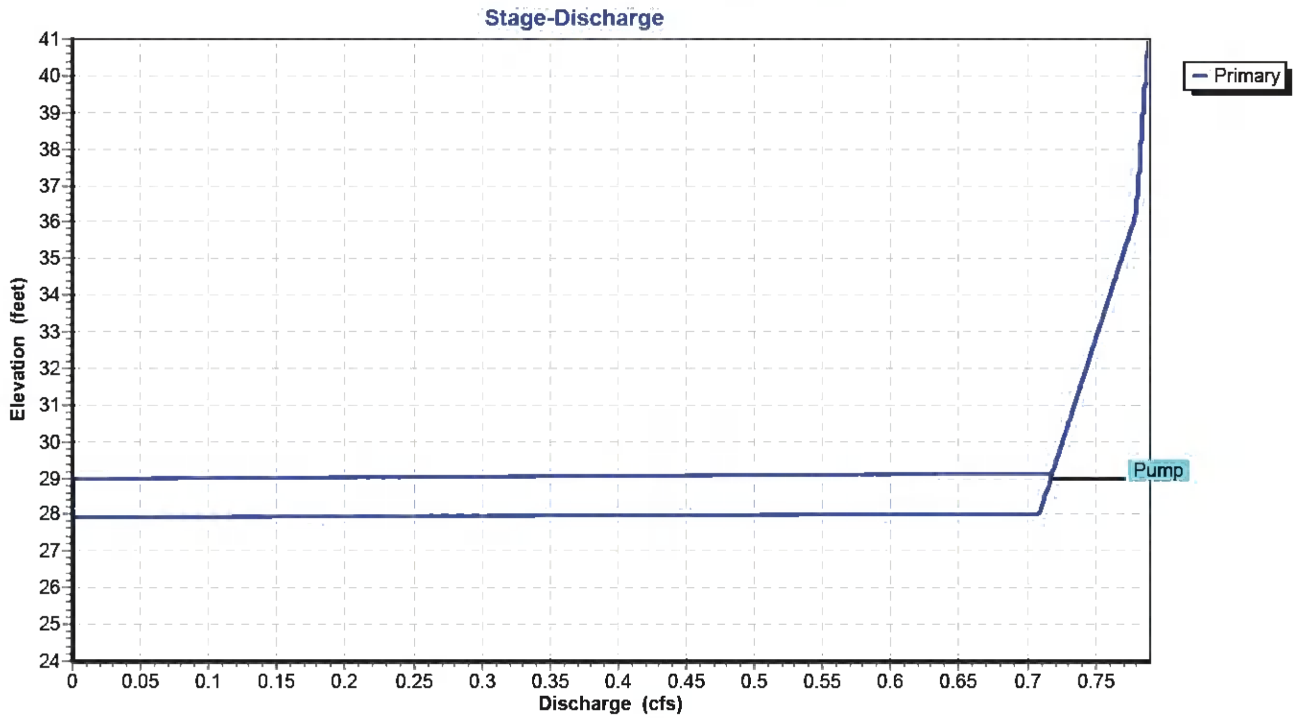
Device	Routin	Invert	Outlet Devices
#1	Primary	29.00'	**Pump** Discharges@41.00' Turns Off@28.00' 6.0" Diam. x 300.0' Long Discharge, Hazen-Williams C= 130 Flow (gpm)= 40.0 100.0 200.0 250.0 300.0 350.0 355.0 Head (feet)= 47.00 40.00 30.00 26.00 20.00 8.00 2.00 -Loss (feet)= 0.06 0.31 1.13 1.71 2.39 3.18 3.27 =Lift (feet)= 46.94 39.69 28.87 24.29 17.61 4.82 -1.27

Primary OutFlow Max=0.77 cfs @ 52.74 hrs HW=35.04' (Free Discharge)
└─**1=Pump** (Pump Controls 0.77 cfs)

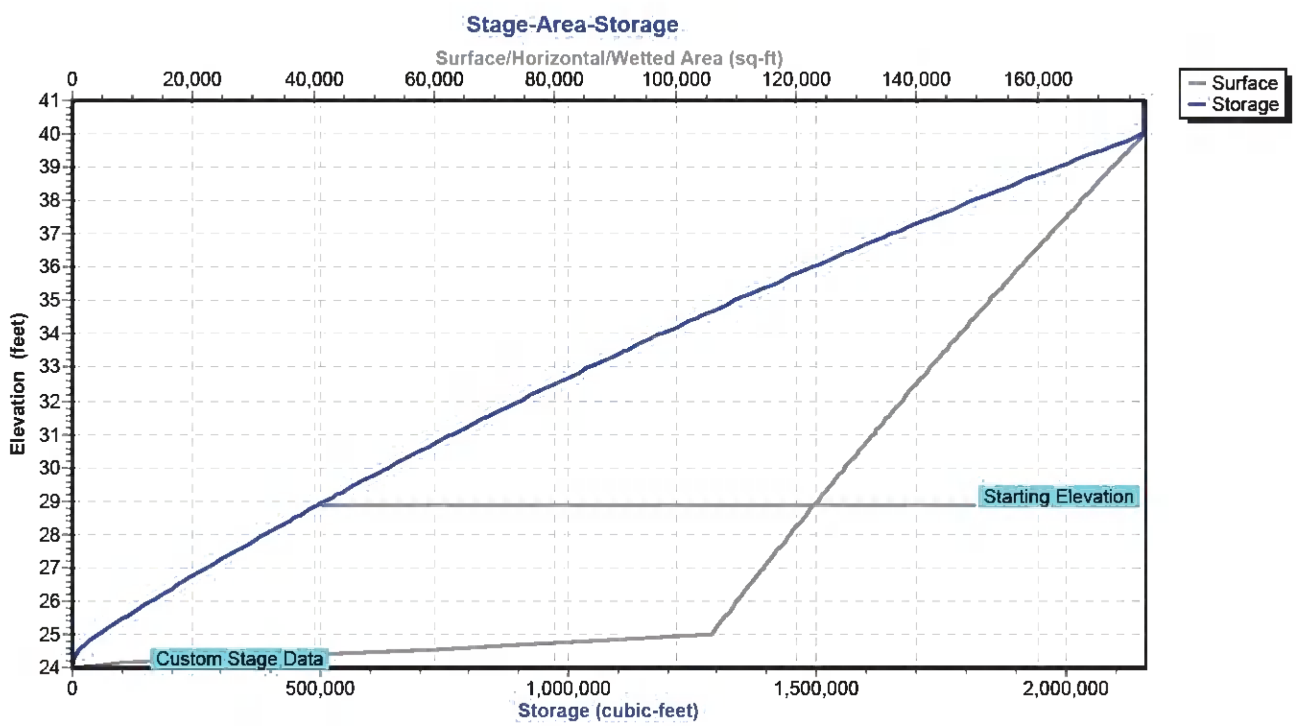
Pond P2: Leachate Pond



Pond P2: Leachate Pond



Pond P2: Leachate Pond



Time span=0.00-120.00 hrs, dt=0.05 hrs, 2401 points
Runoff by SCS TR-20 method, UH=SCS, Weighted-CN
Reach routing by Dyn-Stor-Ind method - Pond routing by Dyn-Stor-Ind method

Subcatchment S1: Cells 4/5 Runoff Area=31.000 ac 0.00% Impervious Runoff Depth=10.54"
Tc=10.0 min CN=97 Runoff=286.98 cfs 27.222 af

Subcatchment S2: Leachate Pond Runoff Area=177,726 sf 100.00% Impervious Runoff Depth=10.90"
Tc=0.0 min CN=100 Runoff=50.08 cfs 3.706 af

Pond P1: Contact Water Cells 4/5 Peak Elev=55.15' Storage=17.523 af Inflow=286.98 cfs 27.222 af
Outflow=16.49 cfs 27.222 af

Pond P2: Leachate Pond Peak Elev=37.07' Storage=1,660,229 cf Inflow=59.55 cfs 30.928 af
Outflow=0.78 cfs 7.378 af

Total Runoff Area = 35.080 ac Runoff Volume = 30.928 af Average Runoff Depth = 10.58"
88.37% Pervious = 31.000 ac 11.63% Impervious = 4.080 ac

Summary for Subcatchment S1: Cells 4/5

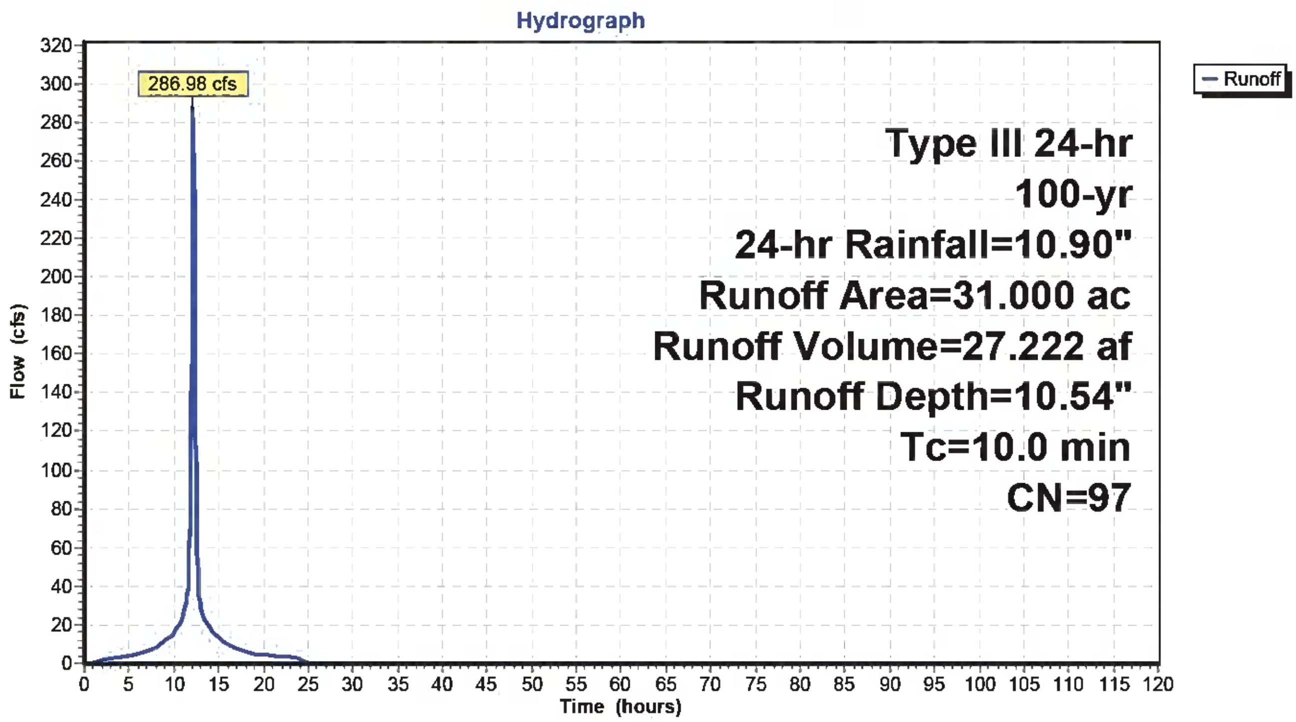
Runoff = 286.98 cfs @ 12.14 hrs, Volume= 27.222 af, Depth=10.54"

Runoff by SCS TR-20 method, UH=SCS, Weighted-CN, Time Span= 0.00-120.00 hrs, dt= 0.05 hrs
Type III 24-hr 100-yr, 24-hr Rainfall=10.90"

	Area (ac)	CN	Description
*	31.000	97	
	31.000		100.00% Pervious Area

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
10.0					**Direct Entry,**

Subcatchment S1: Cells 4/5



Summary for Subcatchment S2: Leachate Pond

[46] Hint: Tc=0 (Instant runoff peak depends on dt)

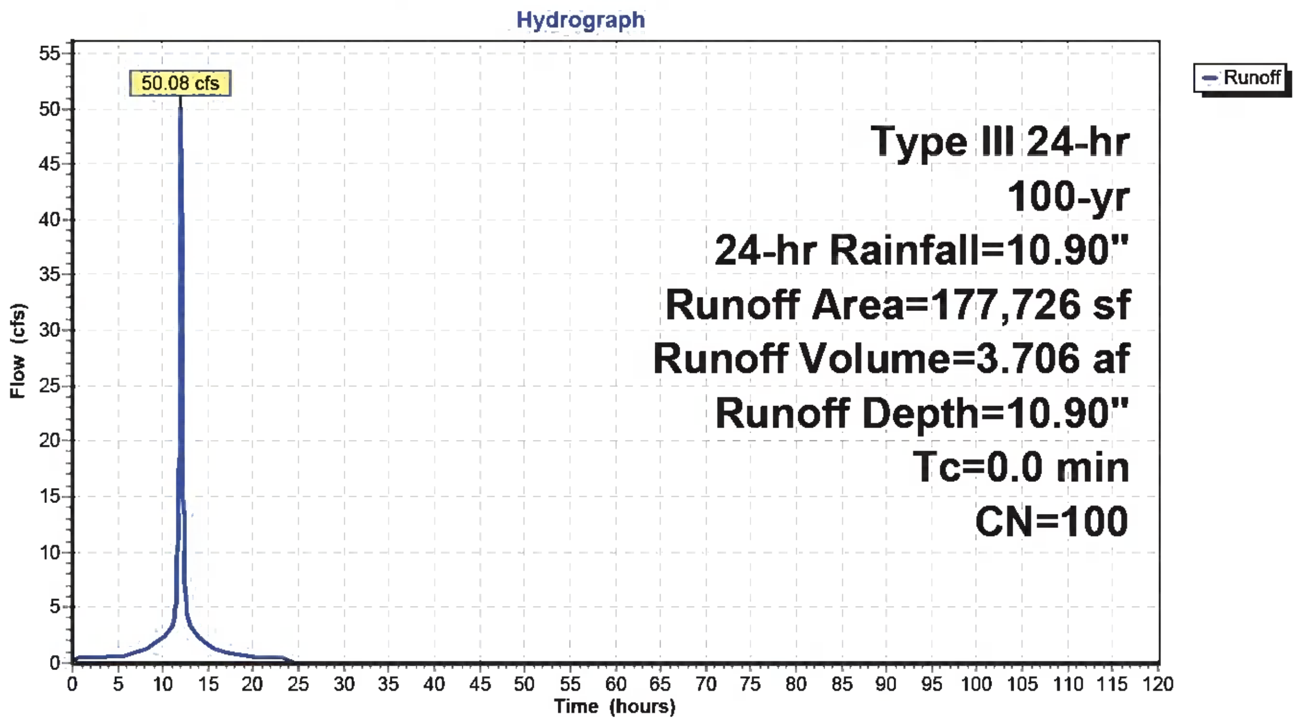
Runoff = 50.08 cfs @ 12.00 hrs, Volume= 3.706 af, Depth=10.90"

Runoff by SCS TR-20 method, UH=SCS, Weighted-CN, Time Span= 0.00-120.00 hrs, dt= 0.05 hrs
Type III 24-hr 100-yr, 24-hr Rainfall=10.90"

	Area (sf)	CN	Description
*	177,726	100	
	177,726		100.00% Impervious Area

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
0.0					**Direct Entry,**

Subcatchment S2: Leachate Pond



Summary for Pond P1: Contact Water Cells 4/5

Inflow Area = 31.000 ac, 0.00% Impervious, Inflow Depth = 10.54" for 100-yr, 24-hr event
Inflow = 286.98 cfs @ 12.14 hrs, Volume= 27.222 af
Outflow = 16.49 cfs @ 14.22 hrs, Volume= 27.222 af, Atten= 94%, Lag= 124.7 min
Primary = 16.49 cfs @ 14.22 hrs, Volume= 27.222 af

Routing by Dyn-Stor-Ind method, Time Span= 0.00-120.00 hrs, dt= 0.05 hrs
Peak Elev= 55.15' @ 14.22 hrs Surf.Area= 31.000 ac Storage= 17.523 af

Plug-Flow detention time= 719.7 min calculated for 27.210 af (100% of inflow)
Center-of-Mass det. time= 720.8 min (1,467.1 - 746.3)

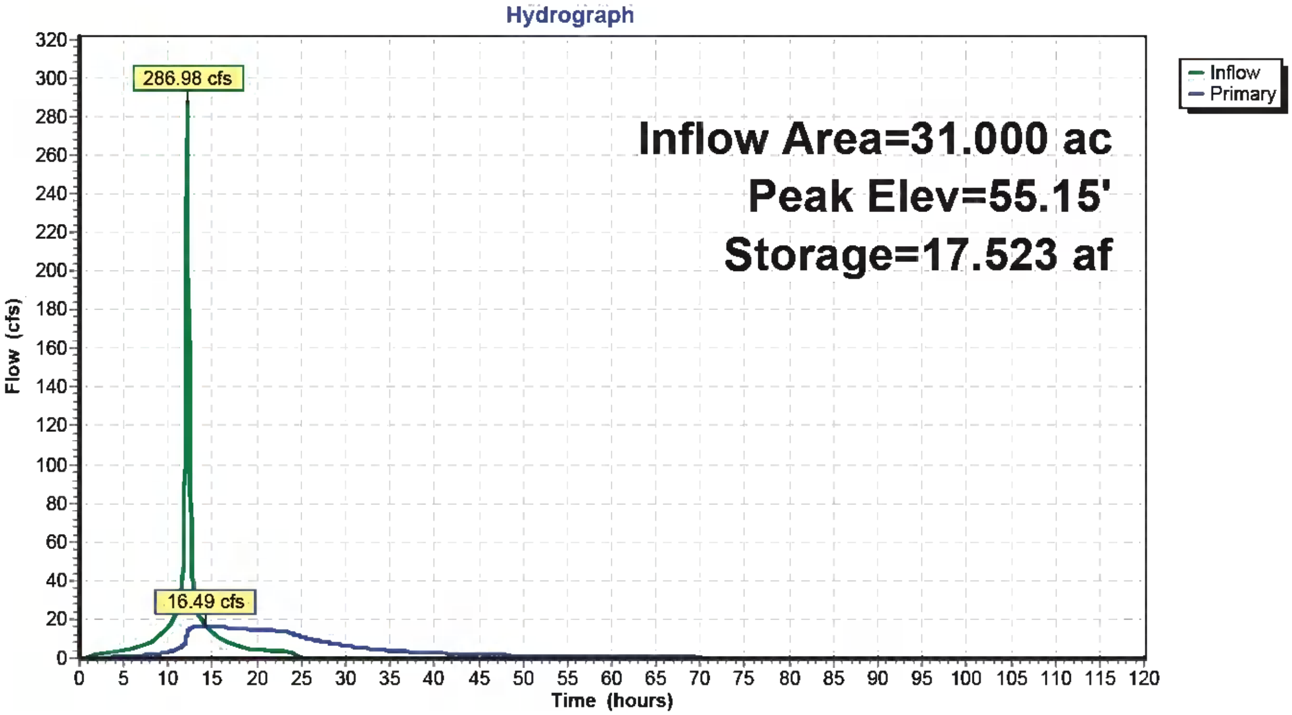
Volume	Invert	Avail.Storage	Storage Description
#1	51.00'	74.999 af	**Custom Stage Data (Conic)**Listed below (Recalc)

Elevation (feet)	Surf.Area (acres)	Inc.Store (acre-feet)	Cum.Store (acre-feet)	Wet.Area (acres)
51.00	0.001	0.000	0.000	0.001
51.20	2.790	0.190	0.190	2.790
54.90	3.340	11.325	11.515	3.351
55.00	31.000	1.484	12.999	31.011
57.00	31.000	62.000	74.999	31.200

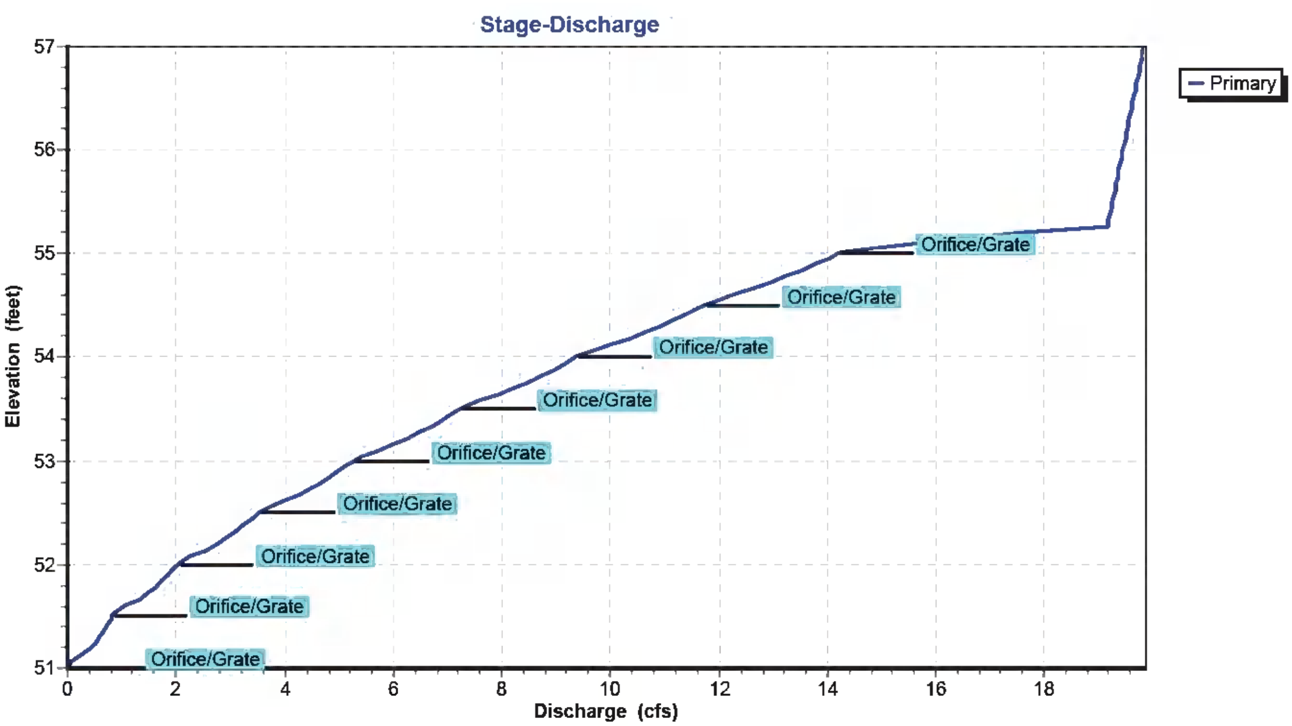
Device	Routing	Invert	Outlet Devices
#1	Primary	36.00'	**18.0" Round Culvert** L= 1,300.0' CPP, projecting, no headwall, Ke= 0.900 Inlet / Outlet Invert= 36.00' / 29.50' S= 0.0050 '/' Cc= 0.900 n= 0.009, Flow Area= 1.77 sf
#2	Device 1	51.00'	**2.0" Vert. Orifice/Grate X 12.00** C= 0.600
#3	Device 1	51.50'	**2.0" Vert. Orifice/Grate X 12.00** C= 0.600
#4	Device 1	52.00'	**2.0" Vert. Orifice/Grate X 12.00** C= 0.600
#5	Device 1	52.50'	**2.0" Vert. Orifice/Grate X 12.00** C= 0.600
#6	Device 1	53.00'	**2.0" Vert. Orifice/Grate X 12.00** C= 0.600
#7	Device 1	53.50'	**2.0" Vert. Orifice/Grate X 12.00** C= 0.600
#8	Device 1	54.00'	**2.0" Vert. Orifice/Grate X 12.00** C= 0.600
#9	Device 1	54.50'	**2.0" Vert. Orifice/Grate X 12.00** C= 0.600
#10	Device 1	55.00'	**36.0" Horiz. Orifice/Grate** C= 0.600 Limited to weir flow at low heads

Primary OutFlow Max=16.49 cfs @ 14.22 hrs HW=55.15' TW=31.24' (Dynamic Tailwater)
└─**1=Culvert** (Passes 16.49 cfs of 19.02 cfs potential flow)
  ├─**2=Orifice/Grate** (Orifice Controls 2.54 cfs @ 9.71 fps)
  ├─**3=Orifice/Grate** (Orifice Controls 2.38 cfs @ 9.09 fps)
  ├─**4=Orifice/Grate** (Orifice Controls 2.21 cfs @ 8.43 fps)
  ├─**5=Orifice/Grate** (Orifice Controls 2.02 cfs @ 7.71 fps)
  ├─**6=Orifice/Grate** (Orifice Controls 1.81 cfs @ 6.92 fps)
  ├─**7=Orifice/Grate** (Orifice Controls 1.58 cfs @ 6.02 fps)
  ├─**8=Orifice/Grate** (Orifice Controls 1.30 cfs @ 4.96 fps)
  ├─**9=Orifice/Grate** (Orifice Controls 0.95 cfs @ 3.61 fps)
  └─**10=Orifice/Grate** (Weir Controls 1.72 cfs @ 1.25 fps)

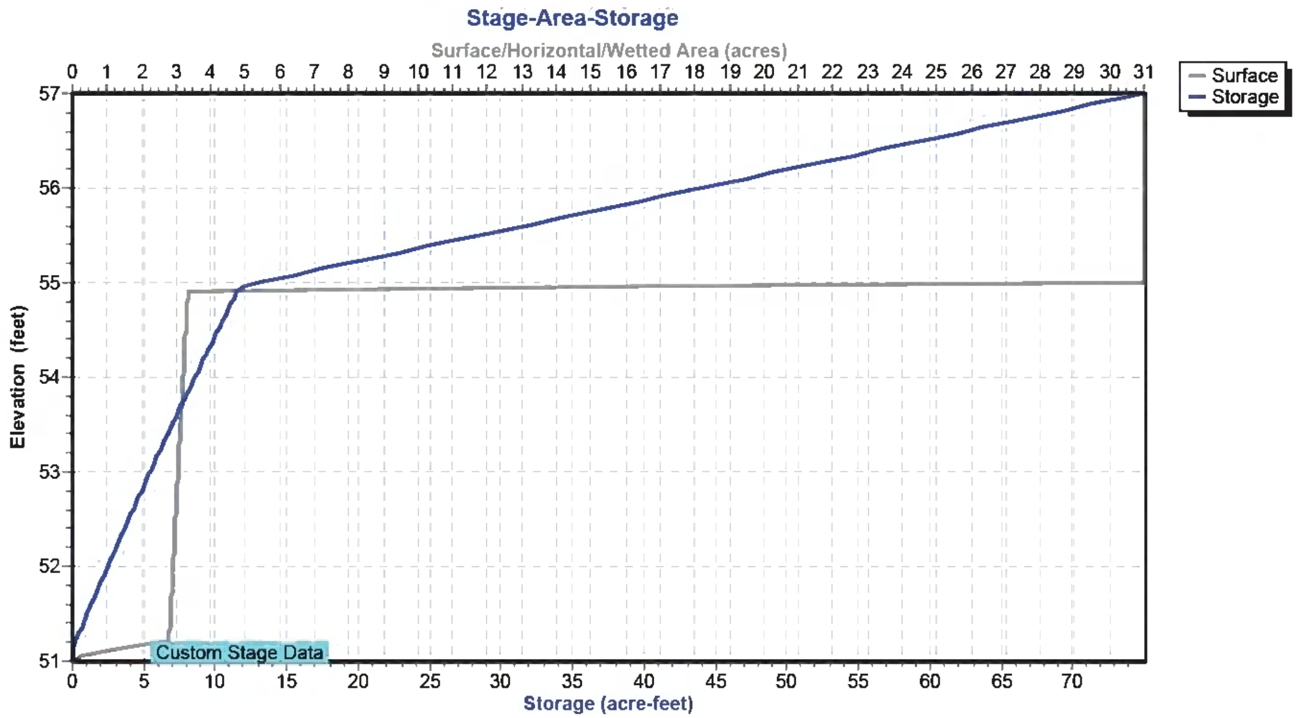
Pond P1: Contact Water Cells 4/5



Pond P1: Contact Water Cells 4/5



Pond P1: Contact Water Cells 4/5



Summary for Pond P2: Leachate Pond

Inflow Area = 35.080 ac, 11.63% Impervious, Inflow Depth = 10.58" for 100-yr, 24-hr event
Inflow = 59.55 cfs @ 12.00 hrs, Volume= 30.928 af
Outflow = 0.78 cfs @ 56.12 hrs, Volume= 7.378 af, Atten= 99%, Lag= 2,647.1 min
Primary = 0.78 cfs @ 56.12 hrs, Volume= 7.378 af

Routing by Dyn-Stor-Ind method, Time Span= 0.00-120.00 hrs, dt= 0.05 hrs
Starting Elev= 28.89' Surf.Area= 123,068 sf Storage= 495,790 cf
Peak Elev= 37.07' @ 56.12 hrs Surf.Area= 162,478 sf Storage= 1,660,229 cf (1,164,440 cf above start)

Plug-Flow detention time= (not calculated: initial storage exceeds outflow)
Center-of-Mass det. time= 2,389.0 min (3,766.5 - 1,377.5)

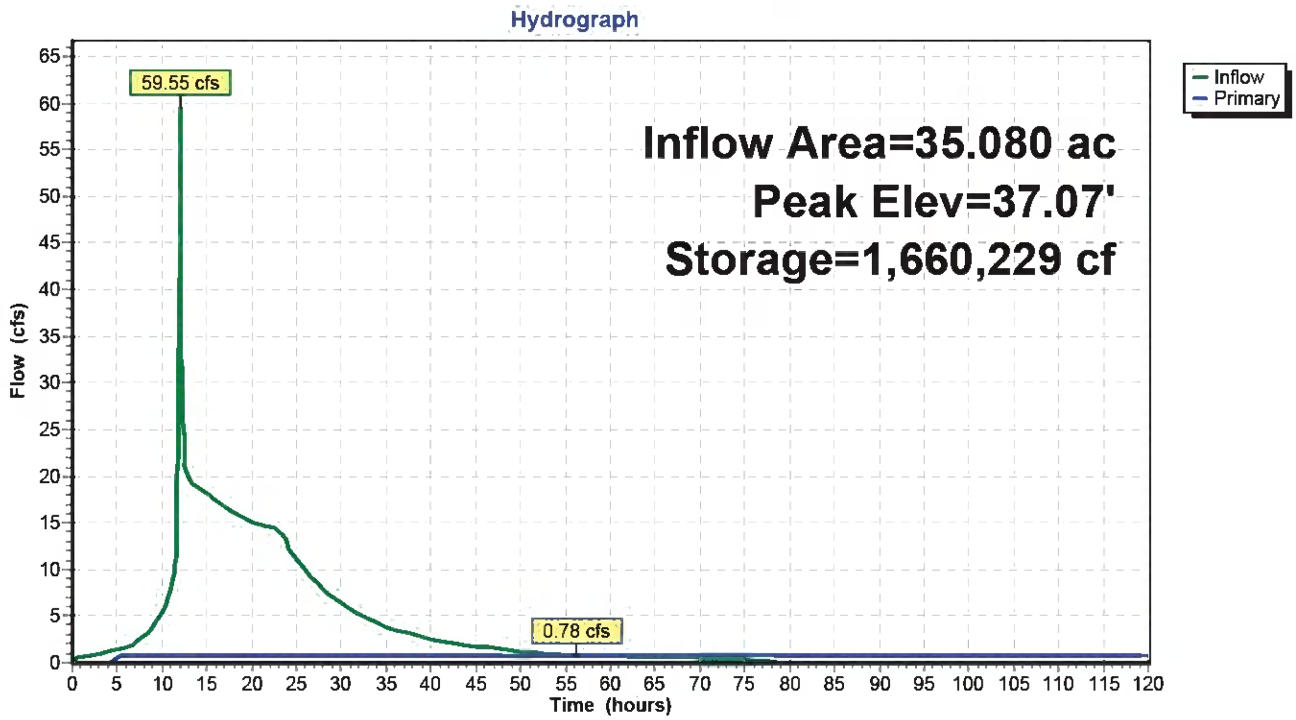
Volume	Invert	Avail.Storage	Storage Description
#1	24.00'	2,158,865 cf	**Custom Stage Data (Conic)**Listed below (Recalc)

Elevation (feet)	Surf.Area (sq-ft)	Inc.Store (cubic-feet)	Cum.Store (cubic-feet)	Wet.Area (sq-ft)
24.00	341	0	0	341
24.50	57,719	10,416	10,416	57,719
25.00	105,931	40,307	50,723	105,934
25.50	108,076	53,501	104,224	108,156
26.00	110,238	54,578	158,802	110,396
26.50	112,416	55,663	214,464	112,653
27.00	114,612	56,756	271,221	114,929
27.50	116,826	57,859	329,079	117,223
28.00	119,056	58,970	388,049	119,535
28.50	121,303	60,089	448,138	121,864
29.00	123,568	61,217	509,355	124,213
29.50	125,850	62,354	571,708	126,579
30.00	128,149	63,499	635,207	128,963
30.50	130,465	64,653	699,860	131,365
31.00	132,798	65,815	765,675	133,785
31.50	135,148	66,986	832,660	136,223
32.00	137,516	68,165	900,825	138,680
32.50	139,900	69,353	970,178	141,154
33.00	142,302	70,550	1,040,728	143,646
33.50	144,721	71,755	1,112,483	146,157
34.00	147,157	72,969	1,185,452	148,685
34.50	149,610	74,191	1,259,643	151,231
35.00	152,080	75,422	1,335,064	153,795
35.50	154,568	76,661	1,411,725	156,378
36.00	157,072	77,909	1,489,635	158,978
36.50	159,594	79,166	1,568,800	161,597
37.00	162,133	80,431	1,649,231	164,234
37.50	164,689	81,705	1,730,936	166,888
38.00	167,262	82,987	1,813,923	169,561
38.50	169,853	84,278	1,898,201	172,252
39.00	172,460	85,577	1,983,778	174,960
39.50	175,085	86,885	2,070,663	177,687
40.00	177,726	88,202	2,158,865	180,431

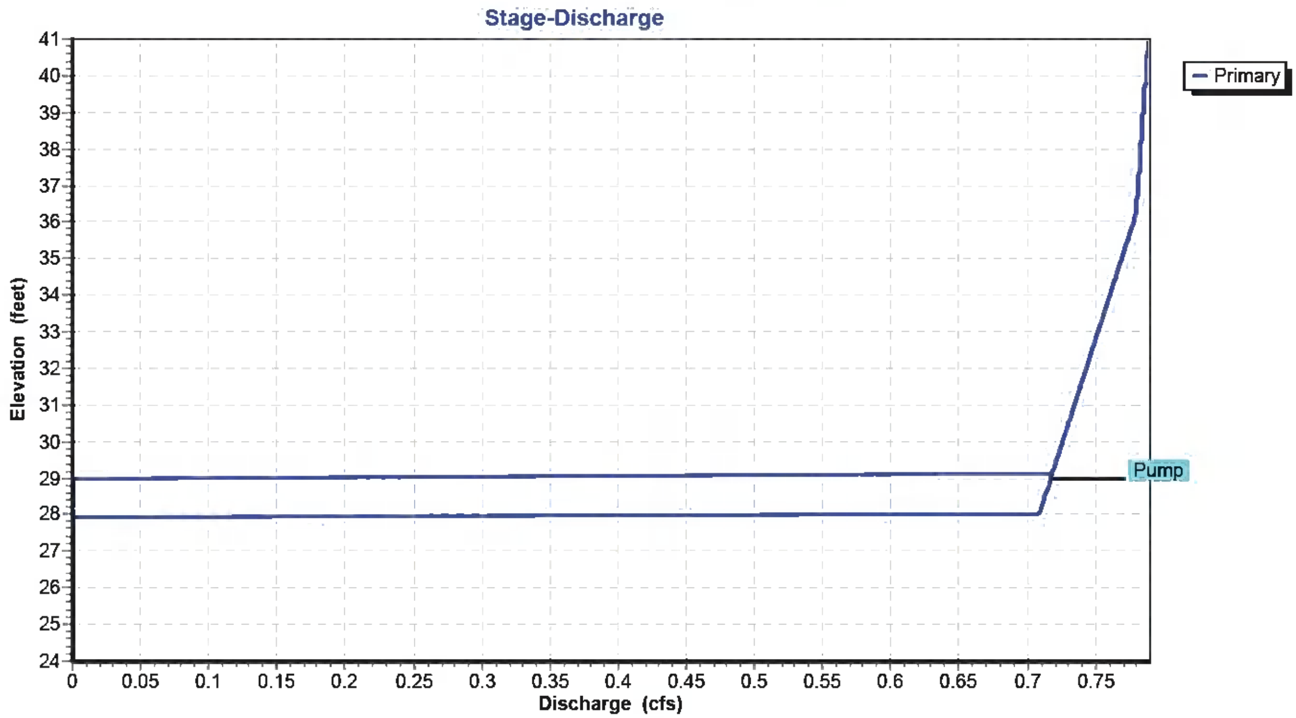
Device	Routin	Invert	Outlet Devices
#1	Primary	29.00'	**Pump** Discharges@41.00' Turns Off@28.00' 6.0" Diam. x 300.0' Long Discharge, Hazen-Williams C= 130 Flow (gpm)= 40.0 100.0 200.0 250.0 300.0 350.0 355.0 Head (feet)= 47.00 40.00 30.00 26.00 20.00 8.00 2.00 -Loss (feet)= 0.06 0.31 1.13 1.71 2.39 3.18 3.27 =Lift (feet)= 46.94 39.69 28.87 24.29 17.61 4.82 -1.27

Primary OutFlow Max=0.78 cfs @ 56.12 hrs HW=37.07' (Free Discharge)
↑—**1=Pump** (Pump Controls 0.78 cfs)

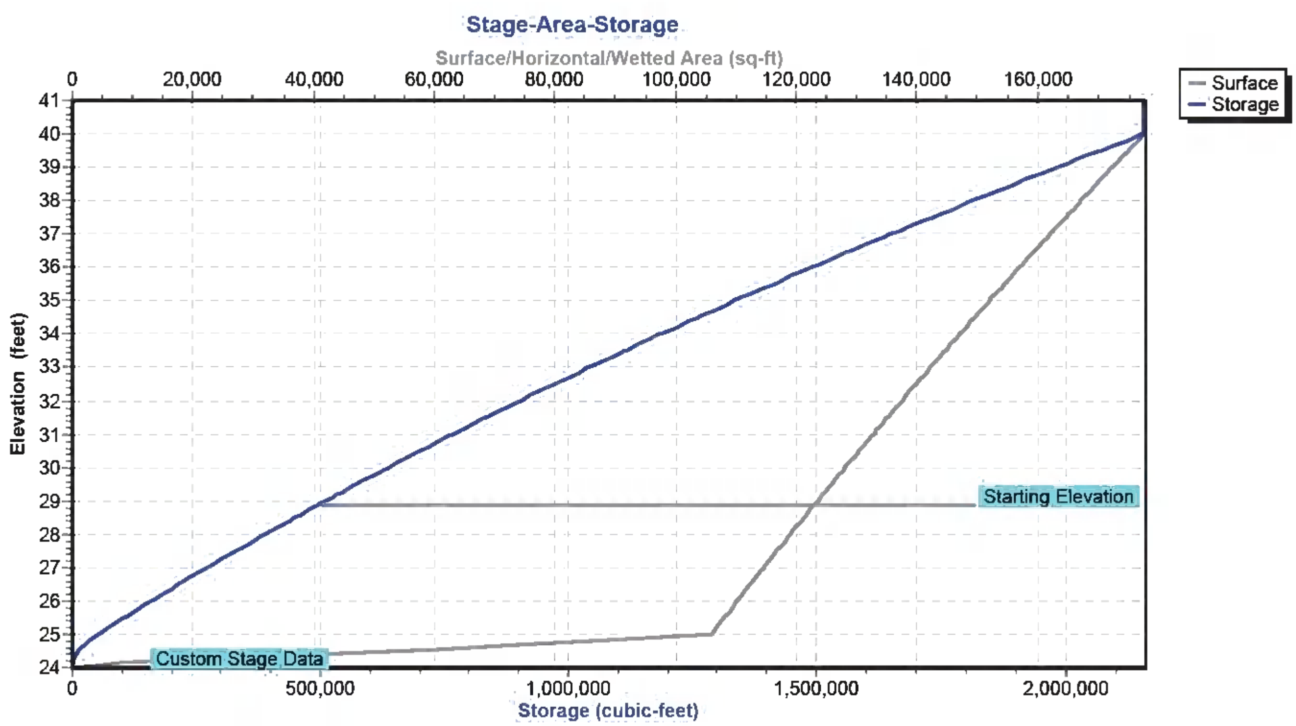
Pond P2: Leachate Pond



Pond P2: Leachate Pond



Pond P2: Leachate Pond



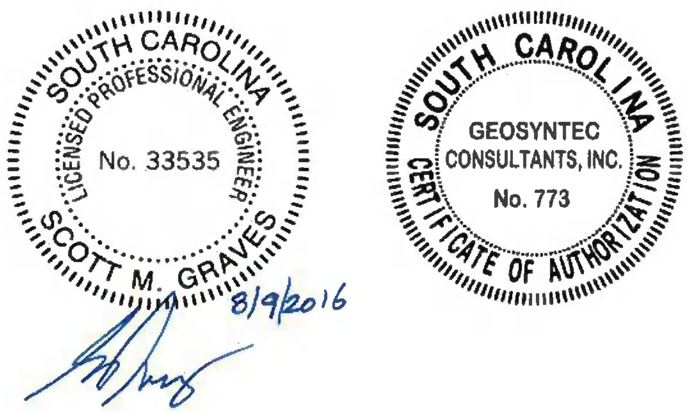
APPENDIX E-6

LEACHATE COLLECTION CORRIDOR AND CONTACT WATER PIPES - STRENGTH DESIGN

Written by:	V. Krishnan	Date:	06/10/16	Reviewed by:	M. Christman	Date:	07/01/16
Client:	Santee Cooper	Project:	Winyah Generating Station	Project No.:	GSC5242	Task No.:	01BT

APPENDIX E-6

LEACHATE COLLECTION CORRIDOR AND CONTACT WATER PIPES - STRENGTH DESIGN



SEALED FOR
CALCULATION PAGES E-6-1
THROUGH E-6-21

PURPOSE

The purpose of this calculation package is to evaluate the strength of the leachate collection corridor pipes, sideslope riser pipes, and contact water discharge pipes for the proposed Class Three Landfill (composed of Landfill Area 1 and Landfill Area 2) at the Winyah Generating Station (WGS). The proposed leachate corridor pipes are 6-in. diameter standard dimension ratio (SDR)-11 (maximum) perforated high-density polyethylene (HDPE). The proposed sideslope riser pipes are 24-in. diameter SDR-11. The proposed contact water discharge pipes are 18-in. diameter SDR-11 (maximum) HDPE.

The function of leachate corridor pipes is to convey leachate collected by the leachate drainage layer to the sumps. The riser pipes extend from the sumps to the crest of the perimeter sideslope. Submersible pumps will be placed inside the riser pipes to transfer leachate from the sump to the leachate transmission system (LTS) forcemain. The function of the contact water discharge pipes is to convey contact water from the decant structure to the outlet leachate pond. The leachate collection corridor pipes, riser pipes, and contact water pipes must have sufficient structural resistance to withstand the applied loads.

Written by:	**V. Krishnan**	Date:	**06/10/16**	Reviewed by:	**M. Christman**	Date:	**07/01/16**
Client:	**Santee Cooper**	Project:	**Winyah Generating Station**	Project No.:	**GSC5242**	Task No.:	**01BT**

METHOD OF ANALYSES

Three potential strength failure mechanisms are considered for plastic pipes: (i) wall crushing; (ii) wall buckling; and (iii) excessive bending strain. These mechanisms are evaluated below using methods presented in the technical literature for flexible plastic pipes [Uni-Bell PVC Pipe Association (Unibell), 1991; and Chevron Phillips Chemical Company (CPChem), 2002]. The design methods for flexible plastic pipe are applicable for both PVC and HDPE pipes (U.S. Army Corps of Engineers, 1997).

Stresses on Pipes

Stresses applied to the pipes are estimated for the post-closure condition, the most critical loading condition for the pipes. As long as sufficient cover is maintained between construction equipment and the pipes, stresses during construction are expected to be significantly lower than post-closure stresses.

During the post-closure condition, the stress applied to the pipe is due to the overburden materials above the pipe (i.e., waste material and final cover soils). This stress is calculated as follows:

$$\sigma_{max} = \gamma_p D_p \qquad \text{(Eqn. 1)}$$

where:

σ_{max} = stress on the pipe, psf;

γ_p = average unit weight of the overburden materials, pcf; and

D_p = thickness of the overburden materials, ft.

The influence of holes on the pipe stress is not normally accounted for in the design process (Bonaparte et al., 2002) and is, therefore, not done so here. Instead, perforation locations that have been demonstrated to be less critical in terms of stress concentrations (Brachman and Krushelnitzky, 2002) have been specified (i.e., perforations are located at the pipe shoulders and haunches).

The structural resistance of the 6-in. diameter leachate corridor pipe is evaluated under loading from 212 ft of waste (i.e., the greatest waste thickness, which occurs below the peak waste elevation in Landfill Area 2) and cover system materials.

The structural resistance of the typical 24-in. diameter riser pipe is evaluated under loading from 30 ft of waste (i.e., the greatest waste thickness at all sumps except that of Subcell 5S) and cover system materials. The structural resistance of the 24-in. diameter riser pipe at Subcell 5S is evaluated under loading from 158 ft of waste (i.e., the waste thickness following the Subcell 5S riser pipe extension in conjunction with Cell 8 filling).

The structural resistance of the 18-in. diameter contact water discharge pipe is evaluated under loading from 166 ft of waste (i.e., the greatest waste thickness located at the proposed decant structure for Cells 4 and 5) and cover system materials.

Wall Crushing

Wall crushing can occur when the stress in the pipe wall, due to external vertical pressure, exceeds the compressive strength of the pipe material. The factor of safety against pipe wall crushing is calculated using the following equation (Phillips 66, 1991):

$$FS_{wc} = \frac{2\sigma_y}{(SDR-1)\sigma_{max}} \qquad \text{(Eqn. 2)}$$

where:

FS_{wc} = factor of safety against pipe wall crushing;

σ_y = compressive yield strength of the pipe, psf;

SDR = standard dimension ratio of the pipe; and

σ_{max} = maximum stress applied to the pipe, psf.

Wall Buckling

Wall buckling (a longitudinal wrinkling in the pipe wall) can occur when the external vertical pressure exceeds the critical buckling pressure of the pipe/bedding aggregate system. The factor of safety against pipe wall buckling is calculated using the following equation:

$$FS_{wb} = \frac{1.2}{\sigma_{max}}\left[\frac{E'E}{(SDR)^3}\right]^{1/2} \qquad \text{(Eqn. 3)}$$

where:

FS_{wb} = factor of safety against pipe wall buckling;

σ_{max} = maximum stress applied to the pipe, psi;

$E' = f(E_s, \nu, k)$ = modulus of soil reaction for pipe bedding material, psi;

E = modulus of elasticity of the pipe material, psi (Figure E-6.1); and

SDR = standard dimension ratio of the pipe.

The modulus of soil reaction, E′, for pipe bedding material is a representative parameter of soil stiffness, which is related to the overburden stress. The modulus of soil reaction is calculated using the constrained modulus of the pipe bedding material (M_s) and an empirical factor (k) based on test data. The constrained modulus, in turn, is calculated as a function of Young's modulus of the pipe bedding material (E_s) and Poisson's ratio of the pipe bedding material (ν):

$$M_s = \frac{E_s(1-\nu)}{(1+\nu)(1-2\nu)} \qquad \text{(Eqn. 4)}$$

where:

M_s = constrained modulus, psi;

E_s = Young's modulus, psi; and

ν = Poisson's ratio.

The Young's modulus and Poisson's ratio were taken from data presented by Selig (1990) for soils at various overburden stress levels. For the leachate corridor pipe analysis, the Young's modulus and Poisson's ratio values are based on a gravel bedding material (i.e., having a classification of GW or GP as defined by the Unified Soil Classification System (USCS)) compacted to 85 percent ASTM D698 at a stress level of 60 psi, the highest stress considered in the Selig (1990) table (Table E-6.1). It is assumed that this material will be an AASHTO No. 57 stone or similar material. The calculations for the riser pipe assume a well-graded or poorly-graded clean sand or gravel bedding material (having a USCS classification of SP, SW, GP, or GW) compacted to 85 percent ASTM D698 at a stress level equal to the lesser of the applied stress or 60 psi. The calculations for the contact water discharge pipe assume a fly ash bedding material (having a USCS classification of ML based on EPRI (2012)) compacted to 85 percent ASTM D698 at a stress level of 60 psi. It is noted that the maximum applied stress on the pipes are higher than 60 psi, as shown in the calculations below. It is therefore anticipated that the constrained modulus will be even higher than the values calculated for a stress level of 60 psi.

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Bending Strain

When a pipe deflects under load, bending strains are induced in the pipe wall. Bending strain occurs in the pipe wall as external pressures are applied to the pipe/bedding aggregate system. The calculated bending strain depends on the ring deflection, which is evaluated using the Modified Iowa Equation (Mosher, 1990; Koerner, 1998) shown below:

$$\Delta X = \frac{D_L K W_c}{\left(EI/r^3\right)+\left(0.061E'\right)} \qquad \text{(Eqn. 5)}$$

where:

ΔX = horizontal deflection or change in diameter, in.;

D_L = deflection lag factor;

K = bedding constant (Figure E-6.2);

W_c = Marston's prism load per unit length of pipe, psi;

E = short-term modulus of elasticity of the pipe, psi (Figure E-6.1);

E' = modulus of soil reaction for bedding material, psi;

I = moment of inertia of the pipe wall per unit length, $in.^4/in.$; and

r = mean radius of the pipe $\left[\frac{D_{od}-t}{2}\right]$, in.

For non-pressure heavy wall HDPE pipe, CPChem (2002) does not recommend a specific "allowable deflection", but instead recommends the bending strain at the predicted deflection be calculated and compared to the allowable strain. Bending strain is calculated using the following equation (Mosher, 1990):

$$\varepsilon_b = f_d \times \frac{t \cdot \Delta y}{D^2} \qquad \text{(Eqn. 6)}$$

where:

ε_b= bending strain, percent;

f_d = deformation shape factor [CPChem (2002) recommends a value of 6 for elliptical cross-sections];

t = minimum wall thickness, in.;

Δy = vertical deflection, in., assumed equal to the horizontal deflection (ΔX); and

D = mean pipe diameter, in.

The following are recommendations for allowable bending strain from the literature and manufacturers:

- an allowable bending strain of 5 percent is recommended in Wilson-Fahmy and Koerner (1994) based on AASHTO guidelines for long term use of smooth polyethylene pipes;
- an allowable bending strain of 4.2 percent is recommended as a conservative upper limit value in CPChem (2002); and
- an allowable bending strain of up to 8 percent is reported by CPChem (2002) as acceptable for a design period of 50 years.

Based on the above information, an allowable strain of 5 percent is selected for HDPE pipe.

CALCULATIONS

Calculations were carried out for 6-in. diameter leachate corridor pipe, 24-in. diameter riser pipe, and 18-in. diameter contact water discharge pipe under expected maximum loads at proposed landfill final grades (i.e., design loading). In addition, the maximum allowable height of waste that the pipes can accommodate (i.e., maximum allowable loading) with adequate factors of safety and allowable strains was calculated. The input parameters and calculated and allowable factors of safety, deflections, and strains are presented in the following calculation sheets:

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6”ϕ SDR-11 HDPE Leachate Corridor (design loading)

Winyah Generating Station - Pipe Strength Design

Performed by: V. Krishnan, 06/10/2016 Reviewed by: M. Christman, 07/01/2016

Input Parameters

Waste

d_c = 212 ft

γ_{avg} = 100 pcf

Pipe

SDR = 11

D_{od} = 6.625 in.

t_{min} = 0.602 in.

E = 30,841 psi

σ_y = 1500 psi

D_L = 1.25

K = 0.11

k = 1.5

Bedding Soil

E_s = 4700 psi

ν = 0.28

Calculated Parameters

σ_{max} = 147.2 psi

M_s = 6009 psi

E' = 9013 psi

W_c = 975 lb/in.

I = 0.01818 in.^{4}/in.

r_{mean} = 3.01 in.

SA = 736.1 psi

Strength Checks

Wall Crushing

$$FS_{WC} = \frac{2\sigma_y}{(SDR - 1)\sigma_{max}}$$

FS_{WC} = 2.0 ≥ 1.5

Wall Buckling

$$FS_{wb} = \frac{1.2}{\sigma_{max}}\left[\frac{E\,E'}{SDR^3}\right]^{1/2}$$

FS_{wb} = 3.7 ≥ 1.5

Ring deflection (Modified Iowa Equation):

$$\Delta X = \frac{D_L K W_c}{(EI / r^3) + (0.061\,E')}$$

Change in diameter, ΔX = 0.235 in.

Ring deflection, ΔX% = 3.55 %

Pipe wall bending strain, ε_b:

$$\varepsilon_b = 6 \cdot \frac{t_{min} \cdot \Delta y}{D^2}$$

Δy = 0.235 in.

D = 6.02 in.

Bending strain, ε_b = 2.34 %

Allowable wall ring bending strain: from 4.2 to 8% (8% for 50 year design life) - [CPChem, 2002]

Varaiable Definition

d_c = maximum thickness of overlying materials, ft;

γ_{avg} = average unit weight of overlying materials (waste, liner and cover), pcf;

SDR = standard dimension ratio of the pipe;

D_{od} = outer diameter of pipe, in [CPChem, 2002];

t_{min} = minimum thickness of the pipe, in. [CPChem, 2002]

E = long-term modulus of elasticity of the pipe material [after 50 years based on S_A, Phillips 66, 1991], psi;

σ_y = compressive yield strength of the pipe;

D_L = deflection lag factor (assume 1.25) [Wilson-Fahmy and Koerner, 1994];

K = bedding constant (0° => 0.110) [Wilson-Fahmy and Koerner, 1994; Figure 2];

k = an empirically derived factor for calculating E' (ranges between 0.7 and 2.3, Selig, 1990);

E_s = Young's modulus of the bedding material, psi;

ν = Poisson's ratio of the bedding material;

σ_{max} = maximum stress applied to the pipe, psi;

M_s = constrained modulus of the bedding material;

E' = the modulus of soil reaction for pipe bedding material [Selig, 1990; Table 2], psi;

W_c = Marston's prism load per unit length of pipe, lb/in. [Wilson -Fahmy and Koerner, 1994]
= (γ_{avg}) (d_c) (D_{od});

I = the moment of inertia of the pipe wall per unit length (t_{min}^3/12), in.^{4}/in.;

r_{mean} = mean radius = (D_{od} - t_{min})/2 , in.

S_A = =(SDR-1)σ_{max} /2

FS_{WC} = factor of safety against wall crushing

FS_{wb} = factor of safety against wall buckling

ΔX = maximum horizontal deflection or change in diameter, in;

ΔX% = the ring deflection, %.
= 100(ΔX/D_{od})

ε_b = Bending strain, %;

Δy = Vertical deflection, in. = ΔX;

D = diameter = Mean diameter (D_{od}-t_{min}), in.;

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6"ϕ SDR-11 HDPE Leachate Corridor (maximum allowable loading)

Winyah Generating Station - Pipe Strength Design

Performed by: V. Krishnan, 06/10/2016   Reviewed by: M. Christman, 07/01/2016

Input Parameters

Waste

d_c = 288 ft

γ_{avg} = 100 pcf

Pipe

SDR = 11

D_{od} = 6.625 in.

t_{min} = 0.602 in.

E = 28,292 psi

σ_y = 1500 psi

D_L = 1.25

K = 0.11

k = 1.5

Bedding Soil

E_s = 4700 psi

ν = 0.28

Calculated Parameters

σ_{max} = 199.9 psi

M_s = 6009 psi

E' = 9013 psi

W_c = 1,324 lb/in.

I = 0.01818 in.^{4}/in.

r_{mean} = 3.01 in.

SA = 999.4 psi

Strength Checks

Wall Crushing

$$FS_{WC} = \frac{2\sigma_y}{(SDR-1)\sigma_{max}}$$

FS_{WC} = 1.5 ≥ 1.5

Wall Buckling

$$FS_{wb} = \frac{1.2}{\sigma_{max}}\left[\frac{E'E}{SDR^3}\right]^{1/2}$$

FS_{wb} = 2.6 ≥ 1.5

Ring deflection (Modified Iowa Equation):

$$\Delta X = \frac{D_L K W_c}{(EI/r^3) + (0.061 E')}$$

Change in diameter, ΔX = 0.320 in.

Ring deflection, ΔX% = 4.83 %

Pipe wall bending strain, ε_b.

$$\varepsilon_b = 6 \cdot \frac{t_{min} \cdot \Delta y}{D^2}$$

Δy = 0.320 in.

D = 6.02 in.

Bending strain, ε_b = 3.19 %

Allowable wall ring bending strain: from 4.2 to 8% (8% for 50 year design life) - [CPChem, 2002]

Varaiable Definition

d_c = maximum thickness of overlying materials, ft;

γ_{avg} = average unit weight of overlying materials (waste, liner and cover), pcf;

SDR = standard dimension ratio of the pipe;

D_{od} = outer diameter of pipe, in [CPChem, 2002];

t_{min} = minimum thickness of the pipe, in. [CPChem, 2002]

E = long-term modulus of elasticity of the pipe material [after 50 years based on S_A, Phillips 66, 1991], psi;

σ_y = compressive yield strength of the pipe;

D_L = deflection lag factor (assume 1.25) [Wilson-Fahmy and Koerner, 1994];

K = bedding constant (0° => 0.110) [Wilson-Fahmy and Koerner, 1994; Figure 2];

k = an empirically derived factor for calculating E' (ranges between 0.7 and 2.3, Selig, 1990);

E_s = Young's modulus of the bedding material, psi;

ν = Poisson's ratio of the bedding material;

σ_{max} = maximum stress applied to the pipe, psi;

M_s = constrained modulus of the bedding material;

E' = the modulus of soil reaction for pipe bedding material [Selig, 1990; Table 2], psi;

W_c = Marston's prism load per unit length of pipe, lb/in. [Wilson -Fahmy and Koerner, 1994]
= (γ_{avg}) (d_c) (D_{od});

I = the moment of inertia of the pipe wall per unit length (t_{min}^3/12), in.^{4}/in.;

r_{mean} = mean radius = (D_{od} - t_{min})/2 , in.

S_A = =(SDR-1)σ_{max} /2

FS_{WC} = factor of safety against wall crushing

FS_{wb} = factor of safety against wall buckling

ΔX = maximum horizontal deflection or change in diameter, in;

ΔX% = the ring deflection, %.
= 100(ΔX/D_{od})

ε_b = Bending strain, %;

Δy = Vertical deflection, in. = ΔX;

D = diameter = Mean diameter (D_{od}-t_{min}), in.;

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24"ϕ SDR-11 HDPE Riser Pipe (design loading)

Winyah Generating Station - Pipe Strength Design

Performed by: V. Krishnan, 06/10/2016 Reviewed by: M. Christman, 07/01/2016

Input Parameters

Waste

d_c = 30 ft

γ_{avg} = 100 pcf

Pipe

SDR = 11

D_{od} = 24.000 in.

t_{min} = 2.182 in.

E = 38,558 psi

σ_y = 1500 psi

D_L = 1.25

K = 0.11

k = 1.5

Bedding Soil

E_s = 3300 psi

ν = 0.19

Calculated Parameters

σ_{max} = 20.8 psi

M_s = 3623 psi

E' = 5434 psi

W_c = 500 lb/in.

I = 0.86551 in.^{4}/in.

r_{mean} = 10.91 in.

SA = 104.2 psi

Strength Checks

Wall Crushing

$$FS_{WC} = \frac{2\sigma_y}{(SDR - 1)\sigma_{max}}$$

FS_{WC} = 14.4 ≥ 1.5

Wall Buckling

$$FS_{wb} = \frac{1.2}{\sigma_{max}}\left[\frac{E'E}{SDR^3}\right]^{1/2}$$

FS_{wb} = 22.9 ≥ 1.5

Ring deflection (Modified Iowa Equation):

$$\Delta X = \frac{D_L K W_c}{(EI/r^3) + (0.061 E')}$$

Change in diameter, ΔX = 0.192 in.

Ring deflection, ΔX% = 0.80 %

Pipe wall bending strain, ε_b.

$$\varepsilon_b = 6 \cdot \frac{t_{min} \cdot \Delta y}{D^2}$$

Δy = 0.192 in.

D = 21.82 in.

Bending strain, ε_b = 0.53 %

Allowable wall ring bending strain: from 4.2 to 8% (8% for 50 year design life) - [CPChem, 2002]

Varaiable Definition

d_c = maximum thickness of overlying materials, ft;

γ_{avg} = average unit weight of overlying materials (waste, liner and cover), pcf;

SDR = standard dimension ratio of the pipe;

D_{od} = outer diameter of pipe, in [CPChem, 2002];

t_{min} = minimum thickness of the pipe, in. [CPChem, 2002]

E = long-term modulus of elasticity of the pipe material [after 50 years based on S_A, Phillips 66, 1991], psi;

σ_y = compressive yield strength of the pipe;

D_L = deflection lag factor (assume 1.25) [Wilson-Fahmy and Koerner, 1994];

K = bedding constant (0° => 0.110) [Wilson-Fahmy and Koerner, 1994; Figure 2];

k = an empirically derived factor for calculating E' (ranges between 0.7 and 2.3, Selig, 1990);

E_s = Young's modulus of the bedding material, psi;

ν = Poisson's ratio of the bedding material;

σ_{max} = maximum stress applied to the pipe, psi;

M_s = constrained modulus of the bedding material;

E' = the modulus of soil reaction for pipe bedding material [Selig, 1990; Table 2], psi;

W_c = Marston's prism load per unit length of pipe, lb/in. [Wilson -Fahmy and Koerner, 1994] = (γ_{avg}) (d_c) (D_{od});

I = the moment of inertia of the pipe wall per unit length (t_{min}^3/12), in.^{4}/in.;

r_{mean} = mean radius = (D_{od} - t_{min})/2 , in.

S_A = =(SDR-1)σ_{max} /2

FS_{WC} = factor of safety against wall crushing

FS_{wb} = factor of safety against wall buckling

ΔX = maximum horizontal deflection or change in diameter, in;

ΔX% = the ring deflection, %. = 100(ΔX/D_{od})

ε_b = Bending strain, %;

Δy = Vertical deflection, in. = ΔX;

D = diameter = Mean diameter (D_{od}-t_{min}), in.;

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24"ϕ SDR-11 HDPE Riser Pipe (design loading at Cell 5S)

Winyah Generating Station - Pipe Strength Design

Performed by: V. Krishnan, 06/10/2016 Reviewed by: M. Christman, 07/01/2016

Input Parameters

Waste

d_c = 158 ft

γ_{avg} = 100 pcf

Pipe

SDR = 11

D_{od} = 24.000 in.

t_{min} = 2.182 in.

E = 32,880 psi

σ_y = 1500 psi

D_L = 1.25

K = 0.11

k = 1.5

Bedding Soil

E_s = 4700 psi

ν = 0.28

Calculated Parameters

σ_{max} = 109.7 psi

M_s = 6009 psi

E' = 9013 psi

W_c = 2,633 lb/in.

I = 0.86551 in.^{4}/in.

r_{mean} = 10.91 in.

SA = 548.6 psi

Strength Checks

Wall Crushing

$$FS_{WC} = \frac{2\sigma_y}{(SDR-1)\sigma_{max}}$$

FS_{WC} = 2.7 ≥ 1.5

Wall Buckling

$$FS_{wb} = \frac{1.2}{\sigma_{max}}\left[\frac{E'E}{SDR^3}\right]^{1/2}$$

FS_{wb} = 5.2 ≥ 1.5

Ring deflection (Modified Iowa Equation):

$$\Delta X = \frac{D_L K W_c}{(EI/r^3)+(0.061E')}$$

Change in diameter, ΔX = 0.633 in.

Ring deflection, ΔX% = 2.64 %

Pipe wall bending strain, ε_b.

$$\varepsilon_b = 6 \cdot \frac{t_{min} \cdot \Delta y}{D^2}$$

Δy = 0.633 in.

D = 21.82 in.

Bending strain, ε_b = 1.74 %

Allowable wall ring bending strain: from 4.2 to 8% (8% for 50 year design life) - [CPChem, 2002]

Variable Definition

d_c = maximum thickness of overlying materials, ft;

γ_{avg} = average unit weight of overlying materials (waste, liner and cover), pcf;

SDR = standard dimension ratio of the pipe;

D_{od} = outer diameter of pipe, in [CPChem, 2002];

t_{min} = minimum thickness of the pipe, in. [CPChem, 2002]

E = long-term modulus of elasticity of the pipe material [after 50 years based on S_A, Phillips 66, 1991], psi;

σ_y = compressive yield strength of the pipe;

D_L = deflection lag factor (assume 1.25) [Wilson-Fahmy and Koerner, 1994];

K = bedding constant (0° => 0.110) [Wilson-Fahmy and Koerner, 1994; Figure 2];

k = an empirically derived factor for calculating E' (ranges between 0.7 and 2.3, Selig, 1990);

E_s = Young's modulus of the bedding material, psi;

ν = Poisson's ratio of the bedding material;

σ_{max} = maximum stress applied to the pipe, psi;

M_s = constrained modulus of the bedding material;

E' = the modulus of soil reaction for pipe bedding material [Selig, 1990; Table 2], psi;

W_c = Marston's prism load per unit length of pipe, lb/in. [Wilson -Fahmy and Koerner, 1994] = (γ_{avg}) (d_c) (D_{od});

I = the moment of inertia of the pipe wall per unit length (t_{min}^3/12), in.^{4}/in.;

r_{mean} = mean radius = (D_{od} - t_{min})/2 , in.

S_A = =(SDR-1)σ_{max} /2

FS_{WC} = factor of safety against wall crushing

FS_{wb} = factor of safety against wall buckling

ΔX = maximum horizontal deflection or change in diameter, in;

ΔX% = the ring deflection, %. = 100(ΔX/D_{od})

ε_b = Bending strain, %;

Δy = Vertical deflection, in. = ΔX;

D = diameter = Mean diameter (D_{od}-t_{min}), in.;

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24"ϕ SDR-11 HDPE Riser Pipe (maximum allowable loading)

Winyah Generating Station - Pipe Strength Design

Performed by: V. Krishnan, 06/10/2016 Reviewed by: M. Christman, 07/01/2016

Input Parameters

Waste

d_c = 288 ft

γ_{avg} = 100 pcf

Pipe

SDR = 11

D_{od} = 24.000 in.

t_{min} = 2.182 in.

E = 28,291 psi

σ_y = 1500 psi

D_L = 1.25

K = 0.11

k = 1.5

Bedding Soil

E_s = 4700 psi

v = 0.28

Calculated Parameters

σ_{max} = 199.9 psi

M_s = 6009 psi

E' = 9013 psi

W_c = 4,797 lb/in.

I = 0.86551 in.^{4}/in.

r_{mean} = 10.91 in.

SA = 999.4 psi

Strength Checks

Wall Crushing

$$FS_{WC} = \frac{2\sigma_y}{(SDR - 1)\sigma_{max}}$$

FS_{WC} = 1.5 ≥ 1.5

Wall Buckling

$$FS_{wb} = \frac{1.2}{\sigma_{max}}\left[\frac{E E'}{SDR^3}\right]^{1/2}$$

FS_{wb} = 2.6 ≥ 1.5

Ring deflection (Modified Iowa Equation):

$$\Delta X = \frac{D_L K W_c}{(EI / r^3) + (0.061 E')}$$

Change in diameter, ΔX = 1.160 in.

Ring deflection, ΔX% = 4.83 %

Pipe wall bending strain, ε_b.

$$\varepsilon_b = 6 \cdot \frac{t_{min} \cdot \Delta y}{D^2}$$

Δy = 1.160 in.

D = 21.82 in.

Bending strain, ε_b = 3.19 %

Allowable wall ring bending strain: from 4.2 to 8% (8% for 50 year design life) - [CPChem, 2002]

Varaiable Definition

d_c = maximum thickness of overlying materials, ft;

γ_{avg} = average unit weight of overlying materials (waste, liner and cover), pcf;

SDR = standard dimension ratio of the pipe;

D_{od} = outer diameter of pipe, in [CPChem, 2002];

t_{min} = minimum thickness of the pipe, in. [CPChem, 2002]

E = long-term modulus of elasticity of the pipe material [after 50 years based on S_A, Phillips 66, 1991], psi;

σ_y = compressive yield strength of the pipe;

D_L = deflection lag factor (assume 1.25) [Wilson-Fahmy and Koerner, 1994];

K = bedding constant (0° => 0.110) [Wilson-Fahmy and Koerner, 1994; Figure 2];

k = an empirically derived factor for calculating E' (ranges between 0.7 and 2.3, Selig, 1990);

E_s = Young's modulus of the bedding material, psi;

v = Poisson's ratio of the bedding material;

σ_{max} = maximum stress applied to the pipe, psi;

M_s = constrained modulus of the bedding material;

E' = the modulus of soil reaction for pipe bedding material [Selig, 1990; Table 2], psi;

W_c = Marston's prism load per unit length of pipe, lb/in. [Wilson -Fahmy and Koerner, 1994]

= (γ_{avg}) (d_c) (D_{od});

I = the moment of inertia of the pipe wall per unit length (t_{min}^3/12), in.^{4}/in.;

r_{mean} = mean radius = (D_{od} - t_{min})/2 , in.

S_A = =(SDR-1)σ_{max} /2

FS_{WC} = factor of safety against wall crushing

FS_{wb} = factor of safety against wall buckling

ΔX = maximum horizontal deflection or change in diameter, in;

ΔX% = the ring deflection, %.

= 100(ΔX/D_{od})

ε_b = Bending strain, %;

Δy = Vertical deflection, in. = ΔX;

D = diameter = Mean diameter (D_{od}-t_{min}), in.;

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18"ϕ SDR-11 HDPE Contact Water Discharge Pipe (design loading)

Winyah Generating Station - Pipe Strength Design

Performed by: V. Krishnan, 06/10/2016 Reviewed by: M. Christman, 07/01/2016

Input Parameters

Waste

d_c = 166 ft

γ_{avg} = 100 pcf

Pipe

SDR = 11

D_{od} = 18.000 in.

t_{min} = 1.636 in.

E = 32,565 psi

σ_y = 1500 psi

D_L = 1.25

K = 0.11

k = 1.5

Bedding Soil

E_s = 1000 psi

ν = 0.41

Calculated Parameters

σ_{max} = 115.3 psi

M_s = 2325 psi

E' = 3487 psi

W_c = 2,075 lb/in.

I = 0.36490 in.^{4}/in.

r_{mean} = 8.18 in.

SA = 576.4 psi

Strength Checks

Wall Crushing

$$FS_{WC} = \frac{2\sigma_y}{(SDR-1)\sigma_{max}}$$

FS_{WC} = 2.6 ≥ 1.5

Wall Buckling

$$FS_{wb} = \frac{1.2}{\sigma_{max}}\left[\frac{E'E}{SDR^3}\right]^{1/2}$$

FS_{wb} = 3.0 ≥ 1.5

Ring deflection (Modified Iowa Equation):

$$\Delta X = \frac{D_L K W_c}{(EI/r^3) + (0.061E')}$$

Change in diameter, ΔX = 1.217 in.

Ring deflection, ΔX% = 6.76 %

Pipe wall bending strain, ε_b.

$$\varepsilon_b = 6 \cdot \frac{t_{min} \cdot \Delta y}{D^2}$$

Δy = 1.217 in.

D = 16.36 in.

Bending strain, ε_b = 4.46 %

Allowable wall ring bending strain: from 4.2 to 8% (8% for 50 year design life) - [CPChem, 2002]

Varaiable Definition

d_c = maximum thickness of overlying materials, ft;

γ_{avg} = average unit weight of overlying materials (waste, liner and cover), pcf;

SDR = standard dimension ratio of the pipe;

D_{od} = outer diameter of pipe, in [CPChem, 2002];

t_{min} = minimum thickness of the pipe, in. [CPChem, 2002]

E = long-term modulus of elasticity of the pipe material [after 50 years based on S_A, Phillips 66, 1991], psi;

σ_y = compressive yield strength of the pipe;

D_L = deflection lag factor (assume 1.25) [Wilson-Fahmy and Koerner, 1994];

K = bedding constant (0° => 0.110) [Wilson-Fahmy and Koerner, 1994; Figure 2];

k = an empirically derived factor for calculating E' (ranges between 0.7 and 2.3, Selig, 1990);

E_s = Young's modulus of the bedding material, psi;

ν = Poisson's ratio of the bedding material;

σ_{max} = maximum stress applied to the pipe, psi;

M_s = constrained modulus of the bedding material;

E' = the modulus of soil reaction for pipe bedding material [Selig, 1990; Table 2], psi;

W_c = Marston's prism load per unit length of pipe, lb/in. [Wilson -Fahmy and Koerner, 1994] = (γ_{avg}) (d_c) (D_{od});

I = the moment of inertia of the pipe wall per unit length (t_{min}^3/12), in.^{4}/in.;

r_{mean} = mean radius = (D_{od} - t_{min})/2 , in.

S_A = =(SDR-1)σ_{max} /2

FS_{WC} = factor of safety against wall crushing

FS_{wb} = factor of safety against wall buckling

ΔX = maximum horizontal deflection or change in diameter, in;

ΔX% = the ring deflection, %. = 100(ΔX/D_{od})

ε_b = Bending strain, %;

Δy = Vertical deflection, in. = ΔX;

D = diameter = Mean diameter (D_{od}-t_{min}), in.;

Written by:	V. Krishnan	Date:	06/10/16	Reviewed by:	M. Christman	Date:	07/01/16
Client:	Santee Cooper	Project:	Winyah Generating Station	Project No.:	GSC5242	Task No.:	01BT

18"ϕ SDR-11 HDPE Contact Water Discharge Pipe (maximum allowable loading)

Winyah Generating Station - Pipe Strength Design

Performed by: V. Krishnan, 06/10/2016 Reviewed by: M. Christman, 07/01/2016

Input Parameters

Waste

d_c = 186 ft

γ_{avg} = 100 pcf

Pipe

SDR = 11

D_{od} = 18.000 in.

t_{min} = 1.636 in.

E = 31,807 psi

σ_y = 1500 psi

D_L = 1.25

K = 0.11

k = 1.5

Bedding Soil

E_s = 1000 psi

v = 0.41

Calculated Parameters

σ_{max} = 129.0 psi

M_s = 2325 psi

E' = 3487 psi

W_c = 2,322 lb/in.

I = 0.36490 $in.^4$/in.

r_{mean} = 8.18 in.

SA = 645.0 psi

Strength Checks

Wall Crushing

$$FS_{WC} = \frac{2\sigma_y}{(SDR-1)\sigma_{max}}$$

FS_{WC} = **2.33 ≥ 1.5**

Wall Buckling

$$FS_{wb} = \frac{1.2}{\sigma_{max}}\left[\frac{E\,E'}{SDR^3}\right]^{1/2}$$

FS_{wb} = **2.7 ≥ 1.5**

Ring deflection (Modified Iowa Equation):

$$\Delta X = \frac{D_L K W_c}{(EI/r^3)+(0.061E')}$$

Change in diameter, ΔX = 1.365 in.

Ring deflection, ΔX% = 7.58 %

Pipe wall bending strain, ε_b.

$$\varepsilon_b = 6\cdot\frac{t_{min}\cdot\Delta y}{D^2}$$

Δy = 1.365 in.

D = 16.36 in.

Bending strain, ε_b = 5.00 %

Allowable wall ring bending strain: from 4.2 to 8% (8% for 50 year design life) - [CPChem, 2002]

Varaiable Definition

d_c = maximum thickness of overlying materials, ft;

γ_{avg} = average unit weight of overlying materials (waste, liner and cover), pcf;

SDR = standard dimension ratio of the pipe;

D_{od} = outer diameter of pipe, in [CPChem, 2002];

t_{min} = minimum thickness of the pipe, in. [CPChem, 2002]

E = long-term modulus of elasticity of the pipe material [after 50 years based on S_A, Phillips 66, 1991], psi;

σ_y = compressive yield strength of the pipe;

D_L = deflection lag factor (assume 1.25) [Wilson-Fahmy and Koerner, 1994];

K = bedding constant (0° => 0.110) [Wilson-Fahmy and Koerner, 1994; Figure 2];

k = an empirically derived factor for calculating E' (ranges between 0.7 and 2.3, Selig, 1990);

E_s = Young's modulus of the bedding material, psi;

v = Poisson's ratio of the bedding material;

σ_{max} = maximum stress applied to the pipe, psi;

M_s = constrained modulus of the bedding material;

E' = the modulus of soil reaction for pipe bedding material [Selig, 1990; Table 2], psi;

W_c = Marston's prism load per unit length of pipe, lb/in. [Wilson -Fahmy and Koerner, 1994] = (γ_{avg}) (d_c) (D_{od});

I = the moment of inertia of the pipe wall per unit length (t_{min}^3/12), $in.^4$/in.;

r_{mean} = mean radius = (D_{od} - t_{min})/2 , in.

S_A = =(SDR-1)σ_{max} /2

FS_{WC} = factor of safety against wall crushing

FS_{wb} = factor of safety against wall buckling

ΔX = maximum horizontal deflection or change in diameter, in;

ΔX% = the ring deflection, %. = 100(ΔX/D_{od})

ε_b = Bending strain, %;

Δy = Vertical deflection, in. = ΔX;

D = diameter = Mean diameter (D_{od}-t_{min}), in.;

Written by:	**V. Krishnan**	Date:	**06/10/16**	Reviewed by:	**M. Christman**	Date:	**07/01/16**
Client:	**Santee Cooper**	Project:	**Winyah Generating Station**	Project No.:	**GSC5242**	Task No.:	**01BT**

SUMMARY AND CONCLUSIONS

6"ϕ SDR-11 HDPE Leachate Corridor Pipes

Under the design loading resulting from a total waste height of 212 ft on top of the leachate corridor pipes, the pipe strength evaluation is summarized as follows:

* Factor of safety against pipe wall crushing, FS_{wc} = 2.0 (OK)
* Factor of safety against pipe wall buckling, FS_{wb} = 3.7 (OK)
* Bending strain = 2.34 % (OK)

The back-calculated maximum height of waste over the corridor that would result in acceptable factors of safety and allowable strains is 288 ft. This is in excess of that proposed, but is provided here as an indication of the tallest waste height that the design could tolerate.

24"ϕ SDR-11 HDPE Riser Pipes

Under the design loading resulting from a total waste height of 30 ft on top of the riser pipes, the pipe strength evaluation is summarized as follows:

* Factor of safety against pipe wall crushing, FS_{wc} = 14.4 (OK)
* Factor of safety against pipe wall buckling, FS_{wb} = 22.9 (OK)
* Bending strain = 0.53 % (OK)

Under the design loading resulting from a total waste height of 158 ft on top of the riser pipe in Subcell 5S, the pipe strength evaluation is summarized as follows:

* Factor of safety against pipe wall crushing, FS_{wc} = 2.7 (OK)
* Factor of safety against pipe wall buckling, FS_{wb} = 5.2 (OK)
* Bending strain = 1.74 % (OK)

The back-calculated maximum height of waste over the riser pipes that would result in acceptable factors of safety and allowable strains is 288 ft. This is in excess of that proposed, but is provided here as an indication of the tallest waste height that the design could tolerate.

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18"ϕ SDR-11 HDPE Contact Water Discharge Pipes

Under the design loading resulting from a total waste height of 166 ft on top of the contact water discharge pipe, the pipe strength evaluation is summarized as follows:

* Factor of safety against pipe wall crushing, FS_{wc} = 2.6 (OK)
* Factor of safety against pipe wall buckling, FS_{wb} = 3.0 (OK)
* Bending strain = 4.46% (OK)

The back-calculated maximum height of waste over the corridor that would result in acceptable factors of safety and allowable strains is 186 ft. This is in excess of that proposed, but is provided here as an indication of the tallest waste height that the design could tolerate.

Based on the above results, the specified pipes are anticipated to perform as designed.

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REFERENCES

Bonaparte, R., Daniel, D.E., and Koerner, R.M., "*Assessment and Recommendations for Improving the Performance of Waste Containment Systems*", U.S. Environmental Protection Agency, Washington, D.C., EPA/600/R-02/099, Dec 2002.

Brachman, R.W.I, and Krushelnitzky, R.P., "Stress Concentrations Around Circular Holes in Perforated Drainage Pipes", *Geosynthetics International*, Vol. 9, No. 2, 2002, pp. 189-213.

CPChem, "*Engineering Manual*", CPChem Performance Pipe, Chevron Phillips Chemical Company LP, Plano, TX, 2002.

EPRI, "*Geotechnical Properties of Fly Ash and Potential for Static Liquefaction, Volume 1 – Summary and Conclusions*" Electric Power Research Institute, Palo Alto, CA, Report 1023743, December 2012.

Koerner, R.M., "Designing with Geosynthetics", Fourth Edition, 1998.

Mosher, A., "*Buried Pipe Design*", McGraw-Hill, New York, 1990, 219 p.

Phillips 66, "*Driscopipe System Design*", Manufacturers' literature, No. 1089-91 A17, Phillips 66, 1991.

Selig, E.T., "*Soil Properties for Plastic Pipe Installations*," Buried Plastic Pipe Technology, ASTM STP1093, Buczala and Cassady, Ed., Oct. 1990, pp. 141-158.

Uni-Bell PVC Pipe Association, "*Handbook of PVC Pipe, Design and Construction*", Uni-Bell Plastic Pipe Association, 1991.

US Army Corps of Engineers, "*Conduits, Culverts, and Pipes*", EM 1110-2-2902, 1997.

Wilson-Fahmy, R.F., and Koerner, R.M., "*Finite Element Analysis of Plastic Sewer Pipe Behavior in Leachate Collection and Removal Systems*", Geosynthetic Research Institute - Drexel University, Philadelphia, PA, June 1994, 105 p.

TABLES

Written by:	V. Krishnan	Date:	06/10/16	Reviewed by:	M. Christman	Date:	07/01/16
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Table E-6.1. Modulus of Soil Reaction for Pipe Bedding Material (from Selig, 1990)

Soil Type: SW, SP, GW, GP

Stress level psi (kPa)	95% D698 E_s	95% D698 B	95% D698 v_s	85% D698 E_s	85% D698 B	85% D698 v_s
1 (7)	1600 (11)	2800 (19)	0.40	1300 (9)	900 (6)	0.26
5 (34)	4100 (28)	3300 (23)	0.29	2100 (14)	1200 (8)	0.21
10 (70)	6000 (41)	3900 (27)	0.24	2600 (18)	1400 (10)	0.19
20 (140)	8600 (59)	5300 (37)	0.23	3300 (23)	1800 (12)	0.19
40 (280)	13000 (90)	8700 (60)	0.25	4100 (28)	2500 (17)	0.23
60 (410)	16000 (110)	13000 (90)	0.29	4700 (32)	3500 (24)	0.28

Soil Type: GM, SM, ML, and GC, SC with < 20% fines

Stress level psi (kPa)	95% D698 E_s	95% D698 B	95% D698 v_s	85% D698 E_s	85% D698 B	85% D698 v_s
1 (7)	1800 (12)	1900 (13)	0.34	600 (4)	400 (3)	0.25
5 (34)	2500 (17)	2000 (14)	0.29	700 (5)	450 (3)	0.24
10 (70)	2900 (20)	2100 (14)	0.27	800 (6)	500 (3)	0.23
20 (140)	3200 (22)	2500 (17)	0.29	850 (6)	700 (5)	0.30
40 (280)	3700 (25)	3400 (23)	0.32	900 (6)	1200 (8)	0.38
60 (410)	4100 (28)	4500 (31)	0.35	1000 (7)	1800 (12)	0.41

Soil Type: CL, MH, GC, SC

Stress level psi (kPa)	95% D698 E_s	95% D698 B	95% D698 v_s	85% D698 E_s	85% D698 B	85% D698 v_s
1 (7)	400 (3)	800 (6)	0.42	100 (1)	100 (1)	0.33
5 (34)	800 (6)	900 (6)	0.35	250 (2)	200 (1)	0.29
10 (70)	1100 (8)	1000 (7)	0.32	400 (3)	300 (2)	0.28
20 (140)	1300 (9)	1100 (8)	0.30	600 (4)	400 (3)	0.25
40 (280)	1400 (10)	1600 (11)	0.35	700 (5)	800 (6)	0.35
60 (410)	1500 (10)	2100 (14)	0.38	800 (6)	1300 (9)	0.40

Note: Units of E_s and B are psi (MPa).

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FIGURES

Written by:	V. Krishnan	Date:	06/10/16	Reviewed by:	M. Christman	Date:	07/01/16
Client:	Santee Cooper	Project:	Winyah Generating Station	Project No.:	GSC5242	Task No.:	01BT

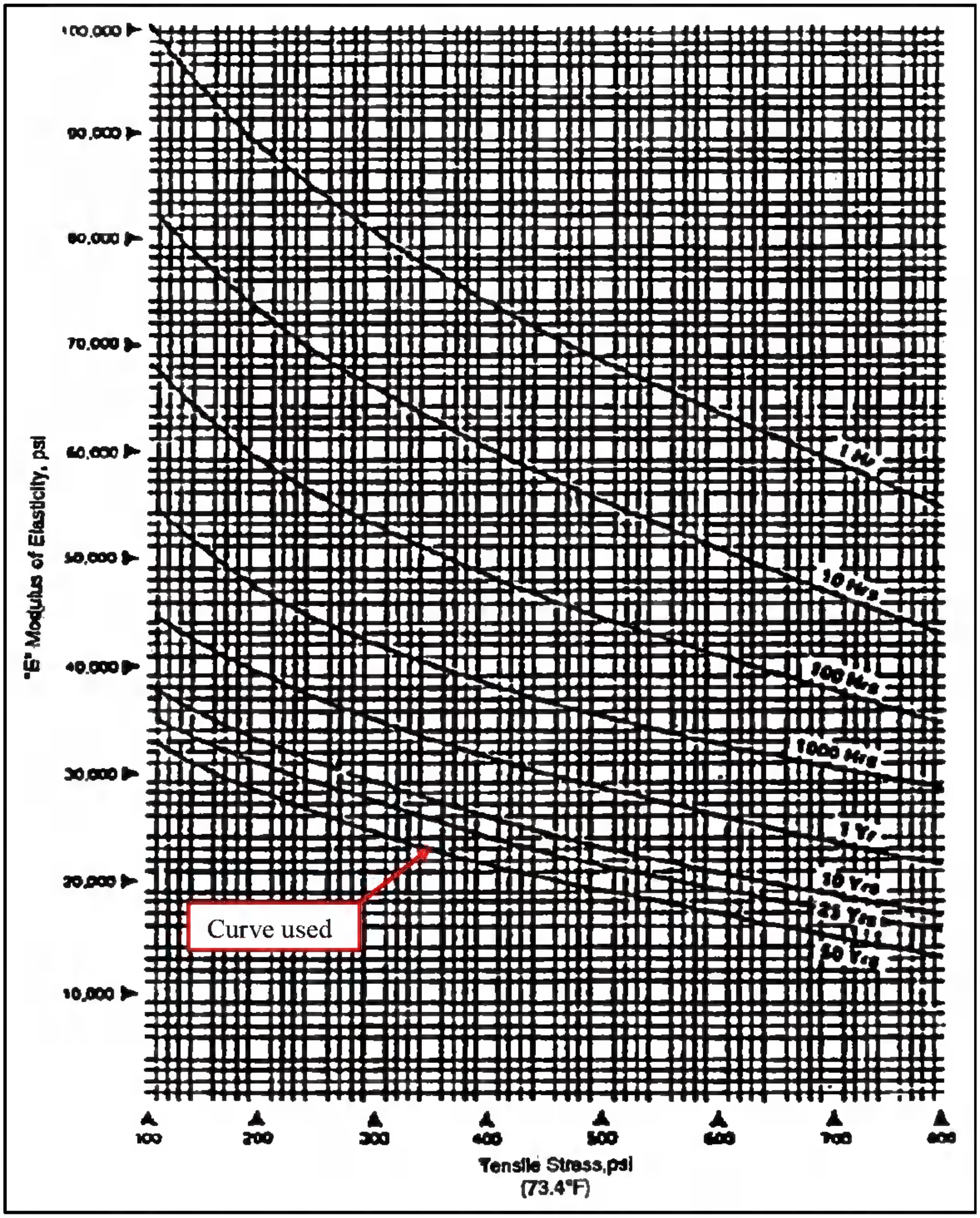

Figure E-6.1. Time Dependent Modulus of Elasticity for Polyethylene Pipe (from Phillips 66, 1991)

Values of Bedding Constant

Bedding Angle (degrees)	K
0	0.110
30	0.108
45	0.105
60	0.102
90	0.096
120	0.090
180	0.083

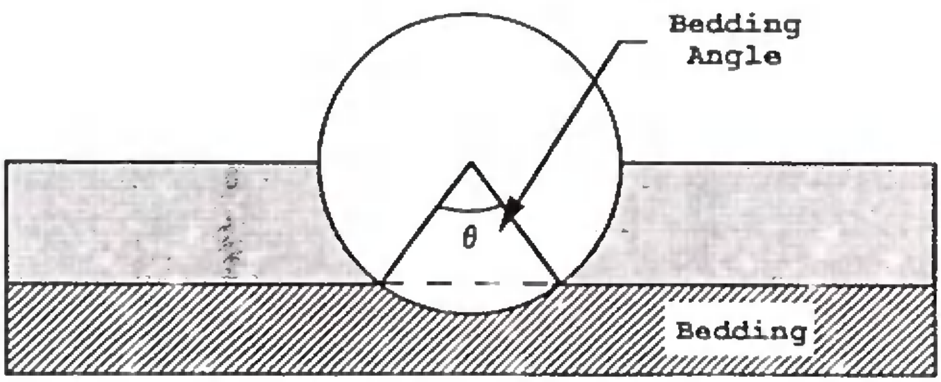

Figure E-6.2. Bedding Constant (from Wilson-Fahmy and Koerner, 1994)

APPENDIX E-7

DURABILITY AND CHEMICAL COMPATIBILITY ASSESSMENT OF GEOMEMBRANE LINER

Written by:	V. Krishnan	Date:	07/14/16	Reviewed by:	B. Gross	Date:	07/22/16
Client:	Santee Cooper	Project:	Winyah Generating Station	Project No.:	GSC5242	Task No.:	01BT

APPENDIX E-7

DURABILITY AND CHEMICAL COMPATIBILITYASSESSMENT OF GEOMEMBRANE LINER

SOUTH CAROLINA LICENSED PROFESSIONAL ENGINEER No. 33535 SCOTT M. GRAVES 8/9/2016

SOUTH CAROLINA CERTIFICATE OF AUTHORIZATION GEOSYNTEC CONSULTANTS, INC. No. 773

SEALED FOR
CALCULATION PAGES E7-1
THROUGH E7-9

PURPOSE

The purpose of this package is to evaluate the physical durability and chemical compatibility of the high-density polyethylene (HDPE) geomembrane component of the liner system for the proposed Class Three Landfill at the Winyah Generating Station (WGS). The evaluation is used to:

- demonstrate the physical durability of the HDPE geomembrane liner by comparing the anticipated conditions of the geomembrane during installation and service to those conditions that could lead to puncture of HDPE geomembranes; and
- demonstrate that the HDPE geomembrane liner will not undergo chemical degradation due to exposure to leachate generated in the landfill by reviewing the published chemical characteristics of leachate from coal combustion residuals (CCR) containment facilities and assessing the resistance of HDPE geomembranes to degradation by liquids with these chemical characteristics.

PHYSICAL DURABILITY OF HDPE GEOMEMBRANE

The physical durability of the HDPE geomembrane liner is demonstrated by assessing the resistance of the geomembrane to puncture by particles or protrusions adjacent to or near the geomembrane. A puncture could potentially occur when a geomembrane is subjected to stresses, such as equipment stress, overburden stress, etc., that cause the geomembrane to deform around particles/protrusions present next to the geomembrane. If such particles/protrusions are likely, a procedure to calculate the factor of safety against puncture proposed by Koerner (2005) may be used. When warranted by the computations, Koerner (2005) recommends that a geotextile cushion be added between the geomembrane and the particles/protrusions to protect the geomembrane from puncture.

As described in Appendix E-1, the HDPE geomembrane liner proposed for the Class Three Landfill will be overlain by a geocomposite drainage layer (geotextile filters bonded to both sides of a geonet drainage core) and underlain by a compacted clay layer or a needlepunched reinforced geosynthetic clay liner (GCL) layer. The geocomposite drainage layer will, in turn, be overlain by a granular protective cover/drainage layer, except along the leachate collection corridors and sumps where it will be directly overlain by coarse aggregate wrapped in a geotextile filter. The cushioning provided by geocomposite drainage layer and the geotextile filter wrapping the coarse aggregate will protect the geomembrane from puncture. Furthermore, the overlying granular materials will be placed with low ground pressure equipment and with a minimum material thickness between the equipment and the geomembrane as specified in the Technical Specifications. For the standard composite liner system that uses a compacted clay layer beneath the geomembrane liner, the Technical Specifications require that the maximum size of particles adjacent to the geomembrane be no greater than 3/8 in. For the alternative composite liner system that uses a GCL beneath the geomembrane liner, the GCL will cushion the geomembrane from particles in the underlying compacted clay layer, which can have a maximum size of 1 in.

Based on these protective measures incorporated into the design of the composite liner system and the installation requirements included in the Technical Specifications, the geomembrane liner is concluded to be physically durable to resist puncture by particles or protrusions in the liner system.

CHEMICAL COMPATIBILITY OF HDPE GEOMEMBRANE

The chemical compatibility of the HDPE geomembrane liner, i.e., the resistance of the geomembrane to chemical degradation, is evaluated by assessing the typical chemical characteristics of leachate from CCR containment facilities, followed by evaluating the resistance of HDPE geomembranes to degradation by liquids with these chemical characteristics.

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Chemical Characteristics of Leachates from CCR Containment Facilities

CCR leachates typically contain a range of inorganic constituents and have a neutral to alkaline pH (Electric Power Research Institute [EPRI], 2006). EPRI (1998) presents the results of a field and laboratory study of the constituents leached from CCR in landfills and surface impoundments. The chemical composition data for porewater (leachate) from 125 core samples collected at eight active CCR landfills is presented in Table E-7.1 (EPRI, 1998). The CCR had been generated from combustion of various types of coal. A subsequent study by EPRI (2006) includes chemistry data from 13 landfills and 15 surface impoundments located in the eastern U.S. Samples from facilities that did not contain flue gas desulfurization (FGD) waste were classified as ash leachate, otherwise the samples were classified as FGD leachate. Ash and FGD leachate constituent data from the EPRI (2006) report are summarized in Table E-7.2. Transition metals presented at the highest concentrations in CCR and FGD leachates in the two EPRI studies are iron, manganese, and molybdenum.

Overview of Chemical Compatibility of HDPE Geomembrane

HDPE geomembranes consist of, by weight percentage, 96% to 97% polyethylene resin, 2% to 3% carbon black, and approximately 0.5% to 1% antioxidants. Polyethylene used for producing geomembranes is essentially chemically inert (Apse, 1989) and does not undergo a change in its molecular structure with organic chemicals such as solvents (USEPA, 1988). Carbon black is added to HDPE geomembranes to reduce penetration of ultraviolet light into the HDPE polymeric compound. Antioxidants are also added to prevent polymer degradation during processing and to extend their service life by delaying polymer degradation caused by oxidation reactions (Hsuan and Koerner, 1998).

The reaction of HDPE geomembranes with chemicals has probably been studied more than any other liner degradation mechanism (Koerner et al. 1990; Rowe et al. 2009; Rowe et al., 2010a,b). In accelerated chemical compatibility testing of geomembranes conducted in the laboratory and in field investigations of geomembranes that have been installed as long as several decades, polyethylene geomembranes have been found to have good resistance to a wide variety of chemicals, including aliphatic and aromatic hydrocarbons, chlorinated and oxygenated solvents, crude petroleum solvents, alcohols, organic and inorganic acids, heavy metals, and salts (Matrecon, Inc., 1988; Hsuan et al., 1991; Brady et al., 1994; Eith and Koerner, 1998; Koerner and Hsuan, 2002; Sangam and Rowe, 2002; Koerner, 2005; Rowe et al., 2010b). This is why HDPE is commonly used for containing pure chemicals in laboratory bottles.

For HDPE geomembranes, the primary degradation mechanisms from chemical exposure consist of: (i) polymer swelling due to absorption of an organic chemical; and (ii) oxidative degradation.

Degradation of HDPE geomembranes used in landfill liners by polymer swelling is essentially reversible for the low levels of contaminants typical of landfill leachates (Sangam and Rowe, 2002; Scheirs, 2009).

With respect to oxidative degradation, HDPE generally does not react with most chemicals because it does not have reactive sites. In addition, HDPE is non-polar and thus does not react readily with polar substances such as water, other inorganic chemicals, and some organic chemicals, such as acetone (Scheirs, 2009). HDPE is relatively inert in both acidic and basic environments, with the exception of oxidizing acids at high concentrations (e.g., sulfuric acid at a concentration greater than 70% (pH of 0.3) (Brydson, 1999; Scheirs, 2009). Further, Rowe et al. (2008) examined the effects of pH on HDPE geomembrane degradation by immersing geomembrane specimens in simulated leachate and distilled water with pHs of 4, 6, 8, and 10 at 185°F for approximately 4.5 months. Their results show no significant difference in antioxidant depletion time for samples in distilled water over the considered pH range.

Although HDPE is not very reactive with metals, the presence of transition metals, such as copper, iron, and manganese, can enhance HDPE oxidation (Sangam and Rowe, 2002). Because of this, the incorporation of transition metal pro-oxidant additives into polyethylene films used to manufacture plastic bags is being studied as a potential solution for controlling plastic film litter.

Compatibility of HDPE Geomembrane with Transition Metals

Transition metals can catalyze abiotic oxidation of polyethylene, resulting in a product that is more susceptible to biodegradation (Corti et al., 2010; Roy et al., 2011; Zheng et al., 2005). However, transition metals are not commonly found in landfill leachates at concentrations that typically cause chemical compatibility issues. In immersion testing of HDPE geomembrane with different leachates, including some that included transition metals in a trace metal solution with a total concentration greater than 3,000 mg/L, Rowe et al. (2008) found the presence of transition metals to have little or no effect on the rate of antioxidant depletion.

Compatibility of HDPE Geomembrane with CCR Leachate

Based on the extensive chemical compatibility studies that have been conducted with HDPE materials and leachates, the HDPE geomembrane liner proposed for the Class Three Landfill is expected to be chemically compatible with the leachate generated in the landfill. Because CCR leachate is essentially a mixture of inorganic constituents, swelling of the geomembrane liner due to absorption of organic chemicals is not a concern for the Class Three Landfill. Furthermore, the expected pH of the CCR leachate is much higher than 0.3. Thus it is not a strong acid, which could accelerate oxidative degradation of HDPE. Lastly, based on the chemical composition data for CCR leachates summarized in Tables E-7.1 and E-7.2, the concentrations of transition metals in the CCR leachate from the Class Three Landfill are expected to be low (i.e., typically concentration less than 500 mg/L) and less than those that could potentially cause transition metal-related degradation of the HDPE geomembrane. Therefore the HDPE geomembrane liner is concluded to be chemically compatible with the expected leachate for the Class Three Landfill.

CONCLUSIONS

Based on the above discussion, it has been demonstrated that the proposed HDPE geomembrane has adequate physical durability and chemical compatibility to function as a liner component for the Class Three Landfill for the WGS.

REFERENCES

Apse, J.L. (1989). "Polyethylene Resins for Geomembrane Applications", In: Koerner, R.M. (Ed.), *Durability and Aging of Geosynthetics*, Elsevier Applied Science Publisher, Amsterdam, Netherlands, pp. 159-176.

Brady, K.C., McMahon, W., and Lamming, G. (1994). "*Thirty Year Ageing of Plastics*", Transport Research Laboratory, Report 11, E472A/BG, ISSN 0968-4093.

Brydson, J.A. (1999). "*Plastics Materials*", 7th Edition, Butterworth Heinemann, Oxford, 920 p.

Corti, A., Muniyasamy, S., Vitali, M., Imam, S.H., and Chiellini, E. (2010). "Oxidation and Biodegradation of Polyethylene Films Containing Pro-oxidant Additives: Synergistic Effects of Sunlight Exposure, Thermal Aging and Fungal Biodegradation", *Polymer Degradation and Stability*, Vol. 95, pp. 1106-1114.

Eith, A.W. and Koerner, G.R. (1998). "Assessment of HDPE Geomembrane Performance in a Municipal Waste Landfill Double Liner System after Eight Years of Service", *Geotextiles and Geomembranes*, Vol. 15, p 277-287.

EPRI (1998). "*Leaching of Inorganic Constituents from Coal Combustion By-Products under Field and Laboratory Conditions: Volume I*", Electric Power Research Institute, TR-111773-V1.

EPRI (2006). "*Characterization of Field Leachates at Coal Combustion Product Management Sites*", Electric Power Research Institute, 1012578.

Hsuan, Y.G. and Koerner, R.M. (1998). "Antioxidant Depletion Lifetime in High Density Polyethylene Geomembranes", *Journal of Geotechnical and Geoenvironmental Engineering*, Vol. 124, No. 6, pp. 532–541.

Hsuan, Y.G., Lord, A.E., and Koerner, R.M. (1991). "Effects of Outdoor Exposure on a High Density Polyethylene Geomembrane", *Geosynthetics '91*, Industrial Fabrics Association International, Vol. 1, pp. 287-302.

Koerner, R.M (2005). "*Designing with Geosynthetics*", 5th edition, 796p.

Koerner, R.M. and Hsuan, G. (2002). "Antioxidant Depletion Time in High Density Polyethylene Geomembranes", Appendix B in *Assessment and Recommendations for Improving the Performance of Waste Containment Systems*, research report published by the U.S. Environmental Protection Agency, National Risk Management Research Laboratory, EPA/600/R-02/099.

Koerner, R.M., Halse, Y.H., and Lord, A.E. Jr. (1990). "Long-Term Durability and Aging of Geomembranes", *Waste Containment Systems: Construction, Regulation and Performance*, Geotechnical Special Publication No. 26, American Society of Engineers, New York, pp. 106-134.

Matrecon, Inc. (1988). "*Lining of Waste Containment and Other Impoundment Facilities*," U.S. Environmental Protection Agency, Risk Reduction Engineering Laboratory, Cincinnati, OH, EPA/600/2-88/052.

Rowe, R.K., Islam, M.Z., and Hsuan, Y.G. (2008). "Leachate Chemical Composition Effects on OIT Depletion in an HDPE Geomembrane", *Geosynthetics International*, Vol. 15, No. 2, pp. 136-151.

Rowe, R.K., Rimal, S., and Sangam, H. (2009). "Ageing of HDPE Geomembrane Exposed to Air, Water, and Leachate at Different Temperatures", *Geotextiles and Geomembranes*, Vol. 27, No. 2, pp. 137-151.

Rowe, R.K., Islam, M.Z., and Hsuan, Y.G. (2010a). "Effects of Thickness on the Ageing of HDPE Geomembranes", *Journal of Geotechnical and Geoenvironmental Engineering*, Vol. 136, No. 2, pp. 199-309.

Rowe, R.K., Islam, M.Z., Brachman, R.W.I., and Arnepalli, D.N. (2010b). "Antioxidant Depletion from an HDPE Geomembrane under Simulated Landfill Condition", *Journal of Geotechnical and Geoenvironmental Engineering*, Vol. 136, No.7, pp. 930-939.

Roy, P.K., Hakkarainen, M., Varma, I.K., and Albertsson, A.-C. (2011). "Degradable Polyethylene: Fantasy or Reality", *Environmental Science and Technology*, Vol. 45, No. 10, pp. 4217-4227.

Sangam, H.P. and Rowe, R.K. (2002). "Durability of HDPE Geomembranes", *Geotextiles and Geomembranes*, Vol. 20, No. 2, pp. 77-95.

Scheirs, J. (2009). "*A Guide to Polymeric Geomembranes: A Practical Approach*", John Wiley & Sons, Ltd., United Kingdom, 572 p.

USEPA (1988). "*Lining of Waste Containment and Other Impoundment Facilities*", EPA/600/2-88/052, Sep. 1988, 12 Chapters plus 13 appendices

Zheng, Y., Yanful, E.K., Bassi, A.S. (2005), "A Review of Plastic Waste Biodegradation", *Critical Reviews in Biotechnology*, Vol. 25, No.4, pp. 243-250.

Written by: V. Krishnan Date: 07/14/16 Reviewed by: B. Gross Date: 07/22/16

Client: Santee Cooper Project: Winyah Generating Station Project No.: GSC5242 Task No.: 01BT

TABLE E-7.1. CHEMICAL COMPOSITION OF POREWATER SAMPLES FROM EIGHT CCR LANDFILLS (MODIFIED FROM EPRI, 1998)

Parameter	Units	Minimum	Maximum	Mean	Median	Std. Deviation	No. of Samples	% of ND Samples
Aluminum	mg/L	ND	763	29.1	1.5	113.8	102	11%
Arsenic	mg/L	ND	5.126	0.308	0.039	0.653	119	31%
Barium	mg/L	ND	12.10	0.24	0.10	1.10	123	26%
Boron	mg/L	ND	173.0	12.8	3.4	25.2	125	5%
Bromide	mg/L	ND	50.49	6.02	1.00	9.75	64	27%
Cadmium	mg/L	ND	0.40	0.062	0.008	0.097	105	68%
Calcium	mg/L	2.8	1318	414	503	276	124	0%
Chloride	mg/L	0.32	1384	77.1	12.0	176.4	123	0%
Chromium	mg/L	ND	6.72	0.211	0.069	0.810	112	52%
Copper	mg/L	ND	5.780	0.204	0.025	0.683	105	53%
Fluoride	mg/L	ND	3.9	1.228	1.010	1.030	86	42%
Iron	mg/L	ND	2540	267.6	0.20	615	93	35%
Lead	mg/L	ND	0.039	0.006	0.005	0.006	68	75%
Magnesium	mg/L	ND	614.5	48.9	8.0	85.5	123	8%
Manganese	mg/L	ND	24.0	2.433	0.026	5.044	110	34%
Molybdenum	mg/L	ND	6.7	1.4	1.0	1.6	105	50%
Nickel	mg/L	ND	8.40	0.423	0.043	1.147	103	36%
Nitrate	mg/L	ND	26.40	2.947	0.862	4.701	123	33%
Nitrite	mg/L	ND	5.082	0.714	0.600	0.819	101	94%
Phosphate	mg/L	ND	1.84	0.81	0.25	2.24	64	95%
Potassium	mg/L	ND	1436	107.1	35.3	235.6	116	3%
Selenium	mg/L	ND	4.930	0.308	0.053	0.746	65	22%
Silicon	mg/L	ND	84.10	13.11	5.58	16.78	119	3%
Silver	mg/L	ND	ND	ND	ND	ND	50	100%
Sodium	mg/L	ND	2875	200	46	396	123	2%
Strontium	mg/L	0.122	54.10	6.463	5.825	5.953	123	0%
Sulfate	mg/L	1.68	12567	2220	1611	2209	125	0%
Sulfite	mg/L	ND	47.97	7.06	1.50	9.51	88	80%
Vanadium	mg/L	ND	1.360	0.161	0.122	0.184	64	6%
Zinc	mg/L	ND	40.0	0.909	0.052	4.120	109	50%
pH	SU	2.80	12.57	NA	NA	NA	125	0%
Eh	mV	-72.8	684	NA	NA	NA	106	0%

NOTES:

1.) Transition metals are shown in italics.

2.) SU denotes Standard Units, mg/L denotes milligrams per liter, mV denotes millivolts.

3.) ND denotes Not Detected; NA indicates Not Available

Written by:	V. Krishnan	Date:	07/14/16	Reviewed by:	B. Gross	Date:	07/22/16
Client:	Santee Cooper	Project:	Winyah Generating Station	Project No.:	GSC5242	Task No.:	01BT

TABLE E-7.2. CHEMICAL COMPOSITION OF ASH AND FGD LEACHATES FROM 13 CCR LANDFILLS AND 15 SURFACE IMPOUNDMENTS (MODIFIED FROM EPRI, 2006)

		Ash Leachate					FGD Leachate				
Parameter	Units	Minimum	Maximum	Median	No. of Samples	% of ND Samples	Minimum	Maximum	Median	No. of Samples	% of ND Samples
Aluminum	mg/L	<0.002	44.4	0.114	67	16	<0.024	0.890	0.179	14	14
Antimony	mg/L	<0.0001	0.059	0.0024	67	3	<0.0001	0.022	0.001	14	29
Arsenic	mg/L	0.0014	1.380	0.025	67	0	0.011	0.230	0.028	14	0
Barium	mg/L	<0.018	0.657	0.108	67	4	<0.030	0.158	0.073	14	7
Beryllium	mg/L	<0.0002	0.0086	<0.0004	67	94	<0.0002	0.0015	<0.0008	14	93
Bicarbonate	mg/L	0.042	535	53	63	0	0.50	87	7.5	14	0
Boron	mg/L	0.207	112	2.16	67	0	1.45	98.5	9.61	14	0
Cadmium	mg/L	<0.0002	0.065	0.0015	67	12	0.0005	0.013	0.0018	14	0
Calcium	mg/L	<2.2	681	55	66	2	234	730	589	14	0
Carbonate	mg/L	<0.01	152	0.60	63	13	<0.010	21	1.0	14	21
Carbonic acid	mg/L	<0.01	3.4	<0.01	63	87	<0.010	0.041	<0.010	14	93
Chloride	mg/L	4.5	92	25	66	0	19	2330	921	14	0
Chromium	mg/L	<0.0002	5.10	0.0006	67	45	<0.0002	0.053	<0.0005	14	64
Cobalt	mg/L	<0.00004	0.133	0.001	67	31	<0.000028	0.078	0.001	14	36
Copper	mg/L	<0.0002	0.494	0.003	67	19	<0.00026	0.044	0.0026	14	14
Iron	mg/L	<0.003	25.6	<0.050	67	52	<0.0046	1.2	<0.050	14	71
Lead	mg/L	<0.0001	0.008	<0.0002	67	73	<0.00014	0.0035	<0.0002	14	64
Lithium	mg/L	<0.001	23.6	0.129	67	13	<0.020	7.07	3.06	14	14
Magnesium	mg/L	<0.05	236	13	66	8	<0.05	5810	8.9	14	14
Manganese	mg/L	<0.0001	4.17	0.055	67	21	<0.0001	1.17	0.113	14	14
Mercury	mg/L	0.00000025	0.000061	0.0000038	22	0	0.00000082	0.000079	0.0000083	8	0
Molybdenum	mg/L	<0.0082	39.6	0.405	67	3	0.164	60.8	0.341	14	0
Nickel	mg/L	<0.0006	0.189	0.0058	67	13	<0.002	0.597	0.0034	14	36
Potassium	mg/L	<2.2	277	11	66	3	10	609	425	14	0
Selenium	mg/L	0.000071	1.76	0.019	67	0	0.0011	2.36	0.0062	14	0
Silicon	mg/L	0.221	19.0	4.65	67	0	0.4	45.4	2.48	14	0
Silver	mg/L	<0.0002	0.002	<0.0002	67	93	<0.0002	<0.0002	<0.0002	14	100
Sodium	mg/L	3.8	3410	52	66	0	108	4630	322	14	0
Strontium	mg/L	<0.030	12	0.83	67	1	1.5	16.9	5.2	14	0
Sulfate	mg/L	45	6690	339	66	0	836	30,500	1615	14	0
Thallium	mg/L	<0.0001	0.018	0.00036	67	46	<0.0001	0.0029	<0.00022	14	86
Uranium	mg/L	<0.00001	0.061	0.0012	67	19	<0.00001	0.016	0.0002	14	36
Vanadium	mg/L	<0.00042	5.02	0.045	67	3	<0.00069	0.4	0.0041	14	21
Zinc	mg/L	<0.0015	0.289	0.005	67	46	<0.002	0.068	<0.005	14	57
pH	SU	4.3	12.0	7.9	64	0	6.2	12.0	9.0	14	0
ORP	mV	-41	411	241	63	2	1.5	356	201	14	0
EC	µmho/cm	174	12,760	990	64	0	2190	26,140	6461	14	0

NOTES:

1.) Transition metals are shown in italics.

2.) SU denotes Standard Units, mg/L denotes milligrams per liter, µmho/cm denotes micromhos per centimeter, mV denotes millivolts.

3.) Less than (<) indicates not detected, value given is detection limit

APPENDIX B

Calculations for Run-on and Run-off Control Systems

APPENDIX F-1

DRAINAGE ANALYSIS – HYDROLOGY

Written by: A. Sivashanthan Date: 6/30/2016 Reviewed by: B. Klenzendorf Date: 7/1/2016

Client: Santee-Cooper Project: Winyah Generating Station Project No.: GSC5242 Phase No.: 01

APPENDIX F-1

DRAINAGE ANALYSIS – HYDROLOGY

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SEALED FOR CALCULATION PAGES F1-1 THROUGH F1-38

1. INTRODUCTION

1.1 Purpose

The purpose of this calculation package is to present the hydrology analysis for the estimation of surface water runoff from the final surface water management system at the proposed Class Three Landfill (composed of Landfill Area 1 and Landfill Area 2) at the Winyah Generating Station (WGS) located in Georgetown County, South Carolina. The specific objectives of the hydrologic analysis include calculating peak discharges and total runoff volumes from the landfill areas during their final conditions that will exist at final buildout (i.e., post-development conditions after closure), when the final cover system for the landfill is in-place, as are the permanent surface water management system conveyance features. The results of these hydrologic calculations are used to design (size) the components of the landfill surface water management system. The hydraulic design of the landfill surface water management features is presented in other calculation packages that accompany the Engineering Report.

1.2 Surface Water Management System - Overview

The Engineering Drawing set that accompanies the Engineering Report shows the final grading plan of each landfill area, and also includes a series of drawings that shows the surface water management system features (identification, layout, and engineering details of the components). Key features of the surface water management system are as follows.

- The final cover system ground surface will be vegetated, and will have sideslopes inclined at three horizontal to one vertical (3H:1V) in-between drainage terraces and top surface slopes (top deck areas) are inclined at a nominal 3 to 5 percent slope.
- The final cover sideslopes have drainage terraces spaced approximately every 30 feet vertically, and with typical drainage profile slopes at 2 percent.
- The drainage terraces will convey water to downdrain pipes spaced periodically around each landfill area.
- Downdrain pipes will outlet into either constructed perimeter drainage channels, or will directly outlet into existing site drainage features (i.e., the discharge canal or cooling pond).
- Where constructed perimeter drainage channels are used, they will convey water to outlets into existing site drainage features (i.e., the intake canal, discharge canal, or cooling pond). Periodic drainage culverts will also be used along the drainage channels as necessary.

Figures 1 and 2 of this calculation package show the layout of the final cover grading of Landfill Areas 1 and 2, respectively, along with a delineation of drainage areas.

2. METHODOLOGY

2.1 HEC-HMS Computer Model

Surface water discharges are estimated using the Hydrologic Modeling System (HEC-HMS) computer program Version 4.1 developed through the Hydraulic Engineering Center (HEC) of the United States Army Corps of Engineers (USACE). The program simulates natural and controlled precipitation-runoff and routing processes of a watershed. HEC-

HMS is the successor to and replacement for the HEC-1 program (USACE, 2000). For precipitation-runoff-routing simulation, HEC-HMS provides the following components:

- Precipitation-specification options can describe an historical precipitation event, a frequency-based hypothetical precipitation event (i.e., design rainfall or storm event), or an event that represents the upper limit of precipitation possible at a given location. For this analysis, the following hypothetical precipitation events were evaluated: the 25-year (4% annual chance), 24-hour duration event (herein referred to as the 25-year, 24-hour event) and the 100-year (1% annual chance), 24-hour duration event (herein referred to as the 100-year, 24-hour event).

- Water loss models can estimate the volume of runoff given the precipitation and properties of the watershed. For this analysis, the Soil Conservation Service (SCS) Curve Number Loss Model was used (USDA, 1986).

- Direct runoff transform models can account for overland flow, storage, and energy losses as surface water runs off a watershed and into the drainage channels. For this analysis, the SCS Unit Hydrograph Model was selected.

- Hydraulic routing models account for storage and energy flux as surface water flows through drainage channels. The Kinematic Wave Model was selected for these analyses.

HEC-HMS was used to model the final landfill conditions. More specifically, HEC-HMS modeling calculates surface water runoff volumes, peak flow rates, and flow characteristics from landfill runoff that drains to and is routed through the perimeter channels. The results are used in separate calculation packages that also accompany the engineering report, which present the hydraulic design of the various landfill surface water management conveyances (terraces, downdrains, channels, culverts).

2.2 Estimation of Time of Concentration for HEC-HMS SCS Curve Number Method

The time of concentration is defined as the time for runoff to flow from the most hydraulically remote point of the drainage area to the point under investigation. The time of concentration (T_c) is a summation of sheet flow travel time, shallow concentrated flow travel time, and open channel flow travel time.

The method to estimate the sheet flow travel time was obtained from the U.S. Department of Agriculture (USDA) document *Urban Hydrology for Small Watersheds, Technical Release 55 (TR-55)* (USDA, 1986). Manning's kinematic solution is used for estimating travel time for sheet flow for flow distances less than 300 ft (USDA, 1986):

$$T_t = \frac{0.007(nL)^{0.8}}{P_{2-24}{}^{0.5} S^{0.4}}$$

where:	T_t	= travel time for overland sheet flow (hr);
	n	= roughness coefficient for sheet flow;
	L	= flow length (ft);
	$P_{2\text{-}24}$	= 2-year, 24-hour rainfall (in.); and
	S	= slope of hydraulic grade line (or land slope) (ft/ft).

The slope of the hydraulic grade line, or land slope (S), for all subcatchment areas of the final cover system is shown in Appendix 1 of this calculation package. To estimate sheet flow travel time (T_t), a roughness coefficient of 0.15 for short grass prairie surfaces was selected for the final cover system, and a roughness coefficient of 0.05 for fallow (no residue) was selected for the gravel access road as shown in Table 1 (USDA, 1986). Maximum flow lengths (L) were measured draining to each stormwater feature (i.e., top deck drainage terraces, sideslope drainage terraces, downdrain pipes, and perimeter drainage channels). As shown in Table 2 (NOAA, 2015), the rainfall depth for the 2-year, 24-hour frequency ($P_{2\text{-}24}$) at the site is 4.42 inches.

Based on the designed conveyance system, runoff will be converted from sheet flow to open channel flow relatively quickly, and therefore shallow concentrated flow should be negligible. Surface water runoff within each subcatchment area will sheet flow along the top deck or sideslopes of the final cover system until the water reaches either a drainage terrace or the perimeter drainage channel, at which point the flow will be classified as open channel flow.

The method selected to estimate the open channel flow travel time is based on guidance provided in TR-55 (USDA, 1986). Travel time for open channel flow is estimated by dividing the longest drainage path by the velocity of runoff:

$$T_t = \frac{L}{V}\left(\frac{1}{60}\right)$$

where: T_t = travel time (min);

L = flow length (ft); and

V = average velocity (ft/sec).

Open channel flow velocities were estimated using Manning's equation based on guidance provided in TR-55 (USDA, 1986). The average flow velocities were determined for bank-full elevation as:

$$V = \frac{1.49}{n} R_h^{2/3} S^{1/2}$$

where: V = average velocity (ft/s);

n = Manning's roughness coefficient;

R_h = hydraulic radius (ft) = A/P;

A = cross sectional area (ft^2);

P = wetted perimeter (ft); and

S = slope of hydraulic grade line (or longitudinal channel slope for normal flow conditions) (ft/ft).

To estimate open channel flow travel time (T_t), a Manning's roughness coefficient (n) was selected for clean and straight earthen open channels as shown in Table 3 (Chow, 1959).

The velocities and times of concentration used in the design are presented in Appendix 1 of this calculation package. A minimum time of concentration of six minutes was used as recommended by TR-55 (USDA, 1986) because small areas with exceedingly short times of concentration could result in design rainfall intensities that are unrealistically high. The lag times calculated for each drainage area are presented in Appendix 1 for use in the SCS Curve Number Method and HEC-HMS software. The lag time is estimated as 0.6 times the time of concentration (USDA, 1986).

3. DESIGN PARAMETERS

The following data and assumptions were utilized to select the relevant engineering parameters to estimate surface water runoff.

3.1 Rainfall

The rainfall depths corresponding to 24-hour duration hypothetical precipitation event and 25- and 100-year frequency return periods for the site are 8.28 inches, 10.9 inches, respectively, as shown in Table 2 (NOAA, 2015). The design storm hyetograph is defined using a SCS Type III rainfall distribution, which is selected based on Figure 3 (USDA, 1986).

3.2 Drainage Areas and Reaches

The drainage areas and reaches were modeled based on the following approach and related assumptions:

- Drainage Areas – The contributing watershed areas for each basin (drainage area) or reach (perimeter drainage channel) associated with the landfill areas are divided into multiple subbasins (subareas). Subbasins are modeled based on the receiving surface water drainage feature and are delineated for the areas draining to perimeter drainage channel reaches. The SCS Curve Number Loss Model was used to estimate the volume of runoff from a given subbasin. The SCS Unit Hydrograph Model was used to estimate the direct runoff flow rates from each subbasin. Each subbasin is assigned a curve number representing the type of ground cover for a given soil for the area. The subbasin area, curve number, and SCS Unit Hydrograph lag time input parameters are included in the HEC-HMS output in Appendix 1.

- Hydrologic Soil Groups (HSGs) – Figure 4 shows a soil map of the immediate site area, taken from the Web Soil Survey tool operated by the USDA Natural Resources Conservation Service (USDA, 2016) for Georgetown County. The predominant soil type at the site are Udorthents, loamy (map unit symbol 58) and Eulonia loamy fine sand (map unit symbol 26A). The on-site soil types have HSG designations as shown in Table 4 (USDA, 2016). To be conservative, soil for the final cover system of the landfill areas is assumed to be a HSG type D soil, which generally provides the highest calculated runoff volumes.

- Curve Number (CN) – Curve numbers are obtained from the TR-55 (USDA, 1986) and are based on the predominant HSG of the drainage area. Table 5 summarizes the CNs chosen for the analyses performed documented within this calculation package. Proposed final cover of the CCR landfill (HSG type D) is generally assumed to be open space (pasture, grassland, or range) with good grass cover conditions (CN = 80).

- Manning's Roughness Coefficients – Values of Manning's roughness coefficients used in the reach routing calculations were obtained from Chow (1959) and Barfuss and Tullis (1989). Table 3 and Figure 5 summarize the Manning's coefficients used in this calculation package. A Manning's roughness coefficient value of 0.027 was selected for the drainage terraces which are proposed to be grass-lined and a value of 0.036 was selected for the drainage terraces which are proposed to be lined with riprap. A Manning's roughness coefficient value of 0.009 was selected for HDPE down drain pipes.

- Perimeter Channel Reaches – Reaches in the HEC-HMS program represent perimeter channels that route surface water from upstream subbasins to downstream junctions. Reaches also may route surface water from upstream reaches. The Kinematic Wave Model is used to model the surface water flow in each of the reaches in the HEC-HMS program. The Kinematic Wave Model accounts for storage and energy flux as surface water moves through stream channels. Average geometric characteristics of the stream channel measured from the existing and proposed topography are input into HEC-HMS.

3.3 Nodal Network Diagrams

Figure 6 of this calculation package presents the nodal network diagrams for the final landfill conditions of both landfill areas (Area 1 and Area 2). The nodal network diagrams show the subbasins, reaches, and discharge locations.

4. RESULTS

Table 6 of this calculation package presents a summary of the HEC-HMS hydrologic modeling results. Additional detailed backup, including screen-shots of the modeling output results, is provided in Appendix 1 of this calculation package. The summary of results in Table 6 shows the calculated peak discharges and total discharge runoff volumes at the landfill outfall locations (which refers to locations where runoff outlets to the existing on-site cooling pond) for both Landfill Area 1 and Landfill Area 2. Results are provided for both the 25-year and 100-year storm events. As previously mentioned, results of these hydrologic calculations are used to design (size) the components of the landfill surface water management system. The hydraulic design of the landfill surface water management features is presented in other calculation packages that accompany the Engineering Report.

5. REFERENCES

Barfuss, S.L. and Tullis, J.P. (1989). *"Friction Factor Tests on 300 mm High Density Polyethylene Pipe"*, Hydraulics Report No. 224, Utah Water Research Laboratory, Paper 611.

Chow, V.T. (1959). *Open-Channel Hydraulics*, McGraw-Hill Book Company, Inc., New York, NY.

HydroCAD (2011). *HydroCAD Stormwater Modeling System Version 10 Owner's Manual*, HydroCAD Software Solutions, Chocorua, NH.

Georgetown County (2000). *Georgetown County Storm Water Design Manual*, adopted 5 September 2000.

NOAA (2015). *Precipitation-Frequency Atlas of the United States.* Atlas 14, Volume 2, Version 3.0, National Oceanic and Atmospheric Administration.

USDA (2016). Web Soil Survey, Soil Survey Staff, Natural Resources Conservation Service (NRCS), United States Department of Agriculture, available online at http://websoilsurvey.nrcs.usda.gov, accessed June 2016.

USACE (2000). *Hydraulic Modeling System HEC-HMS Technical Reference Manual*, US Army Corps of Engineers, Hydrologic Engineering Center, CPD-74B, March 2000.

USDA (1986). *Urban Hydrology for Small Watersheds*, Technical Release 55 (TR-55), United States Department of Agriculture, Science and Education Administration, Agriculture Handbook Number 537.

TABLES

- Table 1. Roughness Coefficient for Sheet Flow (from USDA, 1986)
- Table 2. Precipitation Frequency for Georgetown, South Carolina (from NOAA, 2015)
- Table 3. Manning's n Values for Open Channels (from Chow, 1959)
- Table 4. Hydrologic Soil Groups for On-Site Soils (from USDA, 2016)
- Table 5. Summary of Runoff Curve Numbers (from USDA, 1986)
- Table 6. Summary of Peak Flow Rates and Total Runoff Volumes at Landfill Outfall Locations

Table 1 – Roughness Coefficient for Sheet Flow

(from USDA, 1986)

Surface description	n [1/]
Smooth surfaces (concrete, asphalt, gravel, or bare soil)	0.011
Fallow (no residue)	0.05
Cultivated soils:	
Residue cover ≤20%	0.06
Residue cover >20%	0.17
Grass:	
Short grass prairie	0.15
Dense grasses [2/]	0.24
Bermudagrass	0.41
Range (natural)	0.13
Woods:[3/]	
Light underbrush	0.40
Dense underbrush	0.80

1 The n values are a composite of information compiled by Engman (1986).

2 Includes species such as weeping lovegrass, bluegrass, buffalo grass, blue grama grass, and native grass mixtures.

3 When selecting n , consider cover to a height of about 0.1 ft. This is the only part of the plant cover that will obstruct sheet flow.

Table 2. Precipitation Frequency for Georgetown, South Carolina (from NOAA, 2015)

NOAA Atlas 14, Volume 2, Version 3
Location name: Georgetown, South Carolina, US*
Latitude: 33.3343°, Longitude: -79.3551°
Elevation: 26 ft*
* source: Google Maps

POINT PRECIPITATION FREQUENCY ESTIMATES

G.M. Bonnin, D. Martin, B. Lin, T. Parzybok, M.Yekta, and D. Riley

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)[1]

Duration	Average recurrence interval (years)									
	1	2	5	10	25	50	100	200	500	1000
5-min	0.502 (0.468–0.543)	0.586 (0.545–0.636)	0.668 (0.619–0.722)	0.761 (0.703–0.823)	0.857 (0.789–0.927)	0.942 (0.863–1.02)	1.02 (0.929–1.10)	1.10 (0.993–1.19)	1.19 (1.07–1.29)	1.28 (1.14–1.39)
10-min	0.803 (0.748–0.868)	0.938 (0.872–1.02)	1.07 (0.992–1.16)	1.22 (1.13–1.32)	1.37 (1.26–1.48)	1.50 (1.38–1.62)	1.62 (1.48–1.75)	1.74 (1.57–1.88)	1.89 (1.69–2.04)	2.02 (1.79–2.19)
15-min	1.00 (0.935–1.09)	1.18 (1.10–1.28)	1.35 (1.25–1.46)	1.54 (1.42–1.67)	1.73 (1.59–1.87)	1.90 (1.74–2.05)	2.05 (1.87–2.22)	2.19 (1.99–2.37)	2.37 (2.13–2.57)	2.53 (2.25–2.75)
30-min	1.38 (1.28–1.49)	1.63 (1.51–1.77)	1.92 (1.78–2.08)	2.23 (2.06–2.41)	2.57 (2.36–2.77)	2.86 (2.62–3.09)	3.14 (2.86–3.39)	3.42 (3.09–3.69)	3.78 (3.38–4.09)	4.10 (3.64–4.45)
60-min	1.72 (1.60–1.85)	2.04 (1.90–2.22)	2.46 (2.29–2.67)	2.90 (2.68–3.14)	3.42 (3.14–3.69)	3.88 (3.55–4.19)	4.32 (3.94–4.67)	4.79 (4.34–5.18)	5.42 (4.86–5.87)	5.98 (5.32–6.50)
2-hr	2.08 (1.92–2.24)	2.49 (2.30–2.69)	3.07 (2.83–3.31)	3.66 (3.37–3.96)	4.37 (4.00–4.72)	5.01 (4.56–5.40)	5.63 (5.10–6.07)	6.27 (5.64–6.76)	7.10 (6.33–7.68)	7.85 (6.93–8.51)
3-hr	2.23 (2.06–2.42)	2.66 (2.45–2.91)	3.30 (3.03–3.59)	3.97 (3.64–4.33)	4.79 (4.37–5.21)	5.56 (5.02–6.03)	6.31 (5.66–6.84)	7.10 (6.33–7.71)	8.19 (7.20–8.91)	9.18 (7.98–10.0)
6-hr	2.66 (2.44–2.91)	3.19 (2.91–3.49)	3.94 (3.60–4.32)	4.75 (4.33–5.20)	5.76 (5.21–6.30)	6.69 (6.01–7.31)	7.61 (6.79–8.32)	8.62 (7.62–9.41)	9.97 (8.71–10.9)	11.2 (9.69–12.3)
12-hr	3.11 (2.84–3.44)	3.72 (3.30–4.11)	4.63 (4.21–5.12)	5.62 (5.10–6.20)	6.85 (6.17–7.54)	8.00 (7.15–8.78)	9.17 (8.12–10.0)	10.4 (9.15–11.4)	12.2 (10.5–13.4)	13.8 (11.8–15.2)
24-hr	3.64 (3.33–3.99)	4.42 (4.06–4.85)	5.71 (5.22–6.26)	6.77 (6.17–7.41)	8.28 (7.50–9.05)	9.53 (8.59–10.4	10.9 (9.74–11.9)	12.3 (11.3–13.5)	14.4 (12.7–15.8)	16.1 (14.1–17.7)

**Table 3. Manning's n Values for Open Channels
(from Chow, 1959)**

Type of channel and description	Minimum	Normal	Maximum
B. LINED OR BUILT-UP CHANNELS			
B-1. Metal			
a. Smooth steel surface			
1. Unpainted	0.011	0.012	0.014
2. Painted	0.012	0.013	0.017
b. Corrugated	0.021	0.025	0.030
B-2. Nonmetal			
e. Gravel bottom with sides of			
1. Formed concrete	0.017	0.020	0.025
2. Random stone in mortar	0.020	0.023	0.026
3. Dry rubble or riprap	0.023	0.033	0.036
C. EXCAVATED OR DREDGED			
a. Earth, straight and uniform			
1. Clean, recently completed	0.016	0.018	0.020
2. Clean, after weathering	0.018	0.022	0.025
3. Gravel, uniform section, clean	0.022	0.025	0.030
4. With short grass, few weeds	0.022	0.027	0.033
b. Earth, winding and sluggish			
1. No vegetation	0.023	0.025	0.030
2. Grass, some weeds	0.025	0.030	0.033
3. Dense weeds or aquatic plants in deep channels	0.030	0.035	0.040
4. Earth bottom and rubble sides	0.028	0.030	0.035
5. Stony bottom and weedy banks	0.025	0.035	0.040
6. Cobble bottom and clean sides	0.030	0.040	0.050

Table 4. Hydrologic Soil Groups for On-Site Soils (from USDA, 2016)

Hydrologic Soil Group— Summary by Map Unit — Georgetown County, South Carolina (SC043)				
Map unit symbol	Map unit name	Rating	Acres in AOI	Percent of AOI
10	Leon sand, 0 to 2 percent slopes	A/D	198.2	4.3%
12A	Yauhannah loamy fine sand, 0 to 2 percent slopes	B/D	638.1	14.0%
13	Bladen loam	C/D	336.6	7.4%
18	Cape Fear loam	C/D	15.1	0.3%
20	Centenary fine sand	A	186.7	4.1%
24B	Chisolm sand, 0 to 4 percent slopes	A	55.0	1.2%
25A	Wakulla sand, 0 to 2 percent slopes	A	36.3	0.8%
26A	Eulonia loamy fine sand, 0 to 2 percent slopes	C/D	551.9	12.1%
27	Rutlege sand	A/D	89.1	2.0%
28	Echaw sand	A	503.0	11.0%
33	Hobonny muck	A/D	55.0	1.2%
34	Johnston loam	A/D	131.5	2.9%
36B	Lakeland fine sand, 0 to 6 percent slopes	A	5.4	0.1%
50	Lynn Haven sand	A/D	42.2	0.9%
54A	Chipley fine sand, 0 to 2 percent slopes	A	4.2	0.1%
55	Witherbee fine sand	A/D	447.4	9.8%
57	Grifton loamy fine sand	B/D	80.2	1.8%
58	Udorthents, loamy	B/D	410.4	9.0%
59	Wahee fine sandy loam	C/D	484.4	10.6%
61	Yemassee loamy fine sand	B/D	39.0	0.9%
W	Water		254.9	5.6%
Totals for Area of Interest			**4,564.7**	**100.0%**

Table 5. Summary of Runoff Curve Numbers[1] (from USDA, 1986)

Cover description		Curve numbers for hydrologic soil group			
Cover type	Hydrologic condition	A	B	C	D
Pasture, grassland, or range—continuous forage for grazing. [2]	Poor	68	79	86	89
	Fair	49	69	79	84
	Good	39	61	74	80
Meadow—continuous grass, protected from grazing and generally mowed for hay.	—	30	58	71	78
Brush—brush-weed-grass mixture with brush the major element. [3]	Poor	48	67	77	83
	Fair	35	56	70	77
	Good	30 [4]	48	65	73
Woods—grass combination (orchard or tree farm). [5]	Poor	57	73	82	86
	Fair	43	65	76	82
	Good	32	58	72	79
Woods. [6]	Poor	45	66	77	83
	Fair	36	60	73	79
	Good	30 [4]	55	70	77
Farmsteads—buildings, lanes, driveways, and surrounding lots.	—	59	74	82	86

1 Average runoff condition, and $I_a = 0.2S$.

2 *Poor:* <50%) ground cover or heavily grazed with no mulch.
Fair: 50 to 75% ground cover and not heavily grazed.
Good: > 75% ground cover and lightly or only occasionally grazed.

3 *Poor:* <50% ground cover.
Fair: 50 to 75% ground cover.
Good: >75% ground cover.

4 Actual curve number is less than 30; use CN = 30 for runoff computations.

5 CN's shown were computed for areas with 50% woods and 50% grass (pasture) cover. Other combinations of conditions may be computed from the CN's for woods and pasture.

6 *Poor:* Forest litter, small trees, and brush are destroyed by heavy grazing or regular burning.
Fair: Woods are grazed but not burned, and some forest litter covers the soil.
Good: Woods are protected from grazing, and litter and brush adequately cover the soil.

Table 6. Summary of Peak Flow Rates and Total Runoff Volumes at Landfill Outfall Locations

Landfill	Outfall	Design Storm	Peak Flow (cfs)	Total Runoff (ac-ft)
Area 1	1A	25-year	143.9	13.0
		100-year	203.4	18.5
	1C	25-year	59.5	5.3
		100-year	83.3	7.5
Area 2	2A	25-year	89.5	7.6
		100-year	125.3	10.8
	2B	25-year	35.1	2.9
		100-year	49.2	4.1
	2C	25-year	102.7	8.6
		100-year	144.1	12.2
	2D	25-year	23.6	1.9
		100-year	33.0	2.7
	2E	25-year	49.0	4.0
		100-year	68.6	5.7
	2F	25-year	187.1	15.8
		100-year	262.1	22.5

FIGURES

- Figure 1. Landfill Area 1 Drainage Plan
- Figure 2. Landfill Area 2 Drainage Plan
- Figure 3. SCS Rainfall Distributions (from USDA, 1986)
- Figure 4. Soil Survey Map (from USDA, 2016)
- Figure 5. Manning's n for HDPE (from Barfuss and Tullis, 1989)
- Figure 6. HEC-HMS Nodal Network

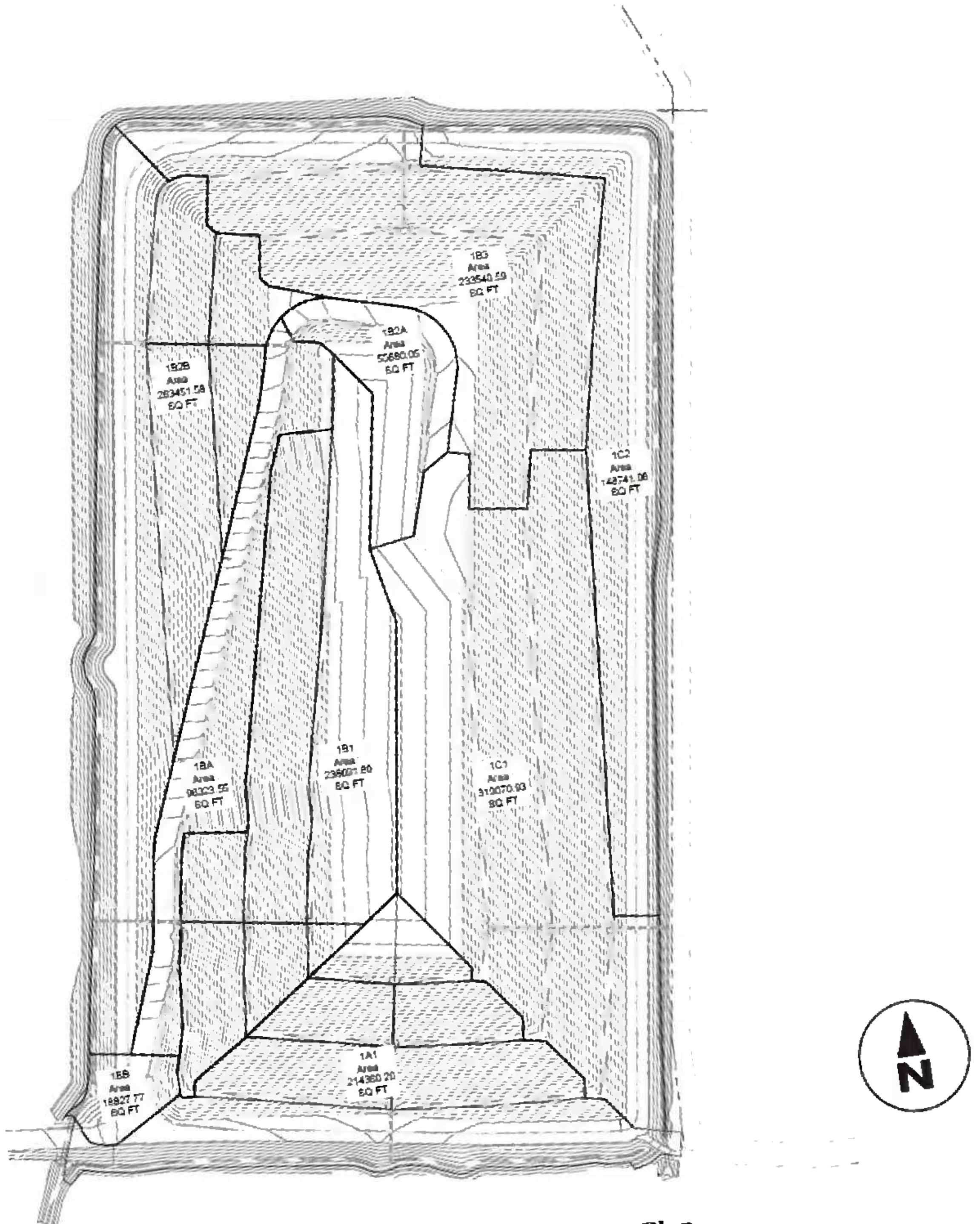


Figure 1. Landfill Area 1 Drainage Plan

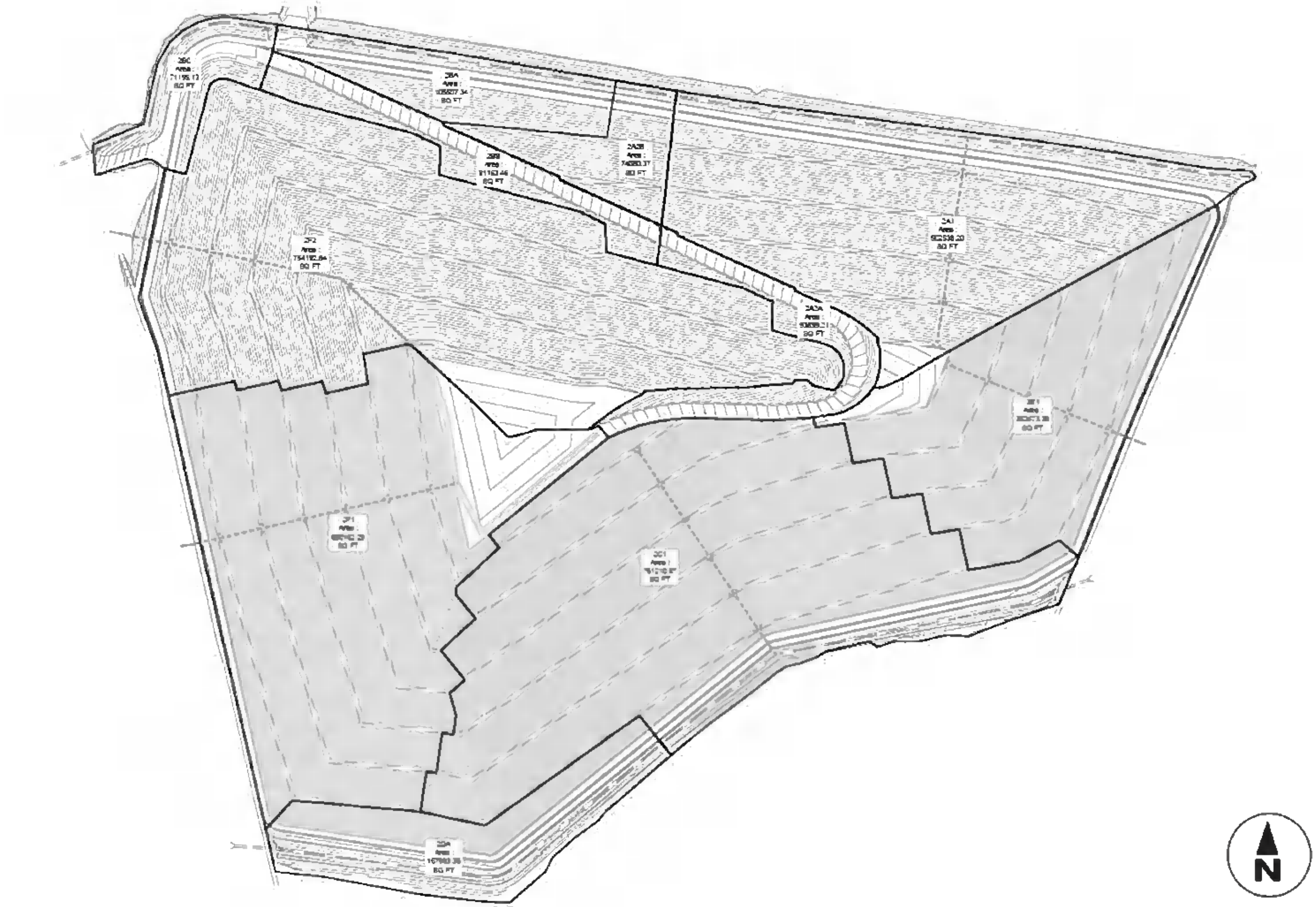


Figure 2. Landfill Area 2 Drainage Plan

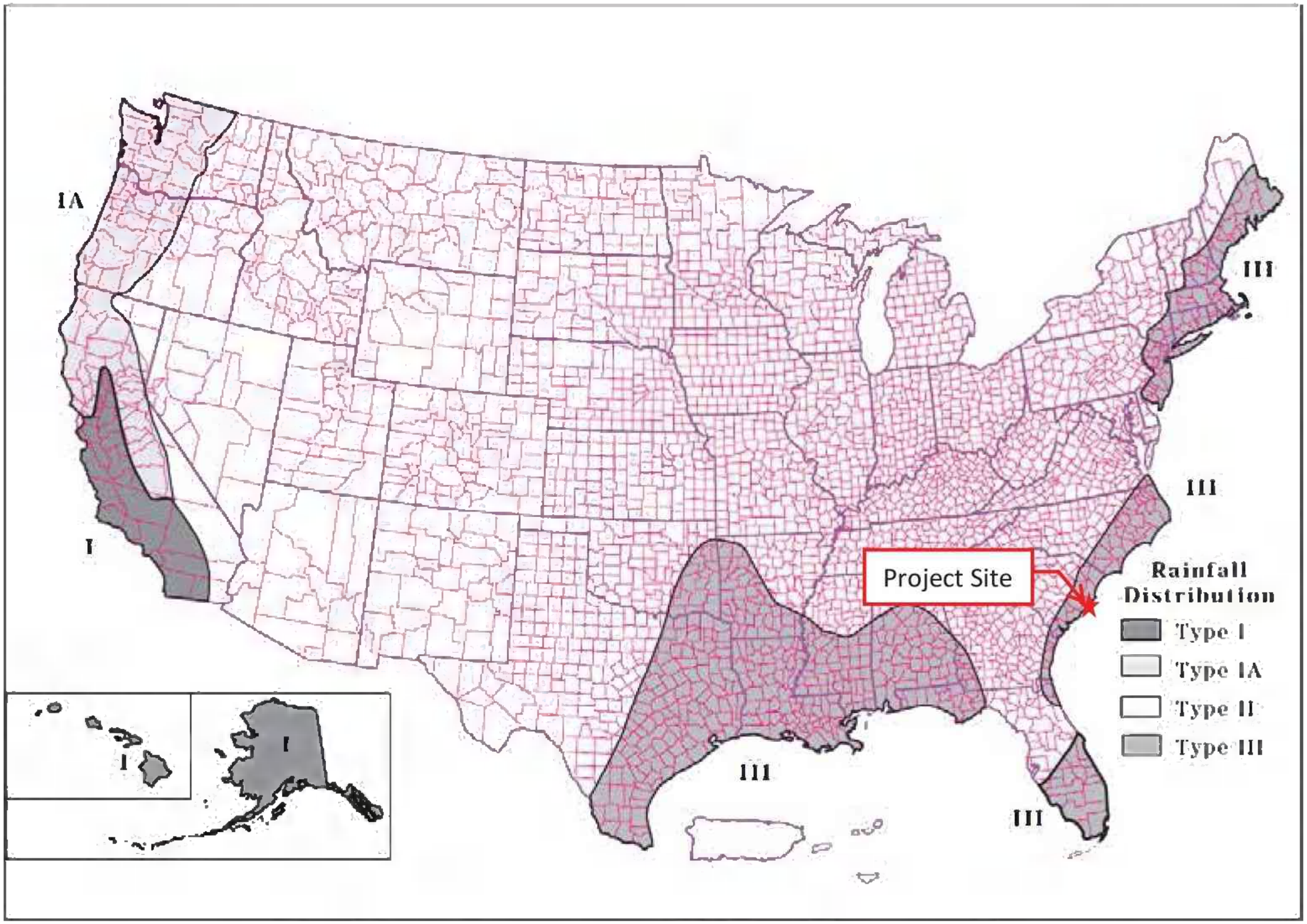


Figure 3. SCS Rainfall Distributions (from USDA, 1986)

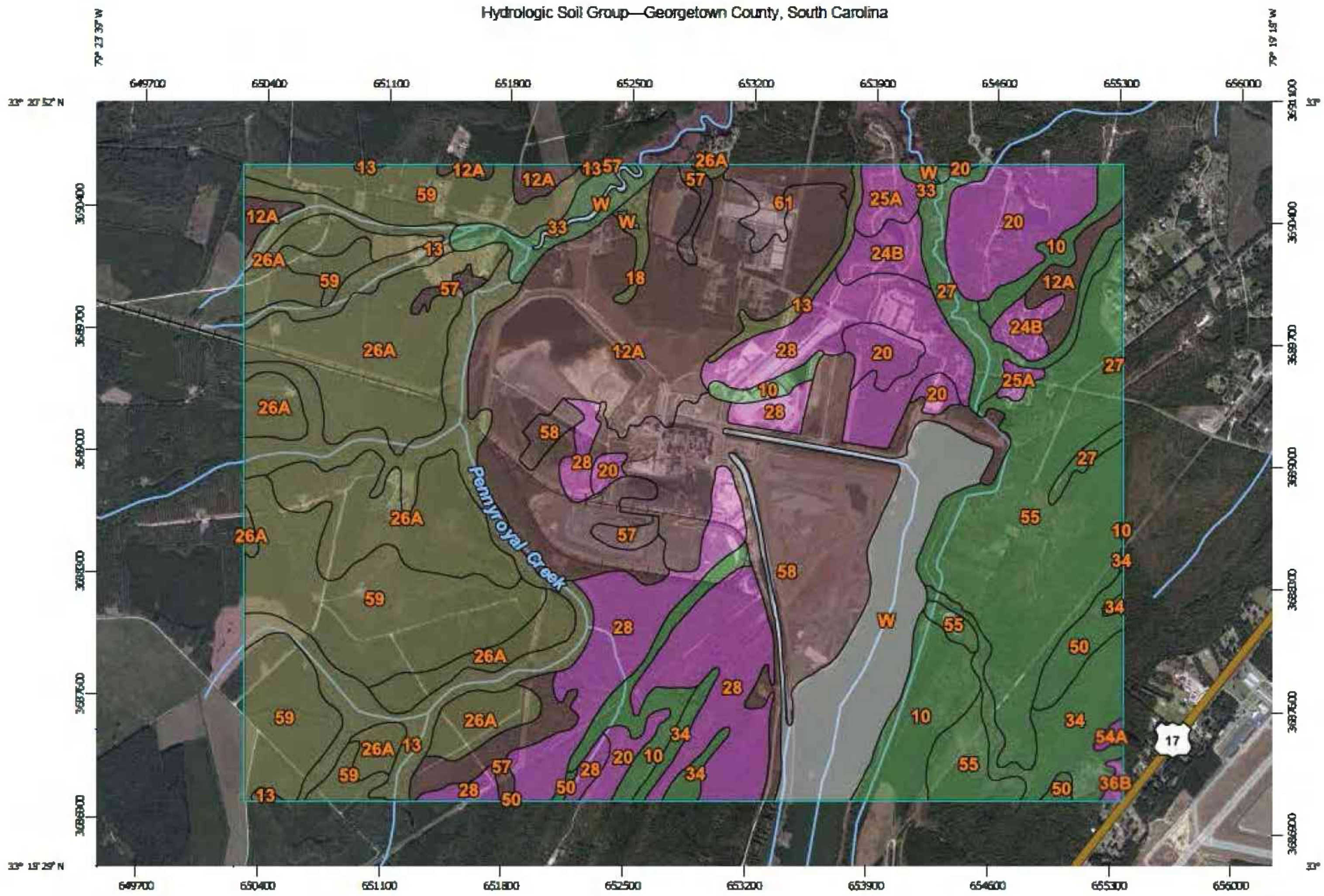
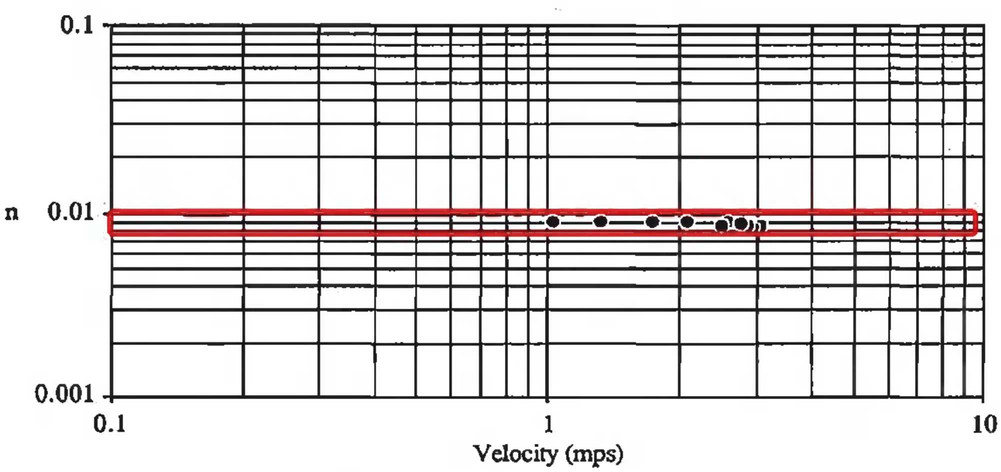


Figure 4. Soil Survey Map (from USDA, 2016)



Manning n vs. Velocity for 300mm Diameter HDPE Pipe

Figure 5. Manning's n for HDPE

(from Barfuss and Tullis, 1989)

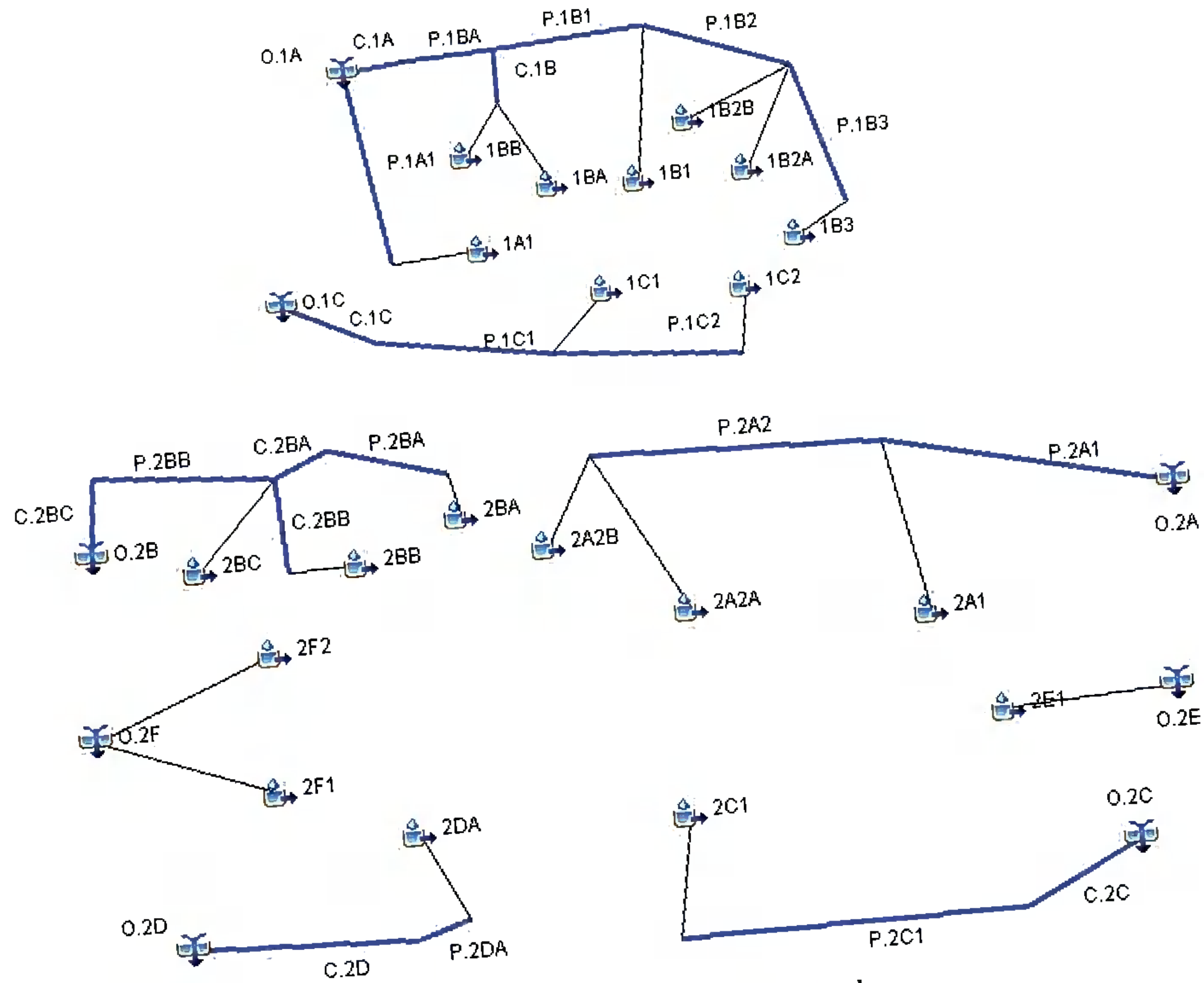


Figure 6. HEC-HMS Nodal Network

APPENDIX 1
HEC-HMS HYDROLOGIC MODEL PARAMETERS

Table 1-1. HEC-HMS Basin Input Parameters for Kinematic Wave Model

POST-DEVELOPMENT CONDITIONS			Watershed Characterization			Sheet Flow				Open Channel Flow - Top Deck or Side Slope Drainage Terrace								
Subcatchment Designation	Area A (mi^2)	Area A (acres)	Initial Abstraction (in)	Curve Number	Impervious Cover (%)	Flow Length (ft)	Manning's n	Slope (ft/ft)	Time T_t (min)	Flow Length (ft)	Depth d (ft)	Area A (ft^2)	Wetted P (ft)	Hydraulic Radius (ft)	Manning's n	Slope (ft/ft)	Velocity (ft/s)	Time T_t (min)
1A1	0.00769	4.92	0.50	80	0.00	108	0.15	0.05	7.57	0	1.0	4.00	8.26	0.48	0.027	0.020	4.81	0.00
1BA	0.00346	2.21	0.50	80	0.00	92	0.15	0.33	2.53	1065	1.0	2.50	5.40	0.46	0.036	0.080	7.01	2.53
1BB	0.00068	0.43	0.50	80	0.00	42	0.15	0.33	1.35	0	1.0	4.00	8.26	0.48	0.027	0.020	4.81	0.00
1B1	0.00856	5.48	0.50	80	0.00	60	0.15	0.05	3.84	370	1.0	11.50	23.19	0.50	0.027	0.005	2.44	2.52
1B2A	0.00200	1.28	0.50	80	0.00	85	0.15	0.05	5.97	149	1.0	2.50	5.40	0.46	0.036	0.080	7.01	0.35
1B2B	0.01070	6.85	0.50	80	0.00	85	0.15	0.05	7.34	68	1.0	4.00	8.26	0.48	0.027	0.020	4.81	0.24
1B3	0.00838	5.36	0.50	80	0.00	97	0.15	0.03	9.62	85	1.0	4.00	8.26	0.48	0.027	0.020	4.81	0.29
1C1	0.01144	7.32	0.50	80	0.00	172	0.15	0.05	8.92	527	1.0	4.00	8.26	0.48	0.027	0.020	4.81	2.53
1C2	0.00533	3.41	0.50	80	0.00	75	0.15	0.33	2.15	0	1.0	4.00	8.26	0.48	0.027	0.005	2.41	0.00
2A1	0.01803	11.54	0.50	80	0.00	126	0.15	0.05	6.95	80	1.0	4.00	8.26	0.48	0.027	0.020	4.81	0.28
2A2A	0.00336	2.15	0.50	80	0.00	34	0.05	0.02	1.46	1233	1.0	2.50	5.40	0.46	0.036	0.080	7.01	2.93
2A2B	0.00266	1.70	0.50	80	0.00	89	0.15	0.33	2.46	230	1.0	4.00	8.26	0.48	0.027	0.020	4.81	0.80
2BA	0.00378	2.42	0.50	80	0.00	163	0.15	0.33	4.00	0	1.0	4.00	8.26	0.48	0.027	0.020	4.81	0.00
2BB	0.00291	1.86	0.50	80	0.00	34	0.05	0.02	1.46	997	1.0	2.50	5.40	0.46	0.036	0.080	7.01	2.37
2BC	0.00255	1.63	0.50	80	0.00	92	0.15	0.33	7.32	0	1.0	4.00	8.26	0.48	0.027	0.020	4.81	0.00
2C1	0.02730	17.47	0.50	80	0.00	93	0.15	0.33	2.55	937	1.0	4.00	8.26	0.48	0.027	0.020	4.81	3.25
2DA	0.00602	3.85	0.50	80	0.00	84	0.15	0.33	2.35	0	1.0	4.00	8.26	0.48	0.027	0.020	4.81	0.00
2E1	0.01264	8.09	0.50	80	0.00	118	0.15	0.05	6.60	73	1.0	4.00	8.26	0.48	0.027	0.020	4.81	0.25
2F1	0.02332	14.93	0.50	80	0.00	121	0.15	0.05	6.73	305	1.0	4.00	8.26	0.48	0.027	0.005	2.41	2.11
2F2	0.02705	17.31	0.50	80	0.00	161	0.15	0.05	8.46	264	1.0	4.00	8.26	0.48	0.027	0.020	4.81	0.91

2-year, 24-hour Design Rainfall Depth = 4.42 inches

Top Deck Drainage Terrace Left Side Slope =	20.0	H:V
Top Deck Drainage Terrace Right Side Slope =	3.0	H:V
Side Slope Drainage Terrace Left Side Slope =	5.0	H:V
Side Slope Drainage Terrace Right Side Slope =	3.0	H:V
Access Road Drainage Terrace Left Side Slope =	2.0	H:V
Access Road Drainage Terrace Right Side Slope =	3.0	H:V

Notes:

1) *Curve number = 80 represents pasture, grassland, or range with good ground cover for hydrologic soil group D (USDA, 1986).*
2) *Manning's roughness coefficient: n = 0.15 represents short grass prairie for sheet flow (USDA, 1984).*
3) *Manning's roughness coefficient: n = 0.027 represents an excavated earth channel that is straight and uniform with short grass and few weeds (Chow, 1959).*
4) *Travel Time (T_t) is calculated using Manning's kinematic solutions for sheet flow (USDA, 1986).*
   $T_t = 0.007(nL)^{0.8} / (P_{2\text{-}24})^{0.5} S^{0.4}$
5) *Velocity factor of 7.0 ft/s corresponds to a value between short grass pasture and woodland from the Upland Method as reported by HydroCAD v.10 Owner's Manual.*
6) *Open channel flow velocity is calculated using Manning's equation (USDA, 1986).*
   $V = (1.49r^{2/3} S^{1/2}) / n$ where: r = hydraulic radius (ft) and is equal to A/P [area (ft^2)/wetted perimeter (ft)]
7) *Design rainfall depth taken from NOAA Atlas 14, Volume 2, Version 3 for Georgetown, South Carolina.*

Table 1-1 (Continued). HEC-HMS Basin Input Parameters for Kinematic Wave Model

	Down Drain Pipe Flow								Open Channel Flow - Perimeter Channel												
Subcatchment Designation	Flow Length (ft)	Area A (ft²)	Wetted P (ft)	Hydraulic Radius (ft)	Manning's n	Slope (ft/ft)	Velocity (ft/s)	Time T_t (min)	Flow Length (ft)	Depth d (ft)	Area A (ft²)	Wetted P (ft)	Hydraulic Radius (ft)	Manning's n	Slope (ft/ft)	Velocity (ft/s)	Time T_t (min)	Design T_c (min)	SCS Lag Time (min)	HMS 25-yr Flow (cfs)	HMS 100-yr Flow (cfs)
1A1	291	1.77	4.71	0.38	0.009	0.333	49.71	0.10	562	2.0	18.00	15.65	1.15	0.027	0.0050	4.28	2.19	9.86	5.92	28.30	39.60
1BA	96	1.77	4.71	0.38	0.009	0.050	19.25	0.08	0	2.0	18.00	15.65	1.15	0.027	0.0050	4.28	0.00	6.00	3.60	13.70	19.10
1BB	0	1.77	4.71	0.38	0.009	0.333	49.71	0.00	113	2.0	18.00	15.65	1.15	0.027	0.0050	4.28	0.44	6.00	3.60	2.70	3.80
1B1	349	1.77	4.71	0.38	0.009	0.333	49.71	0.12	484	2.0	16.00	16.52	0.97	0.027	0.0050	3.82	2.11	8.59	5.15	32.30	45.10
1B2A	0	1.77	4.71	0.38	0.009	0.333	49.71	0.00	0	2.0	18.00	15.65	1.15	0.027	0.0050	4.28	0.00	6.32	3.79	7.90	11.00
1B2B	181	1.77	4.71	0.38	0.009	0.333	49.71	0.06	0	2.0	18.00	15.65	1.15	0.027	0.0050	4.28	0.00	7.63	4.58	41.10	57.50
1B3	168	1.77	4.71	0.38	0.009	0.333	49.71	0.06	406	2.0	18.00	15.65	1.15	0.027	0.0050	4.28	1.58	11.55	6.93	29.80	41.80
1C1	300	1.77	4.71	0.38	0.009	0.333	49.71	0.10	329	2.0	18.00	15.65	1.15	0.027	0.0050	4.28	1.28	12.82	7.69	39.70	55.70
1C2	0	1.77	4.71	0.38	0.009	0.333	49.71	0.00	1600	2.0	18.00	15.65	1.15	0.027	0.0050	4.28	6.22	8.37	5.02	20.10	28.20
2A1	498	1.77	4.71	0.38	0.009	0.333	49.71	0.17	680	2.0	18.00	15.65	1.15	0.027	0.0050	4.28	2.65	10.04	6.03	66.00	92.50
2A2A	0	1.77	4.71	0.38	0.009	0.333	49.71	0.00	0	2.0	18.00	15.65	1.15	0.027	0.0050	4.28	0.00	6.00	3.60	13.30	18.60
2A2B	156	1.77	4.71	0.38	0.009	0.333	49.71	0.05	0	2.0	18.00	15.65	1.15	0.027	0.0050	4.28	0.00	6.00	3.60	10.50	14.70
2BA	0	1.77	4.71	0.38	0.009	0.333	49.71	0.00	423	2.0	18.00	15.65	1.15	0.027	0.0050	4.28	1.65	6.00	3.60	14.90	20.90
2BB	95	1.77	4.71	0.38	0.009	0.050	19.25	0.08	0	2.0	18.00	15.65	1.15	0.027	0.0050	4.28	0.00	6.00	3.60	11.50	16.10
2BC	0	1.77	4.71	0.38	0.009	0.333	49.71	0.00	729	2.0	18.00	15.65	1.15	0.027	0.0050	4.28	2.84	10.16	6.09	9.30	13.00
2C1	85	1.77	4.71	0.38	0.009	0.333	49.71	0.03	646	2.0	18.00	15.65	1.15	0.027	0.0050	4.28	2.51	8.34	5.00	103.30	144.70
2DA	0	1.77	4.71	0.38	0.009	0.333	49.71	0.00	981	2.0	18.00	15.65	1.15	0.027	0.0050	4.28	3.82	6.17	3.70	23.70	33.20
2E1	504	1.77	4.71	0.38	0.009	0.333	49.71	0.17	0	2.0	18.00	15.65	1.15	0.027	0.0050	4.28	0.00	7.02	4.21	49.00	68.60
2F1	668	1.77	4.71	0.38	0.009	0.333	49.71	0.22	0	2.0	18.00	15.65	1.15	0.027	0.0050	4.28	0.00	9.07	5.44	87.20	122.00
2F2	745	1.77	4.71	0.38	0.009	0.333	49.71	0.25	0	2.0	18.00	15.65	1.15	0.027	0.0050	4.28	0.00	9.62	5.77	100.00	140.10

Down Drain Pipe Diameter = 1.5 ft

Perimeter Channel Left Side Slope = 3.0 H:V
Perimeter Channel Right Side Slope = 3.0 H:V
Perimeter Channel Base Width = 3.00 ft

Table 1-2. 25-year, 24-hour Precipitation Event Nodal Areas, Peak Flow Rates, and Runoff Volumes for Landfill Area 1 and Landfill Area 2

Global Summary Results for Run "25-Year"

Project: Winyah Landfills_Areas 1 an   Simulation Run: 25-Year

Start of Run: 01Jan2016, 00:00   Basin Model: Post Dev
End of Run: 06Jan2016, 00:00   Meteorologic Model: 25-Year
Compute Time: 30Jul2016, 15:36:49   Control Specifications: Control 1

Show Elements: All Elements   Volume Units: IN (•) AC-FT   Sorting: Alphabetic

Hydrologic Element	Drainage Area (MI2)	Peak Discharge (CFS)	Time of Peak	Volume (AC-FT)
C.1A	0.03378	115.8	01Jan2016, 12:10	10.6
C.1B	0.00414	16.3	01Jan2016, 12:05	1.3
C.1C	0.01677	59.5	01Jan2016, 12:10	5.3
C.2BA	0.00370	14.9	01Jan2016, 12:06	1.2
C.2BB	0.00291	11.5	01Jan2016, 12:05	0.9
C.2BC	0.00924	35.1	01Jan2016, 12:09	2.9
C.2C	0.02730	102.7	01Jan2016, 12:10	8.6
C.2D	0.00602	23.6	01Jan2016, 12:06	1.9
O.1A	0.04147	143.9	01Jan2016, 12:10	13.0
O.1C	0.01677	59.5	01Jan2016, 12:10	5.3
O.2A	0.02405	89.5	01Jan2016, 12:09	7.6
O.2B	0.00924	35.1	01Jan2016, 12:09	2.9
O.2C	0.02730	102.7	01Jan2016, 12:10	8.6
O.2D	0.00602	23.6	01Jan2016, 12:06	1.9
O.2E	0.01264	49.0	01Jan2016, 12:06	4.0
O.2I	0.05037	107.1	01Jan2016, 12:07	15.8
P.1A1	0.00769	28.1	01Jan2016, 12:09	2.4
P.1BA	0.03378	115.9	01Jan2016, 12:09	10.6
P.1B1	0.02964	102.7	01Jan2016, 12:10	9.3
P.1B2	0.02108	73.4	01Jan2016, 12:10	6.6
P.1B3	0.00830	29.6	01Jan2016, 12:11	2.6
P.1C1	0.01677	59.6	01Jan2016, 12:10	5.3
P.1C2	0.00533	20.1	01Jan2016, 12:08	1.7
P.2A1	0.02405	89.5	01Jan2016, 12:09	7.6
P.2A2	0.00602	23.7	01Jan2016, 12:08	1.9
P.2BA	0.00378	14.9	01Jan2016, 12:05	1.2
P.2BB	0.00924	35.2	01Jan2016, 12:08	2.9
P.2C1	0.02730	102.8	01Jan2016, 12:10	8.6
P.2DA	0.00602	23.6	01Jan2016, 12:05	1.9

Global Summary Results for Run "25-Year"

Project: Winyah Landfills_Areas 1 an   Simulation Run: 25-Year

Start of Run: 01Jan2016, 00:00   Basin Model: Post Dev
End of Run: 06Jan2016, 00:00   Meteorologic Model: 25-Year
Compute Time: 30Jul2016, 15:36:49   Control Specifications: Control 1

Show Elements: All Elements   Volume Units: IN (•) AC-FT   Sorting: Alphabetic

Hydrologic Element	Drainage Area (MI2)	Peak Discharge (CFS)	Time of Peak	Volume (AC-FT)
1A1	0.00769	28.3	01Jan2016, 12:07	2.4
1BA	0.00346	13.7	01Jan2016, 12:05	1.1
1BB	0.00068	2.7	01Jan2016, 12:05	0.2
1B1	0.00856	32.3	01Jan2016, 12:07	2.7
1B2A	0.00200	7.9	01Jan2016, 12:05	0.6
1B2B	0.01070	41.1	01Jan2016, 12:06	3.4
1B3	0.00838	29.8	01Jan2016, 12:08	2.6
1C1	0.01144	39.7	01Jan2016, 12:09	3.6
1C2	0.00533	20.1	01Jan2016, 12:06	1.7
2A1	0.01803	66.0	01Jan2016, 12:08	5.7
2A2A	0.00336	13.3	01Jan2016, 12:05	1.1
2A2B	0.00266	10.5	01Jan2016, 12:05	0.8
2BA	0.00378	14.9	01Jan2016, 12:05	1.2
2BB	0.00291	11.5	01Jan2016, 12:05	0.9
2BC	0.00255	9.3	01Jan2016, 12:08	0.8
2C1	0.02730	103.3	01Jan2016, 12:06	8.6
2DA	0.00602	23.7	01Jan2016, 12:05	1.9
2E1	0.01264	49.0	01Jan2016, 12:06	4.0
2F1	0.02332	87.2	01Jan2016, 12:07	7.3
2F2	0.02705	100.0	01Jan2016, 12:07	8.5

Table 1-3. P 100-year, 24-hour Precipitation Event Nodal Areas, Peak Flow Rates, and Runoff Volumes for Landfill Area 1 and Landfill Area 2

Global Summary Results for Run "100-Year"

Project: Winyah Landfills_Areas 1 an   Simulation Run: 100-Year

Start of Run: 01Jan2016, 00:00   Basin Model: Post Dev
End of Run: 06Jan2016, 00:00   Meteorologic Model: 100-Year
Compute Time: 30Jul2016, 15:39:06   Control Specifications: Control 1

Show Elements: All Elements   Volume Units: IN (●) AC-FT   Sorting: Alphabetic

Hydrologic Element	Drainage Area (MI2)	Peak Discharge (CFS)	Time of Peak	Volume (AC-FT)
C.1A	0.03378	163.9	01Jan2016, 12:09	15.1
C.1B	0.00414	22.9	01Jan2016, 12:05	1.9
C.1C	0.01677	83.3	01Jan2016, 12:10	7.5
C.2BA	0.00378	20.8	01Jan2016, 12:05	1.7
C.2BB	0.00291	16.1	01Jan2016, 12:05	1.3
C.2BC	0.00924	49.2	01Jan2016, 12:08	4.1
C.2C	0.02730	144.1	01Jan2016, 12:10	12.2
C.2D	0.00602	33.0	01Jan2016, 12:06	2.7
O.1A	0.04147	203.4	01Jan2016, 12:09	18.5
O.1C	0.01677	83.3	01Jan2016, 12:10	7.5
O.2A	0.02405	125.3	01Jan2016, 12:09	10.8
O.2B	0.00924	49.2	01Jan2016, 12:08	4.1
O.2C	0.02730	144.1	01Jan2016, 12:10	12.2
O.2D	0.00602	33.0	01Jan2016, 12:06	2.7
O.2E	0.01264	68.6	01Jan2016, 12:06	5.7
O.2F	0.05037	262.1	01Jan2016, 12:07	22.5
P.1A1	0.00769	39.5	01Jan2016, 12:09	3.4
P.1BA	0.03370	164.1	01Jan2016, 12:09	15.1
P.1B1	0.02964	144.7	01Jan2016, 12:09	13.3
P.1B2	0.02103	103.5	01Jan2016, 12:10	9.4
P.1B3	0.00838	41.6	01Jan2016, 12:11	3.7
P.1C1	0.01677	83.3	01Jan2016, 12:10	7.5
P.1C2	0.00533	28.1	01Jan2016, 12:06	2.4
P.2A1	0.02405	125.3	01Jan2016, 12:09	10.8
P.2A2	0.00602	33.1	01Jan2016, 12:07	2.7
P.2BA	0.00378	20.8	01Jan2016, 12:05	1.7
P.2BB	0.00924	49.3	01Jan2016, 12:08	4.1
P.2C1	0.02730	144.2	01Jan2016, 12:10	12.2
P.2DA	0.00602	33.1	01Jan2016, 12:05	2.7

Global Summary Results for Run "100-Year"

Project: Winyah Landfills_Areas 1 an   Simulation Run: 100-Year

Start of Run: 01Jan2016, 00:00   Basin Model: Post Dev
End of Run: 06Jan2016, 00:00   Meteorologic Model: 100-Year
Compute Time: 30Jul2016, 15:39:06   Control Specifications: Control 1

Show Elements: All Elements   Volume Units: IN (●) AC-FT   Sorting: Alphabetic

Hydrologic Element	Drainage Area (MI2)	Peak Discharge (CFS)	Time of Peak	Volume (AC-FT)
1A1	0.00769	39.6	01Jan2016, 12:07	3.4
1BA	0.00346	19.1	01Jan2016, 12:05	1.5
1BB	0.00068	3.8	01Jan2016, 12:05	0.3
1B1	0.00856	45.1	01Jan2016, 12:07	3.8
1B2A	0.00200	11.0	01Jan2016, 12:05	0.9
1B2B	0.01070	57.5	01Jan2016, 12:06	4.8
1B3	0.00838	41.8	01Jan2016, 12:08	3.7
1C1	0.01144	55.7	01Jan2016, 12:09	5.1
1C2	0.00533	28.2	01Jan2016, 12:06	2.4
2A1	0.01803	92.5	01Jan2016, 12:07	8.1
2A2A	0.00336	18.6	01Jan2016, 12:05	1.5
2A2B	0.00266	14.7	01Jan2016, 12:05	1.2
2BA	0.00378	20.9	01Jan2016, 12:05	1.7
2BB	0.00291	16.1	01Jan2016, 12:05	1.3
2BC	0.00255	13.0	01Jan2016, 12:07	1.1
2C1	0.02730	144.7	01Jan2016, 12:06	12.2
2DA	0.00602	33.2	01Jan2016, 12:05	2.7
2E1	0.01264	68.6	01Jan2016, 12:06	5.7
2F1	0.02332	122.0	01Jan2016, 12:07	10.4
2F2	0.02705	140.1	01Jan2016, 12:07	12.1

HEC-HMS HYDROLOGIC MODEL INPUT PARAMETERS

Basin: Post Dev
Last Modified Date: 29 July 2016
Last Modified Time: 13:23:23
Version: 4.1
Filepath Separator: \
Unit System: English
Missing Flow To Zero: No
Enable Flow Ratio: No
Compute Local Flow At Junctions: No

Enable Sediment Routing: No

Enable Quality Routing: No
End:

Subbasin: 2F2
Last Modified Date: 1 July 2016
Last Modified Time: 16:18:23
Canvas X: 12885.616843649492
Canvas Y: 4794.7598030265435
Area: 0.02705
Downstream: O.2F

Canopy: None
Plant Uptake Method: None

Surface: None

LossRate: SCS
Percent Impervious Area: 0.0
Curve Number: 80

Transform: SCS
Lag: 5.77
Unitgraph Type: STANDARD

Baseflow: None
End:

Subbasin: 2F1
Last Modified Date: 1 July 2016
Last Modified Time: 16:18:23
Canvas X: 12744.106111734043
Canvas Y: -677.804127648571
Area: 0.02332
Downstream: O.2F

Canopy: None
Plant Uptake Method: None

Surface: None

LossRate: SCS
Percent Impervious Area: 0.0
Curve Number: 80

Transform: SCS
Lag: 5.44
Unitgraph Type: STANDARD

Baseflow: None
End:

Junction: O.2F
Last Modified Date: 13 June 2016
Last Modified Time: 20:31:32
Canvas X: 5692.872341359882
Canvas Y: 1847.8327345456855
End:

Subbasin: 1B2B
Last Modified Date: 2 August 2016
Last Modified Time: 22:13:49
Canvas X: -8211.903636640676
Canvas Y: 7069.025599933324
From Canvas X: 10493.873085339166
From Canvas Y: 362.69146608314986
Label X: 0.0
Label Y: 1.0
Area: 0.01070
Downstream: P.1B2

Canopy: None
Plant Uptake Method: None

Surface: None

LossRate: SCS
Percent Impervious Area: 0.0
Curve Number: 80

Transform: SCS
Lag: 4.58
Unitgraph Type: STANDARD

Baseflow: None
End:

Subbasin: 1B3
Last Modified Date: 30 July 2016
Last Modified Time: 20:36:01
Canvas X: -3887.0462606038272
Canvas Y: 1894.7386703252268
Area: 0.00838
Downstream: P.1B3

Canopy: None
Plant Uptake Method: None

Surface: None

LossRate: SCS
Percent Impervious Area: 0.0
Curve Number: 80

Transform: SCS
Lag: 6.93
Unitgraph Type: STANDARD

Baseflow: None
End:

Reach: P.1B3
Last Modified Date: 29 July 2016
Last Modified Time: 12:57:46
Canvas X: -3713.78669946232
Canvas Y: 8938.33764631685
From Canvas X: -1811.6481445504396
From Canvas Y: 3056.9616153151255
Downstream: P.1B2

Route: Kinematic Wave
Channel: Kinematic Wave
Length: 800
Energy Slope: 0.005
Mannings n: 0.027
Shape: Trapezoid
Number of Subreaches: 2
Width: 3
Side Slope: 3
Channel Loss: None
End:

Subbasin: 1B2A
Last Modified Date: 29 July 2016
Last Modified Time: 12:56:14
Canvas X: -5879.428452015083
Canvas Y: 4800.296032799975
Area: 0.00200
Downstream: P.1B2

Canopy: None
Plant Uptake Method: None

Surface: None

LossRate: SCS
Percent Impervious Area: 0.0
Curve Number: 80

Transform: SCS
Lag: 3.79
Unitgraph Type: STANDARD

Baseflow: None
End:

Reach: P.1B2
Last Modified Date: 29 July 2016
Last Modified Time: 12:57:00
Canvas X: -9744.809909356078
Canvas Y: 11051.39687314094
From Canvas X: -3713.78669946232
From Canvas Y: 8938.33764631685
Label X: -18.0
Label Y: 11.0
Downstream: P.1B1

Route: Kinematic Wave
Channel: Kinematic Wave
Length: 953
Energy Slope: 0.005
Mannings n: 0.027
Shape: Trapezoid
Number of Subreaches: 2
Width: 3
Side Slope: 3
Channel Loss: None
End:

Subbasin: 1B1
Last Modified Date: 30 July 2016
Last Modified Time: 20:36:35
Canvas X: -10405.140917738609
Canvas Y: 4668.197125443163
Area: 0.00856
Downstream: P.1B1

Canopy: None
Plant Uptake Method: None

Surface: None

LossRate: SCS
Percent Impervious Area: 0.0
Curve Number: 80

Transform: SCS
Lag: 5.15
Unitgraph Type: STANDARD

Baseflow: None
End:

Reach: P.1B1
Last Modified Date: 2 August 2016
Last Modified Time: 22:13:40
Canvas X: -16128.009657053855
Canvas Y: 10567.154133660419
From Canvas X: -9744.809909356078
From Canvas Y: 11051.39687314094
Label X: -28.0
Label Y: 10.0
Downstream: P.1BA

Route: Kinematic Wave
Channel: Kinematic Wave
Length: 215
Energy Slope: 0.005
Mannings n: 0.027
Shape: Trapezoid
Number of Subreaches: 2
Width: 3
Side Slope: 3
Channel Loss: None
End:

Subbasin: 1BA
Last Modified Date: 1 July 2016
Last Modified Time: 16:16:48
Canvas X: -14102.994564680768
Canvas Y: 4712.219192668666
Area: 0.00346
Downstream: C.1B

Canopy: None
Plant Uptake Method: None

Surface: None

LossRate: SCS
Percent Impervious Area: 0.0
Curve Number: 80

Transform: SCS
Lag: 3.60
Unitgraph Type: STANDARD

Baseflow: None
End:

Subbasin: 1BB
Last Modified Date: 1 July 2016
Last Modified Time: 16:16:48
Canvas X: -17580.73787549542
Canvas Y: 6164.947411110228
Area: 0.00068
Downstream: C.1B

Canopy: None
Plant Uptake Method: None

Surface: None

LossRate: SCS
Percent Impervious Area: 0.0
Curve Number: 80

Transform: SCS
Lag: 3.60
Unitgraph Type: STANDARD

Baseflow: None
End:

Reach: C.1B
Last Modified Date: 15 June 2016
Last Modified Time: 15:02:49
Canvas X: -16128.009657053855
Canvas Y: 10567.154133660419
From Canvas X: -16128.009657053859
From Canvas Y: 8278.006637934319
Downstream: P.1BA

Route: Kinematic Wave
Channel: Kinematic Wave
Length: 100
Energy Slope: 0.005
Mannings n: 0.012
Shape: Circular
Number of Subreaches: 2
Width: 2
Channel Loss: None
End:

Reach: P.1BA
Last Modified Date: 2 August 2016
Last Modified Time: 22:13:38
Canvas X: -19693.79710231951
Canvas Y: 10347.04379753291
From Canvas X: -16128.009657053855
From Canvas Y: 10567.154133660419
Label X: -26.0
Label Y: 8.0
Downstream: C.1A

Route: Kinematic Wave
Channel: Kinematic Wave
Length: 105
Energy Slope: 0.005
Mannings n: 0.027
Shape: Trapezoid
Number of Subreaches: 2
Width: 3
Side Slope: 3
Channel Loss: None
End:

Reach: C.1A
Last Modified Date: 2 August 2016
Last Modified Time: 22:13:36
Canvas X: -22482.613380442213
Canvas Y: 10030.299285254518
From Canvas X: -19693.79710231951
From Canvas Y: 10347.04379753291
Label X: -26.0
Label Y: 14.0
Downstream: O.1A

Route: Kinematic Wave
Channel: Kinematic Wave
Length: 100
Energy Slope: 0.005
Mannings n: 0.012
Shape: Circular
Number of Subreaches: 2
Width: 4
Channel Loss: None
End:

Subbasin: 1A1
Last Modified Date: 1 July 2016
Last Modified Time: 16:16:48
Canvas X: -17184.539270465903
Canvas Y: 2202.9613608150576
Area: 0.00769
Downstream: P.1A1

Canopy: None
Plant Uptake Method: None

Surface: None

LossRate: SCS
Percent Impervious Area: 0.0
Curve Number: 80

Transform: SCS
Lag: 5.92
Unitgraph Type: STANDARD

Baseflow: None
End:

Reach: P.1A1
Last Modified Date: 22 June 2016
Last Modified Time: 16:17:54
Canvas X: -22482.613380442213
Canvas Y: 10030.299285254518
From Canvas X: -21014.459119084568
From Canvas Y: 1850.7848230110412
Downstream: O.1A

Route: Kinematic Wave
Channel: Kinematic Wave
Length: 525
Energy Slope: 0.005
Mannings n: 0.027
Shape: Trapezoid
Number of Subreaches: 2
Width: 3
Side Slope: 3
Channel Loss: None
End:

Junction: O.1A
Last Modified Date: 5 July 2016
Last Modified Time: 14:11:58
Canvas X: -22482.613380442213
Canvas Y: 10030.299285254518
Label X: -64.0
Label Y: 12.0
End:

Subbasin: 2C1
Last Modified Date: 2 August 2016
Last Modified Time: 22:14:51
Canvas X: 28600.6668154754
Canvas Y: -2681.3308981947193
Area: 0.02730
Downstream: P.2C1

Canopy: None
Plant Uptake Method: None

Surface: None

LossRate: SCS
Percent Impervious Area: 0.0
Curve Number: 80

Transform: SCS
Lag: 5.00
Unitgraph Type: STANDARD

Baseflow: None
End:

Reach: P.2C1
Last Modified Date: 5 July 2016
Last Modified Time: 14:12:56
Canvas X: 41571.21717786729
Canvas Y: -7368.745639044617
From Canvas X: 27887.27355553725
From Canvas Y: -7571.470729745801
Label X: -6.0
Label Y: -14.0
Downstream: C.2C

Route: Kinematic Wave
Channel: Kinematic Wave
Length: 1350
Energy Slope: 0.005
Mannings n: 0.027
Shape: Trapezoid
Number of Subreaches: 2
Width: 3
Side Slope: 3
Channel Loss: None
End:

Reach: C.2C
Last Modified Date: 2 August 2016
Last Modified Time: 22:14:49
Canvas X: 46137.8514921873
Canvas Y: -5041.691204269195
From Canvas X: 41571.21717786729
From Canvas Y: -7368.745639044617
Label X: -15.0
Label Y: -17.0
Downstream: O.2C

Route: Kinematic Wave
Channel: Kinematic Wave
Length: 100
Energy Slope: 0.005
Mannings n: 0.012
Shape: Circular
Number of Subreaches: 2
Width: 4
Channel Loss: None
End:

Junction: O.2C
Last Modified Date: 2 August 2016
Last Modified Time: 22:14:45
Canvas X: 46137.8514921873
Canvas Y: -5041.691204269195
Label X: -32.0
Label Y: 22.0
End:

Subbasin: 2A1
Last Modified Date: 1 July 2016
Last Modified Time: 16:18:23
Canvas X: 38631.70336270011
Canvas Y: 4490.672166974764
Area: 0.01803

Downstream: P.2A1

Canopy: None
Plant Uptake Method: None

Surface: None

LossRate: SCS
Percent Impervious Area: 0.0
Curve Number: 80

Transform: SCS
Lag: 6.03
Unitgraph Type: STANDARD

Baseflow: None
End:

Subbasin: 2A2A
Last Modified Date: 1 July 2016
Last Modified Time: 16:16:48
Canvas X: 29306.349190445544
Canvas Y: 5301.572529779507
Area: 0.00336
Downstream: P.2A2

Canopy: None
Plant Uptake Method: None

Surface: None

LossRate: SCS
Percent Impervious Area: 0.0
Curve Number: 80

Transform: SCS
Lag: 3.60
Unitgraph Type: STANDARD

Baseflow: None
End:

Subbasin: 2A2B
Last Modified Date: 2 August 2016
Last Modified Time: 22:14:36
Canvas X: 23891.31095583533
Canvas Y: 8196.617848016158
Area: 0.00266
Downstream: P.2A2

Canopy: None
Plant Uptake Method: None

Surface: None

LossRate: SCS
Percent Impervious Area: 0.0
Curve Number: 80

Transform: SCS
Lag: 3.60
Unitgraph Type: STANDARD

Baseflow: None
End:

Reach: P.2A2
Last Modified Date: 2 August 2016
Last Modified Time: 22:14:38
Canvas X: 37212.6277277918
Canvas Y: 11281.962705464492
From Canvas X: 25758.77291482666
From Canvas Y: 11546.252075256181
Label X: -22.0
Label Y: 10.0
Downstream: P.2A1

Route: Kinematic Wave
Channel: Kinematic Wave
Length: 690
Energy Slope: 0.005
Mannings n: 0.027
Shape: Trapezoid
Number of Subreaches: 2
Width: 3
Side Slope: 3
Channel Loss: None
End:

Reach: P.2A1
Last Modified Date: 15 June 2016
Last Modified Time: 15:06:13
Canvas X: 48463.87026170762
Canvas Y: 8849.261617050259
From Canvas X: 37212.6277277918
From Canvas Y: 11281.962705464492
Downstream: O.2A

Route: Kinematic Wave
Channel: Kinematic Wave
Length: 600
Energy Slope: 0.005
Mannings n: 0.027
Shape: Trapezoid
Number of Subreaches: 2
Width: 3
Side Slope: 3
Channel Loss: None
End:

Junction: O.2A
Last Modified Date: 2 August 2016
Last Modified Time: 22:14:42
Canvas X: 48463.87026170762
Canvas Y: 8849.261617050259
Label X: -26.0
Label Y: -28.0
End:

Subbasin: 1C1
Last Modified Date: 30 July 2016
Last Modified Time: 20:36:01

Canvas X: -12166.023606758685
Canvas Y: 133.92420121646865
Area: 0.01144
Downstream: P.1C1

Canopy: None
Plant Uptake Method: None

Surface: None

LossRate: SCS
Percent Impervious Area: 0.0
Curve Number: 80

Transform: SCS
Lag: 7.69
Unitgraph Type: STANDARD

Baseflow: None
End:

Subbasin: 1C2
Last Modified Date: 30 July 2016
Last Modified Time: 20:36:35
Canvas X: -6294.508075225764
Canvas Y: -97.64352108602907
Area: 0.00533
Downstream: P.1C2

Canopy: None
Plant Uptake Method: None

Surface: None

LossRate: SCS
Percent Impervious Area: 0.0
Curve Number: 80

Transform: SCS
Lag: 5.02
Unitgraph Type: STANDARD

Baseflow: None
End:

Reach: P.1C2
Last Modified Date: 29 July 2016
Last Modified Time: 13:20:42
Canvas X: -14675.281438612294
Canvas Y: -2375.333630637142
From Canvas X: -6684.710789452354
From Canvas Y: -2980.3543311221183
Label X: 2.0
Label Y: 14.0
Downstream: P.1C1

Route: Kinematic Wave
Channel: Kinematic Wave
Length: 415
Energy Slope: 0.005
Mannings n: 0.027
Shape: Trapezoid
Number of Subreaches: 2
Width: 3
Side Slope: 3
Channel Loss: None
End:

Reach: P.1C1
Last Modified Date: 15 June 2016
Last Modified Time: 15:04:00
Canvas X: -21960.93356443286
Canvas Y: -1384.8371180633476
From Canvas X: -14675.281438612294
From Canvas Y: -2375.333630637142
Downstream: C.1C

Route: Kinematic Wave
Channel: Kinematic Wave
Length: 340
Energy Slope: 0.005
Mannings n: 0.027
Shape: Trapezoid
Number of Subreaches: 2
Width: 3
Side Slope: 3
Channel Loss: None
End:

Reach: C.1C
Last Modified Date: 29 July 2016
Last Modified Time: 13:27:37
Canvas X: -25834.875480277027
Canvas Y: 376.04557095673226
From Canvas X: -21960.93356443286
From Canvas Y: -1384.8371180633476
Downstream: O.1C

Route: Kinematic Wave
Channel: Kinematic Wave
Length: 100
Energy Slope: 0.005
Mannings n: 0.012
Shape: Circular
Number of Subreaches: 2
Width: 6
Channel Loss: None
End:

Junction: O.1C
Last Modified Date: 13 June 2016
Last Modified Time: 19:27:14
Canvas X: -25834.875480277027
Canvas Y: 376.04557095673226
End:

Subbasin: 2E1
Last Modified Date: 1 July 2016
Last Modified Time: 16:18:23
Canvas X: 41342.38115307616
Canvas Y: 437.24065926194817
Area: 0.01264

Downstream: O.2E

Canopy: None
Plant Uptake Method: None

Surface: None

LossRate: SCS
Percent Impervious Area: 0.0
Curve Number: 80

Transform: SCS
Lag: 4.21
Unitgraph Type: STANDARD

Baseflow: None
End:

Junction: O.2E
Last Modified Date: 2 August 2016
Last Modified Time: 22:14:47
Canvas X: 48010.634599872006
Canvas Y: 886.0654104885907
Label X: -27.0
Label Y: -23.0
End:

Subbasin: 2BA
Last Modified Date: 1 July 2016
Last Modified Time: 16:16:48
Canvas X: 20589.395913598288
Canvas Y: 9600.133674793733
Area: 0.00378
Downstream: P.2BA

Canopy: None
Plant Uptake Method: None

Surface: None

LossRate: SCS
Percent Impervious Area: 0.0
Curve Number: 80

Transform: SCS
Lag: 3.60
Unitgraph Type: STANDARD

Baseflow: None
End:

Reach: P.2BA
Last Modified Date: 2 August 2016
Last Modified Time: 22:14:32
Canvas X: 15538.680105655912
Canvas Y: 12520.77702451859
From Canvas X: 20225.430884358313
From Canvas Y: 11302.987275198375
Label X: -23.0
Label Y: 9.0
Downstream: C.2BA

Route: Kinematic Wave
Channel: Kinematic Wave
Length: 100
Energy Slope: 0.005
Mannings n: 0.027
Shape: Trapezoid
Number of Subreaches: 2
Width: 3
Side Slope: 3
Channel Loss: None
End:

Reach: C.2BA
Last Modified Date: 2 August 2016
Last Modified Time: 22:14:29
Canvas X: 13413.084311893013
Canvas Y: 11607.068275270633
From Canvas X: 15538.680105655912
From Canvas Y: 12520.77702451859
Label X: -40.0
Label Y: 10.0
Downstream: P.2BB

Route: Kinematic Wave
Channel: Kinematic Wave
Length: 100
Energy Slope: 0.005
Mannings n: 0.012
Shape: Circular
Number of Subreaches: 2
Width: 2
Channel Loss: None
End:

Subbasin: 2BB
Last Modified Date: 1 July 2016
Last Modified Time: 16:16:48
Canvas X: 16575.52671264449
Canvas Y: 8018.912474417995
Area: 0.00291
Downstream: C.2BB

Canopy: None
Plant Uptake Method: None

Surface: None

LossRate: SCS
Percent Impervious Area: 0.0
Curve Number: 80

Transform: SCS
Lag: 3.60
Unitgraph Type: STANDARD

Baseflow: None
End:

Reach: C.2BB
Last Modified Date: 2 August 2016

Last Modified Time: 22:15:01
Canvas X: 13413.084311893013
Canvas Y: 11607.068275270633
From Canvas X: 13717.16531196527
From Canvas Y: 7897.280074389091
Label X: -7.0
Label Y: -1.0
Downstream: P.2BB

Route: Kinematic Wave
Channel: Kinematic Wave
Length: 100
Energy Slope: 0.005
Mannings n: 0.012
Shape: Circular
Number of Subreaches: 2
Width: 2
Channel Loss: None
End:

Subbasin: 2BC
Last Modified Date: 1 July 2016
Last Modified Time: 16:16:48
Canvas X: 10250.641911141538
Canvas Y: 8262.1772744758
Area: 0.00255
Downstream: P.2BB

Canopy: None
Plant Uptake Method: None

Surface: None

LossRate: SCS
Percent Impervious Area: 0.0
Curve Number: 80

Transform: SCS
Lag: 6.09
Unitgraph Type: STANDARD

Baseflow: None
End:

Reach: P.2BB
Last Modified Date: 2 August 2016
Last Modified Time: 22:14:28
Canvas X: 6358.405110216641
Canvas Y: 12154.414075400697
From Canvas X: 13413.084311893013
From Canvas Y: 11607.068275270633
Label X: -41.0
Label Y: 9.0
Downstream: C.2BC

Route: Kinematic Wave
Channel: Kinematic Wave
Length: 700
Energy Slope: 0.005
Mannings n: 0.027
Shape: Trapezoid
Number of Subreaches: 2
Width: 3
Side Slope: 3
Channel Loss: None
End:

Reach: C.2BC
Last Modified Date: 29 July 2016
Last Modified Time: 13:27:37
Canvas X: 6115.140310158829
Canvas Y: 9052.78787466367
From Canvas X: 6358.405110216641
From Canvas Y: 12154.414075400697
Label X: -59.0
Label Y: 2.0
Downstream: O.2B

Route: Kinematic Wave
Channel: Kinematic Wave
Length: 100
Energy Slope: 0.005
Mannings n: 0.012
Shape: Circular
Number of Subreaches: 2
Width: 4
Channel Loss: None
End:

Junction: O.2B
Last Modified Date: 13 June 2016
Last Modified Time: 19:50:06
Canvas X: 6115.140310158829
Canvas Y: 9052.78787466367
End:

Subbasin: 2DA
Last Modified Date: 2 August 2016
Last Modified Time: 22:14:53
Canvas X: 17988.033892342843
Canvas Y: -2615.0019424251404
Area: 0.00602
Downstream: P.2DA

Canopy: None
Plant Uptake Method: None

Surface: None

LossRate: SCS
Percent Impervious Area: 0.0
Curve Number: 80

Transform: SCS
Lag: 3.70
Unitgraph Type: STANDARD

Baseflow: None
End:

Reach: P.2DA
Last Modified Date: 2 August 2016

Last Modified Time: 22:14:55
Canvas X: 17658.926938518853
Canvas Y: -6838.545244216715
From Canvas X: 19772.49226240484
From Canvas Y: -6157.806019961932
Label X: -11.0
Label Y: -16.0
Downstream: C.2D

Route: Kinematic Wave
Channel: Kinematic Wave
Length: 100
Energy Slope: 0.005
Mannings n: 0.027
Shape: Trapezoid
Number of Subreaches: 2
Width: 3
Side Slope: 3
Channel Loss: None
End:

Reach: C.2D
Last Modified Date: 5 July 2016
Last Modified Time: 14:13:10
Canvas X: 8897.406134285062
Canvas Y: -6572.885643172613
From Canvas X: 17658.926938518853
From Canvas Y: -6838.545244216715
Label X: -4.0
Label Y: -18.0
Downstream: O.2D

Route: Kinematic Wave
Channel: Kinematic Wave
Length: 100
Energy Slope: 0.005
Mannings n: 0.012
Shape: Circular
Number of Subreaches: 2
Width: 2
Channel Loss: None
End:

Junction: O.2D
Last Modified Date: 5 July 2016
Last Modified Time: 14:13:08
Canvas X: 8897.406134285062
Canvas Y: -6572.885643172613
Label X: -55.0
Label Y: 14.0
End:

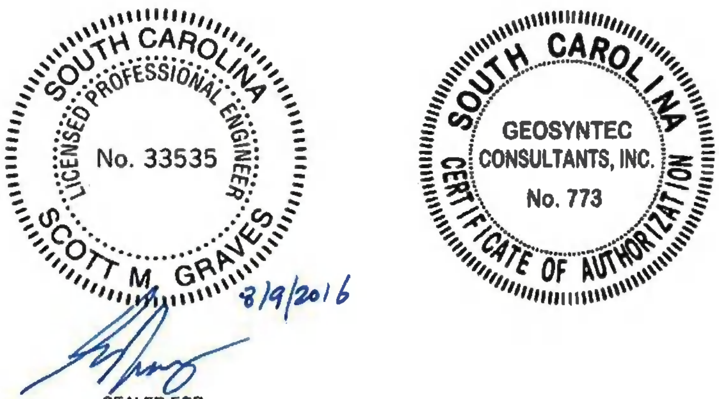
Basin Schematic Properties:
Last View N: 19455.428323308017
Last View S: -11423.177499083657
Last View W: 1299.4355545324634
Last View E: 52378.99845699345
Maximum View N: 12642.493068261829
Maximum View S: -7856.82463190575
Maximum View W: -27712.616632896763
Maximum View E: 51152.5117771321
Extent Method: Elements
Buffer: 5
Draw Icons: Yes
Draw Icon Labels: Name
Draw Map Objects: No
Draw Gridlines: No
Draw Flow Direction: No
Fix Element Locations: No
Fix Hydrologic Order: No
End:

APPENDIX F-2

DRAINAGE TERRACE AND DOWNDRAIN DESIGN

APPENDIX F-2

DRAINAGE TERRACE AND DOWNDRAIN DESIGN



SEALED FOR
CALCULATION PAGES F2-1
THROUGH F2-40

1. INTRODUCTION

1.1 Purpose

The purpose of this calculation package is to present the design of the top deck drainage terraces, sideslope drainage terraces, access road drainage terrace, and downdrain pipe features for the final facility surface water management system of the proposed Class Three Landfill (composed of Landfill Area 1 and Landfill Area 2) at the Winyah Generating Station (WGS) located in Georgetown County, South Carolina. Specifically, this calculation package presents the design criteria for the drainage terraces and downdrains, and presents hydraulic computations and the resulting design of the terraces and downdrains.

1.2 Drainage Terrace and Downdrain Overview

The Engineering Drawing set that accompanies the Engineering Report shows the final grading plan of each landfill area, and also includes a series of drawings that shows the surface water management system features (identification, layout, and engineering details of the components). An overview of the drainage terraces and downdrains is provided below.

- Near the crest of the landfill top-deck (i.e., the areas inclined at 3 to 5 percent nominal surface slopes), "top-deck drainage terraces" are positioned where necessary, to capture sheet flow runoff from top deck areas. These terraces convey water to nearby sideslope drainage terraces.

- The landfill areas are graded with final cover sideslopes inclined at three horizontal to one vertical (3H:1V) in-between drainage terraces. These "sideslope drainage terraces" are spaced approximately every 30 feet vertically, and with typical drainage profile slopes at 2 percent. Sheet flow runoff upgradient from each sideslope drainage terrace will be captured in the terraces, where the water will be conveyed to downdrain pipes spaced periodically around each landfill area.
- Each landfill area will include a final cover access road, with an adjacent drainage channel ("access road drainage terrace") located on the inside edge of the road. The access road drainage terraces will receive primarily the sheet flow runoff from the road along with some sheet flow from adjacent landfill sideslopes, and will convey water to either a culvert beneath the access road or to downdrain pipes spaced periodically around each landfill area.
- The downdrain pipes are oriented approximately perpendicular to the landfill sideslopes, and will convey water from the terraces down the landfill slopes to outlets at the toe of slope (perimeter of the landfills) into either constructed perimeter drainage channels, or directly into existing site drainage features (i.e., the discharge canal or cooling pond).
- At each location where downdrain pipes outlet to the perimeter drainage channel, a concrete outlet apron, also equipped with concrete energy dissipator blocks, will be used. At each location where downdrain pipes outlet directly to existing drainage features (discharge canal or cooling pond), riprap aprons will be used to dissipate energy.

2. DESIGN CRITERIA

The drainage terraces and downdrains are designed to meet or exceed the applicable requirements of the State of South Carolina Department of Health and Environmental Control (DHEC) regulations for Class Three Landfills. The following design criteria have been adopted:

- the drainage terraces and downdrains are designed to control at least the water volume resulting from a 25-year, 24-hour storm;
- the drainage terraces:

  - are sized to convey runoff from the 25-year, 24-hour design rainfall event with a hydraulic head not more than 1-ft;
  - are sized to convey the 100-year, 24-hour design rainfall event without overtopping;
  - have channel lining materials selected to resist the velocities and/or tractive stresses produced by the 25-year, 24-hour rainfall event; and
- the downdrain pipes are designed to convey the 100-year, 24-hour rainfall event without flowing full (to avoid pressurized flow).

3. METHODOLOGY

The hydraulic capacity of the drainage terraces (top deck, sideslope, and access road) and each downdrain pipe is calculated by solving Manning's equation for the depth of flow within each feature. Manning's equation (Chow, 1959) is expressed as:

$$Q = \frac{1.49}{n} A R^{2/3} S^{1/2}$$

where:

Q = discharge (cfs),

n = Manning's roughness coefficient,

A = area of cross-section of flow (ft^2),

P = wetted perimeter (ft),

R = hydraulic radius = A/P (ft), and

S = slope of hydraulic grade line (or longitudinal slope for normal flow conditions) (ft/ft).

The average tractive stresses in the drainage terraces for various flows from the design events are estimated by the following equation (Chow, 1959):

$$\tau_o = \gamma_w R S$$

where:

τ_o = average tractive stress (lb/ft^2),

γ_w = unit weight of water (lb/ft^3),

R = hydraulic radius = A/P (ft), and

S = channel slope (ft/ft).

Manning's equation was used to calculate the resulting flow depths and velocities to confirm that the drainage features are appropriately sized to meet the design criteria. The tractive stress equation was also used to evaluate the channel linings and confirm that they are appropriately selected to meet the design criteria. The calculated 25-year average velocity was used as the basis for design of the channel lining system for each drainage feature.

The flow rates (i.e., peak discharge rates, Q) were derived from the Hydrologic Modeling System (HEC-HMS) computer program hydrologic modeling results from the analysis of the design storm events, as presented in a separate calculation package (see Appendix F-1). More specifically, the flow rates from the HEC-HMS modeling results relate to the contributing drainage areas for each downdrain pipe feature. Each individual sideslope drainage terrace and top-deck drainage terraces constitute a portion of the total contributing area to a downdrain pipe feature. Therefore, the total flow rate within the downdrain pipe feature was scaled based on area to estimate the expected flow rate to each drainage terrace within the overall contributing area. For example, the subcatchment contributing to downdrain pipe feature 1A1 has a total area of 4.92 acres. The subcatchment can be divided into six separate subareas corresponding to each sideslope drainage terraces within this subcatchment. The farthest downgradient sideslope drainage terrace on the west side of the downdrain pipe feature 1A1 (i.e., one of the six drainage terraces for this downdrain pipe) has an area of 0.69 acres. The expected flow rate within that drainage terrace is estimated as 0.69 acres divided by the total area (4.92 acres) times the peak flow rate (28.3 cfs for a 25-year event). This calculation results in an expected peak flow rate of 3.95 cfs for this specific sideslope drainage terrace. Similar calculations were conducted for every proposed drainage terrace, and the largest subarea contributing to a drainage terrace (i.e., the largest expected flow rate within all the drainage terraces) was used to calculate the critical peak flow rate required for designing adequate capacity within the drainage terrace.

4. DESIGN PARAMETERS

The design parameters used in these calculations, including channel geometry, Manning's roughness coefficient, and calculated peak flows for the 25-year and 100-year events are summarized for each drainage terrace type and downdrain pipe feature in Tables 1 and 2, respectively, of this calculation package. Further discussion of the design parameters for each feature is provided in the sections that follow.

4.1 Top-Deck Drainage Terraces

The top-deck drainage terraces are designed as grass-lined v-shaped channels (i.e., trapezoidal channels with bottom width equal to zero). The top-deck drainage terraces will be formed by constructing tack-on berms near the edge of the top deck surface. As such, one side of the v-shaped channel will be inclined at 5% based on the top deck surface grades, and the other side of the v-shaped channel will be inclined at 3H:1V based on the constructed berm. The top deck drainage terraces will have a constant depth of 1.5 ft and a longitudinal slope of approximately 0.5%.

A Manning's roughness coefficient of 0.027 was selected for the top-deck terraces, based on recommendations for excavated earth channels that are straight and uniform with short grass and few weeds (Chow, 1959) as shown in Table 3 of this calculation package. A maximum permissible velocity of 6.0 ft/s was selected, based on recommendations from Georgetown County (2006) for bermuda grass-lined drainage channels, as shown in Table 4 of this calculation package.

The critical top-deck drainage terrace, used as the basis for design, is located in subcatchment 1B1 of the Area 1 Landfill as shown on the attached Figure 1. This location is critical because it has the largest contributing drainage area of 2.25 acres. The resulting peak discharge rates from the 25-year and 100-year storm events are 13.3 cfs and 18.5 cfs, respectively.

4.2 Sideslope Drainage Terraces

The sideslope drainage terraces are designed as grass-lined v-shaped channels graded into the 3H:1V sideslopes of the final cover system. As such, the sideslopes of the sideslope drainage terrace are 3H:1V on the up-slope side of the terrace. The down-slope side is inclined at a 5H:1V angle. The sideslope drainage terraces have a constant depth of 1.5 ft and a longitudinal slope of approximately 2 percent (2%).

As with the top-deck terraces, a Manning's roughness coefficient of 0.027 was used, and a maximum permissible velocity of 6.0 ft/s was used.

The critical sideslope drainage terrace, used as the basis for design, is located in subcatchment 2F2 of the Area 2 Landfill as shown on the attached Figure 2. This location is critical because it has the largest contributing drainage area of 3.07 acres. The resulting peak discharge rates from the 25-year and 100-year storm events are 17.7 cfs and 24.9 cfs, respectively.

4.3 Access Road Drainage Terraces

The access road drainage terraces are designed as riprap-lined v-shaped channels. The sideslopes of the access road drainage terrace are 3H:1V on the up-slope side of the terrace. The down-slope side is inclined at a 2H:1V angle. The access road drainage terraces have a constant depth of 1.5 ft and a longitudinal slope of approximately 8 percent (8%).

The channel lining material will be six-inch diameter (average) riprap stones. A Manning's roughness coefficient of 0.036 was selected for the access road drainage terraces, based on recommendations from Chow (1959), as shown in Table 5. A tractive stress approach was used to check the stability of the channel lining, and a permissible tractive stress of 2.4 pounds per square foot (psf) was used based on recommendations by FHWA (2005) as shown in Table 6.

The critical access road drainage terrace, used as the basis for design, is located in subcatchment 1BA of the Area 1 Landfill as shown on the attached Figure 1. This location is critical because it has the largest contributing drainage area of 2.21 acres. The resulting peak discharge rates from the 25-year and 100-year storm events are 13.7 cfs and 19.1 cfs, respectively.

4.4 Downdrains

The downdrains will be 18-inch diameter, smooth-interior, high-density polyethylene (HDPE) pipe. A Manning's roughness coefficient of 0.009 was selected for the downdrains, based on recommendations from Barfuss and Tullis (1989), as shown in Figure 3. In general, the downdrain pipes will follow the slope of the landfill sideslopes, and as such will generally be inclined at a longitudinal slope of 3H:1V. However, the slope will flatten to four percent (4%) at locations where downdrain pipes cross the sideslope drainage terraces, as well as at the landfill perimeter. This much flatter 4% slope governs the hydraulic sizing, and is therefore used to size the downdrains.

Multiple pipes are used side-by-side to form a cluster of pipes at each downdrain feature location, based on the number of pipes needed to collect the runoff generated from each contributing drainage area.

5. RESULTS

The depth of flow, velocity, and average tractive stress for the peak discharges into each representative top deck drainage terrace, sideslope drainage terrace, access road drainage terrace, and downdrain pipe feature were calculated using the methodology described above. These calculation results are summarized in Table 7 (for drainage terraces) and Table 8 (for downdrain pipe features). In addition, the downdrain pipe feature configurations and associated number of pipes to be utilized at each sideslope drainage terrace were tabulated in Table 9.

Appendix 1 of this calculation package provides spreadsheets used for calculating the results for the downdrain pipe features, access road drainage terraces, top deck drainage terrace, and sideslope drainage terrace with the greatest flow rates (i.e., the critical design cases). The set of Engineering Drawings that accompanies the Engineering Report shows the location and layout of the drainage structures discussed within this calculation package.

The calculation results presented herein demonstrate that:

- Each drainage terrace is designed with the capacity to convey the flows from the 25-year, 24-hour design rainfall event with a hydraulic head not more than 1-ft and to convey the 100-year, 24-hour design rainfall event without overtopping.
- The selected channel lining can adequately resist the velocities and/or tractive stresses produced by the 25-year, 24-hour rainfall event.
- Downdrain pipe features are designed to flow partially full (avoiding pressurized flow) during the 25-year, 24-hour design rainfall event and the 100-year, 24-hour rainfall event, and using multiple pipes side-by-side at locations where more than one pipe is needed to convey the flow from the contributing drainage area.

6. REFERENCES

Barfuss, S.L. and Tullis, J.P. (1989). *Friction Factor Tests on 300 mm High Density Polyethylene Pipe*, Hydraulics Report No. 224, Utah Water Research Laboratory, Paper 611.

Chow, V.T. (1959). *Open-Channel Hydraulics*, McGraw-Hill Book Company, Inc., New York, NY.

FHWA (2005). *Design of Roadside Channels with Flexible Linings*, Federal Highway Administration, United States Department of Transportation, Hydraulic Engineering Circular No. 15, Third Edition.

Georgetown County (2006), *Storm Water Management Design Manual*, Georgetown County, South Carolina, revised 14 November 2006.

TABLES

- Table 1. Design Parameter Summary for Drainage Terraces
- Table 2. Design Parameter Summary for Downdrain Pipe Features
- Table 3. Manning's n Values for Open Channels (from Chow, 1959)
- Table 4. Maximum Velocities for Vegetative Channel Linings (Georgetown County, 2006)
- Table 5. Typical Roughness Coefficients for Lined or Built-Up Channels (from Chow, 1959)
- Table 6. Typical Permissible Shear Stresses for Bare Soil and Stone Linings (FHWA, 2005)
- Table 7. Summary of Calculated Results for Drainage Terraces
- Table 8. Summary of Calculated Results for Downdrain Pipe Features
- Table 9. Distribution of Downdrain Pipes at each Sideslope Drainage Terrace Location

Table 1. Design Parameter Summary for Drainage Terraces

Drainage Feature	Channel Shape	Longitudinal Channel Slope (%)	Manning's n	Bottom Width (ft)	Depth (ft)	Left Side Slope (H:V)	Right Side Slope (H:V)	25-year Flow Rate Q_{25} (cfs)	100-year Flow Rate Q_{100} (cfs)
Sideslope	V-shaped	2.0	0.027	0.0	1.5	5:1	3:1	17.7	24.9
Top Deck	V-shaped	0.50	0.027	0.0	1.5	20:1	3:1	13.3	18.5
Access Road	V-shaped	8.00	0.036	0.0	1.5	3:1	2:1	13.7	19.1

Table 2. Design Parameter Summary for Downdrain Pipe Features

Downdrain Feature[1]	No. of Pipes	Pipe Dimensions			25-year	100-year
		Pipe Diameter (ft)	Minimum Pipe Slope (%)	Manning's Roughness n	Peak Flow (cfs)	Peak Flow (cfs)
1A1	1	1.5	4.0	0.009	15.9	22.3
1B1	2	1.5	4.0	0.009	32.3	45.1
1B2	2	1.5	4.0	0.009	23.7	33.1
1B3	2	1.5	4.0	0.009	24.6	34.5
1C1	2	1.5	4.0	0.009	36.1	50.7
2A1	3	1.5	4.0	0.009	46.0	64.5
2A2	1	1.5	4.0	0.009	21.0	29.4
2C1	5	1.5	4.0	0.009	85.7	120.1
2E1	2	1.5	4.0	0.009	42.4	59.3
2F1	4	1.5	4.0	0.009	79.3	110.9
2F2	5	1.5	4.0	0.009	96.3	134.9

Note:

1. The downdrain pipe feature consists of multiple downdrain pipes with varying lengths.

Table 3. Manning's n Values for Open Channels (from Chow, 1959)

Type of channel and description	Minimum	Normal	Maximum
C. EXCAVATED OR DREDGED			
a. Earth, straight and uniform			
1. Clean, recently completed	0.016	0.018	0.020
2. Clean, after weathering	0.018	0.022	0.025
3. Gravel, uniform section, clean	0.022	0.025	0.030
4. With short grass, few weeds	0.022	0.027	0.033
b. Earth, winding and sluggish			
1. No vegetation	0.023	0.025	0.030
2. Grass, some weeds	0.025	0.030	0.033
3. Dense weeds or aquatic plants in deep channels	0.030	0.035	0.040
4. Earth bottom and rubble sides	0.028	0.030	0.035
5. Stony bottom and weedy banks	0.025	0.035	0.040
6. Cobble bottom and clean sides	0.030	0.040	0.050
c. Dragline-excavated or dredged			
1. No vegetation	0.025	0.028	0.033
2. Light brush on banks	0.035	0.050	0.060
d. Rock cuts			
1. Smooth and uniform	0.025	0.035	0.040
2. Jagged and irregular	0.035	0.040	0.050
e. Channels not maintained, weeds and brush uncut			
1. Dense weeds, high as flow depth	0.050	0.080	0.120
2. Clean bottom, brush on sides	0.040	0.050	0.080
3. Same, highest stage of flow	0.045	0.070	0.110
4. Dense brush, high stage	0.080	0.100	0.140

Table 4. Maximum Velocities for Vegetative Channel Linings (from Georgetown County, 2006)

Vegetation Type	Slope Range (%)[1]	Maximum Velocity[2] (ft/s)
Bermuda Grass	0 - 5	6
	5 - 10	5
Bahia	All	4
Tall Fescue Grass	0 - 10	4
Mixtures[3]		
Kentucky Bluegrass	0 - 5	5
Buffalo Grass	5 - 10	4
	>10	3
Grass Mixture	0 - 5[1]	4
	5 - 10	3
Sericea Lespedeza,	0 - 5[4]	2.5
Weeping Lovegrass, Alfalfa	All	
Annuals[5]	0 - 5	2.5
Sod	All	4
Lapped Sod	All	5.5

[1] Do not use on slopes steeper than 10 percent except for side-slope in combination channels.
[2] Use velocities exceeding 5 feet per second only where good stands can be established and maintained.
[3] Mixtures of Tall Fescue, Bahia, and or Bermuda.
[4] Do not use on slopes steeper than 5 percent except for side-slope in combination channels.
[5] Annuals - used on mild slopes or as temporary protection until permanent covers are established.

Table 5. Typical Roughness Coefficients for Lined or Built-Up Channels (from Chow, 1959)

Type of channel and description	Minimum	Normal	Maximum
B. LINED OR BUILT-UP CHANNELS			
B-1. Metal			
a. Smooth steel surface			
1. Unpainted	0.011	0.012	0.014
2. Painted	0.012	0.013	0.017
b. Corrugated	0.021	0.025	0.030
B-2. Nonmetal			
a. Cement			
1. Neat, surface	0.010	0.011	0.013
2. Mortar	0.011	0.013	0.015
b. Wood			
1. Planed, untreated	0.010	0.012	0.014
2. Planed, creosoted	0.011	0.012	0.015
3. Unplaned	0.011	0.013	0.015
4. Plank with battens	0.012	0.015	0.018
5. Lined with roofing paper	0.010	0.014	0.017
c. Concrete			
1. Trowel finish	0.011	0.013	0.015
2. Float finish	0.013	0.015	0.016
3. Finished, with gravel on bottom	0.015	0.017	0.020
4. Unfinished	0.014	0.017	0.020
5. Gunite, good section	0.016	0.019	0.023
6. Gunite, wavy section	0.018	0.022	0.025
7. On good excavated rock	0.017	0.020	
8. On irregular excavated rock	0.022	0.027	
d. Concrete bottom float finished with sides of			
1. Dressed stone in mortar	0.015	0.017	0.020
2. Random stone in mortar	0.017	0.020	0.024
3. Cement rubble masonry, plastered	0.016	0.020	0.024
4. Cement rubble masonry	0.020	0.025	0.030
5. Dry rubble or riprap	0.020	0.030	0.035
e. Gravel bottom with sides of			
1. Formed concrete	0.017	0.020	0.025
2. Random stone in mortar	0.020	0.023	0.026
3. Dry rubble or riprap	0.023	0.033	0.036

Table 6. Typical Permissible Shear Stresses for Bare Soil and Stone Linings (FHWA, 2005)

Lining Category	Lining Type	Permissible Shear Stress N/m^2	Permissible Shear Stress lb/ft^2
Bare Soil[1] Cohesive (PI = 10)	Clayey sands	1.8-4.5	0.037-0.095
	Inorganic silts	1.1-4.0	0.027-0.11
	Silty sands	1.1-3.4	0.024-0.072
Bare Soil[1] Cohesive (PI $\geq$ 20)	Clayey sands	4.5	0.094
	Inorganic silts	4.0	0.083
	Silty sands	3.5	0.072
	Inorganic clays	6.6	0.14
Bare Soil[2] Non-cohesive (PI < 10)	Finer than coarse sand D_{75}<1.3 mm (0.05 in)	1.0	0.02
	Fine gravel D_{75}=7.5 mm (0.3 in)	5.6	0.12
	Gravel D_{75}=15 mm (0.6 in)	11	0.24
Gravel Mulch[3]	Coarse gravel D_{50} = 25 mm (1 in)	19	0.4
	Very coarse gravel D_{50} = 50 mm (2 in)	38	0.8
Rock Riprap[3]	D_{50} = 0.15 m (0.5 ft)	113	2.4
	D_{50} = 0.30 m (1.0 ft)	227	4.8

[1]Based on Equation 4.6 assuming a soil void ratio of 0.5 (USDA, 1987).
[2]Based on Equation 4.5 derived from USDA (1987)
[3]Based on Equation 6.7 with Shield's parameter equal to 0.047.

Table 7. Summary of Calculated Results for Drainage Terraces

	25-year, 24-hour Design Event				100-year, 24-hour Design Event				
Drainage Feature	**Flow Rate Q_{25} (cfs)**	**Depth of Flow (ft)**	**Average Velocity (ft/s)**	**Average Tractive Stress (psf)**	**Flow Rate Q_{100} (cfs)**	**Depth of Flow (ft)**	**Average Velocity (ft/s)**	**Average Tractive Stress (psf)**	**Channel Lining**
Sideslope	17.7	0.97	4.71	0.59	24.9	1.10	5.13	0.66	Vegetation
Top Deck	13.3	0.75	2.03	0.12	18.5	0.85	2.20	0.13	Vegetation
Access Road 1B2A	7.9	0.74	5.74	1.71	11.0	0.84	6.23	1.94	Riprap
Access Road 1BA	13.7	0.91	6.59	2.11	19.1	1.03	7.15	2.39	Riprap
Access Road 2A2A	13.3	0.90	6.54	2.08	18.6	1.02	7.11	2.36	Riprap
Access Road 2BB	11.5	0.85	6.30	1.97	16.1	0.97	6.86	2.24	Riprap

Table 8. Summary of Calculated Results for Downdrain Pipe Features

Drainage Feature[1]	25-year Flow Rate Q_{25} (cfs)	Depth of Flow (ft)	Average Velocity (ft/s)	100-year Flow Rate Q_{100} (cfs)	Depth of Flow (ft)	Average Velocity (ft/s)	No. of Pipes
1A1	15.9	0.8	17.4	22.3	0.95	18.8	1
1B1	32.3	0.8	17.5	45.1	0.96	18.9	2
1B2	23.7	0.6	16.1	33.1	0.79	17.6	2
1B3	24.6	0.7	16.3	34.5	0.81	17.8	2
1C1	36.1	0.8	18.0	50.7	1.05	19.3	2
2A1	46.0	0.8	17.3	64.5	0.93	18.7	3
2A2	21.0	0.9	18.6	29.4	1.19	19.6	1
2C1	85.7	0.8	17.7	120.1	1.01	19.1	5
2E1	42.4	0.9	18.6	59.3	1.20	19.6	2
2F1	79.3	0.9	18.3	110.9	1.13	19.5	4
2F2	96.3	0.9	18.2	134.9	1.10	19.5	5

Note:

1. "Drainage feature" refers to each downdrain location/designation, as shown on the surface water management system design drawings presented in the set of Engineering Drawings that accompanies the Engineering Report.

Table 9. Distribution of Downdrain Pipes at each Sideslope Drainage Terrace Location

Drainage Feature[1]	No. of Pipes	Number of Pipes Required at Sideslope Drainage Terraces (SSDTs)					
		SSDT 1	SSDT 2	SSDT 3	SSDT 4	SSDT 5	SSDT 6
1A1	1	1	1	1	N/A	N/A	N/A
1B1	2	2	2	1	N/A	N/A	N/A
1B2	2	2	1	1	N/A	N/A	N/A
1B3	2	2	1	N/A	N/A	N/A	N/A
1C1	2	2	2	1	N/A	N/A	N/A
2A1	3	3	2	1	1	1	N/A
2A2	1	1	1	1	N/A	N/A	N/A
2C1	5	5	4	3	2	1	1
2E1	2	2	2	1	1	1	N/A
2F1	4	4	3	3	2	2	1
2F2	5	5	4	3	2	1	1

Note:

1. "Drainage feature" refers to each downdrain location/designation, as shown on the surface water management system design drawings presented in the set of Engineering Drawings that accompanies the Engineering Report. The SSDTs at each downdrain location are numbered starting with "1" being the lowest drainage terrace, and in ascending order going up the slope. "N/A" (not applicable) means that SSDT number does not exist at that specific downdrain location.

FIGURES

- Figure 1. Landfill Area 1 Drainage Plan with Critical Terrace Areas
- Figure 2. Landfill Area 2 Drainage Plan with Critical Terrace Areas
- Figure 3. Manning's n for HDPE (from Barfuss and Tullis, 1989)

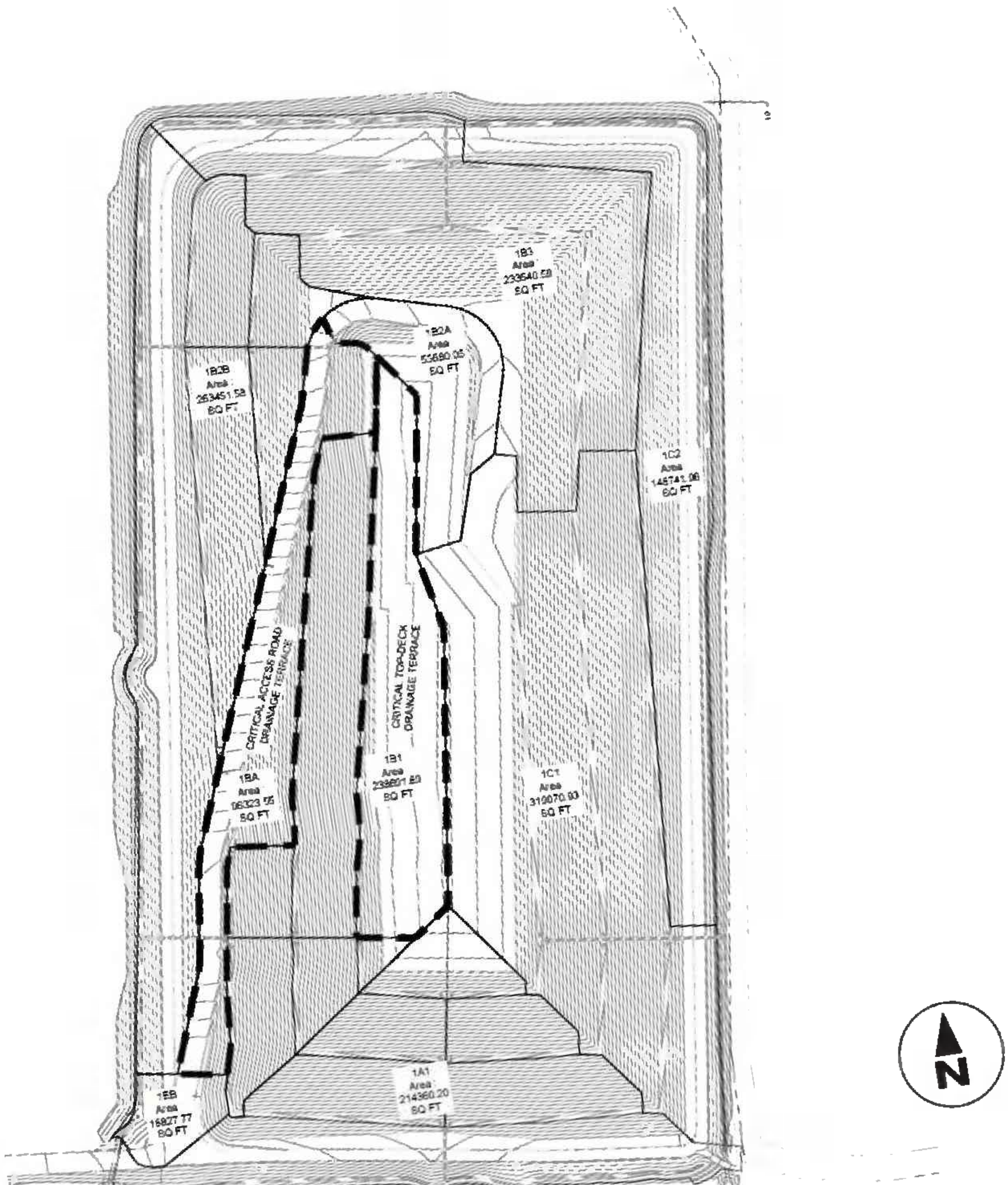


Figure 1. Landfill Area 1 Drainage Plan with Critical Terrace Areas

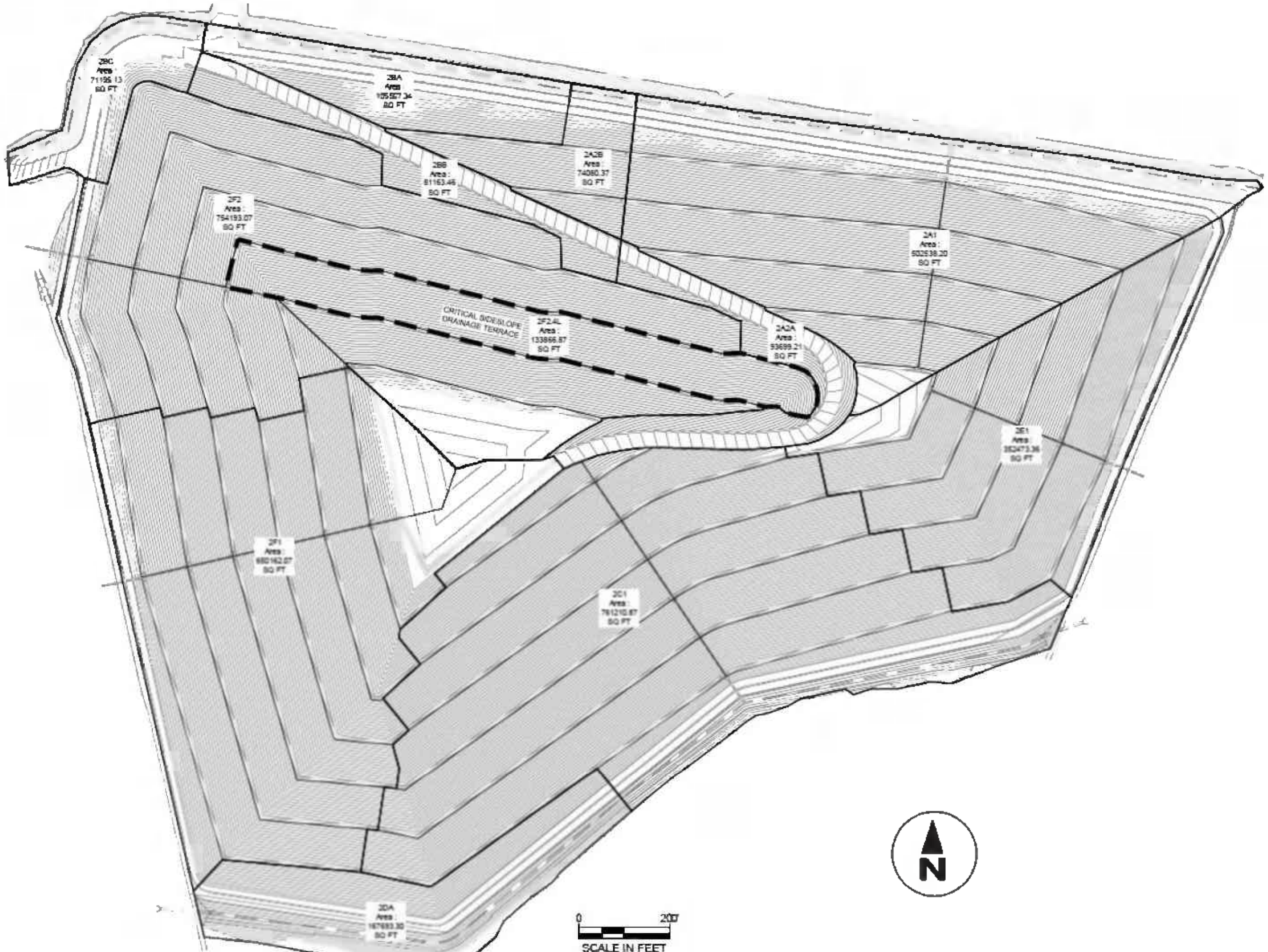
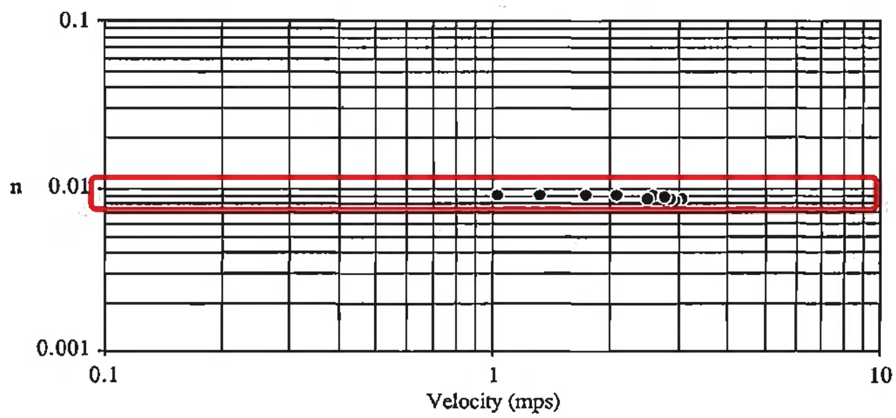


Figure 2. Landfill Area 2 Drainage Plan with Critical Terrace Areas



Manning n vs. Velocity for 300mm Diameter HDPE Pipe

Figure 3. Manning's n for HDPE

(from Barfuss and Tullis, 1989)

Appendix 1

Drainage Feature Calculations

Design/Check: Trapezoidal/Triangular Channel

Methodology: Manning's Equation

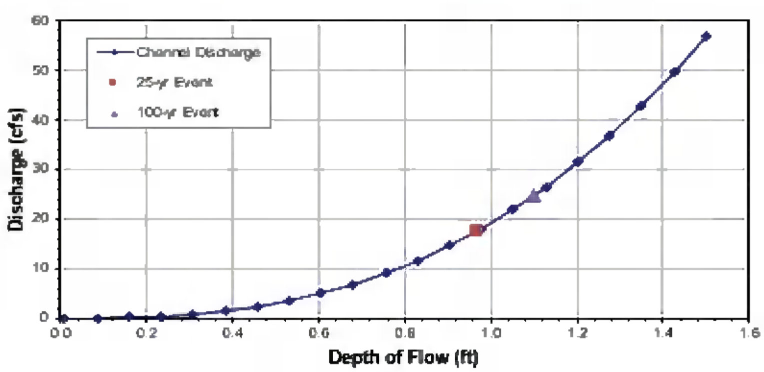
Project: Santee Cooper - Winyah Generating Station

Ditch ID: **Side Slope Drainage Terrace**

Peak Discharge, Q_{25} =	17.75	cfs (25-yr Event)
Peak Discharge, Q_{100} =	24.87	cfs (100-yr Event)
Bottom Width, B =	0.00	ft
Left Side Slope, Z_1 =	5.0	horizontal :1 vertical
Right Side Slope, Z_2 =	3.0	horizontal :1 vertical
Channel Depth, Y =	1.50	ft
Top Width, T =	12.0	ft
Manning's Roughness Coeff., n =	0.027	
Longitudinal Channel Slope, S_o =	0.0200	ft/ft

Depth of Flow Y ft	Area of Flow A ft²	Wetted Perimeter P ft	Hydraulic Radius R=A/P ft	Average Velocity V ft/s	Discharge (Flow Rate) Q=AV ft³/s	Avg. Tractive Stress τ_0 lb/ft²	Comments
0.01	0.00	0.08	0.00	0.22	0.0	0.01	
0.08	0.03	0.70	0.04	0.93	0.0	0.05	
0.16	0.10	1.31	0.08	1.41	0.1	0.10	
0.23	0.22	1.93	0.11	1.82	0.4	0.14	
0.31	0.38	2.54	0.15	2.19	0.8	0.19	
0.38	0.59	3.16	0.19	2.53	1.5	0.23	
0.46	0.84	3.78	0.22	2.85	2.4	0.28	
0.53	1.13	4.39	0.26	3.16	3.6	0.32	
0.61	1.47	5.01	0.29	3.44	5.1	0.37	
0.68	1.85	5.62	0.33	3.72	6.9	0.41	
0.76	2.28	6.24	0.37	3.99	9.1	0.46	
0.83	2.75	6.85	0.40	4.25	11.7	0.50	
0.90	3.27	7.47	0.44	4.50	14.7	0.55	
0.98	3.83	8.08	0.47	4.74	18.2	0.59	
1.05	4.44	8.70	0.51	4.98	22.1	0.64	
1.13	5.09	9.31	0.55	5.21	26.5	0.68	
1.20	5.78	9.93	0.58	5.44	31.4	0.73	
1.28	6.52	10.55	0.62	5.66	36.9	0.77	
1.35	7.30	11.16	0.65	5.88	42.9	0.82	
1.43	8.13	11.78	0.69	6.09	49.5	0.86	
1.50	9.00	12.39	0.73	6.31	56.7	0.91	
0.97	3.76	8.01	0.47	4.71	17.73	0.59	Q (25-yr Event)
1.10	4.84	9.09	0.53	5.13	24.81	0.66	Q (100-yr Event)

Discharge versus Depth Relationship



Design/Check: Trapezoidal/Triangular Channel

Methodology: Manning's Equation

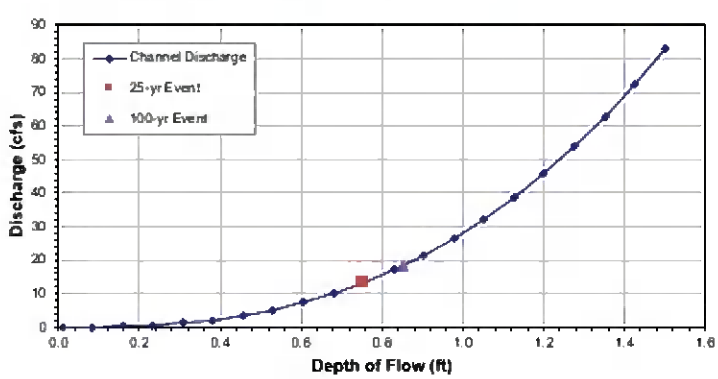
Project: Santee Cooper - Winyah Generating Station

Ditch ID: **Top Deck Drainage Terrace**

Peak Discharge, Q_{25} =	13.27	cfs (25-yr Event)
Peak Discharge, Q_{100} =	18.52	cfs (100-yr Event)
Bottom Width, B =	0.00	ft
Left Side Slope, Z_1 =	20.0	horizontal:1 vertical
Right Side Slope, Z_2 =	3.0	horizontal:1 vertical
Channel Depth, Y =	1.50	ft
Top Width, T =	34.5	ft
Manning's Roughness Coeff., n =	0.027	
Longitudinal Channel Slope, S_o =	0.0050	ft/ft

Depth of Flow Y ft	Area of Flow A ft²	Wetted Perimeter P ft	Hydraulic Radius R=A/P ft	Average Velocity V ft/s	Discharge (Flow Rate) Q=AV ft³/s	Avg. Tractive Stress τ_0 lb/ft²	Comments
0.01	0.00	0.23	0.00	0.11	0.0	0.00	
0.08	0.08	1.96	0.04	0.47	0.0	0.01	
0.16	0.29	3.69	0.08	0.72	0.2	0.02	
0.23	0.63	5.41	0.12	0.93	0.6	0.04	
0.31	1.09	7.14	0.15	1.11	1.2	0.05	
0.38	1.68	8.87	0.19	1.29	2.2	0.06	
0.46	2.40	10.60	0.23	1.45	3.5	0.07	
0.53	3.25	12.32	0.26	1.60	5.2	0.08	
0.61	4.22	14.05	0.30	1.75	7.4	0.09	
0.68	5.33	15.78	0.34	1.89	10.1	0.11	
0.76	6.56	17.51	0.37	2.03	13.3	0.12	
0.83	7.91	19.23	0.41	2.16	17.1	0.13	
0.90	9.40	20.96	0.45	2.29	21.5	0.14	
0.98	11.01	22.69	0.49	2.41	26.5	0.15	
1.05	12.75	24.42	0.52	2.53	32.3	0.16	
1.13	14.62	26.14	0.56	2.65	38.7	0.17	
1.20	16.62	27.87	0.60	2.76	45.9	0.19	
1.28	18.74	29.60	0.63	2.88	53.9	0.20	
1.35	20.99	31.33	0.67	2.99	62.7	0.21	
1.43	23.37	33.05	0.71	3.10	72.4	0.22	
1.50	25.88	34.78	0.74	3.20	82.9	0.23	
0.75	6.55	17.50	0.37	2.03	13.26	0.12	Q (25-yr Event)
0.85	8.39	19.80	0.42	2.20	18.45	0.13	Q (100-yr Event)

Discharge versus Depth Relationship



Design/Check: Trapezoidal/Triangular Channel

Methodology: Manning's Equation

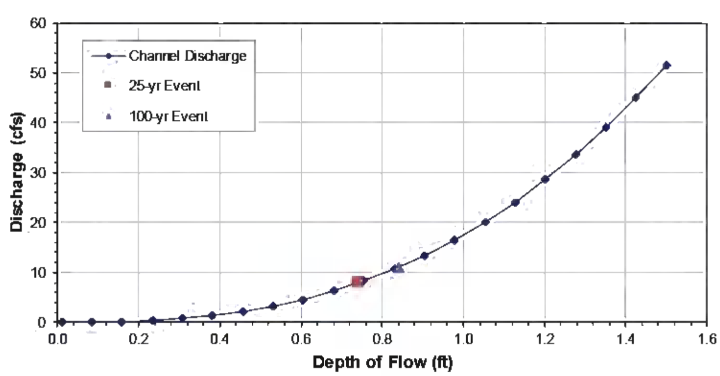
Project: Santee Cooper - Winyah Generating Station

Ditch ID: 1B2A Access Road Drainage Terrace

Peak Discharge, Q_{25} =	7.90	cfs (25-yr Event)
Peak Discharge, Q_{100} =	11.00	cfs (100-yr Event)
Bottom Width, B =	0.00	ft
Left Side Slope, Z_1 =	3.0	horizontal :1 vertical
Right Side Slope, Z_2 =	2.0	horizontal :1 vertical
Channel Depth, Y =	1.50	ft
Top Width, T =	7.5	ft
Manning's Roughness Coeff., n =	0.036	
Longitudinal Channel Slope, S_o =	0.0800	ft/ft

Depth of Flow Y ft	Area of Flow A ft^2	Wetted Perimeter P ft	Hydraulic Radius R=A/P ft	Average Velocity V ft/s	Discharge (Flow Rate) Q=AV ft^3/s	Avg. Tractive Stress τ_0 lb/ft^2	Comments
0.01	0.00	0.05	0.00	0.32	0.0	0.02	
0.08	0.02	0.46	0.04	1.35	0.0	0.20	
0.16	0.06	0.86	0.07	2.05	0.1	0.37	
0.23	0.14	1.26	0.11	2.66	0.4	0.54	
0.31	0.24	1.66	0.14	3.19	0.8	0.71	
0.38	0.37	2.06	0.18	3.69	1.3	0.88	
0.46	0.52	2.47	0.21	4.16	2.2	1.06	
0.53	0.71	2.87	0.25	4.60	3.2	1.23	
0.61	0.92	3.27	0.28	5.02	4.6	1.40	
0.68	1.16	3.67	0.32	5.42	6.3	1.57	
0.76	1.43	4.08	0.35	5.81	8.3	1.75	
0.83	1.72	4.48	0.38	6.18	10.6	1.92	
0.90	2.04	4.88	0.42	6.55	13.4	2.09	
0.98	2.39	5.28	0.45	6.90	16.5	2.26	
1.05	2.77	5.68	0.49	7.25	20.1	2.43	
1.13	3.18	6.09	0.52	7.59	24.1	2.61	
1.20	3.61	6.49	0.56	7.92	28.6	2.78	
1.28	4.07	6.89	0.59	8.24	33.6	2.95	
1.35	4.56	7.29	0.63	8.56	39.1	3.12	
1.43	5.08	7.70	0.66	8.87	45.1	3.30	
1.50	5.63	8.10	0.69	9.18	51.6	3.47	
0.74	1.37	4.00	0.34	5.74	7.87	1.71	Q (25-yr Event)
0.84	1.76	4.53	0.39	6.23	10.98	1.94	Q (100-yr Event)

Discharge versus Depth Relationship



Design/Check: Trapezoidal/Triangular Channel

Methodology: Manning's Equation

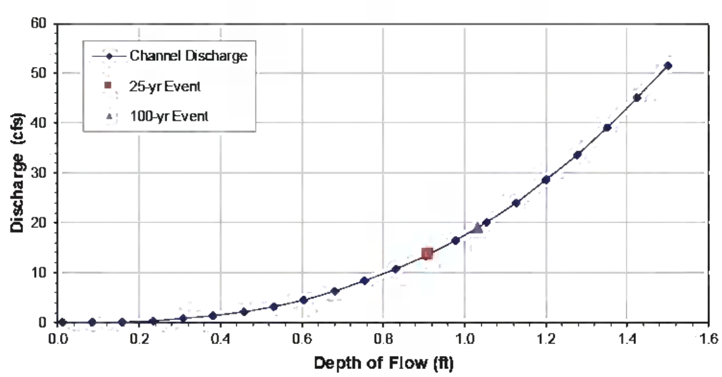
Project: Santee Cooper - Winyah Generating Station

Ditch ID: **1BA Access Road Drainage Terrace**

Peak Discharge, Q_{25} =	13.70	cfs (25-yr Event)
Peak Discharge, Q_{100} =	19.10	cfs (100-yr Event)
Bottom Width, B =	0.00	ft
Left Side Slope, Z_1 =	3.0	horizontal:1 vertical
Right Side Slope, Z_2 =	2.0	horizontal:1 vertical
Channel Depth, Y =	1.50	ft
Top Width, T =	7.5	ft
Manning's Roughness Coeff., n =	0.036	
Longitudinal Channel Slope, S_o =	0.0800	ft/ft

Depth of Flow Y ft	Area of Flow A ft^2	Wetted Perimeter P ft	Hydraulic Radius R=A/P ft	Average Velocity V ft/s	Discharge (Flow Rate) Q=AV ft^3/s	Avg. Tractive Stress τ_0 lb/ft^2	Comments
0.01	0.00	0.05	0.00	0.32	0.0	0.02	
0.08	0.02	0.46	0.04	1.35	0.0	0.20	
0.16	0.06	0.86	0.07	2.05	0.1	0.37	
0.23	0.14	1.26	0.11	2.66	0.4	0.54	
0.31	0.24	1.66	0.14	3.19	0.8	0.71	
0.38	0.37	2.06	0.18	3.69	1.3	0.88	
0.46	0.52	2.47	0.21	4.16	2.2	1.06	
0.53	0.71	2.87	0.25	4.60	3.2	1.23	
0.61	0.92	3.27	0.28	5.02	4.6	1.40	
0.68	1.16	3.67	0.32	5.42	6.3	1.57	
0.76	1.43	4.08	0.35	5.81	8.3	1.75	
0.83	1.72	4.48	0.38	6.18	10.6	1.92	
0.90	2.04	4.88	0.42	6.55	13.4	2.09	
0.98	2.39	5.28	0.45	6.90	16.5	2.26	
1.05	2.77	5.68	0.49	7.25	20.1	2.43	
1.13	3.18	6.09	0.52	7.59	24.1	2.61	
1.20	3.61	6.49	0.56	7.92	28.6	2.78	
1.28	4.07	6.89	0.59	8.24	33.6	2.95	
1.35	4.56	7.29	0.63	8.56	39.1	3.12	
1.43	5.08	7.70	0.66	8.87	45.1	3.30	
1.50	5.63	8.10	0.69	9.18	51.6	3.47	
0.91	2.08	4.92	0.42	6.59	13.68	2.11	Q (25-yr Event)
1.03	2.66	5.57	0.48	7.15	19.06	2.39	Q (100-yr Event)

Discharge versus Depth Relationship



Design/Check: Trapezoidal/Triangular Channel

Methodology: Manning's Equation

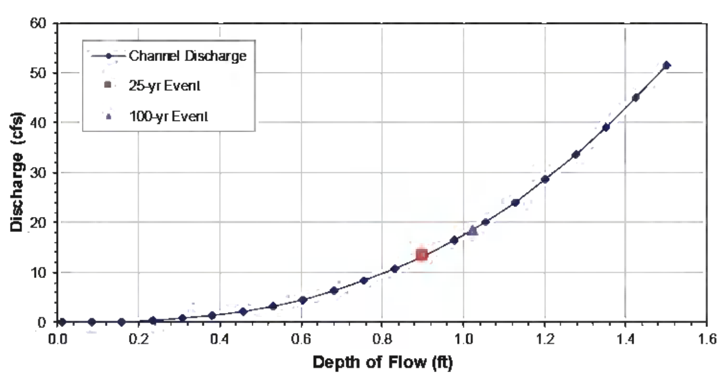
Project: Santee Cooper - Winyah Generating Station

Ditch ID: 2A2A Access Road Drainage Terrace

Peak Discharge, Q_{25} =	13.30	cfs (25-yr Event)
Peak Discharge, Q_{100} =	18.60	cfs (100-yr Event)
Bottom Width, B =	0.00	ft
Left Side Slope, Z_1 =	3.0	horizontal :1 vertical
Right Side Slope, Z_2 =	2.0	horizontal :1 vertical
Channel Depth, Y =	1.50	ft
Top Width, T =	7.5	ft
Manning's Roughness Coeff., n =	0.036	
Longitudinal Channel Slope, S_o =	0.0800	ft/ft

Depth of Flow Y ft	Area of Flow A ft^2	Wetted Perimeter P ft	Hydraulic Radius R=A/P ft	Average Velocity V ft/s	Discharge (Flow Rate) Q=AV ft^3/s	Avg. Tractive Stress τ_0 lb/ft^2	Comments
0.01	0.00	0.05	0.00	0.32	0.0	0.02	
0.08	0.02	0.46	0.04	1.35	0.0	0.20	
0.16	0.06	0.86	0.07	2.05	0.1	0.37	
0.23	0.14	1.26	0.11	2.66	0.4	0.54	
0.31	0.24	1.66	0.14	3.19	0.8	0.71	
0.38	0.37	2.06	0.18	3.69	1.3	0.88	
0.46	0.52	2.47	0.21	4.16	2.2	1.06	
0.53	0.71	2.87	0.25	4.60	3.2	1.23	
0.61	0.92	3.27	0.28	5.02	4.6	1.40	
0.68	1.16	3.67	0.32	5.42	6.3	1.57	
0.76	1.43	4.08	0.35	5.81	8.3	1.75	
0.83	1.72	4.48	0.38	6.18	10.6	1.92	
0.90	2.04	4.88	0.42	6.55	13.4	2.09	
0.98	2.39	5.28	0.45	6.90	16.5	2.26	
1.05	2.77	5.68	0.49	7.25	20.1	2.43	
1.13	3.18	6.09	0.52	7.59	24.1	2.61	
1.20	3.61	6.49	0.56	7.92	28.6	2.78	
1.28	4.07	6.89	0.59	8.24	33.6	2.95	
1.35	4.56	7.29	0.63	8.56	39.1	3.12	
1.43	5.08	7.70	0.66	8.87	45.1	3.30	
1.50	5.63	8.10	0.69	9.18	51.6	3.47	
0.90	2.03	4.87	0.42	6.54	13.29	2.08	Q (25-yr Event)
1.02	2.61	5.52	0.47	7.11	18.55	2.36	Q (100-yr Event)

Discharge versus Depth Relationship



Design/Check: Trapezoidal/Triangular Channel

Methodology: Manning's Equation

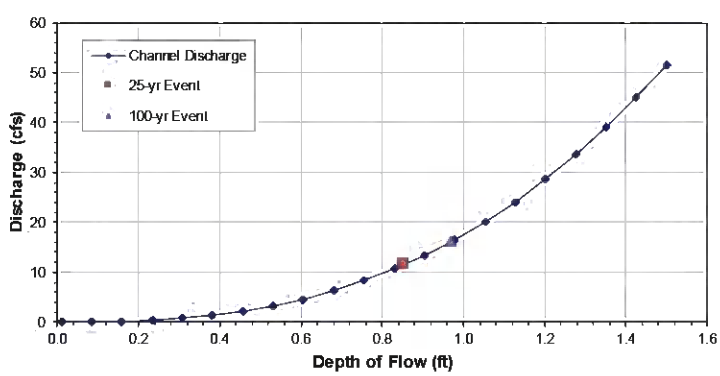
Project: Santee Cooper - Winyah Generating Station

Ditch ID: **2BB Access Road Drainage Terrace**

Peak Discharge, Q_{25} =	11.50	cfs (25-yr Event)
Peak Discharge, Q_{100} =	16.10	cfs (100-yr Event)
Bottom Width, B =	0.00	ft
Left Side Slope, Z_1 =	3.0	horizontal :1 vertical
Right Side Slope, Z_2 =	2.0	horizontal :1 vertical
Channel Depth, Y =	1.50	ft
Top Width, T =	7.5	ft
Manning's Roughness Coeff., n =	0.036	
Longitudinal Channel Slope, S_o =	0.0800	ft/ft

Depth of Flow Y ft	Area of Flow A ft^2	Wetted Perimeter P ft	Hydraulic Radius R=A/P ft	Average Velocity V ft/s	Discharge (Flow Rate) Q=AV ft^3/s	Avg. Tractive Stress τ_0 lb/ft^2	Comments
0.01	0.00	0.05	0.00	0.32	0.0	0.02	
0.08	0.02	0.46	0.04	1.35	0.0	0.20	
0.16	0.06	0.86	0.07	2.05	0.1	0.37	
0.23	0.14	1.26	0.11	2.66	0.4	0.54	
0.31	0.24	1.66	0.14	3.19	0.8	0.71	
0.38	0.37	2.06	0.18	3.69	1.3	0.88	
0.46	0.52	2.47	0.21	4.16	2.2	1.06	
0.53	0.71	2.87	0.25	4.60	3.2	1.23	
0.61	0.92	3.27	0.28	5.02	4.6	1.40	
0.68	1.16	3.67	0.32	5.42	6.3	1.57	
0.76	1.43	4.08	0.35	5.81	8.3	1.75	
0.83	1.72	4.48	0.38	6.18	10.6	1.92	
0.90	2.04	4.88	0.42	6.55	13.4	2.09	
0.98	2.39	5.28	0.45	6.90	16.5	2.26	
1.05	2.77	5.68	0.49	7.25	20.1	2.43	
1.13	3.18	6.09	0.52	7.59	24.1	2.61	
1.20	3.61	6.49	0.56	7.92	28.6	2.78	
1.28	4.07	6.89	0.59	8.24	33.6	2.95	
1.35	4.56	7.29	0.63	8.56	39.1	3.12	
1.43	5.08	7.70	0.66	8.87	45.1	3.30	
1.50	5.63	8.10	0.69	9.18	51.6	3.47	
0.85	1.82	4.60	0.39	6.30	11.46	1.97	Q (25-yr Event)
0.97	2.34	5.23	0.45	6.86	16.08	2.24	Q (100-yr Event)

Discharge versus Depth Relationship



Down Drain Feature: 1A1
Flow Through Circular Pipe

Number of Pipes, N=	1	
Diameter of pipe, D=	18	inches
Longitudinal Slope, So=	0.040	ft/ft
Manning's n=	0.009	
Density of flowing liquid, rho=	1.94	slugs/ft^3
Peak Discharge, Q_{25} =	15.9	cfs
Peak Discharge, Q_{100} =	22.3	cfs

Theta radians	Theta degrees	Depth of Flow y inches	Area of Flow A ft^2	Wetted Perimeter P ft	Hydraulic Radius R ft	Average Velocity V ft/s	Discharge Q=A*V cfs	Force F lbf
0.00	0	0.0	0.000	0.00		0.0	0.00	0.0
0.25	14	0.1	0.001	0.19	0.00	0.8	0.00	0.0
0.50	29	0.3	0.006	0.38	0.02	2.0	0.01	0.0
0.75	43	0.6	0.019	0.56	0.03	3.5	0.07	0.5
1.00	57	1.1	0.045	0.75	0.06	5.0	0.22	2.2
1.25	72	1.7	0.085	0.94	0.09	6.7	0.56	7.3
1.50	86	2.4	0.141	1.13	0.13	8.3	1.17	18.9
1.75	100	3.2	0.215	1.31	0.16	9.9	2.14	41.1
2.00	115	4.1	0.307	1.50	0.20	11.5	3.52	78.5
2.25	129	5.1	0.414	1.69	0.25	13.0	5.37	135.1
2.50	143	6.2	0.535	1.88	0.29	14.3	7.67	213.4
2.75	158	7.2	0.666	2.06	0.32	15.6	10.38	313.7
3.00	172	8.4	0.804	2.25	0.36	16.7	13.40	433.4
3.25	186	9.5	0.944	2.44	0.39	17.6	16.62	567.1
3.50	201	10.6	1.083	2.63	0.41	18.3	19.87	707.1
3.75	215	11.7	1.215	2.81	0.43	18.9	23.00	844.2
4.00	229	12.7	1.338	3.00	0.45	19.3	25.85	969.0
4.25	244	13.7	1.447	3.19	0.45	19.6	28.29	1073.2
4.50	258	14.7	1.541	3.38	0.46	19.6	30.23	1151.0
4.75	272	15.5	1.617	3.56	0.45	19.6	31.61	1199.0
5.00	286	16.2	1.676	3.75	0.45	19.3	32.43	1217.3
5.25	301	16.8	1.718	3.94	0.44	19.0	32.72	1208.8
5.50	315	17.3	1.745	4.13	0.42	18.7	32.56	1178.4
5.75	329	17.7	1.760	4.31	0.41	18.2	32.06	1132.7
6.00	344	17.9	1.766	4.50	0.39	17.7	31.34	1078.7
6.25	358	18.0	1.767	4.69	0.38	17.3	30.53	1022.9
3.20	183	9.2	0.914	2.40	0.38	17.4	15.92	537.7
3.69	212	11.4	1.186	2.77	0.43	18.8	22.30	813.8

Down Drain Feature: 1B1
Flow Through Circular Pipe

Number of Pipes, N=	2	
Diameter of pipe, D=	18	inches
Longitudinal Slope, So=	0.040	ft/ft
Manning's n=	0.009	
Density of flowing liquid, rho=	1.94	slugs/ft^3
Peak Discharge, Q_{25} =	32.3	cfs
Peak Discharge, Q_{100} =	45.1	cfs

Theta radians	Theta degrees	Depth of Flow y inches	Area of Flow A ft^2	Wetted Perimete P ft	Hydraulic Radius R ft	Average Velocity V ft/s	Discharge Q=A*V cfs	Force F lbf
0.00	0	0.0	0.000	0.00		0.0	0.00	0.0
0.25	14	0.1	0.001	0.19	0.00	0.8	0.00	0.0
0.50	29	0.3	0.006	0.38	0.02	2.0	0.02	0.0
0.75	43	0.6	0.019	0.56	0.03	3.5	0.13	0.5
1.00	57	1.1	0.045	0.75	0.06	5.0	0.45	2.2
1.25	72	1.7	0.085	0.94	0.09	6.7	1.13	7.3
1.50	86	2.4	0.141	1.13	0.13	8.3	2.35	18.9
1.75	100	3.2	0.215	1.31	0.16	9.9	4.27	41.1
2.00	115	4.1	0.307	1.50	0.20	11.5	7.05	78.5
2.25	129	5.1	0.414	1.69	0.25	13.0	10.74	135.1
2.50	143	6.2	0.535	1.88	0.29	14.3	15.34	213.4
2.75	158	7.2	0.686	2.06	0.32	15.8	20.76	313.7
3.00	172	8.4	0.804	2.25	0.36	16.7	26.80	433.4
3.25	186	9.5	0.944	2.44	0.39	17.6	33.23	567.1
3.50	201	10.6	1.083	2.63	0.41	18.3	39.74	707.1
3.75	215	11.7	1.215	2.81	0.43	18.9	45.99	844.2
4.00	229	12.7	1.338	3.00	0.45	19.3	51.70	969.0
4.25	244	13.7	1.447	3.19	0.45	19.6	56.59	1073.2
4.50	258	14.7	1.541	3.38	0.46	19.6	60.46	1151.0
4.75	272	15.5	1.617	3.56	0.45	19.6	63.23	1199.0
5.00	286	16.2	1.676	3.75	0.45	19.3	64.86	1217.3
5.25	301	16.8	1.718	3.94	0.44	19.0	65.44	1208.8
5.50	315	17.3	1.745	4.13	0.42	18.7	65.12	1178.4
5.75	329	17.7	1.760	4.31	0.41	18.2	64.12	1132.7
6.00	344	17.9	1.766	4.50	0.39	17.7	62.67	1078.7
6.25	358	18.0	1.767	4.69	0.38	17.3	61.05	1022.9
3.21	184	9.3	0.924	2.41	0.38	17.5	32.27	547.0
3.71	213	11.5	1.196	2.78	0.43	18.9	45.11	824.9

Down Drain Feature: 1B2
Flow Through Circular Pipe

Number of Pipes, N=	2	
Diameter of pipe, D=	18	inches
Longitudinal Slope, So=	0.040	ft/ft
Manning's n=	0.009	
Density of flowing liquid, rho=	1.94	slugs/ft^3
Peak Discharge, Q_{25} =	23.7	cfs
Peak Discharge, Q_{100} =	33.1	cfs

Theta radians	Theta degrees	Depth of Flow y inches	Area of Flow A ft^2	Wetted Perimeter P ft	Hydraulic Radius R ft	Average Velocity V ft/s	Discharge Q=A*V cfs	Force F lbf
0.00	0	0.0	0.000	0.00		0.0	0.00	0.0
0.25	14	0.1	0.001	0.19	0.00	0.8	0.00	0.0
0.50	29	0.3	0.006	0.38	0.02	2.0	0.02	0.0
0.75	43	0.6	0.019	0.56	0.03	3.5	0.13	0.5
1.00	57	1.1	0.045	0.75	0.06	5.0	0.45	2.2
1.25	72	1.7	0.085	0.94	0.09	6.7	1.13	7.3
1.50	86	2.4	0.141	1.13	0.13	8.3	2.35	18.9
1.75	100	3.2	0.215	1.31	0.16	9.9	4.27	41.1
2.00	115	4.1	0.307	1.50	0.20	11.5	7.05	78.5
2.25	129	5.1	0.414	1.69	0.25	13.0	10.74	135.1
2.50	143	6.2	0.535	1.88	0.29	14.3	15.34	213.4
2.75	158	7.2	0.666	2.06	0.32	15.6	20.76	313.7
3.00	172	8.4	0.804	2.25	0.36	16.7	26.80	433.4
3.25	186	9.5	0.944	2.44	0.39	17.6	33.23	567.1
3.50	201	10.6	1.083	2.63	0.41	18.3	39.74	707.1
3.75	215	11.7	1.215	2.81	0.43	18.9	45.99	844.2
4.00	229	12.7	1.338	3.00	0.45	19.3	51.70	969.0
4.25	244	13.7	1.447	3.19	0.45	19.6	56.59	1073.2
4.50	258	14.7	1.541	3.38	0.46	19.6	60.46	1151.0
4.75	272	15.5	1.617	3.56	0.45	19.6	63.23	1199.0
5.00	286	16.2	1.676	3.75	0.45	19.3	64.86	1217.3
5.25	301	16.8	1.718	3.94	0.44	19.0	65.44	1208.8
5.50	315	17.3	1.745	4.13	0.42	18.7	65.12	1178.4
5.75	329	17.7	1.760	4.31	0.41	18.2	64.12	1132.7
6.00	344	17.9	1.766	4.50	0.39	17.7	62.67	1078.7
6.25	358	18.0	1.767	4.69	0.38	17.3	61.05	1022.9
2.87	165	7.8	0.733	2.15	0.34	16.1	23.67	370.7
3.25	186	9.5	0.943	2.44	0.39	17.6	33.16	565.6

Down Drain Feature: 1B3

Flow Through Circular Pipe

Number of Pipes, N=	2	
Diameter of pipe, D=	18	inches
Longitudinal Slope, So=	0.040	ft/ft
Manning's n=	0.009	
Density of flowing liquid, rho=	1.94	slugs/ft^3
Peak Discharge, Q_{25}=	24.6	cfs
Peak Discharge, Q_{100}=	34.5	cfs

Theta radians	Theta degrees	Depth of Flow y inches	Area of Flow A ft^2	Wetted Perimeter P ft	Hydraulic Radius R ft	Average Velocity V ft/s	Discharge Q=A*V cfs	Force F lbf
0.00	0	0.0	0.000	0.00		0.0	0.00	0.0
0.25	14	0.1	0.001	0.19	0.00	0.8	0.00	0.0
0.50	29	0.3	0.006	0.38	0.02	2.0	0.02	0.0
0.75	43	0.6	0.019	0.56	0.03	3.5	0.13	0.5
1.00	57	1.1	0.045	0.75	0.06	5.0	0.45	2.2
1.25	72	1.7	0.085	0.94	0.09	6.7	1.13	7.3
1.50	86	2.4	0.141	1.13	0.13	8.3	2.35	18.9
1.75	100	3.2	0.215	1.31	0.16	9.9	4.27	41.1
2.00	115	4.1	0.307	1.50	0.20	11.5	7.05	78.5
2.25	129	5.1	0.414	1.69	0.25	13.0	10.74	135.1
2.50	143	6.2	0.535	1.88	0.29	14.3	15.34	213.4
2.75	158	7.2	0.666	2.06	0.32	15.6	20.76	313.7
3.00	172	8.4	0.804	2.25	0.36	16.7	26.80	433.4
3.25	186	9.5	0.944	2.44	0.39	17.6	33.23	567.1
3.50	201	10.6	1.083	2.63	0.41	18.3	39.74	707.1
3.75	215	11.7	1.215	2.81	0.43	18.9	45.99	844.2
4.00	229	12.7	1.338	3.00	0.45	19.3	51.70	969.0
4.25	244	13.7	1.447	3.19	0.45	19.6	56.59	1073.2
4.50	258	14.7	1.541	3.38	0.46	19.6	60.46	1151.0
4.75	272	15.5	1.617	3.56	0.45	19.6	63.23	1199.0
5.00	286	16.2	1.676	3.75	0.45	19.3	64.86	1217.3
5.25	301	16.8	1.718	3.94	0.44	19.0	65.44	1208.8
5.50	315	17.3	1.745	4.13	0.42	18.7	65.12	1178.4
5.75	329	17.7	1.760	4.31	0.41	18.2	64.12	1132.7
6.00	344	17.9	1.766	4.50	0.39	17.7	62.67	1078.7
6.25	358	18.0	1.767	4.69	0.38	17.3	61.05	1022.9
2.91	167	8.0	0.753	2.18	0.35	16.3	24.53	387.7
3.30	189	9.7	0.971	2.47	0.39	17.8	34.49	594.1

Down Drain Feature: 1C1
Flow Through Circular Pipe

Number of Pipes, N=	2	
Diameter of pipe, D=	18	inches
Longitudinal Slope, So=	0.040	ft/ft
Manning's n=	0.009	
Density of flowing liquid, rho=	1.94	slugs/ft^3
Peak Discharge, Q_{25}=	36.1	cfs
Peak Discharge, Q_{100}=	50.7	cfs

Theta radians	Theta degrees	Depth of Flow y inches	Area of Flow A ft^2	Wetted Perimeter P ft	Hydraulic Radius R ft	Average Velocity V ft/s	Discharge Q=A*V cfs	Force F lbf
0.00	0	0.0	0.000	0.00		0.0	0.00	0.0
0.25	14	0.1	0.001	0.19	0.00	0.8	0.00	0.0
0.50	29	0.3	0.006	0.38	0.02	2.0	0.02	0.0
0.75	43	0.6	0.019	0.56	0.03	3.5	0.13	0.5
1.00	57	1.1	0.045	0.75	0.06	5.0	0.45	2.2
1.25	72	1.7	0.085	0.94	0.09	6.7	1.13	7.3
1.50	86	2.4	0.141	1.13	0.13	8.3	2.35	18.9
1.75	100	3.2	0.215	1.31	0.16	9.9	4.27	41.1
2.00	115	4.1	0.307	1.50	0.20	11.5	7.05	78.5
2.25	129	5.1	0.414	1.69	0.25	13.0	10.74	135.1
2.50	143	6.2	0.535	1.88	0.29	14.3	15.34	213.4
2.75	158	7.2	0.666	2.06	0.32	15.6	20.76	313.7
3.00	172	8.4	0.804	2.25	0.36	16.7	26.80	433.4
3.25	186	9.5	0.944	2.44	0.39	17.6	33.23	567.1
3.50	201	10.6	1.083	2.63	0.41	18.3	39.74	707.1
3.75	215	11.7	1.215	2.81	0.43	18.9	45.99	844.2
4.00	229	12.7	1.338	3.00	0.45	19.3	51.70	969.0
4.25	244	13.7	1.447	3.19	0.45	19.8	56.59	1073.2
4.50	258	14.7	1.541	3.38	0.46	19.6	60.46	1151.0
4.75	272	15.5	1.617	3.56	0.45	19.6	63.23	1199.0
5.00	286	16.2	1.676	3.75	0.45	19.3	64.86	1217.3
5.25	301	16.8	1.718	3.94	0.44	19.0	65.44	1208.8
5.50	315	17.3	1.745	4.13	0.42	18.7	65.12	1178.4
5.75	329	17.7	1.760	4.31	0.41	18.2	64.12	1132.7
6.00	344	17.9	1.766	4.50	0.39	17.7	62.67	1078.7
6.25	358	18.0	1.767	4.69	0.38	17.3	61.05	1022.9
3.36	193	10.0	1.007	2.52	0.40	18.0	36.15	629.7
3.96	227	12.6	1.317	2.97	0.44	19.3	50.76	948.8

Down Drain Feature: 2A1
Flow Through Circular Pipe

Number of Pipes, N=	3	
Diameter of pipe, D=	18	inches
Longitudinal Slope, So=	0.040	ft/ft
Manning's n=	0.009	
Density of flowing liquid, rho=	1.94	slugs/ft^3
Peak Discharge, Q_{25} =	46.0	cfs
Peak Discharge, Q_{100} =	64.5	cfs

Theta radians	Theta degrees	Depth of Flow y inches	Area of Flow A ft^2	Wetted Perimeter P ft	Hydraulic Radius R ft	Average Velocity V ft/s	Discharge Q=A*V cfs	Force F lbf
0.00	0	0.0	0.000	0.00		0.0	0.00	0.0
0.25	14	0.1	0.001	0.19	0.00	0.8	0.00	0.0
0.50	29	0.3	0.006	0.38	0.02	2.0	0.04	0.0
0.75	43	0.6	0.019	0.56	0.03	3.5	0.20	0.5
1.00	57	1.1	0.045	0.75	0.06	5.0	0.67	2.2
1.25	72	1.7	0.085	0.94	0.09	6.7	1.69	7.3
1.50	86	2.4	0.141	1.13	0.13	8.3	3.52	18.9
1.75	100	3.2	0.215	1.31	0.16	9.9	6.41	41.1
2.00	115	4.1	0.307	1.50	0.20	11.5	10.57	78.5
2.25	129	5.1	0.414	1.69	0.25	13.0	16.11	135.1
2.50	143	6.2	0.535	1.88	0.29	14.3	23.01	213.4
2.75	158	7.2	0.666	2.06	0.32	15.6	31.13	313.7
3.00	172	8.4	0.804	2.25	0.36	16.7	40.21	433.4
3.25	186	9.5	0.944	2.44	0.39	17.6	49.85	567.1
3.50	201	10.6	1.083	2.63	0.41	18.3	59.61	707.1
3.75	215	11.7	1.215	2.81	0.43	18.9	68.99	844.2
4.00	229	12.7	1.338	3.00	0.45	19.3	77.55	969.0
4.25	244	13.7	1.447	3.19	0.45	19.6	84.88	1073.2
4.50	258	14.7	1.541	3.38	0.46	19.6	90.70	1151.0
4.75	272	15.5	1.617	3.56	0.45	19.6	94.84	1199.0
5.00	286	16.2	1.676	3.75	0.45	19.3	97.29	1217.3
5.25	301	16.8	1.718	3.94	0.44	19.0	98.16	1208.8
5.50	315	17.3	1.745	4.13	0.42	18.7	97.68	1178.4
5.75	329	17.7	1.760	4.31	0.41	18.2	96.17	1132.7
6.00	344	17.9	1.766	4.50	0.39	17.7	94.01	1078.7
6.25	358	18.0	1.767	4.69	0.38	17.3	91.58	1022.9
3.15	181	9.0	0.889	2.36	0.38	17.3	46.00	513.1
3.63	208	11.2	1.153	2.72	0.42	18.7	64.59	780.0

Down Drain Feature: 2A2
Flow Through Circular Pipe

Number of Pipes, N=	1	
Diameter of pipe, D=	18	inches
Longitudinal Slope, So=	0.040	ft/ft
Manning's n=	0.009	
Density of flowing liquid, rho=	1.94	slugs/ft^3
Peak Discharge, Q_{25} =	21.0	cfs
Peak Discharge, Q_{100} =	29.4	cfs

Theta radians	Theta degrees	Depth of Flow y inches	Area of Flow A ft^2	Wetted Perimeter P ft	Hydraulic Radius R ft	Average Velocity V ft/s	Discharge Q=A*V cfs	Force F lbf
0.00	0	0.0	0.000	0.00		0.0	0.00	0.0
0.25	14	0.1	0.001	0.19	0.00	0.8	0.00	0.0
0.50	29	0.3	0.006	0.38	0.02	2.0	0.01	0.0
0.75	43	0.6	0.019	0.56	0.03	3.5	0.07	0.5
1.00	57	1.1	0.045	0.75	0.06	5.0	0.22	2.2
1.25	72	1.7	0.085	0.94	0.09	6.7	0.56	7.3
1.50	86	2.4	0.141	1.13	0.13	8.3	1.17	18.9
1.75	100	3.2	0.215	1.31	0.16	9.9	2.14	41.1
2.00	115	4.1	0.307	1.50	0.20	11.5	3.52	78.5
2.25	129	5.1	0.414	1.69	0.25	13.0	5.37	135.1
2.50	143	6.2	0.535	1.88	0.29	14.3	7.67	213.4
2.75	158	7.2	0.666	2.06	0.32	15.6	10.38	313.7
3.00	172	8.4	0.804	2.25	0.36	16.7	13.40	433.4
3.25	186	9.5	0.944	2.44	0.39	17.6	16.62	567.1
3.50	201	10.6	1.083	2.63	0.41	18.3	19.87	707.1
3.75	215	11.7	1.215	2.81	0.43	18.9	23.00	844.2
4.00	229	12.7	1.338	3.00	0.45	19.3	25.85	969.0
4.25	244	13.7	1.447	3.19	0.45	19.6	28.29	1073.2
4.50	258	14.7	1.541	3.38	0.46	19.6	30.23	1151.0
4.75	272	15.5	1.617	3.56	0.45	19.6	31.61	1199.0
5.00	286	16.2	1.676	3.75	0.45	19.3	32.43	1217.3
5.25	301	16.8	1.718	3.94	0.44	19.0	32.72	1208.8
5.50	315	17.3	1.745	4.13	0.42	18.7	32.56	1178.4
5.75	329	17.7	1.760	4.31	0.41	18.2	32.06	1132.7
6.00	344	17.9	1.766	4.50	0.39	17.7	31.34	1078.7
6.25	358	18.0	1.767	4.69	0.38	17.3	30.53	1022.9
3.59	206	11.0	1.132	2.69	0.42	18.6	21.04	758.3
4.39	252	14.3	1.502	3.29	0.46	19.6	29.47	1121.4

Down Drain Feature: 2C1
Flow Through Circular Pipe

Number of Pipes, N=	5	
Diameter of pipe, D=	18	inches
Longitudinal Slope, So=	0.040	ft/ft
Manning's n=	0.009	
Density of flowing liquid, rho=	1.94	slugs/ft^3
Peak Discharge, Q_{25} =	85.7	cfs
Peak Discharge, Q_{100} =	120.1	cfs

Theta radians	Theta degrees	Depth of Flow y inches	Area of Flow A ft^2	Wetted Perimeter P ft	Hydraulic Radius R ft	Average Velocity V ft/s	Discharge Q=A*V cfs	Force F lbf
0.00	0	0.0	0.000	0.00		0.0	0.00	0.0
0.25	14	0.1	0.001	0.19	0.00	0.8	0.00	0.0
0.50	29	0.3	0.006	0.38	0.02	2.0	0.06	0.0
0.75	43	0.6	0.019	0.56	0.03	3.5	0.33	0.5
1.00	57	1.1	0.045	0.75	0.06	5.0	1.12	2.2
1.25	72	1.7	0.085	0.94	0.09	6.7	2.82	7.3
1.50	86	2.4	0.141	1.13	0.13	8.3	5.86	18.9
1.75	100	3.2	0.215	1.31	0.16	9.9	10.69	41.1
2.00	115	4.1	0.307	1.50	0.20	11.5	17.62	78.5
2.25	129	5.1	0.414	1.69	0.25	13.0	26.85	135.1
2.50	143	6.2	0.535	1.88	0.29	14.3	38.35	213.4
2.75	158	7.2	0.666	2.06	0.32	15.6	51.89	313.7
3.00	172	8.4	0.804	2.25	0.36	16.7	67.01	433.4
3.25	186	9.5	0.944	2.44	0.39	17.6	83.08	567.1
3.50	201	10.6	1.083	2.63	0.41	18.3	99.34	707.1
3.75	215	11.7	1.215	2.81	0.43	18.9	114.99	844.2
4.00	229	12.7	1.338	3.00	0.45	19.3	129.25	969.0
4.25	244	13.7	1.447	3.19	0.45	19.6	141.47	1073.2
4.50	258	14.7	1.541	3.38	0.46	19.6	151.16	1151.0
4.75	272	15.5	1.617	3.56	0.45	19.6	158.07	1199.0
5.00	286	16.2	1.676	3.75	0.45	19.3	162.15	1217.3
5.25	301	16.8	1.718	3.94	0.44	19.0	163.60	1208.8
5.50	315	17.3	1.745	4.13	0.42	18.7	162.80	1178.4
5.75	329	17.7	1.760	4.31	0.41	18.2	160.29	1132.7
6.00	344	17.9	1.766	4.50	0.39	17.7	156.68	1078.7
6.25	358	18.0	1.767	4.69	0.38	17.3	152.63	1022.9
3.29	189	9.7	0.967	2.47	0.39	17.7	85.78	590.3
3.84	220	12.1	1.261	2.88	0.44	19.1	120.34	891.4

Down Drain Feature: 2E1
Flow Through Circular Pipe

Number of Pipes, N=	2	
Diameter of pipe, D=	18	inches
Longitudinal Slope, So=	0.040	ft/ft
Manning's n=	0.009	
Density of flowing liquid, rho=	1.94	slugs/ft^3
Peak Discharge, Q_{25} =	42.4	cfs
Peak Discharge, Q_{100} =	59.3	cfs

Theta radians	Theta degrees	Depth of Flow y inches	Area of Flow A ft^2	Wetted Perimeter P ft	Hydraulic Radius R ft	Average Velocity V ft/s	Discharge Q=A*V cfs	Force F lbf
0.00	0	0.0	0.000	0.00		0.0	0.00	0.0
0.25	14	0.1	0.001	0.19	0.00	0.8	0.00	0.0
0.50	29	0.3	0.006	0.38	0.02	2.0	0.02	0.0
0.75	43	0.6	0.019	0.56	0.03	3.5	0.13	0.5
1.00	57	1.1	0.045	0.75	0.06	5.0	0.45	2.2
1.25	72	1.7	0.085	0.94	0.09	6.7	1.13	7.3
1.50	86	2.4	0.141	1.13	0.13	8.3	2.35	18.9
1.75	100	3.2	0.215	1.31	0.16	9.9	4.27	41.1
2.00	115	4.1	0.307	1.50	0.20	11.5	7.05	78.5
2.25	129	5.1	0.414	1.69	0.25	13.0	10.74	135.1
2.50	143	6.2	0.535	1.88	0.29	14.3	15.34	213.4
2.75	158	7.2	0.666	2.06	0.32	15.6	20.76	313.7
3.00	172	8.4	0.804	2.25	0.36	16.7	26.80	433.4
3.25	186	9.5	0.944	2.44	0.39	17.6	33.23	567.1
3.50	201	10.6	1.083	2.63	0.41	18.3	39.74	707.1
3.75	215	11.7	1.215	2.81	0.43	18.9	45.99	844.2
4.00	229	12.7	1.338	3.00	0.45	19.3	51.70	969.0
4.25	244	13.7	1.447	3.19	0.45	19.6	56.59	1073.2
4.50	258	14.7	1.541	3.38	0.46	19.6	60.46	1151.0
4.75	272	15.5	1.617	3.56	0.45	19.6	63.23	1199.0
5.00	286	16.2	1.676	3.75	0.45	19.3	64.86	1217.3
5.25	301	16.8	1.718	3.94	0.44	19.0	65.44	1208.8
5.50	315	17.3	1.745	4.13	0.42	18.7	65.12	1178.4
5.75	329	17.7	1.760	4.31	0.41	18.2	64.12	1132.7
6.00	344	17.9	1.766	4.50	0.39	17.7	62.67	1078.7
6.25	358	18.0	1.767	4.69	0.38	17.3	61.05	1022.9
3.61	207	11.1	1.140	2.70	0.42	18.6	42.45	766.5
4.43	254	14.4	1.515	3.32	0.46	19.6	59.47	1132.1

Down Drain Feature: 2F1
Flow Through Circular Pipe

Number of Pipes, N=	4	
Diameter of pipe, D=	18	inches
Longitudinal Slope, So=	0.040	ft/ft
Manning's n=	0.009	
Density of flowing liquid, rho=	1.94	slugs/ft^3
Peak Discharge, Q_{25} =	79.3	cfs
Peak Discharge, Q_{100} =	110.9	cfs

Theta radians	Theta degrees	Depth of Flow y inches	Area of Flow A ft^2	Wetted Perimeter P ft	Hydraulic Radius R ft	Average Velocity V ft/s	Discharge Q=A*V cfs	Force F lbf
0.00	0	0.0	0.000	0.00		0.0	0.00	0.0
0.25	14	0.1	0.001	0.19	0.00	0.8	0.00	0.0
0.50	29	0.3	0.006	0.38	0.02	2.0	0.05	0.0
0.75	43	0.6	0.019	0.56	0.03	3.5	0.27	0.5
1.00	57	1.1	0.045	0.75	0.06	5.0	0.90	2.2
1.25	72	1.7	0.085	0.94	0.09	6.7	2.26	7.3
1.50	86	2.4	0.141	1.13	0.13	8.3	4.69	18.9
1.75	100	3.2	0.215	1.31	0.16	9.9	8.55	41.1
2.00	115	4.1	0.307	1.50	0.20	11.5	14.10	78.5
2.25	129	5.1	0.414	1.69	0.25	13.0	21.48	135.1
2.50	143	6.2	0.535	1.88	0.29	14.3	30.68	213.4
2.75	158	7.2	0.666	2.06	0.32	15.6	41.51	313.7
3.00	172	8.4	0.804	2.25	0.36	16.7	53.61	433.4
3.25	186	9.5	0.944	2.44	0.39	17.6	66.47	567.1
3.50	201	10.6	1.083	2.63	0.41	18.3	79.47	707.1
3.75	215	11.7	1.215	2.81	0.43	18.9	91.99	844.2
4.00	229	12.7	1.338	3.00	0.45	19.3	103.40	969.0
4.25	244	13.7	1.447	3.19	0.45	19.6	113.17	1073.2
4.50	258	14.7	1.541	3.38	0.46	19.6	120.93	1151.0
4.75	272	15.5	1.617	3.56	0.45	19.6	126.45	1199.0
5.00	286	16.2	1.676	3.75	0.45	19.3	129.72	1217.3
5.25	301	16.8	1.718	3.94	0.44	19.0	130.88	1208.8
5.50	315	17.3	1.745	4.13	0.42	18.7	130.24	1178.4
5.75	329	17.7	1.760	4.31	0.41	18.2	128.23	1132.7
6.00	344	17.9	1.766	4.50	0.39	17.7	125.35	1078.7
6.25	358	18.0	1.767	4.69	0.38	17.3	122.10	1022.9
3.50	200	10.6	1.081	2.62	0.41	18.3	79.33	705.7
4.19	240	13.5	1.423	3.14	0.45	19.5	111.14	1052.2

Down Drain Feature: 2F2
Flow Through Circular Pipe

Number of Pipes, N=	5	
Diameter of pipe, D=	18	inches
Longitudinal Slope, So=	0.040	ft/ft
Manning's n=	0.009	
Density of flowing liquid, rho=	1.94	slugs/ft^3
Peak Discharge, Q_{25} =	96.3	cfs
Peak Discharge, Q_{100} =	134.9	cfs

Theta radians	Theta degrees	Depth of Flow y inches	Area of Flow A ft^2	Wetted Perimeter P ft	Hydraulic Radius R ft	Average Velocity V ft/s	Discharge Q=A*V cfs	Force F lbf
0.00	0	0.0	0.000	0.00		0.0	0.00	0.0
0.25	14	0.1	0.001	0.19	0.00	0.8	0.00	0.0
0.50	29	0.3	0.006	0.38	0.02	2.0	0.06	0.0
0.75	43	0.6	0.019	0.56	0.03	3.5	0.33	0.5
1.00	57	1.1	0.045	0.75	0.06	5.0	1.12	2.2
1.25	72	1.7	0.085	0.94	0.09	6.7	2.82	7.3
1.50	86	2.4	0.141	1.13	0.13	8.3	5.86	18.9
1.75	100	3.2	0.215	1.31	0.16	9.9	10.69	41.1
2.00	115	4.1	0.307	1.50	0.20	11.5	17.62	78.5
2.25	129	5.1	0.414	1.69	0.25	13.0	26.85	135.1
2.50	143	6.2	0.535	1.88	0.29	14.3	38.35	213.4
2.75	158	7.2	0.666	2.06	0.32	15.6	51.89	313.7
3.00	172	8.4	0.804	2.25	0.36	16.7	67.01	433.4
3.25	186	9.5	0.944	2.44	0.39	17.6	83.08	567.1
3.50	201	10.6	1.083	2.63	0.41	18.3	99.34	707.1
3.75	215	11.7	1.215	2.81	0.43	18.9	114.99	844.2
4.00	229	12.7	1.338	3.00	0.45	19.3	129.25	969.0
4.25	244	13.7	1.447	3.19	0.45	19.6	141.47	1073.2
4.50	258	14.7	1.541	3.38	0.46	19.6	151.16	1151.0
4.75	272	15.5	1.617	3.56	0.45	19.6	158.07	1199.0
5.00	286	16.2	1.676	3.75	0.45	19.3	162.15	1217.3
5.25	301	16.8	1.718	3.94	0.44	19.0	163.60	1208.8
5.50	315	17.3	1.745	4.13	0.42	18.7	162.80	1178.4
5.75	329	17.7	1.760	4.31	0.41	18.2	160.29	1132.7
6.00	344	17.9	1.766	4.50	0.39	17.7	156.68	1078.7
6.25	358	18.0	1.767	4.69	0.38	17.3	152.63	1022.9
3.45	198	10.4	1.058	2.59	0.41	18.2	96.36	681.4
4.12	236	13.2	1.390	3.09	0.45	19.5	135.26	1021.0

APPENDIX F-3

PERIMETER CHANNELS AND CULVERTS DESIGN

Written by: **A. Sivashanthan** Date: **7/1/2016** Reviewed by: **B. Klenzendorf** Date: **7/5/2016**

Client: **Santee Cooper** Project: **Winyah Generating Station** Project No.: **GSC5242** Phase No.: **01**

APPENDIX F-3

PERIMETER CHANNELS AND CULVERTS DESIGN

SOUTH CAROLINA LICENSED PROFESSIONAL ENGINEER No. 33535 SCOTT M. GRAVES 8/9/2016

SOUTH CAROLINA GEOSYNTEC CONSULTANTS, INC. No. 773 CERTIFICATE OF AUTHORIZATION

SEALED FOR
CALCULATION PAGES F3-1
THROUGH F3-35

1. INTRODUCTION

1.1 Purpose

The purpose of this calculation package is to present the design of the perimeter drainage channels and roadway culverts for the final facility surface water management system of the proposed Class Three Landfill (composed of Landfill Area 1 and Landfill Area 2) at the Winyah Generating Station (WGS) located in Georgetown County, South Carolina. Specifically, this calculation package presents the design criteria for the drainage channels and culverts, and presents hydraulic computations and the resulting design of these features. Additionally, the riprap aprons located at the outlet of culverts and downdrain pipes are also designed within this calculation package.

1.2 Overview of Perimeter Channels and Culverts

The Engineering Drawing set that accompanies the Engineering Report includes series of drawings that shows the surface water management system features, including the identification, layout, and engineering details of the perimeter drainage channels and culverts. An overview of these features is provided below.

- Overall, the perimeter drainage channels will receive surface water runoff from up-gradient landfill areas, and will convey the surface water runoff through a series of reaches and culverts which ultimately outlet to the on-site cooling pond (either directly into the cooling pond, or into the associated intake and discharge canals connected to the cooling pond). A reach is defined as a segment of a drainage channel with a specific contributing flow rate.

- Landfill Area 1:
  - Will be surrounded by perimeter drainage channels on all sides.
  - Will have a perimeter drainage channel high point is located at about the middle of the northern perimeter. Runoff west of the high point drains to the intake canal via what is termed "Outfall 1A", whereas runoff east of the high point drains to the intake canal via what is termed "Outfall 1C". Additionally, a perimeter drainage channel located along the southern border of Landfill Area 1 has a high point at the eastern end of the channel (i.e., near the southeastern corner of the landfill), and the channel flows from east to west, to Outfall 1A. The locations of these reaches and outlets are shown on the Engineering Drawings.
  - Will have two proposed culverts ("Culverts 1A" and "1C") to convey runoff underneath the perimeter roadway into the intake canal near the southwestern and southeastern corners of the landfill, respectively. In addition, a third proposed culvert ("Culvert 1B") will convey runoff underneath the landfill access road to perimeter drainage channel.
- Landfill Area 2:
  - Will have perimeter drainage channels located along the northern and southern sides. On the western and eastern sides of Landfill Area 2, perimeter drainage channels will not be used, and instead the landfill downdrain pipes at those locations will directly discharge into the discharge canal or cooling pond.
  - Will have perimeter channel high points located in the middle of both the northern and southern perimeter drainage channels. Runoff west of the high points will drain to the discharge canal (via "Outfall 2B" from the northern channel and "Outfall 2D" from the southern channel), whereas runoff east of the high points will drain to the cooling pond (via "Outfall 2A" from the northern channel and "Outfall 2C" from the southern channel). The locations of these reaches and outlets are shown on the Engineering

Drawings.

- Will include culverts to convey flow beneath roads where the landfill perimeter access roads cross the perimeter drainage channels. Specifically, three proposed culverts ("Culverts 2BC", "2C", and "2D") convey runoff underneath the perimeter roadway into either the cooling pond or the discharge canal near the northwestern, southeastern, and southwestern corners of the landfill, respectively. Two additional proposed culverts convey runoff underneath the bridge crossing and the access road into the perimeter drainage channel. "Culvert 2BA" is located at the proposed bridge crossing within perimeter channel reach 2BA and is upstream from the proposed culvert ("Culvert 2BB") beneath the access road.

2. DESIGN CRITERIA

The perimeter drainage channels and culverts are designed to meet or exceed the applicable requirements of the State of South Carolina Department of Health and Environmental Control (DHEC) regulations for Class Three Landfills, by being capable of controlling at least the water volume resulting from a 25-year, 24-hour storm. The following design criteria have been adopted for these features:

- the perimeter drainage channels are sized to convey the 100-year, 24-hour design rainfall event without overtopping;
- the perimeter drainage channels have channel lining materials selected to resist the velocities and/or tractive stresses produced by the 25-year, 24-hour rainfall event;
- the drainage culverts are designed to adequately convey the 100-year, 24-hour design rainfall event.

Additionally, the riprap aprons located at channel and culvert outlets are sized to have apron dimensions and stone particle sizes as recommended by the Federal Highway Administration (FHWA) (2006) to help protect against erosion and scour from the predicted 25-year, 24-hour peak outflows.

Written by: A. Sivashanthan Date: 7/1/2016 Reviewed by: B. Klenzendorf Date: 7/5/2016

Client: Santee Cooper Project: Winyah Generating Station Project No.: GSC5242 Phase No.: 01

3. METHODOLOGY

3.1 Perimeter Drainage Channels

Final cover areas contributing to each perimeter drainage channel reach are modeled in the Hydrologic Modeling System (HEC-HMS) computer program developed through the Hydraulic Engineering Center (HEC) of the United States Army Corps of Engineers (USACE, 2000) for the post-development site conditions, and peak flow rates are subsequently computed for each reach. The details including the methodology, design parameters, and hydrologic calculations (i.e., peak surface water runoff rate) of this analysis are provided in the *Drainage Analysis – Hydrology* calculations located in Appendix F-1. Each reach is designed to convey the peak surface water runoff rate corresponding to the 100-year (i.e., one percent annual chance of occurrence), 24-hour duration rainfall event flowing to the channel reach.

The hydraulic capacity of each perimeter drainage channel reach is calculated and assessed by solving Manning's equation. Manning's equation (Chow, 1959) is expressed as:

$$Q = \frac{1.49}{n} A R^{2/3} S^{1/2}$$

where:

Q = discharge (cfs),

n = Manning's roughness coefficient,

A = area of cross-section of flow (ft^2),

P = wetted perimeter (ft),

R = hydraulic radius = A/P (ft), and

S = slope of hydraulic grade line (or longitudinal channel slope for normal flow conditions) (ft/ft).

The peak velocity calculated from Manning's equation based on the channel geometry and peak flow rate is used to select the appropriate channel lining.

3.2 Culverts

All the culverts are designed by utilizing the HY-8 Culvert Analysis Program v.7.4 (HY-8). HY-8 was originally developed by the Federal Highway Administration (FHWA, 2012) and has since been updated and revised to its current version (Version 7.4). The performance of the culverts is modeled and evaluated based on boundary conditions, culvert characteristics, and design flow rates. The performance of the culverts is assessed for the 100-yr, 24-hr rainfall event assuming the tailwater conditions correspond to the calculated flow depth in the downstream channel for the rainfall event. Flow rates to the proposed culverts were obtained from the HEC-HMS model developed as part of Appendix F-1.

3.3 Riprap Apron Design

The riprap aprons are designed to protect against erosion and scour from the peak outflow based on the 25-year, 24-hour rainfall event. The selected design guidance from the Federal Highway Administration (FHWA) provides a methodology for calculating the median riprap size (d_{50}) and required length of apron (L_a) based on the culvert diameter and design flow rate. The d_{50} is the stone size of the riprap for which 50% of the riprap stones are smaller than d_{50} by mass. The median riprap size is calculated using the following equation (FHWA, 2006):

$$d_{50} = 0.2D\left[\frac{Q}{\sqrt{g}D^{2.5}}\right]^{4/3}\frac{D}{TW}$$

where:

d_{50} = median riprap size (ft),

Q = design discharge (cfs),

D = culvert diameter (ft),

TW = tailwater depth (ft), and

g = gravitational constant (32.2 ft/s^2).

The tailwater depth should be limited to between 0.4D and D. FHWA (2006) recommends the use of a tailwater depth equal to 0.4D if the tailwater conditions are unknown.

Once the median riprap size is calculated, the required length and depth (i.e., thickness) of the riprap apron can be estimated using Table 1 provided by the FHWA (2006) based on the median riprap size and culvert rise. The width of the riprap apron is selected as three times the culvert diameter based on the riprap apron detail presented in Figure 1 from

FHWA (2006) or the bottom width of the channel for culverts that discharge directly to channels. For culverts with multiple barrels, the apron width should be limited to one pipe width wider than the extent of the pipe barrels. Based on the detail, the apron width will also widen downstream of the outlet by one foot per three feet in length at each edge. Figure 1 of this calculation package provides the typical geometry for the riprap apron.

4. DESIGN PARAMETERS

4.1 Perimeter Drainage Channels

All the perimeter drainage channels are designed to be grass-lined trapezoidal shaped channels with the exception of perimeter channel reach 1C1 located near the southeastern portion of Area 1 Landfill. The perimeter channel reach 1C1 is designed to be a concrete-lined rectangular channel due to space restrictions associated with an adjacent easement. The design parameters for each perimeter drainage channel reach, including channel geometry and calculated peak flow rates (as computed by the HEC-HMS model described in Appendix F-1) for the 25-year and 100-year, 24-hour rainfall events, are summarized in Table 2. A Manning's roughness coefficient of 0.027 was selected for the grass-lined perimeter drainage channels, based on recommendations for excavated earth channels that are straight and uniform with short grass and few weeds (Chow, 1959) as shown in Table 3 of this calculation package. A Manning's roughness coefficient of 0.011 was selected for the concrete-lined trowel finish perimeter drainage channel reach 1C1 as shown in Table 3 (Chow, 1959). A maximum permissible velocity of 6.0 ft/s was selected, based on recommendations from Georgetown County (2006) for Bermuda grass-lined drainage channels, as shown in Table 4 of this calculation package. The allowable velocity of 18.0 ft/s was chosen for the concrete-lined channel as shown in Table 5 (USDA, 2007). The perimeter drainage channels are designed to have a calculated average velocity less than the permissible velocity for a 25-year, 24-hour rainfall event.

4.2 Culverts

The culverts around the landfill perimeter areas will be reinforced concrete pipe (RCP). The design parameters for each culvert, including inlet invert elevation, outlet invert elevation, slope, approximate length, and design flow rates for each culvert (as computed by the HEC-HMS model described in Appendix F-1) for the 25-year and 100-year, 24-hour rainfall events, are summarized in Table 6. A Manning's roughness coefficient of 0.012 was selected based on guidance from Chow (1959) as shown in Table 7. All culverts were designed using the following parameters to convey the peak flow rate for the 100-year, 24-hour rainfall event.

The inflow structure into the culvert influences the conveyance of surface water through the culvert. The culvert inflow structures were modeled as a square edge with a headwall.

4.3 Riprap Apron Design

Riprap aprons were sized for the outflow of each proposed culvert, as well as for the downdrain pipe locations which discharge directly to the canals or cooling pond. The riprap apron design parameters including flow rates, culvert diameter, and riprap apron dimensions depth are presented in Table 8. For the purposes of riprap apron design only, proposed culvert crossings with multiple barrels were modeled with a flow rate evenly distributed to each culvert based on the number of barrels.

5. RESULTS

The depth of flow and velocity for the calculated discharge for each perimeter drainage channel reach during the design rainfall events were calculated using the methodology described above. Calculations for each perimeter drainage channel reach were performed using spreadsheet-based computations, with results summarized in Table 9. Spreadsheet results for each perimeter drainage channel are presented in Appendix 1 of this calculation package. The performance of each culvert from HY-8 modeling during the design rainfall events is presented in Table 10. The riprap apron median size, apron depth, and apron length for each culvert and down drain pipe feature are presented in Table 8. The calculation results presented herein demonstrate that:

- Each perimeter drainage channel reach is designed to be able to convey the 100-year, 24-hour rainfall event without overtopping.
- The selected channel lining can adequately resist the velocities or tractive stresses produced by the 25-year, 24-hour rainfall event.
- The culvert designs contain the capacity to convey the design flows without overtopping the downstream roadway for a 100-year, 24-hour rainfall event.

- The minimum d_{50} size of the riprap apron was computed for the outflow of each necessary discharge structure. In addition, the selected riprap class, apron depths and lengths are provided within Table 8 for each riprap apron.

Note that FHWA (2006) recommends an apron width of three times the outlet diameter or one pipe width wider than the extent of the pipe barrels at the up gradient end of the apron near the culvert outlet and a 3:1 rate of expansion at each edge along the length of the apron. However, several structures are discharging into a stabilized trapezoidal channel and the dimensions of the riprap aprons are restricted by the channel dimensions. Therefore, the entire width of the channel should be lined with riprap in these cases.

6. REFERENCES

Chow, V.T (1959). *Open Channel-Hydraulics,* McGraw-Hill.

FHWA (2006). *Hydraulic Design of Energy Dissipators for Culverts and Channels*, Federal Highway Administration, US Department of Transportation, Hydraulic Engineering Circular No. 14, Third Edition.

FHWA (2012). *HY-8 Version 7.4*, Federal Highway Administration, 13 August 2014.

Georgetown County (2000). *Georgetown County Storm Water Design Manual*, adopted 5 September 2000.

USACE (2000). *Hydraulic Modeling System HEC-HMS Technical Reference Manual*, US Army Corps of Engineers, Hydrologic Engineering Center, CPD-74B, March 2000.

USDA (2007). *Stream Restoration Design,* Chapter 8 - Threshold Channel Design, United States Department of Agriculture, Natural Resources Conservation Service, National Engineering Handbook Part 654, issued August 2007.

TABLES

- Table 1. Riprap Classes and Apron Dimensions (from FHWA, 2006)
- Table 2. Design Parameter Summary for Perimeter Drainage Channels
- Table 3. Manning's n Values for Open Channels (from Chow, 1959)
- Table 4. Maximum Velocities for Vegetative Channel Linings (from Georgetown County, 2006)
- Table 5. Allowable Velocity and Shear Stress for Selected Lining Materials (from USDA, 2007)
- Table 6. Design Parameter Summary for Culverts
- Table 7. Manning's n Values for Closed Conduits (from Chow, 1959)
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- Table 9. Perimeter Drainage Channel Capacity Calculation Results
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Table 1. Riprap Classes and Apron Dimensions

(from FHWA, 2006)

Class	D_{50} (mm)	D_{50} (in)	Apron Length[1]	Apron Depth
1	125	5	4D	$3.5D_{50}$
2	150	6	4D	$3.3D_{50}$
3	250	10	5D	$2.4D_{50}$
4	350	14	6D	$2.2D_{50}$
5	500	20	7D	$2.0D_{50}$
6	550	22	8D	$2.0D_{50}$

[1]D is the culvert rise.

Table 2. Design Parameter Summary for Perimeter Drainage Channels

Perimeter Channel	Channel Shape	Longitudinal Channel Slope (%)	Manning's n	Bottom Width (ft)	Depth (ft)	Side Slopes (H:V)	Channel Lining	25-year Flow Rate Q_{25} (cfs)	100-year Flow Rate Q_{100} (cfs)
Perimeter Reach 1A1	Trapezoid	0.50	0.027	3.0	3.0	3:1	Vegetation	28.1	39.5
Perimeter Reach 1BA	Trapezoid	0.50	0.027	3.0	3.0	3:1	Vegetation	115.9	164.1
Perimeter Reach 1B1	Trapezoid	0.50	0.027	3.0	3.0	3:1	Vegetation	102.7	144.7
Perimeter Reach 1B2	Trapezoid	0.50	0.027	3.0	3.0	3:1	Vegetation	73.4	103.5
Perimeter Reach 1B3	Trapezoid	0.50	0.027	3.0	3.0	3:1	Vegetation	29.6	41.6
Perimeter Reach 1C1	Trapezoid	0.50	0.011	3.0	3.2	0	Concrete	59.6	83.3
Perimeter Reach 1C2	Trapezoid	0.50	0.027	2.0	1.7	3:1	Vegetation	20.1	28.1
Perimeter Reach 2A1	Trapezoid	0.50	0.027	3.0	3.0	3:1	Vegetation	89.5	125.3
Perimeter Reach 2A2	Trapezoid	0.50	0.027	3.0	3.0	3:1	Vegetation	23.7	33.1
Perimeter Reach 2BA	Trapezoid	0.50	0.027	3.0	3.0	3:1	Vegetation	14.9	20.8
Perimeter Reach 2BB	Trapezoid	0.50	0.027	3.0	3.0	3:1	Vegetation	35.2	49.3
Perimeter Reach 2C1	Trapezoid	0.50	0.027	3.0	3.0	3:1	Vegetation	102.8	144.2
Perimeter Reach 2DA	Trapezoid	0.50	0.027	3.0	3.0	3:1	Vegetation	23.6	33.1

Table 3. Manning's n Values for Open Channels

(from Chow, 1959)

Type of channel and description	Minimum	Normal	Maximum
B. LINED OR BUILT-UP CHANNELS			
B-1. Metal			
a. Smooth steel surface			
1. Unpainted	0.011	0.012	0.014
2. Painted	0.012	0.013	0.017
b. Corrugated	0.021	0.025	0.030
B-2. Nonmetal			
a. Cement			
1. Neat, surface	0.010	0.011	0.013
2. Mortar	0.011	0.013	0.015
b. Wood			
1. Planed, untreated	0.010	0.012	0.014
2. Planed, creosoted	0.011	0.012	0.015
3. Unplaned	0.011	0.013	0.015
4. Plank with battens	0.012	0.015	0.018
5. Lined with roofing paper	0.010	0.014	0.017
c. Concrete			
1. Trowel finish	0.011	0.013	0.015
2. Float finish	0.013	0.015	0.016
3. Finished, with gravel on bottom	0.015	0.017	0.020
C. EXCAVATED OR DREDGED			
a. Earth, straight and uniform			
1. Clean, recently completed	0.016	0.018	0.020
2. Clean, after weathering	0.018	0.022	0.025
3. Gravel, uniform section, clean	0.022	0.025	0.030
4. With short grass, few weeds	0.022	0.027	0.033
b. Earth, winding and sluggish			
1. No vegetation	0.023	0.025	0.030
2. Grass, some weeds	0.025	0.030	0.033
3. Dense weeds or aquatic plants in deep channels	0.030	0.035	0.040
4. Earth bottom and rubble sides	0.028	0.030	0.035
5. Stony bottom and weedy banks	0.025	0.035	0.040
6. Cobble bottom and clean sides	0.030	0.040	0.050
c. Dragline-excavated or dredged			
1. No vegetation	0.025	0.028	0.033
2. Light brush on banks	0.035	0.050	0.060

Table 4. Maximum Velocities for Vegetative Channel Linings (from Georgetown County, 2006)

Vegetation Type	Slope Range (%)[1]	Maximum Velocity[2] (ft/s)
Bermuda Grass	0 - 5	6
	5 - 10	5
Bahia	All	4
Tall Fescue Grass	0 - 10	4
Mixtures[3]		
Kentucky Bluegrass	0 - 5	5
Buffalo Grass	5 - 10	4
	>10	3
Grass Mixture	0 - 5[1]	4
	5 - 10	3
Sericea Lespedeza,	0 - 5[4]	2.5
Weeping Lovegrass, Alfalfa	All	
Annuals[5]	0 - 5	2.5
Sod	All	4
Lapped Sod	All	5.5

[1] Do not use on slopes steeper than 10 percent except for side-slope in combination channels.
[2] Use velocities exceeding 5 feet per second only where good stands can be established and maintained.
[3] Mixtures of Tall Fescue, Bahia, and or Bermuda.
[4] Do not use on slopes steeper than 5 percent except for side-slope in combination channels.
[5] Annuals - used on mild slopes or as temporary protection until permanent covers are established.

Table 5. Allowable Velocity and Shear Stress for Selected Lining Materials (from USDA, 2007)

Boundary category	Boundary type	Allowable velocity (ft/s)	Allowable shear stress (lb/ft²)
Temporary degradable reinforced erosion control products (RECP)	Jute net	1–2.5	0.45
	Straw with net	1–3	1.5–1.65
	Coconut fiber with net	3–4	2.25
	Fiberglass roving	2.5–7	2
Nondegradable RECP	Unvegetated	5–7	3
	Partially established	7.5–15	4–6
	Fully vegetated	8–21	8
Hard surface	Gabions	1–19	10
	Concrete	>18	12.5

Table 6. Design Parameter Summary for Culverts

Culvert Designation	25-year Flow Rate Q_{25} (cfs)	100-year Flow Rate Q_{100} (cfs)	Approx. Length (ft)	Slope (%)	Description	Inlet Elevation (ft)	Outlet Elevation (ft)	Roadway Elevation (ft)
Culvert 1A	115.8	163.9	96	0.50	1 - 4-ft Diameter RCP	24.81	24.33	35.01
Culvert 1B	16.3	22.9	104	5.77	1 - 2-ft Diameter RCP	33.00	27.00	37.12
Culvert 1C	59.5	83.3	36	0.50	2 - 2.5-ft Diameter RCP	25.09	24.91	30.52
Culvert 2BA	14.9	20.8	112	0.50	1 - 2-ft Diameter RCP	32.39	31.83	38.84
Culvert 2BB	11.5	16.1	107	5.61	1 - 2-ft Diameter RCP	39.00	33.00	42.24
Culvert 2BC	35.1	49.2	109	2.30	2 - 2-ft Diameter RCP	23.50	21.00	27.48
Culvert 2C	102.7	144.1	108	0.50	1 - 3.5-ft Diameter RCP	26.95	26.41	40.72
Culvert 2D	23.6	33.0	108	0.50	1 - 2-ft Diameter RCP	26.99	26.45	40.92

Table 7. Manning's n Values for Closed Conduits (from Chow, 1959)

Type of channel and description	Minimum	Normal	Maximum
A-2. Nonmetal			
a. Lucite	0.008	0.009	0.010
b. Glass	0.009	0.010	0.013
c. Cement			
1. Neat, surface	0.010	0.011	0.013
2. Mortar	0.011	0.013	0.015
d. Concrete			
1. Culvert, straight and free of debris	0.010	0.011	0.013
2. Culvert with bends, connections, and some debris	0.011	0.013	0.014
3. Finished	0.011	0.012	0.014
4. Sewer with manholes, inlet, etc., straight	0.013	0.015	0.017
5. Unfinished, steel form	0.012	0.013	0.014
6. Unfinished, smooth wood form	0.012	0.014	0.016
7. Unfinished, rough wood form	0.015	0.017	0.020
e. Wood			
1. Stave	0.010	0.012	0.014
2. Laminated, treated	0.015	0.017	0.020
f. Clay			
1. Common drainage tile	0.011	0.013	0.017
2. Vitrified sewer	0.011	0.014	0.017
3. Vitrified sewer with manholes, inlet, etc.	0.013	0.015	0.017
4. Vitrified subdrain with open joint	0.014	0.016	0.018
g. Brickwork			
1. Glazed	0.011	0.013	0.015
2. Lined with cement mortar	0.012	0.015	0.017
h. Sanitary sewers coated with sewage slimes, with bends and connections	0.012	0.013	0.016
i. Paved invert, sewer, smooth bottom	0.016	0.019	0.020
j. Rubble masonry, cemented	0.018	0.025	0.030

Table 8. Design Parameter and Results Summary for Riprap Aprons

Riprap Apron Feature	25-year Flow Rate Q_{25} (cfs)	Number of Barrels	Culvert Diameter (ft)	Median Riprap Size (in)	Riprap Class	Apron Depth (ft)	Apron Length (ft)
Culvert 1A	115.8	1	4.0	13.2	3	2.6	20.0
Culvert 1B	16.3	1	2.0	4.9	1	1.4	8.0
Culvert 1C	59.5	2	2.5	6.4	2	1.8	10.0
Culvert 2BA	14.9	1	2.0	4.3	1	1.3	8.0
Culvert 2BB	11.5	1	2.0	3.1	1	0.9	8.0
Culvert 2BC	35.1	2	2.0	5.4	2	1.5	8.0
Culvert 2C	102.7	1	3.5	15.3	4	2.8	21.0
Culvert 2D	23.6	1	2.0	8.0	3	1.6	10.0
2E1	42.4	2	1.5	13.5	3	2.7	7.5
2F1	79.3	4	1.5	12.3	4	2.3	9.0
2F2	96.3	5	1.5	11.9	4	2.2	9.0

Note: See Table 1 for the riprap size associated with each Riprap Class.

Table 9. Perimeter Drainage Channel Capacity Calculation Results

Drainage Channel Reach	25-year, 24-hour Design Event				100-year, 24-hour Design Event			
	Peak Flow (cfs)	Peak Depth (ft)	Peak Velocity (ft/s)	Tractive Stress (psf)	Peak Flow (cfs)	Peak Depth (ft)	Peak Velocity (ft/s)	Tractive Stress (psf)
Perimeter Reach 1A1	28.1	1.26	3.30	0.24	39.5	1.47	3.60	0.28
Perimeter Reach 1BA	115.9	2.39	4.75	0.42	164.1	2.78	5.19	0.48
Perimeter Reach 1B1	102.7	2.27	4.61	0.40	144.7	2.64	5.03	0.46
Perimeter Reach 1B2	73.4	1.96	4.23	0.35	103.5	2.28	4.62	0.40
Perimeter Reach 1B3	29.6	1.29	3.34	0.25	41.6	1.51	3.65	0.28
Perimeter Reach 1C1	59.6	2.39	8.31	0.29	83.3	3.13	8.86	0.32
Perimeter Reach 1C2	20.1	1.18	3.06	0.22	28.1	1.38	3.33	0.25
Perimeter Reach 2A1	89.5	2.14	4.45	0.38	125.3	2.48	4.85	0.43
Perimeter Reach 2A2	23.7	1.16	3.15	0.23	33.1	1.36	3.44	0.26
Perimeter Reach 2BA	14.9	0.92	2.78	0.19	20.8	1.09	3.04	0.22
Perimeter Reach 2BB	35.2	1.40	3.50	0.26	49.3	1.63	3.82	0.30
Perimeter Reach 2C1	102.8	2.27	4.61	0.40	144.2	2.63	5.02	0.46
Perimeter Reach 2DA	23.6	1.15	3.15	0.23	33.1	1.36	3.44	0.26

Table 10. Culvert Capacity Analysis Results

	25-year, 24-hour storm event				100-year, 24-hour storm event		
Culvert	**Peak Flow Rate (cfs)**	**Pipe Velocity (fps)**	**Headwater Elev (ft)**	**25-year Freeboard (ft)**	**Peak Flow Rate (cfs)**	**Pipe Velocity (fps)**	**Headwater Elev (ft)**
Culvert 1A	115.8	10.61	30.84	4.17	163.9	13.53	34.21
Culvert 1B	16.3	13.37	35.36	1.76	22.9	14.58	36.38
Culvert 1C	59.5	7.28	28.23	2.29	83.3	9.23	29.60
Culvert 2BA	14.9	6.19	34.63	4.21	20.8	7.57	35.46
Culvert 2BB	11.5	14.42	40.81	1.43	16.1	15.92	41.34
Culvert 2BC	35.1	9.89	26.06	1.42	49.2	10.81	27.23
Culvert 2C	102.6	11.39	33.77	6.95	144.1	15.67	38.25
Culvert 2D	23.6	8.20	30.62	10.30	33.0	10.71	32.93

FIGURES

- Figure 1. Typical Geometry of Riprap Aprons at Culverts (from FHWA, 2006)

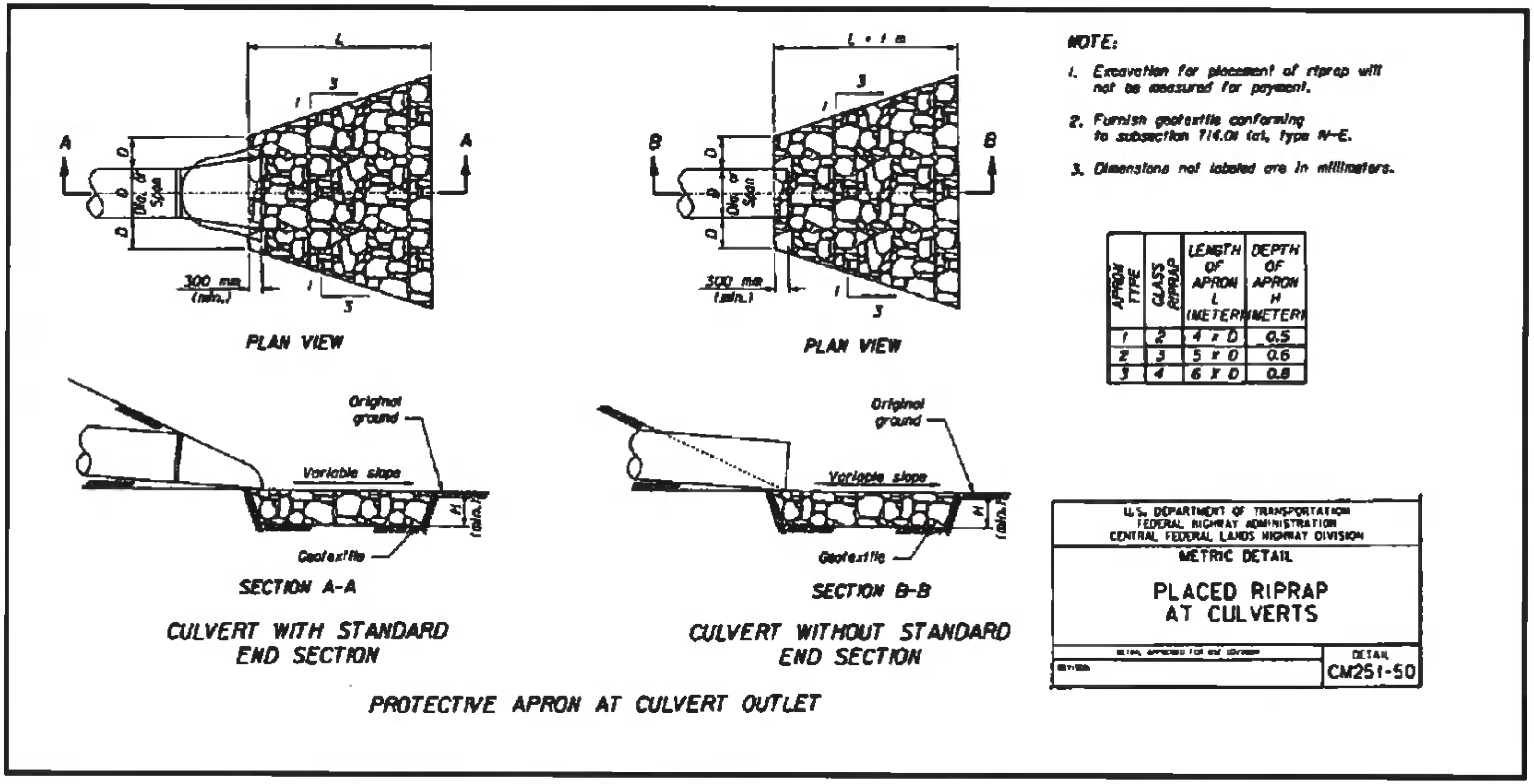


Figure 1. Typical Geometry of Riprap Aprons at Culverts

(from FHWA, 2006)

Appendix 1

Perimeter Channel Calculations

Design/Check: Trapezoidal/Triangular Channel

Methodology: Manning's Equation

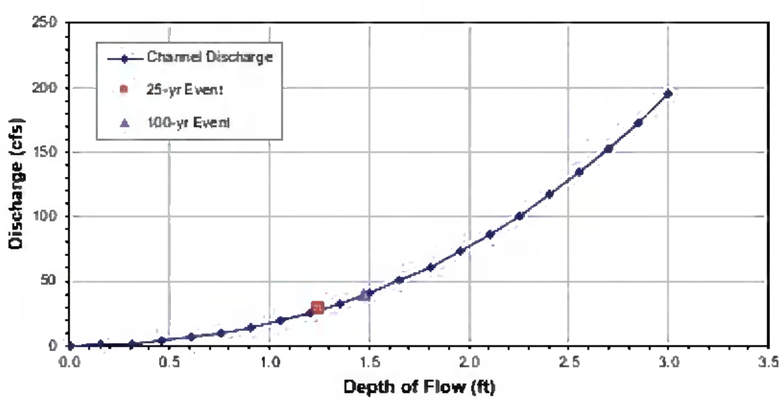
Project: Santee Cooper - Winyah Generating Station

Ditch ID: **Perimeter Reach 1A1** **Design**

Peak Discharge, Q_{25} =	28.10	cfs (25-yr Event)
Peak Discharge, Q_{100} =	39.50	cfs (100-yr Event)
Bottom Width, B =	3.00	ft
Left Side Slope, Z_1 =	3.00	horizontal : 1 vertical
Right Side Slope, Z_2 =	3.00	horizontal : 1 vertical
Channel Depth, Y =	3.00	ft
Top Width, T =	21.0	ft
Manning's Roughness Coeff., n =	0.027	
Longitudinal Channel Slope, S_o =	0.005	ft/ft

Depth of Flow Y ft	Area of Flow A ft^2	Wetted Perimeter P ft	Hydraulic Radius R=A/P ft	Average Velocity V ft/s	Discharge (Flow Rate) Q=AV ft^3/s	Avg. Tractive Stress τ_0 lb/ft^2	Comments
0.01	0.03	3.06	0.01	0.18	0.0	0.00	
0.16	0.55	4.01	0.14	1.04	0.6	0.04	
0.31	1.21	4.95	0.24	1.53	1.9	0.08	
0.46	2.01	5.90	0.34	1.90	3.8	0.11	
0.61	2.93	6.85	0.43	2.22	6.5	0.13	
0.76	3.99	7.79	0.51	2.50	10.0	0.16	
0.91	5.19	8.74	0.59	2.76	14.3	0.19	
1.06	6.52	9.68	0.67	3.00	19.5	0.21	
1.21	7.98	10.63	0.75	3.22	25.7	0.23	
1.36	9.58	11.57	0.83	3.44	32.9	0.26	
1.51	11.31	12.52	0.90	3.65	41.2	0.28	
1.65	13.16	13.46	0.98	3.85	50.7	0.31	
1.80	15.18	14.41	1.05	4.04	61.3	0.33	
1.95	17.31	15.36	1.13	4.23	73.2	0.35	
2.10	19.58	16.30	1.20	4.41	86.3	0.37	
2.25	21.98	17.25	1.27	4.59	100.8	0.40	
2.40	24.51	18.19	1.35	4.76	116.7	0.42	
2.55	27.18	19.14	1.42	4.93	134.1	0.44	
2.70	29.99	20.08	1.49	5.10	152.9	0.47	
2.85	32.93	21.03	1.57	5.26	173.3	0.49	
3.00	36.00	21.97	1.64	5.42	195.3	0.51	
1.26	8.49	10.94	0.78	3.30	27.98	0.24	Q (25-yr Event)
1.47	10.93	12.32	0.89	3.60	39.41	0.28	Q (100-yr Event)

Discharge versus Depth Relationship



Design/Check: Trapezoidal/Triangular Channel

Methodology: Manning's Equation

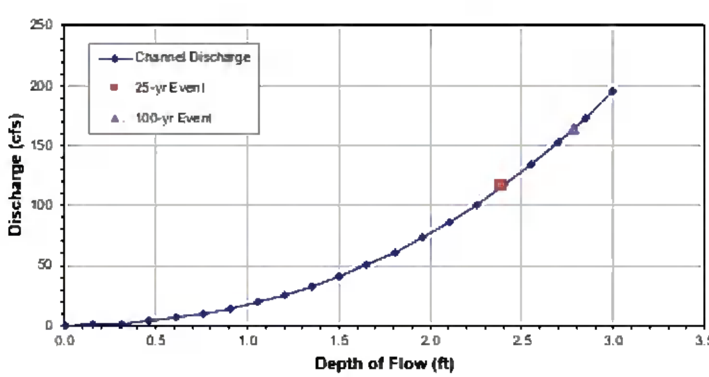
Project: Santee Cooper - Winyah Generating Station

Ditch ID: **Perimeter Reach 1BA** **Design**

Peak Discharge, Q_{25} =	115.90	cfs (25-yr Event)
Peak Discharge, Q_{100} =	164.10	cfs (100-yr Event)
Bottom Width, B =	3.00	ft
Left Side Slope, Z_1 =	3.00	horizontal : 1 vertical
Right Side Slope, Z_2 =	3.00	horizontal : 1 vertical
Channel Depth, Y =	3.00	ft
Top Width, T =	21.0	ft
Manning's Roughness Coeff., n =	0.027	
Longitudinal Channel Slope, S_o =	0.005	ft/ft

Depth of Flow Y ft	Area of Flow A ft^2	Wetted Perimeter P ft	Hydraulic Radius R=A/P ft	Average Velocity V ft/s	Discharge (Flow Rate) Q=AV ft^3/s	Avg. Tractive Stress τ_0 lb/ft^2	Comments
0.01	0.03	3.06	0.01	0.18	0.0	0.00	
0.16	0.55	4.01	0.14	1.04	0.6	0.04	
0.31	1.21	4.95	0.24	1.53	1.9	0.08	
0.46	2.01	5.90	0.34	1.90	3.8	0.11	
0.61	2.93	6.85	0.43	2.22	6.5	0.13	
0.76	3.99	7.79	0.51	2.50	10.0	0.16	
0.91	5.19	8.74	0.59	2.76	14.3	0.19	
1.06	6.52	9.68	0.67	3.00	19.5	0.21	
1.21	7.98	10.63	0.75	3.22	25.7	0.23	
1.36	9.58	11.57	0.83	3.44	32.9	0.26	
1.51	11.31	12.52	0.90	3.65	41.2	0.28	
1.65	13.18	13.46	0.98	3.85	50.7	0.31	
1.80	15.18	14.41	1.05	4.04	61.3	0.33	
1.95	17.31	15.36	1.13	4.23	73.2	0.35	
2.10	19.58	16.30	1.20	4.41	66.3	0.37	
2.25	21.96	17.25	1.27	4.59	100.8	0.40	
2.40	24.51	18.19	1.35	4.76	116.7	0.42	
2.55	27.18	19.14	1.42	4.93	134.1	0.44	
2.70	29.99	20.08	1.49	5.10	152.9	0.47	
2.85	32.93	21.03	1.57	5.26	173.3	0.49	
3.00	36.00	21.97	1.64	5.42	195.3	0.51	
2.39	24.38	18.14	1.34	4.75	115.86	0.42	Q (25-yr Event)
2.78	31.59	20.60	1.53	5.19	163.91	0.48	Q (100-yr Event)

Discharge versus Depth Relationship



Design/Check: Trapezoidal/Triangular Channel

Methodology: Manning's Equation

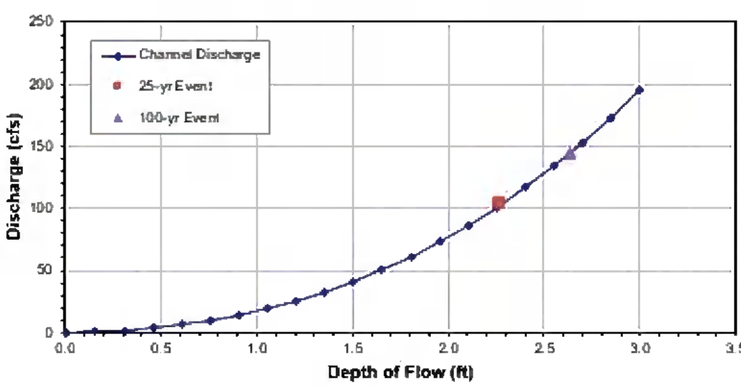
Project: Santee Cooper - Winyah Generating Station

Ditch ID: **Perimeter Reach 1B1** **Design**

Peak Discharge, Q_{25} =	102.70	cfs (25-yr Event)
Peak Discharge, Q_{100} =	144.70	cfs (100-yr Event)
Bottom Width, B =	3.00	ft
Left Side Slope, Z_1 =	3.00	horizontal:1 vertical
Right Side Slope, Z_2 =	3.00	horizontal:1 vertical
Channel Depth, Y =	3.00	ft
Top Width, T =	21.0	ft
Manning's Roughness Coeff., n =	0.027	
Longitudinal Channel Slope, S_o =	0.005	ft/ft

Depth of Flow Y ft	Area of Flow A ft^2	Wetted Perimeter P ft	Hydraulic Radius R=A/P ft	Average Velocity V ft/s	Discharge (Flow Rate) Q=AV ft^3/s	Avg. Tractive Stress τ_0 lb/ft^2	Comments
0.01	0.03	3.06	0.01	0.18	0.0	0.00	
0.16	0.55	4.01	0.14	1.04	0.6	0.04	
0.31	1.21	4.95	0.24	1.53	1.9	0.08	
0.46	2.01	5.90	0.34	1.90	3.6	0.11	
0.61	2.93	6.85	0.43	2.22	6.5	0.13	
0.76	3.99	7.79	0.51	2.50	10.0	0.16	
0.91	5.19	8.74	0.59	2.76	14.3	0.19	
1.06	6.52	9.68	0.67	3.00	19.5	0.21	
1.21	7.98	10.63	0.75	3.22	25.7	0.23	
1.36	9.58	11.57	0.83	3.44	32.9	0.26	
1.51	11.31	12.52	0.90	3.65	41.2	0.28	
1.65	13.18	13.46	0.98	3.85	50.7	0.31	
1.80	15.18	14.41	1.05	4.04	61.3	0.33	
1.95	17.31	15.36	1.13	4.23	73.2	0.35	
2.10	19.58	16.30	1.20	4.41	86.3	0.37	
2.25	21.98	17.25	1.27	4.59	100.8	0.40	
2.40	24.51	18.19	1.35	4.76	116.7	0.42	
2.55	27.18	19.14	1.42	4.93	134.1	0.44	
2.70	29.99	20.08	1.49	5.10	152.9	0.47	
2.85	32.93	21.03	1.57	5.26	173.3	0.49	
3.00	36.00	21.97	1.64	5.42	195.3	0.51	
2.27	22.27	17.36	1.28	4.61	102.63	0.40	Q (25-yr Event)
2.64	28.75	19.67	1.46	5.03	144.51	0.46	Q (100-yr Event)

Discharge versus Depth Relationship



Design/Check: Trapezoidal/Triangular Channel

Methodology: Manning's Equation

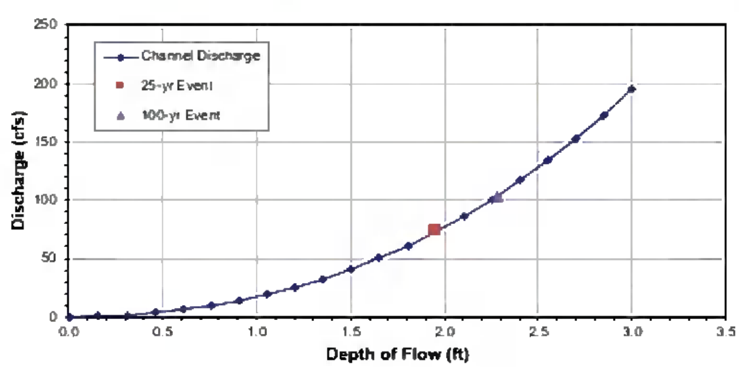
Project: Santee Cooper - Winyah Generating Station

Ditch ID: **Perimeter Reach 1B2** **Design**

Peak Discharge, Q_{25} =	73.40	cfs (25-yr Event)
Peak Discharge, Q_{100} =	103.50	cfs (100-yr Event)
Bottom Width, B =	3.00	ft
Left Side Slope, Z_1 =	3.00	horizontal : 1 vertical
Right Side Slope, Z_2 =	3.00	horizontal : 1 vertical
Channel Depth, Y =	3.00	ft
Top Width, T =	21.0	ft
Manning's Roughness Coeff., n =	0.027	
Longitudinal Channel Slope, S_o =	0.005	ft/ft

Depth of Flow Y ft	Area of Flow A ft^2	Wetted Perimeter P ft	Hydraulic Radius R=A/P ft	Average Velocity V ft/s	Discharge (Flow Rate) Q=AV ft^3/s	Avg. Tractive Stress τ_0 lb/ft^2	Comments
0.01	0.03	3.06	0.01	0.18	0.0	0.00	
0.16	0.55	4.01	0.14	1.04	0.6	0.04	
0.31	1.21	4.95	0.24	1.53	1.9	0.08	
0.46	2.01	5.90	0.34	1.90	3.8	0.11	
0.61	2.93	6.85	0.43	2.22	6.5	0.13	
0.76	3.99	7.79	0.51	2.50	10.0	0.16	
0.91	5.19	8.74	0.59	2.76	14.3	0.19	
1.06	6.52	9.66	0.87	3.00	19.5	0.21	
1.21	7.98	10.63	0.75	3.22	25.7	0.23	
1.36	9.58	11.57	0.83	3.44	32.9	0.26	
1.51	11.31	12.52	0.90	3.65	41.2	0.28	
1.65	13.18	13.46	0.98	3.85	50.7	0.31	
1.80	15.18	14.41	1.05	4.04	61.3	0.33	
1.95	17.31	15.36	1.13	4.23	73.2	0.35	
2.10	19.56	16.30	1.20	4.41	86.3	0.37	
2.25	21.98	17.25	1.27	4.59	100.8	0.40	
2.40	24.51	18.19	1.35	4.76	116.7	0.42	
2.55	27.18	19.14	1.42	4.93	134.1	0.44	
2.70	29.99	20.08	1.49	5.10	152.9	0.47	
2.85	32.93	21.03	1.57	5.26	173.3	0.49	
3.00	38.00	21.97	1.64	5.42	195.3	0.51	
1.96	17.35	15.37	1.13	4.23	73.39	0.35	Q (25-yr Event)
2.28	22.40	17.41	1.29	4.62	103.40	0.40	Q (100-yr Event)

Discharge versus Depth Relationship



Design/Check: Trapezoidal/Triangular Channel

Methodology: Manning's Equation

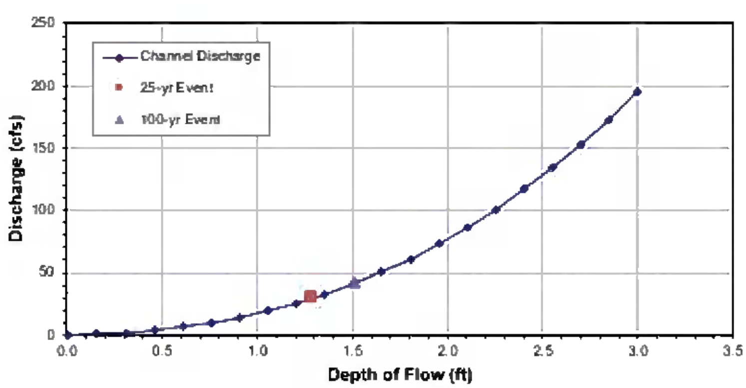
Project: Santee Cooper - Winyah Generating Station

Ditch ID: **Perimeter Reach 1B3** **Design**

Peak Discharge, Q_{25} =	29.60	cfs (25-yr Event)
Peak Discharge, Q_{100} =	41.60	cfs (100-yr Event)
Bottom Width, B =	3.00	ft
Left Side Slope, Z_1 =	3.00	horizontal : 1 vertical
Right Side Slope, Z_2 =	3.00	horizontal : 1 vertical
Channel Depth, Y =	3.00	ft
Top Width, T =	21.0	ft
Manning's Roughness Coeff., n =	0.027	
Longitudinal Channel Slope, S_o =	0.005	ft/ft

Depth of Flow Y ft	Area of Flow A ft^2	Wetted Perimeter P ft	Hydraulic Radius R=A/P ft	Average Velocity V ft/s	Discharge (Flow Rate) Q=AV ft^3/s	Avg. Tractive Stress τ_0 lb/ft^2	Comments
0.01	0.03	3.06	0.01	0.18	0.0	0.00	
0.16	0.55	4.01	0.14	1.04	0.6	0.04	
0.31	1.21	4.95	0.24	1.53	1.9	0.08	
0.46	2.01	5.90	0.34	1.90	3.8	0.11	
0.61	2.93	6.85	0.43	2.22	6.5	0.13	
0.76	3.99	7.79	0.51	2.50	10.0	0.16	
0.91	5.19	8.74	0.59	2.76	14.3	0.19	
1.06	6.52	9.68	0.67	3.00	19.5	0.21	
1.21	7.98	10.63	0.75	3.22	25.7	0.23	
1.36	9.58	11.57	0.83	3.44	32.9	0.26	
1.51	11.31	12.52	0.90	3.65	41.2	0.28	
1.65	13.18	13.46	0.98	3.85	50.7	0.31	
1.80	15.16	14.41	1.05	4.04	61.3	0.33	
1.95	17.31	15.36	1.13	4.23	73.2	0.35	
2.10	19.58	16.30	1.20	4.41	86.3	0.37	
2.25	21.98	17.25	1.27	4.59	100.8	0.40	
2.40	24.51	18.19	1.35	4.76	116.7	0.42	
2.55	27.18	19.14	1.42	4.93	134.1	0.44	
2.70	29.99	20.08	1.49	5.10	152.9	0.47	
2.85	32.93	21.03	1.57	5.26	173.3	0.49	
3.00	36.00	21.97	1.64	5.42	195.3	0.51	
1.29	8.82	11.13	0.79	3.34	29.47	0.25	Q (25-yr Event)
1.51	11.38	12.55	0.91	3.65	41.58	0.28	Q (100-yr Event)

Discharge versus Depth Relationship



Design/Check: Trapezoidal/Triangular Channel

Methodology: Manning's Equation

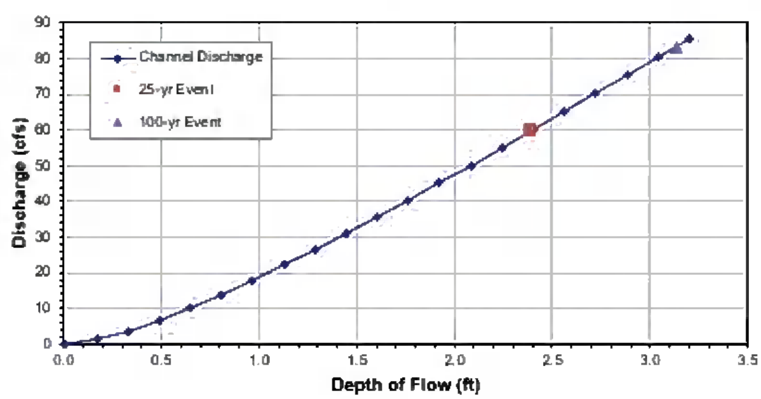
Project: Santee Cooper - Winyah Generating Station

Ditch ID: **Perimeter Reach 1C1** **Design**

Peak Discharge, Q_{25} =	59.60	cfs (25-yr Event)
Peak Discharge, Q_{100} =	83.30	cfs (100-yr Event)
Bottom Width, B =	3.00	ft
Left Side Slope, Z_1 =	0.00	horizontal : 1 vertical
Right Side Slope, Z_2 =	0.00	horizontal : 1 vertical
Channel Depth, Y =	3.20	ft
Top Width, T =	3.0	ft
Manning's Roughness Coeff., n =	0.012	
Longitudinal Channel Slope, S_o =	0.005	ft/ft

Depth of Flow Y ft	Area of Flow A ft²	Wetted Perimeter P ft	Hydraulic Radius R=A/P ft	Average Velocity V ft/s	Discharge (Flow Rate) Q=AV ft³/s	Avg. Tractive Stress τ_0 lb/ft²	Comments
0.01	0.03	3.02	0.01	0.41	0.0	0.00	
0.17	0.51	3.34	0.15	2.50	1.3	0.05	
0.33	0.99	3.66	0.27	3.66	3.6	0.08	
0.49	1.47	3.98	0.37	4.51	6.6	0.11	
0.65	1.94	4.30	0.45	5.17	10.1	0.14	
0.81	2.42	4.62	0.52	5.71	13.8	0.16	
0.97	2.90	4.93	0.59	6.16	17.9	0.18	
1.13	3.38	5.25	0.64	6.54	22.1	0.20	
1.29	3.86	5.57	0.69	6.87	26.5	0.22	
1.45	4.34	5.89	0.74	7.16	31.0	0.23	
1.61	4.82	6.21	0.78	7.41	35.7	0.24	
1.76	5.29	6.53	0.81	7.63	40.4	0.25	
1.92	5.77	6.85	0.84	7.83	45.2	0.26	
2.08	6.25	7.17	0.87	8.01	50.1	0.27	
2.24	6.73	7.49	0.90	8.18	55.0	0.28	
2.40	7.21	7.81	0.92	8.33	60.0	0.29	
2.56	7.69	8.12	0.95	8.46	65.0	0.30	
2.72	8.16	8.44	0.97	8.59	70.1	0.30	
2.88	8.64	8.76	0.99	8.70	75.2	0.31	
3.04	9.12	9.08	1.00	8.81	80.3	0.31	
3.20	9.60	9.40	1.02	8.90	85.5	0.32	
2.39	7.17	7.78	0.92	8.31	59.60	0.29	Q (25-yr Event)
3.13	9.40	9.27	1.01	8.86	83.30	0.32	Q (100-yr Event)

Discharge versus Depth Relationship



Design/Check: Trapezoidal/Triangular Channel

Methodology: Manning's Equation

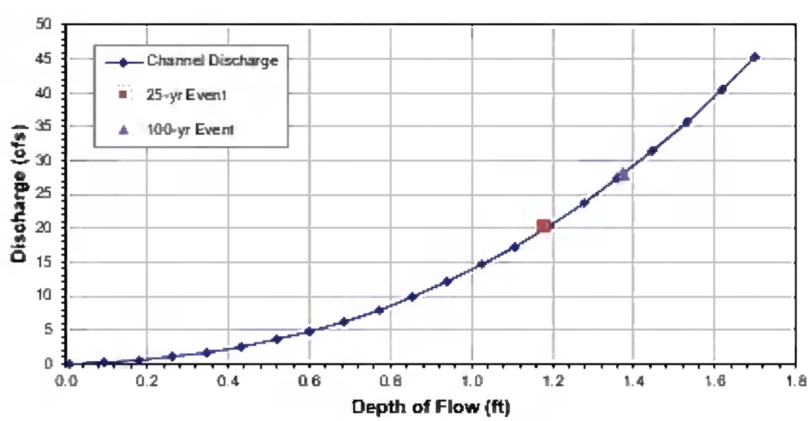
Project: Santee Cooper - Winyah Generating Station

Ditch ID: **Perimeter Reach 1C2** **Design**

Peak Discharge, Q_{25} =	20.10	cfs (25-yr Event)
Peak Discharge, Q_{100} =	28.10	cfs (100-yr Event)
Bottom Width, B =	2.00	ft
Left Side Slope, Z_1 =	3.00	horizontal : 1 vertical
Right Side Slope, Z_2 =	3.00	horizontal : 1 vertical
Channel Depth, Y =	1.70	ft
Top Width, T =	12.2	ft
Manning's Roughness Coeff., n =	0.027	
Longitudinal Channel Slope, S_o =	0.005	ft/ft

Depth of Flow Y ft	Area of Flow A ft²	Wetted Perimeter P ft	Hydraulic Radius R=A/P ft	Average Velocity V ft/s	Discharge (Flow Rate) Q=AV ft³/s	Avg. Tractive Stress τ_0 lb/ft²	Comments
0.01	0.02	2.06	0.01	0.18	0.0	0.00	
0.09	0.22	2.60	0.08	0.74	0.2	0.03	
0.18	0.45	3.13	0.14	1.08	0.5	0.05	
0.26	0.74	3.67	0.20	1.34	1.0	0.06	
0.35	1.06	4.20	0.25	1.56	1.6	0.08	
0.43	1.43	4.74	0.30	1.75	2.5	0.09	
0.52	1.84	5.27	0.35	1.93	3.5	0.11	
0.60	2.29	5.80	0.39	2.10	4.8	0.12	
0.69	2.78	6.34	0.44	2.25	6.3	0.14	
0.77	3.32	6.87	0.48	2.40	8.0	0.15	
0.86	3.90	7.41	0.53	2.55	9.9	0.16	
0.94	4.53	7.94	0.57	2.66	12.1	0.18	
1.02	5.19	8.48	0.61	2.81	14.6	0.19	
1.11	5.90	9.01	0.66	2.94	17.4	0.20	
1.19	6.66	9.55	0.70	3.07	20.4	0.22	
1.28	7.45	10.08	0.74	3.19	23.8	0.23	
1.36	8.29	10.61	0.78	3.31	27.4	0.24	
1.45	9.17	11.15	0.82	3.43	31.4	0.26	
1.53	10.09	11.68	0.86	3.54	35.7	0.27	
1.62	11.06	12.22	0.91	3.65	40.4	0.28	
1.70	12.07	12.75	0.95	3.76	45.4	0.30	
1.18	6.57	9.49	0.69	3.06	20.09	0.22	Q (25-yr Event)
1.38	8.43	10.70	0.79	3.33	28.08	0.25	Q (100-yr Event)

Discharge versus Depth Relationship



Design/Check: Trapezoidal/Triangular Channel

Methodology: Manning's Equation

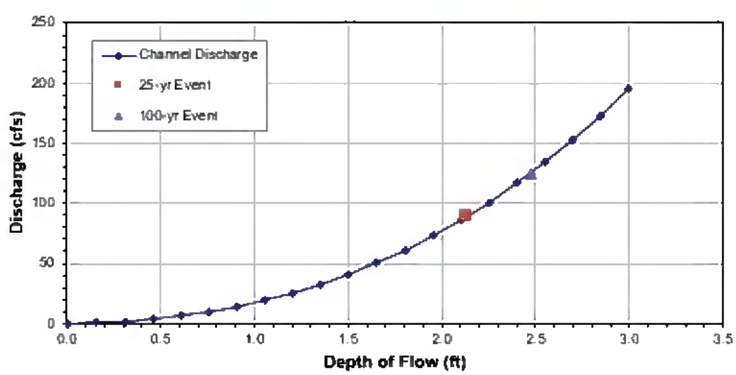
Project: Santee Cooper - Winyah Generating Station

Ditch ID: **Perimeter Reach 2A1** **Design**

Peak Discharge, Q_{25} =	89.50	cfs (25-yr Event)
Peak Discharge, Q_{100} =	125.30	cfs (100-yr Event)
Bottom Width, B =	3.00	ft
Left Side Slope, Z_1 =	3.00	horizontal : 1 vertical
Right Side Slope, Z_2 =	3.00	horizontal : 1 vertical
Channel Depth, Y =	3.00	ft
Top Width, T =	21.0	ft
Manning's Roughness Coeff., n =	0.027	
Longitudinal Channel Slope, S_o =	0.005	ft/ft

Depth of Flow Y ft	Area of Flow A ft²	Wetted Perimeter P ft	Hydraulic Radius R=A/P ft	Average Velocity V ft/s	Discharge (Flow Rate) Q=AV ft³/s	Avg. Tractive Stress τ_0 lb/ft²	Comments
0.01	0.03	3.06	0.01	0.18	0.0	0.00	
0.16	0.55	4.01	0.14	1.04	0.6	0.04	
0.31	1.21	4.95	0.24	1.53	1.9	0.08	
0.46	2.01	5.90	0.34	1.90	3.8	0.11	
0.61	2.93	6.85	0.43	2.22	6.5	0.13	
0.76	3.99	7.79	0.51	2.50	10.0	0.16	
0.91	5.19	8.74	0.59	2.76	14.3	0.19	
1.06	6.52	9.68	0.67	3.00	19.5	0.21	
1.21	7.98	10.63	0.75	3.22	25.7	0.23	
1.36	9.58	11.57	0.83	3.44	32.9	0.26	
1.51	11.31	12.52	0.90	3.65	41.2	0.28	
1.65	13.18	13.46	0.98	3.85	50.7	0.31	
1.80	15.18	14.41	1.05	4.04	61.3	0.33	
1.95	17.31	15.36	1.13	4.23	73.2	0.35	
2.10	19.58	16.30	1.20	4.41	86.3	0.37	
2.25	21.98	17.25	1.27	4.59	100.8	0.40	
2.40	24.51	18.19	1.35	4.76	116.7	0.42	
2.55	27.18	19.14	1.42	4.93	134.1	0.44	
2.70	29.99	20.08	1.49	5.10	152.9	0.47	
2.85	32.93	21.03	1.57	5.26	173.3	0.49	
3.00	36.00	21.97	1.64	5.42	195.3	0.51	
2.14	20.09	16.51	1.22	4.45	89.38	0.38	Q (25-yr Event)
2.48	25.82	18.66	1.38	4.85	125.12	0.43	Q (100-yr Event)

Discharge versus Depth Relationship



Design/Check: Trapezoidal/Triangular Channel

Methodology: Manning's Equation

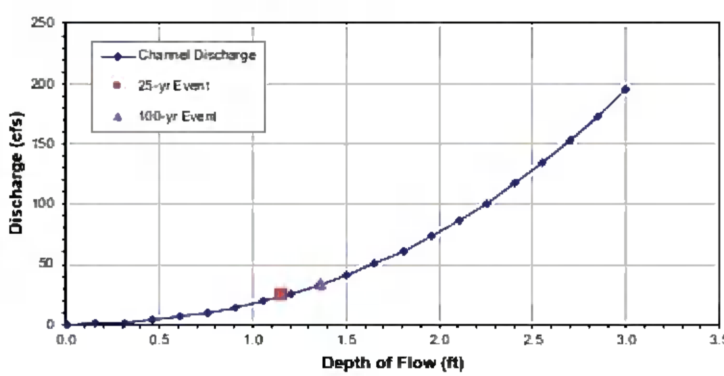
Project: Santee Cooper - Winyah Generating Station

Ditch ID: **Perimeter Reach 2A2** **Design**

Peak Discharge, Q_{25} =	23.70	cfs (25-yr Event)
Peak Discharge, Q_{100} =	33.10	cfs (100-yr Event)
Bottom Width, B =	3.00	ft
Left Side Slope, Z_1 =	3.00	horizontal : 1 vertical
Right Side Slope, Z_2 =	3.00	horizontal : 1 vertical
Channel Depth, Y =	3.00	ft
Top Width, T =	21.0	ft
Manning's Roughness Coeff., n =	0.027	
Longitudinal Channel Slope, S_o =	0.005	ft/ft

Depth of Flow Y ft	Area of Flow A ft^2	Wetted Perimeter P ft	Hydraulic Radius R=A/P ft	Average Velocity V ft/s	Discharge (Flow Rate) Q=AV ft^3/s	Avg. Tractive Stress τ_0 lb/ft^2	Comments
0.01	0.03	3.06	0.01	0.18	0.0	0.00	
0.16	0.55	4.01	0.14	1.04	0.6	0.04	
0.31	1.21	4.95	0.24	1.53	1.9	0.08	
0.46	2.01	5.90	0.34	1.90	3.8	0.11	
0.61	2.93	6.85	0.43	2.22	6.5	0.13	
0.76	3.99	7.79	0.51	2.50	10.0	0.16	
0.91	5.19	8.74	0.59	2.76	14.3	0.19	
1.06	6.52	9.68	0.67	3.00	19.5	0.21	
1.21	7.98	10.63	0.75	3.22	25.7	0.23	
1.36	9.58	11.57	0.83	3.44	32.9	0.26	
1.51	11.31	12.52	0.90	3.65	41.2	0.28	
1.65	13.18	13.46	0.98	3.85	50.7	0.31	
1.80	15.18	14.41	1.05	4.04	61.3	0.33	
1.95	17.31	15.36	1.13	4.23	73.2	0.35	
2.10	19.58	16.30	1.20	4.41	86.3	0.37	
2.25	21.98	17.25	1.27	4.59	100.8	0.40	
2.40	24.51	18.19	1.35	4.76	116.7	0.42	
2.55	27.18	19.14	1.42	4.93	134.1	0.44	
2.70	29.99	20.08	1.49	5.10	152.9	0.47	
2.85	32.93	21.03	1.57	5.26	173.3	0.49	
3.00	36.00	21.97	1.64	5.42	195.3	0.51	
1.16	7.49	10.32	0.73	3.15	23.59	0.23	Q (25-yr Event)
1.36	9.61	11.59	0.83	3.44	33.09	0.26	Q (100-yr Event)

Discharge versus Depth Relationship



Design/Check: Trapezoidal/Triangular Channel

Methodology: Manning's Equation

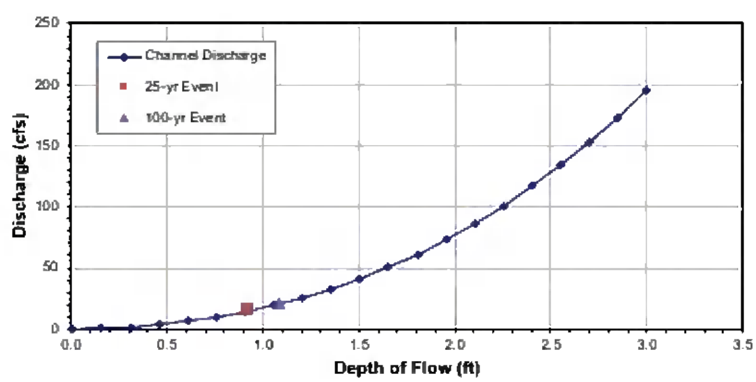
Project: Santee Cooper - Winyah Generating Station

Ditch ID: **Perimeter Reach 2BA** **Design**

Peak Discharge, Q_{25} =	14.90	cfs (25-yr Event)
Peak Discharge, Q_{100} =	20.80	cfs (100-yr Event)
Bottom Width, B =	3.00	ft
Left Side Slope, Z_1 =	3.00	horizontal : 1 vertical
Right Side Slope, Z_2 =	3.00	horizontal : 1 vertical
Channel Depth, Y =	3.00	ft
Top Width, T =	21.0	ft
Manning's Roughness Coeff., n =	0.027	
Longitudinal Channel Slope, S_o =	0.005	ft/ft

Depth of Flow Y ft	Area of Flow A ft^2	Wetted Perimeter P ft	Hydraulic Radius R=A/P ft	Average Velocity V ft/s	Discharge (Flow Rate) Q=AV ft^3/s	Avg. Tractive Stress τ_0 lb/ft^2	Comments
0.01	0.03	3.06	0.01	0.18	0.0	0.00	
0.16	0.55	4.01	0.14	1.04	0.6	0.04	
0.31	1.21	4.95	0.24	1.53	1.9	0.08	
0.46	2.01	5.90	0.34	1.90	3.8	0.11	
0.61	2.93	6.85	0.43	2.22	6.5	0.13	
0.76	3.99	7.79	0.51	2.50	10.0	0.16	
0.91	5.19	8.74	0.59	2.76	14.3	0.19	
1.06	6.52	9.68	0.67	3.00	19.5	0.21	
1.21	7.98	10.63	0.75	3.22	25.7	0.23	
1.36	9.58	11.57	0.83	3.44	32.9	0.26	
1.51	11.31	12.52	0.90	3.65	41.2	0.28	
1.65	13.16	13.46	0.98	3.85	50.7	0.31	
1.80	15.18	14.41	1.05	4.04	61.3	0.33	
1.95	17.31	15.36	1.13	4.23	73.2	0.35	
2.10	19.58	16.30	1.20	4.41	86.3	0.37	
2.25	21.98	17.25	1.27	4.59	100.8	0.40	
2.40	24.51	18.19	1.35	4.76	116.7	0.42	
2.55	27.18	19.14	1.42	4.93	134.1	0.44	
2.70	29.99	20.08	1.49	5.10	152.9	0.47	
2.85	32.93	21.03	1.57	5.26	173.3	0.49	
3.00	36.00	21.97	1.64	5.42	195.3	0.51	
0.92	5.33	8.84	0.60	2.78	14.85	0.19	Q (25-yr Event)
1.09	6.81	9.87	0.69	3.04	20.72	0.22	Q (100-yr Event)

Discharge versus Depth Relationship



Design/Check: Trapezoidal/Triangular Channel

Methodology: Manning's Equation

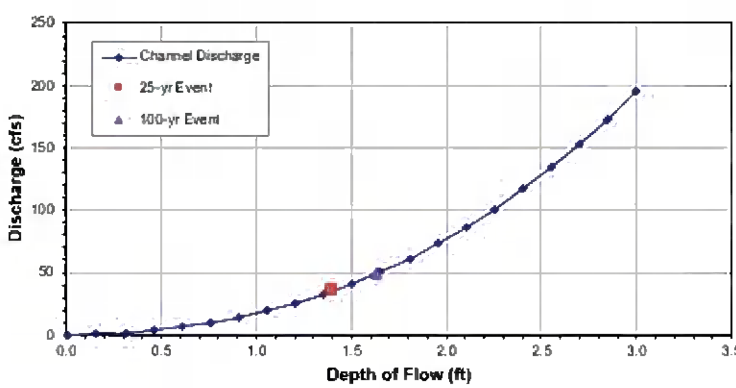
Project: Santee Cooper - Winyah Generating Station

Ditch ID: **Perimeter Reach 2BB** **Design**

Peak Discharge, Q_{25} =	35.20	cfs (25-yr Event)
Peak Discharge, Q_{100} =	49.30	cfs (100-yr Event)
Bottom Width, B =	3.00	ft
Left Side Slope, Z_1 =	3.00	horizontal : 1 vertical
Right Side Slope, Z_2 =	3.00	horizontal : 1 vertical
Channel Depth, Y =	3.00	ft
Top Width, T =	21.0	ft
Manning's Roughness Coeff., n =	0.027	
Longitudinal Channel Slope, S_o =	0.005	ft/ft

Depth of Flow Y ft	Area of Flow A ft^2	Wetted Perimeter P ft	Hydraulic Radius R=A/P ft	Average Velocity V ft/s	Discharge (Flow Rate) Q=AV ft^3/s	Avg. Tractive Stress τ_0 lb/ft^2	Comments
0.01	0.03	3.06	0.01	0.18	0.0	0.00	
0.16	0.55	4.01	0.14	1.04	0.6	0.04	
0.31	1.21	4.95	0.24	1.53	1.9	0.08	
0.46	2.01	5.90	0.34	1.90	3.8	0.11	
0.61	2.93	6.85	0.43	2.22	6.5	0.13	
0.76	3.99	7.79	0.51	2.50	10.0	0.16	
0.91	5.19	8.74	0.59	2.76	14.3	0.19	
1.06	6.52	9.68	0.67	3.00	19.5	0.21	
1.21	7.98	10.63	0.75	3.22	25.7	0.23	
1.36	9.58	11.57	0.83	3.44	32.9	0.26	
1.51	11.31	12.52	0.90	3.65	41.2	0.28	
1.65	13.18	13.46	0.98	3.85	50.7	0.31	
1.60	15.18	14.41	1.05	4.04	61.3	0.33	
1.95	17.31	15.36	1.13	4.23	73.2	0.35	
2.10	19.58	16.30	1.20	4.41	86.3	0.37	
2.25	21.96	17.25	1.27	4.59	100.8	0.40	
2.40	24.51	18.19	1.35	4.76	116.7	0.42	
2.55	27.18	19.14	1.42	4.93	134.1	0.44	
2.70	29.99	20.08	1.49	5.10	152.9	0.47	
2.85	32.93	21.03	1.57	5.26	173.3	0.49	
3.00	36.00	21.97	1.64	5.42	195.3	0.51	
1.40	10.04	11.83	0.85	3.50	35.09	0.26	Q (25-yr Event)
1.63	12.89	13.33	0.97	3.82	49.23	0.30	Q (100-yr Event)

Discharge versus Depth Relationship



Design/Check: Trapezoidal/Triangular Channel

Methodology: Manning's Equation

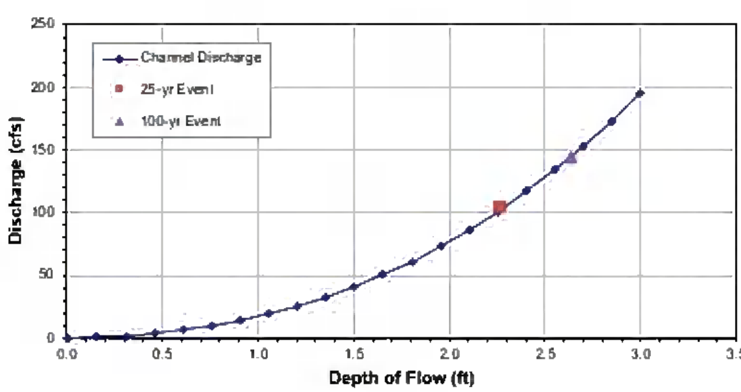
Project: Santee Cooper - Winyah Generating Station

Ditch ID: **Perimeter Reach 2C1** **Design**

Peak Discharge, Q_{25} =	102.80	cfs (25-yr Event)
Peak Discharge, Q_{100} =	144.20	cfs (100-yr Event)
Bottom Width, B =	3.00	ft
Left Side Slope, Z_1 =	3.00	horizontal : 1 vertical
Right Side Slope, Z_2 =	3.00	horizontal : 1 vertical
Channel Depth, Y =	3.00	ft
Top Width, T =	21.0	ft
Manning's Roughness Coeff., n =	0.027	
Longitudinal Channel Slope, S_o =	0.005	ft/ft

Depth of Flow Y ft	Area of Flow A ft^2	Wetted Perimeter P ft	Hydraulic Radius R=A/P ft	Average Velocity V ft/s	Discharge (Flow Rate) Q=AV ft^3/s	Avg. Tractive Stress τ_0 lb/ft^2	Comments
0.01	0.03	3.06	0.01	0.18	0.0	0.00	
0.16	0.55	4.01	0.14	1.04	0.6	0.04	
0.31	1.21	4.95	0.24	1.53	1.9	0.08	
0.46	2.01	5.90	0.34	1.90	3.8	0.11	
0.61	2.93	6.85	0.43	2.22	6.5	0.13	
0.76	3.99	7.79	0.51	2.50	10.0	0.16	
0.91	5.19	8.74	0.59	2.76	14.3	0.19	
1.06	6.52	9.68	0.67	3.00	19.5	0.21	
1.21	7.98	10.63	0.75	3.22	25.7	0.23	
1.36	9.58	11.57	0.83	3.44	32.9	0.26	
1.51	11.31	12.52	0.90	3.65	41.2	0.28	
1.65	13.18	13.46	0.98	3.85	50.7	0.31	
1.80	15.18	14.41	1.05	4.04	61.3	0.33	
1.95	17.31	15.36	1.13	4.23	73.2	0.35	
2.10	19.58	16.30	1.20	4.41	86.3	0.37	
2.25	21.98	17.25	1.27	4.59	100.8	0.40	
2.40	24.51	18.19	1.35	4.76	116.7	0.42	
2.55	27.18	19.14	1.42	4.93	134.1	0.44	
2.70	29.99	20.08	1.49	5.10	152.9	0.47	
2.85	32.93	21.03	1.57	5.26	173.3	0.49	
3.00	36.00	21.97	1.64	5.42	195.3	0.51	
2.27	22.29	17.36	1.28	4.61	102.72	0.40	Q (25-yr Event)
2.63	28.68	19.65	1.46	5.02	144.01	0.46	Q (100-yr Event)

Discharge versus Depth Relationship



Design/Check: Trapezoidal/Triangular Channel

Methodology: Manning's Equation

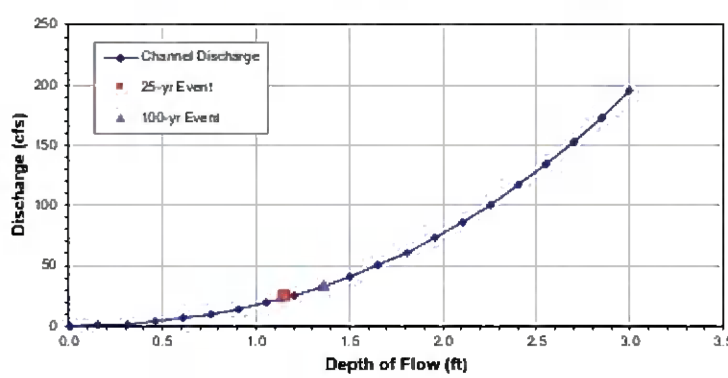
Project: Santee Cooper - Winyah Generating Station

Ditch ID: **Perimeter Reach 2DA** **Design**

Peak Discharge, Q_{25} =	23.60	cfs (25-yr Event)
Peak Discharge, Q_{100} =	33.10	cfs (100-yr Event)
Bottom Width, B =	3.00	ft
Left Side Slope, Z_1 =	3.00	horizontal : 1 vertical
Right Side Slope, Z_2 =	3.00	horizontal : 1 vertical
Channel Depth, Y =	3.00	ft
Top Width, T =	21.0	ft
Manning's Roughness Coeff., n =	0.027	
Longitudinal Channel Slope, S_o =	0.005	ft/ft

Depth of Flow Y ft	Area of Flow A ft^2	Wetted Perimeter P ft	Hydraulic Radius R=A/P ft	Average Velocity V ft/s	Discharge (Flow Rate) Q=AV ft^3/s	Avg. Tractive Stress τ_0 lb/ft^2	Comments
0.01	0.03	3.06	0.01	0.18	0.0	0.00	
0.16	0.55	4.01	0.14	1.04	0.6	0.04	
0.31	1.21	4.95	0.24	1.53	1.9	0.08	
0.46	2.01	5.90	0.34	1.90	3.8	0.11	
0.61	2.93	6.65	0.43	2.22	6.5	0.13	
0.76	3.99	7.79	0.51	2.50	10.0	0.16	
0.91	5.19	6.74	0.59	2.76	14.3	0.19	
1.06	6.52	9.68	0.67	3.00	19.5	0.21	
1.21	7.98	10.63	0.75	3.22	25.7	0.23	
1.36	9.58	11.57	0.83	3.44	32.9	0.26	
1.51	11.31	12.52	0.90	3.65	41.2	0.28	
1.65	13.18	13.46	0.98	3.85	50.7	0.31	
1.80	15.18	14.41	1.05	4.04	61.3	0.33	
1.95	17.31	15.36	1.13	4.23	73.2	0.35	
2.10	19.58	16.30	1.20	4.41	86.3	0.37	
2.25	21.98	17.25	1.27	4.59	100.8	0.40	
2.40	24.51	18.19	1.35	4.76	116.7	0.42	
2.55	27.18	19.14	1.42	4.93	134.1	0.44	
2.70	29.99	20.08	1.49	5.10	152.9	0.47	
2.85	32.93	21.03	1.57	5.26	173.3	0.49	
3.00	36.00	21.97	1.64	5.42	195.3	0.51	
1.15	7.46	10.30	0.72	3.15	23.49	0.23	Q (25-yr Event)
1.36	9.61	11.59	0.83	3.44	33.09	0.26	Q (100-yr Event)

Discharge versus Depth Relationship



APPENDIX F-4

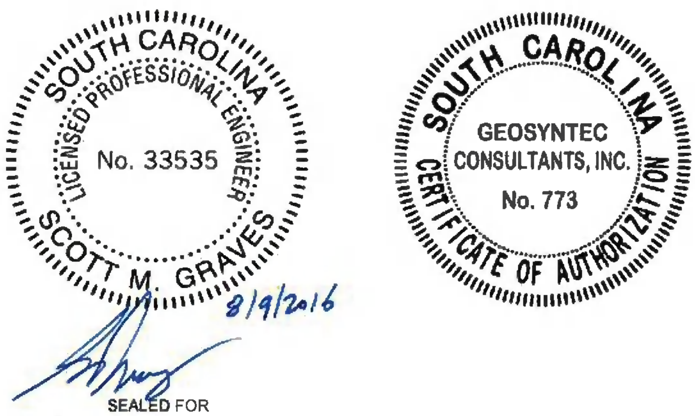
FINAL COVER SOIL EROSION LOSS CALCULATIONS

Written by: **A. Sivashanthan** Date: **6/15/2016** Reviewed by: **B. Klenzendorf** Date: **6/16/2016**

Client: **Santee Cooper** Project: **Winyah Generating Station** Project No.: **GSC5242** Phase No.: **01**

APPENDIX F-4

FINAL COVER SOIL EROSION LOSS CALCULATIONS



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CALCULATION PAGES F4-1
THROUGH F4-13

1 PURPOSE

The purpose of this calculation package is to present the evaluation of the long term effects of erosion and soil loss for the completed final cover system of the proposed Class Three Landfill at the Winyah Generating Station (WGS) located in Georgetown, South Carolina. This package provides calculations for the annual soil loss per acre from the final cover, including top deck surfaces with a grade of 5% and sideslopes with a slope of 3H:1V (horizontal:vertical) (i.e., 33.3%) as described in the Engineering Report and shown on the Engineering Drawings. The estimated amount of erosion was calculated using the Revised Universal Soil Loss Equation (RUSLE).

2 FINAL COVER SOIL EROSION LOSS CALCULATION METHODOLOGY

The method to calculate the estimated soil erosion loss over the project area was obtained from the guidance document *Predicting Soil Erosion by Water: A Guide to Conservation Planning with the Revised Universal Soil Loss Equation (RUSLE)* (USDA, 1997) as well as previously published information provided by USDA. This document presents the RUSLE methodology and guidance for each of the equation's parameters. The RUSLE is described as follows:

$$A = R \times K \times LS \times C \times P$$

where: A = the computed spatial average annual soil loss (tons/acre/year),

R = the average annual rainfall runoff erosivity factor,

K = the soil erodibility factor,

LS = the topographic factor,

C = the cover management factor, and

P = the erosion control practice factor.

3 RUSLE INPUT PARAMETERS

3.1 Rainfall Runoff Erosivity Factor (R)

The rainfall runoff erosivity factor is defined as the average annual rainfall erosion index specific for the project area. Based on USDA (1997), the value was determined to be approximately 400 for Georgetown, SC, as shown in Figure 1 at the end of this document.

3.2 Soil Erodibility Factor (K)

The soil erodibility factor is a function of the physical and chemical properties of the soil and is specific to the source of the cover material. The soil erodibility factor can be thought of as the ease with which soil is detached by splash during rainfall or by surface flow. More specifically, K is an estimated measure of the erodibility of a soil's fine-earth fraction; that is, the fraction of material less than two mm in size. For soil loss calculation purposes, assessments were made of on-site soils and those nearby, using the Georgetown County soil survey (USDA, 2016). Figure 2 shows that the site and nearby areas have a combination of a number of soil classifications including the following: Udorthents loamy soil (map symbol 58); Witherbee fine sand (map symbol 55); Eulonia loamy fine sand (map symbol 26A); Yauhannah loamy fine sand (map symbol 12A); and Echaw sand (map symbol 28)..

The Web Soil Survey tool operated by the USDA Natural Resources Conservation Service (NRCS) (USDA, 2016) was consulted for Georgetown County for information on the corresponding soil erodibility factors. Although the exact borrow source for the final cover system vegetative layer has not been designated, it is reasonable to assume that near-surface soils (i.e., topsoil) from nearby areas in and around the WGS site will be used to construct the topsoil layer of the final cover system. The value of K for the project location for soils near the surface varies from 0.02 to 0.32, where the estimate considers the erodibility of fine-earth fraction for material less than two mm in size (using the Kf erosion factor provided in Table 1). The main surface layer soils in the area are Eulonia loamy fine sand and Udorthents loamy soil, and value of Kf for these soils is 0.20 and 0.17, respectively. The use of 0.20 in the calculation is using a conservative value of the formations that are most predominant at the site and surrounding areas (i.e., a likely candidate source of future final cover topsoil).

3.3 Topographic Factor (LS)

The slope length factor, L, and slope steepness factor, S, are typically combined into one topographic factor, LS, to facilitate field application of these equation components. USDA (1997) presents values of the LS factor for slope lengths in feet up to 1,000 feet and percent slopes up to 60%, as shown in Table 2, for soils with vegetated cover with consolidated soil conditions.

The longest slope lengths for the sideslope and top deck surfaces of the final cover system for both landfills (i.e., Area 1 and Area 2) were used to select the LS factor for each area, and these lengths were applied to compute the soil loss for both portions of the landfill. The top deck surface will consist of a 5% nominal maximum slope with a maximum length of 110 ft. The final cover system will consist of 3H:1V (33.3%) sideslopes with a maximum length of 100 ft between drainage terraces. Also, a computation was performed for a hypothetical scenario of a 300 ft long sideslope at 33% (in order to back-calculate the maximum terrace spacing that would yield an acceptably low soil loss design). Based on these slope lengths, the following LS factors were selected (and interpolated if necessary) from Table 2:

- Sideslopes – 3H:1V (33.3%) over the maximum landfill design slope length (between terraces) of 100 ft, LS = 5.63
- Sideslopes – 3H:1V (33.3%) over a hypothetical design slope length (between terraces) of 300 ft, LS = 9.78
- Top Deck – 5% slope over the maximum landfill design slope length of 110 ft, LS = 0.63

3.4 Cover Management Factor (C)

The cover management factor is a function of land cover, and is based on vegetative cover in direct contact with the soil surface, protection from erosion offered by canopy cover, and the effects of past cropping and management. The final cover of the landfill is categorized as having no appreciable canopy with a vegetated cover of grass, grass-like plants, or decaying compacted duff or litter ("duff or litter" refers to mulch and similar organic matter) at least 2 inches deep. There is assumed to be 90% ground cover, which represents a reasonable expectation of the long-term post-closure ground cover condition. This is considered a reasonable and appropriate long-term condition because of the climate and because of the Post-Closure Plan's requirements for ongoing inspections, maintenance and mowing, and repairs. These ground cover conditions result in a C value (interpolated from Table 3 (USDA, 1977)) of C = 0.0063.

3.5 Erosion Control Practice Factor (P)

The erosion control practice factor considers topographical practices that will reduce erosion by altering runoff drainage patterns. This factor generally applies to agricultural cropping practices and is not anticipated for the landfill. Therefore, the P factor is assumed to be equal to one (1).

3.6 Tolerable Soil Loss (T)

The calculated soil loss should be compared to the tolerable (i.e., permissible) soil loss (T). Several sources of information exist to select the permissible soil loss. The USDA soil-specific survey of Georgetown County soils (USDA, 2016) lists the "T" factors recommended for each soil type. This value represents the maximum average annual rate of soil erosion "that can occur without affecting crop productivity over a sustained period". For the landfill case, the term "crop productivity" refers to vegetation sustainability (lack of excessive erosion). As shown in Table 2, the USDA's recommended permissible soil loss rate for Eulonia loamy fine sand and Udorthents loamy soil (the predominant soil types at the site and surrounding areas (i.e., a likely candidate source of future final cover topsoil)) is 5 tons/acre/year.

4 SOIL EROSION LOSS RESULTS

Applying the RUSLE with the parameters defined above, the computed soil loss in tons/acre/year is calculated as follows:

$$A = R \times K \times LS \times C \times P$$

- Final Cover Sideslopes, Base Design Case (maximum spacing of 100 ft between terraces):

  A = 400 × 0.20 × 5.63 × 0.0063 × 1 = 2.84 tons/acre/year

- Final Cover Side Slopes, Back-Calculated Hypothetical Case (300 ft between terraces):

  A = 400 × 0.20 × 9.78× 0.0063 × 1 = 4.93 tons/acre/year

- Final Cover Top Deck, Base Design Case (maximum slope length of 110 ft):

  A = 400 × 0.20 × 0.63 × 0.0063 × 1 = 0.32 tons/acre/year

5 CONCLUSIONS

Based on the analyses presented herein, the following conclusions are drawn:

- Overall, the calculated soil loss from the final cover system design is below the permissible soil loss of 5 tons/acre/year recommended by USDA (2016) for the area/site-specific soils. Specifically, results are:
  - As discussed in Section 3.2 (Soil Erodibility Factor, K), the native site soils as well as the locally available soils have K values of 0.20 to 0.17 (Table 1).
  - The average annual soil loss from the final cover on the 3H:1V landfill sideslopes as-designed for all of the variables selected as the base design case is 2.84 tons/acre/year, which is below permissible soil loss recommended by USDA (2016) for the area/site-specific soils.
  - The average annual soil loss from the final cover on the top deck surface as-designed for all of the variables selected as the base design case is 0.32 tons/acre/year. This is much lower than permissible soil loss recommended by USDA (2016) for the area/site-specific soils.
  - To provide effective erosional stability against soil loss, the maximum

spacing of the final cover sideslope drainage terraces on the 3H:1V landfill sideslopes should be 300 ft or less. The final cover design meets this spacing requirement.

6 REFERENCES

USDA (1977). Procedure for Computing Sheet and Rill Erosion on Project Areas, United States Department of Agriculture, Soil Conservation Services, Engineering Division and Ecological Sciences and Technology Division, Technical Release No. 51, Revision 2.

USDA (1978). Predicting Rainfall Erosion Losses: A Guide to Conservation Planning, United States Department of Agriculture, Science and Education Administration, Agriculture Handbook Number 537.

USDA (1981). Soil Survey of Tarrant County, Texas, United States Department of Agriculture and Soil Conservation Service, with Texas Agricultural Experiment Station, Issued 1981.

USDA (1997). Predicting Soil Erosion by Water: A Guide to Conservation Planning with the Revised Universal Soil Loss Equation (RUSLE), United States Department of Agriculture, Agricultural Research Service, Agriculture Handbook Number 703.

USDA (2016). Web Soil Survey, Soil Survey Staff, Natural Resources Conservation Service (NRCS), United States Department of Agriculture, available online at http://websoilsurvey.nrcs.usda.gov, accessed June 2016.

TABLES

- Table 1. Soil Erodibility Factor K for Site Soils (from USDA, 2016)
- Table 2. Values for Topographic Factor, LS, for Low Ratio of Rill to Interrill Erosion (from USDA, 1997)
- Table 3. C Factor Cover Values for Permanent Pasture, Rangeland, Idle Land, and Grazed Woodland (from USDA, 1977)

Table 1. Soil Erodibility Factor K for Site Soils (from USDA, 2016)

RUSLE2 Related Attributes–Georgetown County, South Carolina								
Map symbol and soil name	Pct. of map unit	Slope length (ft)	Hydrologic group	Kf	T factor	Representative value		
						% Sand	% Silt	% Clay
10—Leon sand, 0 to 2 percent slopes								
Leon	84	151	A/D	.02	5	95.0	2.0	3.0
12A—Yauhannah loamy fine sand, 0 to 2 percent slopes								
Yauhannah	95	656	B/D	.17	5	83.5	6.5	10.0
13—Bladen loam								
Bladen	100	656	C/D	.32	5	44.3	40.7	15.0
18—Cape Fear loam								
Cape Fear	100	656	C/D	.24	5	46.0	37.0	17.0
20—Centenary fine sand								
Centenary	100	656	A	.02	5	94.9	0.6	4.5
24B—Chisolm sand, 0 to 4 percent slopes								
Chisolm	100	492	A	.05	5	92.7	1.3	6.0
25A—Wakulla sand, 0 to 2 percent slopes								
Wakulla	100	656	A	.02	5	93.6	1.4	5.0
26A—Eulonia loamy fine sand, 0 to 2 percent slopes								
Eulonia	97	656	C/D	.20	5	83.5	6.5	10.0
27—Rutlege sand								
Rutlege	100	656	A/D	.02	5	92.1	1.9	6.0
28—Echaw sand								
Echaw	95	656	A	.02	5	94.1	1.4	4.5
33—Hobonny muck								
Hobonny	100	656	A/D	—	2	—	—	0.0
34—Johnston loam								
Johnston	100	656	A/D	.32	4	45.3	43.2	11.5
36B—Lakeland fine sand, 0 to 6 percent slopes								
Lakeland	100	656	A	.02	5	94.4	0.6	5.0
50—Lynn Haven sand								
Lynn Haven	100	656	A/D	.02	5	96.0	1.5	2.5
54A—Chipley fine sand, 0 to 2 percent slopes								
Chipley	85	656	A	.02	5	95.5	1.5	3.0
55—Witherbee fine sand								
Witherbee	95	656	A/D	.02	5	95.5	1.5	3.0
57—Grifton loamy fine sand								
Grifton	100	656	B/D	.20	5	87.3	6.7	6.0
58—Udorthents, loamy								
Udorthents	100	656	B/D	.17	5	55.5	14.5	30.0
59—Wahee fine sandy loam								
Wahee	95	656	C/D	.24	4	67.3	20.2	12.5
61—Yemassee loamy fine sand								
Yemassee	95	656	B/D	.24	5	83.5	6.5	10.0

Table 2. Values for Topographic Factor, LS, for Low Ratio of Rill to Interrill Erosion[1] (from USDA, 1997)

Values for topographic factor, LS, for low ratio of rill to interrill erosion.[1]

Slope (%)	Horizontal slope length (ft) <3	6	9	12	15	25	50	75	100	150	200	250	300	400	600	800	1000
0.2	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05
0.5	0.08	0.08	0.08	0.08	0.08	0.08	0.08	0.08	0.09	0.09	0.09	0.09	0.09	0.09	0.09	0.09	0.09
1.0	0.12	0.12	0.12	0.12	0.12	0.13	0.13	0.14	0.14	0.15	0.15	0.15	0.15	0.16	0.16	0.17	0.17
2.0	0.20	0.20	0.20	0.20	0.20	0.21	0.23	0.25	0.26	0.27	0.28	0.29	0.30	0.31	0.33	0.34	0.35
3.0	0.26	0.26	0.26	0.26	0.26	0.29	0.33	0.36	0.38	0.40	0.43	0.44	0.46	0.48	0.52	0.55	0.57
4.0	0.33	0.33	0.33	0.33	0.33	0.36	0.43	0.46	0.50	0.54	0.58	0.61	0.63	0.67	0.74	0.78	0.82
5.0	0.38	0.38	0.38	0.38	0.38	0.44	0.52	0.57	0.62	0.68	0.73	0.78	0.81	0.87	0.97	1.04	1.10
6.0	0.44	0.44	0.44	0.44	0.44	0.50	0.61	0.68	0.74	0.83	0.90	0.95	1.00	1.08	1.21	1.31	1.40
8.0	0.54	0.54	0.54	0.54	0.54	0.64	0.79	0.90	0.99	1.12	1.23	1.32	1.40	1.53	1.74	1.91	2.05
10.0	0.60	0.63	0.65	0.66	0.68	0.81	1.03	1.19	1.31	1.51	1.67	1.80	1.92	2.13	2.45	2.71	2.93
12.0	0.61	0.70	0.75	0.80	0.83	1.01	1.31	1.52	1.69	1.97	2.20	2.39	2.56	2.85	3.32	3.70	4.02
14.0	0.63	0.76	0.86	0.92	0.98	1.20	1.58	1.85	2.08	2.44	2.73	2.99	3.21	3.60	4.23	4.74	5.18
16.0	0.65	0.82	0.94	1.04	1.12	1.38	1.85	2.18	2.46	2.91	3.28	3.60	3.88	4.37	5.17	5.82	6.39
20.0	0.68	0.93	1.11	1.26	1.39	1.74	2.37	2.84	3.22	3.85	4.38	4.83	5.24	5.95	7.13	8.10	8.94
25.0	0.73	1.05	1.30	1.51	1.70	2.17	3.00	3.63	4.16	5.03	5.76	6.39	6.96	7.97	9.65	11.04	12.26
30.0	0.77	1.16	1.48	1.75	2.00	2.57	3.60	4.40	5.06	6.18	7.11	7.94	8.68	9.99	12.19	14.04	15.66
40.0	0.85	1.36	1.79	2.17	2.53	3.30	4.73	5.84	6.78	8.37	9.71	10.91	11.99	13.92	17.19	19.88	22.41
50.0	0.91	1.52	2.06	2.54	3.00	3.95	5.74	7.14	8.33	10.37	12.11	13.65	15.06	17.59	21.88	25.55	28.82
60.0	0.97	1.67	2.29	2.86	3.41	4.52	6.63	8.29	9.72	12.16	14.26	16.13	17.84	20.92	26.17	30.68	34.71

[1] Such as for rangeland and other consolidated soil conditions with cover (applicable to thawing soil where both interrill and rill erosion are significant).

Table 3. C Factor Cover Values for Permanent Pasture, Rangeland, Idle Land, and Grazed Woodland[1] (from USDA, 1977)

Vegetal Canopy		Cover That Contacts the Surface						
Type and Height of Raised Canopy[2/]	Canopy Cover[3/] %	Type[4/]	Percent Ground Cover					
			0	20	40	60	80	95-100
No appreciable canopy		G	.45	.20	.10	.042	.013	.003
		W	.45	.24	.15	.090	.043	.011
Canopy of tall weeds or short brush (0.5 m fall ht.)	25	G	.36	.17	.09	.038	.012	.003
		W	.36	.20	.13	.082	.041	.011
	50	G	.26	.13	.07	.035	.012	.003
		W	.26	.16	.11	.075	.039	.011
	75	G	.17	.10	.06	.031	.011	.003
		W	.17	.12	.09	.067	.038	.011
Appreciable brush or bushes (2 m fall ht.)	25	G	.40	.18	.09	.040	.013	.003
		W	.40	.22	.14	.085	.042	.011
	50	G	.34	.16	.085	.038	.012	.003
		W	.34	.19	.13	.081	.041	.011
	75	G	.28	.14	.08	.036	.012	.003
		W	.28	.17	.12	.077	.040	.011
Trees but no appreciable low brush (4 m fall ht.)	25	G	.42	.19	.10	.041	.013	.003
		W	.42	.23	.14	.087	.042	.011
	50	G	.39	.18	.09	.040	.013	.003
		W	.39	.21	.14	.085	.042	.011
	75	G	.36	.17	.09	.039	.012	.003
		W	.36	.20	.13	.083	.041	.011

[1/] All values shown assume: (1) random distribution of mulch or vegetation, and (2) mulch of appreciable depth where it exists. Idle land refers to land with undisturbed profiles for at least a period of three consecutive years. Also to be used for burned forest land and forest land that has been harvested less than three years ago.

[2/] Average fall height of waterdrops from canopy to soil surface: m = meters.

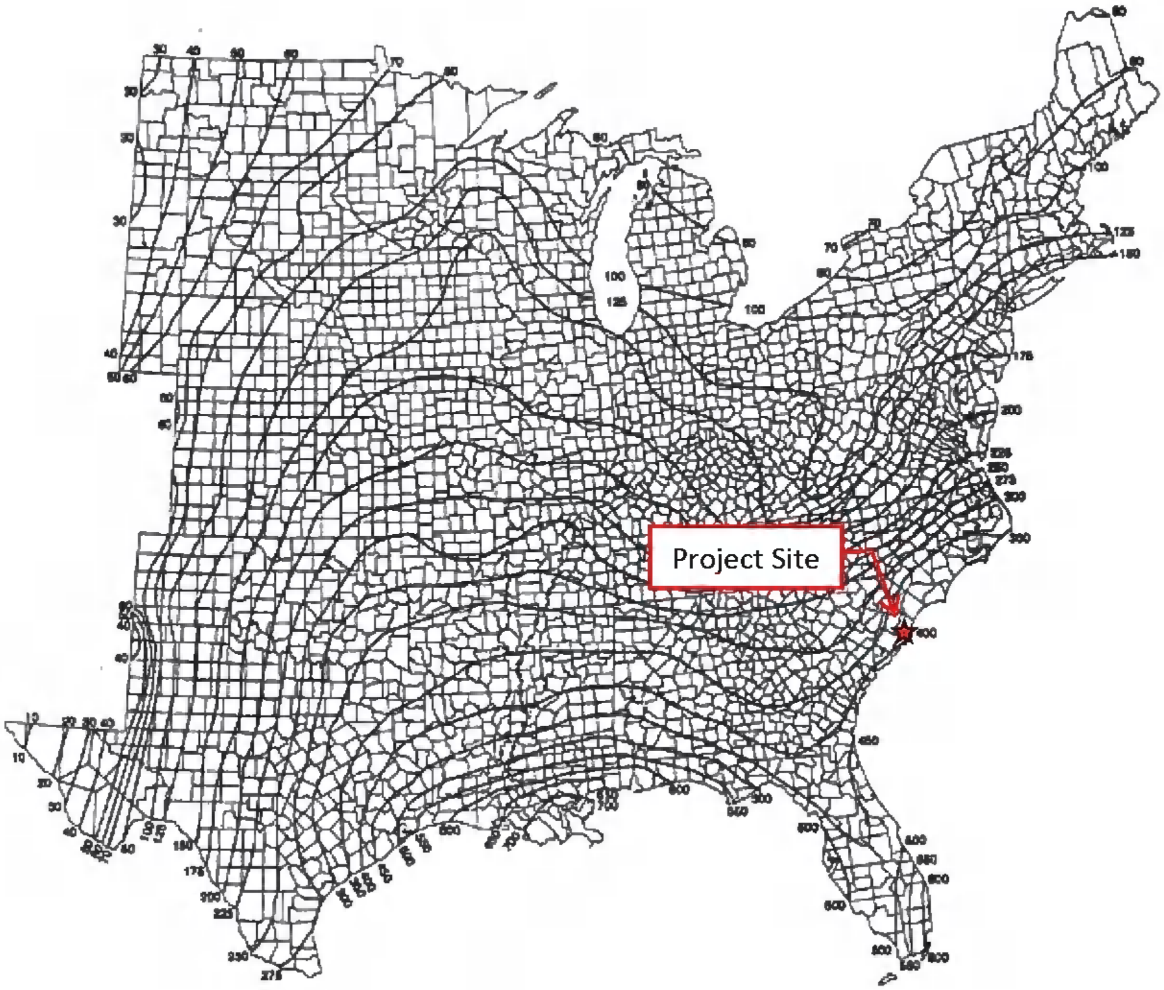
[3/] Portion of total-area surface that would be hidden from view by canopy in a vertical projection, (a bird's-eye view).

[4/] G: Cover at surface is grass, grasslike plants, decaying compacted duff, or litter at least 2 inches deep.

W: Cover at surface is mostly broadleaf herbaceous plants (as weeds with little lateral-root network near the surface), and/or undecayed residue.

FIGURES

- Figure 1. Average Annual Rainfall Runoff Erosivity Factor, R, Isoerodent Map (from USDA, 1997)
- Figure 2. Soil Survey Map (from USDA, 2016)



ft·tonf·in(ac·h·yr)$^{-1}$.

Figure 1. Average Annual Rainfall Runoff Erosivity Factor, R, Isoerodent Map (from USDA, 1997)

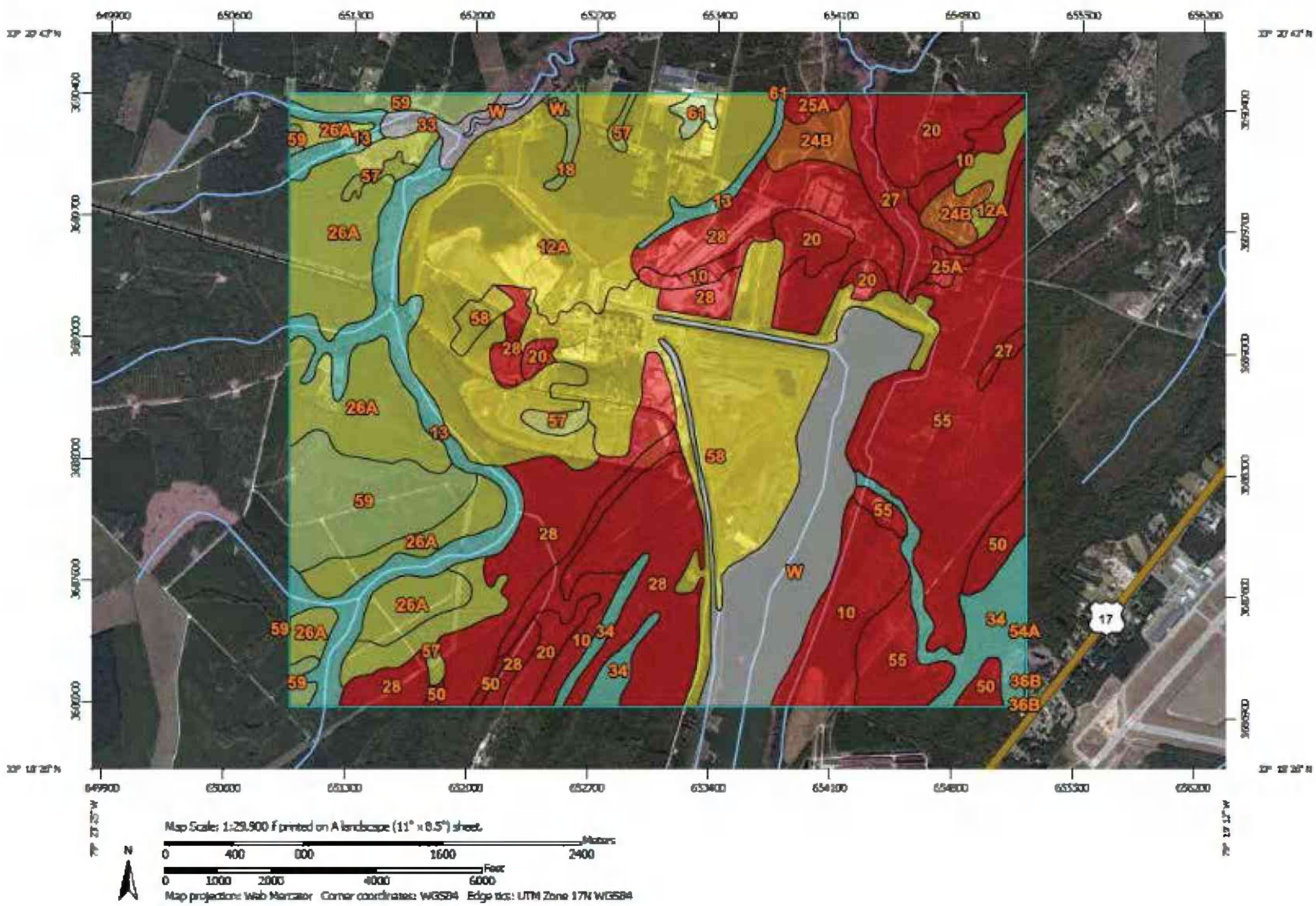


Figure 2. Soil Survey Map (from USDA, 2016)

APPENDIX F-5

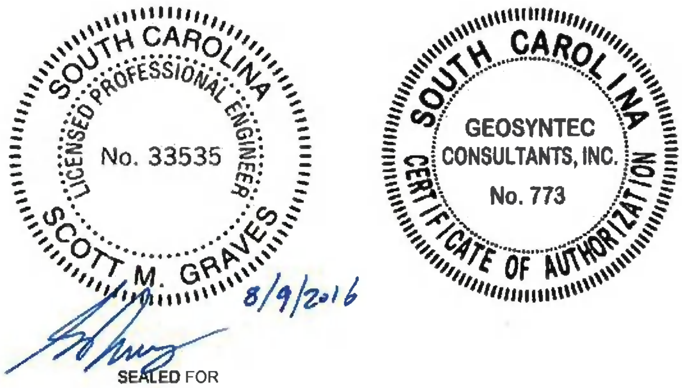
ACTIVE AREA RUN-ON CONTROL DESIGN

Written by: **A. Sivashanthan** Date: **6/15/2016** Reviewed by: **B. Klenzendorf** Date: **6/17/2016**

Client: **Santee Cooper** Project: **Winyah Generating Station** Project No.: **GSC5242** Phase No.: **01**

APPENDIX F-5

ACTIVE AREA RUN-ON CONTROL DESIGN



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CALCULATION PAGES F5-1
THROUGH F5-13

1 INTRODUCTION

The purpose of this calculation package is to present the analysis for the sizing of the temporary diversion berms to be utilized as necessary at the active face (i.e., diversions around areas of exposed waste) during development of the proposed Class Three Landfill at the Winyah Generating Station (WGS) located in Georgetown County, South Carolina.

Diversion berms are temporary soil berms that will be constructed as necessary up-gradient from the active working face (exposed waste areas) to intercept storm water runoff flow before it comes in contact with waste. If these temporary diversion berms are used, they will route the non-contact (clean) storm water around active areas into the surface water management system and away from the active face. To provide operator flexibility to adapt to differing conditions, rather than provide just one required design berm size, this calculation package provides the sizing needed for a variety of cases. These are intended to capture the expected range of operational conditions, while providing operational flexibility to choose the appropriate minimum berm size based on the conditions that exist up-gradient from each specific active area at any point in time. As such, for these analyses, the maximum up-gradient drainage area which can be managed by each given diversion berm size and for the 25-year rainfall event is calculated.

2 ASSUMPTIONS AND PROCEDURES

The section discusses the assumptions and procedures for the design of the temporary diversion berms.

It is assumed that temporary diversion berms will be installed with flow line (longitudinal) slopes ranging from 0.5% to 2%. Temporary diversion berms will be placed as needed up-gradient from the active working face. The temporary diversion berms are assumed to be "tack-on" berms with a 2.5:1 side slope (see Figure 1 of this calculation package) to form a v-shaped channel. A channel depth of 2.0 feet was assumed (i.e., this is a fixed parameter of these calculations).

The Rational Method described in the Georgetown County *Storm Water Management Design Manual* (2006) is used to calculate the peak surface water discharge (this is the specified method for drainage areas of 20 acres or less). This approach is considered appropriate given this local recommendation, and because it is expected that drainage areas to a given diversion berm will be on the order of 20 acres or less. The channels were sized assuming they are flowing full, which is considered adequate since they are interior and temporary site features, and given other conservative selections of parameters as documented herein. The following steps were utilized to calculate the drainage areas that each diversion berm can accommodate.

1. Compute the discharge capacity of diversion berms with 0.5%, 1%, 1.5%, and 2% slopes using Manning's Equation for open channel flow.
2. Apply the Rational Method to compute the up-gradient drainage area that would produce the discharge capacity calculated in Step 1.

Manning's equation was used to estimate the peak discharge capacity of the v-shaped channel created by a temporary diversion berm. Manning's equation (Chow, 1959) is expressed as:

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Client: **Santee Cooper** Project: **Winyah Generating Station** Project No.: **GSC5242** Phase No.: **01**

$$Q = \frac{1.49}{n} A R^{2/3} S^{1/2} \tag{1}$$

where:

Q = discharge (cfs),

n = Manning's roughness coefficient,

A = area of cross-section of flow (ft^2),

P = wetted perimeter (ft),

R = hydraulic radius = A/P (ft), and

S = longitudinal slope (ft/ft).

The peak discharge from the contributing drainage area by the Rational Method (Georgetown County, 2006) can be computed by:

$$Q = C \times C_f \times i \times A \tag{2}$$

where:

Q = peak design discharge (cfs),

C = runoff coefficient (dimensionless),

C_f = frequency factor based on recurrence Interval

i = design rainfall intensity (in/hr), and

A = drainage area (acres).

The design rainfall intensity in Equation (2) is calculated using guidance in the Georgetown County *Storm Water Design Manual* (2006). The manual estimates the design rainfall intensity as shown in Equation (3):

$$i = \frac{a}{(b+t_c)^c} \tag{3}$$

where:

i = design rainfall intensity (in/hr),

t_c = time of concentration (minutes [min]), and

a, b, and c = coefficients for specific return period.

Equation (2) is rearranged, and the watershed drainage area was back-calculated for each potential flow line slope of a temporary diversion berm.

3 DESIGN PARAMETERS

This section discusses the justification behind the selected design parameters for the temporary diversion berms.

The Manning's roughness coefficient (*n*) for the diversion berm was selected as 0.02 for unlined, bare soil channels with flow depths larger than 0.5 ft, as shown in Table 1 (Georgetown County, 2006). The peak discharge flowing to the channel is calculated using the Rational Method.

A runoff coefficient of C = 0.60 was selected based on information provided by the *Storm Water Management Design Manual* (Georgetown County, 2006) for graded areas with no plant cover and clayey soils having an average slope of 5% to 10% as shown in Table 2. A runoff coefficient frequency factor of C_f = 1.1 was selected based on information provided by the *Storm Water Management Design Manual* (Georgetown County, 2006), as shown in Table 3.

For a conservative design approach, a minimum time of concentration of 5 minutes was used to calculate the rainfall intensity by Equation (3). Because the drainage areas for this analysis are much smaller, a time of concentration of 5 minutes is used.

To calculate the design rainfall intensity using Equation (3), the coefficients *a*, *b*, and *c* were selected for a 25-year return period from Table 4 as 288.87, 29.41, and 0.996 respectively (Georgetown County, 2006). The corresponding rainfall intensity for a time of concentration of 5 minutes was selected as 8.51 in/hr for a 25-year storm event.

4 RESULTS

The results of the temporary diversion berm calculations are summarized in Table 5 for each assumed flow line slope. The calculated drainage areas represent the maximum drainage area that each temporary diversion berm configuration can accommodate for the 25-year design rainfall event. It should be noted that if, during operations, a larger area than those calculated in Table 5 will be draining towards the active face, multiple diversion berms may be constructed to comply with the drainage area requirements presented herein for the given berm height and the selected flow line slope.

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Client: **Santee Cooper** Project: **Winyah Generating Station** Project No.: **GSC5242** Phase No.: **01**

5 REFERENCES

Chow, V.T. (1959), *Open Channel-Hydraulics,* McGraw-Hill.

Georgetown County (2006), *Storm Water Management Design Manual*, Georgetown County, South Carolina, revised 14 November 2006.

TABLES

- Table 1. Manning's Roughness Coefficients for Artificial Channels (from Georgetown County, 2006)
- Table 2. Runoff Coefficients for Rational Method (from Georgetown County, 2006)
- Table 3. Rational Method Runoff Coefficient Frequency Factors (from Georgetown County, 2006)
- Table 4. Rainfall Intensity Coefficient for Georgetown County (from Georgetown County, 2006)
- Table 5. Diversion Berm Drainage Area Sizing

Table 1. Manning's Roughness Coefficients for Artificial Channels (from Georgetown County, 2006)

Lining Category	Lining Type	"n" at various flow depths		
		0 - 0.5 ft	0.5 - 2.0 ft	>2.0 ft
Rigid	Concrete	0.015	0.013	0.013
	Grouted Riprap	0.040	0.030	0.028
	Stone Masonry	0.042	0.032	0.030
	Soil Cement	0.025	0.022	0.020
	Asphalt	0.018	0.016	0.016
Unlined	Bare Soil	0.023	0.020	0.020
	Rock Cut	0.045	0.035	0.025
Temporary[1]	Woven Paper Net	0.016	0.015	0.015
	Jute Net	0.028	0.022	0.019
	Fiberglass Roving	0.028	0.022	0.019
	Straw with Net	0.065	0.033	0.025
	Curled Wood Mat	0.066	0.035	0.028
	Synthetic Mat	0.036	0.025	0.021
Gravel	1-inch D_{50}	0.044	0.033	0.030
	2-inch D_{50}	0.066	0.041	0.034
Rock Riprap	6-inch D_{50}	0.104	0.069	0.035
	12-inch D_{50}	---	0.078	0.040

Source: Federal Highway Administration. Design of Roadside Channels with Flexible Linings, HEC-15, 1988.
Note: Values listed are representative values for the respective depth ranges. Manning's "n" varies with the flow depth.
[1]Some "temporary" linings become permanent when buried.

Table 2. Runoff Coefficients for Rational Method

(from Georgetown County, 2006)

Description of Area	Runoff Coefficients "C"
Lawns:	
Sandy soil, flat, 2%	0.10
Sandy soil, average, 2 - 7%	0.15
Sandy soil, steep, > 7%	0.20
Clay soil, flat, 2%	0.17
Clay soil, average, 2 - 7%	0.22
Clay soil, steep, > 7%	0.35
Business:	
Downtown areas	0.95
Neighborhood areas	0.70
Residential:	
Single-family areas	0.50
Multi-units, detached	0.60
Multi-units, attached	0.70
Suburban	0.40
Apartment dwelling areas	0.70
Industrial:	
Light areas	0.70
Heavy areas	0.80
Parks and cemeteries	0.25
Playgrounds	0.35
Railroad yard areas	0.40
Unimproved areas (forest)	0.30
Streets:	
Asphalt and Concrete	0.95
Brick	0.85
Drives, walks, and roofs	0.95
Gravel areas	0.50
Graded or no plant cover	
Sandy soil, flat, 0 - 5%	0.30
Sandy soil, flat, 5 - 10%	0.40
Clayey soil, flat, 0 - 5%	0.50
Clayey soil, average, 5 - 10%	0.60

Table 3. Rational Method Runoff Coefficient Frequency Factors
(from Georgetown County, 2006)

Recurrence Interval (years)	Frequency Factor, C_f
25	1.1
50	1.2
100	1.25
Note: The product of C_f times C shall not exceed 1.0.	

Table 4. Rainfall Intensity Coefficient for Georgetown County

(from Georgetown County, 2006)

Frequency (years)	a	b	c	i (in/hr) t_c=5min	i (in/hr) t_c=10min	i (in/hr) t_c=15min
2	249.76	34.10	1.026	5.80	5.13	4.59
5	261.38	32.32	1.015	6.63	5.84	5.21
10	269.35	31.13	1.007	7.26	6.37	5.68
25	288.87	29.41	0.996	8.51	7.43	6.60
50	288.87	28.24	0.989	9.04	7.87	6.97
100	296.41	27.09	0.981	9.86	8.55	7.55
Source: South Carolina Department of Transportation, September 1997						

Table 5. Diversion Berm Drainage Area Sizing

Depth of Channel (ft)	Diversion Berm Flow Line Slope (%)	Maximum Predicted Flow Velocity (ft/s)	Maximum Predicted Flow Rate (cfs)	Maximum Drainage Area (ac)
2.0	0.5%	5.05	55.58	9.9
	1.0%	7.15	78.60	14.0
	1.5%	8.75	96.27	17.1
	2.0%	10.11	111.16	19.8

Note: The back-calculated maximum allowable drainage area for the channel dimensions (geometry and slope) given above, as calculated by the Rational Method, assumes that the channel created by the diversion berm is flowing full when conveying the peak discharge during the 25-year rainfall event and from the maximum contributing drainage area.

FIGURES

- Figure 1. Typical/Schematic (Conceptual) of Active Fill Area Section

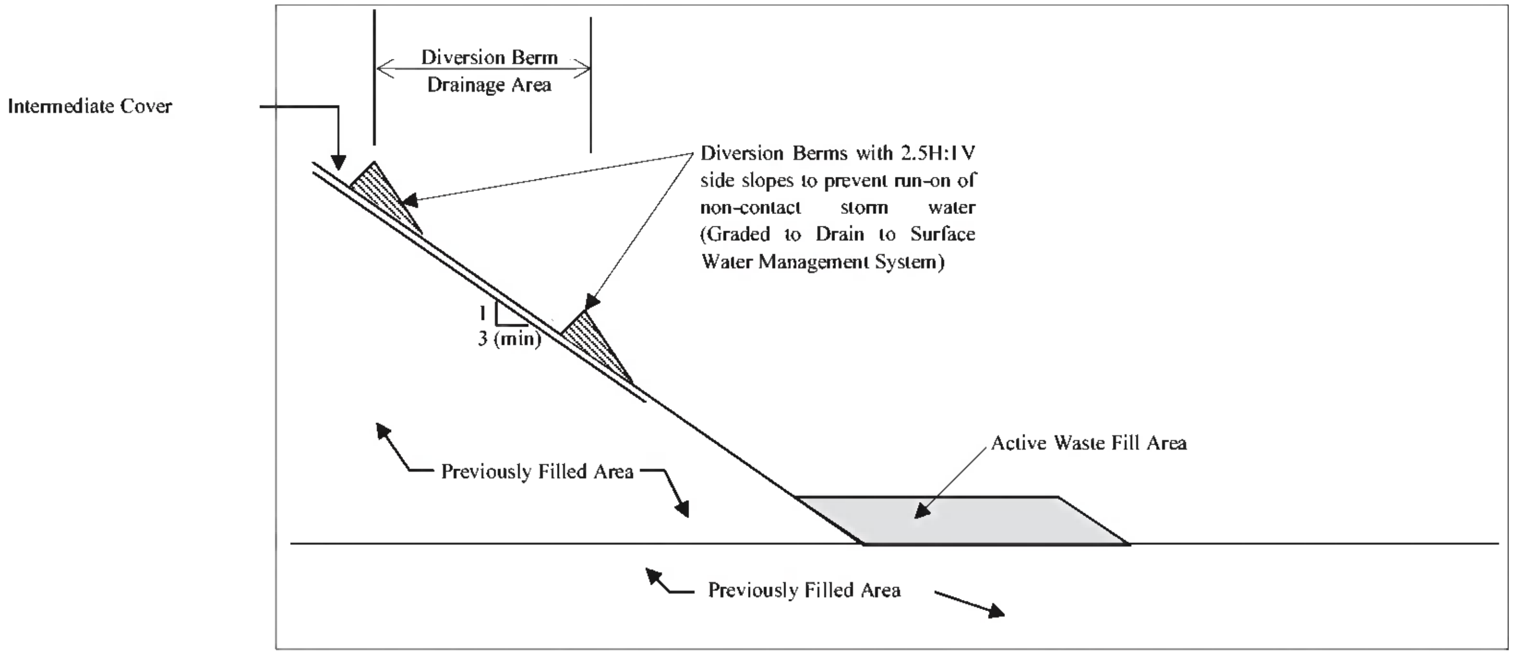

Figure 1. Typical/Schematic (Conceptual) of Active Fill Area Section

(portraying possible conditions where diversion berms may be used) (Not to Scale (NTS))

APPENDIX F-6

SITE WATERSHED ANALYSIS

Written by:	A. Sivashanthan	Date:	7/22/2016	Reviewed by:	B. Klenzendorf	Date:	8/4/2016
Client:	Santee Cooper	Project:	Winyah Generating Station	Project No.:	GSC5242	Phase No.:	01

APPENDIX F-6

SITE WATERSHED ANALYSIS

SOUTH CAROLINA LICENSED PROFESSIONAL ENGINEER No. 33535 SCOTT M. GRAVES 8/9/16

SOUTH CAROLINA GEOSYNTEC CONSULTANTS, INC. No. 773 CERTIFICATE OF AUTHORIZATION

SEALED FOR CALCULATION PAGES F6-1 THROUGH F6-54

1. INTRODUCTION

1.1 Purpose

The purpose of this calculation package is to evaluate the site watershed hydrology under pre-development and post-development conditions, for the portion of the Winyah Generating Station (WGS) site that includes the proposed Class Three Landfill areas (Landfill Area 1 and Landfill Area 2). The objectives of this site watershed hydrologic analysis are as follows:

- evaluate and demonstrate that sufficient capacity will exist in the on-site industrial cooling pond (on-site receiving water body) to contain the design storm event (25-year, 24-hour) following the proposed development of on-site landfill (i.e., Landfill Area 1 and Landfill Area 2); and

- compare the values of peak discharge, runoff volume, and cooling pond peak water surface elevation under post-development conditions to those that exist under pre-development existing conditions in order to demonstrate that the landfill design does not adversely alter, to any significant degree, the total quantity of runoff currently entering the cooling pond and the drainage patterns of the watershed in the vicinity of the cooling pond.

Note that the hydraulic design of the landfill surface water management features is presented in other calculation packages that accompany the Engineering Report.

1.2 Site Watershed Overview

WGS and the surface impoundments on site are located within the Sampit River watershed (ID: 03040207-01). The Sampit River Watershed occupies 105,260 acres in the Lower Coastal Plain and Coastal Zone deposits of South Carolina and consists primarily of the Sampit River and its tributaries, Pennyroyal Creek and Turkey Creek (Geosyntec, 2016). Pennyroyal Creek borders the western boundary of the WGS property and flows in a northeasterly direction. Turkey Creek is located east of the WGS property and flows in a northern direction towards Pennyroyal Creek where the two creeks meet about one-mile north of the WGS property line and one-half mile south of the Sampit River. The industrial cooling pond was formed by constructing a compacted earthen dam within the existing Turkey Creek. A portion of Turkey Creek was subsequently relocated to a man-made channel along the east side of the cooling pond to divert off-site runon areas to the south of the WGS site around the cooling pond.

The following definitions pertain to the two conditions analyzed in this package:

- Pre-Development Conditions – represent the existing conditions at the WGS site for drainage areas contributing runoff to the cooling pond.
- Post-Development Conditions – represent proposed conditions of the site once the landfills have been fully developed, with the final cover and permanent surface water management system installed; all drainage areas contributing runoff to the cooling pond outside of the proposed landfill areas were modeled as pre-development conditions for this scenario.

Figure 1 of this calculation package shows the pre-development drainage areas contributing runoff to the cooling pond.

2. METHODOLOGY

2.1 HEC-HMS Computer Model

Surface water discharges are estimated using the Hydrologic Modeling System (HEC-HMS) computer program Version 4.1 developed through the Hydraulic Engineering Center (HEC) of the United States Army Corps of Engineers (USACE). The program simulates natural and controlled precipitation-runoff and routing processes of a watershed. HEC-HMS is the successor to and replacement for the HEC-1 program (USACE, 2000). For precipitation-runoff-routing simulation, HEC-HMS provides the following components:

- Precipitation-specification options can describe an historical precipitation event, a frequency-based hypothetical precipitation event (i.e., design rainfall or storm event), or an event that represents the upper limit of precipitation possible at a given location. For this analysis, the hypothetical precipitation event used was the 25-year (4% annual chance), 24-hour duration event (herein referred to as the 25-year, 24-hour event).

- Water loss models can estimate the volume of runoff given the precipitation and properties of the watershed. For this analysis, the Soil Conservation Service (SCS) Curve Number Loss Model was used (USDA, 1986).

- Direct runoff transform models can account for overland flow, storage, and energy losses as surface water runs off a watershed and into the drainage channels. For this analysis, the SCS Unit Hydrograph Model was selected.

- Hydraulic routing models account for storage and energy flux as surface water flows through drainage channels. The Kinematic Wave Model was selected for these analyses.

- Hydraulic models of water-control measures such as surface water pond outfall structures.

HEC-HMS was used to model the pre-development conditions and the post-development conditions. More specifically, HEC-HMS modeling calculates surface water runoff volumes, peak flow rates, and flow characteristics for the drainage areas contributing runoff to the cooling pond.

2.2 Estimation of Time of Concentration for HEC-HMS SCS Curve Number Method

The time of concentration is defined as the time for runoff to flow from the most hydraulically remote point of the drainage area to the point under investigation. The time of concentration (T_c) is a summation of sheet flow travel time, shallow concentrated flow travel time, and open channel flow travel time.

The method to estimate the sheet flow travel time was obtained from the U.S. Department of Agriculture (USDA) document *Urban Hydrology for Small Watersheds, Technical Release 55 (TR-55)* (USDA, 1986). Manning's kinematic solution is used for estimating travel time for sheet flow for flow distances less than 300 ft (USDA, 1986):

$$T_t = \frac{0.007(nL)^{0.8}}{P_{2-24}^{\;0.5} S^{0.4}}$$

where: T_t = travel time for overland sheet flow (hr);

n = roughness coefficient for sheet flow;

L = flow length (ft);

P_{2-24} = 2-year, 24-hour rainfall (in.); and

S = slope of hydraulic grade line (or land slope) (ft/ft).

The slope of the hydraulic grade line, or land slope (S), for all subcatchment areas is shown in Appendix 1 of this calculation package for the pre-development scenario. Characteristics for the post-development scenario are provided in Appendix F-1 for the proposed landfills whereas other subcatchment areas (outside the landfill areas) maintain pre-development conditions in the post-development model; the pre-development Unit 2 Slurry Pond and Ash Pond A are proposed to be converted to Area 1 Landfill and Area 2 Landfill, respectively for post-development conditions. To estimate sheet flow travel time (T_t), a roughness coefficient (n) of 0.40 for woods with light underbrush and a roughness coefficient of 0.05 for fallow (no residue) was selected for pre-development conditions as shown in Table 1 (USDA, 1986). Maximum flow lengths (L) were selected for each drainage area based on the existing site topography. As shown in Table 2 (NOAA, 2015), the rainfall depth for the 2-year, 24-hour frequency (P_{2-24}) at the site is 4.42 inches.

The method used to estimate shallow concentrated flow was obtained from the Upland Method (USDA, 1986) using the equation below.

$$V = K_v \sqrt{S}$$

where: V = average velocity (ft/sec),

K_v = shallow concentrated flow velocity factor (ft/sec), and

S = land slope (ft/ft).

The shallow concentrated flow velocity factor (K_v) is selected from Table 3 (HydroCAD, 2011) based on surface type. A velocity factor of K_v = 10.0 ft/sec was selected for developed areas and impoundment areas based on a nearly bare surface description. The land slopes were estimated from the topographic maps of existing conditions.

The method selected to estimate the shallow concentrated flow and open channel flow travel time is based on guidance provided in TR-55 (USDA, 1986). Travel time for shallow concentrated flow and open channel flow is estimated by dividing the longest drainage path by the velocity of runoff:

$$T_t = \frac{L}{V}\left(\frac{1}{60}\right)$$

where: T_t = travel time (min);

L = flow length (ft); and

V = average velocity (ft/sec).

The shallow concentrated flow velocities are defined above and open channel flow velocities were estimated using Manning's equation based on guidance provided in TR-55 (USDA, 1986). The average flow velocities were determined for bank-full elevation as:

$$V = \frac{1.49}{n} R_h^{2/3} S^{1/2}$$

where:

- V = average velocity (ft/s);
- n = Manning's roughness coefficient;
- R_h = hydraulic radius (ft) = A/P;
- A = cross sectional area (ft^2);
- P = wetted perimeter (ft); and
- S = slope of hydraulic grade line (or longitudinal channel slope for normal flow conditions) (ft/ft).

To estimate open channel flow travel time (T_t), a Manning's roughness coefficient (n) value of 0.028 was selected to represent a dragline-excavated or dredged channel with no vegetation as shown in Table 4 (Chow, 1959).

The velocities and times of concentration used in the design are presented in Appendix 1. A minimum time of concentration of six minutes was used as recommended by TR-55 (USDA, 1986) because small areas with exceedingly short times of concentration could result in design rainfall intensities that are unrealistically high. The lag times calculated for each drainage area are presented in Appendix 1 for use in the SCS Curve Number Method and HEC-HMS software. The lag time is estimated as 0.6 times the time of concentration (USDA, 1986).

3. DESIGN PARAMETERS

The following data and assumptions were utilized in selecting engineering parameters to estimate surface water runoff.

3.1 Rainfall

- The rainfall depths corresponding to 24-hour duration hypothetical precipitation event and 25-year frequency return periods for the site is 8.28 inches as shown in Table 2 (NOAA, 2015). The design storm hyetograph is defined using a SCS Type III rainfall distribution, which is selected based on Figure 2 (USDA, 1986).

3.2 Drainage Areas and Reaches

- Drainage Areas – The contributing site watershed for the pre-development model is divided into multiple subbasins (subcatchments). Subbasins are modeled based on the management of on-site surface water drainage and are delineated for the following areas: developed area (i.e., WGS Plant Area) and existing surface impoundments areas (i.e., Ash Pond A, Ash Pond B, Slurry Pond 3 and 4 plus West Ash Pond, South Ash Pond, and Unit 2 Slurry Pond). The SCS Curve Number Loss Model was used to estimate the volume of runoff from a given subbasin. The SCS Unit Hydrograph Model was used to estimate the direct runoff flow rates from each subbasin. Each subbasin is assigned a curve number representing the type of ground cover for a given soil for the area. The subbasin area, curve number, and SCS Unit Hydrograph lag time input parameters associated with pre-development conditions are included in the HEC-HMS output in Appendix 1.

- Hydrologic Soil Groups (HSGs) – Figure 3 shows a soil map of the WGS site area, taken from the Web Soil Survey tool operated by the USDA Natural Resources Conservation Service (USDA, 2016) for Georgetown County. The predominant soil type at the site are Udorthents, loamy (map unit symbol 58) and Eulonia loamy fine sand (map unit symbol 26A). The on-site soil types have HSG designations as shown in Table 5 (USDA, 2016). To be conservative, both pre-development and post-development soils are assumed to be a HSG type D soil, which generally provides the highest calculated runoff volumes.

- Curve Number (CN) – Curve numbers are obtained from the TR-55 (USDA, 1986) and are based on the predominant HSG of the drainage area. Table 6 summarizes the CNs chosen for the analysis performed for pre-development conditions documented within this calculation package. A composite weighted CN value was calculated for drainage areas with surface water ponds. Table 7 summarizes the area of each cover type selected to calculate the composite CN value for the pre-development subbasins. Areas covered in standing water or exposed geomembrane are assumed to have a large CN value in order to convert the majority of the rainfall into runoff; a CN value of 98 which corresponds to impervious cover (paved areas) was selected for these areas.

3.3 Pre-Development Condition

Figure 1 delineates existing drainage areas contributing runoff to the cooling pond for pre-development conditions. Existing topographic information was compiled via LIDAR survey data (obtained from the Georgetown County, South Carolina) collected in 2005. The pre-development drainage area of 1,257 acres includes the proposed 120-acre landfill footprints as well as additional areas draining to the cooling pond. The total drainage area for pre-development conditions matches the post-development drainage area which allows for direct comparison between the two scenarios.

The Ash Pond A drainage area of approximately 83 acres is bounded by the intake canal to the north, the cooling pond to the east, Ash Pond B to the south, and the discharge canal to the west. Ash Pond A currently receives low volume wastewater from the existing coal-fired electric generating units and contact wastewater from Unit 2 Slurry Pond via a pump with a maximum capacity of 5.79 cfs. Ash Ponds A and B are hydraulically connected through a 30-inch diameter corrugated metal pipe (CMP), a 48-inch diameter smooth steel pipe, and a 24- inch diameter HDPE pipe, and an emergency spillway at elevation 37 ft (Thomas and Hutton, 2016; Thomas and Hutton, 2012). The Ash Pond A area will be converted to the Area 2 Landfill for post-development conditions.

The Unit 2 Slurry Pond drainage area of approximately 37 acres is bounded by the intake canal to the south, the WGS industrial plant to the west, and undeveloped private properties to the north and east. The Unit 2 Slurry Pond discharges to Ash Pond A via a pump with a maximum capacity of 5.79 cfs. The Unity 2 Slurry Pond area will be converted to the Area 1 Landfill for post-development conditions.

The Ash Pond B drainage area of approximately 74 acres is bounded by Ash Pond A to the north, the cooling pond to the east and south, and the discharge canal to the west. A 24-inch diameter smooth interior, corrugated exterior high density polyethylene pipe culvert conveys water from the riser structure to the discharge canal of the cooling pond (Santee Cooper, 2012; Thomas and Hutton, 2016).

The South Ash Pond drainage area of approximately 76 acres is bounded by the railroad line and coal pile to the north, the discharge canal to the east, and a forested area to the south and west. The South Ash Pond currently receives low volume wastewater (from Units 3 and 4) and other process water inflows (from Unit 3 and 4 fly ash sluice and blowdown from the SEFA Star Facility). Water is discharged from the South Ash Pond through a riser structure and outlet pipe to the discharge canal (Lockwood Greene, 1978).

The Slurry Pond 3 & 4 plus West Ash Pond drainage area of approximately 172 acres is bounded by Pennyroyal Creek to the west, the WGS industrial plant to the east, undeveloped land and the South Ash Pond to the south, and undeveloped land to the north. Slurry Pond 3 & 4 is separated from the West Ash Pond by a dike. Slurry Pond 3 & 4 receives process water and stormwater from the plant areas. Slurry Pond 3 & 4 discharges to the discharge canal via a two pumps with a combined maximum capacity of 3,075 gpm. Slurry Pond 3 & 4 has additional pump stations which discharge to the discharge canal during normal operating conditions but become inundated during larger rainfall events (i.e., the 100-year, 24-hour storm event). Slurry Pond 3 & 4 is designed to contain stormwater runoff for a 25-year, 24-hour storm event; rainfall events greater than the 25-year, 24-hour event will discharge via an emergency spillway to Pennyroyal Creek and the two pumps to the discharge canal. For this analysis, only the discharges to the discharge canal were considered. Therefore, the two pumps that are not inundated during extreme events were the only flows considered from the Slurry Pond 3 & 4. This conservatively estimates the flow from Slurry Pond 3 & 4 during extreme events as additional flow will discharge via the emergency spillway to Pennyroyal Creek. Portions of the coal pile drain to a pump station that can route water to Slurry Pond 3 & 4; however, for the purposes of this analysis, it was assumed that these pumps are not operational during extreme rainfall events and runoff from the coal pile flows directly into the discharge canal.

The West Ash Pond is approximately 64 acres (of the total 172 acres of the drainage area) and is located immediately south of Slurry Pond 3 & 4. The West Ash Pond gravity drains to Slurry Pond 3 & 4 through two 36 inch diameter culverts, four 22 inch diameter culverts, and an emergency spillway. The West Ash Pond and Slurry Pond 3 & 4 were modeled as a single drainage area and the hydraulic connection between the two ponds was not considered since the ultimate discharge to the cooling pond is controlled by the two pumps.

The developed drainage area of approximately 266 acres consists of the WGS power blocks, paved roadways, coal pile area, and associated impervious cover and infrastructure. The developed area drains directly into the intake canal or discharge canal. On-site stormwater infrastructure, such as smaller ponds, were not explicitly considered in this analysis; these ponds were conservatively considered to be completely full for modeling purposes.

The undeveloped area located in the southern portion of the WGS property to the west of the cooling pond and south of the South Ash Pond was not considered in this analysis. The undeveloped area consists of a combination of grass-covered land and wetland areas which drain to infrastructure immediately west of the cooling pond, in the form of long pond areas that intercept runoff from the undeveloped area and route it to the south and around the cooling pond via the Turkey Creek bypass channel (immediately east of the cooling pond) according to a Santee Cooper report (Carter, 1995). All of the undeveloped area was assumed to drain to the Turkey Creek bypass channel.

The cooling pond drainage area of approximately 550 acres is conservatively assumed to be entirely standing water. The outlet structure for the cooling pond consists of an emergency spillway at elevation 20.93 ft. An existing concrete riser structure is also located in the cooling pond but has been removed from operation and was not considered in this analysis; the emergency spillway was modeled in this analysis as the only outlet structure for the cooling pond. The elevation-storage curve for the cooling pond in Appendix 1 was derived from the Santee Cooper report (Carter, 1995).

The all the pre-development drainage areas drain directly to either the cooling pond or the intake canal or discharge canal which are hydraulically connected to the cooling pond.

3.4 Post-Development Condition

Figure 1 and Figure 2 of Appendix F-1 shows the delineation of drainage areas contributing to each drainage feature. Also, the final configuration of the proposed surface water management system design for the landfill is shown on the Engineering Drawings that accompany the permit application. The proposed surface water management system will utilize drainage terraces, downdrain pipes, perimeter drainage channels, and culverts to control surface water runoff from the site. The facility area associated with the proposed landfills (i.e., post-development conditions) is approximately 120.2 acres (37.2 acres for Area 1 Landfill and 83.0 acres for Area 2 Landfill). All contributing areas draining to the cooling pond outside of the landfill areas remain unchanged for post-development conditions.

3.5 Industrial Cooling Pond

The on-site cooling pond is incorporated in the pre-development and post-development condition analyses. The cooling pond is analyzed to verify containment of the 25-year, 24-

hour rainfall event for both pre-development and post-development runoff. The cooling pond is accounted for in the HEC-HMS program as "reservoir" node. The elevation-storage relationship is input for the cooling pond to describe the volume of storage provided by the cooling pond, which is computed based on the pond geometry (Carter, 1995). WGS National Pollutant Discharge Elimination System (NPDES) permit (Permit No. SC0022471) includes two outfalls for the cooling pond: Outfall 001 discharges to Turkey Creek and Outfall 002 discharges to the North Santee River. Outfall 001 consists of the emergency spillway, whereas Outfall 002 is operated by a pump station. For the purposes of the site watershed analysis, only Outfall 001 is utilized and the pump station at Outfall 002 was not considered. Input and output files for the site watershed analysis are provided in Appendix 1.

3.6 Nodal Network Diagrams

Nodal network diagrams used in HEC-HMS for the site watershed analysis are discussed below and correspond to the output files included in Appendix 1.

- Pre-Development Nodal Network – Figure 4 of this calculation package presents the nodal network drawing for the pre-development conditions. The pre-development nodal network diagram shows the subbasins and discharge locations on Figure 1.
- Post-Development Nodal Network – Figure 5 presents the nodal network drawing for the post-development conditions. The post-development nodal network diagram shows the landfill subbasins, reaches, and discharge locations together with the pre-development areas outside of the landfill areas draining to the cooling pond.

4. RESULTS

A summary of the results are presented in Table 8 of this calculation package, and detailed modeling results are in the appendix included with this calculation package. Table 8 summarizes analysis results for the pre- and post-development peak discharges and total runoff volumes from the site, as well as water surface elevations in the cooling pond. Inspection of Table 8 reveals that both pre-development and post-development flows result in zero-discharge from the cooling pond for the 25-year, 24-hour event.

Thus, the cooling pond is adequate to retain the runoff from the contributing site watershed (including developed landfill areas), and further; the landfill is not anticipated to adversely affect or significantly alter the drainage patterns in the vicinity of the site.

5. REFERENCES

Carter, M. (1995). *Report on the Proposed Improvements to the Winyah Generating Station Cooling Pond*, Inter-Office Communication, Santee Cooper Geotechnical Services, 21 December 1995.

Chow, V.T. (1959). *Open-Channel Hydraulics*, McGraw-Hill Book Company, Inc., New York, NY.

HydroCAD (2011). *HydroCAD Stormwater Modeling System Version 10 Owner's Manual*, HydroCAD Software Solutions, Chocorua, NH.

Lockwood Greene. (1978). *South Carolina Public Service Authority - Georgetown Generating Station.*

Georgetown County (2000). *Georgetown County Storm Water Design Manual*, adopted 5 September 2000.

Geosyntec Consultants, Inc., (2016). *Site Hydrogeologic Characterization Study Report*, Winyah Generating Station, Georgetown, South Carolina, Professional Geologist of Record: Matthew Wissler, dated 29 April 2016, as revised.

NOAA (2015). *Precipitation-Frequency Atlas of the United States.* Atlas 14, Volume 2, Version 3.0, National Oceanic and Atmospheric Administration.

Santee Cooper. (2012). *Inter-Office Communication - WGS Ash Pond B - Abandon Existing Drawdown Structure.* dated 2 March 2012.

Thomas and Hutton. (2012). *Topographic Survey of a Portion of Santee Cooper Winyah Generating Station.* revised 2012.

Thomas and Hutton. (2016). *Topographic Survey of the Dike Crests at Santee Cooper Winyah Generating Station.*

USDA (2016). Web Soil Survey, Soil Survey Staff, Natural Resources Conservation Service (NRCS), United States Department of Agriculture, available online at http://websoilsurvey.*nrcs.usda.gov*, accessed June 2016.

USACE (2000). *Hydraulic Modeling System HEC-HMS Technical Reference Manual*, US Army Corps of Engineers, Hydrologic Engineering Center, CPD-74B, March 2000.

USDA (1986). *Urban Hydrology for Small Watersheds*, Technical Release 55 (TR-55), United States Department of Agriculture, Science and Education Administration, Agriculture Handbook Number 537.

TABLES

- Table 1. Roughness Coefficient for Sheet Flow (from USDA, 1986)
- Table 2. Precipitation Frequency for Georgetown, South Carolina (from NOAA, 2015)
- Table 3. Upland Method Velocity Factors for Shallow Concentrated Flow (from HydroCAD, 2011)
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Table 1. Roughness Coefficient for Sheet Flow

(from USDA, 1986)

Surface description	n [1]
Smooth surfaces (concrete, asphalt, gravel, or bare soil)	0.011
Fallow (no residue)	0.05
Cultivated soils:	
Residue cover ≤20%	0.06
Residue cover >20%	0.17
Grass:	
Short grass prairie	0.15
Dense grasses [2]	0.24
Bermudagrass	0.41
Range (natural)	0.13
Woods: [3]	
Light underbrush	0.40
Dense underbrush	0.80

1 The n values are a composite of information compiled by Engman (1986).

2 Includes species such as weeping lovegrass, bluegrass, buffalo grass, blue grama grass, and native grass mixtures.

3 When selecting n , consider cover to a height of about 0.1 ft. This is the only part of the plant cover that will obstruct sheet flow.

Table 2. Precipitation Frequency for Georgetown, South Carolina (from NOAA, 2015)

NOAA Atlas 14, Volume 2, Version 3
Location name: Georgetown, South Carolina, US*
Latitude: 33.3343°, Longitude: -79.3551°
Elevation: 26 ft*
* source: Google Maps

POINT PRECIPITATION FREQUENCY ESTIMATES

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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)[1]

Duration	Average recurrence interval (years)									
	1	2	5	10	25	50	100	200	500	1000
5-min	0.502 (0.468-0.543)	0.586 (0.545-0.636)	0.668 (0.619-0.722	0.761 (0.703-0.823)	0.857 (0.789-0.927)	0.942 (0.863-1.02)	1.02 (0.929-1.10)	1.10 (0.993-1.19)	1.19 (1.07-1.29)	1.28 (1.14-1.39)
10-min	0.803 (0.748-0.868)	0.938 (0.872-1.02)	1.07 (0.992-1.16)	1.22 (1.13-1.32)	1.37 (1.26-1.48)	1.50 (1.38-1.62)	1.62 (1.48-1.75)	1.74 (1.57-1.88)	1.89 (1.69-2.04)	2.02 (1.79-2.19)
15-min	1.00 (0.935-1.09)	1.18 (1.10-1.28)	1.35 (1.25-1.46)	1.54 (1.42-1.67)	1.73 (1.59-1.87)	1.90 (1.74-2.05)	2.05 (1.87-2.22)	2.19 (1.99-2.37)	2.37 (2.13-2.57)	2.53 (2.25-2.75)
30-min	1.38 (1.28-1.49)	1.63 (1.51-1.77)	1.92 (1.78-2.08)	2.23 (2.06-2.41)	2.57 (2.36-2.77)	2.86 (2.62-3.09)	3.14 (2.86-3.39)	3.42 (3.09-3.69)	3.78 (3.38-4.09)	4.10 (3.64-4.45)
60-min	1.72 (1.60-1.85)	2.04 (1.90-2.22)	2.46 (2.29-2.67)	2.90 (2.68-3.14)	3.42 (3.14-3.69)	3.88 (3.55-4.19)	4.32 (3.94-4.67)	4.79 (4.34-5.18)	5.42 (4.86-5.87)	5.98 (5.32-6.50)
2-hr	2.08 (1.92-2.24)	2.49 (2.30-2.69)	3.07 (2.83-3.31)	3.66 (3.37-3.96)	4.37 (4.00-4.72)	5.01 (4.56-5.40)	5.63 (5.10-6.07)	6.27 (5.64-6.76)	7.10 (6.33-7.68)	7.85 (6.93-8.51)
3-hr	2.23 (2.06-2.42)	2.66 (2.45-2.91)	3.30 (3.03-3.59)	3.97 (3.64-4.33)	4.79 (4.37-5.21)	5.55 (5.02-6.03)	6.31 (5.66-6.84)	7.10 (6.33-7.71)	8.19 (7.20-8.91)	9.18 (7.98-10.0)
6-hr	2.66 (2.44-2.91)	3.19 (2.91-3.49)	3.94 (3.60-4.32)	4.75 (4.33-5.20)	5.76 (5.21-6.30)	6.69 (6.01-7.31)	7.61 (6.79-8.32)	8.62 (7.62-9.41)	9.97 (8.71-10.9)	11.2 (9.69-12.3)
12-hr	3.11 (2.84-3.44)	3.72 (3.39-4.11)	4.63 (4.21-5.12)	5.62 (5.10-6.20)	6.85 (6.17-7.54)	8.00 (7.15-8.78)	9.17 (8.12-10.0)	10.4 (9.15-11.4)	12.2 (10.5-13.4)	13.8 (11.8-15.2)
24-hr	3.64 (3.33-3.99)	4.42 (4.06-4.85)	5.71 (5.22-6.26)	6.77 (6.17-7.41)	8.28 (7.50-9.05)	9.53 (8.59-10.4)	10.9 (9.74-11.9)	12.3 (11.0-13.5)	14.4 (12.7-15.8)	16.1 (14.1-17.7)

Table 3. Upland Method Velocity Factors for Shallow Concentrated Flow (from HydroCAD, 2011)

Surface Description	K_v [ft/sec]	K_v [m/sec]
Paved	20.33	6.2
Unpaved	16.13	4.92
Grassed Waterway	15.0	4.57
Nearly Bare & Untilled	10.0	3.05
Cultivated Straight Rows	9.0	2.74
Short Grass Pasture	7.0	2.13
Woodland	5.0	1.52
Forest w/Heavy Litter	2.5	0.76

Table 4. Manning's n Values for Open Channels (from Chow, 1959)

Type of channel and description	Minimum	Normal	Maximum
C. EXCAVATED OR DREDGED			
a. Earth, straight and uniform			
1. Clean, recently completed	0.016	0.018	0.020
2. Clean, after weathering	0.018	0.022	0.025
3. Gravel, uniform section, clean	0.022	0.025	0.030
4. With short grass, few weeds	0.022	0.027	0.033
b. Earth, winding and sluggish			
1. No vegetation	0.023	0.025	0.030
2. Grass, some weeds	0.025	0.030	0.033
3. Dense weeds or aquatic plants in deep channels	0.030	0.035	0.040
4. Earth bottom and rubble sides	0.028	0.030	0.035
5. Stony bottom and weedy banks	0.025	0.035	0.040
6. Cobble bottom and clean sides	0.030	0.040	0.050
c. Dragline-excavated or dredged			
1. No vegetation	0.025	0.028	0.033
2. Light brush on banks	0.035	0.050	0.060
d. Rock cuts			
1. Smooth and uniform	0.025	0.035	0.040
2. Jagged and irregular	0.035	0.040	0.050
e. Channels not maintained, weeds and brush uncut			
1. Dense weeds, high as flow depth	0.050	0.080	0.120
2. Clean bottom, brush on sides	0.040	0.050	0.080
3. Same, highest stage of flow	0.045	0.070	0.110
4. Dense brush, high stage	0.080	0.100	0.140

Table 5. Hydrologic Soil Groups for On-Site Soils (from USDA, 2016)

Hydrologic Soil Group— Summary by Map Unit — Georgetown County, South Carolina (SC043)				
Map unit symbol	Map unit name	Rating	Acres in AOI	Percent of AOI
10	Leon sand, 0 to 2 percent slopes	A/D	198.2	4.3%
12A	Yauhannah loamy fine sand, 0 to 2 percent slopes	B/D	638.1	14.0%
13	Bladen loam	C/D	336.6	7.4%
18	Cape Fear loam	C/D	15.1	0.3%
20	Centenary fine sand	A	186.7	4.1%
24B	Chisolm sand, 0 to 4 percent slopes	A	55.0	1.2%
25A	Wakulla sand, 0 to 2 percent slopes	A	36.3	0.8%
26A	Eulonia loamy fine sand, 0 to 2 percent slopes	C/D	551.9	12.1%
27	Rutlege sand	A/D	89.1	2.0%
28	Echaw sand	A	503.0	11.0%
33	Hobonny muck	A/D	55.0	1.2%
34	Johnston loam	A/D	131.5	2.9%
36B	Lakeland fine sand, 0 to 6 percent slopes	A	5.4	0.1%
50	Lynn Haven sand	A/D	42.2	0.9%
54A	Chipley fine sand, 0 to 2 percent slopes	A	4.2	0.1%
55	Witherbee fine sand	A/D	447.4	9.8%
57	Grifton loamy fine sand	B/D	80.2	1.8%
58	Udorthents, loamy	B/D	410.4	9.0%
59	Wahee fine sandy loam	C/D	484.4	10.6%
61	Yemassee loamy fine sand	B/D	39.0	0.9%
W	Water		254.9	5.6%
Totals for Area of Interest			**4,564.7**	**100.0%**

Table 6. Summary of Runoff Curve Numbers (from USDA, 1986)

Table 2-2a Runoff curve numbers for urban areas [1]

Cover description		Curve numbers for hydrologic soil group			
Cover type and hydrologic condition	Average percent impervious area [2]	A	B	C	D
Fully developed urban areas (vegetation established)					
Open space (lawns, parks, golf courses, cemeteries, etc.) [3]:					
Poor condition (grass cover < 50%)		68		86	89
Fair condition (grass cover 50% to 75%)		49	69	79	84
Good condition (grass cover > 75%)		39	61	74	80
Impervious areas:					
Paved parking lots, roofs, driveways, etc. (excluding right-of-way)		98	98	98	98
Streets and roads:					
Paved; curbs and storm sewers (excluding right-of-way)		98	98	98	98
Paved; open ditches (including right-of-way)		83	89	92	93
Gravel (including right-of-way)		76	85	89	91
Dirt (including right-of-way)		72	82	87	89
Western desert urban areas:					
Natural desert landscaping (pervious areas only) [4]		63	77	85	88
Artificial desert landscaping (impervious weed barrier, desert shrub with 1- to 2-inch sand or gravel mulch and basin borders)		96	96	96	96
Urban districts:					
Commercial and business	85	89	92	94	95
Industrial	72	81	88	91	93

Table 7. Composite Curve Numbers

Area Designation	Cover Type 1	Cover Type 1 Area (ac)	Cover Type 1 CN	Cover Type 2	Cover Type 2 Area (ac)	Cover Type 2 CN	Composite CN
Developed	Industrial	265.5	93	N/A	N/A	N/A	93
Unit 2 Slurry Pond	Exposed Geomembrane	37.3	98	N/A	N/A	N/A	98
Slurry Pond 3&4 plus West Ash Pond	Open space with grass cover <50%	96.7	89	Water or Exposed Geomembrane	75.0	98	92.9
South Ash Pond	Open space with grass cover <50%	64.0	89	Water	11.6	98	90.4
Ash Pond A	Open space with grass cover <50%	83.0	89	N/A	N/A	N/A	89
Ash Pond B	Open space with grass cover <50%	67.3	89	Water	7.0	98	89.8
Cooling Pond	Water	549.7	98	N/A	N/A	N/A	98

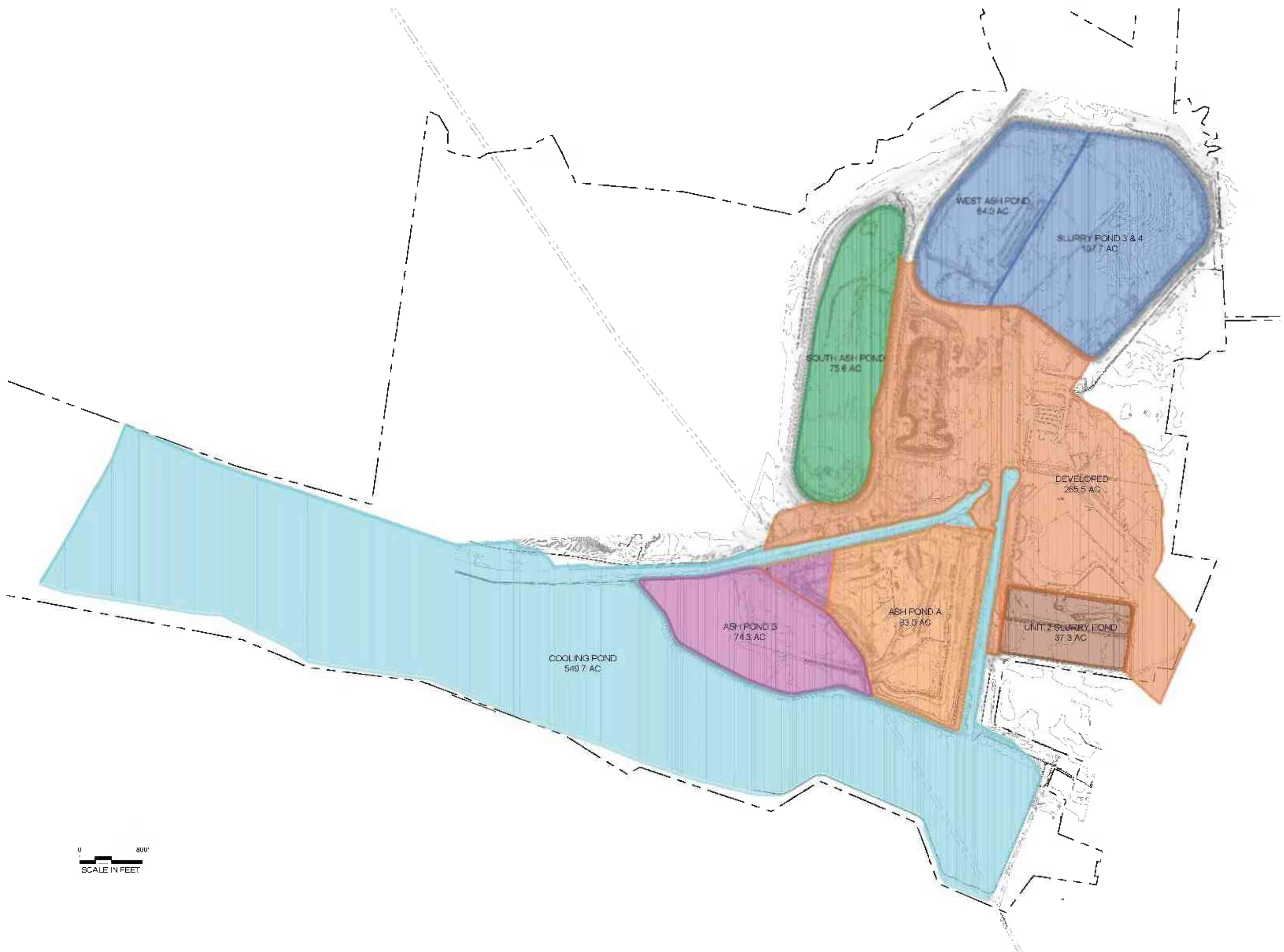
Table 8. Summary of Peak Discharge and Total Discharge Volumes at Cooling Pond Outfall

	Pre-Development 25-Year, 24-Hour Event	Post-Development 25-Year, 24-Hour Event
Cooling Pond Peak Outflow (cfs)	0.0	0.0
Cooling Pond Peak Inflow (cfs)	4,273	4,855
Cooling Pond Inflow Volume (ac-ft)	704.7	691.0
Cooling Pond Water Surface Elevation (ft)	20.9	20.8

FIGURES

- Figure 1. Pre-Development Drainage Plan
- Figure 2. SCS Rainfall Distributions (from USDA, 1986)
- Figure 3. Soil Survey Map
- Figure 5. Pre-Development HEC-HMS Nodal Network
- Figure 6. Post-Development HEC-HMS Nodal Network

Figure 1. Pre-Development Drainage Plan



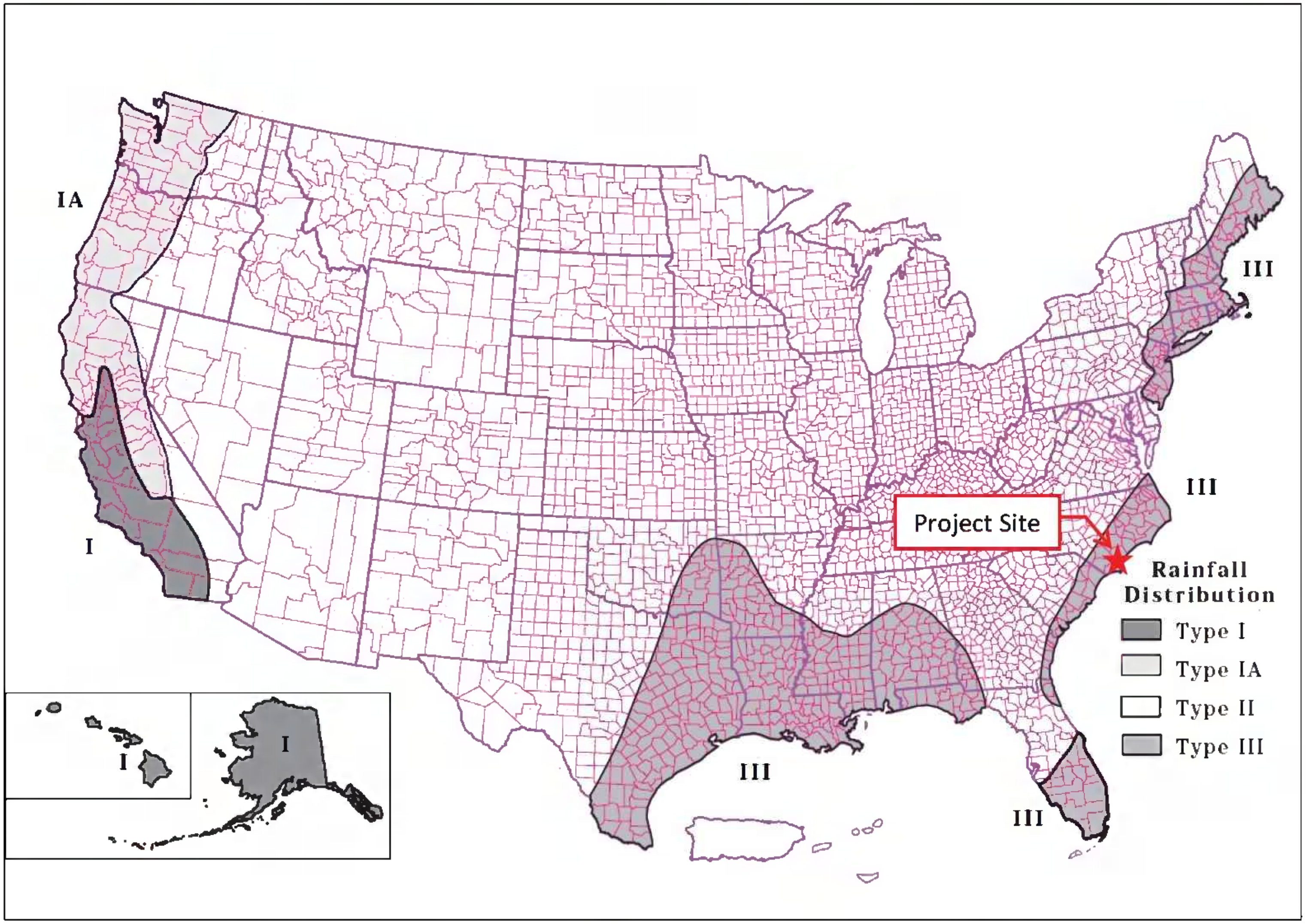


Figure 2. SCS Rainfall Distributions (from USDA, 1986)

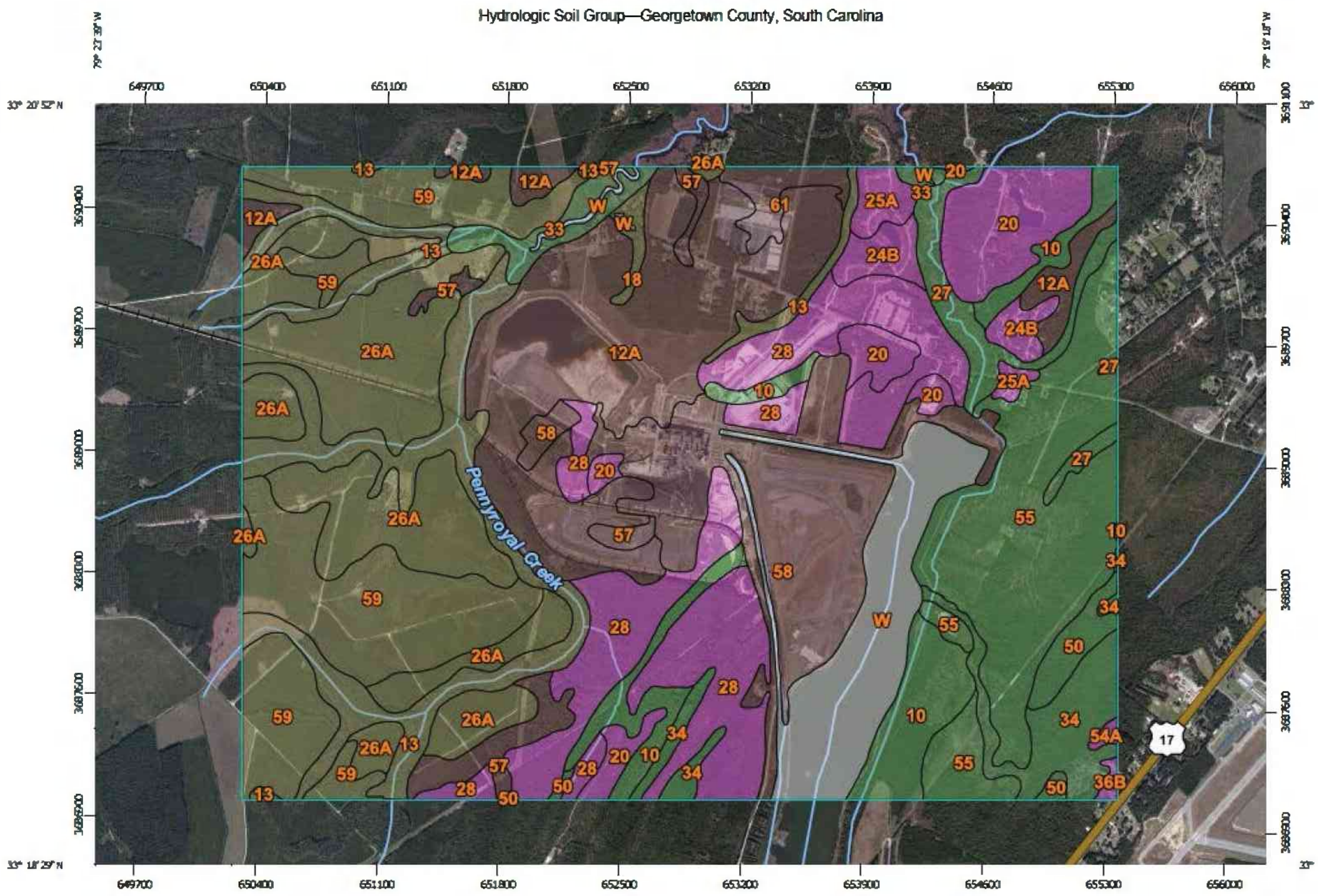


Figure 3. Soil Survey Map

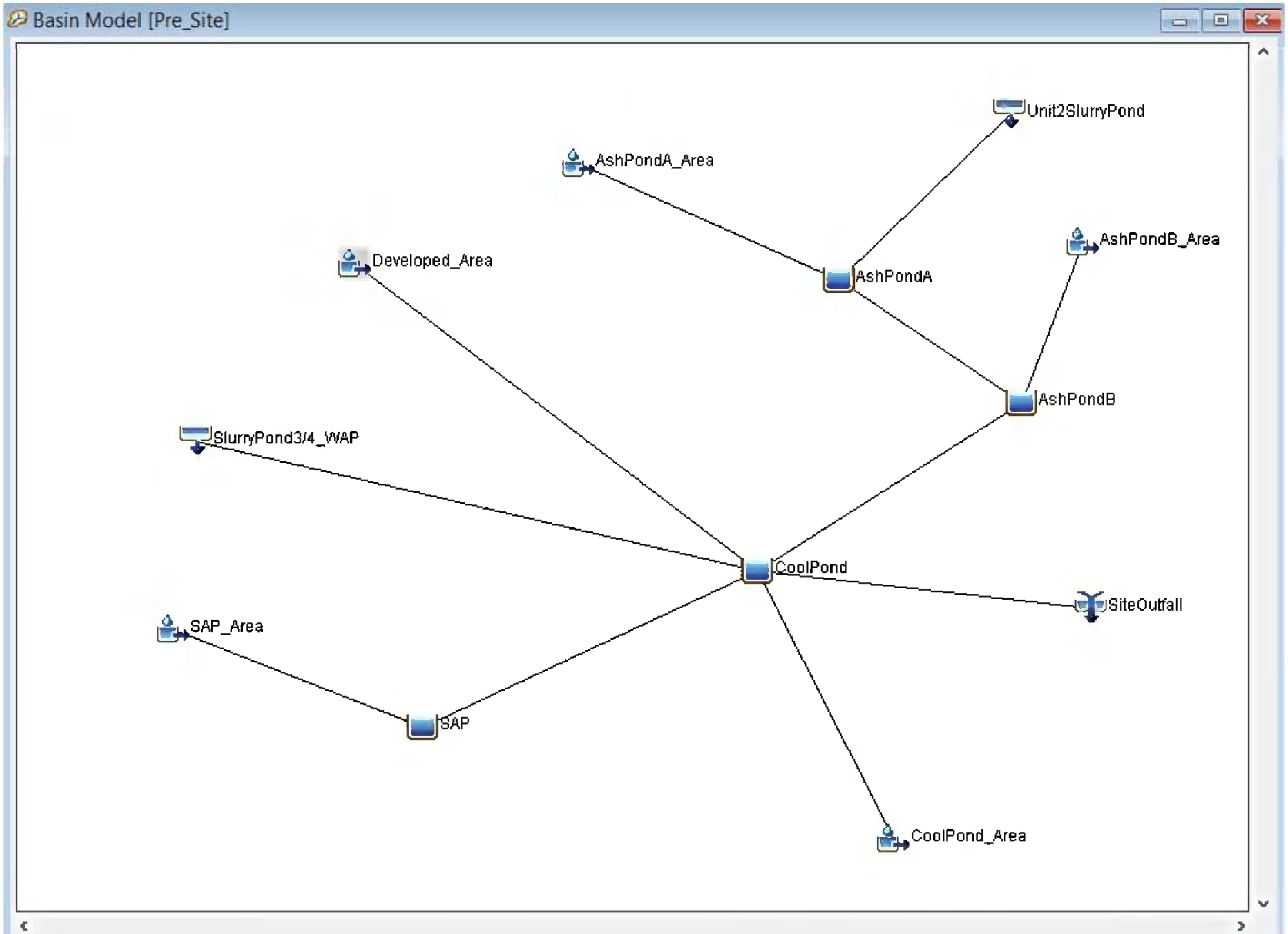


Figure 4. Pre-Development HEC-HMS Nodal Network

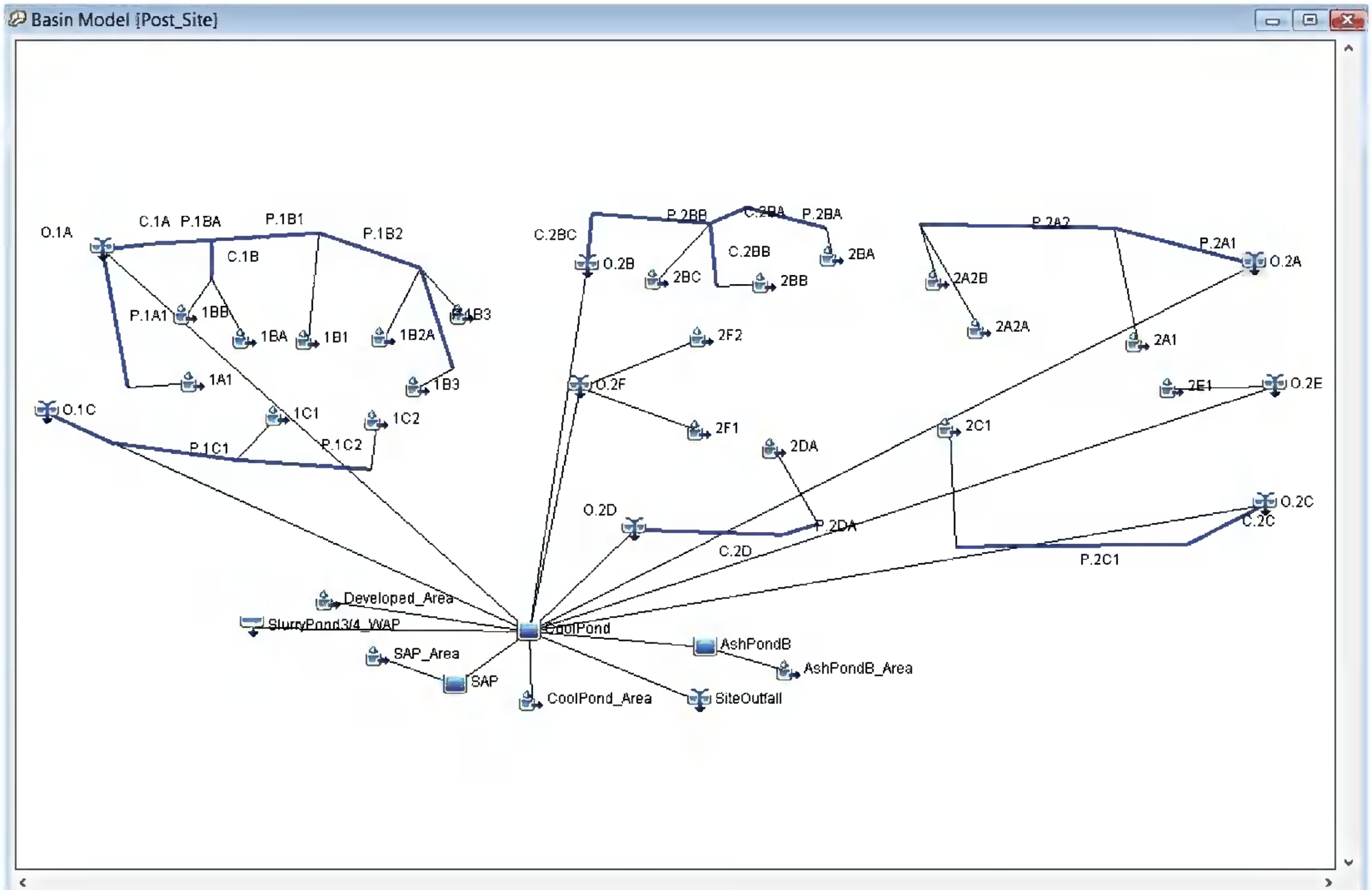


Figure 5. Post-Development HEC-HMS Nodal Network

APPENDIX 1

HEC-HMS HYDROLOGIC MODEL PARAMETERS

Table 1-1. Pre-Development 25-year, 24-hour Precipitation Event Nodal Areas, Peak Flow Rates, and Runoff Volumes

Global Summary Results for Run "Pre_Site_25-Year"

Project: Winyah Landfills_Areas 1 an   Simulation Run: Pre_Site_25-Year

Start of Run: 01Jan2016, 00:00   Basin Model: Pre_Site
End of Run: 06Jan2016, 00:00   Meteorologic Model: 25-Year
Compute Time: 04Aug2016, 18:21:34   Control Specifications: Control 1

Show Elements: All Elements   Volume Units: IN (•) AC-FT   Sorting: Alphabetic

Hydrologic Element	Drainage Area (MI2)	Peak Discharge (CFS)	Time of Peak	Volume (AC-FT)
AshPondA	0.18785	414.1	01Jan2016, 12:16	61.4
AshPondA_Area	0.12963	481.4	01Jan2016, 12:11	48.1
AshPondB	0.30388	57.8	01Jan2016, 15:19	113.5
AshPondB_Area	0.11603	245.7	01Jan2016, 12:38	43.7
CoolPond	1.96398	0.0	01Jan2016, 00:00	0.0
CoolPond_Area	0.85885	4052.2	01Jan2016, 12:05	368.3
Developed_Area	0.41491	578.1	01Jan2016, 13:17	164.6
SAP	0.11812	137.7	01Jan2016, 13:29	44.9
SAP_Area	0.11812	198.0	01Jan2016, 12:56	44.9
SiteOutfall	1.96398	0.0	01Jan2016, 00:00	0.0
SlurryPond3/4_WAP	0.26822	6.8	01Jan2016, 00:00	13.4
Unit2SlurryPond	0.05822	5.8	01Jan2016, 00:00	11.5

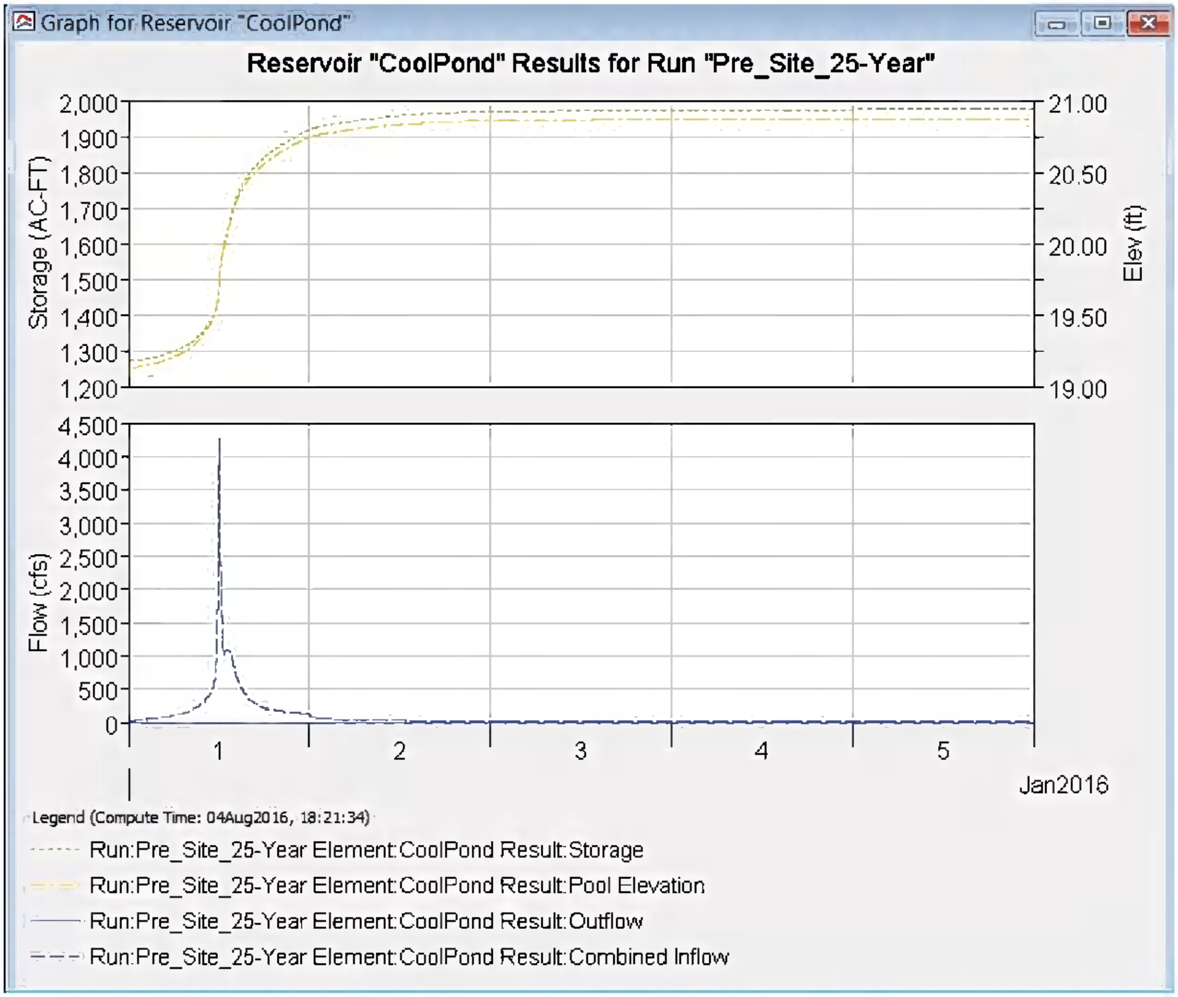


Figure 1-1. Pre-Development 24-year, 24-hour Precipitation Event Cooling Pond Surface Water Pond Hydrograph and Elevation/Storage Relationships

Table 1-2. Cooling Pond Elevation-Storage Relationship

Paired Data | Table | Graph

Elevation (FT)	Storage (AC-FT)
9.63	0.00001
11.63	30.00000
13.63	105.00000
15.63	290.00000
17.63	735.00000
19.63	1450.00000
20.63	1870.00000
20.93	2000.00000
21.63	2300.00000
23.63	3000.00000

Table 1-3. Ash Pond A Elevation-Area Relationship

Paired Data | Table | Graph

Elevation (FT)	Area (AC)
28.0	0.015
30.0	0.162
32.0	0.273
34.0	0.869
36.0	1.525
38.0	3.293
38.8	8.432

Table 1-4. Ash Pond B Elevation-Area Relationship

Paired Data | Table | Graph

Elevation (FT)	Area (AC)
34.00	0.006
36.00	30.021
38.00	59.773
39.68	62.211

Table 1-5. South Ash Pond Elevation-Area Relationship

Paired Data | Table | Graph

Elevation (FT)	Area (AC)
12.0	0.001
14.0	0.007
16.0	0.033
18.0	0.314
20.0	2.071
22.0	3.080
24.0	4.152
26.0	5.225
28.0	6.456
30.0	10.286
32.0	20.794
34.0	41.028
36.0	56.316
36.9	64.301

Table 1-6. Post-Development 25-year, 24-hour Precipitation Event Nodal Areas, Peak Flow Rates, and Runoff Volumes

Global Summary Results for Run "Post_Site_25-Year"

Project: Winyah Landfills_Areas 1 an   Simulation Run: Post_Site_25-Year

Start of Run: 01Jan2016, 00:00   Basin Model: Post_Site
End of Run: 06Jan2016, 00:00   Meteorologic Model: 25-Year
Compute Time:04Aug2016, 18:23:52   Control Specifications:Control 1

Show Elements: All Elements   Volume Units: IN ◉ AC-FT   Sorting: Alphabetic

Hydrologic Element	Drainage Area (MI2)	Peak Discharge (CFS)	Time of Peak	Volume (AC-FT)
AshPondB	0.11603	29.8	01Jan2016, 14:56	40.7
AshPondB_Area	0.11603	245.7	01Jan2016, 12:38	43.7
CoolPond	1.96399	0.0	01Jan2016, 00:00	0.0
CoolPond_Area	0.85885	4052.2	01Jan2016, 12:05	368.3
C.1A	0.03378	115.8	01Jan2016, 12:10	10.6
C.1B	0.00414	16.3	01Jan2016, 12:05	1.3
C.1C	0.01677	59.5	01Jan2016, 12:10	5.3
C.2BA	0.00378	14.9	01Jan2016, 12:06	1.2
C.2BB	0.00291	11.5	01Jan2016, 12:05	0.9
C.2BC	0.00924	35.1	01Jan2016, 12:09	2.9
C.2C	0.02730	102.7	01Jan2016, 12:10	8.6
C.2D	0.00602	23.6	01Jan2016, 12:06	1.9
Developed_Area	0.41491	578.1	01Jan2016, 13:17	164.6
O.1A	0.04147	143.9	01Jan2016, 12:10	13.0
O.1C	0.01677	59.5	01Jan2016, 12:10	5.3
O.2A	0.02405	89.5	01Jan2016, 12:09	7.6
O.2B	0.00924	35.1	01Jan2016, 12:09	2.9
O.2C	0.02730	102.7	01Jan2016, 12:10	8.6
O.2D	0.00602	23.6	01Jan2016, 12:06	1.9
O.2E	0.01264	49.0	01Jan2016, 12:06	4.0
O.2F	0.05037	187.1	01Jan2016, 12:07	15.8
P.1A1	0.00769	28.1	01Jan2016, 12:09	2.4
P.1BA	0.03378	115.9	01Jan2016, 12:09	10.6
P.1B1	0.02964	102.7	01Jan2016, 12:10	9.3
P.1B2	0.02108	73.4	01Jan2016, 12:10	6.6
P.1B3	0.00838	29.6	01Jan2016, 12:11	2.6
P.1C1	0.01677	59.6	01Jan2016, 12:10	5.3
P.1C2	0.00533	20.1	01Jan2016, 12:08	1.7

Table 1-6 (Continued). Post-Development 25-year, 24-hour Precipitation Event Nodal Areas, Peak Flow Rates, and Runoff Volumes

Global Summary Results for Run "Post_Site_25-Year"

Project: Winyah Landfills_Areas 1 an Simulation Run: Post_Site_25-Year

Start of Run: 01Jan2016, 00:00 Basin Model: Post_Site
End of Run: 06Jan2016, 00:00 Meteorologic Model: 25-Year
Compute Time: 04Aug2016, 18:23:52 Control Specifications: Control 1

Show Elements: All Elements Volume Units: IN AC-FT Sorting: Alphabetic

Hydrologic Element	Drainage Area (MI2)	Peak Discharge (CFS)	Time of Peak	Volume (AC-FT)
P.2A1	0.02405	89.5	01Jan2016, 12:09	7.6
P.2A2	0.00602	23.7	01Jan2016, 12:08	1.9
P.2BA	0.00378	14.9	01Jan2016, 12:05	1.2
P.2BB	0.00924	35.2	01Jan2016, 12:08	2.9
P.2C1	0.02730	102.8	01Jan2016, 12:10	8.6
P.2DA	0.00602	23.6	01Jan2016, 12:05	1.9
SAP	0.11812	137.7	01Jan2016, 13:29	44.9
SAP_Area	0.11812	198.0	01Jan2016, 12:56	44.9
SiteOutfall	1.96399	0.0	01Jan2016, 00:00	0.0
SlurryPond3/4_WAP	0.26822	6.8	01Jan2016, 00:00	13.4
1A1	0.00769	28.3	01Jan2016, 12:07	2.4
1BA	0.00346	13.7	01Jan2016, 12:05	1.1
1BB	0.00068	2.7	01Jan2016, 12:05	0.2
1B1	0.00856	32.3	01Jan2016, 12:07	2.7
1B2A	0.00200	7.9	01Jan2016, 12:05	0.6
1B2B	0.01070	41.1	01Jan2016, 12:06	3.4
1B3	0.00838	29.8	01Jan2016, 12:08	2.6
1C1	0.01144	39.7	01Jan2016, 12:09	3.6
1C2	0.00533	20.1	01Jan2016, 12:06	1.7
2A1	0.01803	66.0	01Jan2016, 12:08	5.7
2A2A	0.00336	13.3	01Jan2016, 12:05	1.1
2A2B	0.00266	10.5	01Jan2016, 12:05	0.8
2BA	0.00378	14.9	01Jan2016, 12:05	1.2
2BB	0.00291	11.5	01Jan2016, 12:05	0.9
2BC	0.00255	9.3	01Jan2016, 12:08	0.8
2C1	0.02730	103.3	01Jan2016, 12:06	8.6
2DA	0.00602	23.7	01Jan2016, 12:05	1.9
2E1	0.01264	49.0	01Jan2016, 12:06	4.0
2F1	0.02332	87.2	01Jan2016, 12:07	7.3
2F2	0.02705	100.0	01Jan2016, 12:07	8.5

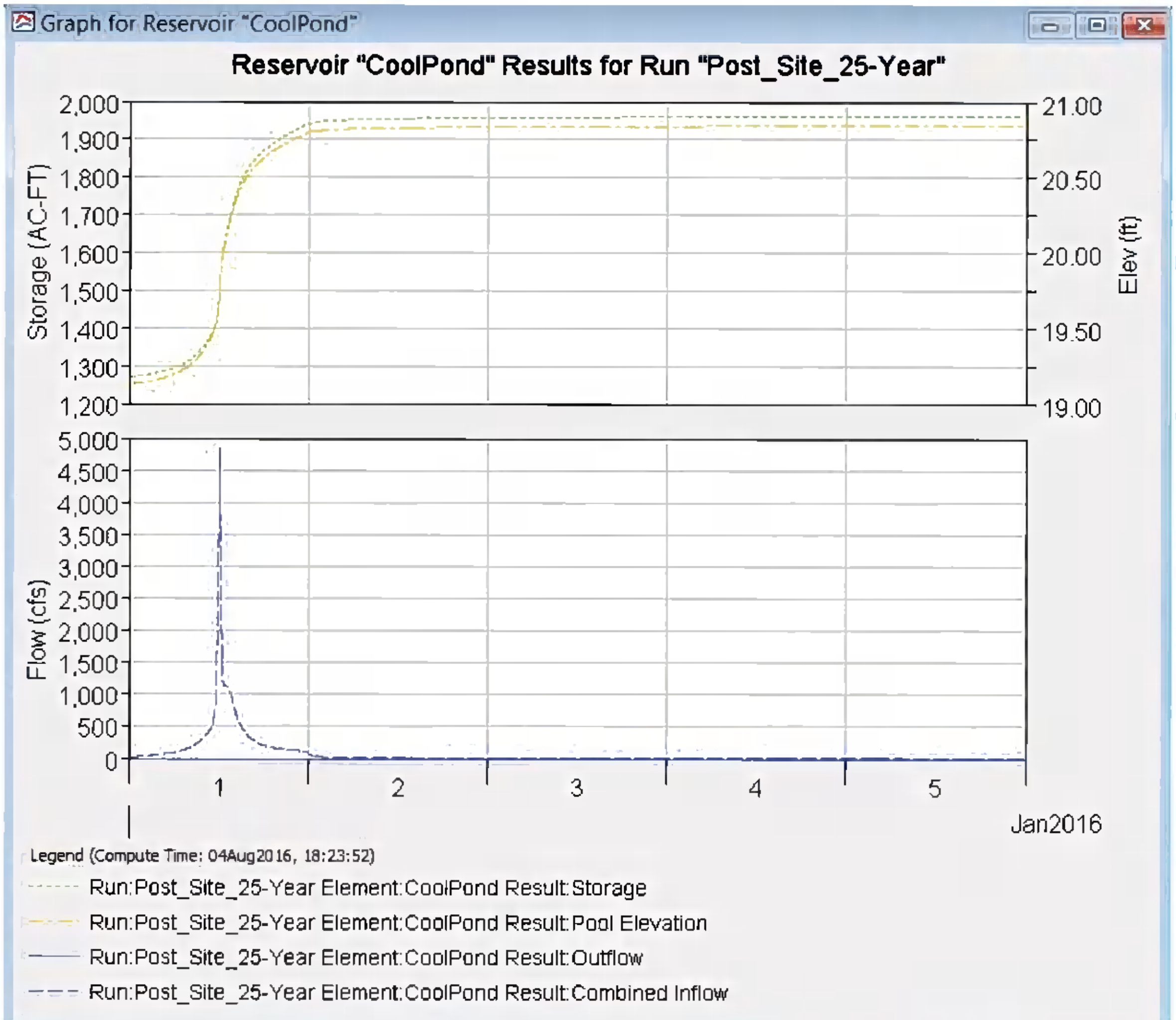


Figure 1-2. Post-Development 24-year, 24-hour Precipitation Event Cooling Pond Surface Water Pond Hydrograph and Elevation/Storage Relationships

HEC-HMS PRE-DEVELOPMENT HYDROLOGIC MODEL INPUT PARAMETERS

Pre-Development HEC-HMS Basin Input Parameters for Kinematic Wave Model

PRE-DEVELOPMENT CONDITIONS

			Watershed Characterization			Sheet Flow				Shallow Concentrated Flow					Open Channel Flow											
Subcatchment Designation	Area A (mi^2)	Area A (acres)	Initial Abstraction (in)	Curve Number	Impervious Cover (%)	Flow Length (ft)	Manning's n	Slope (ft/ft)	Time T_t (min)	Flow Length (ft)	Velocity Factor (ft/s)	Slope (ft/ft)	Average Velocity (ft/s)	Time T_t (min)	Flow Length (ft)	Depth d (ft)	Area A (ft^2)	Wetted P (ft)	Hydraulic Radius (ft)	Manning's n	Slope (ft/ft)	Velocity (ft/s)	Time T_t (min)	Design T_c (min)	SCS Lag Time (min)	HMS 25-yr Flow (cfs)
Developed	0.41491	265.5	0.15	93.0	0.00	300	0.40	0.010	57.5	2777	10.00	0.005	0.71	65.46										123	73.8	578.1
Slurry Pond 3&4 +WAP	0.26822	171.7	0.15	92.9	0.00	218	0.05	0.018	6.6	3036	10.00	0.0003	0.18	278.84	1475	2.0	62.00	37.65	1.65	0.028	0.005	5.25	4.69	290	174.1	6.8
SAP	0.11812	75.6	0.21	90.4	0.00	300	0.05	0.007	12.8	3223	10.00	0.005	0.71	75.98										89	53.3	198.0
Ash Pond A	0.12963	83.0	0.25	89.0	0.00	151	0.05	0.013	5.6						3152	2.0	62.00	37.65	1.65	0.028	0.005	5.25	10.01	16	9.4	481.4
Ash Pond B	0.11603	74.3	0.23	89.8	0.00	300	0.05	0.005	14.4	1737	10.00	0.005	0.71	40.94	1000	2.0	22.00	17.65	1.25	0.028	0.005	4.36	3.82	59	35.5	245.7
Unit 2 Slurry Pond	0.05822	37.3	0.04	98.0	0.00	142	0.05	0.014	5.2	3261	10.00	0.005	0.68	80.15										85	51.2	5.8

2-year, 24-hour Design Rainfall Depth = 4.50 inches

Perimeter Channel Left Side Slope = 3.0 H:V
Perimeter Channel Right Side Slope = 3.0 H:V
Perimeter Channel Bottom Width = 25.0 ft
Interior Channel Bottom Width = 5.0 ft

Notes:

1) *Curve number = 93 represents industrial or fully developed urban areas for hydrologic soil group D (USDA, 1986).*
2) *Curve number = 89 represents open space with no grass for hydrologic soil group D (USDA, 1986).*
3) *Curve number = 90 represents 10 % of water with open space with no grass for hydrologic soil group D (USDA, 1986).*
4) *Curve number = 73 represents brush-weed grass mixture with good cover for hydrologic soil group D (USDA, 1986).*
5) *Manning's roughness coefficient: n = 0.40 represents woods with light underbrush for sheet flow (USDA, 1984).*
6) *Manning's roughness coefficient: n = 0.05 represents fallow with no residue for sheet flow (USDA, 1984).*
7) *Manning's roughness coefficient: n = 0.028 represents a dragline-excavated or dredged channel with no vegetation (Chow, 1959).*
8) *Travel Time (T_t) is calculated using Manning's kinematic solutions for sheet flow (USDA, 1986).*

$$T_t = 0.007(nL)^{0.8} / (P_{2\text{-}24})^{0.5} S^{0.4}$$

9) *Velocity factor of 10.0 ft/s corresponds to a value for nearly bare and untilled surfaces from the Upland Method as reported by HydroCAD v.10 Owner's Manual.*
10) *Open channel flow velocity is calculated using Manning's equation (USDA, 1986).*

$$V = (1.49 r^{2/3} S^{1/2}) / n \quad \text{where: } r = \text{hydraulic radius (ft) and is equal to } A/P \text{ [area (ft}^2\text{)/wetted perimeter (ft)]}$$

Basin: Pre_Site
Last Modified Date: 1 August 2016
Last Modified Time: 13:47:27
Version: 4.1
Filepath Separator: \
Unit System: English
Missing Flow To Zero: No
Enable Flow Ratio: No
Compute Local Flow At Junctions: No

Enable Sediment Routing: No

Enable Quality Routing: No
End:

Subbasin: CoolPond_Area
Last Modified Date: 30 July 2016
Last Modified Time: 22:00:38
Canvas X: 16949.950824572217
Canvas Y: -22428.927249820517
Area: 0.85885
Downstream: CoolPond

Canopy: None
Plant Uptake Method: None

Surface: None

LossRate: SCS
Percent Impervious Area: 0.0
Curve Number: 98

Transform: SCS
Lag: 3.6
Unitgraph Type: STANDARD

Baseflow: None
End:

Subbasin: Developed_Area
Last Modified Date: 3 August 2016
Last Modified Time: 17:50:42
Canvas X: 5771.395371927307
Canvas Y: -10492.754390167414
Area: 0.41491
Downstream: CoolPond

Canopy: None
Plant Uptake Method: None

Surface: None

LossRate: SCS
Percent Impervious Area: 0.0
Curve Number: 93

Transform: SCS
Lag: 73.8
Unitgraph Type: STANDARD

Baseflow: None
End:

Subbasin: AshPondA_Area
Last Modified Date: 3 August 2016
Last Modified Time: 17:50:12
Canvas X: 10414.849851559466
Canvas Y: -8423.35429603773
Area: 0.12963
Downstream: AshPondA

Canopy: None
Plant Uptake Method: None

Surface: None

LossRate: SCS
Percent Impervious Area: 0.0
Curve Number: 89

Transform: SCS
Lag: 9.4
Unitgraph Type: STANDARD

Baseflow: None
End:

Source: Unit2SlurryPond
Last Modified Date: 3 August 2016
Last Modified Time: 17:50:11
Canvas X: 19371.13211305606
Canvas Y: -7389.937112018892
Area: 0.05822
Downstream: AshPondA

Flow Method: GAGE_FLOW
Flow Gage: SlurryPond2_DG
End Flow Method:
End:

Reservoir: AshPondA
Last Modified Date: 3 August 2016
Last Modified Time: 17:50:09
Canvas X: 15811.584034768952
Canvas Y: -10834.661058748352
Downstream: AshPondB

Route: Controlled Outflow
Routing Curve: Elevation-Area
Initial Outflow Equals Inflow: Yes
Elevation-Area Table: Ash Pond A_EA
Adaptive Control: On
Main Tailwater Condition: None
Auxiliary Tailwater Condition: None

Conduit: Culvert
Conduit Outlet: Main
Culvert Shape: Circular
Chart Number: 1
Scale Number: 1
Solution Control: Automatic
Diameter: 2.5
Number Barrels: 1
Culvert Length: 40.8
Entrance Loss Coefficient: 0.5
Exit Loss Coefficient: 1
Top Manning's n: 0.025
Inlet Invert Elevation: 37.5
Outlet Invert Elevation: 36.52
End Conduit:

Conduit: Culvert
Conduit Outlet: Main
Culvert Shape: Circular

Chart Number: 1
Scale Number: 1
Solution Control: Automatic
Diameter: 4
Number Barrels: 1
Culvert Length: 30.9
Entrance Loss Coefficient: 0.5
Exit Loss Coefficient: 1
Top Manning's n: 0.012
Inlet Invert Elevation: 35.49
Outlet Invert Elevation: 35.28
End Conduit:

Conduit: Culvert
Conduit Outlet: Main
Culvert Shape: Circular
Chart Number: 1
Scale Number: 1
Solution Control: Automatic
Diameter: 3.5
Number Barrels: 1
Culvert Length: 24.6
Entrance Loss Coefficient: 0.5
Exit Loss Coefficient: 1
Top Manning's n: 0.012
Inlet Invert Elevation: 36.2
Outlet Invert Elevation: 35.7
End Conduit:

Spillway: Broad-Crested Spillway
Spillway Outlet: Main
Spillway Crest Length: 100
Spillway Crest Elevation: 37
Spillway Coefficient: 2.6
End Spillway:

Evaporation Method: Zero Evaporation
End Evaporation:
End:

Subbasin: AshPondB_Area
Last Modified Date: 3 August 2016
Last Modified Time: 17:50:35
Canvas X: 20878.91005805196
Canvas Y: -10038.703402333067
Area: 0.11603
Downstream: AshPondB

Canopy: None
Plant Uptake Method: None

Surface: None

LossRate: SCS
Percent Impervious Area: 0.0
Curve Number: 89.9

Transform: SCS
Lag: 35.5
Unitgraph Type: STANDARD

Baseflow: None
End:

Reservoir: AshPondB
Last Modified Date: 3 August 2016
Last Modified Time: 17:49:29
Canvas X: 19606.460413120556
Canvas Y: -13380.19146382773
Downstream: CoolPond

Route: Controlled Outflow
Routing Curve: Elevation-Area
Initial Outflow Equals Inflow: Yes
Elevation-Area Table: Ash Pond B_EA
Adaptive Control: On
Main Tailwater Condition: None
Auxiliary Tailwater Condition: None

Conduit: Culvert
Conduit Outlet: Main
Culvert Shape: Circular
Chart Number: 1
Scale Number: 1
Solution Control: Automatic
Diameter: 1.8
Number Barrels: 1
Culvert Length: 113.3
Entrance Loss Coefficient: 0.5
Exit Loss Coefficient: 1
Top Manning's n: 0.013
Inlet Invert Elevation: 34
Outlet Invert Elevation: 17.99
End Conduit:

Spillway: Broad-Crested Spillway
Spillway Outlet: Main
Spillway Crest Length: 4
Spillway Crest Elevation: 34.9
Spillway Coefficient: 3
End Spillway:

Evaporation Method: Zero Evaporation
End Evaporation:
End:

Source: SlurryPond3/4_WAP
Last Modified Date: 3 August 2016
Last Modified Time: 17:50:40
Canvas X: 2469.2063694956887
Canvas Y: -14166.439655372591
Area: 0.26822
Downstream: CoolPond

Flow Method: GAGE_FLOW
Flow Gage: SlurryPond-WAP_DG
End Flow Method:
End:

Subbasin: SAP_Area
Last Modified Date: 3 August 2016
Last Modified Time: 17:50:39
Canvas X: 2015.1553816613414
Canvas Y: -18087.789095760138
Area: 0.11812
Downstream: SAP

Canopy: None
Plant Uptake Method: None

Surface: None

LossRate: SCS
Percent Impervious Area: 0.0

Curve Number: 90.4

Transform: SCS
Lag: 53.3
Unitgraph Type: STANDARD

Baseflow: None
End:

Reservoir: SAP
Last Modified Date: 3 August 2016
Last Modified Time: 17:50:37
Canvas X: 7174.825697960743
Canvas Y: -20110.379859749504
Downstream: CoolPond

Route: Controlled Outflow
Routing Curve: Elevation-Area
Initial Outflow Equals Inflow: Yes
Elevation-Area Table: SAP_EA
Adaptive Control: On
Main Tailwater Condition: None
Auxiliary Tailwater Condition: None

Spillway: Broad-Crested Spillway
Spillway Outlet: Main
Spillway Crest Length: 30
Spillway Crest Elevation: 28.73
Spillway Coefficient: 3
End Spillway:

Evaporation Method: Zero Evaporation
End Evaporation:
End:

Reservoir: CoolPond
Last Modified Date: 5 August 2016
Last Modified Time: 01:22:36
Canvas X: 14164.3532470499
Canvas Y: -16841.59376915017
Downstream: SiteOutfall

Route: Controlled Outflow
Routing Curve: Elevation-Storage
Initial Elevation: 19.13
Elevation-Storage Table: ES_Cool_Pond
Adaptive Control: On
Main Tailwater Condition: None
Auxiliary Tailwater Condition: None

Spillway: Broad-Crested Spillway
Spillway Outlet: Main
Spillway Crest Length: 63.29
Spillway Crest Elevation: 20.93
Spillway Coefficient: 3
End Spillway:

Evaporation Method: Zero Evaporation
End Evaporation:
End:

Junction: SiteOutfall
Last Modified Date: 3 August 2016
Last Modified Time: 17:50:33
Canvas X: 21044.01950817354
Canvas Y: -17633.73810792579
End:

Basin Schematic Properties:
Last View N: -5886.038098238729
Last View S: -23932.82626360068
Last View W: 112.26896901012151
Last View E: 22946.90592082476
Maximum View N: -5886.038098238729
Maximum View S: -23932.82626360068
Maximum View W: 112.26896901012151
Maximum View E: 22946.90592082476
Extent Method: Elements
Buffer: 20
Draw Icons: Yes
Draw Icon Labels: Name
Draw Map Objects: No
Draw Gridlines: No
Draw Flow Direction: No
Fix Element Locations: No
Fix Hydrologic Order: No
End:

HEC-HMS POST-DEVELOPMENT HYDROLOGIC MODEL INPUT PARAMETERS

Basin: Post_Site
Last Modified Date: 30 July 2016
Last Modified Time: 21:57:03
Version: 4.1
Filepath Separator: \
Unit System: English
Missing Flow To Zero: No
Enable Flow Ratio: No
Compute Local Flow At Junctions: No

Enable Sediment Routing: No

Enable Quality Routing: No
End:

Subbasin: CoolPond_Area
Last Modified Date: 3 August 2016
Last Modified Time: 17:52:03
Canvas X: 2821.678332635842
Canvas Y: -16678.674874247386
Area: 0.85885
Downstream: CoolPond

Canopy: None
Plant Uptake Method: None

Surface: None

LossRate: SCS
Percent Impervious Area: 0.0
Curve Number: 98

Transform: SCS
Lag: 3.6
Unitgraph Type: STANDARD

Baseflow: None
End:

Subbasin: Developed_Area
Last Modified Date: 3 August 2016
Last Modified Time: 17:51:48
Canvas X: -9203.552192078438
Canvas Y: -10741.21730266971
Area: 0.41491
Downstream: CoolPond

Canopy: None
Plant Uptake Method: None

Surface: None

LossRate: SCS
Percent Impervious Area: 0.0
Curve Number: 93

Transform: SCS
Lag: 73.8
Unitgraph Type: STANDARD

Baseflow: None

End:

Source: SlurryPond3/4_WAP
Last Modified Date: 3 August 2016
Last Modified Time: 17:51:50
Canvas X: -13637.855948066828
Canvas Y: -12319.528809038457
Area: 0.26822
Downstream: CoolPond

Flow Method: GAGE_FLOW
Flow Gage: SlurryPond-WAP_DG
End Flow Method:
End:

Subbasin: SAP_Area
Last Modified Date: 3 August 2016
Last Modified Time: 17:52:01
Canvas X: -6272.402251679334
Canvas Y: -14048.15569696614
Area: 0.11812
Downstream: SAP

Canopy: None
Plant Uptake Method: None

Surface: None

LossRate: SCS
Percent Impervious Area: 0.0
Curve Number: 90.4

Transform: SCS
Lag: 53.3
Unitgraph Type: STANDARD

Baseflow: None
End:

Reservoir: SAP
Last Modified Date: 3 August 2016
Last Modified Time: 17:52:00
Canvas X: -1687.783114132013
Canvas Y: -15701.62489411435
Downstream: CoolPond

Route: Controlled Outflow
Routing Curve: Elevation-Area
Initial Outflow Equals Inflow: Yes
Elevation-Area Table: SAP_EA
Adaptive Control: On
Main Tailwater Condition: None
Auxiliary Tailwater Condition: None

Spillway: Broad-Crested Spillway
Spillway Outlet: Main
Spillway Crest Length: 30
Spillway Crest Elevation: 28.73
Spillway Coefficient: 3
End Spillway:

Evaporation Method: Zero Evaporation
End Evaporation:
End:

Subbasin: AshPondB_Area
Last Modified Date: 3 August 2016
Last Modified Time: 17:52:13
Canvas X: 18003.53187008762
Canvas Y: -14874.890295540245
Area: 0.11603
Downstream: AshPondB

Canopy: None
Plant Uptake Method: None

Surface: None

LossRate: SCS
Percent Impervious Area: 0.0
Curve Number: 89.9

Transform: SCS
Lag: 35.5
Unitgraph Type: STANDARD

Baseflow: None
End:

Reservoir: AshPondB
Last Modified Date: 3 August 2016
Last Modified Time: 17:52:09
Canvas X: 13118.281969422442
Canvas Y: -13446.894170730426
Downstream: CoolPond

Route: Controlled Outflow
Routing Curve: Elevation-Area
Initial Outflow Equals Inflow: Yes
Elevation-Area Table: Ash Pond B_EA
Adaptive Control: On
Main Tailwater Condition: None
Auxiliary Tailwater Condition: None

Conduit: Culvert
Conduit Outlet: Main
Culvert Shape: Circular
Chart Number: 1
Scale Number: 1
Solution Control: Automatic
Diameter: 1.8
Number Barrels: 1
Culvert Length: 113.3
Entrance Loss Coefficient: 0.5
Exit Loss Coefficient: 1
Top Manning's n: 0.013
Inlet Invert Elevation: 34
Outlet Invert Elevation: 17.99
End Conduit:

Spillway: Broad-Crested Spillway
Spillway Outlet: Main
Spillway Crest Length: 4
Spillway Crest Elevation: 34.9
Spillway Coefficient: 3
End Spillway:

Evaporation Method: Zero Evaporation
End Evaporation:
End:

Subbasin: 2F2
Last Modified Date: 1 July 2016
Last Modified Time: 16:18:23
Canvas X: 12885.616843649492
Canvas Y: 4794.7598030265435
Area: 0.02705
Downstream: O.2F

Canopy: None
Plant Uptake Method: None

Surface: None

LossRate: SCS
Percent Impervious Area: 0.0
Curve Number: 80

Transform: SCS
Lag: 5.77
Unitgraph Type: STANDARD

Baseflow: None
End:

Subbasin: 2F1
Last Modified Date: 1 July 2016
Last Modified Time: 16:18:23
Canvas X: 12744.106111734043
Canvas Y: -677.804127648571
Area: 0.02332
Downstream: O.2F

Canopy: None
Plant Uptake Method: None

Surface: None

LossRate: SCS
Percent Impervious Area: 0.0
Curve Number: 80

Transform: SCS
Lag: 5.44
Unitgraph Type: STANDARD

Baseflow: None
End:

Junction: O.2F
Last Modified Date: 30 July 2016
Last Modified Time: 21:57:46
Canvas X: 5692.872341359882

Canvas Y: 1847.8327345456855
Downstream: CoolPond
End:

Subbasin: 1B2B
Last Modified Date: 1 July 2016
Last Modified Time: 16:16:48
Canvas X: -1231.8523918511928
Canvas Y: 6143.726782000704
From Canvas X: 10493.873085339166
From Canvas Y: 362.69146608314986
Label X: 0.0
Label Y: 1.0
Area: 0.01070
Downstream: P.1B2

Canopy: None
Plant Uptake Method: None

Surface: None

LossRate: SCS
Percent Impervious Area: 0.0
Curve Number: 80

Transform: SCS
Lag: 4.58
Unitgraph Type: STANDARD

Baseflow: None
End:

Subbasin: 1B3
Last Modified Date: 30 July 2016
Last Modified Time: 20:36:01
Canvas X: -3887.0462606038272
Canvas Y: 1894.7386703252268
Area: 0.00838
Downstream: P.1B3

Canopy: None
Plant Uptake Method: None

Surface: None

LossRate: SCS
Percent Impervious Area: 0.0
Curve Number: 80

Transform: SCS
Lag: 6.93
Unitgraph Type: STANDARD

Baseflow: None
End:

Reach: P.1B3
Last Modified Date: 29 July 2016
Last Modified Time: 12:57:46
Canvas X: -3713.78669946232
Canvas Y: 8938.33764631685
From Canvas X: -1811.6481445504396
From Canvas Y: 3056.9616153151255
Downstream: P.1B2

Route: Kinematic Wave
Channel: Kinematic Wave
Length: 800
Energy Slope: 0.005
Mannings n: 0.027
Shape: Trapezoid
Number of Subreaches: 2
Width: 3
Side Slope: 3
Channel Loss: None
End:

Subbasin: 1B2A
Last Modified Date: 29 July 2016
Last Modified Time: 12:56:14
Canvas X: -5879.428452015083
Canvas Y: 4800.296032799975
Area: 0.00200
Downstream: P.1B2

Canopy: None
Plant Uptake Method: None

Surface: None

LossRate: SCS
Percent Impervious Area: 0.0
Curve Number: 80

Transform: SCS
Lag: 3.79
Unitgraph Type: STANDARD

Baseflow: None
End:

Reach: P.1B2
Last Modified Date: 29 July 2016
Last Modified Time: 12:57:00
Canvas X: -9744.809909356078
Canvas Y: 11051.39687314094
From Canvas X: -3713.78669946232
From Canvas Y: 8938.33764631685
Label X: -18.0
Label Y: 11.0
Downstream: P.1B1

Route: Kinematic Wave
Channel: Kinematic Wave
Length: 953
Energy Slope: 0.005
Mannings n: 0.027
Shape: Trapezoid
Number of Subreaches: 2
Width: 3
Side Slope: 3
Channel Loss: None

End:

Subbasin: 1B1
Last Modified Date: 30 July 2016
Last Modified Time: 20:36:35
Canvas X: -10405.140917738609
Canvas Y: 4668.197125443163
Area: 0.00856
Downstream: P.1B1

Canopy: None
Plant Uptake Method: None

Surface: None

LossRate: SCS
Percent Impervious Area: 0.0
Curve Number: 80

Transform: SCS
Lag: 5.15
Unitgraph Type: STANDARD

Baseflow: None
End:

Reach: P.1B1
Last Modified Date: 5 July 2016
Last Modified Time: 14:12:06
Canvas X: -16128.009657053855
Canvas Y: 10567.154133660419
From Canvas X: -9744.809909356078
From Canvas Y: 11051.39687314094
Label X: -12.0
Label Y: 12.0
Downstream: P.1BA

Route: Kinematic Wave
Channel: Kinematic Wave
Length: 215
Energy Slope: 0.005
Mannings n: 0.027
Shape: Trapezoid
Number of Subreaches: 2
Width: 3
Side Slope: 3
Channel Loss: None
End:

Subbasin: 1BA
Last Modified Date: 1 July 2016
Last Modified Time: 16:16:48
Canvas X: -14102.994564680768
Canvas Y: 4712.219192668666
Area: 0.00346
Downstream: C.1B

Canopy: None
Plant Uptake Method: None

Surface: None

LossRate: SCS
Percent Impervious Area: 0.0
Curve Number: 80

Transform: SCS
Lag: 3.60
Unitgraph Type: STANDARD

Baseflow: None
End:

Subbasin: 1BB
Last Modified Date: 1 July 2016
Last Modified Time: 16:16:48
Canvas X: -17580.73787549542
Canvas Y: 6164.947411110228
Area: 0.00068
Downstream: C.1B

Canopy: None
Plant Uptake Method: None

Surface: None

LossRate: SCS
Percent Impervious Area: 0.0
Curve Number: 80

Transform: SCS
Lag: 3.60
Unitgraph Type: STANDARD

Baseflow: None
End:

Reach: C.1B
Last Modified Date: 15 June 2016
Last Modified Time: 15:02:49
Canvas X: -16128.009657053855
Canvas Y: 10567.154133660419
From Canvas X: -16128.009657053859
From Canvas Y: 8278.006637934319
Downstream: P.1BA

Route: Kinematic Wave
Channel: Kinematic Wave
Length: 100
Energy Slope: 0.005
Mannings n: 0.012
Shape: Circular
Number of Subreaches: 2
Width: 2
Channel Loss: None
End:

Reach: P.1BA
Last Modified Date: 5 July 2016
Last Modified Time: 14:12:03
Canvas X: -19693.79710231951
Canvas Y: 10347.04379753291

From Canvas X: -16128.009657053855
From Canvas Y: 10567.154133660419
Label X: -13.0
Label Y: 14.0
Downstream: C.1A

Route: Kinematic Wave
Channel: Kinematic Wave
Length: 105
Energy Slope: 0.005
Mannings n: 0.027
Shape: Trapezoid
Number of Subreaches: 2
Width: 3
Side Slope: 3
Channel Loss: None
End:

Reach: C.1A
Last Modified Date: 29 July 2016
Last Modified Time: 13:27:37
Canvas X: -22482.613380442213
Canvas Y: 10030.299285254518
From Canvas X: -19693.79710231951
From Canvas Y: 10347.04379753291
Label X: -3.0
Label Y: 18.0
Downstream: O.1A

Route: Kinematic Wave
Channel: Kinematic Wave
Length: 100
Energy Slope: 0.005
Mannings n: 0.012
Shape: Circular
Number of Subreaches: 2
Width: 4
Channel Loss: None
End:

Subbasin: 1A1
Last Modified Date: 1 July 2016
Last Modified Time: 16:16:48
Canvas X: -17184.539270465903
Canvas Y: 2202.9613608150576
Area: 0.00769
Downstream: P.1A1

Canopy: None
Plant Uptake Method: None

Surface: None

LossRate: SCS
Percent Impervious Area: 0.0
Curve Number: 80

Transform: SCS
Lag: 5.92
Unitgraph Type: STANDARD

Baseflow: None
End:

Reach: P.1A1
Last Modified Date: 22 June 2016
Last Modified Time: 16:17:54
Canvas X: -22482.613380442213
Canvas Y: 10030.299285254518
From Canvas X: -21014.459119084568
From Canvas Y: 1850.7848230110412
Downstream: O.1A

Route: Kinematic Wave
Channel: Kinematic Wave
Length: 525
Energy Slope: 0.005
Mannings n: 0.027
Shape: Trapezoid
Number of Subreaches: 2
Width: 3
Side Slope: 3
Channel Loss: None
End:

Junction: O.1A
Last Modified Date: 30 July 2016
Last Modified Time: 21:57:39
Canvas X: -22482.613380442213
Canvas Y: 10030.299285254518
Label X: -64.0
Label Y: 12.0
Downstream: CoolPond
End:

Subbasin: 2C1
Last Modified Date: 1 July 2016
Last Modified Time: 16:16:48
Canvas X: 27481.82337413488
Canvas Y: -577.4551005548856
Area: 0.02730
Downstream: P.2C1

Canopy: None
Plant Uptake Method: None

Surface: None

LossRate: SCS
Percent Impervious Area: 0.0
Curve Number: 80

Transform: SCS
Lag: 5.00
Unitgraph Type: STANDARD

Baseflow: None
End:

Reach: P.2C1
Last Modified Date: 5 July 2016
Last Modified Time: 14:12:56

Canvas X: 41571.21717786729
Canvas Y: -7368.745639044617
From Canvas X: 27887.27355553725
From Canvas Y: -7571.470729745801
Label X: -6.0
Label Y: -14.0
Downstream: C.2C

Route: Kinematic Wave
Channel: Kinematic Wave
Length: 1350
Energy Slope: 0.005
Mannings n: 0.027
Shape: Trapezoid
Number of Subreaches: 2
Width: 3
Side Slope: 3
Channel Loss: None
End:

Reach: C.2C
Last Modified Date: 29 July 2016
Last Modified Time: 13:27:37
Canvas X: 46137.8514921873
Canvas Y: -5041.691204269195
From Canvas X: 41571.21717786729
From Canvas Y: -7368.745639044617
Downstream: O.2C

Route: Kinematic Wave
Channel: Kinematic Wave
Length: 100
Energy Slope: 0.005
Mannings n: 0.012
Shape: Circular
Number of Subreaches: 2
Width: 4
Channel Loss: None
End:

Junction: O.2C
Last Modified Date: 30 July 2016
Last Modified Time: 21:57:58
Canvas X: 46137.8514921873
Canvas Y: -5041.691204269195
Downstream: CoolPond
End:

Subbasin: 2A1
Last Modified Date: 1 July 2016
Last Modified Time: 16:18:23
Canvas X: 38631.70336270011
Canvas Y: 4490.672166974764
Area: 0.01803
Downstream: P.2A1

Canopy: None
Plant Uptake Method: None

Surface: None

LossRate: SCS
Percent Impervious Area: 0.0
Curve Number: 80

Transform: SCS
Lag: 6.03
Unitgraph Type: STANDARD

Baseflow: None
End:

Subbasin: 2A2A
Last Modified Date: 1 July 2016
Last Modified Time: 16:16:48
Canvas X: 29306.349190445544
Canvas Y: 5301.572529779507
Area: 0.00336
Downstream: P.2A2

Canopy: None
Plant Uptake Method: None

Surface: None

LossRate: SCS
Percent Impervious Area: 0.0
Curve Number: 80

Transform: SCS
Lag: 3.60
Unitgraph Type: STANDARD

Baseflow: None
End:

Subbasin: 2A2B
Last Modified Date: 1 July 2016
Last Modified Time: 16:16:48
Canvas X: 26772.285556680727
Canvas Y: 8139.723799596108
Area: 0.00266
Downstream: P.2A2

Canopy: None
Plant Uptake Method: None

Surface: None

LossRate: SCS
Percent Impervious Area: 0.0
Curve Number: 80

Transform: SCS
Lag: 3.60
Unitgraph Type: STANDARD

Baseflow: None
End:

Reach: P.2A2
Last Modified Date: 15 June 2016

Last Modified Time: 15:06:19
Canvas X: 37212.6277277918
Canvas Y: 11281.962705464492
From Canvas X: 25758.77291482666
From Canvas Y: 11546.252075256181
Downstream: P.2A1

Route: Kinematic Wave
Channel: Kinematic Wave
Length: 690
Energy Slope: 0.005
Mannings n: 0.027
Shape: Trapezoid
Number of Subreaches: 2
Width: 3
Side Slope: 3
Channel Loss: None
End:

Reach: P.2A1
Last Modified Date: 3 August 2016
Last Modified Time: 17:52:28
Canvas X: 45467.788634463366
Canvas Y: 9198.417880803721
From Canvas X: 37212.6277277918
From Canvas Y: 11281.962705464492
Downstream: O.2A

Route: Kinematic Wave
Channel: Kinematic Wave
Length: 600
Energy Slope: 0.005
Mannings n: 0.027
Shape: Trapezoid
Number of Subreaches: 2
Width: 3
Side Slope: 3
Channel Loss: None
End:

Junction: O.2A
Last Modified Date: 3 August 2016
Last Modified Time: 17:52:28
Canvas X: 45467.788634463366
Canvas Y: 9198.417880803721
Downstream: CoolPond
End:

Subbasin: 1C1
Last Modified Date: 30 July 2016
Last Modified Time: 20:36:01
Canvas X: -12166.023606758685
Canvas Y: 133.92420121646865
Area: 0.01144
Downstream: P.1C1

Canopy: None
Plant Uptake Method: None

Surface: None

LossRate: SCS
Percent Impervious Area: 0.0
Curve Number: 80

Transform: SCS
Lag: 7.69
Unitgraph Type: STANDARD

Baseflow: None
End:

Subbasin: 1C2
Last Modified Date: 30 July 2016
Last Modified Time: 20:36:35
Canvas X: -6294.508075225764
Canvas Y: -97.64352108602907
Area: 0.00533
Downstream: P.1C2

Canopy: None
Plant Uptake Method: None

Surface: None

LossRate: SCS
Percent Impervious Area: 0.0
Curve Number: 80

Transform: SCS
Lag: 5.02
Unitgraph Type: STANDARD

Baseflow: None
End:

Reach: P.1C2
Last Modified Date: 29 July 2016
Last Modified Time: 13:20:42
Canvas X: -14675.281438612294
Canvas Y: -2375.333630637142
From Canvas X: -6684.710789452354
From Canvas Y: -2980.3543311221183
Label X: 2.0
Label Y: 14.0
Downstream: P.1C1

Route: Kinematic Wave
Channel: Kinematic Wave
Length: 415
Energy Slope: 0.005
Mannings n: 0.027
Shape: Trapezoid
Number of Subreaches: 2
Width: 3
Side Slope: 3
Channel Loss: None
End:

Reach: P.1C1
Last Modified Date: 15 June 2016
Last Modified Time: 15:04:00

Canvas X: -21960.93356443286
Canvas Y: -1384.8371180633476
From Canvas X: -14675.281438612294
From Canvas Y: -2375.333630637142
Downstream: C.1C

Route: Kinematic Wave
Channel: Kinematic Wave
Length: 340
Energy Slope: 0.005
Mannings n: 0.027
Shape: Trapezoid
Number of Subreaches: 2
Width: 3
Side Slope: 3
Channel Loss: None
End:

Reach: C.1C
Last Modified Date: 29 July 2016
Last Modified Time: 13:27:37
Canvas X: -25834.875480277027
Canvas Y: 376.04557095673226
From Canvas X: -21960.93356443286
From Canvas Y: -1384.8371180633476
Downstream: O.1C

Route: Kinematic Wave
Channel: Kinematic Wave
Length: 100
Energy Slope: 0.005
Mannings n: 0.012
Shape: Circular
Number of Subreaches: 2
Width: 6
Channel Loss: None
End:

Junction: O.1C
Last Modified Date: 30 July 2016
Last Modified Time: 21:57:32
Canvas X: -25834.875480277027
Canvas Y: 376.04557095673226
Downstream: CoolPond
End:

Subbasin: 2E1
Last Modified Date: 3 August 2016
Last Modified Time: 17:52:22
Canvas X: 40625.996794706356
Canvas Y: 1829.2509956677422
Area: 0.01264
Downstream: O.2E

Canopy: None
Plant Uptake Method: None

Surface: None

LossRate: SCS
Percent Impervious Area: 0.0
Curve Number: 80

Transform: SCS
Lag: 4.21
Unitgraph Type: STANDARD

Baseflow: None
End:

Junction: O.2E
Last Modified Date: 3 August 2016
Last Modified Time: 17:52:20
Canvas X: 46713.76974784296
Canvas Y: 1979.5663772266707
Downstream: CoolPond
End:

Subbasin: 2BA
Last Modified Date: 1 July 2016
Last Modified Time: 16:16:48
Canvas X: 20589.395913598288
Canvas Y: 9600.133674793733
Area: 0.00378
Downstream: P.2BA

Canopy: None
Plant Uptake Method: None

Surface: None

LossRate: SCS
Percent Impervious Area: 0.0
Curve Number: 80

Transform: SCS
Lag: 3.60
Unitgraph Type: STANDARD

Baseflow: None
End:

Reach: P.2BA
Last Modified Date: 29 July 2016
Last Modified Time: 13:24:10
Canvas X: 15538.680105655912
Canvas Y: 12520.77702451859
From Canvas X: 20225.430884358313
From Canvas Y: 11302.987275198375
Downstream: C.2BA

Route: Kinematic Wave
Channel: Kinematic Wave
Length: 100
Energy Slope: 0.005
Mannings n: 0.027
Shape: Trapezoid
Number of Subreaches: 2
Width: 3
Side Slope: 3
Channel Loss: None
End:

Reach: C.2BA
Last Modified Date: 29 July 2016
Last Modified Time: 13:25:58
Canvas X: 13413.084311893013
Canvas Y: 11607.068275270633
From Canvas X: 15538.680105655912
From Canvas Y: 12520.77702451859
Downstream: P.2BB

Route: Kinematic Wave
Channel: Kinematic Wave
Length: 100
Energy Slope: 0.005
Mannings n: 0.012
Shape: Circular
Number of Subreaches: 2
Width: 2
Channel Loss: None
End:

Subbasin: 2BB
Last Modified Date: 1 July 2016
Last Modified Time: 16:16:48
Canvas X: 16575.52671264449
Canvas Y: 8018.912474417995
Area: 0.00291
Downstream: C.2BB

Canopy: None
Plant Uptake Method: None

Surface: None

LossRate: SCS
Percent Impervious Area: 0.0
Curve Number: 80

Transform: SCS
Lag: 3.60
Unitgraph Type: STANDARD

Baseflow: None
End:

Reach: C.2BB
Last Modified Date: 29 July 2016
Last Modified Time: 13:23:07
Canvas X: 13413.084311893013
Canvas Y: 11607.068275270633
From Canvas X: 13717.16531196527
From Canvas Y: 7897.280074389091
Downstream: P.2BB

Route: Kinematic Wave
Channel: Kinematic Wave
Length: 100
Energy Slope: 0.005
Mannings n: 0.012
Shape: Circular
Number of Subreaches: 2
Width: 2
Channel Loss: None
End:

Subbasin: 2BC
Last Modified Date: 1 July 2016
Last Modified Time: 16:16:48
Canvas X: 10250.641911141538
Canvas Y: 8262.1772744758
Area: 0.00255
Downstream: P.2BB

Canopy: None
Plant Uptake Method: None

Surface: None

LossRate: SCS
Percent Impervious Area: 0.0
Curve Number: 80

Transform: SCS
Lag: 6.09
Unitgraph Type: STANDARD

Baseflow: None
End:

Reach: P.2BB
Last Modified Date: 29 July 2016
Last Modified Time: 13:24:06
Canvas X: 6358.405110216641
Canvas Y: 12154.414075400697
From Canvas X: 13413.084311893013
From Canvas Y: 11607.068275270633
Downstream: C.2BC

Route: Kinematic Wave
Channel: Kinematic Wave
Length: 700
Energy Slope: 0.005
Mannings n: 0.027
Shape: Trapezoid
Number of Subreaches: 2
Width: 3
Side Slope: 3
Channel Loss: None
End:

Reach: C.2BC
Last Modified Date: 29 July 2016
Last Modified Time: 13:27:37
Canvas X: 6115.140310158829
Canvas Y: 9052.78787466367
From Canvas X: 6358.405110216641
From Canvas Y: 12154.414075400697
Label X: -59.0
Label Y: 2.0
Downstream: O.2B

Route: Kinematic Wave

Channel: Kinematic Wave
Length: 100
Energy Slope: 0.005
Mannings n: 0.012
Shape: Circular
Number of Subreaches: 2
Width: 4
Channel Loss: None
End:

Junction: O.2B
Last Modified Date: 30 July 2016
Last Modified Time: 21:57:53
Canvas X: 6115.140310158829
Canvas Y: 9052.78787466367
Downstream: CoolPond
End:

Subbasin: 2DA
Last Modified Date: 1 July 2016
Last Modified Time: 16:16:48
Canvas X: 17198.998598498627
Canvas Y: -1757.9620139287435
Area: 0.00602
Downstream: P.2DA

Canopy: None
Plant Uptake Method: None

Surface: None

LossRate: SCS
Percent Impervious Area: 0.0
Curve Number: 80

Transform: SCS
Lag: 3.70
Unitgraph Type: STANDARD

Baseflow: None
End:

Reach: P.2DA
Last Modified Date: 15 June 2016
Last Modified Time: 15:05:07
Canvas X: 17658.926938518853
Canvas Y: -6838.545244216715
From Canvas X: 19772.49226240484
From Canvas Y: -6157.806019961932
Downstream: C.2D

Route: Kinematic Wave
Channel: Kinematic Wave
Length: 100
Energy Slope: 0.005
Mannings n: 0.027
Shape: Trapezoid
Number of Subreaches: 2
Width: 3
Side Slope: 3
Channel Loss: None
End:

Reach: C.2D
Last Modified Date: 5 July 2016
Last Modified Time: 14:13:10
Canvas X: 8897.406134285062
Canvas Y: -6572.885643172613
From Canvas X: 17658.926938518853
From Canvas Y: -6838.545244216715
Label X: -4.0
Label Y: -18.0
Downstream: O.2D

Route: Kinematic Wave
Channel: Kinematic Wave
Length: 100
Energy Slope: 0.005
Mannings n: 0.012
Shape: Circular
Number of Subreaches: 2
Width: 2
Channel Loss: None
End:

Junction: O.2D
Last Modified Date: 30 July 2016
Last Modified Time: 21:57:42
Canvas X: 8897.406134285062
Canvas Y: -6572.885643172613
Label X: -55.0
Label Y: 14.0
Downstream: CoolPond
End:

Reservoir: CoolPond
Last Modified Date: 3 August 2016
Last Modified Time: 17:51:46
Canvas X: 2671.3629510769133
Canvas Y: -12545.001881376855
Downstream: SiteOutfall

Route: Controlled Outflow
Routing Curve: Elevation-Storage
Initial Elevation: 19.13
Elevation-Storage Table: ES_Cool_Pond
Adaptive Control: On
Main Tailwater Condition: None
Auxiliary Tailwater Condition: None

Spillway: Broad-Crested Spillway
Spillway Outlet: Main
Spillway Crest Length: 63.29
Spillway Crest Elevation: 20.93
Spillway Coefficient: 3
End Spillway:

Evaporation Method: Zero Evaporation
End Evaporation:
End:

Junction: SiteOutfall

Last Modified Date: 3 August 2016
Last Modified Time: 17:52:11
Canvas X: 12742.493515525119
Canvas Y: -16678.674874247386
End:

Basin Schematic Properties:
Last View N: 13250.763321987739
Last View S: -17408.661171716536
Last View W: -27692.344123826642
Last View E: 50321.33890525724
Maximum View N: 13250.763321987739
Maximum View S: -17408.661171716536
Maximum View W: -27692.344123826642
Maximum View E: 50321.33890525724
Extent Method: Elements
Buffer: 5
Draw Icons: Yes
Draw Icon Labels: Name
Draw Map Objects: No
Draw Gridlines: No
Draw Flow Direction: No
Fix Element Locations: No
Fix Hydrologic Order: No
End:

APPENDIX G

FINAL COVER SYSTEM DRAINAGE LAYER DESIGN

Written by:	**V. Krishnan**	Date:	**06/10/16**	Reviewed by:	**B. Gross**	Date:	**07/14/16**
Client:	**Santee Cooper**	Project:	**Winyah Generating Station**	Project No.:	**GSC5242**	Task No.:	**01BT**

APPENDIX G

FINAL COVER SYSTEM DRAINAGE LAYER DESIGN

SOUTH CAROLINA LICENSED PROFESSIONAL ENGINEER No. 33535 SCOTT M. GRAVES 8/9/2016

SOUTH CAROLINA GEOSYNTEC CONSULTANTS, INC. No. 773 CERTIFICATE OF AUTHORIZATION

SEALED FOR
CALCULATION PAGES G-1
THROUGH G-14

PURPOSE

The purpose of this calculation package is to present the design of the geosynthetic drainage layer of the final cover system for the proposed Class Three Landfill at the Winyah Generating Station (WGS). The geocomposite drainage layer is composed of a high density polyethylene (HDPE) geonet core with a needlepunched non-woven geotextile bonded to its top and bottom surfaces (i.e., a double-sided geocomposite). The geocomposite drainage layer will be located between a 2-ft thick layer of soil capable of supporting native vegetation (sand or clay) and a 20-mil (minimum) thick textured linear low-density polyethylene (LLDPE) geomembrane.

The design criteria evaluated include: (i) filtration capability and specifications for the geotextile component of the geocomposite drainage layer; (ii) survivability specifications for the geotextile component; and (iii) hydraulic capacities of the geosynthetic drainage layer and testing conditions for verifying that the required capacities are achieved.

METHOD OF ANALYSIS

Geotextile Filtration

The filtration characteristics of the geotextile component of the geocomposite layer are evaluated using a retention criterion, a permeability criterion, a porosity criterion, and a thickness criterion based on methods proposed by Holtz et al. (1998) and Giroud (2010). These criteria are summarized in Table G.1.

Table G.1. Filtration Criteria for Geotextile Components (adapted from Holtz et al., 1998; and Giroud, 2010)

1. Retention Criterion

1.1 Soils with less than 50% particles < 0.075 mm (US Sieve No. 200)

Density index of the soil (Relative density)		Linear coefficient of uniformity of the soil	
		$1 < C'_u < 3$	$C'_u > 3$
loose soil	$I_D \leq 35\%$	$O_{95} \leq (C'_u)^{0.3}\, d'_{85}$	$O_{95} < \frac{9}{(C'_u)^{1.7}}\, d'_{85}$
medium dense soil	$35\% < I_D < 65\%$	$O_{95} \leq 1.5\,(C'_u)^{0.3}\, d'_{85}$	$O_{95} < \frac{13.5}{(C'_u)^{1.7}}\, d'_{85}$
dense soil	$I_D \geq 65\%$	$O_{95} \leq 2\,(C'_u)^{0.3}\, d'_{85}$	$O_{95} < \frac{18}{(C'_u)^{1.7}}\, d'_{85}$

1.2 Soils with more than 50% particles < 0.075 mm (US Sieve No. 200)

$O_{95} \leq 210\ \mu m$ (US Sieve No. 70)

2 Permeability Criterion

$k_{geotextile} \geq \max(i_{soil}\, k_{soil}, k_{soil})$

3. Porosity Criterion

Nonwoven geotextiles: $n_g \geq 55\%$

4. Thickness Criterion

Nonwoven geotextiles: $N_{constrictions} \geq 25$

Notes:
- O_{95} is the apparent opening size (AOS) of the geotextile
- C'_u = linear coefficient of uniformity = $\sqrt{d'_{100}/d'_0}$ where d'_{100} and d'_0 is the top and bottom extremities, respectively, of a line drawn through the soil particle-size distribution curve and tangent at d_{50}.
- d'_{85} is the "linear particle size" for which 85% of particles are finer by weight, derived from the straight line drawn through the soil particle-size distribution curve.
- I_D = relative density or density index = $(e_{max} - e)/(e_{max} - e_{min})$, where e = soil void ratio; e_{min} = soil minimum void ratio, and e_{max} = soil maximum void ratio.-
- $k_{geotextile}$ = geotextile hydraulic conductivity; k_{soil} = soil hydraulic conductivity; i_{soil} = hydraulic gradient in the soil next to the geotextile,
- porosity, n_g (dimensionless) is calculated as follows: $n_g = 1 - \mu_g/(\rho_g\, t_g)$, where: μ_g = geotextile mass per unit area, ρ_g = polymer density, and t_g = geotextile thickness

- Number of constrictions ($N_{constrictions}$) is calculated as follows: ($N_{constrictions}$) $=\mu_g/[\rho_g d_f\sqrt{1-n_g}]$, where: μ_g = geotextile mass per unit area; ρ_g = polymer density; d_f = geotextile fiber diameter; and n_g = geotextile porosity.

Geotextile Survivability

Survivability requirements (grab, tear, and puncture strengths) are considered so that the geotextile component of the geocomposite will have adequate resistance to stresses applied to the geotextile during construction (i.e., when concentrated stresses should be the highest), using the method presented in GRI-GT13 (2012). The procedure involves two steps: (i) establish the required degree of survivability as a function of subgrade conditions, type of construction equipment operation above the geotextile, and lift thickness using Table G.2; and (ii) establish the recommended minimum values of certain mechanical strength properties (i.e., grab strength, puncture resistance, and trapezoidal tear strength) using Table G.3. The survivability requirements are then compared to characteristics of geotextile products on the current market to check that products are available to meet the calculated minimum strengths.

Drainage Layer Hydraulic Capacity

The drainage layer hydraulic capacity design evaluation is performed using the design-by-function concept presented by Koerner (2005) and based on Darcy's equation (flow rate = hydraulic conductivity × hydraulic gradient × cross-sectional area of flow) for hydraulic flow in porous, saturated media. The approach herein then follows the design methodologies presented in Giroud et al. (2000) and GRI-GC8 (2013).

The design method involves the following steps:

Step 1) Calculate the required (design) transmissivity (θ_{req}) based on results of leachate generation calculations using the USEPA Hydrologic Evaluation of Landfill Performance (*HELP*) model.

Step 2) Apply a global factor of safety (FS) to find the allowable flow rate and corresponding "Long-Term In-Soil" (LTIS) transmissivity (θ_{LTIS}).

Step 3) Apply partial reduction factors (RFs) for creep, chemical clogging, and biological clogging to account for the long-term decrease in flow capacity behavior, and calculate the baseline flow rate and corresponding baseline transmissivity (θ_{100}).

Step 4) Determine the critical operational case for θ_{100} by comparing required θ_{100} to typical θ_{100} for biplanar geocomposites at various loading conditions.

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Step 5) Identify GRI-GC8 test conditions to measure θ_{100}. The resulting θ_{100} from Step 4 is a product specification for the baseline laboratory test transmissivity that should be achieved if tested in accordance with GRI-GC8, Part 6 (2013). Therefore, it is necessary to identify test conditions which simulate site-specific loading conditions and boundary conditions.

Step 6) Calculate the index transmissivity that corresponds to the baseline transmissivity from previous steps. Geocomposite manufacturers typically provide product index transmissivities based on laboratory tests in which the drainage layer is sandwiched between two steel plates as opposed to site specific boundary conditions. The index transmissivity is determined by applying a reduction factor to θ_{100} to account for geotextile/soil intrusion.

Table G.2. Required Degree of Survivability as a Function of Subgrade Conditions, Construction Equipment, and Lift Thickness (GRI-GT13)*

Subgrade Conditions	Low ground-pressure equipment (≤ 3.6 psi)	Medium ground-pressure equipment (> 3.6 psi, ≤ 7.3 psi)	High ground-pressure equipment (> 7.3 psi)
Subgrade has been cleared of all obstacles except grass, leaves, and fine wood debris. Surface is smooth and level so that any shallow depressions and humps do not exceed 18 in. in depth or height. All larger depressions are filled. Alternatively, a smooth working table may be placed.	Low	Moderate	High
Subgrade has been cleared of obstacles larger than small to moderate-sized tree limbs and rocks. Tree trunks and stumps should be removed or covered with a partial working table. Depressions and humps should not exceed 18 in. in depth or height. Larger depressions should be filled.	Moderate	High	Very High
Minimal site preparation is required. Trees may be felled, delimbed, and left in place. Stumps should be cut to project not more than ± 6 in. above subgrade. Fabric may be draped directly over the tree trunks, stumps, large depressions and humps, holes, stream channels, and large boulders. Items should be removed only if placing the fabric and cover material over them will distort the finished road surface.	High	Very High	Not Recommended

* Recommendations are for 6 to 12 in. initial lift thickness. For other initial lift thicknesses:

12 to 18 in.: reduce survivability requirement one level;
18 to 24 in.: reduce survivability requirement two levels;
> 24 in.: reduce survivability requirement three levels

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Table G.3. GRI-GT13 Geotextile Strength Property Requirements

			Geotextile Classification [1]					
			Class 1 (high)		Class 2 (moderate)		Class 3 (low)	
Tests	Test Methods	Units	Elongation < 50%	Elongation ≥ 50%	Elongation < 50%	Elongation ≥ 50%	Elongation < 50%	Elongation ≥ 50%
Grab Tensile Strength	ASTM D 4632	lb	315	203	248	158	180	113
Trapezoid Tear Strength	ASTM D 4533	lb	112	79	90	56	68	41
CBR Puncture Strength	ASTM D 6241	lb	630	440	500	320	380	230
Permittivity	ASTM D 4491	sec^{-1}	0.02	0.02	0.02	0.02	0.02	0.02
Apparent Opening Size	ASTM D 4751	in.	0.024	0.024	0.024	0.024	0.024	0.024
Ultraviolet stability [2]	ASTM D 7238	% Ret. @ 500 hrs	50	50	50	50	50	50

Notes: (1) All values are minimum average roll values (MARV) except AOS, which is a maximum average roll value (MaxARV) and UV stability which is a minimum average value.

(2) Evaluation to be on 2-in. strip tensile specimens after 500 hours exposure.

FILTRATION EVALUATION RESULTS

Geotextile Retention

The geotextile must have openings that are small enough to retain fine-grained soil particles so that they do not enter the geocomposite drainage layer, which could result in clogging or flow capacity reduction of the drainage layer. Therefore, the apparent opening size (AOS, hereafter referred to as O_{95}) of the geotextile must be less than a maximum value.

The geocomposite drainage layer in the final cover system will be overlain by a layer of soil capable of supporting native vegetation. The upper 6-in. of this soil layer will be topsoil, and the lower 18-in. will be protective cover. The Unified Soil Classifications specified for the protective cover are SP-SC, SC, and CL. The soil layer will be spread over the cover system geosynthetics with a medium ground-pressure bulldozer. It is anticipated that the soil placed with this technique will have a medium dense relative density.

The O_{95} is calculated depending on the type of soil used for the protective cover as follows:

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If the soil used for the protective cover is fine grained, i.e., more than 50 percent of particles are finer than 0.075 mm (U.S. Sieve No. 200) (e.g., a CL soil), then by applying the criterion in Table G.1, O_{95} is calculated as:

$$O_{95} \leq 210\ \mu m \text{ (U.S. Sieve No. 70)}$$

If, for example, an SP-SC soil having $C'_u = 1.5$ and $d'_{85} = 0.15$ mm (U.S. Sieve No. 100) is used as protective cover, then by applying the criterion in Table G.1, O_{95} is calculated as:

$$O_{95} \leq 1.5\ (C'_u)^{0.3}\ d'_{85}$$

$$O_{95} \leq 1.5\ (1.5)^{0.3}\ (0.15 \text{ mm}) = 0.25 \text{ mm} = 250\ \mu m \text{ (U.S. Sieve No. 60)}$$

Considering a more well-graded protective cover, for example, a SC soil having $C'_u = 10$ and $d'_{85} = 0.84$ mm (US Sieve No. 20), O_{95} is calculated as:

$$O_{95} < \frac{13.5}{(C'_u)^{1.7}}\ d'_{85}$$

$$O_{95} < \frac{13.5}{(10)^{1.7}}\ (0.84 \text{ mm}) = 0.23 \text{ mm} = 230\ \mu m \text{ (U.S. Sieve No. 60-70)}$$

The range of geotextile mass per unit areas anticipated for use as a filtration layer or drainage layer component are 6 to 16 oz/yd^2 (200 to 540 g/m^2). Typical O_{95} values for 6 to 16 oz/yd^2 geotextiles on the current market range from 90 to 850 µm (IFAI, 2015); thus, products are available that can meet this specification.

Geotextile Permeability

Based on the Unified Soil Classifications specified for the protective cover, it is anticipated that the hydraulic conductivity of the protective cover will fall within the range of 1×10^{-5} to 1×10^{-2} cm/s. The geotextile must have openings that are large enough to allow infiltrating water to pass through the retained soil/geotextile interface without significant flow impedance. Thus, the hydraulic conductivity or permeability of the geotextile must be greater than a minimum required value. The hydraulic gradient in the protective cover is assumed to be <10 based on typical values in Giroud (2010). A hydraulic gradient of 10 will be used in the calculations.

Applying the permeability criterion of Table G.1, the calculated hydraulic conductivity of the geotextile, $k_{geotextile}$, is:

$$k_{geotextile} \geq \max\ (i_{soil}\, k_{soil},\ k_{soil})$$

$$\geq 10 \times (1 \times 10^{-2} \text{ cm/s}) = 0.1 \text{ cm/s}$$

This requirement is achievable by most geotextiles. Note that some manufacturers report the permeability property as "permittivity" (Ψ), which is defined as Ψ=k/t. Based on the range of geotextile mass per unit areas and thicknesses anticipated for the project (6 to 16 oz/yd^2 (200 to 540 g/m^2) and 1.3 to 5.7 mm, respectively), typical $k_{geotextile}$ values (calculated from typical permittivities and thicknesses) for needlepunched non-woven geotextiles are 0.2 to 0.4 cm/s.

Geotextile Porosity

The geotextile filter must have enough openings so that blocking some of them will not significantly clog the geotextile and inhibit flow into the geonet. Thus, the porosity of the geotextile must be greater than a minimum value. As shown in Table G.1, for non-woven geotextiles, the geotextile porosity n_g is required to be:

$$n_g > 55\%$$

The porosity criterion requirements apply for the geotextile component of the geocomposite drainage layer. Geotextile porosity is not a property that is directly measured or reported by manufacturers, however it can be calculated as indicated in Table G.1 (i.e., $n_g = 1 - \mu_g/(\rho_g\ t_g)$). Typical resulting n_g values for non-woven geotextiles are 50 to 95%. Based on the geotextile density of polypropylene or polyethylene and the range of mass per unit areas and thicknesses anticipated for the project (6 to 16 oz/yd^2 (200 to 540 g/m^2) and 1.3 to 5.7 mm, respectively), the calculated n_g values range from approximately 80% to 90%, which is well in excess of the minimum porosity required to prevent clogging.

Geotextile Thickness

For non-woven geotextiles, such as those proposed for the final cover system geocomposite drainage layer, the geotextile filter must be thick enough to have a sufficient number of constrictions. From Table G.1, the number of constrictions, $N_{constrictions}$, needs to be at least 25.

The number of constrictions in non-woven geotextiles is a function of mass per unit area, porosity, polymer density, and geotextile fiber diameter:

$$N_{constrictions} = \mu_g/(\rho_g d_f \sqrt{1 - n_g})$$

Based on data for non-woven needlepunched geotextiles presented by Palmeira and Gardoni (2000) and Faure et al. (2006), as well as data compiled by Geosyntec from manufacturers, most non-woven needlepunched geotextiles that have at least 25 constrictions have a minimum thickness of 2.3 mm. Therefore, a minimum thickness of 2.3 mm will be specified for the geotextile filter.

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SURVIVABILITY EVALUATION RESULTS

Survivability refers to the ability of the geotextile to withstand the stresses during installation and handling in the field. The degree of survivability is first evaluated using Table G.2 with the anticipated installation conditions. The following conditions are conservatively assumed to apply: (i) smooth and level subgrade condition; (ii) initial lift thickness of protective cover placed above geotextile is 12 in.; and (iii) maximum equipment ground pressure of 5 psi (i.e., medium ground-pressure equipment is used). Using Table G.2, a "moderate" degree of survivability is used.

In the second step, the minimum required values for the mechanical properties of the geotextile are established from Table G.3 based on the "moderate" or "Class 2" survivability requirement. The chart provides minimum required values for two ranges of geotextile extensibility. Values were selected for the more extensible range because this range is applicable to non-woven materials that are required for the geotextile. These survivability requirements, which are outlined in Table G.3, apply for the geotextile component of the geocomposite drainage layer.

HYDRAULIC CAPACITY EVALUATION

Step 1) Calculate Required (Design) Transmissivity, θ_{req}

As presented in Appendix E-1, the *HELP* model was used to calculate the required (design) in-plane hydraulic conductivity and equivalent transmissivity of the geocomposite drainage layer of the final cover system. The required transmissivity is based on maintaining the peak daily average head on the underlying geomembrane less than or equal to the approximate thickness of the drainage layer. Maintaining the head within the geocomposite thickness is recommended to provide veneer stability of the final cover system on the side slope. The required (design) transmissivity, θ_{req}, was calculated based on the operational condition corresponding to the landfill at final grades after installation of the final cover system (Cases D-1 and D-2 of Appendix E-1) as 4.1×10^{-5} m^2/s.

Step 2) Calculate Allowable "Long Term In Soil" Transmissivity, θ_{LTIS}

The allowable "Long Term In Soil" transmissivity, θ_{LTIS} is calculated by applying a factor of safety to increase the minimum required transmissivity. A factor of safety (FS) of 2 was assumed for the final cover system drainage layer in the analysis.

$$\theta_{LTIS} = \theta_{req} \times FS \qquad \text{(Eqn. 1)}$$

The θ_{LTIS} was calculated for the final cover drainage layer as: $4.1 \times 10^{-5} \times 2 = 8.2 \times 10^{-5}$ m^2/s.

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Step 3) Calculate Baseline Geocomposite Transmissivity, θ_{100}

Factors which account for additional long-term transmissivity reduction due to creep, chemical clogging, and biological clogging were applied to determine the minimum baseline product transmissivity, θ_{100}, for laboratory testing results as shown in Eqns. 2 and 3.

$$\theta_{LTIS} = \frac{\theta_{100}}{RF_{CR}RF_{CC}RF_{BC}} \quad \text{(Eqn. 2)}$$

Where RF_{CR} = reduction factor for creep, RF_{CC} = reduction factor for chemical clogging and/or precipitation of chemicals, and RF_{BC} = reduction factor for biological clogging.

Creep is the long-term reduction in thickness of the drainage layer under a sustained compressive stress. For landfill final cover systems, Koerner (2005) recommends that reduction factors for creep range from 1.2 to 1.4. The reduction factor for creep for the final cover system geocomposite is assumed to be 1.4.

GRI (2013) provides guidance for clogging reduction factors for landfill final cover systems. Chemical and biological clogging can increase over time as infiltrating water passes through the geocomposite. GRI (2013) recommends a chemical clogging reduction factor between 1.0 and 1.2 and a biological clogging reduction factor between 1.2 and 3.5 at final conditions. Based on recommendations by GRI, the chemical clogging reduction factor is assumed as 1.2. The final cover geocomposite is potentially susceptible to biological clogging due to root penetrations from the vegetative cover, therefore the biological clogging factor is assumed as 3.0, which is close to the upper bound of the recommended range.

Rearranging Eqn. 2 and substituting θ_{LTIS} and the reduction factors above, we obtain the following equation:

$$\theta_{100} = \theta_{LTIS}\ RF_{CR}\ RF_{CC}\ RF_{BC} \quad \text{(Eqn. 3)}$$

The θ_{100} was calculated for each operational condition, as shown below.

Case Designation	θ_{LTIS} (m^2/s)	RF_{CR}	RF_{CC}	RF_{BC}	RF_{total}	θ_{100} (m^2/s)
Final Cover Drainage Layer	8.2×10^{-5}	1.4	1.2	3.0	5.0	4.1×10^{-4}

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Step 4) Calculate the Critical Operational Case for θ_{100}

Geosyntec contacted SKAPS Industries to obtain typical θ_{100} data for a common biplanar geocomposite on the market. The data correspond to the product, TN 270-2-8, a geocomposite with non-woven geotextile on both sides of the geonet. This does not constitute specification or endorsement of this product; it is merely intended to compare the required transmissivities to a commercially available product to check reasonableness of the design and availability of products. The TN 270-2-8 geocomposite transmissivity was measured at a gradient of 0.1 while sandwiched between soil and a geomembrane for a seating time of 100 hours under four different normal stresses.

To compare the required θ_{100} to the typical θ_{100} on the market, the normal stress on the geocomposite drainage layer should be calculated. The stress can be determined from the thickness of fill to be placed above the drainage layer as follows:

$$p = \gamma_{cover} \times h_{cover} \qquad \text{(Eqn. 4)}$$

where: p represents the normal stress; γ_{cover} represents the density of the protective cover soil, assumed as 120 pcf; and h_{cover} represents the thickness of the protective cover soil. Therefore, the stress for the final cover geocomposite is 120 pcf × 2 ft = 240 psf.

The required (minimum) θ_{100} is plotted versus the calculated stress in Figure G.1. The expected θ_{100} data for a typical biplanar geocomposite (SKAPS TN 270-2-8) is shown for reference. As shown in Figure G.1, the required θ_{100} for the final cover geocomposite is less than θ_{100} for a typical biplanar geocomposite at the corresponding stress condition. Therefore, geocomposite products are available that meet or exceed the minimum baseline transmissivity required.

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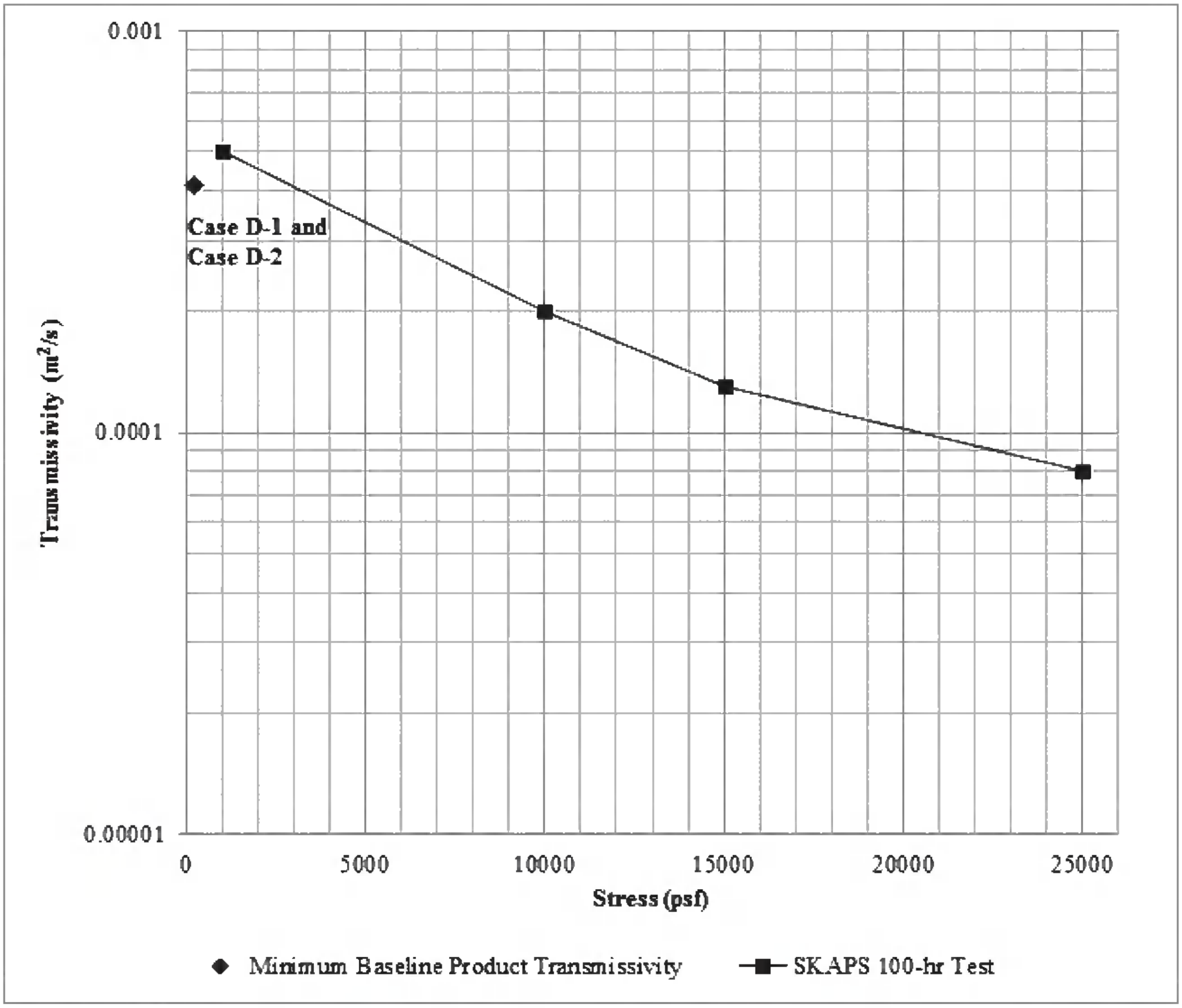

Figure G.1. Comparison of Required θ_{100} to Typical θ_{100} Test Results at Various Normal Stresses. Note: The typical product information shown does not constitute an endorsement of these products, nor does this require the use of any specific manufacturer or product. This information is presented for comparison purposes only.

Step 5) Identify Site-Specific Conditions for Evaluating θ_{100}

The testing conditions to be used in evaluating θ_{100} using GRI Standard GC8, Part 6 are: (i) the testing configuration (i.e., stratum configuration); (ii) the applied stress; and (iii) the hydraulic gradient. These conditions are specified below:

- The recommended testing configuration for transmissivity testing of the final cover system drainage layer should consist of a 20-mil LLDPE geomembrane on one side of the geocomposite specimen (to simulate site-specific final cover design) and soil representative of the protective cover on the other side of the geocomposite specimen.

- The stress to be applied in testing the final cover system drainage layer should be equivalent to the in-situ stress expected on the geocomposite as calculated in Step 4. Therefore, the stress on the final cover drainage layer geocomposite material to be used in determining θ_{100} is 240 psf.
- The geocomposite drainage layer slopes at about 33% on the side slope of the final cover system. Therefore, the hydraulic gradient to be used in determining θ_{100} for the geocomposite is 0.33.

Step 6) Determine Index Transmissivity, θ_{INDEX}, Based on θ_{100}

While the θ_{100} given above is suitable for use as a specification if desired, it is usually more convenient to report the transmissivity between two steel plates for a short duration test since manufacturers of geocomposite drainage materials often present the hydraulic capacities of their product in this manner. These transmissivities are usually higher than those that would be obtained using the site specific boundary conditions of soil on one side and a geomembrane on the other side. This is because the short duration test does not completely account for the time-delayed intrusion of the geotextile into the transmissive core resulting from the deformation of the geotextile under sustained loading. Additionally, the steel plate boundary condition of the short duration test will not account for a reduction in transmissivity due to particle migration into the transmissive core.

To compare the specified θ_{100} of the final cover drainage layer with index values reported by the manufacturer, factors can be applied to account for the reduction of the transmissivity that may be experienced due to intrusion and particulate clogging when testing the drainage layer with boundary materials other than steel plates. The index transmissivity, θ_{INDEX}, which accounts for intrusion and particulate clogging, can be determined as shown in Eqn. 5:

$$\theta_{INDEX} = \theta_{100} * RF_{INT}\, RF_{PC} \qquad \text{(Eqn. 5)}$$

Koerner (2005) recommends using an intrusion reduction factor (RF_{INT}) between 1.3 and 1.5. An intrusion factor of 1.4 is assumed for the final cover geocomposite drainage layer. The geotextile is expected to adequately retain particulates to avoid potential clogging of the transmissive core; however, a particulate clogging reduction factor (RF_{PC}) of 1.1 is applied. The index transmissivity, θ_{INDEX}, for the final cover geocomposite drainage layer is found to be:

$$\theta_{INDEX} = 4.1 \times 10^{-4}\ (m^2/s) \times 1.4 \times 1.1$$

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$$\theta_{INDEX} = 6.3 \times 10^{-4}\ m^2/s$$

CONCLUSIONS

Based on the evaluations herein, the following specification is recommended for the final cover system drainage layer geocomposite.

- Filtration of Geotextile Components:
  - Apparent Opening Size, $O_{95} \leq 210$ µm (U.S. Sieve No. 70)
  - Geotextile Water Permeability, $k_{geotextile} \geq 0.1$ cm/s for geotextile component of the geocomposite overlain by the protective cover
  - Geotextile Thickness, $t_{geotextile} \geq 2.3$ mm for geotextile component of the geocomposite overlain by the protective cover
- Survivability (Mechanical) Properties of Geotextile Components:
  - Grab Strength = 158 lbs
  - Trapezoid Tear Strength = 56 lbs
  - CBR Puncture Strength = 320 lbs
- Hydraulic Capacity (Transmissivity) of Geocomposite Drainage Layer
  - $\theta_{INDEX} = 6.3 \times 10^{-4}\ m^2/s$ (when tested between two steel plates with an applied stress of 240 psf at a gradient of 0.33) based on the site-specific design calculations.

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REFERENCES

Faure, Y.H, Baudoin, P., and Pierson, O., "A Contribution for Predicting Geotextile Clogging During Filtration of Suspended Solids", *Geotextiles and Geomembranes*, 24(1), 2006, pp. 11-20.

Geosynthetic Research Institute (GRI), "Standard Guide for Determination of the Allowable Flow Rate of a Drainage Geocomposite", *GRI Standard GC8,* 2013.

Geosynthetic Research Institute (GRI), "Standard Specification for Test Methods and Properties for Geotextiles Used as Separation Between Subgrade Soil and Aggregate", *GRI Standard GT-13*, 2012.

Giroud, J.P., "Development of Criteria for Geotextiles and Granular Filters", *Proceedings of the 9th International Conference on Geosynthetics*, Guaruja, Brazil, Vol. 1, 2010, pp. 45-64.

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

Holtz, R.D., Christopher, B.R., and Berg, R.R., "Geosynthetic Design and Construction Guidelines Participant Notebook, National Highway Institute (NHI) Course No. 13213", U.S. Department of Transportation, Federal Highway Administration, Rep. No. FHWA-HI-95-038, 1998.

Industrial Fabrics Association International (IFAI), *Geosynthetics Specifier's Guide 2015*, Roseville, MN, January 2015.

Kim, B., Prezzi, M., Salgado, R., "Geotechnical Properties of Fly and Bottom Ash Mixtures for Use in Highway Embankments", *Journal of Geotechnical and Geoenvironmental Engineering*, July 2005, pp. 914-924.

Koerner, R.M., "*Designing with Geosynthetics*", Fifth Edition, Prentice Hall, 2005, 767 p.

Palmeira, E.M. and Gardoni, M.G., "The Influence of Partial Clogging and Pressure on the Behavior of Geotextiles in Drainage Systems", *Geosynthetics International*, Vol. 7, Nos. 4-6, 2000, pp. 403-431.