

Prepared for



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RUN-ON AND RUN-OFF CONTROL SYSTEM PLAN LANDFILL AREA 1 WINYAH GENERATING STATION

Prepared by



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Project Number: 26009458

September 2026

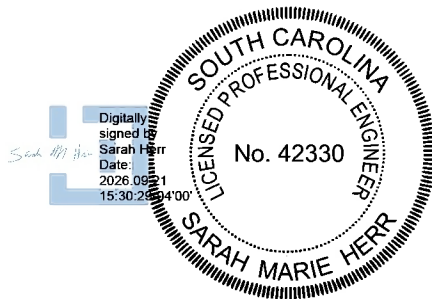


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1. INTRODUCTION

This Run-On and Run-Off Control System (ROROCS) Plan was prepared for the coal combustion residuals (CCR) Landfill Area 1 at Winyah Generating Station Class 3 Landfill (Winyah) as a periodic update to document how the run-on and run-off control systems have been designed and constructed to meet the regulatory requirements of 40 Code of Federal Regulations (CFR) 257.81 (run-on and run-off controls for CCR landfills). The supporting calculations for the run-on and run-off controls systems are provided as an appendix to this ROROCS Plan.

The initial ROROCS Plan was prepared and certified in 2018 when the CCR Landfill Area 1 was newly built and was placed into the facility operating record prior to the date of initial receipt of CCR. The first subsequent plan (i.e., the first periodic update, being completed within the required five-year timeframe) was prepared in November 2021. This ROROCS Plan is the second subsequent plan (i.e., the second periodic update, being completed within the required five-year timeframe), and will be placed in Winyah's operating record, as required by 40 CFR 257.105(g)(3).

On 9 October 2024, the South Carolina Department of Environmental Services (SCDES) approved closure construction of Landfill Area 1 pursuant to South Carolina Regulation 61-107.19, Part V, Subpart F, Section 258.60, and the landfill entered the post-closure care period. This periodic update has been refined since the 2021 Periodic Update to limit the scope of design, analysis, and control measure construction to be applicable to the post-closure conditions. Various calculations, such as leachate generation assessments, pump and pipe capacities, stability of the landfill, and contact water runoff have been excluded from this update.

2. REGULATORY REQUIREMENTS

2.1 Federal CCR Rule

As required by 40 CFR 257.53 and 40 CFR 257.81, the owner or operator of a CCR landfill must design, construct, operate, and maintain:

- 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; and
- 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.

In the context of the CCR Rule, "active portion" is defined in 40 CFR 257.53 as that part of a CCR unit that has received or is receiving CCR or non-CCR wastes and that has not completed closure in accordance with Section 257.102.

However, Landfill Area 1 has entered the post-closure care period. As required by 40 CFR 257.104, post-closure care maintenance must include maintaining the integrity and effectiveness of the final cover system, including preventing run-on and run-off from eroding or otherwise damaging the final cover.

2.2 Preamble to the Federal CCR Rule

Further clarification on the intent of this requirement of the CCR Rule is provided in the Preamble for the CCR Rule:

The owner or operator must design, construct, operate, and maintain the CCR landfill in such a way that any runoff generated from at least a 24-hour, 25-year storm must be collected through hydraulic structures, such as drainage ditches, toe drains, swales, or other means, and controlled so as to not adversely affect the condition of the CCR landfill. EPA has promulgated these requirements to minimize the detention time of run-off on the CCR landfill and minimize infiltration into the CCR landfill, to dissipate stormwater run-off velocity, and to minimize erosion of CCR landfill slopes. An additional concern with run-off from CCR landfills is the water quality of the run-off, which may collect suspended solids from the landfill slopes.

A description of run-on and run-off controls systems for Landfill Area 1 is included in subsequent sections of this ROROCS Plan.

3. DESIGN METHODOLOGY

3.1 Design Storms

Run-on and run-off controls systems were designed and constructed for hydraulic capacity for the 25-year, 24-hour storm event required by local and state regulations. Site-specific precipitation estimates were obtained from the National Oceanic and Atmospheric Administration (NOAA) Atlas 14, Volume 2, Version 3 for Georgetown, South Carolina, which indicates that the 25-year, 24-hour storm event generates 8.24 inches of precipitation at Winyah as presented in Table 1. Design calculations that support run-on and run-off control features against the assumed design storm are included in Appendix I of this ROROCS Plan. Final cover stormwater features consist of downdrains to convey stormwater off the cap to the perimeter channel that were designed and constructed considering the 25-year, 24-hour storm event published for Georgetown County as described within the approved landfill permit application and in the landfill operations fill plan.

3.2 Run-Off and Routing Methodology

HydroCAD Version 10.20-3h was used to apply the Soil Conservation Services (SCS) Technical Release 20 (TR-20) method to estimate runoff for post-development conditions. Modeling was applied to calculate surface water run-off volumes, peak flow rates, routing of rainfall event hydrographs through perimeter channels, and run-off discharge quantities supporting the final cover design of Landfill Area 1. Appendix I presents detailed drainage calculations and results, including a detailed discussion of the parameters used in the analysis.

3.3 Design Drawings

Final conditions and construction details for run-on and run-off control system features are presented within the approved Permit Drawings [Geosyntec, 2017] of the Winyah Generating Station Class Three Landfill Permit Application approved by the former South Carolina Department of Health and Environmental Control (DHEC) on 15 September 2017 (Permit #LF3-00042). Topographic information used in this evaluation was obtained from the 2024 survey provided by Santee Cooper.

4. RUN-ON CONTROL

Run-on is defined as water that may flow towards any portion of a landfill. This section summarizes the proposed surface water management system design and describes the drainage features and components of the facility to divert stormwater run-on from reaching Landfill Area 1.

Landfill Area 1 is located within the footprint of the closed Unit 2 Slurry Pond and is bounded by a perimeter berm providing access to the landfill. The landfill is topographically higher than the surrounding areas; therefore, run-on potential is limited to stormwater flows from the perimeter berm around the landfill footprint.

Calculations related to run-on from the perimeter road of the landfill are found in Appendix I.

5. RUN-OFF CONTROL

Run-off is defined as stormwater, leachate, or liquid flows generated from a project area (landfill and associated features). Stormwater run-off management encompasses two classifications of run-off:

- Contact water (run-off that has contacted CCR) and
- Stormwater (run-off that has not contacted CCR).

Because Landfill Area 1 is closed and no CCR placement or active waste management activities occur at the site, stormwater does not come into contact with CCR. As a result, contact water is not generated, and all run-off from the landfill is managed as non-contact stormwater. Run-off controls for stormwater at Landfill Area 1 are described below.

As part of this periodic update, runoff control will be confirmed for the landfill in accordance to the methods prescribed in the original 2018 plan and within 40 CFR 257.3-3.

5.1 Stormwater Run-Off

The landfill is closed and covered with a final cover system designed to minimize infiltration and prevent stormwater from contacting CCR. The landfill has side slopes graded at 3 horizontal to 1 vertical (3H:1V) between drainage benches and top deck slopes graded at 5 percent to maximize the removal of run-off. Drainage swales collect and divert surface water run-off into downdrains

to a perimeter channel at the toe of the landfill and then to the stormwater management areas. Stormwater run-off controls are capable of conveying flow from the 25-year, 24-hour storm event. Design calculations are provided in Appendix I.

A review of the current conditions of the landfill from survey information, as well as a visual inspection indicates that the landfill is being operated in accordance with the approved lift plans and permit documents to comply with federal regulations related to run-off control prevention measures.

6. CONCLUSIONS

As required by 40 CFR 257.81, the run-on control system at Landfill Area 1 was designed and built and is being maintained to prevent flow onto the closed CCR unit during peak discharge from the 25-year, 24-hour storm event, and the run-off control system at Landfill Area 2 was designed and built and is being maintained to collect and control at least the water volume from the 25-year, 24-hour storm event.

TABLES

Table 1. NOAA Atlas 14 Precipitation Data



NOAA Atlas 14, Volume 2, Version 3
Location name: Georgetown, South Carolina,
USA*

Latitude: 33.33°, Longitude: -79.3591°

Elevation: 25 ft**

* source: ESRI Maps

** source: USGS



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 & aerals](#)

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 (0.467-0.542)	0.585 (0.544-0.634)	0.665 (0.617-0.720)	0.759 (0.701-0.821)	0.855 (0.786-0.924)	0.940 (0.861-1.02)	1.02 (0.927-1.10)	1.10 (0.991-1.18)	1.19 (1.07-1.29)	1.28 (1.14-1.39)
10-min	0.801 (0.746-0.866)	0.936 (0.869-1.01)	1.06 (0.988-1.15)	1.21 (1.12-1.31)	1.36 (1.25-1.47)	1.50 (1.37-1.62)	1.62 (1.47-1.75)	1.74 (1.57-1.88)	1.88 (1.69-2.04)	2.01 (1.79-2.18)
15-min	1.00 (0.933-1.08)	1.18 (1.09-1.28)	1.35 (1.25-1.46)	1.54 (1.42-1.66)	1.73 (1.59-1.87)	1.90 (1.74-2.05)	2.04 (1.86-2.21)	2.19 (1.98-2.37)	2.37 (2.12-2.56)	2.53 (2.24-2.74)
30-min	1.37 (1.28-1.48)	1.62 (1.51-1.76)	1.91 (1.78-2.07)	2.22 (2.06-2.41)	2.56 (2.35-2.76)	2.86 (2.62-3.08)	3.13 (2.85-3.38)	3.41 (3.08-3.68)	3.76 (3.38-4.08)	4.09 (3.63-4.44)
60-min	1.71 (1.60-1.85)	2.04 (1.89-2.21)	2.45 (2.28-2.66)	2.90 (2.68-3.13)	3.40 (3.13-3.68)	3.87 (3.54-4.18)	4.31 (3.93-4.66)	4.78 (4.33-5.17)	5.40 (4.84-5.86)	5.97 (5.30-6.48)
2-hr	2.07 (1.92-2.24)	2.48 (2.29-2.69)	3.05 (2.81-3.30)	3.65 (3.36-3.94)	4.36 (3.99-4.70)	4.99 (4.55-5.38)	5.61 (5.08-6.05)	6.25 (5.62-6.74)	7.08 (6.31-7.65)	7.83 (6.92-8.48)
3-hr	2.22 (2.05-2.42)	2.66 (2.44-2.90)	3.28 (3.01-3.58)	3.96 (3.62-4.31)	4.77 (4.35-5.19)	5.53 (5.00-6.01)	6.28 (5.64-6.82)	7.08 (6.30-7.69)	8.16 (7.18-8.88)	9.15 (7.96-9.98)
6-hr	2.66 (2.44-2.90)	3.17 (2.90-3.48)	3.92 (3.58-4.30)	4.73 (4.31-5.18)	5.73 (5.19-6.27)	6.66 (5.99-7.28)	7.58 (6.77-8.29)	8.58 (7.59-9.37)	9.93 (8.68-10.9)	11.2 (9.66-12.3)
12-hr	3.10 (2.84-3.43)	3.71 (3.38-4.10)	4.61 (4.19-5.09)	5.59 (5.07-6.17)	6.82 (6.14-7.50)	7.97 (7.12-8.75)	9.14 (8.08-10.0)	10.4 (9.12-11.4)	12.1 (10.5-13.3)	13.8 (11.8-15.1)
24-hr	3.62 (3.32-3.97)	4.40 (4.04-4.83)	5.69 (5.20-6.23)	6.74 (6.14-7.38)	8.24 (7.47-9.01)	9.48 (8.55-10.4)	10.8 (9.69-11.8)	12.2 (10.9-13.4)	14.3 (12.6-15.7)	16.0 (14.0-17.6)
2-day	4.28 (3.93-4.68)	5.18 (4.76-5.68)	6.63 (6.07-7.25)	7.82 (7.14-8.54)	9.52 (8.64-10.4)	10.9 (9.86-11.9)	12.4 (11.1-13.6)	14.0 (12.5-15.4)	16.3 (14.4-18.0)	18.2 (16.0-20.1)
3-day	4.53 (4.16-4.97)	5.48 (5.03-6.01)	6.97 (6.38-7.62)	8.18 (7.47-8.94)	9.89 (8.99-10.8)	11.3 (10.2-12.4)	12.8 (11.5-14.0)	14.4 (12.9-15.8)	16.7 (14.8-18.3)	18.5 (16.3-20.4)
4-day	4.79 (4.40-5.25)	5.78 (5.31-6.34)	7.30 (6.69-8.00)	8.53 (7.80-9.34)	10.3 (9.33-11.2)	11.7 (10.6-12.8)	13.2 (11.9-14.4)	14.8 (13.2-16.2)	17.0 (15.1-18.7)	18.8 (16.6-20.8)
7-day	5.60 (5.17-6.09)	6.74 (6.23-7.33)	8.42 (7.76-9.14)	9.75 (8.97-10.6)	11.6 (10.6-12.6)	13.1 (11.9-14.2)	14.6 (13.3-15.9)	16.3 (14.7-17.7)	18.6 (16.7-20.3)	20.5 (18.3-22.4)
10-day	6.34 (5.89-6.85)	7.60 (7.06-8.21)	9.33 (8.65-10.1)	10.7 (9.88-11.5)	12.5 (11.5-13.5)	13.9 (12.8-15.0)	15.4 (14.1-16.6)	17.0 (15.5-18.3)	19.1 (17.3-20.6)	20.8 (18.7-22.6)
20-day	8.57 (8.02-9.18)	10.2 (9.55-10.9)	12.3 (11.5-13.2)	14.0 (13.1-15.0)	16.2 (15.1-17.3)	18.0 (16.7-19.2)	19.8 (18.3-21.2)	21.6 (19.9-23.2)	24.1 (22.1-25.9)	26.1 (23.8-28.1)
30-day	10.6 (9.95-11.2)	12.5 (11.8-13.3)	14.9 (14.0-15.8)	16.6 (15.6-17.6)	19.0 (17.8-20.1)	20.8 (19.4-22.0)	22.6 (21.0-23.9)	24.4 (22.6-25.9)	26.8 (24.8-28.5)	28.7 (26.4-30.6)
45-day	13.4 (12.6-14.2)	15.8 (14.9-16.7)	18.4 (17.4-19.5)	20.4 (19.3-21.6)	23.1 (21.7-24.4)	25.1 (23.5-26.6)	27.0 (25.3-28.7)	29.0 (27.0-30.8)	31.5 (29.3-33.6)	33.5 (31.0-35.8)
60-day	15.9 (15.0-16.8)	18.8 (17.7-19.8)	21.8 (20.5-23.0)	24.0 (22.6-25.3)	26.9 (25.3-28.4)	29.0 (27.3-30.7)	31.1 (29.2-32.9)	33.2 (31.0-35.2)	35.9 (33.5-38.2)	38.0 (35.3-40.5)

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.

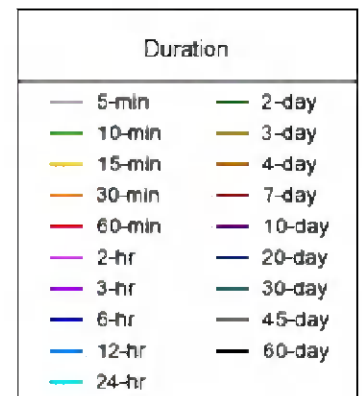
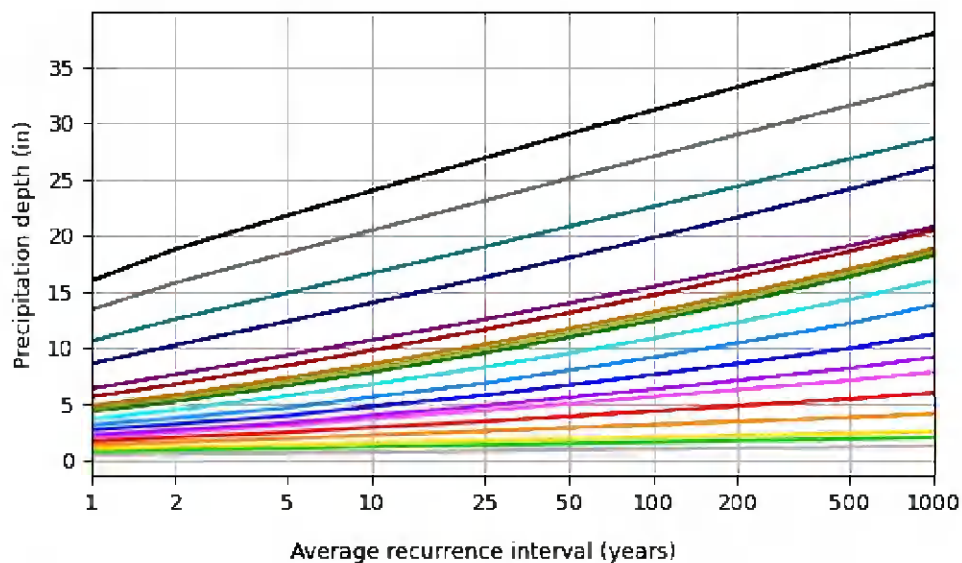
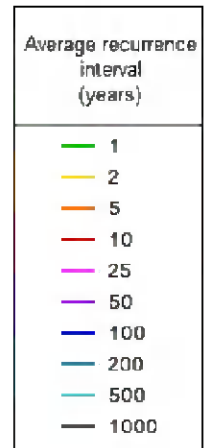
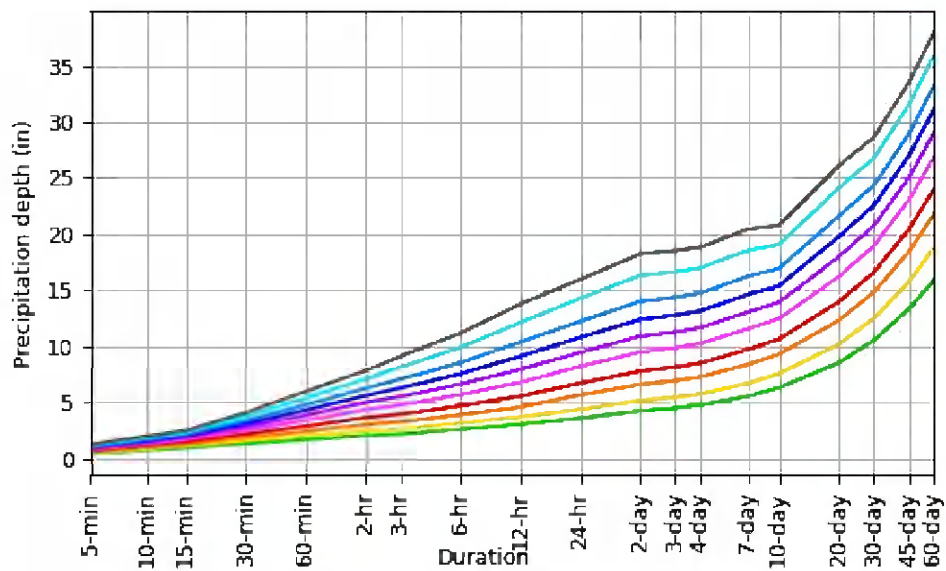
Please refer to NOAA Atlas 14 document for more information.

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PF graphical

PDS-based depth-duration-frequency (DDF) curves

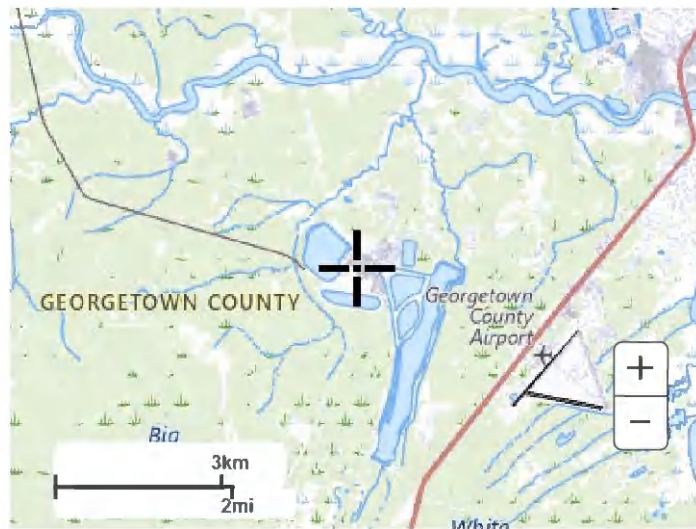
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Maps & aerials

Small scale terrain



Large scale terrain



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APPENDICES

APPENDIX I

Calculations for Run-On and Run-Off Perimeter Controls

Written by: A. Brand Date: 9/11/2026 Reviewed by: S. Herr Date: 9/21/2026

Client: Santee Cooper Project: Winyah Generating Station Project No.: 26009458 Phase No.: 005

SURFACE WATER MANAGEMENT SYSTEM DESIGN CLOSURE TURF® FINAL COVER SYSTEM



SEALED FOR

CALCULATION PAGES 1 THROUGH 79

1. INTRODUCTION

1.1 Purpose

The purpose of this calculation package is to present the analysis for the estimation of surface water runoff and evaluation of the final surface water management system hydraulic capacity at Landfill Area 1 at Winyah Generating Station (WGS) located in Georgetown County, South Carolina. On 9 October 2024, the South Carolina Department of Environmental Services (SCDES) approved closure construction of Landfill Area 1 pursuant to South Carolina Regulation 61-107.19, Part V, Subpart F, Section 258.60, and the landfill entered the post-closure care period.

The hydrologic analysis includes the calculation of peak discharge flow rates and total runoff volumes from the landfill during final conditions (i.e., during post-development conditions), as are the permanent surface water management system conveyance features. The results of the hydrologic calculations are used to evaluate the capacity of the hydraulic components of the landfill surface water management system.

1.2 Surface Water Management System - Overview

The 2024 survey provided by Santee Cooper shows the final grading plan for Landfill Area 1 (Santee Cooper, 2024). The Construction Drawings (Geosyntec, 2017) accompanying the Engineering Report in the approved permit application include a series of drawings that show the surface water management system features. Key features of the surface water

Written by: A. Brand Date: 9/11/2026 Reviewed by: S. Herr Date: 9/21/2026

Client: Santee Cooper Project: Winyah Generating Station Project No.: 26009458 Phase No.: 005

management system at Landfill Area 1 include:

- The final cover system ground surface has sideslopes inclined at three horizontal to one vertical (3H:1V) in-between drainage terraces and top surface slopes (top deck areas) inclined at a nominal 3 to 5 percent. The sideslope drainage terraces are spaced approximately every 30 feet (ft) vertically and with typical drainage profile slopes at 2 percent. Sheet flow runoff upgradient from each sideslope drainage terrace is captured in the terraces, where the water is conveyed to down drain pipes spaced periodically around the landfill.
- Down drain pipes outlet into either perimeter drainage reaches or into existing site drainage features (i.e., the discharge canal or cooling pond). At each location where down drain pipes outlet to the perimeter drainage channel, a concrete outlet apron is used. At each location where down drain pipes outlet directly to existing drainage features (discharge canal or cooling pond), riprap aprons are used to dissipate energy.
- Perimeter drainage reaches convey water to outlets into existing site drainage features (i.e., the intake canal, discharge canal, or cooling pond). Periodic or extensive drainage culverts are located along the drainage channels, as necessary. Construction conditions at Landfill Area 1 converted some perimeter drainage channels to subreaches, some of which are open channels or closed pipes, equivalent to the permitted hydraulic capacity. These surface water management features have been evaluated and their designations are depicted in Figure 1.
- The landfill includes 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 receive sheet flow runoff from the road along with some sheet flow from adjacent landfill sideslopes and conveys water to a culvert beneath the access road. A culvert conveys flow beneath the road where the landfill perimeter access roads cross the perimeter reach. The hydraulic capacity of the roadway culvert was also evaluated.

Figure 2 of this calculation package shows the layout of the final cover grading of Landfill Area 1, along with a delineation of drainage areas.

Written by: A. Brand Date: 9/11/2026 Reviewed by: S. Herr Date: 9/21/2026

Client: Santee Cooper Project: Winyah Generating Station Project No.: 26009458 Phase No.: 005

2. METHODOLOGY

2.1 HydroCAD Computer Model

Surface water discharges are estimated using *HydroCAD Version 10.20-3h* (HydroCAD, 2024). *HydroCAD* utilizes frequency-based precipitation events, in conjunction with watershed properties, to calculate peak runoff by several accepted methods. For precipitation-runoff-routing simulation, *HydroCAD* provides the following components:

- Precipitation-specification options can describe a frequency-based hypothetical precipitation event (i.e., design rainfall or storm event). For this analysis, the following hypothetical precipitation events were evaluated: the 25-year (4 percent annual chance of occurrence), 24-hour duration event (herein referred to as the 25-year, 24-hour event); and the 100-year (1 percent annual chance of occurrence), 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) Technical Release 20 (TR-20) method was applied in *HydroCAD* to calculate stormwater runoff volumes (SCS, 1982).
- *HydroCAD* was used to model the transformation of rainfall excess into direct runoff, accounting for watershed storage and flow timing. The Standard SCS Unit Hydrograph from the TR-20 method was selected for this analysis (SCS, 1982).

HydroCAD 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 under final closure conditions. The results are used to evaluate the capacity of the hydraulic design of the surface water management conveyances (i.e., top deck drainage terraces, sideslope drainage terraces, down drain pipes, and perimeter drainage channels and pipes).

2.1.1 Estimation of Time of Concentration for *HydroCAD* NRCS Curve Number Method

The method to estimate the sheet flow travel time was obtained from the United States Department of Agriculture (USDA) document *Urban Hydrology for Small Watersheds, Technical Release 55 (TR-55)* (USDA, 1986). Manning's kinematic solution is used for

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estimating travel time for sheet flow for flow distances less than 100 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 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 0.11 for ClosureTurf® was selected for the final cover system (WatershedGeo, 2019). Maximum flow lengths (L) were measured draining to each stormwater feature. As shown in Table 1, the rainfall depth for the 2-year, 24-hour frequency (P_{2-24}) at the site is 4.40 inches (NOAA, 2026).

Based on the designed conveyance system, runoff is 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 sheet flows 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 is 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

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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 (ft²);
- 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) of 0.027 was selected for open channels lined with vegetation as shown in Table 2 (Chow, 1959).

The velocities and times of concentration used in the analysis 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.

2.2 Drainage Terraces and Downdrains

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:

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$$Q = \frac{1.49}{n} AR^{\frac{2}{3}} S^{\frac{1}{2}}$$

where:

- Q = discharge (cfs),
- n = Manning's roughness coefficient,
- A = area of cross-section of flow (ft²),
- 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):

$$\tau_o = \gamma_w RS$$

where:

- τ_o = average tractive stress (lb/ft²),
- γ_w = unit weight of water (lb/ft³),
- 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. The tractive stress equation was also used to evaluate the channel lining. 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) from the *HydroCAD* modeling results relate to the contributing drainage areas for each down drain pipe feature. Each individual sideslope drainage terrace and top-deck drainage terraces constitute a portion of the total contributing area to a down drain pipe feature. Therefore, the total flow rate within the down drain 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 down drain pipe feature 1A1 has a total area of 3.81 acres. The subcatchment can be divided into four separate subareas corresponding to each sideslope drainage terrace within this

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subcatchment. The farthest downgradient sideslope drainage terrace on the east side of the downdrain pipe feature 1A1 (i.e., one of the four drainage terraces for this downdrain pipe) has an area of 0.71 acres. The expected flow rate within that drainage terrace is estimated as 0.71 acres divided by the total area (3.81 acres) times the peak flow rate (22.42 cfs for a 25-year event). This calculation results in an expected peak flow rate of 3.97 cfs for this specific sideslope drainage terrace. Similar calculations were conducted for every drainage terrace, and the combined runoff from all contributing subareas was used to calculate the critical peak flow rate required for designing adequate capacity within the drainage terrace.

2.3 Perimeter Drainage Reaches and Roadway Culverts

2.3.1 Perimeter Drainage Reaches

Final cover areas contributing to each perimeter drainage reach are modeled in *HydroCAD*. Each reach is designed to convey the peak surface water runoff rate corresponding to the 100-year (i.e., 1 percent annual chance of occurrence), 24-hour duration rainfall event flowing to the channel reach. The hydraulic capacity of each perimeter drainage channel or perimeter culvert is calculated and assessed by solving Manning's equation from Section 2.2. The peak velocity for the perimeter drainage channels calculated from Manning's equation was used to evaluate the channel lining.

In perimeter reach culverts that were flowing full during the 100-year, 24-hour event, the cumulative energy losses (friction losses, minor losses, and exit losses) were calculated to estimate the hydraulic grade line (HGL) along the system. Friction losses and minor losses were estimated and the 100-year, 24-hour HGL elevation was calculated at each junction box. Friction loss, or the energy required to overcome the roughness of the pipe, is expressed as (FHWA, 2013):

$$H_f = S_f L$$

where:

H_f = friction loss (ft),

S_f = friction slope (ft/ft), and

L = length of pipe (ft).

The equation for calculating the friction slope along each pipe segment is provided below (FHWA, 2013):

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$$S_f = \frac{(Q n)^2}{(K_Q D^{2.67})}$$

where: Q = flow (ft³/s),
 n = roughness coefficient,
 K_Q = 0.46 for English units, and
 D = pipe diameter (ft).

Minor losses due to the presence of junction boxes were calculated using the following equation (UDFCD, 2001):

$$H = K \left(\frac{V^2}{2g} \right)$$

where: H = minor loss (ft),
 K = bend loss coefficient = 0.025 from Figure 4 (UDFCD, 2001),
 V = pipe velocity (ft/s),
 g = acceleration due to gravity (ft/s²) = 32.2 ft/s².

Exit loss from the storm drain outfall was calculated using the following equation (FHWA, 2013):

$$H_o = K \left[\frac{V_o^2}{2g} - \frac{V_d^2}{2g} \right]$$

where: H_o = exit loss (ft),
 K = exit loss coefficient,
 V_o = average outlet velocity (ft/s),
 V_d = channel velocity downstream of outlet (ft/s), and
 g = acceleration due to gravity (ft/s²) = 32.2 ft/s².

An exit loss coefficient (K) of 1.0 is usually recommended by the Federal Highway Administration (FHWA) (2009); however, a loss coefficient of 0.5 was selected for this analysis because the pipe segment at the outlet is flowing at a depth approximately 50 percent

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of the pipe diameter. The downstream channel velocity (V_d) represents the open channel velocity between the pipe outlet headwall and the intake channel. A downstream velocity of 2 feet per second (ft/s) was selected to represent non-erosive velocities in this section of channel. This is assumed to be a conservative assumption since the actual downstream velocity will likely be greater prior to reaching the intake channel.

2.3.2 Roadway Culverts

The hydraulic capacity of the roadway culvert was assessed by utilizing the HY-8 Culvert Analysis Program v.7.5 (HY-8). The performance of the culvert is modeled and evaluated based on boundary conditions, culvert characteristics, and design flow rates for the 100-year, 24-hour rainfall event. Tailwater conditions were assumed to correspond to the calculated flow depth in the downstream channel.

2.3.3 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 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 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/s²).

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 3 provided by the FHWA (2006). The width of the riprap

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apron is selected as three times the culvert diameter based on the riprap apron detail presented in Figure 5 from FHWA (2006) or the bottom width of the channel for culverts that discharge directly to channels.

3. DESIGN PARAMETERS

The following data and assumptions were utilized to select the relevant engineering parameters to estimate surface water runoff and hydraulic capacity.

3.1 *HydroCAD*

3.1.1 Rainfall

The rainfall depths corresponding to 24-hour duration hypothetical precipitation event and 25- and 100-year frequency return periods for the site are 8.24 inches and 10.8 inches, respectively, as shown in Table 1 (NOAA, 2026). The design storm hyetograph is defined using a NRCS Type III rainfall distribution, which is selected based on Figure 6 (USDA, 1986).

3.1.2 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 or pipe) 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 NRCS Curve Number Loss Model was used to estimate the volume of runoff from a given subbasin. The NRCS 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 NRCS Unit Hydrograph lag time input parameters are included in the HEC-HMS output in Appendix I.
- **Curve Number (CN)** – The curve number for ClosureTurf® was obtained from the WatershedGeo Design Guidelines Manual (WatershedGeo, 2019). Final cover of the

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CCR landfill will have a CN = 95, as shown in Table 5. The curve number for the gravel access road (CN = 96) was obtained from Table 2-2 of TR-55 (USDA, 1986).

- 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 2 and Figure 8 summarize the Manning's coefficients used in this calculation package. A Manning's roughness coefficient value of 0.036 was selected for the drainage terraces which are proposed to be lined with ClosureTurf® overlain with riprap. A Manning's roughness coefficient value of 0.027 was selected for perimeter drainage channels which are proposed to be grass-lined. A Manning's roughness coefficient value of 0.009 was selected for HDPE down drain pipes.
- Perimeter Channel Reaches – Perimeter channels were modeled in *HydroCAD* as channel reaches. Reach parameters, including length, slope, cross-section geometry, and Manning's roughness coefficient, were developed from the available design plans to evaluate channel conveyance capacity and flow routing.

3.1.3 Nodal Network Diagrams

Figure 9 of this calculation package presents the nodal network diagram for the final landfill conditions of the landfill. The nodal network diagram shows the subbasins, reaches, and discharge locations.

3.2 Drainage Terrace and Downdrains

The hydraulic capacity of the drainage terraces and downdrains were evaluated to meet or exceed the applicable requirements of the SCDES 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

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stresses produced by the 25-year, 24-hour rainfall event; and

- the down drain pipes are designed to convey the 100-year, 24-hour rainfall event without flowing full (to avoid pressurized flow).

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 down drain pipe feature in Table 6 and Table 7, respectively, of this calculation package. Further discussion of the design parameters for each feature is provided in the sections that follow.

3.2.1 Top-Deck Drainage Terraces

The top-deck drainage terraces are designed as v-shaped channels (i.e., trapezoidal channels with bottom width equal to zero). The top-deck drainage terraces are formed by constructing tack-on berms near the edge of the top deck surface. As such, one side of the v-shaped channel is inclined at 5 percent based on the top deck surface grades, and the other side of the v-shaped channel is inclined at 3H:1V based on the constructed berm. The top deck drainage terraces have a constant depth of 1.5 ft and a longitudinal slope of approximately 0.5 percent.

A Manning's roughness coefficient of 0.036 was selected for the top-deck terraces, based on recommendations for excavated earth channels with riprap (Chow, 1959) as shown in Table 2. 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 8.

The critical top-deck drainage terrace, used as the basis for design, is located in subcatchment 1C1.1 of the Landfill Area 1 as shown on the attached Figure 10. This location is critical because it has the largest contributing drainage area of 3.83 acres. The resulting peak discharge rates from the 25-year and 100-year storm events are 24.2 cfs and 31.9 cfs, respectively.

3.2.2 Sideslope Drainage Terraces

The sideslope drainage terraces are designed as riprap-lined trapezoidal channels graded into the 3H:1V sideslopes of the final cover system. The sideslope drainage terraces have been modified to be trapezoidal in shape with 2H:1V sides on both the up-slope and down-slope sides of the terrace. The sideslope drainage terraces have a constant depth of 1.5 ft, bottom width of 6.0 ft, and a longitudinal slope of approximately 2 percent. As with the top-deck

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terraces, a Manning's roughness coefficient of 0.036 was used, and 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 8.

The critical sideslope drainage terrace, used as the basis for design, is located in subcatchment 1C1.2 of the Landfill Area 1 as shown on the attached Figure 10. This location is critical because it has the largest contributing drainage area of 2.33 acres. The resulting peak discharge rates from the 25-year and 100-year storm events are 14.7 cfs and 19.4 cfs, respectively.

3.2.3 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.

The channel lining material will be 6-inch (in.) 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 2. 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 8.

The critical access road drainage terrace, used as the basis for design, is located in subcatchment 1BA of the Landfill Area 1 as shown on the attached Figure 10. This location is critical because it has the largest contributing drainage area of 1.71 acres. The resulting peak discharge rates from the 25-year and 100-year storm events are 13.4 cfs and 17.6 cfs, respectively.

3.2.4 Downdrains

The downdrains are 18-in. 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 8. In general, the downdrain pipes follow the slope of the landfill sideslopes and as such are generally inclined at a longitudinal slope of 3H:1V. However, the slope flattens to 4 percent at locations where downdrain pipes cross the sideslope drainage terraces, as well as at the landfill perimeter.

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This much flatter 4 percent 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.

3.3 Perimeter Drainage Reaches and Roadway Culverts

3.3.1 Perimeter Drainage Reaches

Perimeter drainage channels are grass-lined trapezoidal shaped channels. The design parameters for each perimeter drainage channel reach, including channel geometry and calculated peak flow rates for the 25-year and 100-year, 24-hour rainfall events, are summarized in Table 10. A Manning's roughness coefficient of 0.027 was selected for perimeter drainage channels with grass-lining (Chow, 1959) as shown in Table 2 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 9 of this calculation package. 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. Perimeter drainage reach pipes/culverts at Landfill Area 1 were designed as reinforced concrete pipe (RCP). The design parameters for each perimeter drainage reach pipe/culvert, including geometry and calculated peak flow rates are also summarized in Table 10. A Manning's roughness coefficient of 0.012 was selected for concrete pipes as shown in Table 11 (Chow, 1959). A maximum permissible velocity of 18 ft/s was selected, based on Table 12 (USDA, 2007).

3.3.2 Roadway Culverts

The roadway culvert near the landfill perimeter is RCP. The design parameters for the roadway culvert, including inlet invert elevation, outlet invert elevation, slope, approximate length, and design flow rates for each culvert for the 25-year and 100-year, 24-hour rainfall events, are summarized in Table 13. A Manning's roughness coefficient of 0.012 was selected based on guidance from Chow (1959) as shown in Table 11. The culvert was 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.

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3.3.3 Riprap Apron Design

Riprap aprons were sized for the outflow of each proposed culvert, as well as for the down drain 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 14.

4. RESULTS

Table 15 of this calculation package presents a summary of the *HydroCAD* 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 15 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 Landfill Area 1. Results are provided for both the 25-year and 100-year storm events.

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 down drain pipe feature were calculated using the methodology described above. These calculation results are summarized in Table 16 (for drainage terraces) and Table 17 (for down drain pipe features). In addition, the down drain pipe feature configurations and associated number of pipes to be utilized at each sideslope drainage terrace were tabulated in Table 18.

Appendix 2 of this calculation package provides spreadsheets used for calculating the results for the down drain 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.

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- 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.

The depth of flow and velocity for the calculated discharge for each perimeter drainage reach channel/pipe during the design rainfall events were calculated using the methodology described above. Calculations for each perimeter drainage reach were performed using spreadsheet-based computations, with results summarized in Table 19. Spreadsheet results for each perimeter drainage reach are presented in Appendix 3 of this calculation package. For perimeter reach pipes/culverts flowing full during the 100-year, 24-hour event, the HGL elevation was calculated at each junction box to ensure the elevation of the HGL was less than the top of junction box, as shown in Table 20.

The performance of the roadway culvert from HY-8 modeling during the design rainfall events is presented in Table 21. The riprap apron median size, apron depth, and apron length for each culvert are presented in Table 14. 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.
- Perimeter drainage reach pipes are designed to convey the 25-year, 24-hour rainfall event without flowing full. Furthermore, for the 100-year rainfall event, the HGL elevation is less than the top of manhole elevation for all perimeter drainage reach pipes except junction W8 at the upstream end of the pipe system; this manhole will be bolted to prevent surcharge during a 100-year, 24-hour rainfall event.
- The roadway culvert design contains 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 14 for each riprap apron.

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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 is lined with riprap in these cases.

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**Table 1. Precipitation Frequency for Georgetown, South Carolina
(from NOAA, 2026)**



NOAA Atlas 14, Volume 2, Version 3
Location name: Georgetown, South Carolina,
USA*
Latitude: 33.33°, Longitude: -79.3591°
Elevation: 25 ft**
* source: ESRI Maps
** source: USGS



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 (0.487-0.542)	0.585 (0.544-0.634)	0.665 (0.617-0.720)	0.759 (0.701-0.821)	0.855 (0.788-0.924)	0.940 (0.861-1.02)	1.02 (0.927-1.10)	1.10 (0.991-1.18)	1.19 (1.07-1.29)	1.28 (1.14-1.39)
10-min	0.801 (0.746-0.866)	0.936 (0.869-1.01)	1.06 (0.989-1.15)	1.21 (1.12-1.31)	1.36 (1.25-1.47)	1.50 (1.37-1.62)	1.62 (1.47-1.75)	1.74 (1.57-1.88)	1.88 (1.69-2.04)	2.01 (1.79-2.18)
15-min	1.00 (0.933-1.08)	1.18 (1.09-1.26)	1.35 (1.25-1.46)	1.54 (1.42-1.68)	1.73 (1.59-1.87)	1.90 (1.74-2.05)	2.04 (1.86-2.21)	2.19 (1.99-2.37)	2.37 (2.12-2.56)	2.53 (2.24-2.74)
30-min	1.37 (1.29-1.48)	1.62 (1.51-1.78)	1.91 (1.79-2.07)	2.22 (2.08-2.41)	2.56 (2.35-2.78)	2.86 (2.62-3.08)	3.13 (2.85-3.36)	3.41 (3.09-3.68)	3.76 (3.39-4.08)	4.09 (3.63-4.44)
60-min	1.71 (1.60-1.85)	2.04 (1.89-2.21)	2.45 (2.28-2.66)	2.90 (2.68-3.13)	3.40 (3.13-3.68)	3.87 (3.54-4.18)	4.31 (3.93-4.69)	4.78 (4.33-5.17)	5.40 (4.84-5.86)	5.97 (5.30-6.48)
2-hr	2.07 (1.92-2.24)	2.48 (2.29-2.69)	3.05 (2.81-3.30)	3.65 (3.39-3.94)	4.36 (3.99-4.70)	4.99 (4.55-5.38)	5.61 (5.08-6.05)	6.25 (5.62-6.74)	7.08 (6.31-7.85)	7.83 (6.92-8.48)
3-hr	2.22 (2.05-2.42)	2.66 (2.44-2.90)	3.28 (3.01-3.58)	3.96 (3.62-4.31)	4.77 (4.35-5.19)	5.53 (5.00-6.01)	6.28 (5.64-6.82)	7.08 (6.30-7.86)	8.16 (7.18-9.88)	9.15 (7.96-9.98)
6-hr	2.66 (2.44-2.90)	3.17 (2.90-3.48)	3.92 (3.58-4.30)	4.73 (4.31-5.18)	5.73 (5.19-6.27)	6.66 (5.99-7.28)	7.58 (6.77-8.29)	8.58 (7.59-9.37)	9.93 (8.68-10.9)	11.2 (9.66-12.3)
12-hr	3.10 (2.84-3.43)	3.71 (3.38-4.10)	4.61 (4.19-5.09)	5.59 (5.07-6.17)	6.82 (6.14-7.50)	7.97 (7.12-8.75)	9.14 (8.08-10.0)	10.4 (9.12-11.4)	12.1 (10.5-13.3)	13.8 (11.8-15.1)
24-hr	3.62 (3.32-3.97)	4.40 (4.04-4.83)	5.69 (5.20-6.23)	6.74 (6.14-7.38)	8.24 (7.47-9.01)	9.48 (8.55-10.4)	10.8 (9.69-11.8)	12.2 (10.9-13.4)	14.3 (12.6-15.7)	16.0 (14.0-17.6)
2-day	4.28 (3.93-4.68)	5.18 (4.76-5.66)	6.63 (6.07-7.25)	7.82 (7.14-8.54)	9.52 (8.64-10.4)	10.9 (9.88-11.9)	12.4 (11.1-13.6)	14.0 (12.5-15.4)	16.3 (14.4-18.0)	18.2 (16.0-20.1)
3-day	4.53 (4.18-4.97)	5.48 (5.03-6.01)	6.97 (6.36-7.62)	8.18 (7.47-8.94)	9.89 (8.99-10.8)	11.3 (10.2-12.4)	12.8 (11.5-14.0)	14.4 (12.9-15.8)	16.7 (14.8-18.3)	18.5 (16.3-20.4)
4-day	4.79 (4.40-5.25)	5.78 (5.31-6.34)	7.30 (6.69-8.00)	8.53 (7.80-9.34)	10.3 (9.33-11.2)	11.7 (10.6-12.8)	13.2 (11.9-14.4)	14.8 (13.2-16.2)	17.0 (15.1-18.7)	18.8 (16.8-20.8)
7-day	5.60 (5.17-6.09)	6.74 (6.23-7.33)	8.42 (7.76-9.14)	9.75 (8.97-10.6)	11.6 (10.6-12.6)	13.1 (11.9-14.2)	14.6 (13.3-15.9)	16.3 (14.7-17.7)	18.6 (16.7-20.3)	20.5 (18.3-22.4)
10-day	6.34 (5.89-6.85)	7.60 (7.00-8.21)	9.33 (8.65-10.1)	10.7 (9.88-11.5)	12.5 (11.5-13.5)	13.9 (12.8-15.0)	15.4 (14.1-16.8)	17.0 (15.5-18.3)	19.1 (17.3-20.6)	20.8 (18.7-22.8)
20-day	8.57 (8.02-9.18)	10.2 (9.55-10.9)	12.3 (11.5-13.2)	14.0 (13.1-15.0)	16.2 (15.1-17.3)	18.0 (16.7-19.2)	19.8 (18.3-21.2)	21.6 (19.9-23.2)	24.1 (22.1-26.9)	26.1 (23.8-28.1)
30-day	10.6 (9.95-11.2)	12.5 (11.8-13.3)	14.9 (14.0-15.8)	16.6 (15.6-17.6)	19.0 (17.8-20.1)	20.8 (19.4-22.0)	22.6 (21.0-23.9)	24.4 (22.6-26.9)	26.8 (24.8-28.5)	28.7 (26.4-30.6)
45-day	13.4 (12.6-14.2)	15.8 (14.9-16.7)	18.4 (17.4-19.5)	20.4 (19.3-21.6)	23.1 (21.7-24.4)	25.1 (23.5-26.6)	27.0 (25.3-28.7)	29.0 (27.0-30.8)	31.5 (29.3-33.6)	33.5 (31.0-35.8)
60-day	15.9 (15.0-16.8)	18.8 (17.7-19.8)	21.8 (20.5-23.0)	24.0 (22.8-25.3)	26.9 (25.3-28.4)	29.0 (27.3-30.7)	31.1 (29.2-32.9)	33.2 (31.0-35.2)	35.9 (33.5-38.2)	38.0 (35.3-40.5)

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.

Table 2. 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 3. 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

¹D is the culvert rise.

**Table 4. Hydrologic Soil Groups for On-Site Soils
(from USDA, 2026)**

Map unit symbol	Map unit name	Rating	Acres in AOI	Percent of AOI
10	Leon sand, 0 to 2 percent slopes	A/D	144.4	3.2%
12A	Yauhannah loamy fine sand, 0 to 2 percent slopes	B/D	742.5	16.3%
13	Bladen loam, 0 to 2 percent slopes	C/D	367.9	8.1%
18	Cape Fear loam	C/D	15.1	0.3%
20	Centenary fine sand	A	226.5	5.0%
24B	Chisolm sand, 0 to 4 percent slopes	A	55.0	1.2%
25A	Wakulla sand, 0 to 2 percent slopes	A	48.8	1.1%
26A	Eulonia loamy fine sand, 0 to 2 percent slopes	C/D	555.6	12.2%
27	Rutlege sand	A/D	109.4	2.4%
28	Echaw sand	A	388.0	8.5%
33	Hobonny muck	A/D	89.0	2.0%
34	Johnston loam	A/D	83.4	1.8%
50	Lynn Haven sand	A/D	36.2	0.8%
54A	Chipley fine sand, 0 to 2 percent slopes	A	2.4	0.1%
55	Witherbee fine sand	A/D	461.7	10.1%
57	Grifton loamy fine sand	B/D	45.3	1.0%
58	Udorthents, loamy	B/D	399.6	8.8%
59	Wahee fine sandy loam	C/D	510.6	11.2%
61	Yemassee loamy fine sand	B/D	81.9	1.8%
W	Water		191.5	4.2%
Totals for Area of Interest			4,554.7	100.0%

Table 5. ClosureTurf® Hydrology Parameters
(from WatershedGeo, 2019)

ClosureTurf® Hydrology		
	TR-55 Data	
	Curve Number Depends on Rain Intensity	92 ¹ - 95
Sheet Flow	Manning's n	0.11
	Flow Length	100'
	2yr-24hr Rain	SCS
	Land Slope	design
	Flow Length	design
	Slope	design
Shallow Concentrated Flow	Surface (paved/unpaved)	Unpaved
	X-Sect Area	ft ²
	Wetted Perimeter	Linear Feet
	Channel Slope	ft/ft
Channel Flow	Manning's n	0.02 - HydroBinder; 0.03 - Sand Infill
	Flow Length	design
	Flow Length	design

1. CN ranging from 92 in High Intensity Rainfalls to 95 in normal rainfall events.

Table 6. 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	0.036	6.0	1.5	2:1	2:1	11.3	15.0
Top Deck	V-shaped	0.50	0.036	0.0	1.5	20:1	3:1	20.6	27.1
Access Road	V-shaped	8.00	0.036	0.0	1.5	3:1	2:1	13.4	17.6

Table 7. 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	16.1	21.3
1B1	2	1.5	4.0	0.009	27.0	35.5
1B2	2	1.5	4.0	0.009	30.1	39.7
1B3	2	1.5	4.0	0.009	30.6	40.4
1C1	2	1.5	4.0	0.009	38.9	51.3

Note:

1. The downdrain pipe feature consists of multiple downdrain pipes with varying lengths.

**Table 8. Typical Permissible Shear Stresses for Bare Soil and Stone Linings
(FHWA, 2005)**

Lining Category	Lining Type	Permissible Shear Stress	
		N/m ²	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 ≥ 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

¹Based on Equation 4.6 assuming a soil void ratio of 0.5 (USDA, 1987).

²Based on Equation 4.5 derived from USDA (1987)

³Based on Equation 6.7 with Shield's parameter equal to 0.047.

Table 9. 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 10. Design Parameter Summary for Perimeter Drainage Reaches

Perimeter Reach	Shape	Longitudinal Slope (ft/ft)	Manning's n	Inlet Invert (ft)	Outlet Invert (ft)	Length (ft)	Bottom Width (ft)	Diameter (in)	Depth (ft)	Side Slopes (H:V)	Lining	25-year Flow Rate Q_{25} (cfs)	100-year Flow Rate Q_{100} (cfs)
Perimeter Reach 1A1	Trapezoid	0.005	0.027	33.93	30.07	775.52	3.00	-	3.00	3:1	Vegetation	21.05	27.91
Pipe 1A1	Circular	0.005	0.012	30.07	29.80	49.00	-	36	-	-	-	21.03	27.88
Perimeter Reach 1B3a	Trapezoid	0.005	0.027	37.60	36.92	136.36	3.00	-	3.00	3:1	Vegetation	31.93	42.11
Perimeter Reach 1B3b	Trapezoid	0.014	0.027	36.92	33.79	229.96	3.00	-	3.00	3:1	Vegetation	31.72	41.85
Pipe 1B2a	Circular	0.010	0.012	33.13	31.99	110.00	-	30	-	-	-	31.66	41.77
Pipe 1B2b	Circular	0.002	0.012	31.99	31.29	294.00	-	42	-	-	-	31.39	41.42
Pipe 1B2c	Circular	0.005	0.012	31.29	26.55	927.00	-	48	-	-	-	87.93	116.5
Pipe 1B1a	Circular	0.005	0.012	26.50	25.21	243.00	-	54	-	-	-	87.69	116.17
Pipe 1B1b	Circular	0.005	0.012	25.21	24.53	129.00	-	60	-	-	-	93.21	123.43
Pipe 1B1c	Circular	0.020	0.012	24.45	22.30	104.00	-	60	-	-	-	113.44	150.53
Perimeter Reach 1C2	Trapezoid	0.005	0.027	36.48	34.72	352.59	2.00	-	1.70	3:1	Vegetation	3.05	4.08
Perimeter Reach 1C1	Trapezoid	0.005	0.027	34.72	31.72	600.00	2.00	-	1.70	3:1	Vegetation	9.03	12.13
Pipe 1C1a	Circular	0.005	0.012	31.72	28.34	669.00	-	30	-	-	-	8.72	11.73
Pipe 1C1b	Circular	0.008	0.012	28.34	25.48	357.00	-	48	-	-	-	54.35	72.26
Pipe 1C1c	Circular	0.010	0.012	22.25	21.21	103.00	-	48	-	-	-	54.20	72.08

**Table 11. 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	0.026	0.030

**Table 12. Allowable Velocity and Shear Stress for Selected Lining Materials
(from USDA, 2007)**

Boundary category	Boundary type	Allowable velocity (ft/s)	Allowable shear stress (lb/ft ²)
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 13. Design Parameter Summary for Roadway 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 1B	13.34	17.55	104	5.77	1 - 2-ft Diameter RCP	33.00	27.00	37.12

Table 14. Design Parameter and Results Summary for Riprap Aprons

Riprap Apron Feature	25-year Flow Rate Q₂₅ (cfs)	Number of Barrels	Culvert Diameter (ft)	Median Riprap Size (in)	Riprap Class	Apron Depth (ft)	Apron Length (ft)
Pipe 1BA	113.5	1	5.0	7.6	3	1.5	25.0
Pipe 1C1c	54.4	1	4.0	4.8	1	1.4	16.0

Note: See Table 3 for the riprap size associated with each Riprap Class.

**Table 15. Summary of Peak Flow Rates and Total Runoff Volumes at Outfall
Locations**

Outfall	Design Storm	Peak Flow (cfs)	Total Runoff (ac-ft)
1A	25-year	113.4	14.4
	100-year	150.5	19.2
1C	25-year	54.2	5.7
	100-year	72.1	7.6

Table 16. Summary of Calculated Results for Drainage Terraces

Drainage Feature	25-year, 24-hour Design Event				100-year, 24-hour Design Event				Channel Lining
	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)	
Sideslope	11.3	0.49	3.28	0.52	15.0	0.58	3.60	0.60	Riprap
Top Deck	20.6	0.99	1.82	0.15	27.1	1.10	1.95	0.17	Riprap
Access Road 1BA	13.4	0.90	6.55	2.09	17.6	1.00	7.01	2.31	Riprap

Table 17. 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	16.2	0.8	17.5	21.3	0.9	18.6	1
1B1	27.0	0.7	16.7	35.5	0.8	17.9	2
1B2	30.1	0.7	17.2	39.7	0.9	18.3	2
1B3	29.9	0.7	17.2	39.4	0.9	18.4	2
1C1	38.9	0.9	18.3	51.3	1.1	19.3	2

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 18. 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	N/A	N/A	N/A	N/A
1B1	2	2	1	N/A	N/A	N/A	N/A
1B2	2	2	1	N/A	N/A	N/A	N/A
1B3	2	2	1	N/A	N/A	N/A	N/A
1C1	2	2	1	N/A	N/A	N/A	N/A

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.

Table 19. Perimeter Drainage Reach 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	21.05	1.09	3.05	0.22	27.91	1.25	3.29	0.24
Pipe 1A1	21.03	1.34	6.89	-	27.88	1.58	7.40	-
Perimeter Reach 1B3a	31.93	1.33	3.41	0.25	42.11	1.52	3.67	0.28
Perimeter Reach 1B3b	31.72	1.05	4.92	0.57	41.85	1.20	5.30	0.63
Pipe 1B2a	31.66	1.56	9.85	-	41.77	2.50	9.11	-
Pipe 1B2b	31.39	2.70	9.75	-	41.42	4.00	8.81	-
Pipe 1B2c	87.93	2.70	9.75	-	116.5	4.00	8.81	-
Pipe 1B1a	87.69	2.46	9.85	-	116.17	4.50	9.53	-
Pipe 1B1b	93.21	2.40	10.01	-	123.43	2.84	10.72	-
Pipe 1BA	113.44	1.82	17.51	-	150.53	2.12	18.92	-
Perimeter Reach 1C2	3.05	0.48	1.85	0.10	4.08	0.55	2.00	0.11
Perimeter Reach 1C1	9.03	0.82	2.48	0.16	12.13	0.94	2.68	0.18
Pipe 1C1a	8.72	0.90	5.48	-	11.73	1.06	5.94	-
Pipe 1C1b	54.35	1.73	10.41	-	72.26	2.04	11.20	-
Pipe 1C1c	54.20	1.63	11.29	-	72.08	1.91	12.17	-

Table 20. 100-year, 24-hour Hydraulic Grade Line for Perimeter Reach Pipes/Culverts Flowing Full

Junctions/Inlets/Outlets		From Station	Length	Q ₁₀₀	Pipe Diameter	Velocity, v	n	Friction Slope	Incremental h _f	Cumulative h _f	K _j	Manhole Minor Loss		Exit Minor Loss		Total Minor Loss		Hydraulic Grade Line Elev.	Top of Manhole Elevation
From (ft)	To (ft)	(ft)	(ft)	(cfs)	(in)	(fps)	(-)	(ft/ft)	(ft)	(ft)	(-)	K _j x v ² /2g	K _o	K _o x [v _o ² /2g - v _d ² /2g]	(ft)	Incremental Minor Loss	Cumulative Minor Loss	(ft)	(ft)
West Storm Drain Inlet	W8	403.12	113.73	41.77	30	8.51	0.01	0.009	1.01	8.12	0.025	0.028	-	0	0	0.028	3.7	39.3	38.7
	W7	516.85	302.46	41.42	42	4.31	0.01	0.001	0.44	7.11	0.025	0.007	-	0	0	0.007	3.7	38.9	41.0
W7	W6	819.31	258.79	116.50	48	9.27	0.01	0.006	1.46	6.67	0.025	0.033	-	0	0	0.033	3.7	37.4	41.0
W6	W5	1078.1	344.01	116.50	48	9.27	0.01	0.006	1.94	5.21	0.025	0.033	-	0	0	0.033	3.6	35.4	41.0
W5	W4	1422.11	344.07	116.50	48	9.27	0.01	0.006	1.94	3.28	0.025	0.033	-	0	0	0.033	3.6	33.5	41.0
W4	W3	1766.18	128.92	116.17	54	7.30	0.01	0.003	0.38	1.34	0.025	0.021	-	0	0	0.021	3.6	33.0	41.0
W3	W2	1895.1	128.92	116.17	54	7.30	0.01	0.003	0.38	0.95	0.025	0.021	-	0	0	0.021	3.5	32.6	41.0
W2	W1	2034.0	136.58	123.43	60	11.60	0.01	0.002	0.26	0.57	0.025	0.052	-	0	0	0.052	3.5	32.4	35.0
W1	West Storm Drain Outlet	2160.6	107.58	150.53	60	21.22	0.01	0.003	0.31	0.31	-	-	0.5	3.46	2.00	3.464	3.5	28.5	-

Notes:

- 1) Junction boxes designated W1-W8 and their associated top of structure elevation represent construction conditions at Landfill Area 1.
- 2) Friction loss and exit minor loss estimated using guidance outlined in Urban Drainage Design Manual, Federal Highway Administration, US Department of Transportation, Hydraulic Engineering Circular No. 22, Third Edition, September 2009, revised August 2013.
- 3) Manhole minor losses were estimated using guidance outlined in Denver Urban Storm Drainage Criteria Manual, Volume 1, Prepared for the Urban Drainage and Flood Control District, Denver, Colorado, June 2001, Revised April 2008.

Table 21. Roadway Culvert Capacity Analysis Results

Culvert	25-year, 24-hour storm event				100-year, 24-hour storm event			
	Peak Flow Rate (cfs)	Pipe Velocity (fps)	Headwater Elev (ft)	25-year Freeboard (ft)	Peak Flow Rate (cfs)	Pipe Velocity (fps)	Headwater Elev (ft)	100-year Freeboard (ft)
Culvert 1B	13.3	15.17	33.65	3.47	17.6	15.46	33.75	3.37

FIGURES

Figure 1. Landfill Area 1 Perimeter Reach Designations

Figure 2. Landfill Area 1 Drainage Plan

Figure 3. Bend Loss Coefficient (from UDFCD, 2001)

Figure 4. Typical Geometry of Riprap Aprons at Culverts (from FHWA, 2006)

Figure 5. NRCS Rainfall Distributions (from USDA, 1986)

Figure 6. Soil Survey Map (from USDA, 2026)

Figure 7. Manning's n for HDPE (from Barfuss and Tullis, 1989)

Figure 8. *HydroCAD* Nodal Network

Figure 9. Landfill Area 1 Drainage Plan with Critical Terrace Areas

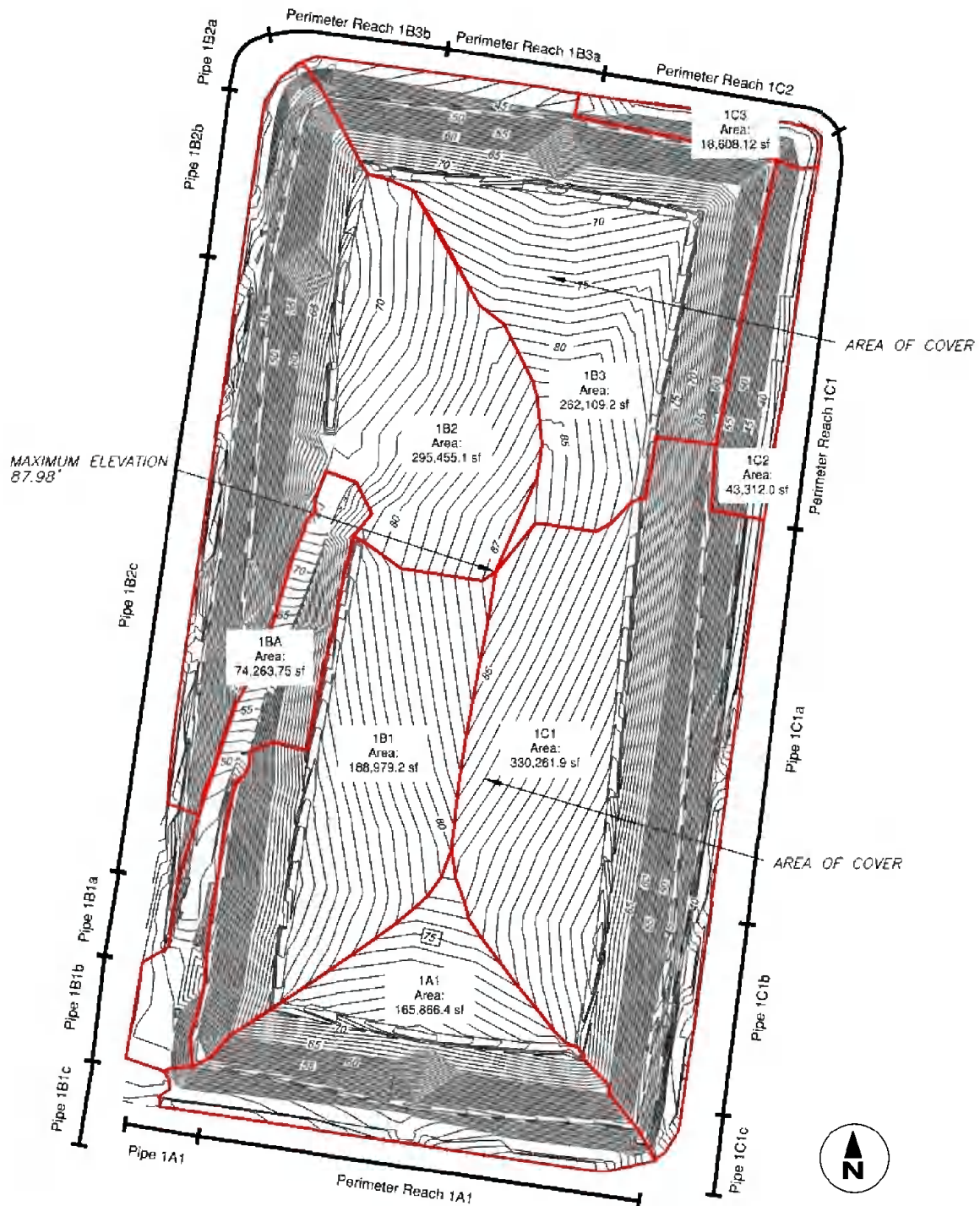


Figure 1. Landfill Area 1 Perimeter Reach Designations

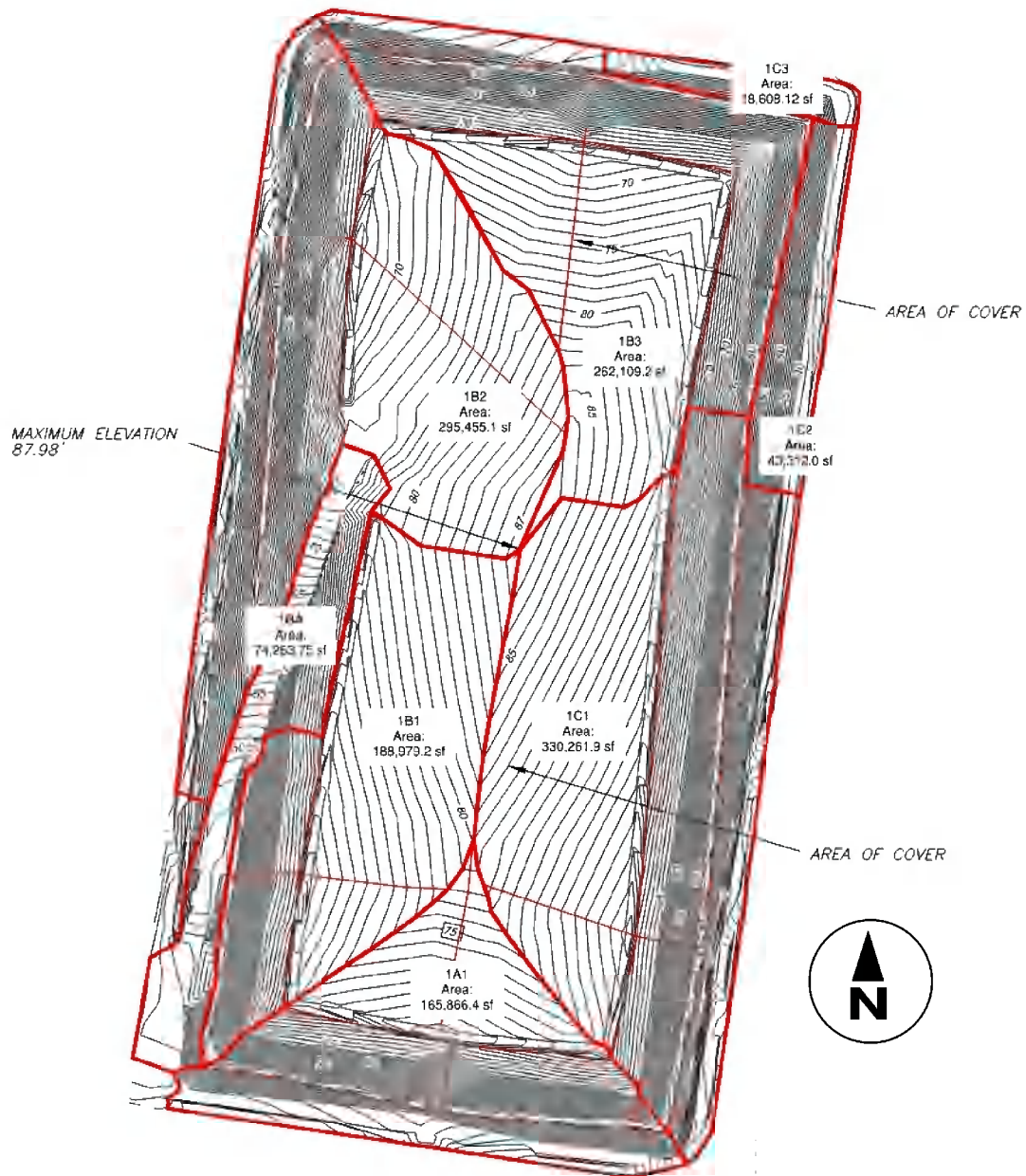


Figure 2. Landfill Area 1 Drainage Plan

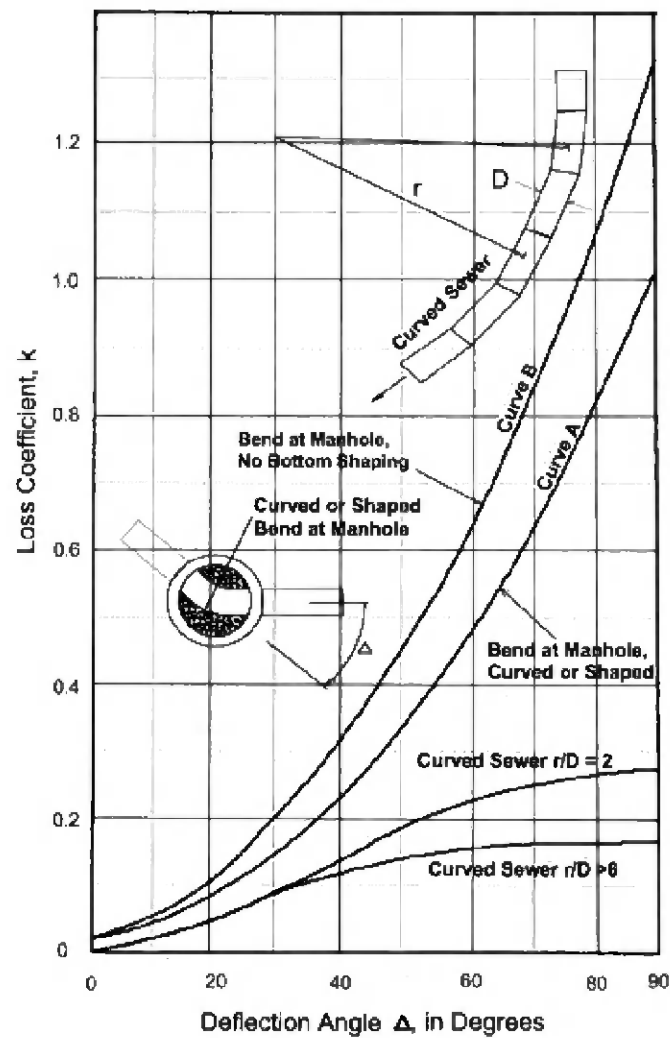


Figure 3. Bend Loss Coefficient (from UDFCD, 2001)

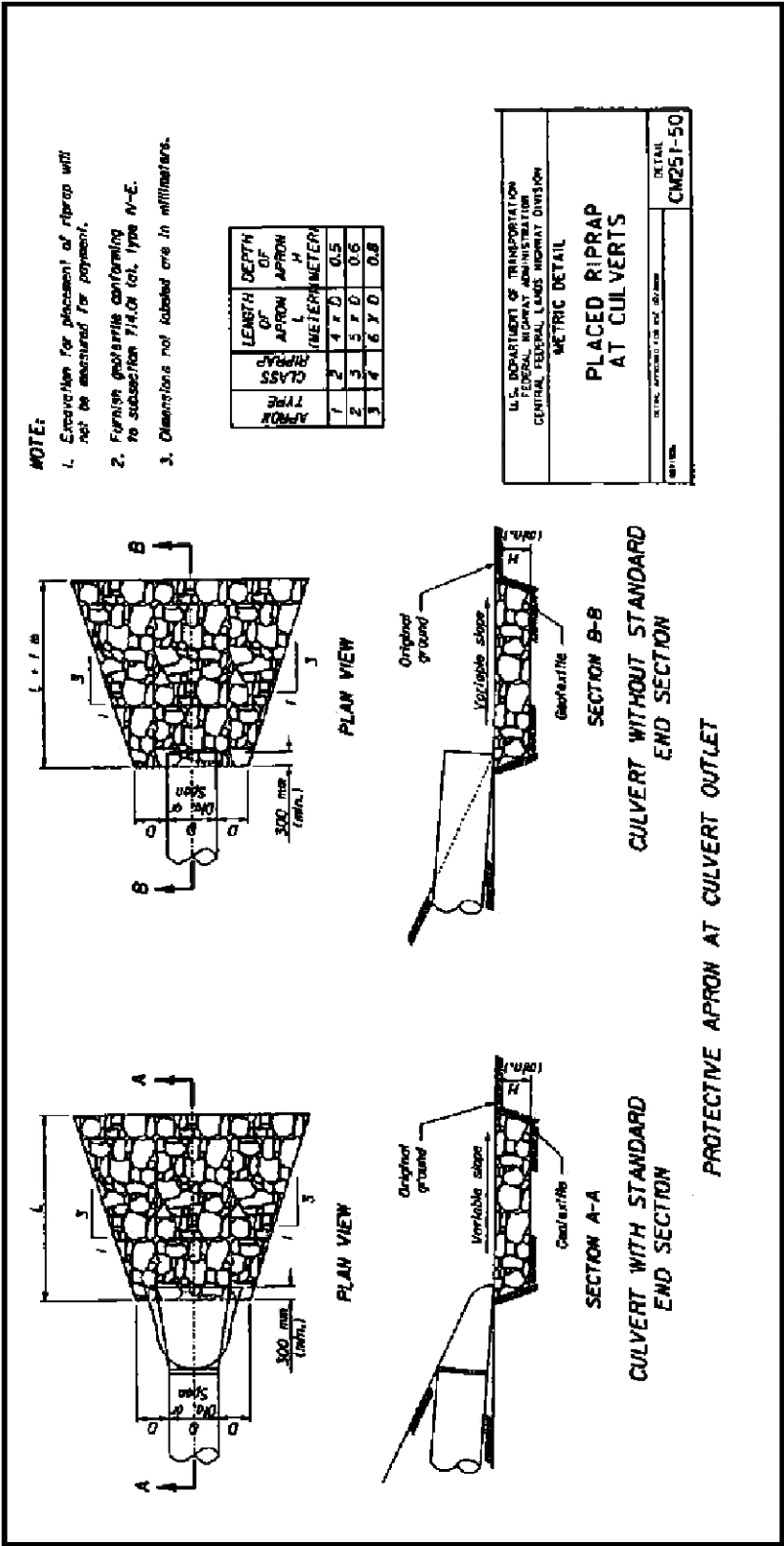


Figure 4. Typical Geometry of Riprap Aprons at Culverts
(from FHWA, 2006)

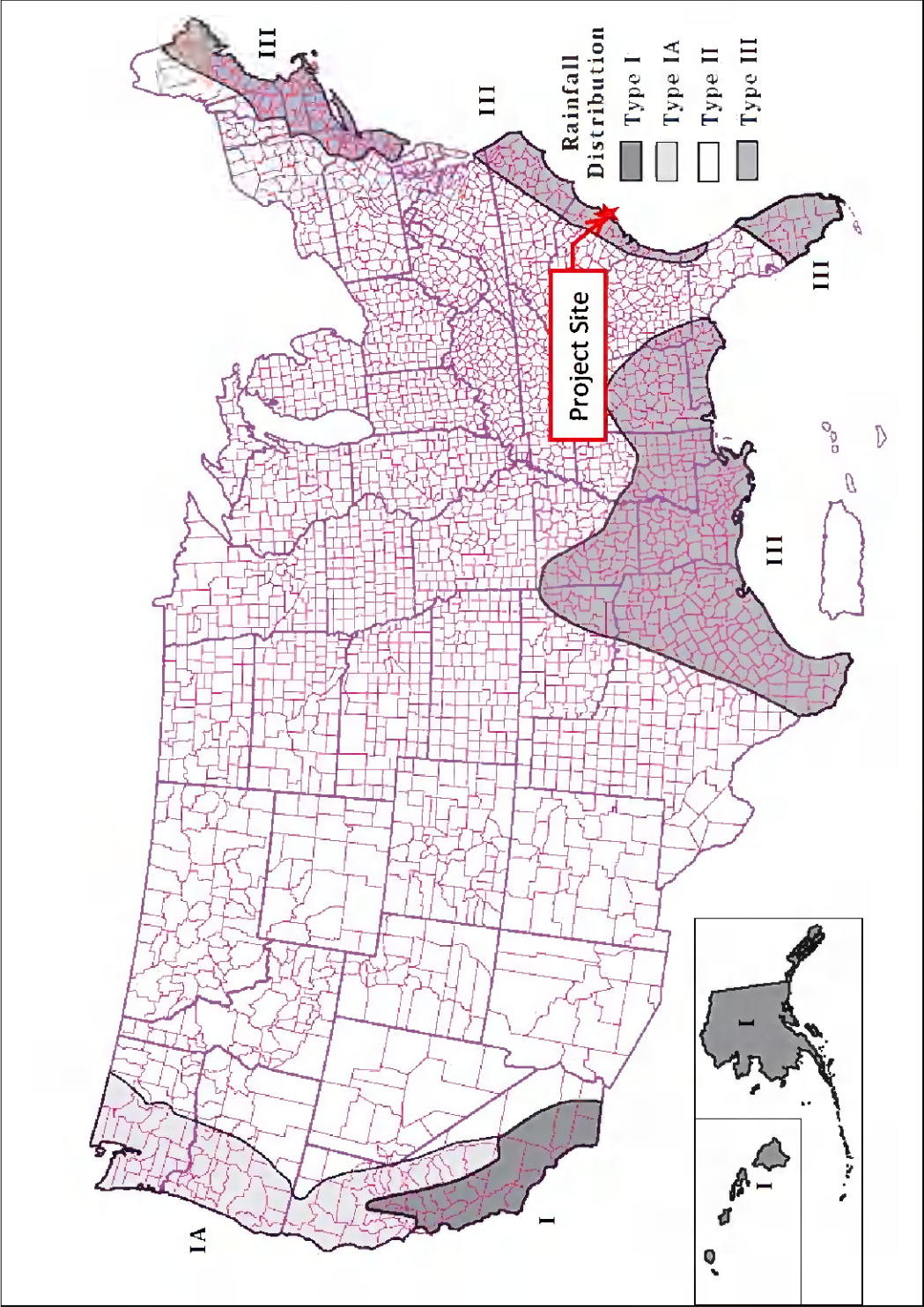


Figure 5. NRCS Rainfall Distributions (from USDA, 1986)

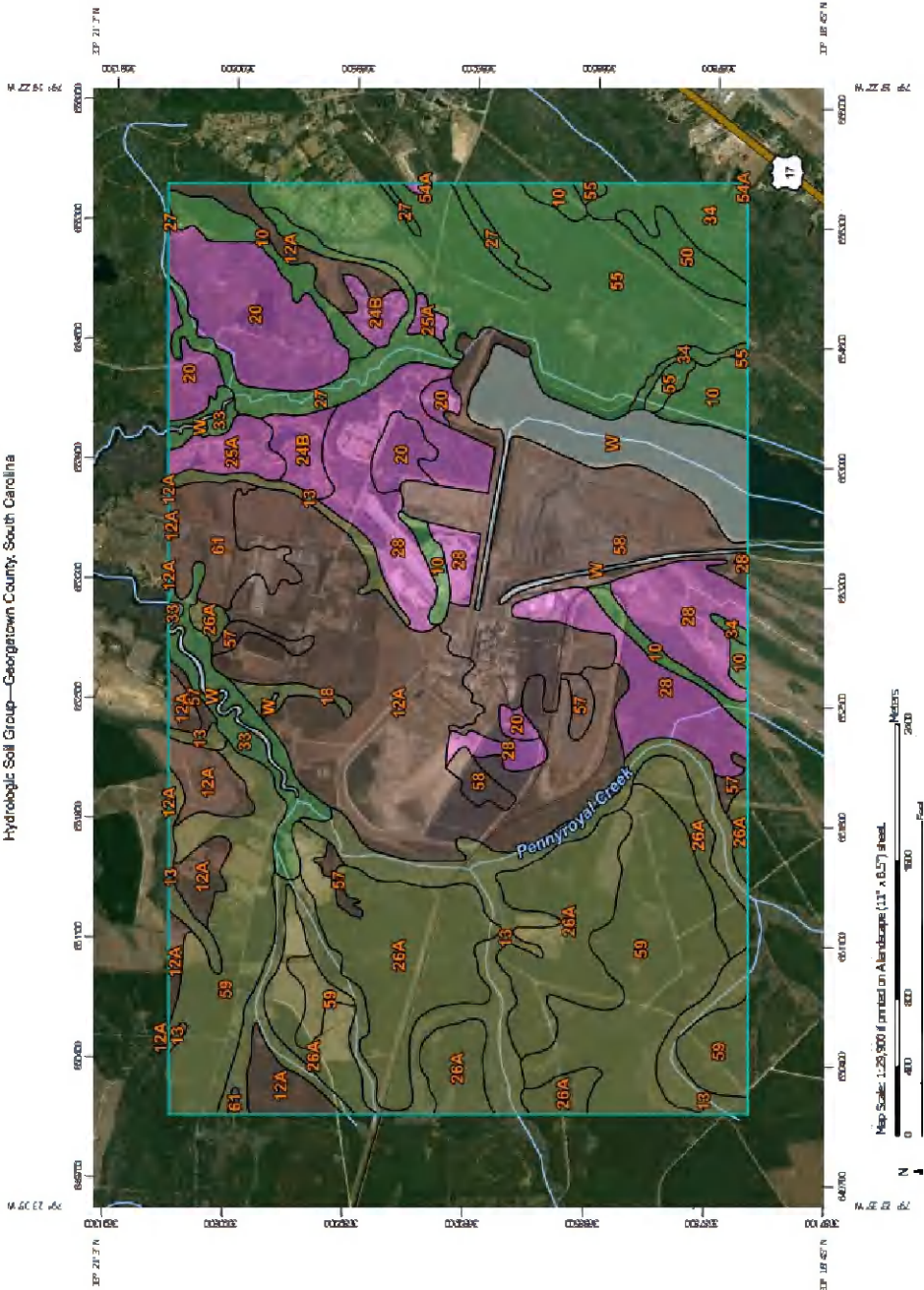
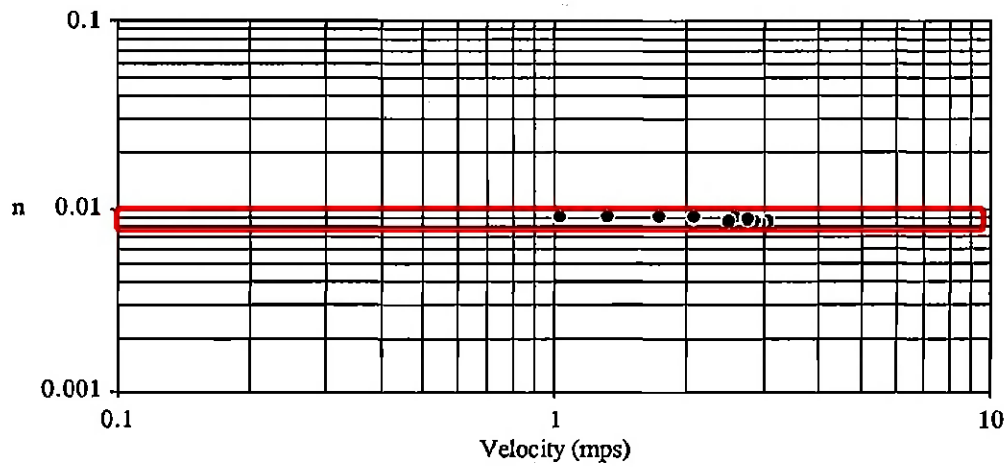


Figure 6. Soil Survey Map (from USDA, 2026)
Note: This map is provided for informational purposes only, but does not affect the analyses, which are based on the non-soil (ClosureTurf®) ground surface for this alternative final cover system option.



Manning n vs. Velocity for 300mm Diameter HDPE Pipe

Figure 7. Manning's n for HDPE
(from Barfuss and Tullis, 1989)

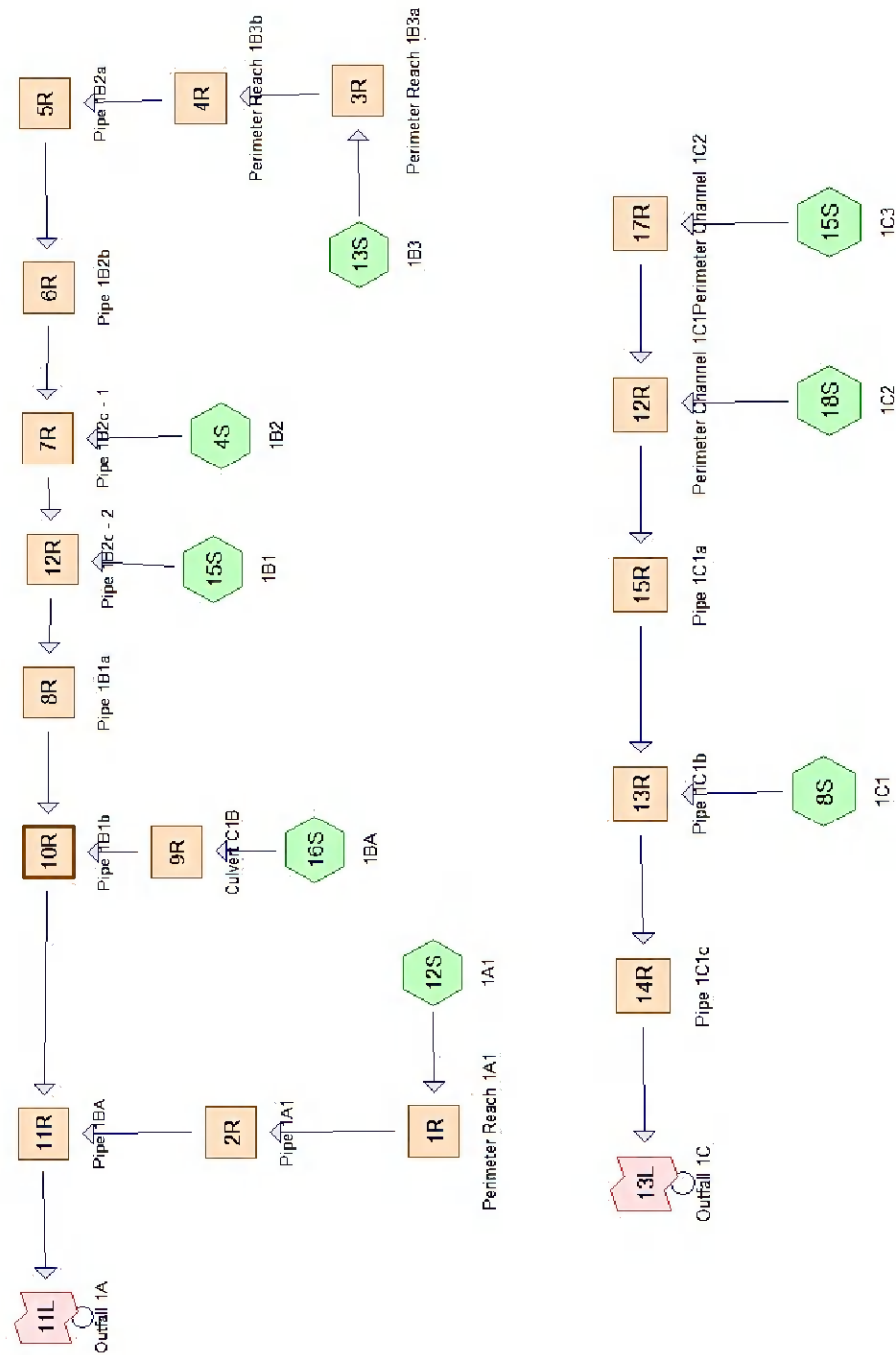


Figure 8. HydroCAD Nodal Network

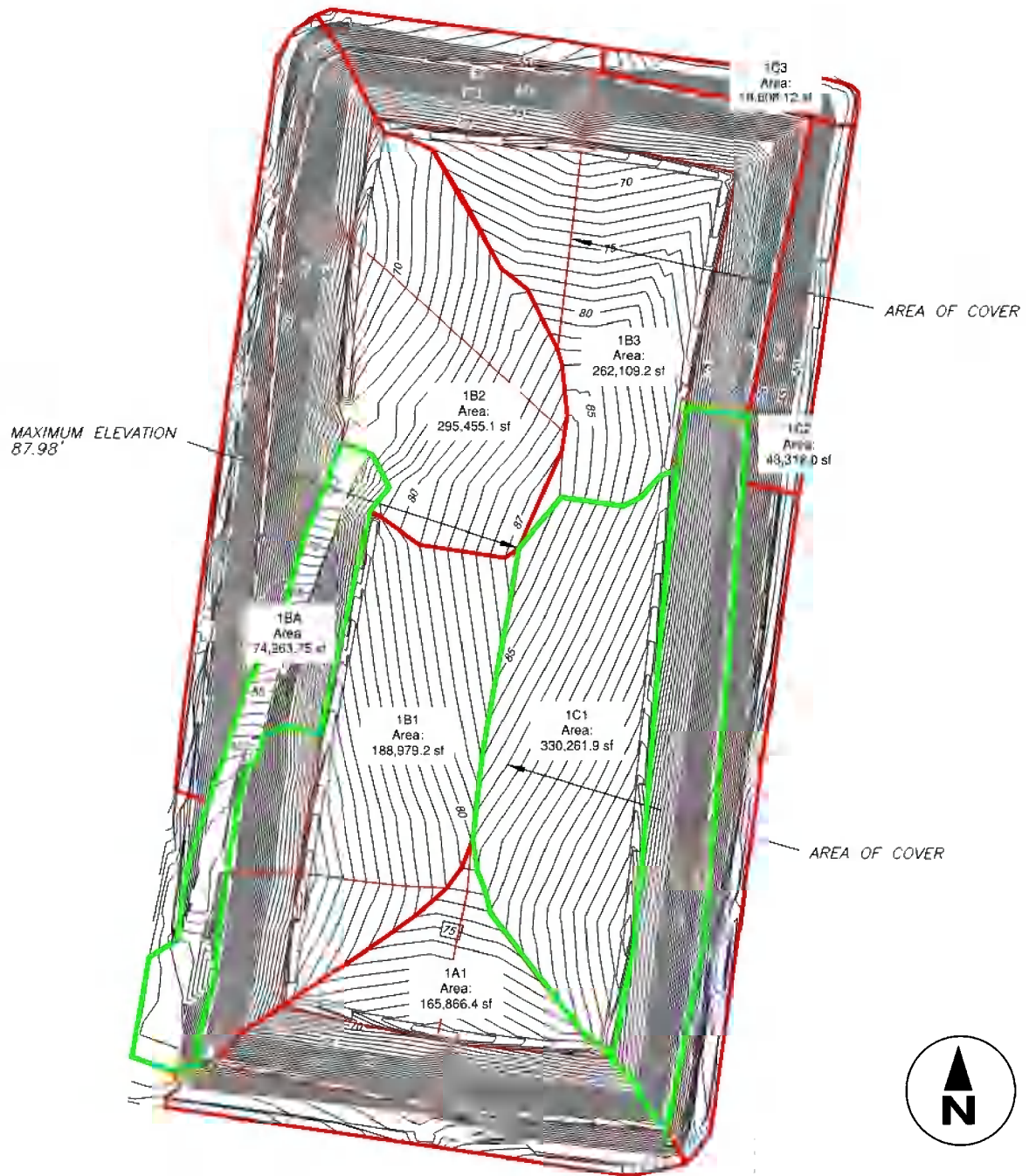


Figure 9. Landfill Area 1 Drainage Plan with Critical Terrace Area

APPENDIX 1

HYDROCAD HYDROLOGIC MODEL PARAMETERS

Table 1-1. HydroCAD Basin Input Parameters

POST-DEVELOPMENT CONDITIONS			Watershed Characterization			Sheet Flow			Open Channel Flow - Top Deck or Side Slope Drainage Terrace									
Subcatchment Designation	Area A (mi ³)	Area A (acres)	Initial Abstraction (in)	Curve Number	Impervious Cover (%)	Flow Length (ft)	Manning's n	Slope (ft/ft)	Time T _i (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 _i (min)
1A1	0.00595	3.808	0.11	95	0.00	288	0.11	0.05	11.65	337	1.0	2.00	4.47	0.45	0.036	0.020	3.42	1.64
1BA	0.00266	1.705	0.11	95	0.00	89	0.11	0.24	2.22	953	1.0	2.50	5.40	0.46	0.036	0.080	7.01	2.27
1B1	0.00678	4.338	0.11	95	0.00	251	0.11	0.06	9.03	542	1.0	11.50	23.19	0.50	0.036	0.005	1.83	4.93
1B2	0.01060	6.783	0.11	95	0.00	423	0.11	0.05	15.08	571	1.0	2.00	4.47	0.45	0.036	0.080	6.85	1.39
1B3	0.00940	6.017	0.11	95	0.00	397	0.11	0.05	15.68	758	1.0	2.00	4.47	0.45	0.036	0.020	3.42	3.69
1C1	0.01185	7.582	0.11	95	0.00	262	0.11	0.05	9.50	776	1.0	2.00	4.47	0.45	0.036	0.020	3.42	3.78
1C2	0.00155	0.994	0.11	95	0.00	86	0.11	0.29	1.97	0	1.0	2.00	4.47	0.45	0.036	0.020	3.42	0.00
1C3	0.00067	0.427	0.11	95	0.00	61	0.11	0.18	1.81	0	1.0	2.00	4.47	0.45	0.036	0.005	1.71	0.00

2-year, 24-hour Design Rainfall Depth = 4.40 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 = 2.0 H:V

Side Slope Drainage Terrace Right Side Slope = 2.0 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 = 95 represents ClosureTurf® (WatershedGeo, 2019).
- 2) Manning's roughness coefficient: $n = 0.11$ represents sheet flow for ClosureTurf® (WatershedGeo, 2019).
- 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) Manning's roughness coefficient: $n = 0.000$ represents HDPE (Barfiuss, 1989).
- 5) Travel Time (T_t) is calculated using Manning's kinematic solutions for sheet flow (USD4, 1986).

$$T_t = 0.007(nL)^{0.8} / (P^{2.34})^{0.5} S^{0.4}$$

$$V = (1.49r^{-2.5} S^{1/3}) / n \quad \text{where: } r = \text{hydraulic radius (ft) and is equal to } A/P \text{ [area (ft}^2\text{) / wetted perimeter (ft)]}$$
- 6) Open channel flow velocity is calculated using Manning's equation (USD4, 1986).
- 7) Design rainfall depth taken from NOAA Atlas 14, Volume 2, Version 3 for Georgetown, South Carolina.

Table 1-1 (Continued). HydroCAD Basin Input Parameters

Down Drain Pipe Flow										Open Channel Flow - Perimeter Channel									
Flow Length (ft)	Area A (ft ²)	Wetted P (ft)	Hydraulic Radius (ft)	Manning's n	Slope (ft/ft)	Velocity (ft/s)	Time T ₁ (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 ₁ (min)	Design T _c (min)	HydroCAD 25-yr Flow (cfs)	HydroCAD 100-yr Flow (cfs)
167	1.77	4.71	0.38	0.009	0.333	49.71	0.06	776	2.0	18.00	15.65	1.15	0.027	0.0050	4.28	3.02	16.4	22.41	29.54
0	3.14	4.71	0.67	0.009	0.050	28.25	0.00	0	2.0	18.00	15.65	1.15	0.027	0.0050	4.28	0.00	6.0	13.38	17.60
211	1.77	4.71	0.38	0.009	0.333	49.71	0.07	0	2.0	8.00	8.94	0.89	0.027	0.0050	3.62	0.00	14.0	26.96	35.54
161	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	16.5	39.83	52.50
154	1.77	4.71	0.38	0.009	0.333	49.71	0.05	366	2.0	18.00	15.65	1.15	0.027	0.0050	4.28	1.43	20.9	32.18	42.42
167	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	13.3	47.89	63.13
0	1.77	4.71	0.38	1.009	0.333	0.44	0.00	600	2.0	18.00	15.65	1.15	0.027	0.0050	4.28	2.33	6.0	7.76	10.33
0	1.77	4.71	0.38	0.009	0.333	49.71	0.00	353	2.0	18.00	15.65	1.15	0.027	0.0050	4.28	1.37	6.0	3.34	4.40
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									

Appendix 2

Drainage Feature Calculations

Design/Check: Trapezoidal/Triangular Channel

Methodology: Manning's Equation

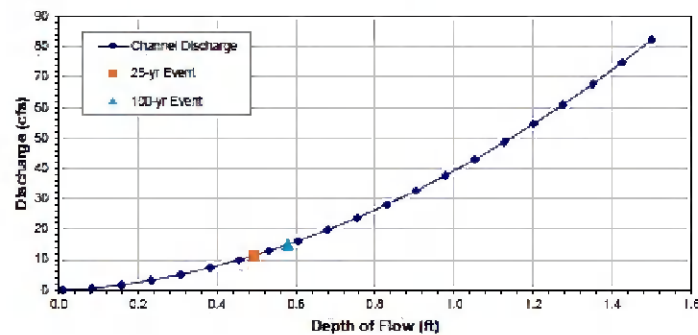
Project: Santee Cooper - Winyah Generating Station

Ditch ID: Side Slope Drainage Terrace

Peak Discharge, Q_{25} =	11.35	cfs (25-yr Event)
Peak Discharge, Q_{100} =	14.96	cfs (100-yr Event)
Bottom Width, B =	6.00	ft
Left Side Slope, Z_L =	2.0	horizontal : 1 vertical
Right Side Slope, Z_R =	2.0	horizontal : 1 vertical
Channel Depth, Y =	1.50	ft
Top Width, T =	12.0	ft
Manning's Roughness Coeff., n =	0.016	
Longitudinal Channel Slope, S_o =	0.0200	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	0.06	6.04	0.01	0.27	0.0	0.01	
0.08	0.52	6.38	0.08	1.10	0.6	0.10	
0.16	1.00	6.71	0.15	1.65	1.7	0.19	
0.23	1.51	7.04	0.21	2.10	3.2	0.27	
0.31	2.04	7.38	0.28	2.48	5.1	0.34	
0.38	2.59	7.71	0.34	2.83	7.3	0.42	
0.46	3.16	8.04	0.39	3.14	9.9	0.49	
0.53	3.75	8.38	0.45	3.43	12.9	0.56	
0.61	4.37	8.71	0.50	3.70	16.1	0.63	
0.68	5.01	9.04	0.55	3.95	19.8	0.69	
0.76	5.67	9.38	0.60	4.18	23.7	0.75	
0.83	6.35	9.71	0.65	4.41	28.0	0.82	
0.90	7.06	10.04	0.70	4.63	32.7	0.88	
0.98	7.79	10.38	0.75	4.83	37.6	0.94	
1.05	8.54	10.71	0.80	5.03	42.9	0.99	
1.13	9.31	11.04	0.84	5.22	48.6	1.05	
1.20	10.10	11.38	0.89	5.41	54.6	1.11	
1.28	10.92	11.71	0.93	5.59	61.0	1.16	
1.35	11.76	12.04	0.98	5.76	67.7	1.22	
1.43	12.62	12.38	1.02	5.93	74.8	1.27	
1.50	13.50	12.71	1.06	6.09	82.5	1.33	
0.49	3.45	8.21	0.42	3.28	11.31	0.52	Q (25-yr Event)
0.58	4.14	8.59	0.48	3.60	14.92	0.60	Q (100-yr Event)

Discharge versus Depth Relationship



Design/Check: Trapezoidal/Triangular Channel

Methodology: Manning's Equation

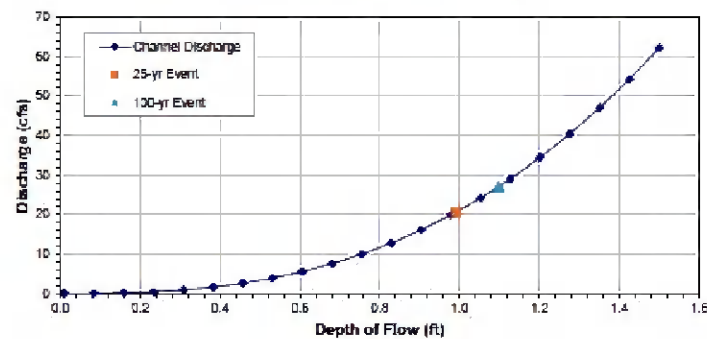
Project: Santee Cooper - Winyah Generating Station

Ditch ID: **Top Deck Drainage Terrace**

Peak Discharge, Q_{25}	=	20.59	cfs (25-yr Event)
Peak Discharge, Q_{100}	=	27.08	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	=	0.036	
Longitudinal Channel Slope, S_o	=	0.0050	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	0.23	0.00	0.08	0.0	0.00	
0.08	0.08	1.96	0.04	0.35	0.0	0.01	
0.16	0.29	3.69	0.08	0.54	0.2	0.02	
0.23	0.63	5.41	0.12	0.69	0.4	0.04	
0.31	1.09	7.14	0.15	0.84	0.9	0.05	
0.38	1.68	8.87	0.19	0.97	1.6	0.06	
0.46	2.40	10.60	0.23	1.09	2.6	0.07	
0.53	3.25	12.32	0.26	1.20	3.9	0.08	
0.61	4.22	14.05	0.30	1.31	5.5	0.09	
0.68	5.33	15.78	0.34	1.42	7.6	0.11	
0.76	6.56	17.51	0.37	1.52	10.0	0.12	
0.83	7.91	19.23	0.41	1.62	12.8	0.13	
0.90	9.40	20.96	0.45	1.71	16.1	0.14	
0.98	11.01	22.69	0.49	1.81	19.9	0.15	
1.05	12.75	24.42	0.52	1.90	24.2	0.16	
1.13	14.62	26.14	0.56	1.99	29.0	0.17	
1.20	16.62	27.87	0.60	2.07	34.4	0.19	
1.28	18.74	29.60	0.63	2.16	40.4	0.20	
1.35	20.99	31.33	0.67	2.24	47.0	0.21	
1.43	23.37	33.05	0.71	2.32	54.3	0.22	
1.50	25.88	34.78	0.74	2.40	62.2	0.23	
0.99	11.28	22.97	0.49	1.82	20.56	0.15	Q (25-yr Event)
1.10	13.88	25.47	0.54	1.95	27.08	0.17	Q (100-yr Event)

Discharge versus Depth Relationship



Design/Check: Trapezoidal/Triangular Channel

Methodology: Manning's Equation

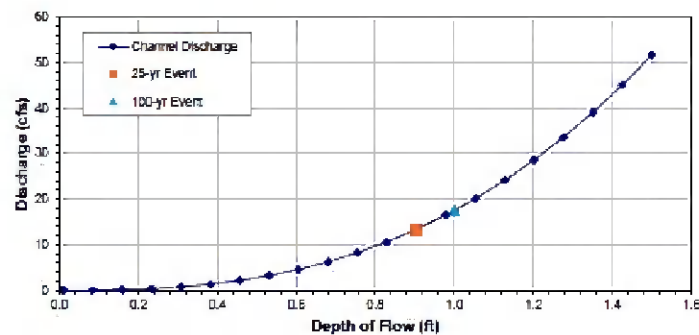
Project: Santee Cooper - Winyah Generating Station

Ditch ID: 1BA Access Road Drainage Terrace

Peak Discharge, Q_{25}	=	13.38	cfs (25-yr Event)
Peak Discharge, Q_{100}	=	17.60	cfs (100-yr Event)
Bottom Width, B	=	0.00	ft
Left Side Slope, Z_L	=	3.0	horizontal:1 vertical
Right Side Slope, Z_R	=	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	=	0.0300	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	
0.02	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	2.04	4.88	0.42	6.55	13.38	2.09	Q (25-yr Event)
1.00	2.50	5.40	0.46	7.01	17.55	2.31	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, S _o =	0.040	ft/ft
Manning's n=	0.009	
Density of flowing liquid, rho=	1.94	slugs/ft ³
Peak Discharge, Q ₁₅ =	16.168	cfs
Peak Discharge, Q ₁₀₀ =	21.312	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
3.22	184	9.3	0.925	2.41	0.38	17.5	16.17	548.2
3.62	207	11.1	1.145	2.71	0.42	18.6	21.34	771.7

Down Drain Feature: 1B1

Flow Through Circular Pipe

Number of Pipes, N=	2	
Diameter of pipe, D=	18	inches
Longitudinal Slope, S _o =	0.040	ft/ft
Manning's n=	0.009	
Density of flowing liquid, rho=	1.94	slugs/ft ³
Peak Discharge, Q ₂₅ =	26.950	cfs
Peak Discharge, Q ₁₀₀ =	35.527	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
3.01	172	8.4	0.807	2.25	0.36	16.7	26.96	436.6
3.34	191	9.9	0.994	2.50	0.40	17.9	35.55	616.6

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 =	30.119	cfs
Peak Discharge, Q ₁₀₀ =	39.700	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
3.13	179	8.9	0.876	2.35	0.37	17.2	30.10	501.4
3.50	200	10.6	1.082	2.62	0.41	18.3	39.71	706.7

Down Drain Feature: 1B3

Flow Through Circular Pipe

Number of Pipes, N=	2	
Diameter of pipe, D=	18	inches
Longitudinal Slope, S _o =	0.040	ft/ft
Manning's n=	0.009	
Density of flowing liquid, rho=	1.94	slugs/ft ³
Peak Discharge, Q ₁₅ =	29.899	cfs
Peak Discharge, Q ₁₀₀ =	39.414	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
3.12	179	8.9	0.872	2.34	0.37	17.1	29.88	496.8
3.49	200	10.5	1.076	2.62	0.41	18.3	39.43	700.6

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 ₁₅ =	38.940	cfs
Peak Discharge, Q ₁₀₀ =	51.331	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
3.47	199	10.5	1.066	2.60	0.41	18.3	38.96	690.3
3.98	228	12.7	1.330	2.99	0.45	19.3	51.37	962.0

Appendix 3

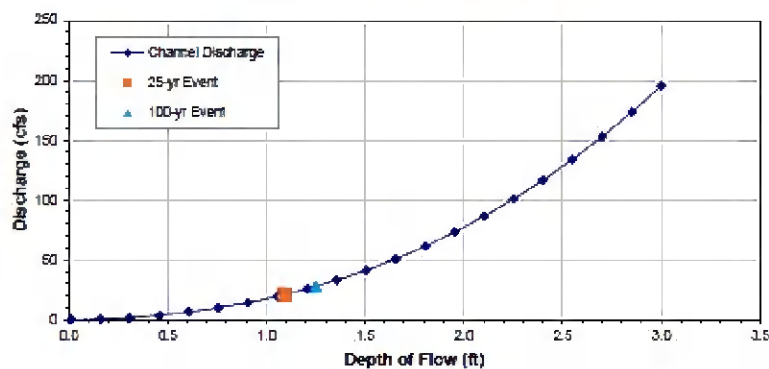
Perimeter Reach Calculations

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} = 21.05 cfs (25-yr Event)
Peak Discharge, Q_{100} = 27.81 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	
1.66	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	
1.09	6.86	9.91	0.69	3.05	20.96	0.22	Q (25-yr Event)
1.25	8.45	10.91	0.77	3.29	27.80	0.24	Q (100-yr Event)

Discharge versus Depth Relationship



Pipe 1A1

Diameter of pipe, D=	36	inches
Longitudinal Slope, So=	0.005	ft/ft
Manning's n=	0.012	
Density of flowing liquid, rho=	1.94	slugs/ft ³
Peak Discharge, Q ₂₅ =	21.030	cfs
Peak Discharge, Q ₁₀₀ =	27.880	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	0.003	0.38	0.01	0.3	0.00	0.0
0.50	29	0.6	0.023	0.75	0.03	0.9	0.02	0.0
0.75	43	1.3	0.077	1.13	0.07	1.5	0.11	0.3
1.00	57	2.2	0.178	1.50	0.12	2.1	0.38	1.6
1.25	72	3.4	0.339	1.88	0.18	2.8	0.95	5.2
1.50	86	4.8	0.565	2.25	0.25	3.5	1.98	13.4
1.75	100	6.5	0.862	2.63	0.33	4.2	3.60	29.2
2.00	115	8.3	1.227	3.00	0.41	4.8	5.93	55.7
2.25	129	10.2	1.656	3.38	0.49	5.5	9.04	95.8
2.50	143	12.3	2.139	3.75	0.57	6.0	12.92	151.3
2.75	158	14.5	2.664	4.13	0.65	6.6	17.48	222.4
3.00	172	16.7	3.216	4.50	0.71	7.0	22.57	307.3
3.25	186	19.0	3.778	4.88	0.77	7.4	27.98	402.1
3.50	201	21.2	4.332	5.25	0.83	7.7	33.46	501.4
3.75	215	23.4	4.862	5.63	0.86	8.0	38.73	598.5
4.00	229	25.5	5.351	6.00	0.89	8.1	43.53	687.0
4.25	244	27.5	5.788	6.38	0.91	8.2	47.65	761.0
4.50	258	29.3	6.162	6.75	0.91	8.3	50.91	816.1
4.75	272	31.0	6.468	7.13	0.91	8.2	53.24	850.2
5.00	286	32.4	6.704	7.50	0.89	8.1	54.61	863.1
5.25	301	33.7	6.873	7.88	0.87	8.0	55.10	857.1
5.50	315	34.6	6.981	8.25	0.85	7.9	54.83	835.5
5.75	329	35.4	7.041	8.63	0.82	7.7	53.99	803.1
6.00	344	35.8	7.064	9.00	0.78	7.5	52.77	764.8
6.25	358	36.0	7.069	9.38	0.75	7.3	51.41	725.3
2.92	168	16.0	3.047	4.39	0.69	6.9	20.99	280.4
3.25	186	18.9	3.767	4.87	0.77	7.4	27.88	400.3

PIPE 1BA

Diameter of pipe, D=	60	inches
Longitudinal Slope, So=	0.020	ft/ft
Manning's n=	0.012	
Density of flowing liquid, rho=	1.94	slugs/ft^3
Peak Discharge, Q _p =	113.440	cfs
Peak Discharge, Q ₁₀₀ =	150.530	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	0.2	0.008	0.63	0.01	1.0	0.01	0.0
0.50	29	0.9	0.064	1.25	0.05	2.4	0.16	0.7
0.75	43	2.1	0.214	1.88	0.11	4.1	0.88	7.0
1.00	57	3.7	0.495	2.50	0.20	6.0	2.96	34.2
1.25	72	5.7	0.941	3.13	0.30	7.9	7.42	113.4
1.50	86	8.0	1.570	3.75	0.42	9.8	15.43	294.1
1.75	100	10.8	2.394	4.38	0.55	11.7	28.11	640.6
2.00	115	13.8	3.408	5.00	0.68	13.6	46.35	1222.9
2.25	129	17.1	4.600	5.63	0.82	15.4	70.63	2103.8
2.50	143	20.5	5.942	6.25	0.95	17.0	100.89	3323.1
2.75	158	24.2	7.401	6.88	1.08	18.4	136.51	4884.9
3.00	172	27.9	8.934	7.50	1.19	19.7	176.30	6749.2
3.25	186	31.6	10.494	8.13	1.29	20.8	218.58	8831.8
3.50	201	35.3	12.034	8.75	1.38	21.7	261.35	11011.7
3.75	215	39.0	13.505	9.38	1.44	22.4	302.51	13146.1
4.00	229	42.5	14.865	10.00	1.49	22.9	340.03	15089.5
4.25	244	45.8	16.078	10.63	1.51	23.1	372.18	16713.6
4.50	258	48.8	17.117	11.25	1.52	23.2	397.68	17924.3
4.75	272	51.6	17.967	11.88	1.51	23.1	415.85	18672.4
5.00	286	54.0	18.622	12.50	1.49	22.9	426.58	18957.7
5.25	301	56.1	19.090	13.13	1.45	22.5	430.39	18824.4
5.50	315	57.7	19.392	13.75	1.41	22.1	428.30	18351.6
5.75	329	58.9	19.557	14.38	1.36	21.6	421.70	17640.0
6.00	344	59.7	19.623	15.00	1.31	21.0	412.21	16798.1
6.25	358	60.0	19.635	15.63	1.26	20.4	401.53	15930.0
2.59	148	21.8	6.445	6.47	1.00	17.5	112.88	3835.3
2.84	163	25.5	7.935	7.10	1.12	18.9	150.13	5510.2

PIPE 1B1a

Diameter of pipe, D= 54 inches
 Longitudinal Slope, So= 0.005 ft/ft
 Manning's n= 0.012
 Density of flowing liquid, rho= 1.94 slugs/ft^3
 Peak Discharge, Q₂₅= 87.690 cfs
 Peak Discharge, Q₁₀₀= 116.170 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	0.2	0.007	0.56	0.01	0.5	0.00	0.0
0.50	29	0.8	0.052	1.13	0.05	1.1	0.06	0.1
0.75	43	1.9	0.173	1.69	0.10	1.9	0.33	1.2
1.00	57	3.3	0.401	2.25	0.18	2.8	1.12	6.0
1.25	72	5.1	0.762	2.81	0.27	3.7	2.80	20.0
1.50	86	7.2	1.272	3.38	0.38	4.6	5.82	51.7
1.75	100	9.7	1.939	3.94	0.49	5.5	10.61	112.7
2.00	115	12.4	2.761	4.50	0.61	6.3	17.50	215.2
2.25	129	15.4	3.726	5.06	0.74	7.2	26.66	370.2
2.50	143	18.5	4.813	5.63	0.86	7.9	38.09	584.7
2.75	158	21.7	5.995	6.19	0.97	8.6	51.54	859.5
3.00	172	25.1	7.237	6.75	1.07	9.2	66.56	1187.5
3.25	186	28.5	8.500	7.31	1.16	9.7	82.52	1553.9
3.50	201	31.8	9.747	7.88	1.24	10.1	98.66	1937.5
3.75	215	35.1	10.939	8.44	1.30	10.4	114.20	2313.0
4.00	229	38.2	12.041	9.00	1.34	10.7	128.37	2655.0
4.25	244	41.2	13.023	9.56	1.36	10.8	140.50	2940.7
4.50	258	44.0	13.865	10.13	1.37	10.8	150.13	3153.7
4.75	272	46.4	14.553	10.69	1.36	10.8	156.99	3285.4
5.00	286	48.6	15.084	11.25	1.34	10.7	161.04	3335.6
5.25	301	50.5	15.463	11.81	1.31	10.5	162.48	3312.1
5.50	315	52.0	15.708	12.38	1.27	10.3	161.69	3228.9
5.75	329	53.0	15.841	12.94	1.22	10.0	159.20	3103.7
6.00	344	53.7	15.895	13.50	1.18	9.8	155.61	2955.6
6.25	358	54.0	15.904	14.06	1.13	9.5	151.59	2802.9
3.33	191	29.5	8.904	7.49	1.19	9.9	87.70	1675.9
6.25	358	54.0	15.904	14.06	1.13	9.5	151.58	2802.6

PIPE 1B1b

Diameter of pipe, D= 60 inches
 Longitudinal Slope, So= 0.005 ft/ft
 Manning's n= 0.012
 Density of flowing liquid, rho= 1.94 slugs/ft³
 Peak Discharge, Q₂₅= 93.210 cfs
 Peak Discharge, Q₁₀₀= 123.430 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	0.2	0.008	0.63	0.01	0.5	0.00	0.0
0.50	29	0.9	0.064	1.25	0.05	1.2	0.08	0.2
0.75	43	2.1	0.214	1.88	0.11	2.1	0.44	1.8
1.00	57	3.7	0.495	2.50	0.20	3.0	1.48	8.5
1.25	72	5.7	0.941	3.13	0.30	3.9	3.71	28.4
1.50	86	8.0	1.570	3.75	0.42	4.9	7.71	73.5
1.75	100	10.8	2.394	4.38	0.55	5.9	14.06	160.1
2.00	115	13.8	3.408	5.00	0.68	6.8	23.18	305.7
2.25	129	17.1	4.600	5.63	0.82	7.7	35.31	526.0
2.50	143	20.5	5.942	6.25	0.95	8.5	50.44	830.8
2.75	158	24.2	7.401	6.88	1.08	9.2	68.26	1221.2
3.00	172	27.9	8.934	7.50	1.19	9.9	88.15	1687.3
3.25	186	31.6	10.494	8.13	1.29	10.4	109.29	2207.9
3.50	201	35.3	12.034	8.75	1.38	10.9	130.68	2752.9
3.75	215	39.0	13.505	9.38	1.44	11.2	151.26	3286.5
4.00	229	42.5	14.865	10.00	1.49	11.4	170.02	3772.4
4.25	244	45.8	16.078	10.63	1.51	11.6	186.09	4178.4
4.50	258	48.8	17.117	11.25	1.52	11.6	198.84	4481.1
4.75	272	51.6	17.967	11.88	1.51	11.6	207.92	4668.1
5.00	286	54.0	18.622	12.50	1.49	11.5	213.29	4739.4
5.25	301	56.1	19.090	13.13	1.45	11.3	215.20	4706.1
5.50	315	57.7	19.392	13.75	1.41	11.0	214.15	4587.9
5.75	329	58.9	19.557	14.38	1.36	10.8	210.85	4410.0
6.00	344	59.7	19.623	15.00	1.31	10.5	206.10	4199.5
6.25	358	60.0	19.635	15.63	1.26	10.2	200.77	3982.5
3.06	175	28.8	9.307	7.65	1.22	10.0	93.13	1807.8
3.42	196	34.1	11.518	8.54	1.35	10.7	123.46	2567.2

PIPE 1B2a

Diameter of pipe, D=	30	inches
Longitudinal Slope, So=	0.010	ft/ft
Manning's n=	0.012	
Density of flowing liquid, rho=	1.94	slugs/ft^3
Peak Discharge, Q _p =	31.660	cfs
Peak Discharge, Q ₁₀₀ =	41.770	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	0.002	0.31	0.01	0.4	0.00	0.0
0.50	29	0.5	0.016	0.63	0.03	1.1	0.02	0.0
0.75	43	1.0	0.053	0.94	0.06	1.8	0.10	0.3
1.00	57	1.8	0.124	1.25	0.10	2.7	0.33	1.7
1.25	72	2.8	0.235	1.56	0.15	3.5	0.83	5.6
1.50	86	4.0	0.393	1.88	0.21	4.4	1.72	14.6
1.75	100	5.4	0.598	2.19	0.27	5.2	3.13	31.8
2.00	115	6.9	0.852	2.50	0.34	6.1	5.16	60.6
2.25	129	8.5	1.150	2.81	0.41	6.8	7.86	104.3
2.50	143	10.3	1.486	3.13	0.48	7.6	11.23	164.8
2.75	158	12.1	1.850	3.44	0.54	8.2	15.20	242.2
3.00	172	13.9	2.233	3.75	0.60	8.8	19.63	334.6
3.25	186	15.8	2.624	4.06	0.65	9.3	24.34	437.9
3.50	201	17.7	3.008	4.38	0.69	9.7	29.10	546.0
3.75	215	19.5	3.376	4.69	0.72	10.0	33.68	651.8
4.00	229	21.2	3.716	5.00	0.74	10.2	37.86	748.2
4.25	244	22.9	4.020	5.31	0.76	10.3	41.44	828.7
4.50	258	24.4	4.279	5.63	0.76	10.3	44.28	888.7
4.75	272	25.8	4.492	5.94	0.76	10.3	46.30	925.8
5.00	286	27.0	4.655	6.25	0.74	10.2	47.49	940.0
5.25	301	28.0	4.773	6.56	0.73	10.0	47.92	933.4
5.50	315	28.9	4.848	6.88	0.71	9.8	47.69	909.9
5.75	329	29.5	4.889	7.19	0.68	9.6	46.95	874.7
6.00	344	29.8	4.906	7.50	0.65	9.4	45.89	832.9
6.25	358	30.0	4.909	7.81	0.63	9.1	44.71	789.9
3.64	209	18.7	3.217	4.55	0.71	9.9	31.70	606.1
6.25	358	30.0	4.909	7.81	0.63	9.1	44.71	790.1

PIPE 1B2b

Diameter of pipe, D=	42	inches
Longitudinal Slope, So=	0.002	ft/ft
Manning's n=	0.012	
Density of flowing liquid, rho=	1.94	slugs/ft^3
Peak Discharge, Q ₂₅ =	31.390	cfs
Peak Discharge, Q ₁₀₀ =	41.420	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	0.2	0.004	0.44	0.01	0.3	0.00	0.0
0.50	29	0.7	0.032	0.88	0.04	0.6	0.02	0.0
0.75	43	1.5	0.105	1.31	0.08	1.1	0.12	0.2
1.00	57	2.6	0.243	1.75	0.14	1.6	0.39	1.2
1.25	72	4.0	0.461	2.19	0.21	2.1	0.97	4.0
1.50	86	5.6	0.769	2.63	0.29	2.6	2.02	10.3
1.75	100	7.5	1.173	3.06	0.38	3.1	3.68	22.4
2.00	115	9.7	1.670	3.50	0.48	3.6	6.07	42.8
2.25	129	11.9	2.254	3.94	0.57	4.1	9.25	73.7
2.50	143	14.4	2.912	4.38	0.67	4.5	13.22	116.4
2.75	158	16.9	3.627	4.81	0.75	4.9	17.88	171.0
3.00	172	19.5	4.378	5.25	0.83	5.3	23.09	236.3
3.25	186	22.1	5.142	5.69	0.90	5.6	28.63	309.2
3.50	201	24.7	5.897	6.13	0.96	5.8	34.23	385.6
3.75	215	27.3	6.617	6.56	1.01	6.0	39.62	460.3
4.00	229	29.7	7.284	7.00	1.04	6.1	44.54	528.4
4.25	244	32.1	7.878	7.44	1.06	6.2	48.75	585.2
4.50	258	34.2	8.387	7.88	1.07	6.2	52.09	627.6
4.75	272	36.1	8.804	8.31	1.06	6.2	54.47	653.8
5.00	286	37.8	9.125	8.75	1.04	6.1	55.88	663.8
5.25	301	39.3	9.354	9.19	1.02	6.0	56.38	659.1
5.50	315	40.4	9.502	9.63	0.99	5.9	56.10	642.6
5.75	329	41.3	9.583	10.06	0.95	5.8	55.24	617.7
6.00	344	41.8	9.615	10.50	0.92	5.6	53.99	588.2
6.25	358	42.0	9.621	10.94	0.88	5.5	52.60	557.8
3.37	193	23.4	5.516	5.90	0.93	5.7	31.40	346.7
3.84	220	28.2	6.868	6.72	1.02	6.0	41.49	486.2

PIPE 1B2c

Diameter of pipe, D= 48 inches
 Longitudinal Slope, So= 0.005 ft/ft
 Manning's n= 0.012
 Density of flowing liquid, rho= 1.94 slugs/ft³
 Peak Discharge, Q_{ps}= 87.930 cfs
 Peak Discharge, Q₁₀₀= 116.500 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	0.2	0.005	0.50	0.01	0.4	0.00	0.0
0.50	29	0.7	0.041	1.00	0.04	1.0	0.04	0.1
0.75	43	1.7	0.137	1.50	0.09	1.8	0.24	0.8
1.00	57	2.9	0.317	2.00	0.16	2.6	0.81	4.1
1.25	72	4.5	0.602	2.50	0.24	3.4	2.04	13.5
1.50	86	6.4	1.005	3.00	0.34	4.2	4.25	34.9
1.75	100	8.6	1.532	3.50	0.44	5.1	7.75	76.1
2.00	115	11.0	2.181	4.00	0.55	5.9	12.78	145.3
2.25	129	13.7	2.944	4.50	0.65	6.6	19.48	249.9
2.50	143	16.4	3.803	5.00	0.76	7.3	27.82	394.8
2.75	158	19.3	4.737	5.50	0.86	7.9	37.64	580.4
3.00	172	22.3	5.718	6.00	0.95	8.5	48.61	801.9
3.25	186	25.3	6.716	6.50	1.03	9.0	60.27	1049.3
3.50	201	28.3	7.702	7.00	1.10	9.4	72.07	1308.3
3.75	215	31.2	8.643	7.50	1.15	9.7	83.42	1561.9
4.00	229	34.0	9.514	8.00	1.19	9.9	93.76	1792.7
4.25	244	36.6	10.290	8.50	1.21	10.0	102.63	1985.7
4.50	258	39.1	10.955	9.00	1.22	10.0	109.66	2129.5
4.75	272	41.3	11.499	9.50	1.21	10.0	114.67	2218.4
5.00	286	43.2	11.918	10.00	1.19	9.9	117.63	2252.3
5.25	301	44.9	12.218	10.50	1.16	9.7	118.68	2236.5
5.50	315	46.2	12.411	11.00	1.13	9.5	118.10	2180.3
5.75	329	47.2	12.517	11.50	1.09	9.3	116.28	2095.8
6.00	344	47.8	12.559	12.00	1.05	9.1	113.66	1995.7
6.25	358	48.0	12.566	12.50	1.01	8.8	110.72	1892.6
3.86	221	32.4	9.033	7.72	1.17	9.8	88.08	1666.2
6.25	358	48.0	12.566	12.50	1.01	8.8	110.72	1892.6

Design/Check: Trapezoidal/Triangular Channel

Methodology: Manning's Equation

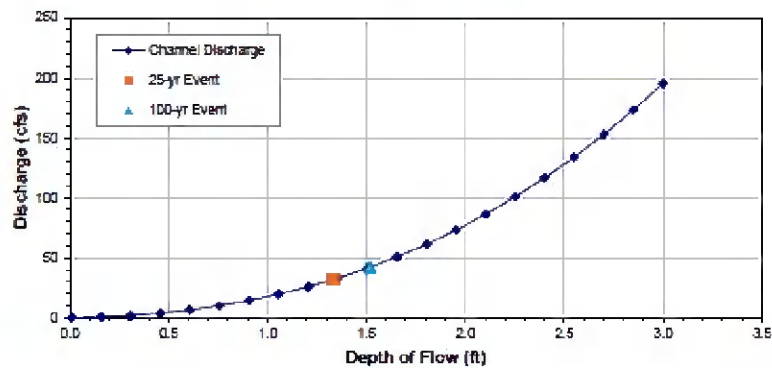
Project: Santee Cooper - Winyah Generating Station

Ditch ID: **Perimeter Reach 1B3a** **Design**

Peak Discharge, Q_{25} = 31.93 cfs (25-yr Event)
 Peak Discharge, Q_{100} = 42.11 cfs (100-yr Event)
 Bottom Width, B = 3.00 ft
 Left Side Slope, Z_L = 3.00 horizontal :1 vertical
 Right Side Slope, Z_R = 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, S₀ = 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	Arg. 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	
1.33	9.35	11.44	0.82	3.41	31.87	0.25	Q (25-yr Event)
1.52	11.48	12.61	0.91	3.67	42.06	0.28	Q (100-yr Event)

Discharge versus Depth Relationship



Design/Check: Trapezoidal/Triangular Channel

Methodology: Manning's Equation

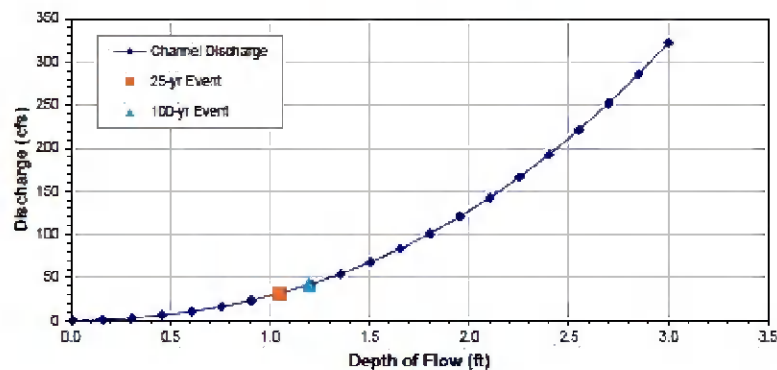
Project: Santee Cooper - Winyah Generating Station

Ditch ID: **Perimeter Reach 1B3b** Design

Peak Discharge, Q_{25} = 31.72 cfs (25-yr Event)
 Peak Discharge, Q_{100} = 41.85 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 = 0.014 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	0.30	0.0	0.01	
0.16	0.55	4.01	0.14	1.72	1.0	0.12	
0.31	1.21	4.95	0.24	2.52	3.1	0.21	
0.46	2.01	5.90	0.34	3.14	6.3	0.29	
0.61	2.93	6.85	0.43	3.66	10.7	0.36	
0.76	3.99	7.79	0.51	4.12	16.5	0.44	
0.91	5.19	8.74	0.59	4.55	23.6	0.50	
1.06	6.52	9.68	0.67	4.94	32.2	0.57	
1.21	7.98	10.63	0.75	5.32	42.5	0.64	
1.36	9.58	11.57	0.83	5.68	54.4	0.70	
1.51	11.31	12.52	0.90	6.02	68.1	0.77	
1.65	13.18	13.46	0.98	6.35	83.6	0.83	
1.80	15.18	14.41	1.05	6.66	101.1	0.89	
1.95	17.31	15.36	1.13	6.97	120.7	0.96	
2.10	19.58	16.30	1.20	7.27	142.4	1.02	
2.25	21.98	17.25	1.27	7.57	166.3	1.08	
2.40	24.51	18.19	1.35	7.86	192.6	1.14	
2.55	27.18	19.14	1.42	8.14	221.2	1.21	
2.70	29.99	20.08	1.49	8.41	252.3	1.27	
2.85	32.93	21.03	1.57	8.68	285.9	1.33	
3.00	36.00	21.97	1.64	8.95	322.2	1.39	
1.05	6.44	9.63	0.67	4.92	31.68	0.57	Q (25-yr Event)
1.20	7.89	10.57	0.75	5.30	41.80	0.63	Q (100-yr Event)

Discharge versus Depth Relationship



Design/Check: Trapezoidal/Triangular Channel

Methodology: Manning's Equation

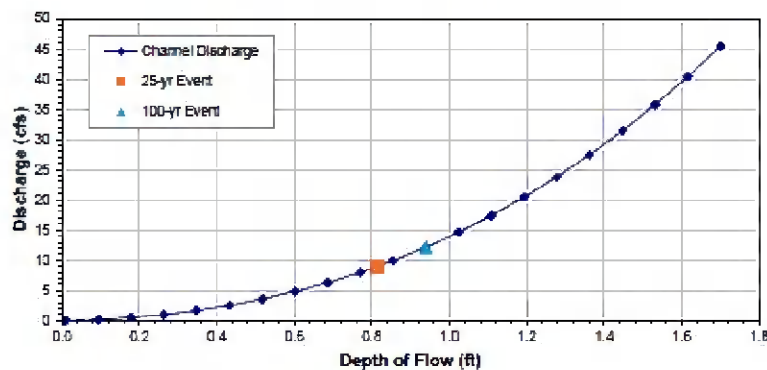
Project: Santee Cooper - Winyah Generating Station

Ditch ID: **Perimeter Reach 1C1** Design

Peak Discharge, Q_{25} =	9.03	cfs (25-yr Event)
Peak Discharge, Q_{100} =	12.13	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_0 =	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	0.02	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	6.1	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	2.68	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	
0.82	3.63	7.16	0.51	2.48	9.00	0.16	Q (25-yr Event)
0.94	4.52	7.94	0.57	2.68	12.13	0.18	Q (100-yr Event)

Discharge versus Depth Relationship



PIPE 1C1a

Diameter of pipe, D=	30	inches
Longitudinal Slope, So=	0.005	ft/ft
Manning's n=	0.012	
Density of flowing liquid, rho=	1.94	slugs/ft ³
Peak Discharge, Q _p =	8.720	cfs
Peak Discharge, Q ₁₀₀ =	11.730	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	0.002	0.31	0.01	0.3	0.00	0.0
0.50	29	0.5	0.016	0.63	0.03	0.8	0.01	0.0
0.75	43	1.0	0.053	0.94	0.06	1.3	0.07	0.2
1.00	57	1.8	0.124	1.25	0.10	1.9	0.23	0.8
1.25	72	2.8	0.235	1.56	0.15	2.5	0.58	2.8
1.50	86	4.0	0.393	1.88	0.21	3.1	1.21	7.3
1.75	100	5.4	0.598	2.19	0.27	3.7	2.21	15.9
2.00	115	6.9	0.852	2.50	0.34	4.3	3.65	30.3
2.25	129	8.5	1.150	2.81	0.41	4.8	5.56	52.2
2.50	143	10.3	1.486	3.13	0.48	5.3	7.94	82.4
2.75	158	12.1	1.850	3.44	0.54	5.8	10.75	121.1
3.00	172	13.9	2.233	3.75	0.60	6.2	13.88	167.3
3.25	186	15.8	2.624	4.06	0.65	6.6	17.21	219.0
3.50	201	17.7	3.008	4.38	0.69	6.8	20.58	273.0
3.75	215	19.5	3.376	4.69	0.72	7.1	23.82	325.9
4.00	229	21.2	3.716	5.00	0.74	7.2	26.77	374.1
4.25	244	22.9	4.020	5.31	0.76	7.3	29.30	414.4
4.50	258	24.4	4.279	5.63	0.76	7.3	31.31	444.4
4.75	272	25.8	4.492	5.94	0.76	7.3	32.74	462.9
5.00	286	27.0	4.655	6.25	0.74	7.2	33.58	470.0
5.25	301	28.0	4.773	6.56	0.73	7.1	33.88	466.7
5.50	315	28.9	4.848	6.88	0.71	7.0	33.72	455.0
5.75	329	29.5	4.889	7.19	0.68	6.8	33.20	437.3
6.00	344	29.8	4.906	7.50	0.65	6.6	32.45	416.5
6.25	358	30.0	4.909	7.81	0.63	6.4	31.61	394.9
2.57	147	10.8	1.584	3.21	0.49	5.5	8.68	92.3
2.83	162	12.7	1.969	3.54	0.56	5.9	11.70	134.9

PIPE 1C1b

Diameter of pipe, D=	48	inches
Longitudinal Slope, S _o =	0.008	ft/ft
Manning's n=	0.012	
Density of flowing liquid, rho=	1.94	slugs/ft ³
Peak Discharge, Q _p =	54.350	cfs
Peak Discharge, Q ₁₀₀ =	72.260	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	0.2	0.005	0.50	0.01	0.5	0.00	0.0
0.50	29	0.7	0.041	1.00	0.04	1.3	0.05	0.1
0.75	43	1.7	0.137	1.50	0.09	2.2	0.31	1.3
1.00	57	2.9	0.317	2.00	0.16	3.3	1.03	6.5
1.25	72	4.5	0.602	2.50	0.24	4.3	2.59	21.6
1.50	86	6.4	1.005	3.00	0.34	5.4	5.38	55.9
1.75	100	8.6	1.532	3.50	0.44	6.4	9.81	121.8
2.00	115	11.0	2.181	4.00	0.55	7.4	16.17	232.5
2.25	129	13.7	2.944	4.50	0.65	8.4	24.63	399.9
2.50	143	16.4	3.803	5.00	0.76	9.3	35.19	631.7
2.75	158	19.3	4.737	5.50	0.86	10.1	47.61	928.6
3.00	172	22.3	5.718	6.00	0.95	10.8	61.49	1283.0
3.25	186	25.3	6.716	6.50	1.03	11.4	76.24	1678.8
3.50	201	28.3	7.702	7.00	1.10	11.8	91.16	2093.2
3.75	215	31.2	8.643	7.50	1.15	12.2	105.51	2499.0
4.00	229	34.0	9.514	8.00	1.19	12.5	118.60	2868.4
4.25	244	36.6	10.290	8.50	1.21	12.6	129.81	3177.1
4.50	258	39.1	10.955	9.00	1.22	12.7	138.71	3407.2
4.75	272	41.3	11.499	9.50	1.21	12.6	145.04	3549.5
5.00	286	43.2	11.918	10.00	1.19	12.5	148.79	3603.7
5.25	301	44.9	12.218	10.50	1.16	12.3	150.12	3578.3
5.50	315	46.2	12.411	11.00	1.13	12.0	149.39	3488.5
5.75	329	47.2	12.517	11.50	1.09	11.8	147.09	3353.2
6.00	344	47.8	12.559	12.00	1.05	11.4	143.78	3193.2
6.25	358	48.0	12.566	12.50	1.01	11.1	140.05	3028.2
2.87	165	20.8	5.209	5.74	0.91	10.4	54.20	1094.3
3.18	182	24.5	6.447	6.37	1.01	11.2	72.21	1569.1

PIPE 1C1c

Diameter of pipe, D=	48	inches
Longitudinal Slope, So=	0.010	ft/ft
Manning's n=	0.012	
Density of flowing liquid, rho=	1.94	slugs/ft^3
Peak Discharge, Q ₂₅ =	54.200	cfs
Peak Discharge, Q ₁₀₀ =	72.080	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	0.2	0.005	0.50	0.01	0.6	0.00	0.0
0.50	29	0.7	0.041	1.00	0.04	1.5	0.06	0.2
0.75	43	1.7	0.137	1.50	0.09	2.5	0.34	1.7
1.00	57	2.9	0.317	2.00	0.16	3.6	1.15	8.1
1.25	72	4.5	0.602	2.50	0.24	4.8	2.89	27.0
1.50	86	6.4	1.005	3.00	0.34	6.0	6.02	69.9
1.75	100	8.6	1.532	3.50	0.44	7.2	10.96	152.2
2.00	115	11.0	2.181	4.00	0.55	8.3	18.08	290.6
2.25	129	13.7	2.944	4.50	0.65	9.4	27.54	499.9
2.50	143	16.4	3.803	5.00	0.76	10.3	39.34	789.6
2.75	158	19.3	4.737	5.50	0.86	11.2	53.24	1160.7
3.00	172	22.3	5.718	6.00	0.95	12.0	68.75	1603.7
3.25	186	25.3	6.716	6.50	1.03	12.7	85.24	2098.6
3.50	201	28.3	7.702	7.00	1.10	13.2	101.92	2616.5
3.75	215	31.2	8.643	7.50	1.15	13.6	117.97	3123.7
4.00	229	34.0	9.514	8.00	1.19	13.9	132.60	3585.5
4.25	244	36.6	10.290	8.50	1.21	14.1	145.14	3971.4
4.50	258	39.1	10.955	9.00	1.22	14.2	155.08	4259.1
4.75	272	41.3	11.499	9.50	1.21	14.1	162.17	4436.8
5.00	286	43.2	11.918	10.00	1.19	14.0	166.35	4504.6
5.25	301	44.9	12.218	10.50	1.16	13.7	167.84	4472.9
5.50	315	46.2	12.411	11.00	1.13	13.5	167.02	4360.6
5.75	329	47.2	12.517	11.50	1.09	13.1	164.45	4191.5
6.00	344	47.8	12.559	12.00	1.05	12.8	160.75	3991.5
6.25	358	48.0	12.566	12.50	1.01	12.5	156.58	3785.2
2.77	158	19.5	4.797	5.53	0.87	11.3	54.16	1186.4
3.05	175	22.9	5.919	6.10	0.97	12.2	72.03	1700.3

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} = 3.05 cfs (25-yr Event)
Peak Discharge, Q_{100} = 4.08 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_0 = 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	0.02	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	2.68	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	
0.48	1.64	5.02	0.33	1.85	3.02	0.10	Q (25-yr Event)
0.55	2.02	5.50	0.37	2.00	4.05	0.11	Q (100-yr Event)

Discharge versus Depth Relationship

