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Run-On and Run-Off Control System Plan Periodic Update for CCR Landfill Area 1, Rev. 1

WINYAH GENERATING STATION
Georgetown, South Carolina

Prepared by

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November 2021

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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 (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 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 appendices to this ROROCS Plan.

The initial ROROCS Plan was prepared and certified 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. This ROROCS Plan is the first subsequent plan (i.e., the first 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).

Since the creation of the original 2018 ROROCS Plan, this periodic update has been refined to limit the scope of design, analysis, and control measure construction to be applicable to the operational conditions. It is for that reason, that various calculations such as leachate generation assessments, pump and pipe capacities, and stability of the landfill 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.

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 the Winyah Area 1 CCR landfill is included in subsequent sections of this ROROCS Plan.

3. DESIGN METHODOLOGY

3.1 Design Storm

Run-on and run-off controls systems were designed and constructed for hydraulic capacity for the 24-hour, 25-year storm events 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-off control features against the assumed design storm are included in Appendix I of this ROROCS Plan. Interim and final cover stormwater features consist of downdrains to convey stormwater off the cap to the perimeter channel, an attenuation basin and decant pipe within the active face and downdrains to collect and manage contact water during operations and perimeter control berms during operations were designed and constructed considering the 24-hour, 25-year design storm 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

The HydroCAD® and HEC-HMS computer programs were used to apply the SCS and Rational method to estimate runoff for interim and 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 the active face attenuation basin, and run-off discharge quantities supporting the design and operation of the CCR landfill. Appendices I-A though I-C present detailed drainage calculations and results, including a detailed discussion of the parameters used in the analyses.

3.3 Design Drawings

Final conditions, topography 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 South Carolina Department of Health and Environmental Control (DHEC) on 15 September 2017 (Permit #LF3-00042).

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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 landfill facility used to divert stormwater run-on from reaching an active portion of the Winyah CCR landfill.

The Class III CCR 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 CCR 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-B.

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).

Run-off controls for contact water and stormwater at the Area I landfill 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 Contact Water Run-Off

Contact water on the exposed CCR surface is being managed based on typical filling conditions for interim or operational filling. The landfill design includes the layout of a series of interim filling condition and assumes that the active portion of the cell is graded towards the center of the active face or cell to drain towards a constructed attenuation basin and decant structure. A perimeter containment berm is necessary in each lift phase scenario to prevent the migration of contact water run-off from leaving the landfill footprint and will be maintained during each lift.

The perimeter access berm effectively serves as a contact water containment berm during initial filling conditions. Stage-storage curves for each phase of construction were developed, based on the refined design of fill plans developed by Geosyntec in 2020 to evaluate the elevation where a temporary soil berm is necessary along the perimeter. Once waste is placed above the elevation of the perimeter berm, interim conditions are implemented and contact water drains towards the interior of each cell or phase, a 24-inch tall (minimum) berm around the cell is also required assuming that the cell is flat, that the full cell footprint contributes to the storage capacity, and that infiltration does not remove contact water during the design storm event.

Temporary containment berms were designed and constructed to manage contact water from a sloped active portion of the CCR Landfill. Temporary containment berm heights were computed based on a range of contributing contact water drainage areas and a range of stormwater collection areas at the base of the slope. The associated calculations are provided in Appendices I-A through I-C.

5.2 Stormwater Run-Off

During operations, exterior side slopes of the landfill may be covered with interim soil cover or approved alternative cover as CCR placement progresses to prevent stormwater from contacting CCR. Stormwater run-off controls for non-contact water during CCR placement were designed and built based on the final cover system design; the interim features will be constructed based on the final cover feature dimensions until final cover is constructed. The landfill has side slopes graded at 3 horizontal to 1 vertical (3H:1V) between drainage benches, and top deck slopes graded at five percent to maximize the removal of run-off. Drainage swales will be constructed on the interim and final cover system to collect and divert surface water run-off into down drains 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 24-hour, 25-year storm event. Design calculations are provided in Appendix I. Some of the stormwater management features were sized to convey non-contact water run-off from the final cover installation which results in a conservative analysis for interim conditions.

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 CCR landfill run-on control system at the Winyah Area 1 CCR landfill has been designed, built, and is being maintained to prevent flow onto the active portion of the CCR unit during peak discharge from a 25-year, 24-hour storm, and the CCR landfill run-off control system at Winyah has been designed and is currently being operated to collect and control at least the water volume from a 25-year, 24-hour design storm.

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: 19.05 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 (0.467-0.542)	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 (0.861-1.01)	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)	1.07 (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 (1.79-2.19)
15-min	1.00 (0.933-1.08)	1.18 (1.09-1.27)	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 (1.28-1.49)	1.63 (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 (2.85-3.39)	3.41 (3.08-3.69)	3.77 (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)	3.41 (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 (5.31-6.48)
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 (6.31-7.69)	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 (8.09-10.0)	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)	7.61 (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 (19.3-21.7)	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

APPENDICES

APPENDIX I

Run-Off Control System Calculations

APPENDIX I-A

Calculations for Active Face Run-Off Control

Written by: **C. Jordan** Date: **9/3/21** Reviewed by: **S. Graves** Date: **9/3/21**

Client: **Santee Cooper** Project: **Winyah Generating Station** Project No.: **GC8100** Task No.: **05**

ACTIVE AREA RUNOFF CONTROL

1. PURPOSE

The purpose of this calculation package is to evaluate the active area runoff control design for the Area 1 Class Three Landfill at the Winyah Generating Station (WGS). The calculations are used to:

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

2. METHOD OF ANALYSIS

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

The contact water drainage area, attenuation basins and decant structures were modeled using HydroCAD Stormwater Modeling Software Version 10 (HydroCAD Software Solutions, LLC, 2011). The original attenuation basin and the decant structure were sized using the model with the objective of draining contact water generated from the 25-year, 24-hour rainfall event within approximately 3 days. In addition, the model was used to evaluate the decant structure and water levels in the active area considering runoff from a 100-year, 24-hour storm event. This update to the original 2016 calculation does not include an analysis for the proposed temporary and permanent leachate storage ponds. The assumptions and analyses performed related to the 2016 evaluation are assumed to be the same, at the time of the writing of this updated calculation.

Within this calculation, the approved filling (lift) plan for the Area 1 Landfill was evaluated on an individual phased basis to assess the capacity of the attenuation basin and decant structure for the 25-year, 24-hour design storm event. In addition, calculations were performed to evaluate the sizing of a “run-off” control berm that would be constructed around the active face of the landfill as each individual lift area is reached. Along with the lift plan, the most recent available landfill survey (performed on 23 June 2021 by Santee Cooper) was analyzed for compliance with the Federal CCR Rule Runoff control requirements.

3. DESIGN PARAMETERS

Rainfall Event

The rainfall depth corresponding to the 25-year, 24-hour storm and 100-year, 24-hour storm were selected based on information from the NOAA Atlas 14 Point Precipitation Frequency Estimates (NOAA Hydrometeorological Design Studies Center, retrieved 26-July-2021). The 25-year, 24-hour rainfall depth is 8.24 in., and the 100-year, 24-hour rainfall depth is 10.8 in., as shown in Table 1. The rainfall intensity used in the analysis was the SCS Type III distribution.

Contact Water Catchment Area

Each lift of the Area 1 landfill was analyzed as well as the survey data. As the landfill is filled above grade, cover soil will be placed on the fill side slopes and the active area will decrease. The surface condition of the catchment area was assumed to be bare (without vegetation) and graded to drain towards the decant structure. The runoff curve number of the surface material was assumed to be 97 based on *HELP* model output files. The time of concentration for contact water to drain from the catchment area to the attenuation basin of the decant structure was assumed to be 6 minutes.

Contact Water Attenuation Basin and Outlet Structures

The attenuation basin is a depressed area surrounding the decant structure that attenuates the contact water inflow by temporarily detaining it while it flows into the decant structure. The attenuation basin is proposed to have a depth of 4 ft during operations and sideslopes graded at approximately at 5H:1V. The decant structure will be extended vertically upward as the landfill is filled. Since the active area and corresponding contact water generation will decrease as the landfill is filled, the size of the attenuation basin required to manage contact water also decreases. For the purposes of this calculation package, the attenuation basin is sized assuming the largest active area and the corresponding approximate waste elevations.

The decant structure consists of a vertical 36-in. diameter perforated reinforced concrete pipe. The pipe is proposed to be installed in 4 ft long segments. The pipe perforations are 2-in. in diameter, spaced at 30-degree intervals around the pipe (i.e., 12 perforations per row), and staggered vertically at 6-in. spacing for a total of 8 rows. The decant structure drains into an 18-

in. diameter HDPE pipe, having a longitudinal slope of 0.5 percent and approximately 1,300 ft long.

The decant structure and attenuation basin within Lifts 1A and 1B were modeled under the following scenarios:

1. Normal operations of the decant structure and attenuation basin;
2. The decant structure is clogged and does not discharge water from the attenuation basin; and
3. The attenuation basin is filled with sediment and does not provide storage.

Runoff Containment Berm

During operations, the active face will be graded to drain contact water towards the attenuation basin and decant structure. Under this analysis, it was assumed that the storage within the active face experiences a period in time where the decant structure openings get clogged by CCR as well as the attenuation basin is filled with sediment during the period between lifts.

As these scenarios occur, a runoff containment measure still needs to be active to prevent waste from leaving the active face under storm conditions. For these scenarios, the total storage within the active face for each lift was evaluated using AutoCAD Civil 3D 2020. A vertical face was assumed at the edge of the active face along the perimeter slopes. Each of these areas was modeled in HydroCAD to estimate the peak elevation of water within the active face. Using this peak water elevation, a perimeter surface berm height was established for runoff control under the assumption there will be 6 inches of freeboard between the top of berm and peak water elevation.

4. RESULTS

Based on the HydroCAD model outputs, the calculated peak water surface elevation in the contact water attenuation basin for the Phase 1A lift was 56.68 ft-MSL for the 25-yr, 24-hr storm and 56.91 ft-MSL for the 100-yr, 24-hr storm, and Phase 1B lift was 86.69 ft-MSL for the 25-yr, 24-hr storm and 86.91 ft-MSL for the 100-yr, 24-hr storm.

As presented in Table 4, under the operating conditions for Phases 1A and 1B, the grading provided for the Area 1 lift plan includes a 2-ft tall perimeter berm the provides secondary containment within the active face. This perimeter runoff control berm provides more than 1-foot of freeboard for the 100-year design storm under the assumption of the attenuation basin being full of sediment. The 2-ft tall runoff control berm provides sufficient freeboard during landfill operations and will contain the runoff during intermediate lifts that are not presented within the approved lift plans.

5. CONCLUSIONS

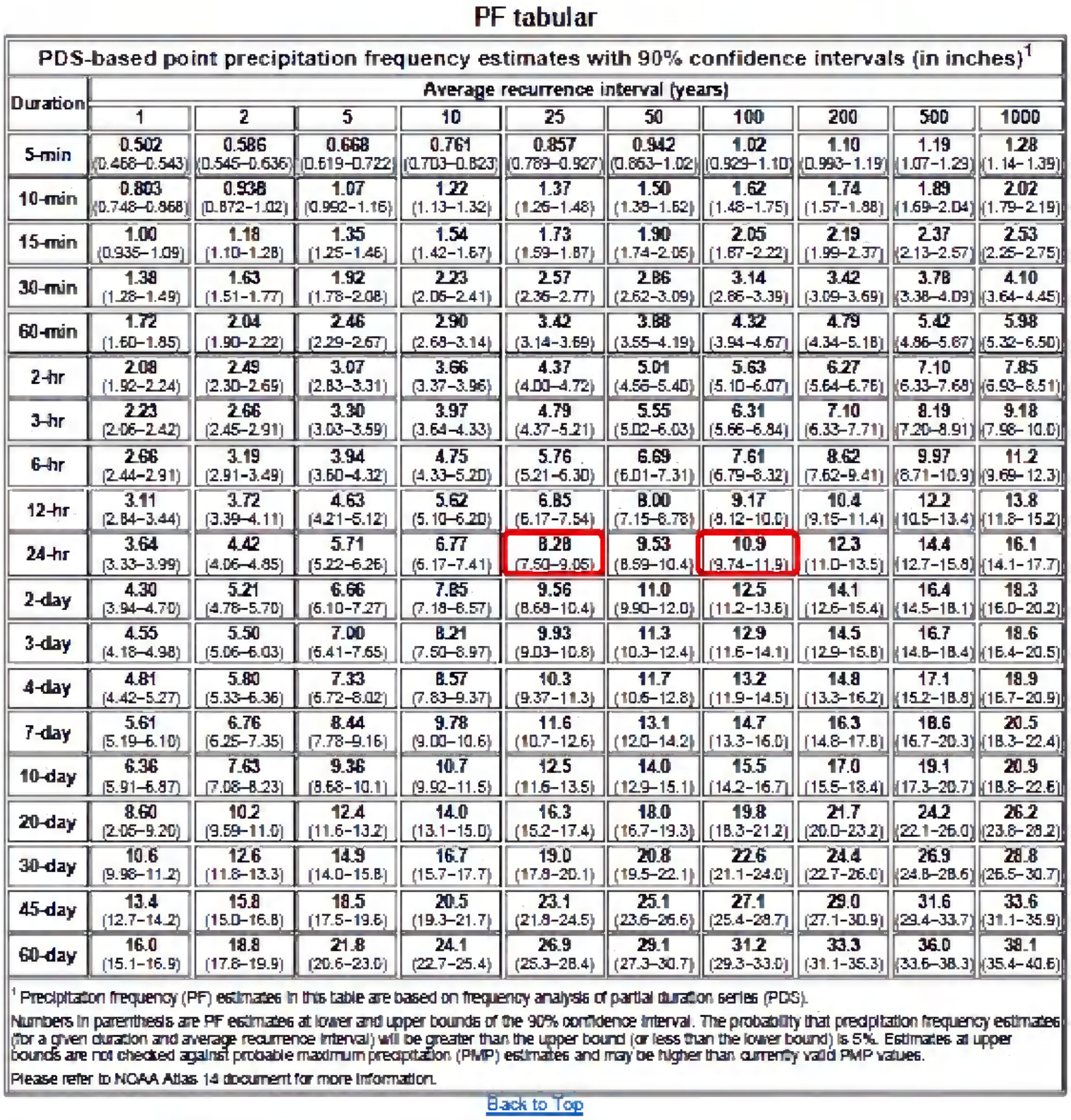
Based on calculation herein, the 2-ft tall runoff control berm that is located around the perimeter of the active face of the Area 1 landfill will contain contact water within the working area, even under a worst-case hypothetical scenario with the decant structure not functioning.

6. REFERENCES

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

NOAA Hydrometeorological Design Studies Center. *Precipitation Frequency Data Server.* Retrieved July 28, 2021, from NOAA Atlas 14 Pont Precipitation Frequency Estimates (South Carolina): http://hdsc.nws.noaa.gov/hdsc/pfds/pfds_map_cont.html

TABLE 1 – NOAA Point Precipitation Data



PF tabular

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

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

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

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

Please refer to NOAA Atlas 14 document for more information.

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TABLE 2 – Construction Lift Phase Area and Volume

Construction Phase	Attenuation Basin Top Elevation (ft)	Attenuation Basin Bottom Elevation (ft)	Area of Bottom of Attenuation Basin (acres)	Area of Top of Attenuation Basin (acres)	Approx. Basin Volume (acre-ft)
1A	54	50	3.1	7.4	16.96
1B	84	80	1.4	2.1	6.74

TABLE 3 – Construction Lift Phase Area and Volume

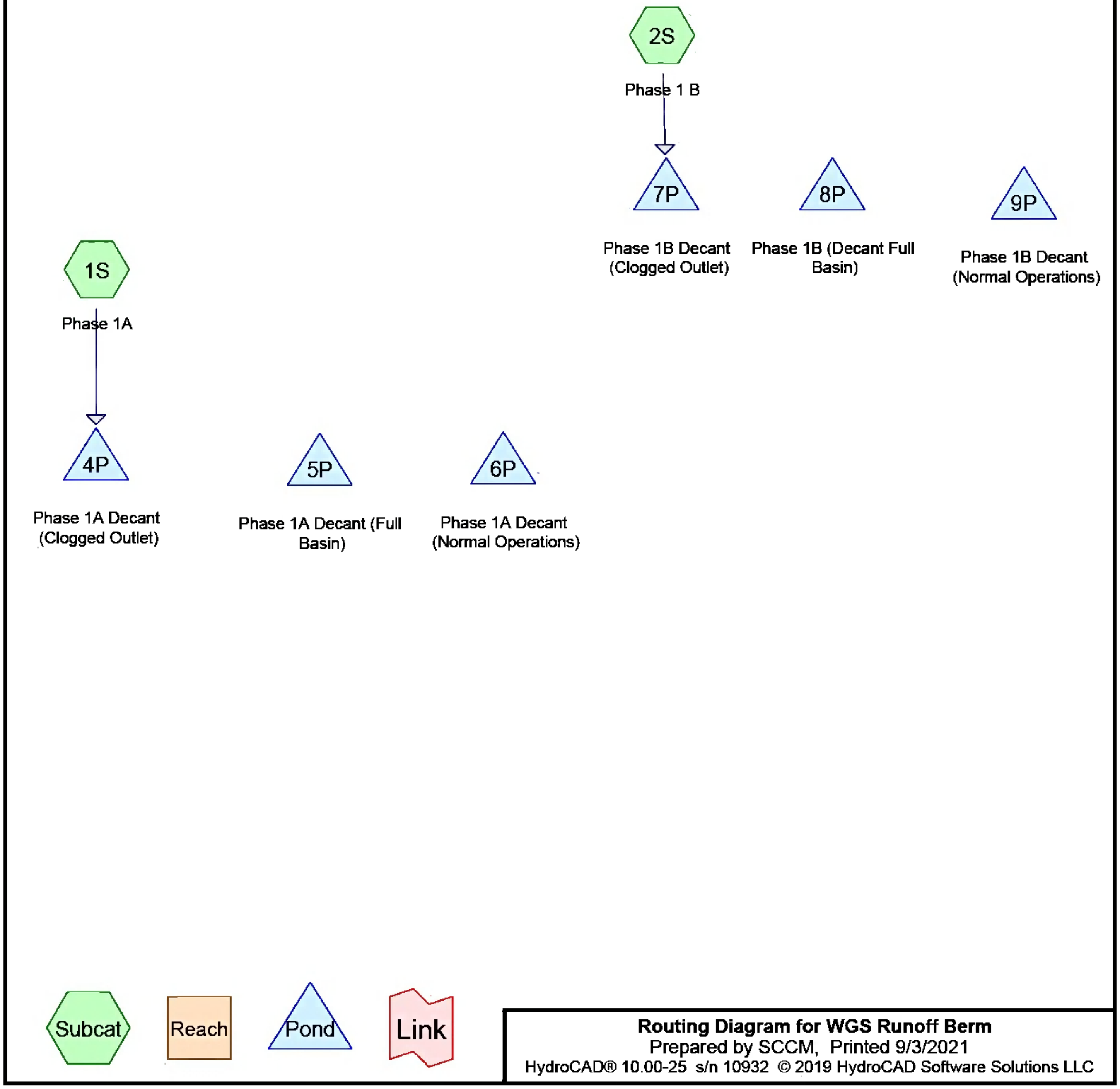
Construction Phase	HydroCAD Scenario	Peak Inflow (cfs)		Peak Outflow (cfs)		Peak Water Surface Elevation (ft)	
		25-yr, 24-hr storm	100-yr, 24-hr storm	25-yr, 24-hr storm	100-yr, 24-hr storm	25-yr, 24-hr storm	100-yr, 24-hr storm
1A	Normal Operations	167.05	220.36	5.26	7.46	53	53.56
	Clogged Decant Pipe	167.05	220.36	0	0	53.56	54.2
	Full Attenuation Basin	167.05	220.36	0	0	56.68	56.91
1B	Normal Operations	92.63	122.19	6.3	9.3	83.23	83.98
	Clogged Decant Pipe	92.63	122.19	0	0	84.34	84.87
	Full Attenuation Basin	92.63	122.19	0	0	86.69	86.91

TABLE 4 – Construction Lift Phase Area and Volume

Construction Phase	HydroCAD Scenario	25-yr, 24-hr Water Elevation (ft)	Fill Plan Berm Elevation (ft)	Freeboard (ft)	100-yr, 24-hr Water Elevation (ft)	Fill Plan Berm Elevation (ft)	Freeboard (ft)
1A	Normal Operations	53	58	5	53.56	58	4.44
	Clogged Decant Pipe	53.56	58	4.44	54.2	58	3.8
	Full Attenuation Basin	56.68	58	1.32	56.91	58	1.09
1B	Normal Operations	83.23	88	4.77	83.98	88	4.02
	Clogged Decant Pipe	84.34	88	3.66	84.87	88	3.13
	Full Attenuation Basin	86.69	88	1.31	86.91	88	1.09

APPENDIX A

HYDROCAD REPORTS



Routing Diagram for WGS Runoff Berm
Prepared by SCCM, Printed 9/3/2021
HydroCAD® 10.00-25 s/n 10932 © 2019 HydroCAD Software Solutions LLC

Area Listing (selected nodes)

Area (acres)	CN	Description (subcatchment-numbers)
32.800	97	(1S, 2S)
32.800	**97**	**TOTAL AREA**

Soil Listing (selected nodes)

Area (acres)	Soil Group	Subcatchment Numbers
0.000	HSG A	
0.000	HSG B	
0.000	HSG C	
0.000	HSG D	
32.800	Other	1S, 2S
32.800		**TOTAL AREA**

Ground Covers (selected nodes)

HSG-A (acres)	HSG-B (acres)	HSG-C (acres)	HSG-D (acres)	Other (acres)	Total (acres)	Ground Cover	Subcatchment Numbers
0.000	0.000	0.000	0.000	32.800	32.800		1S, 2S
0.000	**0.000**	**0.000**	**0.000**	**32.800**	**32.800**	**TOTAL AREA**	

Pipe Listing (selected nodes)

Line#	Node Number	In-Invert (feet)	Out-Invert (feet)	Length (feet)	Slope (ft/ft)	n	Diam/Width (inches)	Height (inches)	Inside-Fill (inches)
1	6P	36.00	29.50	1,300.0	0.0050	0.009	18.0	0.0	0.0
2	9P	36.00	29.50	1,300.0	0.0050	0.009	36.0	0.0	0.0

Time span=0.00-36.00 hrs, dt=0.05 hrs, 721 points
Runoff by SCS TR-20 method, UH=SCS, Weighted-CN
Reach routing by Dyn-Stor-Ind method - Pond routing by Dyn-Stor-Ind method

Subcatchment 1S: Phase 1A	Runoff Area=21.100 ac 0.00% Impervious Runoff Depth=7.92" Tc=6.0 min CN=97 Runoff=167.05 cfs 13.926 af
Subcatchment 2S: Phase 1 B	Runoff Area=11.700 ac 0.00% Impervious Runoff Depth=7.92" Tc=6.0 min CN=97 Runoff=92.63 cfs 7.722 af
Pond 4P: Phase 1A Decant (Clogged	Peak Elev=53.56' Storage=606,623 cf Inflow=167.05 cfs 13.926 af Outflow=0.00 cfs 0.000 af
Pond 5P: Phase 1A Decant (Full Basin)	Peak Elev=56.68' Storage=606,623 cf Inflow=167.05 cfs 13.926 af Outflow=0.00 cfs 0.000 af
Pond 6P: Phase 1A Decant (Normal	Peak Elev=53.00' Storage=461,628 cf Inflow=167.05 cfs 13.926 af Outflow=5.26 cfs 7.806 af
Pond 7P: Phase 1B Decant (Clogged	Peak Elev=84.34' Storage=336,374 cf Inflow=92.63 cfs 7.722 af Outflow=0.00 cfs 0.000 af
Pond 8P: Phase 1B (Decant Full Basin)	Peak Elev=86.69' Storage=336,374 cf Inflow=92.63 cfs 7.722 af Outflow=0.00 cfs 0.000 af
Pond 9P: Phase 1B Decant (Normal	Peak Elev=83.23' Storage=225,334 cf Inflow=92.63 cfs 7.722 af Outflow=6.30 cfs 5.723 af

Total Runoff Area = 32.800 ac Runoff Volume = 21.648 af Average Runoff Depth = 7.92"
100.00% Pervious = 32.800 ac 0.00% Impervious = 0.000 ac

Summary for Subcatchment 1S: Phase 1A

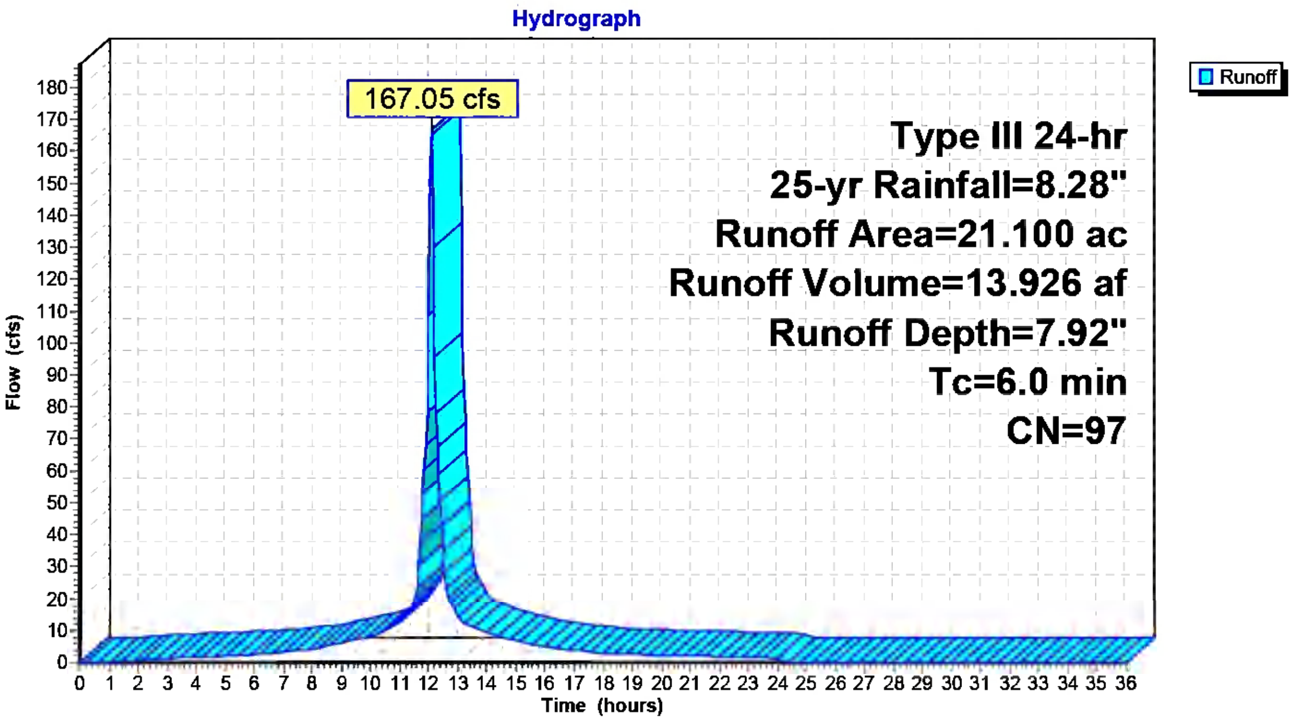
Runoff = 167.05 cfs @ 12.09 hrs, Volume= 13.926 af, Depth= 7.92"

Runoff by SCS TR-20 method, UH=SCS, Weighted-CN, Time Span= 0.00-36.00 hrs, dt= 0.05 hrs
Type III 24-hr 25-yr Rainfall=8.28"

	Area (ac)	CN	Description
*	21.100	97	
	21.100		100.00% Pervious Area

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

Subcatchment 1S: Phase 1A



Summary for Subcatchment 2S: Phase 1 B

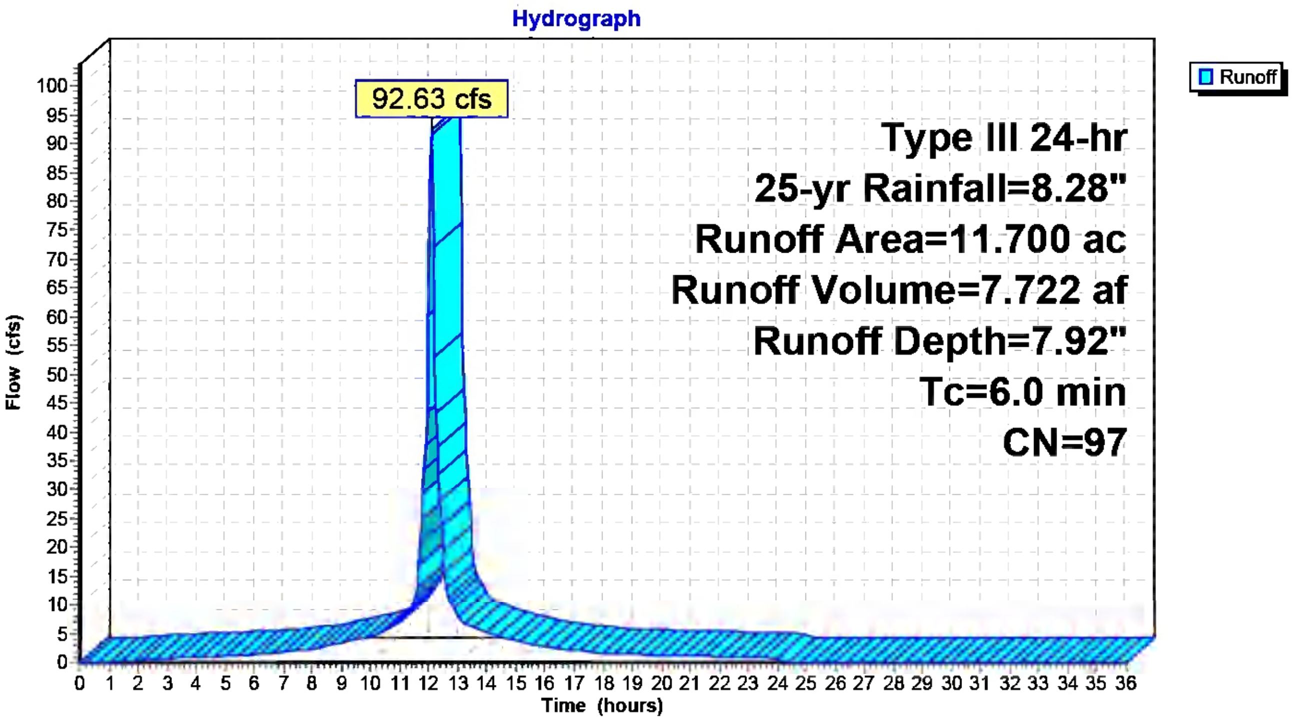
Runoff = 92.63 cfs @ 12.09 hrs, Volume= 7.722 af, Depth= 7.92"

Runoff by SCS TR-20 method, UH=SCS, Weighted-CN, Time Span= 0.00-36.00 hrs, dt= 0.05 hrs
Type III 24-hr 25-yr Rainfall=8.28"

	Area (ac)	CN	Description
*	11.700	97	
	11.700		100.00% Pervious Area

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

Subcatchment 2S: Phase 1 B



Summary for Pond 4P: Phase 1A Decant (Clogged Outlet)

```
Inflow Area =      21.100 ac,  0.00% Impervious,  Inflow Depth =  7.92"   for  25-yr event
Inflow      =    167.05 cfs @  12.09 hrs,  Volume=        13.926 af
Outflow     =      0.00 cfs @   0.00 hrs,  Volume=         0.000 af,  Atten= 100%,  Lag= 0.0 min
```

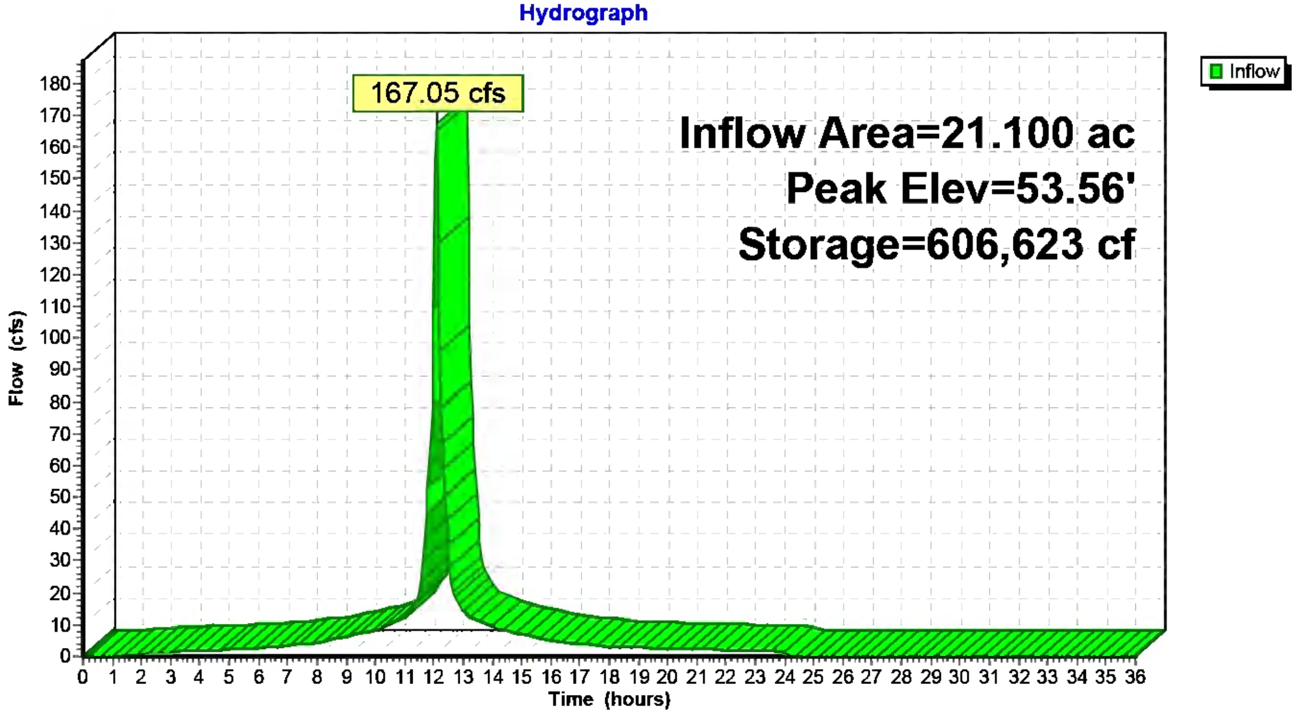
Routing by Dyn-Stor-Ind method, Time Span= 0.00-36.00 hrs, dt= 0.05 hrs
Peak Elev= 53.56' @ 24.40 hrs Surf.Area= 282,229 sf Storage= 606,623 cf

Plug-Flow detention time= (not calculated: initial storage exceeds outflow)
Center-of-Mass det. time= (not calculated: no outflow)

Volume	Invert	Avail.Storage	Storage Description
#1	50.00'	3,746,462 cf	**Custom Stage Data (Prismatic)**Listed below (Recalc)

Elevation (feet)	Surf.Area (sq-ft)	Inc.Store (cubic-feet)	Cum.Store (cubic-feet)
50.00	133,207	0	0
52.00	142,034	275,241	275,241
54.00	321,524	463,558	738,799
56.00	884,036	1,205,560	1,944,359
58.00	918,067	1,802,103	3,746,462

Pond 4P: Phase 1A Decant (Clogged Outlet)



Summary for Pond 5P: Phase 1A Decant (Full Basin)

```
Inflow Area =      21.100 ac,   0.00% Impervious,  Inflow Depth =   7.92"    for  25-yr event
Inflow      =    167.05 cfs @   12.09 hrs,  Volume=          13.926 af
Outflow     =      0.00 cfs @    0.00 hrs,  Volume=           0.000 af,  Atten= 100%,  Lag= 0.0 min
```

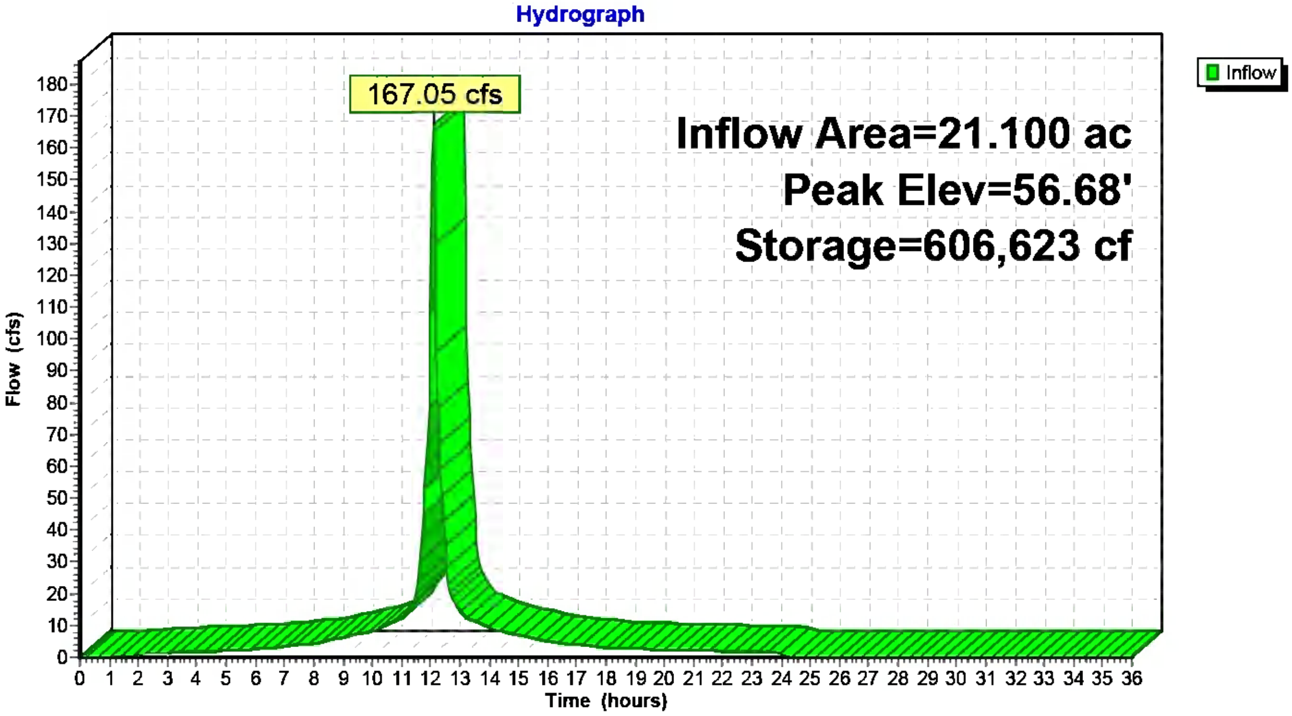
Routing by Dyn-Stor-Ind method, Time Span= 0.00-36.00 hrs, dt= 0.05 hrs
Peak Elev= 56.68' @ 24.40 hrs   Surf.Area= 895,636 sf   Storage= 606,623 cf

Plug-Flow detention time= (not calculated: initial storage exceeds outflow)
Center-of-Mass det. time= (not calculated: no outflow)

Volume	Invert	Avail.Storage	Storage Description
#1	56.00'	1,802,103 cf	**Custom Stage Data (Prismatic)** Listed below (Recalc)

Elevation (feet)	Surf.Area (sq-ft)	Inc.Store (cubic-feet)	Cum.Store (cubic-feet)
56.00	884,036	0	0
58.00	918,067	1,802,103	1,802,103

Pond 5P: Phase 1A Decant (Full Basin)



Summary for Pond 6P: Phase 1A Decant (Normal Operations)

```
Inflow Area =      21.100 ac,   0.00% Impervious,  Inflow Depth =  7.92"   for  25-yr event
Inflow      =     167.05 cfs @  12.09 hrs,  Volume=        13.926 af
Outflow     =       5.26 cfs @  15.75 hrs,  Volume=         7.806 af,  Atten= 97%,  Lag= 220.0 min
Primary     =       5.26 cfs @  15.75 hrs,  Volume=         7.806 af
```

Routing by Dyn-Stor-Ind method, Time Span= 0.00-36.00 hrs, dt= 0.05 hrs
Peak Elev= 53.00' @ 15.75 hrs   Surf.Area= 231,578 sf   Storage= 461,628 cf

Plug-Flow detention time= 701.6 min calculated for 7.806 af (56% of inflow)
Center-of-Mass det. time= 583.7 min (1,330.3 - 746.6)

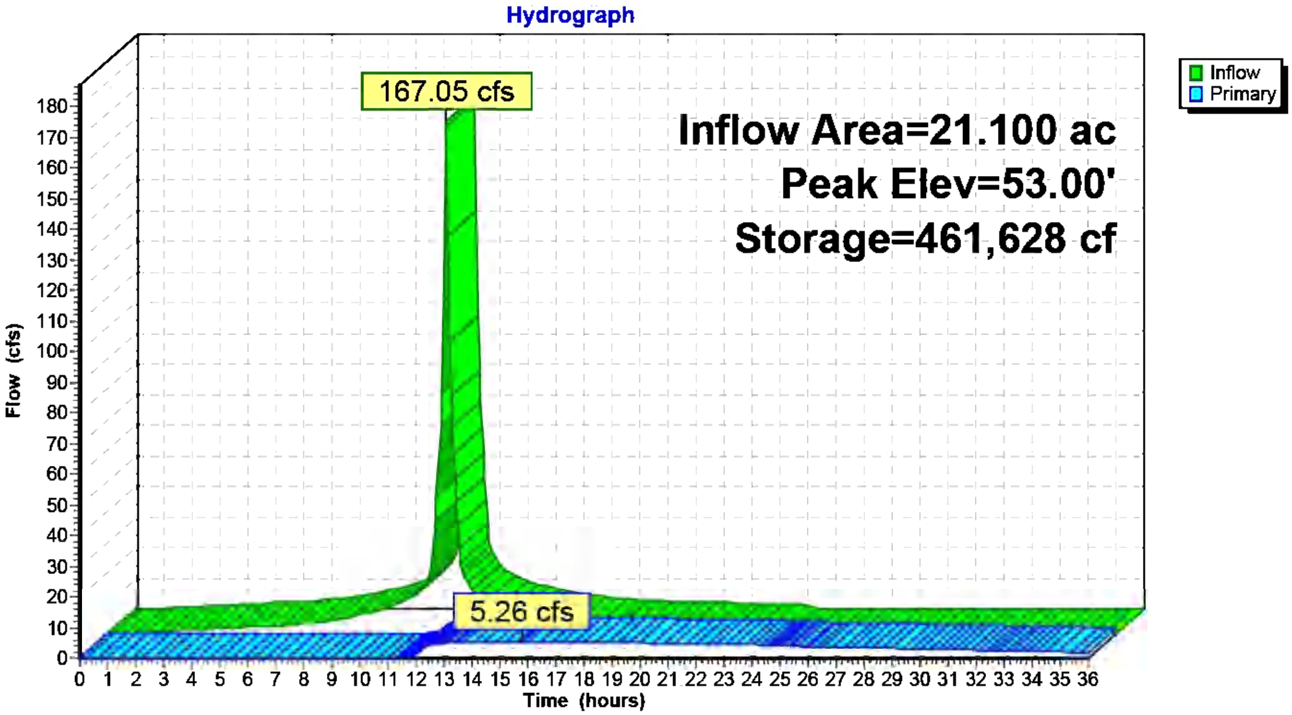
Volume	Invert	Avail.Storage	Storage Description
#1	50.00'	3,746,462 cf	**Custom Stage Data (Prismatic)**Listed below (Recalc)

Elevation (feet)	Surf.Area (sq-ft)	Inc.Store (cubic-feet)	Cum.Store (cubic-feet)
50.00	133,207	0	0
52.00	142,034	275,241	275,241
54.00	321,524	463,558	738,799
56.00	884,036	1,205,560	1,944,359
58.00	918,067	1,802,103	3,746,462

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

```
Primary OutFlow Max=5.26 cfs @ 15.75 hrs  HW=53.00'  (Free Discharge)
└─1=Culvert (Passes 5.26 cfs of 18.24 cfs potential flow)
   ├─2=Orifice/Grate ( Controls 0.00 cfs)
   ├─3=Orifice/Grate ( Controls 0.00 cfs)
   ├─4=Orifice/Grate ( Controls 0.00 cfs)
   ├─5=Orifice/Grate ( Controls 0.00 cfs)
   ├─6=Orifice/Grate ( Controls 0.00 cfs)
   ├─7=Orifice/Grate (Orifice Controls 0.81 cfs @ 3.10 fps)
   ├─8=Orifice/Grate (Orifice Controls 1.21 cfs @ 4.60 fps)
   ├─9=Orifice/Grate (Orifice Controls 1.50 cfs @ 5.73 fps)
   └─10=Orifice/Grate (Orifice Controls 1.74 cfs @ 6.66 fps)
```

Pond 6P: Phase 1A Decant (Normal Operations)



Summary for Pond 7P: Phase 1B Decant (Clogged Outlet)

```
Inflow Area =      11.700 ac,   0.00% Impervious,  Inflow Depth =   7.92"    for  25-yr event
Inflow      =      92.63 cfs @  12.09 hrs,  Volume=          7.722 af
Outflow     =       0.00 cfs @   0.00 hrs,  Volume=          0.000 af,  Atten= 100%,  Lag= 0.0 min
```

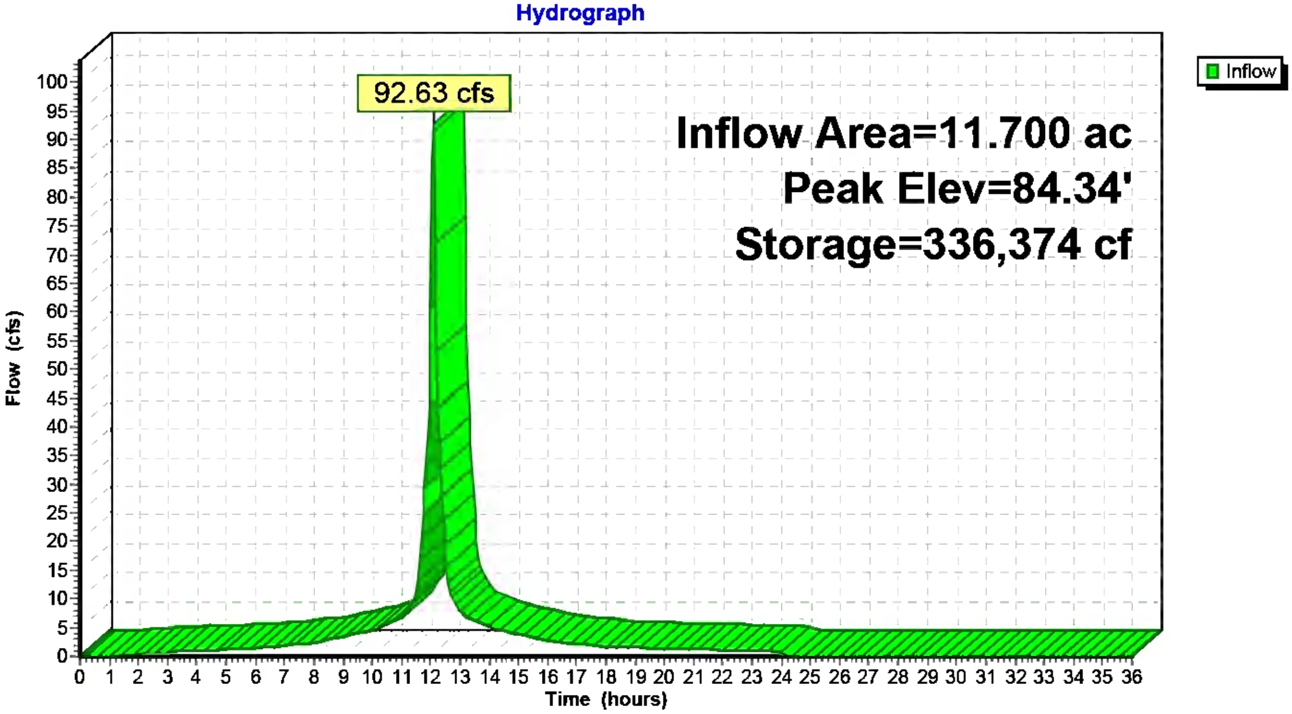
Routing by Dyn-Stor-Ind method, Time Span= 0.00-36.00 hrs, dt= 0.05 hrs
Peak Elev= 84.34' @ 24.40 hrs Surf.Area= 159,611 sf Storage= 336,374 cf

Plug-Flow detention time= (not calculated: initial storage exceeds outflow)
Center-of-Mass det. time= (not calculated: no outflow)

Volume	Invert	Avail.Storage	Storage Description
#1	80.00'	1,867,138 cf	**Custom Stage Data (Prismatic)**Listed below (Recalc)

Elevation (feet)	Surf.Area (sq-ft)	Inc.Store (cubic-feet)	Cum.Store (cubic-feet)
80.00	62,804	0	0
82.00	68,978	131,782	131,782
84.00	92,642	161,620	293,402
86.00	485,761	578,403	871,805
88.00	509,572	995,333	1,867,138

Pond 7P: Phase 1B Decant (Clogged Outlet)



Summary for Pond 8P: Phase 1B (Decant Full Basin)

```
Inflow Area =      11.700 ac,   0.00% Impervious,  Inflow Depth =   7.92"    for  25-yr event
Inflow      =     92.63 cfs @  12.09 hrs,  Volume=          7.722 af
Outflow     =      0.00 cfs @   0.00 hrs,  Volume=          0.000 af,  Atten= 100%,  Lag= 0.0 min
```

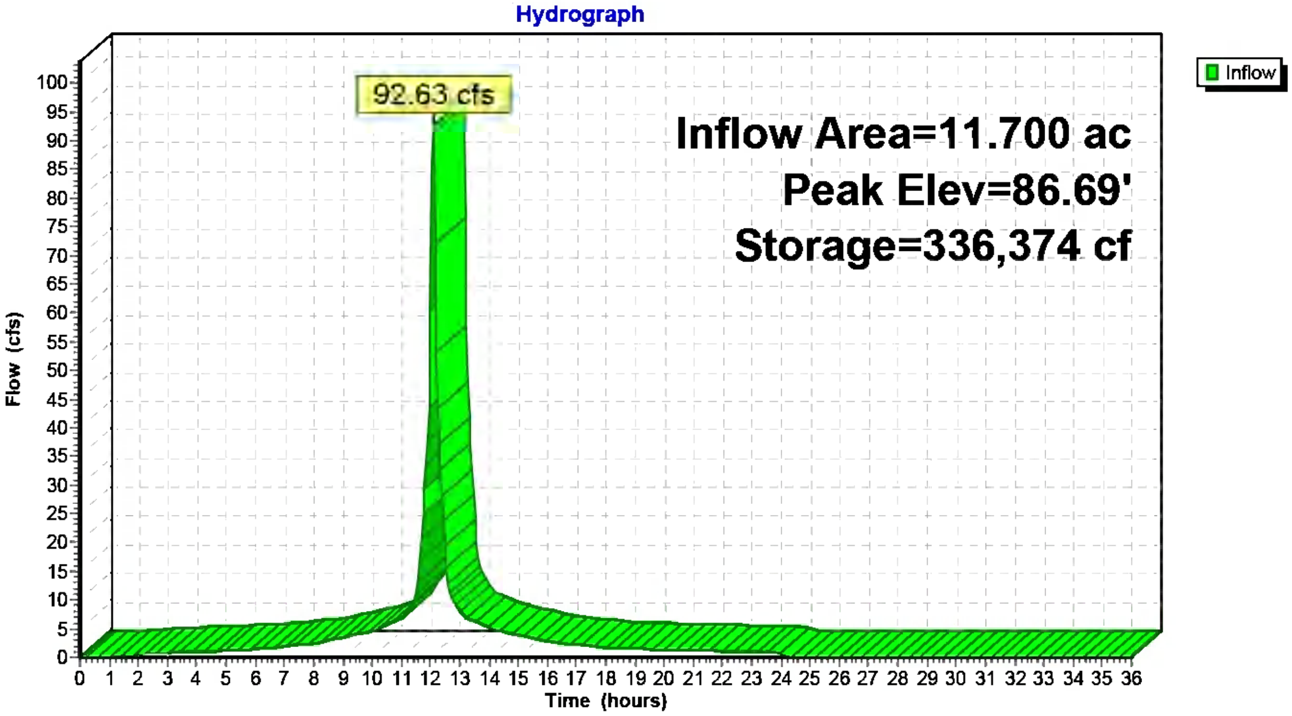
Routing by Dyn-Stor-Ind method, Time Span= 0.00-36.00 hrs, dt= 0.05 hrs
Peak Elev= 86.69' @ 24.40 hrs Surf.Area= 493,936 sf Storage= 336,374 cf

Plug-Flow detention time= (not calculated: initial storage exceeds outflow)
Center-of-Mass det. time= (not calculated: no outflow)

Volume	Invert	Avail.Storage	Storage Description
#1	86.00'	995,333 cf	**Custom Stage Data (Prismatic)**Listed below (Recalc)

Elevation (feet)	Surf.Area (sq-ft)	Inc.Store (cubic-feet)	Cum.Store (cubic-feet)
86.00	485,761	0	0
88.00	509,572	995,333	995,333

Pond 8P: Phase 1B (Decant Full Basin)



Summary for Pond 9P: Phase 1B Decant (Normal Operations)

Inflow Area = 11.700 ac, 0.00% Impervious, Inflow Depth = 7.92" for 25-yr event
Inflow = 92.63 cfs @ 12.09 hrs, Volume= 7.722 af
Outflow = 6.30 cfs @ 13.44 hrs, Volume= 5.723 af, Atten= 93%, Lag= 81.3 min
Primary = 6.30 cfs @ 13.44 hrs, Volume= 5.723 af

Routing by Dyn-Stor-Ind method, Time Span= 0.00-36.00 hrs, dt= 0.05 hrs
Peak Elev= 83.23' @ 13.44 hrs Surf.Area= 83,497 sf Storage= 225,334 cf

Plug-Flow detention time= 513.4 min calculated for 5.723 af (74% of inflow)
Center-of-Mass det. time= 425.0 min (1,171.6 - 746.6)

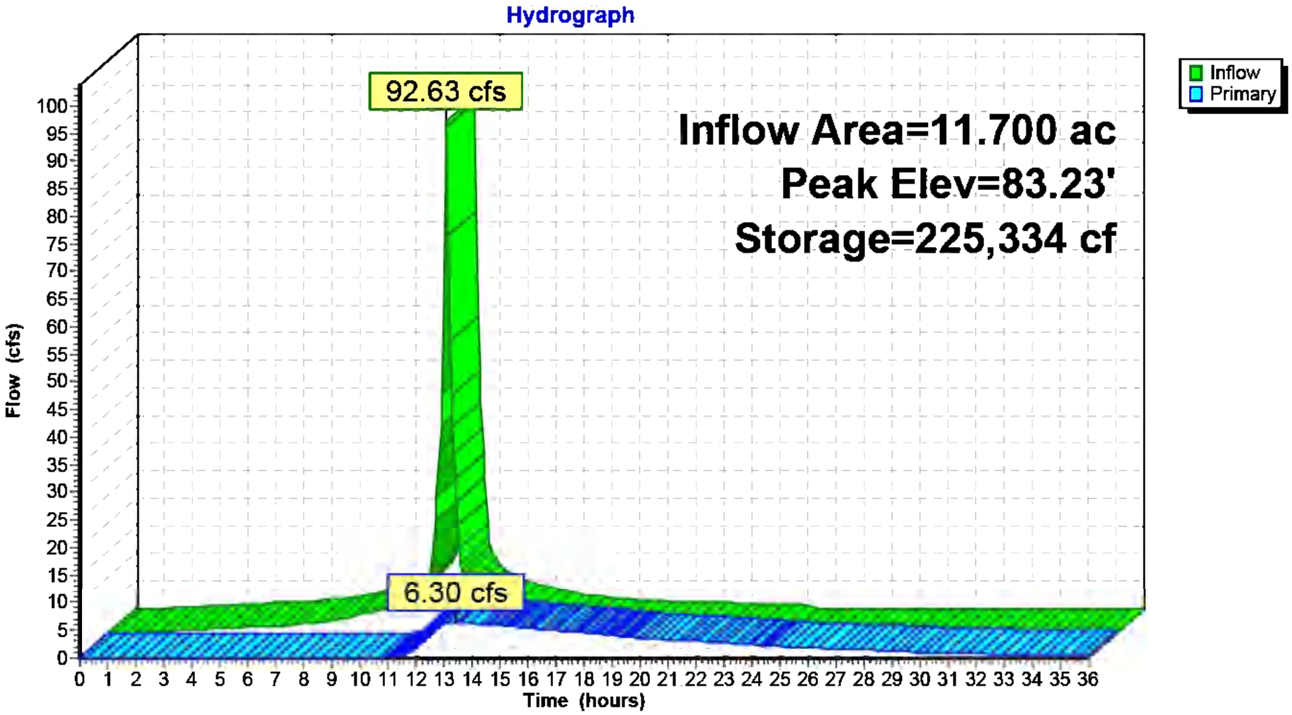
Volume	Invert	Avail.Storage	Storage Description
#1	80.00'	1,867,138 cf	**Custom Stage Data (Prismatic)**Listed below (Recalc)

Elevation (feet)	Surf.Area (sq-ft)	Inc.Store (cubic-feet)	Cum.Store (cubic-feet)
80.00	62,804	0	0
82.00	68,978	131,782	131,782
84.00	92,642	161,620	293,402
86.00	485,761	578,403	871,805
88.00	509,572	995,333	1,867,138

Device	Routing	Invert	Outlet Devices
#1	Primary	36.00'	**36.0" Round Culvert** L= 1,300.0' CPP, projecting, no headwall, Ke= 0.900 Inlet / Outlet Invert= 36.00' / 29.50' S= 0.0050 '/' Cc= 0.900 n= 0.009, Flow Area= 7.07 sf
#2	Device 1	85.00'	**36.0" Horiz. Orifice/Grate** C= 0.600 Limited to weir flow at low heads
#3	Primary	84.50'	**2.0" Vert. Orifice/Grate X 12.00** C= 0.600
#4	Primary	84.00'	**2.0" Vert. Orifice/Grate X 12.00** C= 0.600
#5	Primary	83.50'	**2.0" Vert. Orifice/Grate X 12.00** C= 0.600
#6	Primary	83.00'	**2.0" Vert. Orifice/Grate X 12.00** C= 0.600
#7	Primary	82.50'	**2.0" Vert. Orifice/Grate X 12.00** C= 0.600
#8	Primary	82.00'	**2.0" Vert. Orifice/Grate X 12.00** C= 0.600
#9	Primary	81.50'	**2.0" Vert. Orifice/Grate X 12.00** C= 0.600
#10	Primary	81.00'	**2.0" Vert. Orifice/Grate X 12.00** C= 0.600

Primary OutFlow Max=6.30 cfs @ 13.44 hrs HW=83.23' (Free Discharge)
↑—**1=Culvert** (Passes 0.00 cfs of 159.41 cfs potential flow)
  ↑—**2=Orifice/Grate** (Controls 0.00 cfs)
├—**3=Orifice/Grate** (Controls 0.00 cfs)
├—**4=Orifice/Grate** (Controls 0.00 cfs)
├—**5=Orifice/Grate** (Controls 0.00 cfs)
├—**6=Orifice/Grate** (Orifice Controls 0.48 cfs @ 1.83 fps)
├—**7=Orifice/Grate** (Orifice Controls 1.01 cfs @ 3.86 fps)
├—**8=Orifice/Grate** (Orifice Controls 1.35 cfs @ 5.15 fps)
├—**9=Orifice/Grate** (Orifice Controls 1.62 cfs @ 6.17 fps)
└—**10=Orifice/Grate** (Orifice Controls 1.85 cfs @ 7.05 fps)

Pond 9P: Phase 1B Decant (Normal Operations)



APPENDIX I-B

Calculations for Run-On and Run-Off Perimeter Controls

SURFACE WATER MANAGEMENT SYSTEM DESIGN
CLOSURE TURF® FINAL COVER SYSTEM

5/27/2020

South Carolina Licensed Professional Engineer No. 33535 Scott M. Graves

South Carolina Geosyntec Consultants, Inc. No. 773 Certificate of Authorization

SEALED FOR
CALCULATION PAGES 1 THROUGH 109

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 the proposed Class Three Landfill (composed of Landfill Area 1 and Landfill Area 2) at the Winyah Generating Station (WGS) located in Georgetown County, South Carolina. Because the proposed alternative geomembrane/artificial turf system known as the ClosureTurf® final cover system will generate higher runoff volume as compared to the currently-permitted final cover system with a vegetated (grass) surface, Geosyntec evaluated the capacities of the existing stormwater structures. The hydrologic analysis includes calculation of peak discharges and total runoff volumes from the landfill areas during final conditions (i.e., post-development conditions after closure), when the final cover system for the landfill is in-place, as are the permanent surface water management system conveyance features. The results of 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 Engineering Drawing set that accompanies the Engineering Report in the approved permit application shows the final grading plan of each landfill area, and also includes a series of drawings that shows the surface water management system features. Key features of the surface water management system at Landfill Area 1 and 2 are as follows:

- The final cover system ground surface will have sideslopes inclined at three horizontal to one vertical (3H:1V) in-between drainage terraces and top surface slopes (top deck areas) are inclined at a nominal 3 to 5 percent slope. The sideslope drainage terraces are spaced approximately every 30 feet vertically, and with typical drainage profile slopes at 2 percent. Sheet flow runoff upgradient from each sideslope drainage terrace will be captured in the terraces, where the water will be conveyed to downdrain pipes spaced periodically around each landfill area.

- Downdrain pipes will outlet into either perimeter drainage reaches, or into existing site drainage features (i.e., the discharge canal or cooling pond). At each location where downdrain pipes outlet to the perimeter drainage channel, a concrete outlet apron will be used. At each location where downdrain pipes outlet directly to existing drainage features (discharge canal or cooling pond), riprap aprons will be used to dissipate energy.

- Perimeter drainage reaches will 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.

- Each landfill area will include a final cover access road, with an adjacent drainage channel ("access road drainage terrace") located on the inside edge of the road. The access road drainage terraces will receive sheet flow runoff from the road along with some sheet flow from adjacent landfill sideslopes and will convey water to either a culvert beneath the access road or to downdrain pipes spaced periodically around each landfill area. Culverts will convey flow beneath roads where the landfill perimeter access roads cross the perimeter reaches. The hydraulic capacity of the roadway culverts was also evaluated.

Figures 2 and 3 of this calculation package show the layout of the final cover grading of Landfill Areas 1 and 2, respectively, along with a delineation of drainage areas.

2. METHODOLOGY

2.1 HEC-HMS Computer Model

Surface water discharges are estimated using the Hydrologic Modeling System (HEC-HMS) computer program Version 4.1 developed through the Hydraulic Engineering Center (HEC) of the United States Army Corps of Engineers (USACE). The program simulates natural and controlled precipitation-runoff and routing processes of a watershed. For precipitation-runoff-routing simulation, HEC-HMS 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% annual chance), 24-hour duration event (herein referred to as the 25-year, 24-hour event); and the 100-year (1% annual chance), 24-hour duration event (herein referred to as the 100-year, 24-hour event).

- Water loss models can estimate the volume of runoff given the precipitation and properties of the watershed. For this analysis, the National Resource Conservation Service (NRCS) Curve Number Loss Model was used (USDA, 1986).

- Direct runoff transform models can account for overland flow, storage, and energy losses as surface water runs off a watershed and into the drainage channels. For this analysis, the NRCS Unit Hydrograph Model was selected.

- Hydraulic routing models account for storage and energy flux as surface water flows through drainage channels. The Kinematic Wave Model was selected for these analyses.

HEC-HMS 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, downdrain pipes, and perimeter drainage channels and pipes).

2.1.1 Estimation of Time of Concentration for HEC-HMS NRCS Curve Number Method

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

$P_{2\text{-}24}$ = 2-year, 24-hour rainfall (in.); and

S = slope of hydraulic grade line (or land slope) (ft/ft).

The slope of the hydraulic grade line, or land slope (*S*), for all subcatchment areas of the final cover system is shown in Appendix 1 of this calculation package. To estimate sheet flow travel time (T_t), a roughness coefficient of 0.11 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 ($P_{2\text{-}24}$) at the site is 4.42 inches (NOAA, 2020).

Based on the designed conveyance system, runoff will be converted from sheet flow to open channel flow relatively quickly, and therefore shallow concentrated flow should be negligible. Surface water runoff within each subcatchment area will sheet flow along the top deck or sideslopes of the final cover system until the water reaches either a drainage terrace or the perimeter drainage channel, at which point the flow will be classified as open channel flow. The method selected to estimate the open channel flow travel time is based on guidance provided in TR-55 (USDA, 1986). Travel time for open channel flow is estimated by dividing the longest drainage path by the velocity of runoff:

$$T_t = \frac{L}{V}\left(\frac{1}{60}\right)$$

where: T_t = travel time (min);

L = flow length (ft); and

V = average velocity (ft/sec).

Open channel flow velocities were estimated using Manning's equation based on guidance provided in TR-55 (USDA, 1986). The average flow velocities were determined for bank-full elevation as:

$$V = \frac{1.49}{n} R_h^{2/3} S^{1/2}$$

where: V = average velocity (ft/s);

n = Manning's roughness coefficient;

R_h = hydraulic radius (ft) = A/P;

A = cross sectional area (ft^2);

P = wetted perimeter (ft); and

S = slope of hydraulic grade line (or longitudinal channel slope for normal flow conditions) (ft/ft).

To estimate open channel flow travel time (T_t), a Manning's roughness coefficient (n) 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. The lag times calculated for each drainage area are presented in Appendix 1 for use in the NRCS Curve Number Method and HEC-HMS software. The lag time is estimated as 0.6 times the time of concentration (USDA, 1986).

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:

$$Q = \frac{1.49}{n} A R^{2/3} S^{1/2}$$

where:

Q = discharge (cfs),

n = Manning's roughness coefficient,

A = area of cross-section of flow (ft^2),

P = wetted perimeter (ft),

R = hydraulic radius = A/P (ft), and

S = slope of hydraulic grade line (or longitudinal slope for normal flow conditions) (ft/ft).

The average tractive stresses in the drainage terraces for various flows from the design events are estimated by the following equation (Chow, 1959):

$$\tau_o = \gamma_w R S$$

where:

τ_0 = average tractive stress (lb/ft^2),

γ_w = unit weight of water (lb/ft^3),

R = hydraulic radius = A/P (ft), and

S = channel slope (ft/ft).

Manning's equation was used to calculate the resulting flow depths and velocities to confirm that the drainage features are appropriately sized. 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 HEC-HMS modeling results relate to the contributing drainage areas for each downdrain pipe feature. Each individual sideslope drainage terrace and top-deck drainage terraces constitute a portion of the total contributing area to a downdrain pipe feature. Therefore, the total flow rate within the downdrain pipe feature was scaled based on area to estimate the expected flow rate to each drainage terrace within the overall contributing area. For example, the subcatchment contributing to downdrain pipe feature 1A1 has a total area of 4.92 acres. The subcatchment can be divided into six separate subareas corresponding to each sideslope drainage terraces within this subcatchment. The farthest downgradient sideslope drainage terrace on the west side of the downdrain pipe feature 1A1 (i.e., one of the six drainage terraces for this downdrain pipe) has an area of 0.69 acres. The expected flow rate within that drainage terrace is estimated as 0.69 acres divided by the total area (4.92 acres) times the peak flow rate (34.58 cfs for a 25-year event). This calculation results in an expected peak flow rate of 4.83 cfs for this specific sideslope drainage terrace. Similar calculations were conducted for every drainage terrace, and the largest subarea contributing to a drainage terrace (i.e., the largest expected flow rate within all the drainage terraces) was used to calculate the critical peak flow rate required for designing adequate capacity within the drainage terrace.

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 HEC-HMS. Each reach is designed to convey the peak surface water runoff rate corresponding to the 100-year (i.e., one percent annual chance of occurrence), 24-hour duration rainfall event flowing to the channel reach. The hydraulic capacity of each perimeter drainage channel 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):

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

where: Q = flow (ft^3/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):

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

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/s^2) = 32.2 ft/s^2.

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

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

where:

H_0= 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/s^2) = 32.2 ft/s^2.

An exit loss coefficient (K) of 1.0 is usually recommended by 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% 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 two feet per second 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 culverts was assessed by utilizing the HY-8 Culvert Analysis Program v.7.5 (HY-8). The performance of the culverts 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 Federal Highway Administration (FHWA) provides a methodology for calculating the median riprap size (d_{50}) and required length of apron (L_a) based on the culvert diameter and design flow rate. The median riprap size is calculated using the following equation (FHWA, 2006):

$$d_{50} = 0.2D\left[\frac{Q}{\sqrt{g}D^{2.5}}\right]^{4/3}\frac{D}{TW}$$

where: d_{50} = median riprap size (ft),

Q = design discharge (cfs),

D = culvert diameter (ft),

TW = tailwater depth (ft), and

g = gravitational constant (32.2 ft/s^2).

The tailwater depth should be limited to between 0.4D and D. FHWA (2006) recommends the use of a tailwater depth equal to 0.4D if the tailwater conditions are unknown. Once the median riprap size is calculated, the required length and depth (i.e., thickness) of the riprap apron can be estimated using Table 3 provided by the FHWA (2006). The width of the riprap 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 HEC-HMS

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 8.28 inches and 10.9 inches, respectively, as shown in Table 1 (NOAA, 2019). 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 1.

- Curve Number (CN) – Curve number for ClosureTurf® were obtained from the WatershedGeo Design Guidelines Manual (WatershedGeo, 2019). Proposed final cover of the CCR landfill will have a CN = 95, as shown in Table 5.

- 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 downdrain pipes.

- Perimeter Channel Reaches –The Kinematic Wave Model is used to model the surface water flow in each of the reaches in the HEC-HMS program. The Kinematic Wave Model accounts for storage and energy flux as surface water moves through stream channels. Average geometric characteristics of the stream channel measured from the existing and proposed topography are input into HEC-HMS.

3.1.3 Nodal Network Diagrams

Figure 9 of this calculation package presents the nodal network diagrams for the final landfill conditions of both landfill areas (Area 1 and Area 2). The nodal network diagrams show the subbasins, reaches, and discharge locations.

3.2 Drainage Terrace and Downdrain

The hydraulic capacity of the drainage terraces and downdrains were evaluated to meet or exceed the applicable requirements of the State of South Carolina Department of Health and Environmental Control (SCDHEC) regulations for Class Three Landfills. The following design criteria have been adopted:

- the drainage terraces and downdrains are designed to control at least the water volume resulting from a 25-year, 24-hour storm;
- the drainage terraces:
  - are sized to convey runoff from the 25-year, 24-hour design rainfall event with a hydraulic head not more than 1-ft;
  - are sized to convey the 100-year, 24-hour design rainfall event without overtopping;
  - have channel lining materials selected to resist the velocities and/or tractive stresses produced by the 25-year, 24-hour rainfall event; and
- the downdrain pipes are designed to convey the 100-year, 24-hour rainfall event without flowing full (to avoid pressurized flow).

The design parameters used in these calculations, including channel geometry, Manning's roughness coefficient, and calculated peak flows for the 25-year and 100-year events are summarized for each drainage terrace type and downdrain pipe feature in Tables 6 and 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 will be formed by constructing tack-on berms near the edge of the top deck surface. As such, one side of the v-shaped channel will be inclined at 5% based on the top deck surface grades, and the other side of the v-shaped channel will be inclined at 3H:1V based on the constructed berm. The top deck drainage terraces will have a constant depth of 1.5 ft and a longitudinal slope of approximately 0.5%.

A Manning's roughness coefficient of 0.036 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 1B1 of the Area 1 Landfill as shown on the attached Figure 10. This location is critical because it has the largest contributing drainage area of 2.25 acres. The resulting peak discharge rates from the 25-year and 100-year storm events are 15.70 cfs and 20.80 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 (2%). As with the top-deck 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 2F2 of the Area 2 Landfill as shown on the attached Figure 11. This location is critical because it has the largest contributing drainage area of 3.07 acres. The resulting peak discharge rates from the 25-year and 100-year storm events are 21.6 cfs and 28.6 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 (8%).

The channel lining material will be six-inch diameter (average) riprap stones. A Manning's roughness coefficient of 0.036 was selected for the access road drainage terraces, based on recommendations from Chow (1959), as shown in Table 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 Area 1 Landfill as shown on the attached Figure 10. This location is critical because it has the largest contributing drainage area of 2.21 acres. The resulting peak discharge rates from the 25-year and 100-year storm events are 16.2 cfs and 21.4 cfs, respectively.

3.2.4 Downdrains

The downdrains will be 18-inch diameter, smooth-interior, high-density polyethylene (HDPE) pipe. A Manning's roughness coefficient of 0.009 was selected for the downdrains, based on recommendations from Barfuss and Tullis (1989), as shown in Figure 8. In general, the downdrain pipes will follow the slope of the landfill sideslopes, and as such will generally be inclined at a longitudinal slope of 3H:1V. However, the slope will flatten to four percent (4%) at locations where downdrain pipes cross the sideslope drainage terraces, as well as at the landfill perimeter. This much flatter 4% slope governs the hydraulic sizing, and is therefore used to size the downdrains.

Multiple pipes are used side-by-side to form a cluster of pipes at each downdrain feature location, based on the number of pipes needed to collect the runoff generated from each contributing drainage area.

3.3 Perimeter Drainage Reaches and Roadway Culverts

3.3.1 Perimeter Drainage Reaches

Perimeter drainage channels are designed to be 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 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 for of 18 ft/s was selected, based on Table 12 (USDA, 2007).

3.3.2 Roadway Culverts

The roadway culverts near the landfill perimeter areas will be RCP. The design parameters for each 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. All culverts were designed using the following parameters to convey the peak flow rate for the 100-year, 24-hour rainfall event. The inflow structure into the culvert influences the conveyance of surface water through the culvert. The culvert inflow structures were modeled as a square edge with a headwall.

3.3.3 Riprap Apron Design

Riprap aprons were sized for the outflow of each proposed culvert, as well as for the downdrain pipe locations which discharge directly to the canals or cooling pond. The riprap apron design parameters including flow rates, culvert diameter, and riprap apron dimensions depth are presented in Table 14. For the purposes of riprap apron design only, proposed culvert crossings with multiple barrels were modeled with a flow rate evenly distributed to each culvert based on the number of barrels.

4. RESULTS

Table 15 of this calculation package presents a summary of the HEC-HMS hydrologic modeling results. Additional detailed backup, including screen-shots of the modeling output results, is provided in Appendix 1 of this calculation package. The summary of results in Table 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 both Landfill Area 1 and Landfill Area 2. 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 downdrain pipe feature were calculated using the methodology described above. These calculation results are summarized in Table 16 (for drainage terraces) and Table 17 (for downdrain pipe features). In addition, the downdrain pipe feature configurations and associated number of pipes to be utilized at each sideslope drainage terrace were tabulated in Table 18.

Appendix 2 of this calculation package provides spreadsheets used for calculating the results for the downdrain pipe features, access road drainage terraces, top deck drainage terrace, and sideslope drainage terrace with the greatest flow rates (i.e., the critical design cases). The set of Engineering Drawings that accompanies the Engineering Report shows the location and layout of the drainage structures discussed within this calculation package.

The calculation results presented herein demonstrate that:

- Each drainage terrace is designed with the capacity to convey the flows from the 25-year, 24-hour design rainfall event with a hydraulic head not more than 1-ft and to convey the 100-year, 24-hour design rainfall event without overtopping.
- The selected channel lining can adequately resist the velocities and/or tractive stresses produced by the 25-year, 24-hour rainfall event.
- Downdrain pipe features are designed to flow partially full (avoiding pressurized flow) during the 25-year, 24-hour design rainfall event and the 100-year, 24-hour rainfall event, and using multiple pipes side-by-side at locations where more than one pipe is needed to convey the flow from the contributing drainage area.

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 each 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 and downdrain pipe feature 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 designs contain the capacity to convey the design flows without overtopping the downstream roadway for a 100-year, 24-hour rainfall event. Proposed modifications to roadway culvert design include updating Culvert 2BC from 2 barrels with 24-inch diameters to 2 barrels with 30-inch diameters.
- 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.

Note that FHWA (2006) recommends an apron width of three times the outlet diameter or one pipe width wider than the extent of the pipe barrels at the up gradient end of the apron near the culvert outlet and a 3:1 rate of expansion at each edge along the length of the apron. However, several structures are discharging into a stabilized trapezoidal channel and the dimensions of the riprap aprons are restricted by the channel dimensions. Therefore, the entire width of the channel should be lined with riprap in these cases.

5. REFERENCES

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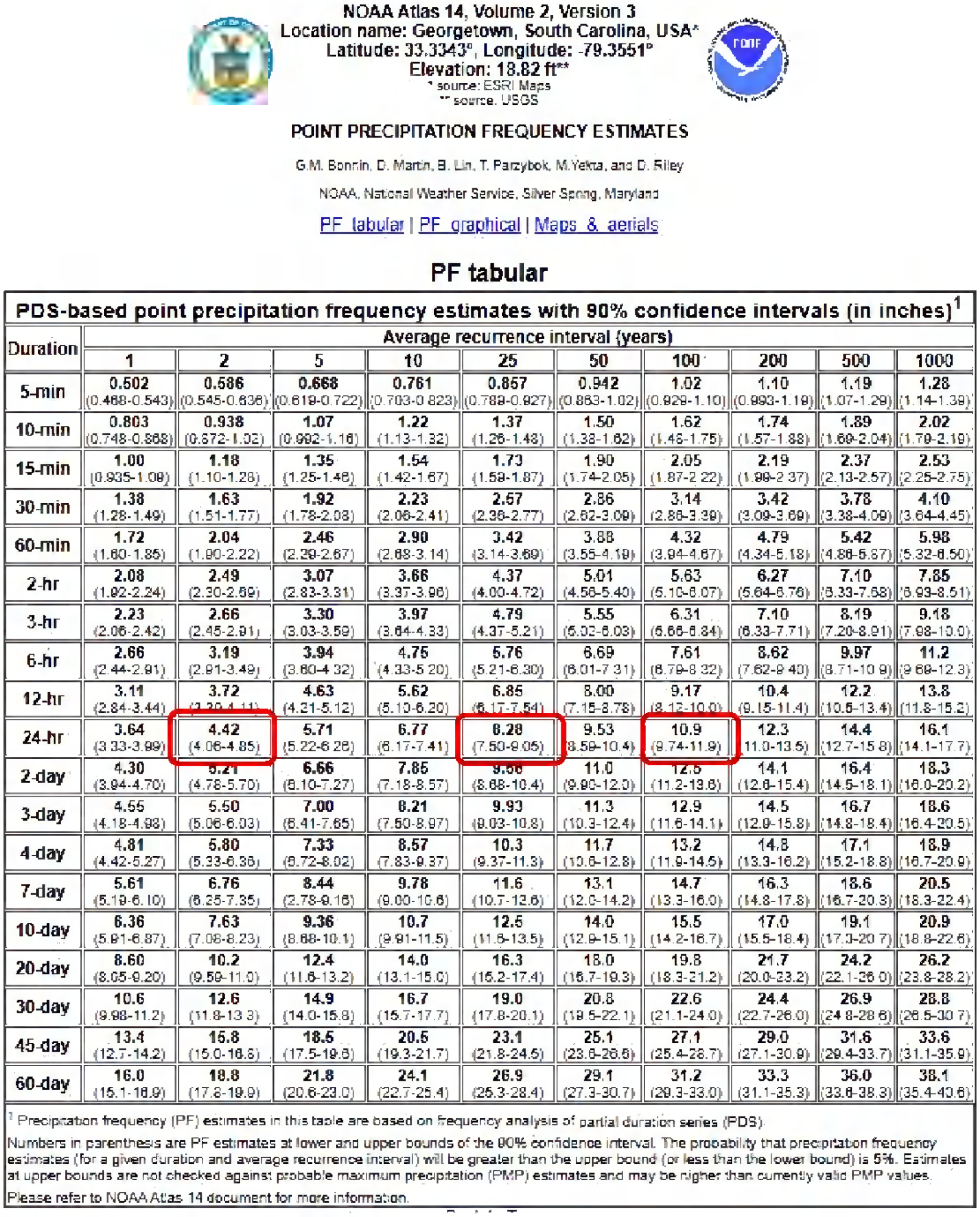
TABLES

- Table 20. 100-year, 24-hour Hydraulic Grade Line for Perimeter Reach Pipes/Culverts Flowing Full
- Table 21. Roadway Culvert Capacity Analysis Results

Table 1. Precipitation Frequency for Georgetown, South Carolina (from NOAA, 2019)



NOAA Atlas 14, Volume 2, Version 3
Location name: Georgetown, South Carolina, USA*
Latitude: 33.3343°, Longitude: -79.3551°
Elevation: 18.82 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]

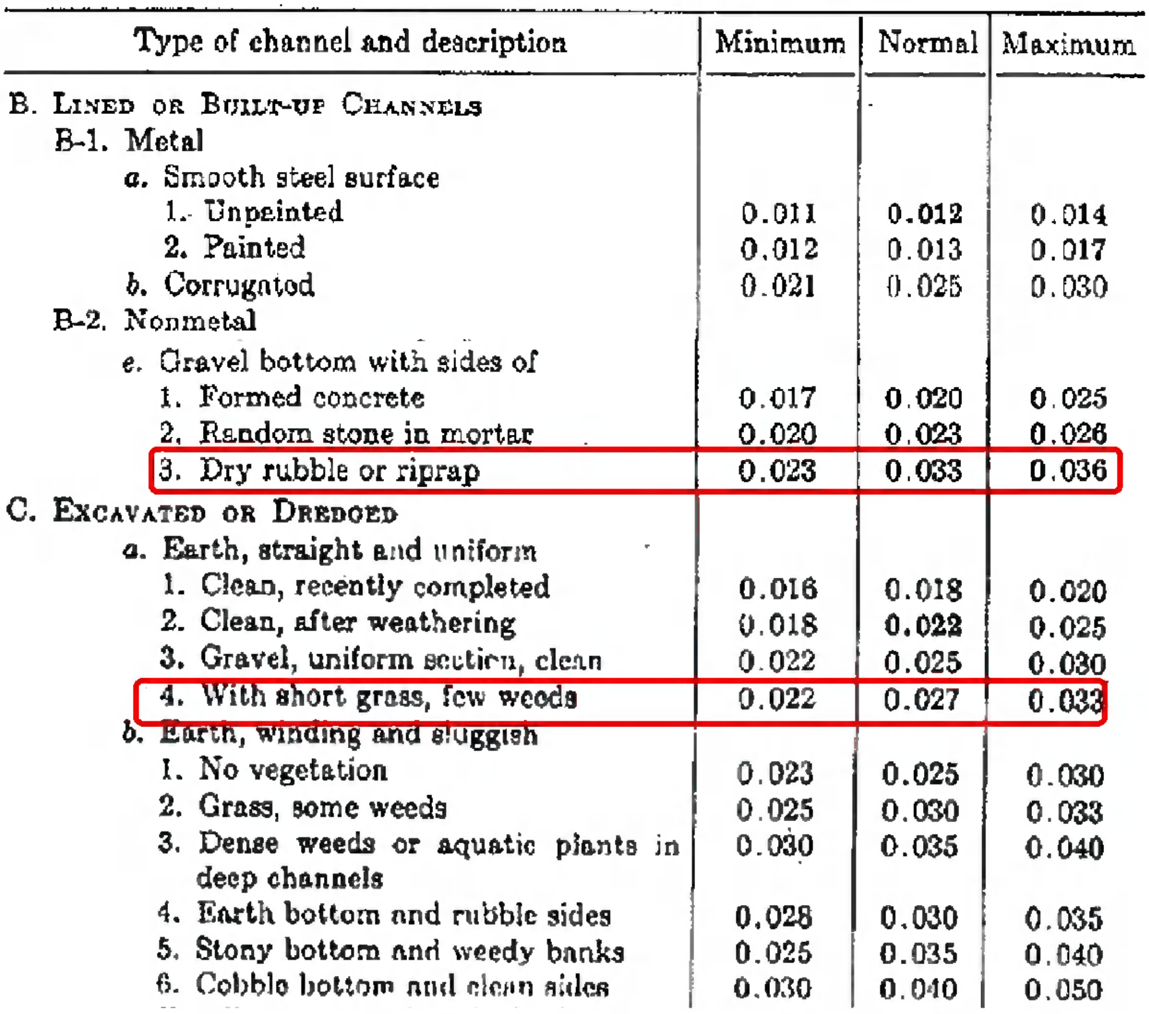
Duration	Average recurrence interval (years)									
	1	2	5	10	25	50	100	200	500	1000
5-min	0.502 (0.468-0.543)	0.586 (0.545-0.636)	0.668 (0.619-0.722)	0.761 (0.703-0.823)	0.857 (0.789-0.927)	0.942 (0.863-1.02)	1.02 (0.929-1.10)	1.10 (0.993-1.19)	1.19 (1.07-1.29)	1.28 (1.14-1.39)
10-min	0.803 (0.748-0.868)	0.938 (0.872-1.02)	1.07 (0.992-1.16)	1.22 (1.13-1.32)	1.37 (1.26-1.48)	1.50 (1.38-1.62)	1.62 (1.46-1.75)	1.74 (1.57-1.88)	1.89 (1.69-2.04)	2.02 (1.79-2.19)
15-min	1.00 (0.935-1.08)	1.18 (1.10-1.28)	1.35 (1.25-1.46)	1.54 (1.42-1.67)	1.73 (1.59-1.87)	1.90 (1.74-2.05)	2.05 (1.87-2.22)	2.19 (1.99-2.37)	2.37 (2.13-2.57)	2.53 (2.25-2.75)
30-min	1.38 (1.28-1.49)	1.63 (1.51-1.77)	1.92 (1.78-2.08)	2.23 (2.06-2.41)	2.57 (2.36-2.77)	2.86 (2.62-3.09)	3.14 (2.86-3.39)	3.42 (3.09-3.69)	3.78 (3.38-4.09)	4.10 (3.64-4.45)
60-min	1.72 (1.60-1.85)	2.04 (1.90-2.22)	2.46 (2.29-2.67)	2.90 (2.68-3.14)	3.42 (3.14-3.69)	3.88 (3.55-4.19)	4.32 (3.94-4.67)	4.79 (4.34-5.18)	5.42 (4.88-5.87)	5.98 (5.32-6.50)
2-hr	2.08 (1.92-2.24)	2.49 (2.30-2.69)	3.07 (2.83-3.31)	3.66 (3.37-3.96)	4.37 (4.00-4.72)	5.01 (4.56-5.40)	5.63 (5.10-6.07)	6.27 (5.64-6.76)	7.10 (6.33-7.68)	7.85 (6.93-8.51)
3-hr	2.23 (2.06-2.42)	2.66 (2.45-2.91)	3.30 (3.03-3.59)	3.97 (3.64-4.33)	4.79 (4.37-5.21)	5.55 (5.02-6.03)	6.31 (5.66-6.84)	7.10 (6.33-7.71)	8.19 (7.20-8.91)	9.18 (7.98-10.0)
6-hr	2.66 (2.44-2.91)	3.19 (2.91-3.49)	3.94 (3.60-4.32)	4.75 (4.33-5.20)	5.76 (5.21-6.30)	6.69 (6.01-7.31)	7.61 (6.79-8.32)	8.62 (7.62-9.40)	9.97 (8.71-10.9)	11.2 (9.69-12.3)
12-hr	3.11 (2.84-3.44)	3.72 (3.39-4.11)	4.63 (4.21-5.12)	5.62 (5.10-6.20)	6.85 (6.17-7.54)	8.00 (7.15-8.78)	9.17 (8.12-10.0)	10.4 (9.15-11.4)	12.2 (10.5-13.4)	13.8 (11.8-15.2)
24-hr	3.64 (3.33-3.99)	4.42 (4.06-4.85)	5.71 (5.22-6.28)	6.77 (6.17-7.41)	8.28 (7.50-9.05)	9.53 (8.59-10.4)	10.9 (9.74-11.9)	12.3 (11.0-13.5)	14.4 (12.7-15.8)	16.1 (14.1-17.7)
2-day	4.30 (3.94-4.70)	5.21 (4.78-5.70)	6.66 (6.10-7.27)	7.85 (7.18-8.57)	9.56 (8.68-10.4)	11.0 (9.90-12.0)	12.5 (11.2-13.6)	14.1 (12.6-15.4)	16.4 (14.5-18.1)	18.3 (16.0-20.2)
3-day	4.55 (4.18-4.98)	5.50 (5.06-6.03)	7.00 (6.41-7.65)	8.21 (7.50-8.97)	9.93 (9.03-10.8)	11.3 (10.3-12.4)	12.9 (11.6-14.1)	14.5 (12.9-15.8)	16.7 (14.8-18.4)	18.6 (16.4-20.5)
4-day	4.81 (4.42-5.27)	5.80 (5.33-6.36)	7.33 (6.72-8.02)	8.57 (7.83-9.37)	10.3 (9.37-11.3)	11.7 (10.6-12.8)	13.2 (11.9-14.5)	14.8 (13.3-16.2)	17.1 (15.2-18.8)	18.9 (16.7-20.9)
7-day	5.61 (5.19-6.10)	6.76 (6.25-7.35)	8.44 (2.78-9.16)	9.78 (9.00-10.6)	11.6 (10.7-12.6)	13.1 (12.0-14.2)	14.7 (13.3-16.0)	16.3 (14.8-17.8)	18.6 (16.7-20.3)	20.5 (18.3-22.4)
10-day	6.36 (5.91-6.87)	7.63 (7.08-8.23)	9.36 (8.68-10.1)	10.7 (9.91-11.5)	12.5 (11.6-13.5)	14.0 (12.9-15.1)	15.5 (14.2-16.7)	17.0 (15.5-18.4)	19.1 (17.3-20.7)	20.9 (18.8-22.6)
20-day	8.60 (8.05-9.20)	10.2 (9.59-11.0)	12.4 (11.6-13.2)	14.0 (13.1-15.0)	16.3 (15.2-17.4)	18.0 (16.7-19.3)	19.8 (18.3-21.2)	21.7 (20.0-23.2)	24.2 (22.1-26.0)	26.2 (23.8-28.2)
30-day	10.6 (9.98-11.2)	12.6 (11.8-13.3)	14.9 (14.0-15.8)	16.7 (15.7-17.7)	19.0 (17.8-20.1)	20.8 (19.5-22.1)	22.6 (21.1-24.0)	24.4 (22.7-26.0)	26.9 (24.8-28.6)	28.8 (26.5-30.7)
45-day	13.4 (12.7-14.2)	15.8 (15.0-16.6)	18.5 (17.5-19.6)	20.5 (19.3-21.7)	23.1 (21.8-24.5)	25.1 (23.6-26.5)	27.1 (25.4-28.7)	29.0 (27.1-30.9)	31.6 (29.4-33.7)	33.6 (31.1-35.9)
60-day	16.0 (15.1-16.9)	18.8 (17.8-19.9)	21.8 (20.6-23.0)	24.1 (22.7-25.4)	26.9 (25.3-28.4)	29.1 (27.3-30.7)	31.2 (29.3-33.0)	33.3 (31.1-35.3)	36.0 (33.6-38.3)	38.1 (35.4-40.6)

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

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

Please refer to NOAA Atlas 14 document for more information.

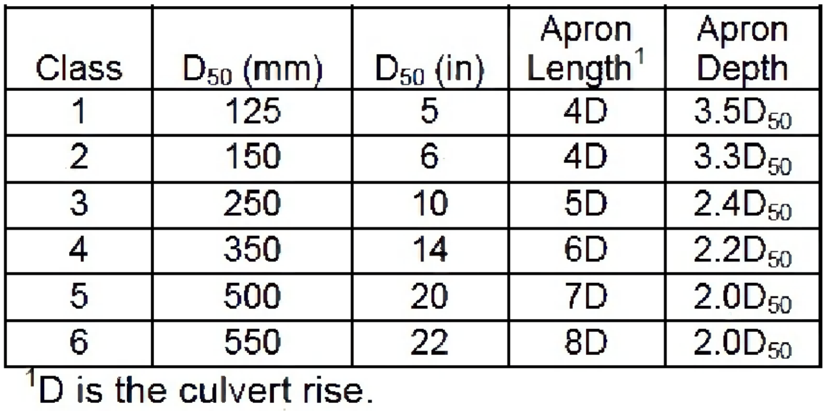
Table 2. Manning's n Values for Open Channels
(from Chow, 1959)



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

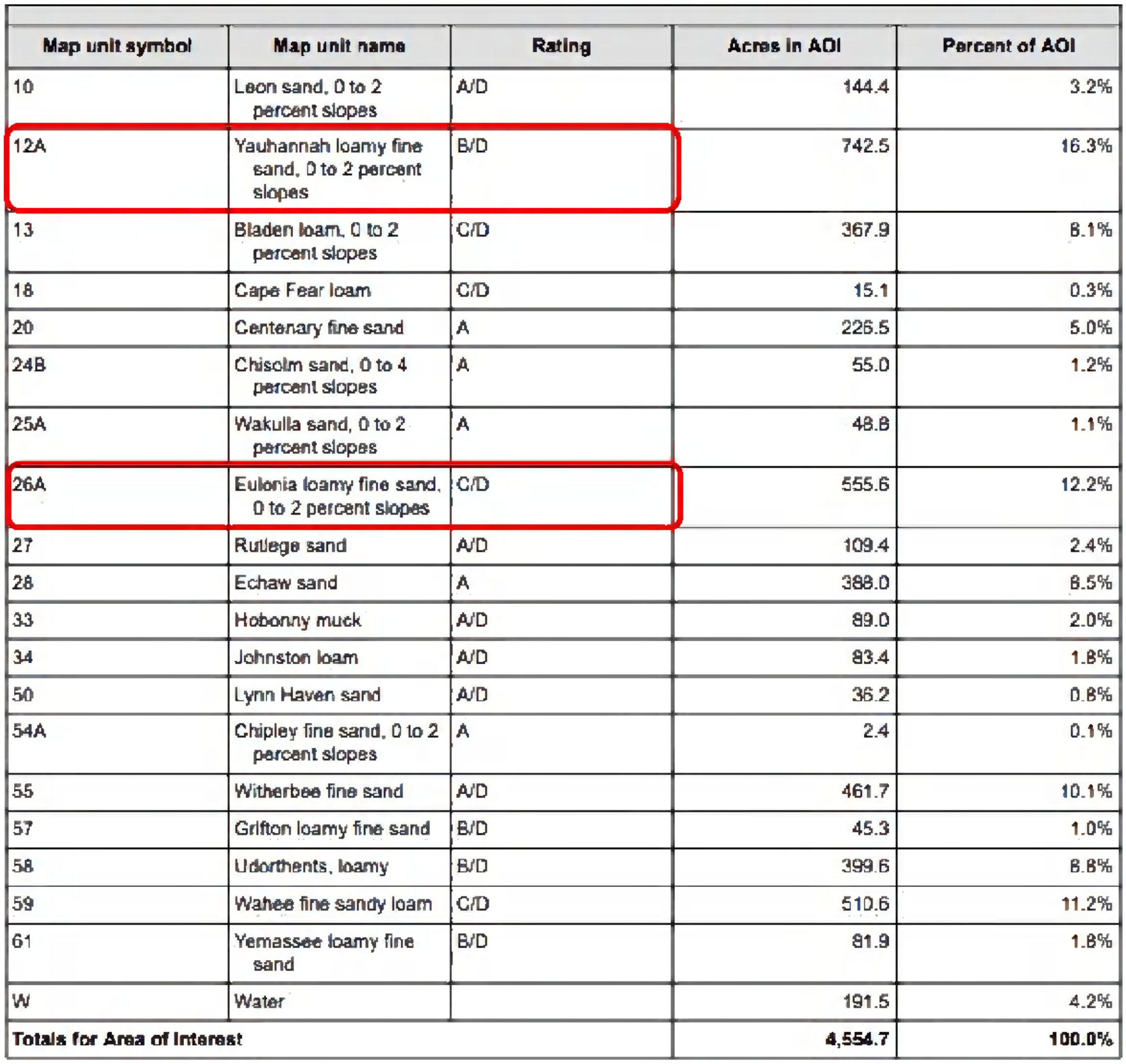
(from FHWA, 2006)



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	$2.0D_{50}$

[1] D is the culvert rise.

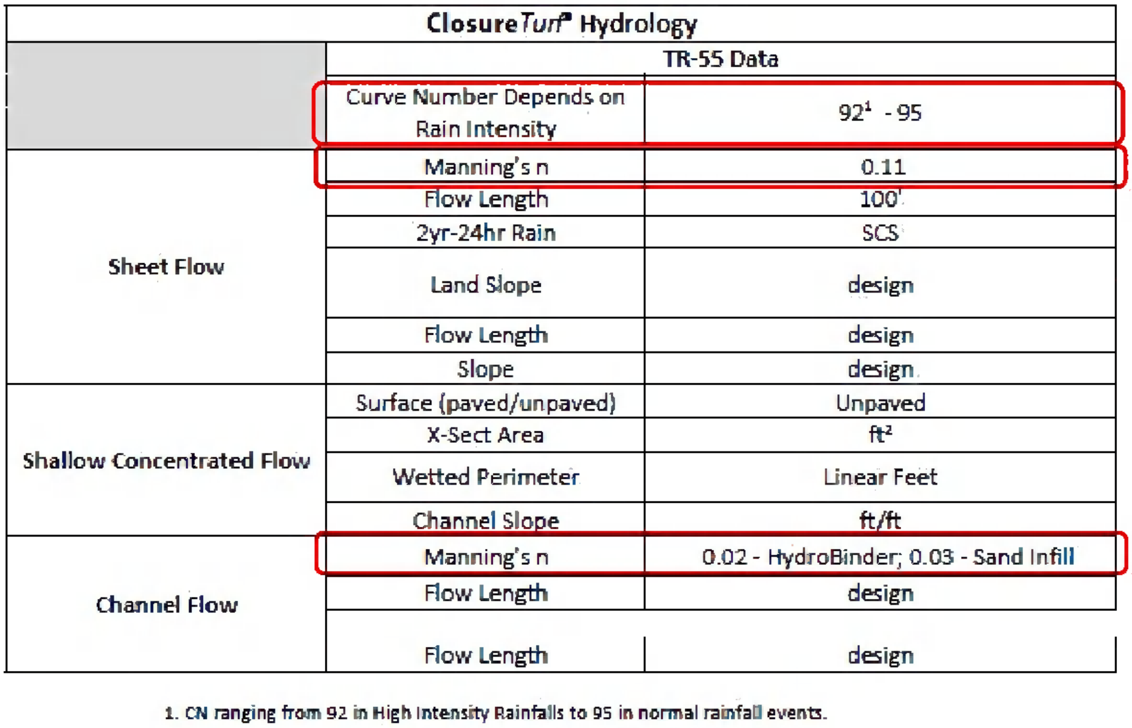
Table 4. Hydrologic Soil Groups for On-Site Soils (from USDA, 2020)



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	1.5	2:1	2:1	21.6	28.6
Top Deck	V-shaped	0.50	0.036	0.0	1.5	20:1	3:1	15.7	20.8
Access Road	V-shaped	8.00	0.036	0.0	1.5	3:1	2:1	16.2	21.4

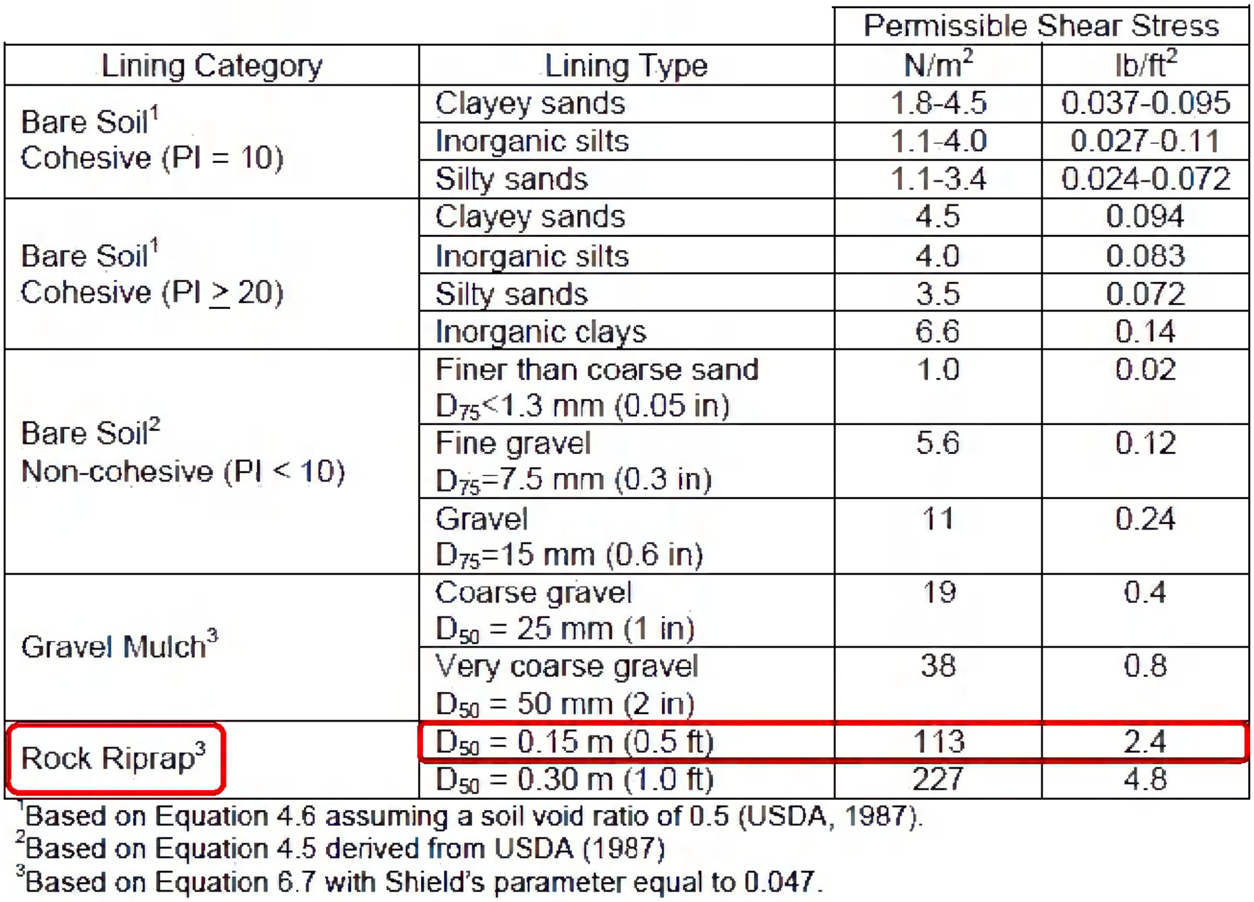
Table 7. Design Parameter Summary for Downdrain Pipe Features

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	19.5	25.7
1B1	2	1.5	4.0	0.009	38.2	50.5
1B2	2	1.5	4.0	0.009	28.5	37.8
1B3	2	1.5	4.0	0.009	30.3	40.1
1C1	2	1.5	4.0	0.009	44.0	58.3
2A1	3	1.5	4.0	0.009	56.1	74.2
2A2	2	1.5	4.0	0.009	24.8	32.8
2C1	5	1.5	4.0	0.009	100.5	133.0
2E1	3	1.5	4.0	0.009	51.0	67.5
2F1	5	1.5	4.0	0.009	95.2	126.0
2F2	6	1.5	4.0	0.009	117.4	155.4

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)



		Permissible Shear Stress	
Lining Category	Lining Type	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 $\geq$ 20)	Clayey sands	4.5	0.094
	Inorganic silts	4.0	0.083
	Silty sands	3.5	0.072
	Inorganic clays	6.6	0.14
Bare Soil[2] Non-cohesive (PI < 10)	Finer than coarse sand D_{75}<1.3 mm (0.05 in)	1.0	0.02
	Fine gravel D_{75}=7.5 mm (0.3 in)	5.6	0.12
	Gravel D_{75}=15 mm (0.6 in)	11	0.24
Gravel Mulch[3]	Coarse gravel D_{50} = 25 mm (1 in)	19	0.4
	Very coarse gravel D_{50} = 50 mm (2 in)	38	0.8
Rock Riprap[3]	D_{50} = 0.15 m (0.5 ft)	113	2.4
	D_{50} = 0.30 m (1.0 ft)	227	4.8

[1] Based on Equation 4.6 assuming a soil void ratio of 0.5 (USDA, 1987).
[2] Based on Equation 4.5 derived from USDA (1987)
[3] Based on Equation 6.7 with Shield's parameter equal to 0.047.

Table 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.

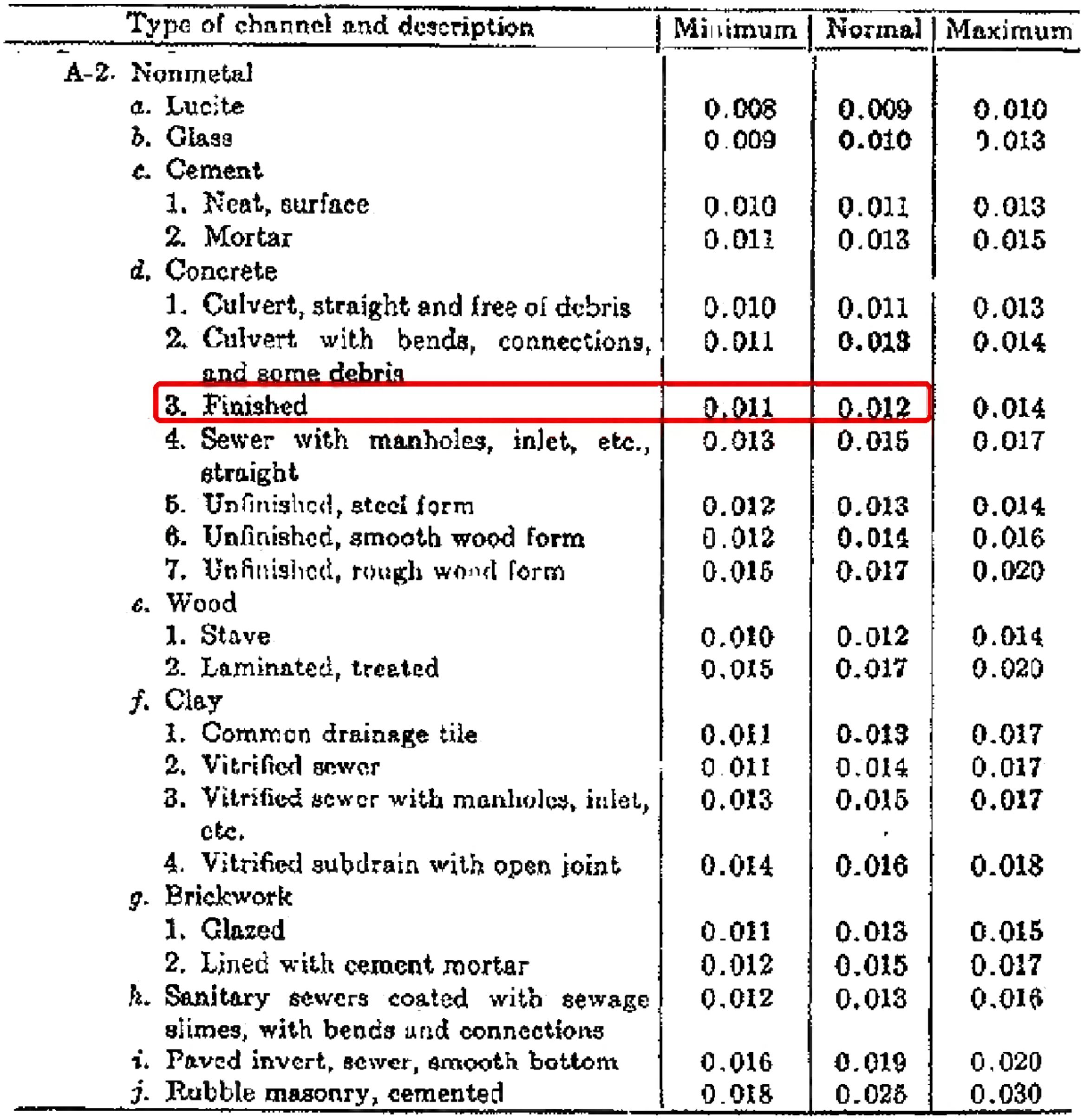
[4] Do not use on slopes steeper than 5 percent except for side-slope in combination channels.

[5] Annuals - used on mild slopes or as temporary protection until permanent covers are established.

Table 10. Design Parameter Summary for Perimeter Drainage Reaches

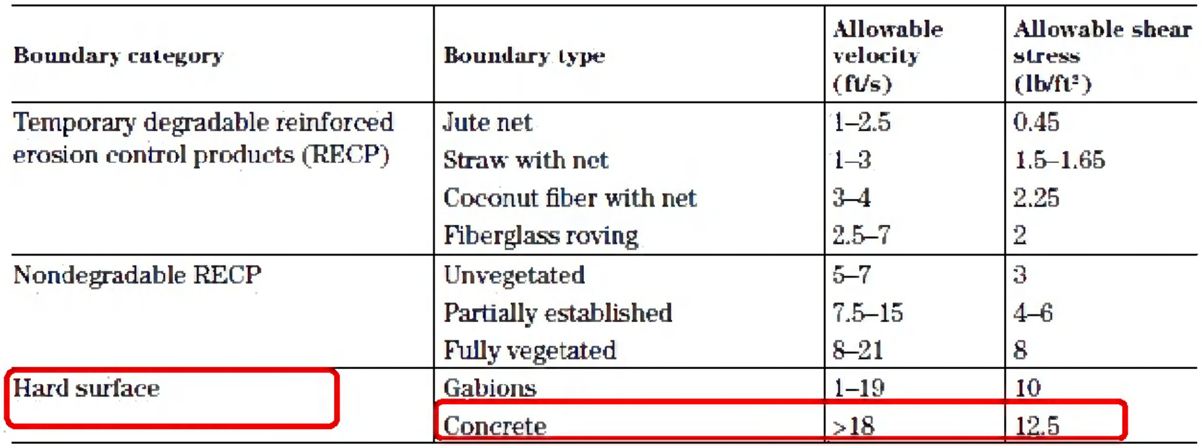
Perimeter Reach	Shape	Longitudinal Slope (ft/ft)	Manning's n	Bottom Width (ft)	Diameter (in)	Depth (ft)	Side Slopes (H:V)	Lining	25-year Flow Rate Q_{25} (cfs)	100-year Flow Rate Q_{100} (cfs)
Perimeter Reach 1A1	Trapezoid	0.005	0.027	3.00	-	3.00	3:1	Vegetation	34.49	45.62
Pipe 1A1	Circular	0.005	0.012	-	36	-	-	-	34.49	45.62
Perimeter Reach 1B3a	Trapezoid	0.005	0.027	3.00	-	3.00	3:1	Vegetation	36.71	48.24
Perimeter Reach 1B3b	Trapezoid	0.014	0.027	3.00	-	3.00	3:1	Vegetation	36.71	48.24
Pipe 1B2a	Circular	0.010	0.012	-	30	-	-	-	36.71	48.24
Pipe 1B2b	Circular	0.002	0.012	-	42	-	-	-	36.71	48.24
Pipe 1B2c	Circular	0.005	0.012	-	48	-	-	-	90.46	120.36
Pipe 1B1a	Circular	0.005	0.012	-	54	-	-	-	126.58	168.37
Pipe 1B1b	Circular	0.005	0.012	-	60	-	-	-	143.04	190.87
Pipe 1BA	Circular	0.020	0.012	-	60	-	-	-	177.21	236.19
Perimeter Reach 1C2	Trapezoid	0.005	0.027	2.00	-	1.70	3:1	Vegetation	23.99	31.78
Perimeter Reach 1C1	Trapezoid	0.005	0.027	2.00	-	1.70	3:1	Vegetation	23.99	31.78
Pipe 1C1a	Circular	0.005	0.012	-	30	-	-	-	23.99	31.78
Pipe 1C1b	Circular	0.008	0.012	-	48	-	-	-	72.21	95.53
Pipe 1C1c	Circular	0.010	0.012	-	48	-	-	-	72.21	95.53
Perimeter Reach 2A1	Trapezoid	0.005	0.027	3.0	-	3.0	3:1	Vegetation	107.8	142.9
Perimeter Reach 2A2	Trapezoid	0.005	0.027	3.0	-	3.0	3:1	Vegetation	28.0	37.0
Perimeter Reach 2BA	Trapezoid	0.005	0.027	3.0	-	3.0	3:1	Vegetation	17.6	23.3
Perimeter Reach 2BB	Trapezoid	0.005	0.027	3.0	-	3.0	3:1	Vegetation	42.2	55.9
Perimeter Reach 2C1	Trapezoid	0.005	0.027	3.0	-	3.0	3:1	Vegetation	120.4	159.6
Perimeter Reach 2DA	Trapezoid	0.005	0.027	3.0	-	3.0	3:1	Vegetation	28.1	37.1

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	0.025	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/ft²)
Temporary degradable reinforced erosion control products (RECP)	Jute net	1–2.5	0.45
	Straw with net	1–3	1.5–1.65
	Coconut fiber with net	3–4	2.25
	Fiberglass roving	2.5–7	2
Nondegradable RECP	Unvegetated	5–7	3
	Partially established	7.5–15	4–6
	Fully vegetated	8–21	8
Hard surface	Gabions	1–19	10
	Concrete	>18	12.5

Table 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	19.3	25.6	104	5.77	1 - 2-ft Diameter RCP	33.00	27.00	37.12
Culvert 2BA	17.6	23.3	112	0.50	1 - 2-ft Diameter RCP	32.39	31.83	38.84
Culvert 2BB	13.6	18.0	107	5.61	1 - 2-ft Diameter RCP	39.00	33.00	42.24
Culvert 2BC	42.2	55.9	109	2.30	2 - 2.5-ft Diameter RCP	23.50	21.00	27.48
Culvert 2C	120.2	159.5	108	0.50	1 - 3.5-ft Diameter RCP	26.95	26.41	40.72
Culvert 2D	28.0	37.1	108	0.50	1 - 2-ft Diameter RCP	26.99	26.45	40.92

Table 14. Design Parameter and Results Summary for Riprap Aprons

Riprap Apron Feature	25-year Flow Rate Q_{25} (cfs)	Number of Barrels	Culvert Diameter (ft)	Median Riprap Size (in)	Riprap Class	Apron Depth (ft)	Apron Length (ft)
Pipe 1BA	177.2	1	5.0	13.8	4	2.5	30.0
Culvert 1B	19.3	1	2.0	6.1	2	1.7	8.0
Pipe 1C1c	72.2	2	4.0	2.8	1	0.8	16.0
Culvert 2BA	17.6	1	2.0	5.4	1	1.6	8.0
Culvert 2BB	13.6	1	2.0	3.8	1	1.1	8.0
Culvert 2BC	42.2	2	2.5	4.1	1	1.2	10.0
Culvert 2C	120.2	1	3.5	18.9	5	3.2	24.5
Culvert 2D	28.0	1	2.0	10.0	3	2.0	10.0
2E1	51.0	3	1.5	10.1	3	2.0	7.5
2F1	95.2	5	1.5	11.7	4	2.1	9.0
2F2	117.4	6	1.5	12.1	4	2.2	9.0

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 Landfill Outfall Locations

Landfill	Outfall	Design Storm	Peak Flow (cfs)	Total Runoff (ac-ft)
Area 1	1A	25-year	177.2	17.0
		100-year	177.2	22.8
	1C	25-year	72.1	6.9
		100-year	72.1	9.2
Area 2	2A	25-year	107.8	9.9
		100-year	107.8	13.2
	2B	25-year	42.2	3.8
		100-year	42.2	5.1
	2C	25-year	120.2	11.2
		100-year	120.2	15.0
	2D	25-year	28.0	2.5
		100-year	28.0	3.3
	2E	25-year	59.0	5.2
		100-year	59.0	6.9
	2F	25-year	226.6	20.6
		100-year	226.6	27.7

Table 16. Summary of Calculated Results for Drainage Terraces

	25-year, 24-hour Design Event				100-year, 24-hour Design Event				
Drainage Feature	**Flow Rate Q_{25} (cfs)**	**Depth of Flow (ft)**	**Average Velocity (ft/s)**	**Average Tractive Stress (psf)**	**Flow Rate Q_{100} (cfs)**	**Depth of Flow (ft)**	**Average Velocity (ft/s)**	**Average Tractive Stress (psf)**	**Channel Lining**
Sideslope	21.6	0.72	4.06	0.72	28.6	0.84	4.44	0.82	Riprap
Top Deck	15.7	0.89	1.70	0.14	20.8	0.99	1.83	0.15	Riprap
Access Road 1B2A	9.3	0.79	5.98	1.82	12.4	0.88	6.41	2.03	Riprap
Access Road 1BA	16.2	0.97	6.86	2.24	21.4	1.08	7.36	2.49	Riprap
Access Road 2A2A	15.7	0.96	6.81	2.22	20.8	1.07	7.31	2.46	Riprap
Access Road 2BB	13.6	0.91	6.57	2.10	18.0	1.01	7.05	2.33	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	19.5	0.9	18.3	25.7	1.06	19.3	1
1B1	38.2	0.9	18.2	50.5	1.04	19.3	2
1B2	28.5	0.7	16.9	37.8	0.86	18.1	2
1B3	30.3	0.7	17.2	40.1	0.89	18.4	2
1C1	44.0	0.9	18.8	58.3	1.18	19.6	2
2A1	56.1	0.9	18.1	74.2	1.03	19.2	3
2A2	24.8	0.7	16.3	32.8	0.78	17.5	2
2C1	100.5	0.9	18.4	133.0	1.09	19.4	5
2E1	51.0	0.8	17.7	67.5	0.96	18.8	3
2F1	95.2	0.9	18.2	126.0	1.04	19.3	5
2F2	117.4	0.9	18.3	155.4	1.06	19.3	6

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	1	N/A	N/A	N/A
1B1	2	2	2	1	N/A	N/A	N/A
1B2	2	2	1	1	N/A	N/A	N/A
1B3	2	2	1	N/A	N/A	N/A	N/A
1C1	2	2	2	1	N/A	N/A	N/A
2A1	3	3	2	1	1	1	N/A
2A2	2	2	1	1	N/A	N/A	N/A
2C1	5	5	4	3	2	1	1
2E1	3	3	2	1	1	1	N/A
2F1	5	5	4	3	2	2	1
2F2	6	6	5	4	3	2	1

Note:

1. "Drainage feature" refers to each downdrain location/designation, as shown on the surface water management system design drawings presented in the set of Engineering Drawings that accompanies the Engineering Report. The SSDTs at each downdrain location are numbered starting with "1" being the lowest drainage terrace, and in ascending order going up the slope. "N/A" (not applicable) means that SSDT number does not exist at that specific downdrain location.

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	34.49	1.38	3.48	0.26	45.62	0.25	1.37	0.06
Pipe 1A1	34.49	1.80	7.78	-	45.62	2.21	8.19	-
Perimeter Reach 1B3a	36.71	1.42	3.53	0.27	48.24	0.26	1.38	0.07
Perimeter Reach 1B3b	36.71	1.12	5.11	0.60	48.24	0.17	1.80	0.13
Pipe 1B2a	36.71	1.73	10.14	-	48.24	2.50	9.11	-
Pipe 1B2b	36.71	2.76	9.80	-	48.24	4.00	8.81	-
Pipe 1B2c	90.46	2.76	9.80	-	120.36	4.00	8.81	-
Pipe 1B1a	126.58	3.15	10.64	-	168.37	4.50	9.53	-
Pipe 1B1b	143.04	3.13	11.08	-	190.87	3.91	11.60	-
Pipe 1BA	177.21	2.33	19.76	-	236.19	2.76	21.22	-
Perimeter Reach 1C2	23.99	1.28	3.20	0.23	31.78	0.30	1.44	0.07
Perimeter Reach 1C1	23.99	1.28	3.20	0.23	31.78	0.30	1.44	0.07
Pipe 1C1a	23.99	1.63	7.07	-	31.78	2.07	7.31	-
Pipe 1C1b	72.21	2.04	11.20	-	95.53	2.43	11.96	-
Pipe 1C1c	72.21	1.91	12.17	-	95.53	2.26	13.04	-
Perimeter Reach 2A1	107.8	2.32	4.66	0.41	142.9	2.62	5.01	0.45
Perimeter Reach 2A2	28.0	1.25	3.29	0.24	37.0	1.43	3.54	0.27
Perimeter Reach 2BA	17.6	1.00	2.91	0.20	23.3	1.15	3.14	0.22
Perimeter Reach 2BB	42.2	1.52	3.67	0.28	55.9	1.73	3.94	0.32
Perimeter Reach 2C1	120.4	2.43	4.80	0.43	159.6	2.75	5.15	0.47
Perimeter Reach 2DA	28.1	1.25	3.29	0.24	37.1	1.43	3.55	0.27

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

								Friction Loss			Manhole Minor Loss		Exit Minor Loss		Total Minor Loss			
Junctions/Inlets/Outlets		From Station	Length	Q_{100}	Pipe Diameter	Velocity, v	n	Friction Slope	Incremental h_f	Cumulative h_f	Kj	Kj x $v^2/2g$	Ko	Ko x $[v_o^2/2g - v_d^2/2g]$	Incremental Minor Loss	Cumulative Minor Loss	Hydraulic Grade Line Elev.	Top of Manhole Elevation
From (ft)	To (ft)	(ft)	(ft)	(cfs)	(in)	(fps)	(-)	(ft/ft)	(ft)	(ft)	(-)	(ft)	(-)	(ft)	(ft)	(ft)	(ft)	(ft)
West Storm Drain Inlet	W8	403.12	113.73	48.24	30	9.83	0.012	0.012	1.35	10.64	0.025	0.037	-	0	0.037	3.8	41.6	38.7
W8	W7	516.85	302.46	48.24	42	5.01	0.012	0.002	0.60	9.29	0.025	0.010	-	0	0.010	3.7	40.9	41.0
W7	W6	819.31	258.79	120.36	48	9.58	0.012	0.006	1.56	8.69	0.025	0.036	-	0	0.036	3.7	39.4	41.0
W6	W5	1078.1	344.01	120.36	48	9.58	0.012	0.006	2.07	7.13	0.025	0.036	-	0	0.036	3.7	37.3	41.0
W5	W4	1422.11	344.07	120.36	48	9.58	0.012	0.006	2.07	5.07	0.025	0.036	-	0	0.036	3.6	35.2	41.0
W4	W3	1766.18	128.92	168.37	54	10.59	0.012	0.006	0.81	3.00	0.025	0.044	-	0	0.044	3.6	34.3	41.0
W3	W2	1895.1	128.92	168.37	54	10.59	0.012	0.006	0.81	2.19	0.025	0.044	-	0	0.044	3.6	33.5	41.0
W2	W1	2024.02	136.58	190.87	60	11.60	0.012	0.005	0.63	1.38	0.025	0.052	-	0	0.052	3.5	32.8	35.0
W1	West Storm Drain Outlet	2160.6	107.58	236.19	60	21.22	0.012	0.007	0.76	0.76	-	-	0.5	3.46	3.464	3.5	28.5	-
													v_d=	2.00	fps			

Notes:

1) Junction boxes designated W1-W8 and their associated top of sturcture elevation represent construction conditons 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) Mahole 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

	25-year, 24-hour storm event				100-year, 24-hour storm event		
Culvert	**Peak Flow Rate (cfs)**	**Pipe Velocity (fps)**	**Headwater Elev (ft)**	**25-year Freeboard (ft)**	**Peak Flow Rate (cfs)**	**Pipe Velocity (fps)**	**Headwater Elev (ft)**
Culvert 1B	19.3	15.26	35.78	1.34	25.6	16.31	36.90
Culvert 2BA	17.6	6.92	34.98	3.86	23.3	8.13	35.97
Culvert 2BB	13.6	15.13	41.04	1.20	18.0	14.9	41.59
Culvert 2BC	42.2	11.84	25.88	1.60	55.9	11.09	26.45
Culvert 2C	120.2	12.9	35.47	5.25	159.5	16.58	40.40
Culvert 2D	28.0	9.3	31.63	9.29	37.1	11.91	34.14

FIGURES

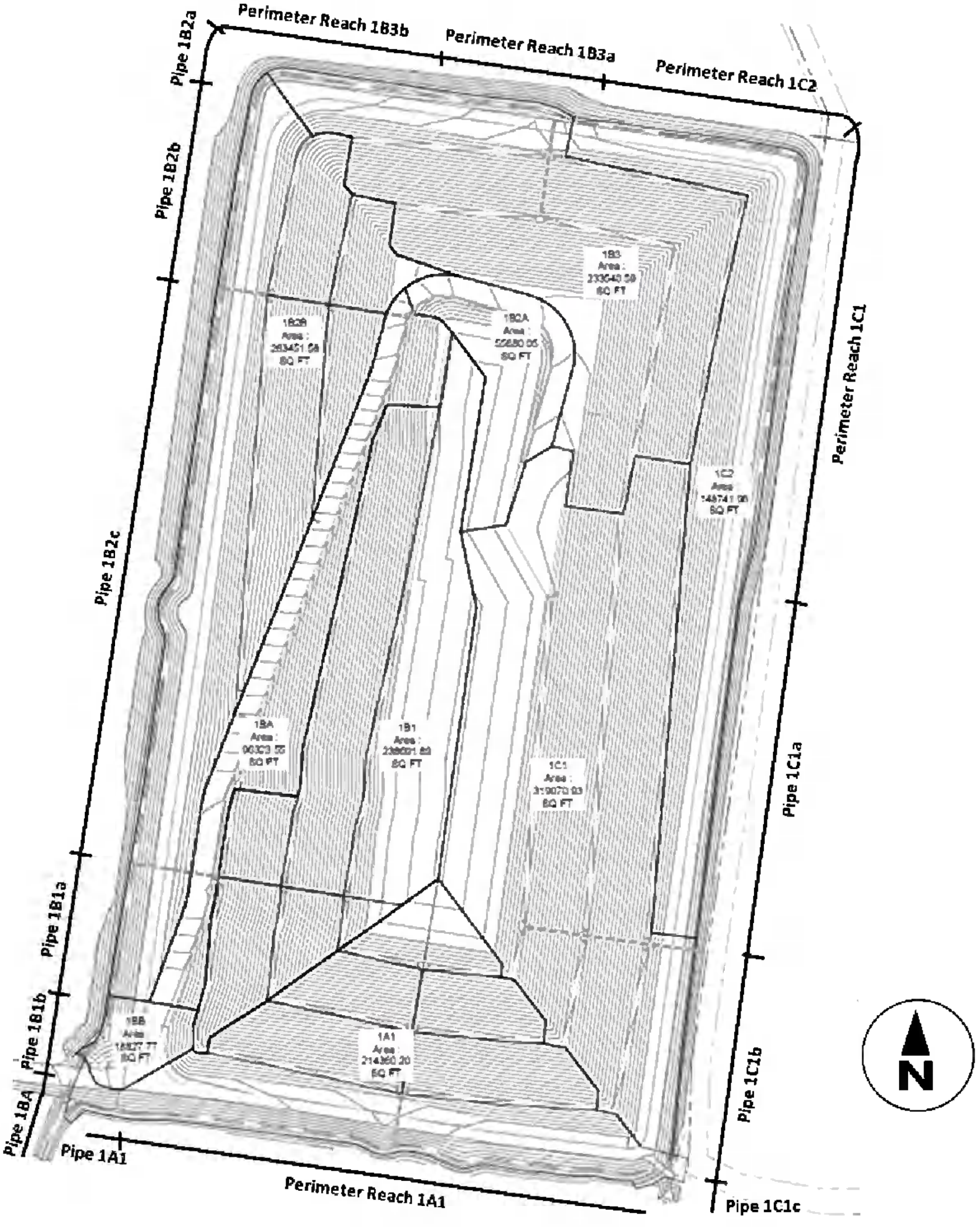


Figure 1. Landfill Area 1 Perimeter Reach Designations

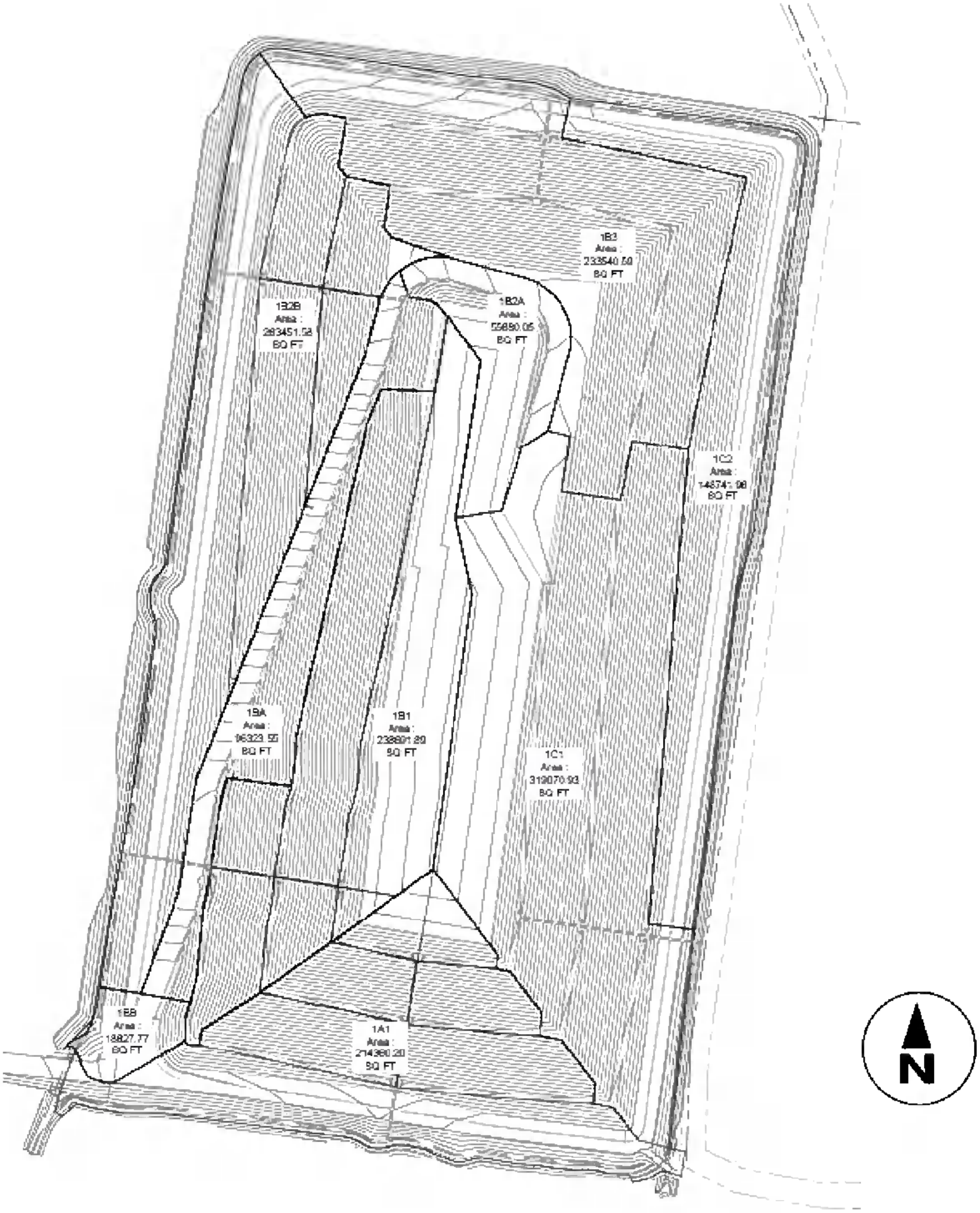


Figure 2. Landfill Area 1 Drainage Plan

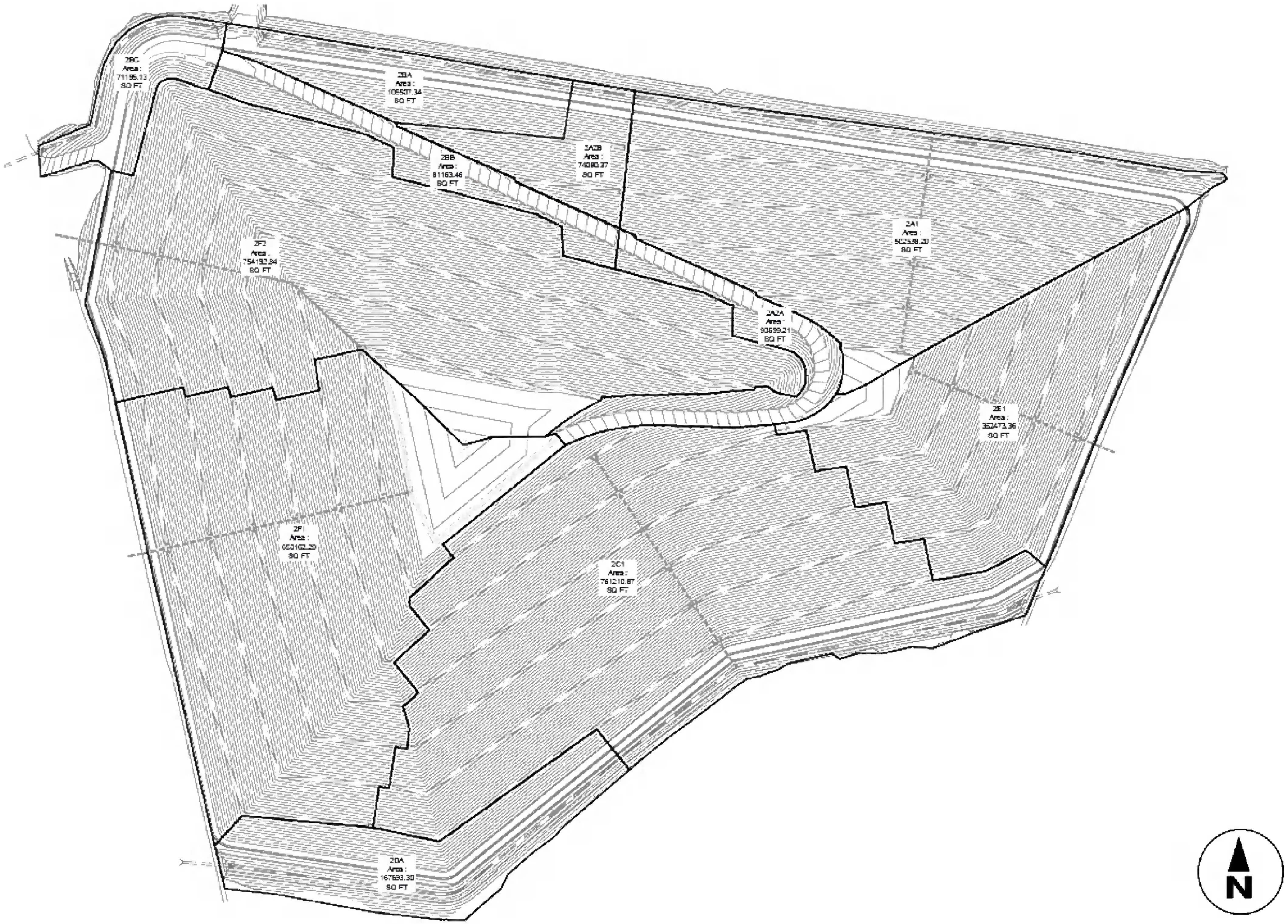


Figure 3. Landfill Area 2 Drainage Plan

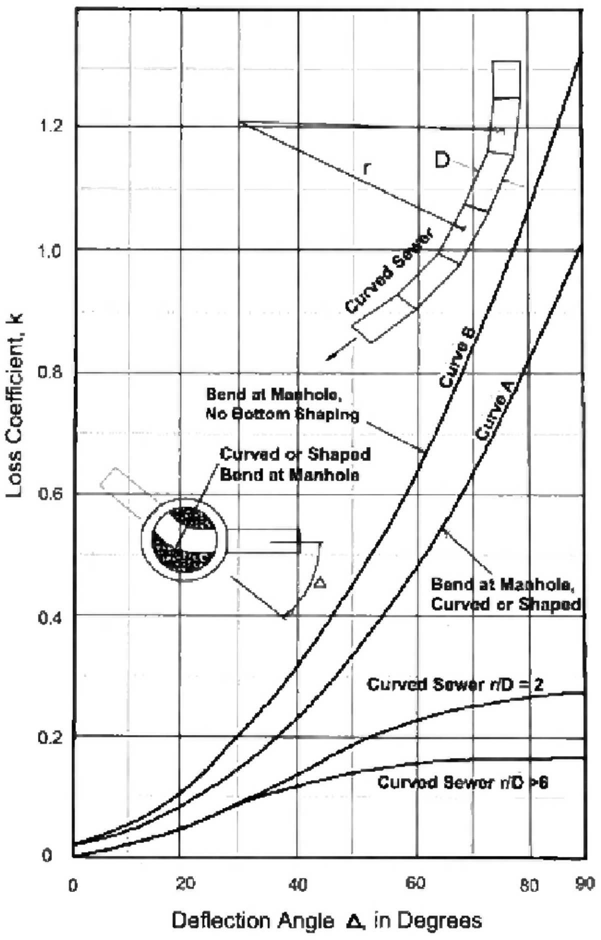
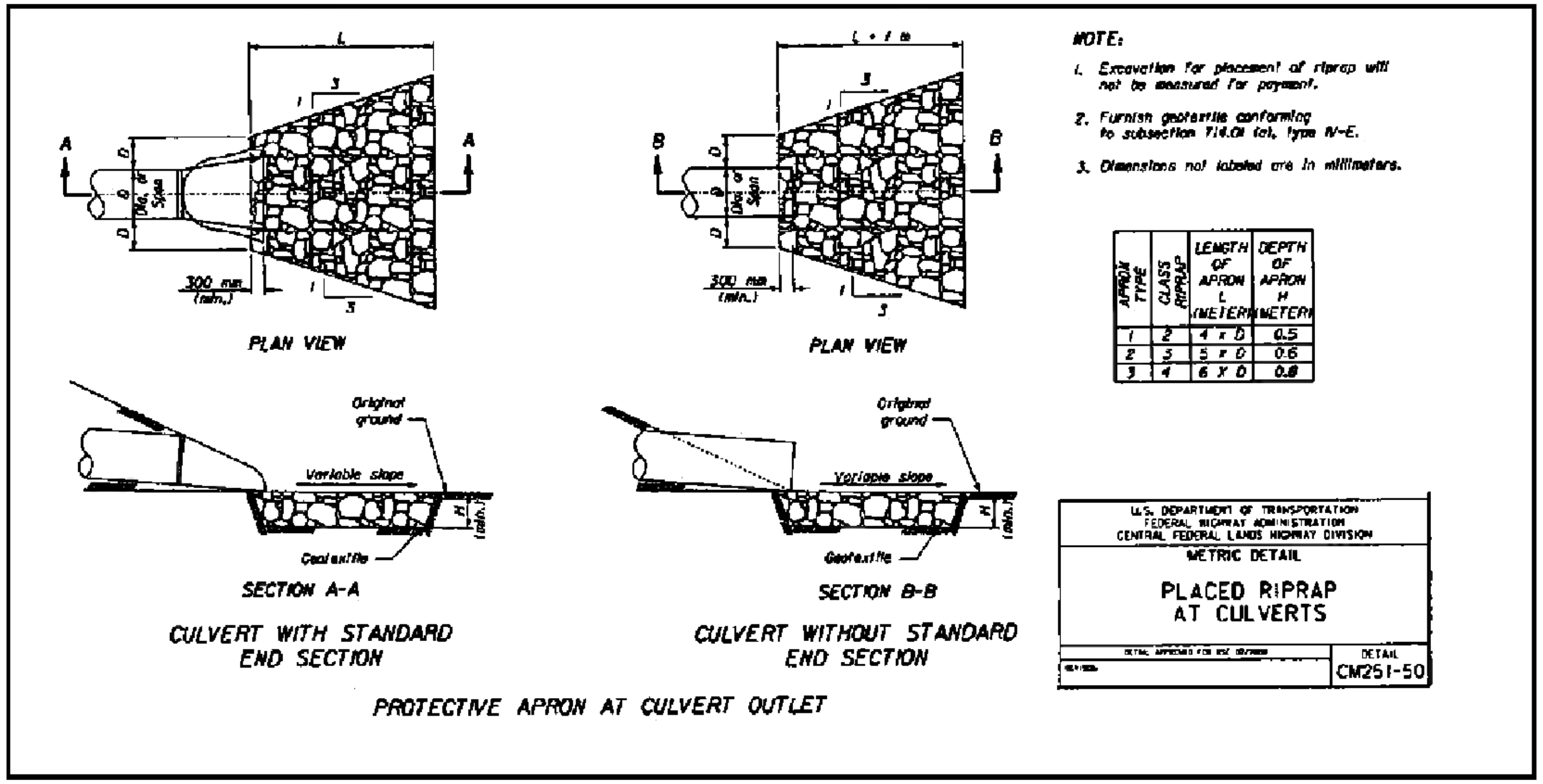


Figure 4. Bend Loss Coefficient (from UDFCD, 2001)



NOTE:

1. Excavation for placement of riprap will not be measured for payment.
2. Furnish geotextile conforming to subsection 714.01 (a), type IV-E.
3. Dimensions not labeled are in millimeters.

APRON TYPE	CLASS RIPRAP	LENGTH OF APRON L (METERS)	DEPTH OF APRON H (METERS)
1	2	4 x D	0.5
2	3	5 x D	0.6
3	4	6 X D	0.8

Figure 5. Typical Geometry of Riprap Aprons at Culverts

(from FHWA, 2006)

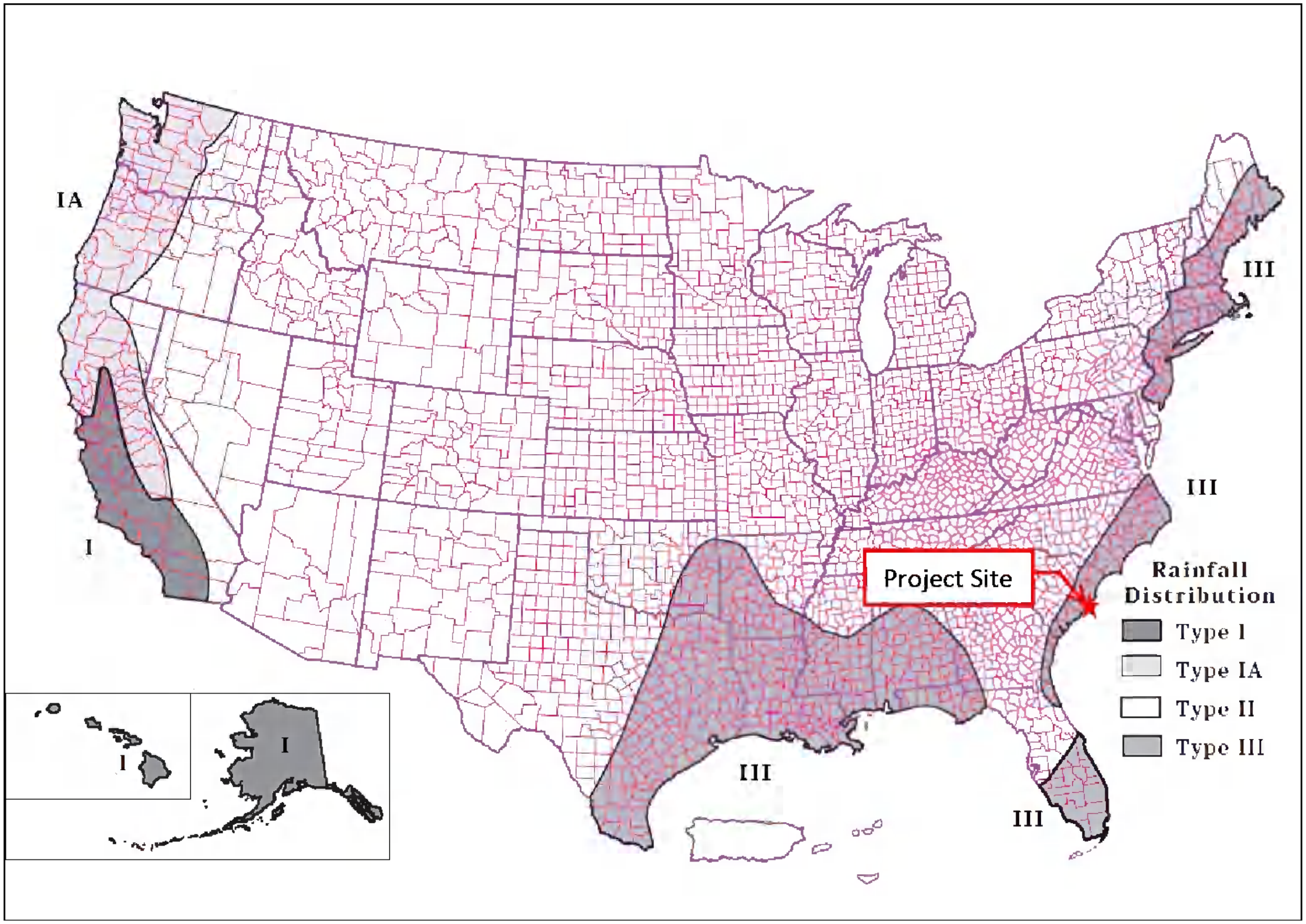


Figure 6. NRCS Rainfall Distributions (from USDA, 1986)

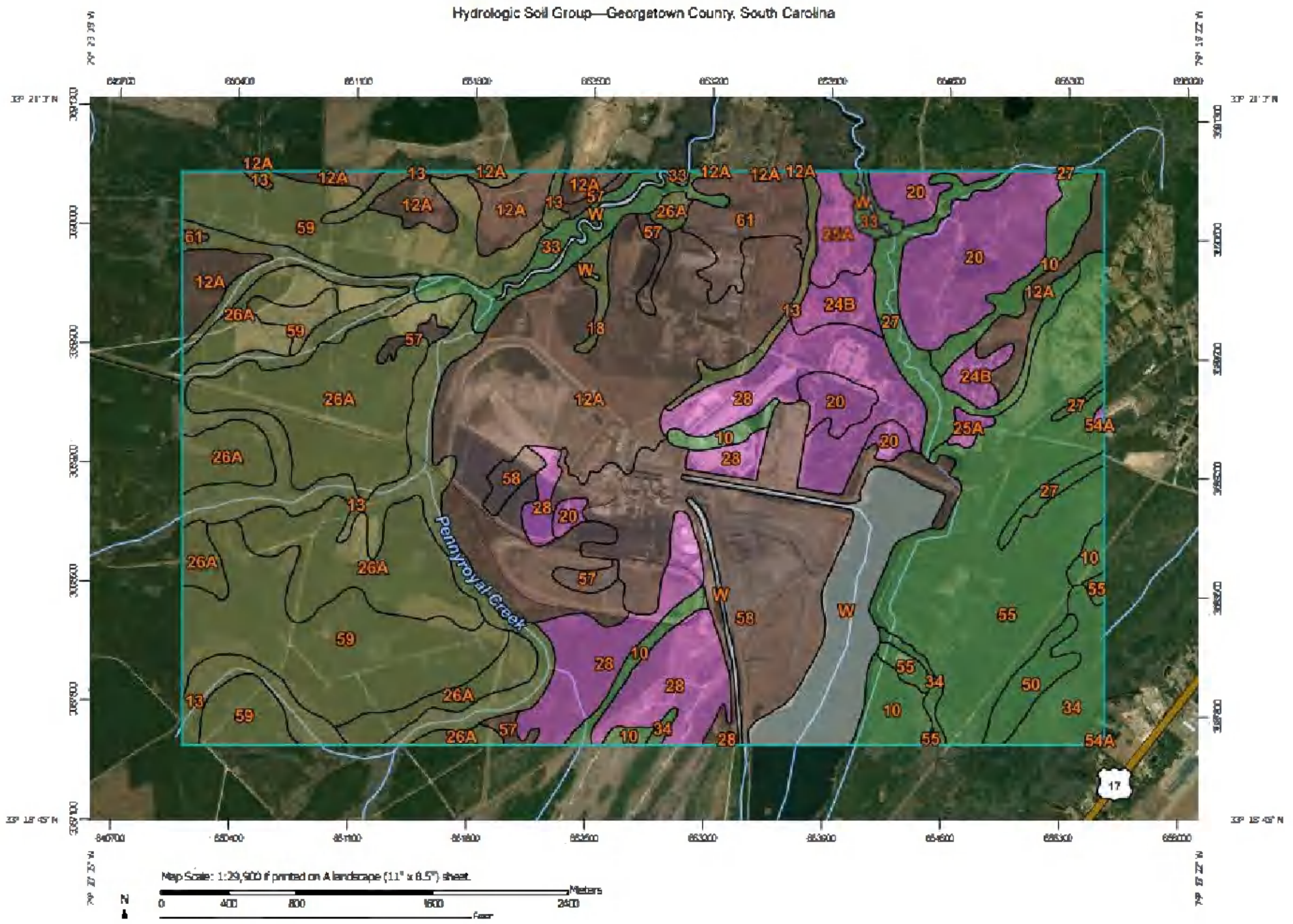
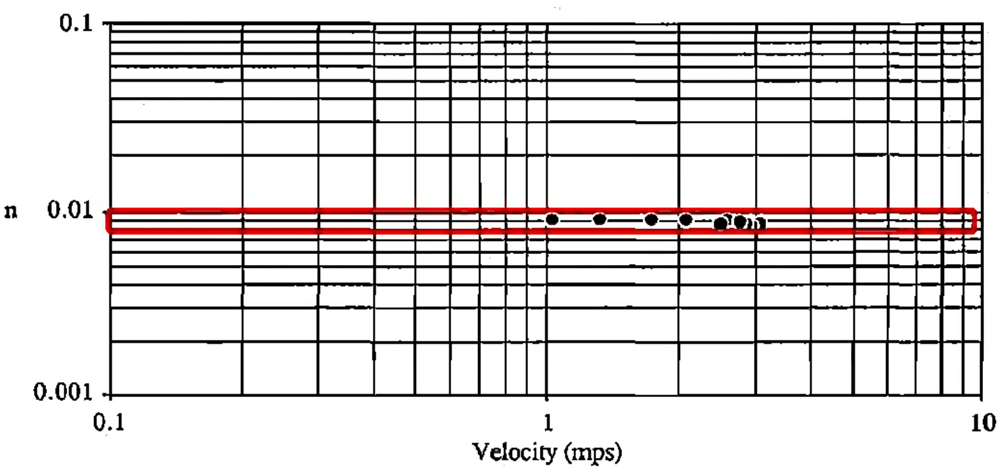


Figure 7. Soil Survey Map (from USDA, 2020)

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 8. Manning's n for HDPE

(from Barfuss and Tullis, 1989)

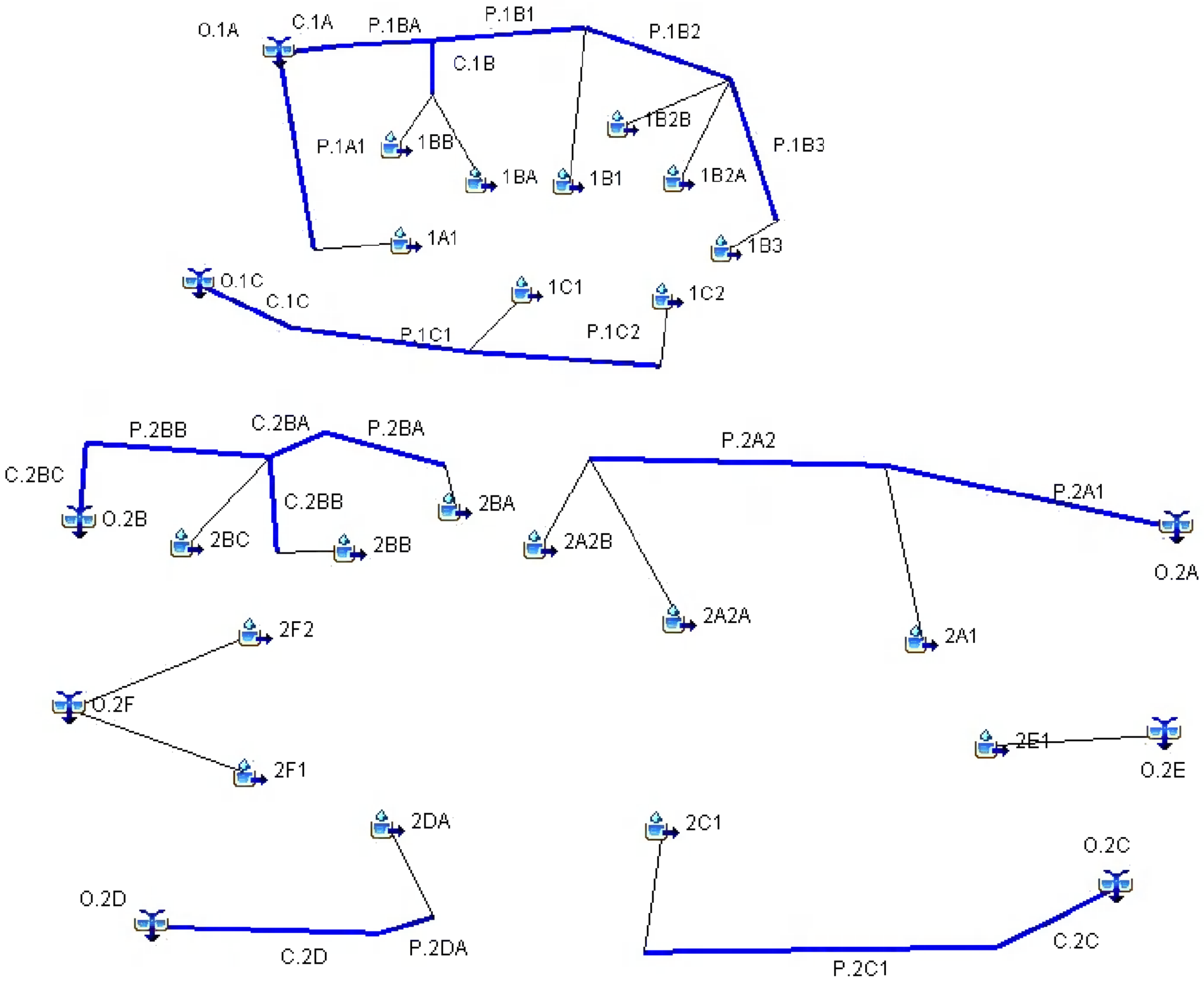


Figure 9. HEC-HMS Nodal Network

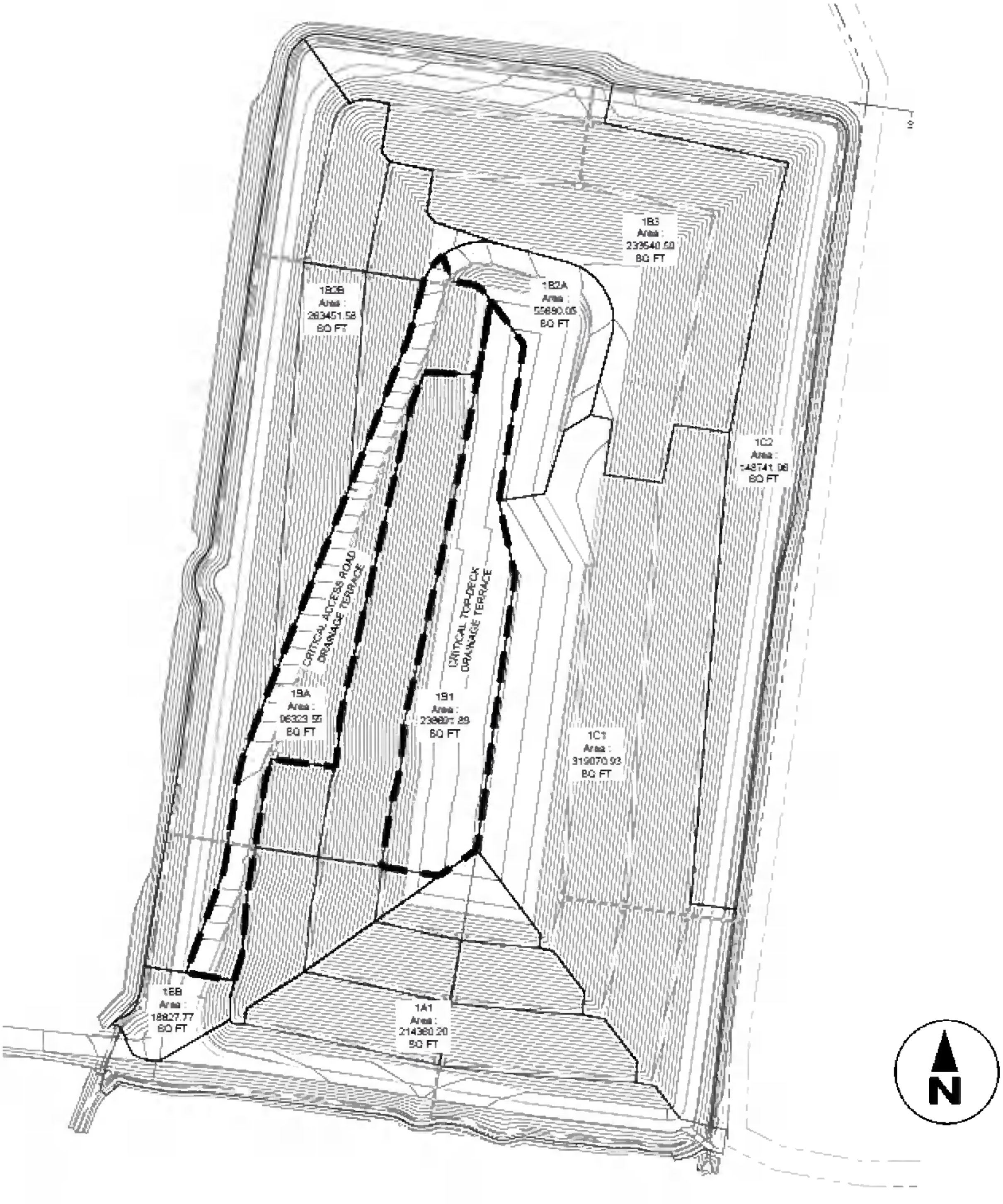


Figure 10. Landfill Area 1 Drainage Plan with Critical Terrace Areas

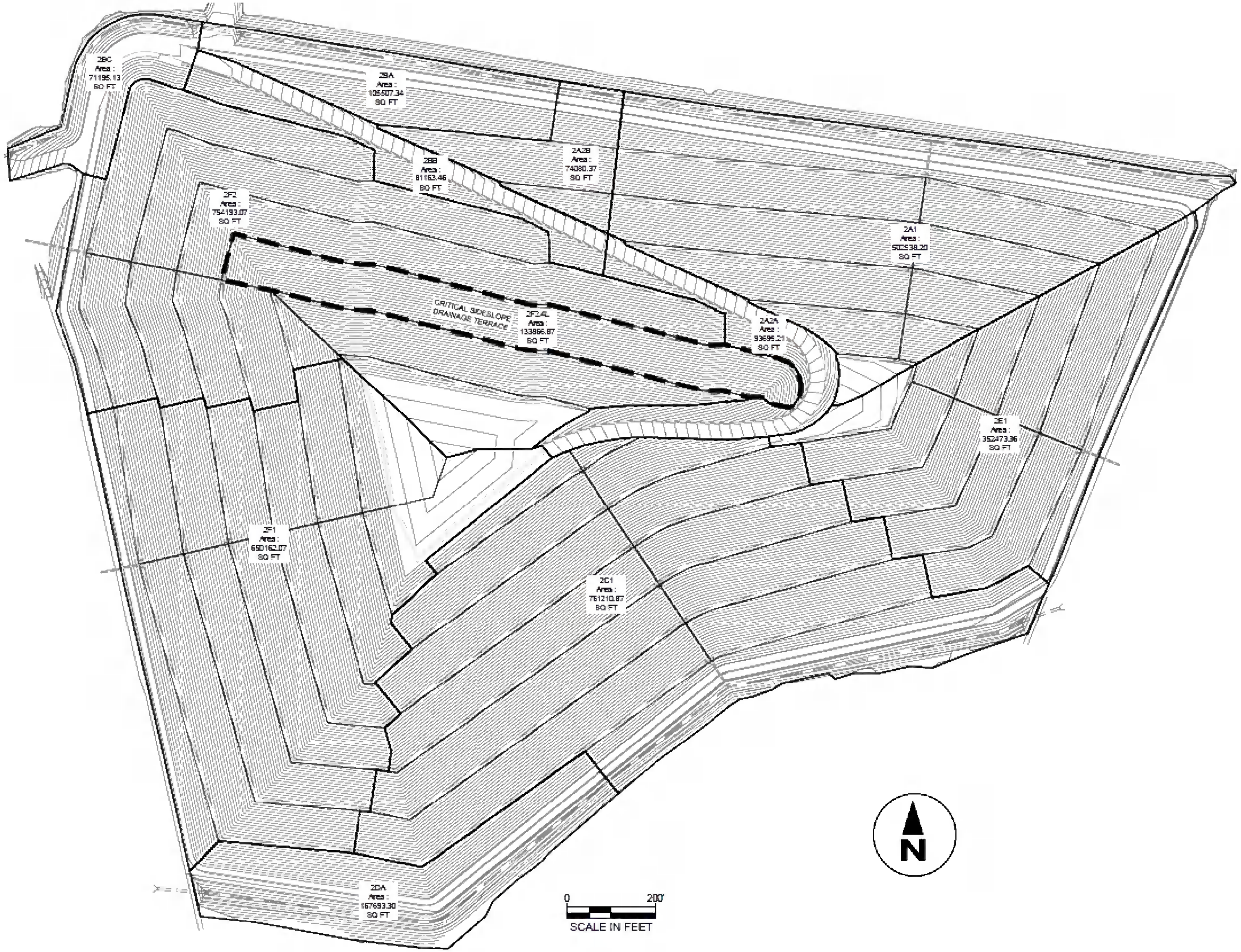


Figure 11. Landfill Area 2 Drainage Plan with Critical Terrace Areas

APPENDIX 1

HEC-HMS HYDROLOGIC MODEL PARAMETERS

Table 1-1. HEC-HMS Basin Input Parameters for Kinematic Wave Model

POST-DEVELOPMENT CONDITIONS			Watershed Characterization			Sheet Flow				Open Channel Flow - Top Deck or Side Slope Drainage Terrace								
Subcatchment Designation	Area A (mi^2)	Area A (acres)	Initial Abstraction (in)	Curve Number	Impervious Cover (%)	Flow Length (ft)	Manning's n	Slope (ft/ft)	Time T_t (min)	Flow Length (ft)	Depth d (ft)	Area A (ft^2)	Wetted P (ft)	Hydraulic Radius (ft)	Manning's n	Slope (ft/ft)	Velocity (ft/s)	Time T_t (min)
1A1	0.00769	4.92	0.11	95	0.00	108	0.11	0.05	5.91	0	1.0	4.00	8.26	0.48	0.036	0.020	3.61	0.00
1BA	0.00346	2.21	0.11	95	0.00	92	0.11	0.33	1.97	1065	1.0	2.50	5.40	0.46	0.036	0.080	7.01	2.53
1BB	0.00068	0.43	0.11	95	0.00	42	0.11	0.33	1.05	0	1.0	4.00	8.26	0.48	0.036	0.020	3.61	0.00
1B1	0.00856	5.48	0.11	95	0.00	60	0.11	0.05	3.00	370	1.0	11.50	23.19	0.50	0.036	0.005	1.83	3.36
1B2A	0.00200	1.28	0.11	95	0.00	85	0.11	0.05	4.66	149	1.0	2.50	5.40	0.46	0.036	0.080	7.01	0.35
1B2B	0.01070	6.85	0.11	95	0.00	85	0.11	0.05	5.73	68	1.0	4.00	8.26	0.48	0.036	0.020	3.61	0.31
1B3	0.00838	5.36	0.11	95	0.00	97	0.11	0.03	7.51	85	1.0	4.00	8.26	0.48	0.036	0.020	3.61	0.39
1C1	0.01144	7.32	0.11	95	0.00	172	0.11	0.05	6.96	527	1.0	4.00	8.26	0.48	0.036	0.020	3.61	3.13
1C2	0.00533	3.41	0.11	95	0.00	75	0.11	0.33	1.68	0	1.0	4.00	8.26	0.48	0.036	0.005	1.80	0.00
2A1	0.01803	11.54	0.11	95	0.00	126	0.11	0.05	5.42	80	1.0	4.00	8.26	0.48	0.036	0.020	3.61	0.37
2A2A	0.00336	2.15	0.11	95	0.00	34	0.11	0.02	2.74	1233	1.0	2.50	5.40	0.46	0.036	0.080	7.01	2.93
2A2B	0.00266	1.70	0.11	95	0.00	89	0.11	0.33	1.92	230	1.0	4.00	8.26	0.48	0.036	0.020	3.61	1.06
2BA	0.00378	2.42	0.11	95	0.00	163	0.11	0.33	3.12	0	1.0	4.00	8.26	0.48	0.036	0.020	3.61	0.00
2BB	0.00291	1.86	0.11	95	0.00	34	0.11	0.02	2.74	997	1.0	2.50	5.40	0.46	0.036	0.080	7.01	2.37
2BC	0.00255	1.63	0.11	95	0.00	92	0.11	0.33	5.71	0	1.0	4.00	8.26	0.48	0.036	0.020	3.61	0.00
2C1	0.02730	17.47	0.11	95	0.00	93	0.11	0.33	1.99	937	1.0	4.00	8.26	0.48	0.036	0.020	3.61	4.33
2DA	0.00602	3.85	0.11	95	0.00	84	0.11	0.33	1.84	0	1.0	4.00	8.26	0.48	0.036	0.020	3.61	0.00
2E1	0.01264	8.09	0.11	95	0.00	118	0.11	0.05	5.15	73	1.0	4.00	8.26	0.48	0.036	0.020	3.61	0.34
2F1	0.02332	14.93	0.11	95	0.00	121	0.11	0.05	5.25	305	1.0	4.00	8.26	0.48	0.036	0.005	1.80	2.82
2F2	0.02705	17.31	0.11	95	0.00	161	0.11	0.05	6.60	264	1.0	4.00	8.26	0.48	0.036	0.020	3.61	1.22

2-year, 24-hour Design Rainfall Depth = 4.42 inches

ainage Terrace Left Side Slope = 20.0 H:V

Top Deck Drainage Terrace Right Side Slope = 3.0 H:V

Side Slope Drainage Terrace Left Side Slope = 5.0 H:V

Side Slope Drainage Terrace Right Side Slope = 3.0 H:V

Access Road Drainage Terrace Left Side Slope = 2.0 H:V

Access Road Drainage Terrace Right Side Slope = 3.0 H:V

Notes:

1) *Curve number = 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 = 0.009 represents HDPE (Barfuss, 1989).*
5) *Travel Time (T_t) is calculated using Manning's kinematic solutions for sheet flow (USDA, 1986).*

$$T_t = 0.007(nL)^{0.8} / (P_{2\text{-}24})^{0.5} S^{0.4}$$

6) *Open channel flow velocity is calculated using Manning's equation (USDA, 1986).*

$$V = (1.49r^{2/3} S^{1/2}) / n \quad \text{where: } r = \text{hydraulic radius (ft) and is equal to A/P [area (ft}^2\text{)/wetted perimeter (ft)]}$$

7) *Design rainfall depth taken from NOAA Atlas 14, Volume 2, Version 3 for Georgetown, South Carolina.*

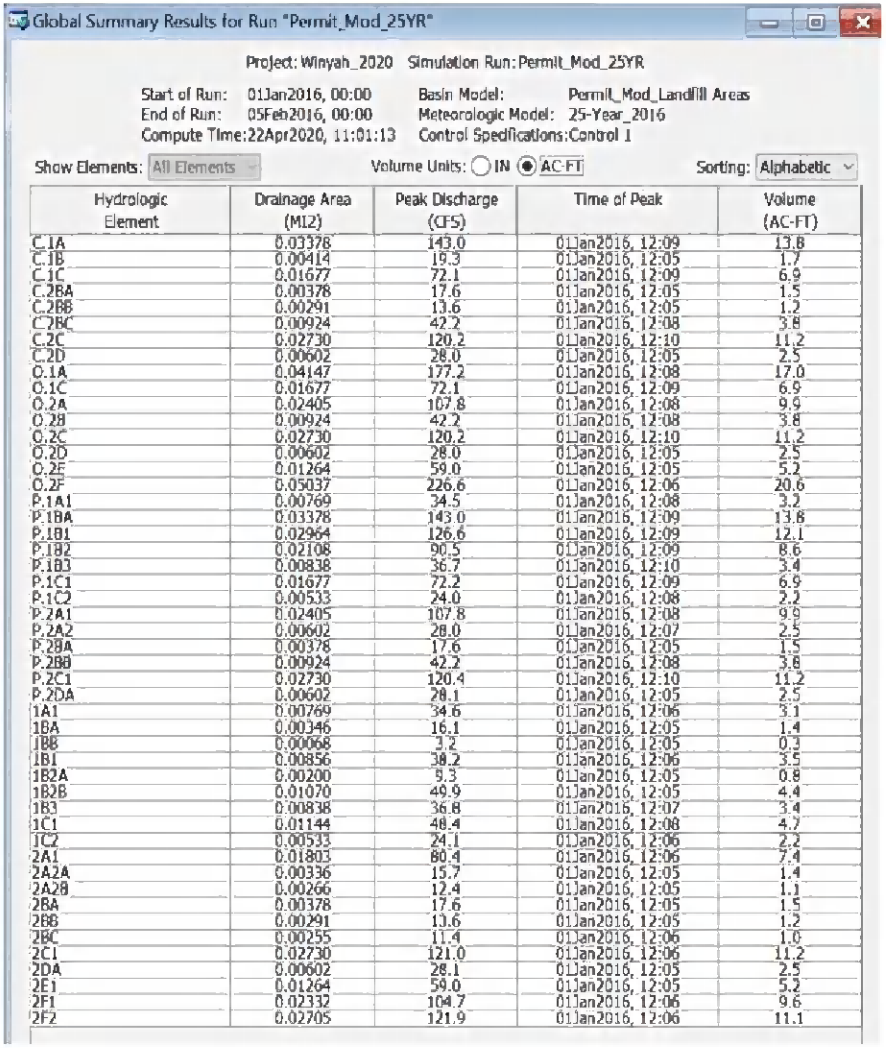
Table 1-1 (Continued). HEC-HMS Basin Input Parameters for Kinematic Wave Model

Down Drain Pipe Flow								Open Channel Flow - Perimeter Channel												
Flow Length (ft)	Area A (ft^2)	Wetted P (ft)	Hydraulic Radius (ft)	Manning's n	Slope (ft/ft)	Velocity (ft/s)	Time T_t (min)	Flow Length (ft)	Depth d (ft)	Area A (ft^2)	Wetted P (ft)	Hydraulic Radius (ft)	Manning's n	Slope (ft/ft)	Velocity (ft/s)	Time T_t (min)	Design T_c (min)	SCS Lag Time (min)	HMS 25-yr Flow (cfs)	HMS 100-yr Flow (cfs)
291	1.77	4.71	0.38	0.009	0.333	49.71	0.10	562	2.0	18.00	15.65	1.15	0.027	0.0050	4.28	2.19	8.19	4.92	34.58	45.77
96	1.77	4.71	0.38	0.009	0.050	19.25	0.08	0	2.0	18.00	15.65	1.15	0.027	0.0050	4.28	0.00	6.00	3.60	16.15	21.37
0	1.77	4.71	0.38	0.009	0.333	49.71	0.00	113	2.0	18.00	15.65	1.15	0.027	0.0050	4.28	0.44	6.00	3.60	3.17	4.20
349	1.77	4.71	0.38	0.009	0.333	49.71	0.12	484	2.0	16.00	16.52	0.97	0.027	0.0050	3.82	2.11	8.59	5.15	38.20	50.56
0	1.77	4.71	0.38	0.009	0.333	49.71	0.00	0	2.0	18.00	15.65	1.15	0.027	0.0050	4.28	0.00	6.00	3.60	9.33	12.35
181	1.77	4.71	0.38	0.009	0.333	49.71	0.06	0	2.0	18.00	15.65	1.15	0.027	0.0050	4.28	0.00	6.10	3.66	49.87	66.00
168	1.77	4.71	0.38	0.009	0.333	49.71	0.06	406	2.0	18.00	15.65	1.15	0.027	0.0050	4.28	1.58	9.54	5.72	36.77	48.67
300	1.77	4.71	0.38	0.009	0.333	49.71	0.10	329	2.0	18.00	15.65	1.15	0.027	0.0050	4.28	1.28	11.47	6.88	48.39	64.05
0	1.77	4.71	0.38	0.009	0.333	49.71	0.00	1600	2.0	18.00	15.65	1.15	0.027	0.0050	4.28	6.22	7.90	4.74	24.10	31.89
498	1.77	4.71	0.38	0.009	0.333	49.71	0.17	680	2.0	18.00	15.65	1.15	0.027	0.0050	4.28	2.65	8.61	5.16	80.42	106.46
0	1.77	4.71	0.38	0.009	0.333	49.71	0.00	0	2.0	18.00	15.65	1.15	0.027	0.0050	4.28	0.00	6.00	3.60	15.68	20.75
156	1.77	4.71	0.38	0.009	0.333	49.71	0.05	0	2.0	18.00	15.65	1.15	0.027	0.0050	4.28	0.00	6.00	3.60	12.41	16.43
0	1.77	4.71	0.38	0.009	0.333	49.71	0.00	423	2.0	18.00	15.65	1.15	0.027	0.0050	4.28	1.65	6.00	3.60	17.64	23.34
95	1.77	4.71	0.38	0.009	0.050	19.25	0.08	0	2.0	18.00	15.65	1.15	0.027	0.0050	4.28	0.00	6.00	3.60	13.58	17.97
0	1.77	4.71	0.38	0.009	0.333	49.71	0.00	729	2.0	18.00	15.65	1.15	0.027	0.0050	4.28	2.84	8.55	5.13	11.39	15.07
85	1.77	4.71	0.38	0.009	0.333	49.71	0.03	646	2.0	18.00	15.65	1.15	0.027	0.0050	4.28	2.51	8.86	5.32	121.04	160.22
0	1.77	4.71	0.38	0.009	0.333	49.71	0.00	981	2.0	18.00	15.65	1.15	0.027	0.0050	4.28	3.82	6.00	3.60	28.09	37.18
504	1.77	4.71	0.38	0.009	0.333	49.71	0.17	0	2.0	18.00	15.65	1.15	0.027	0.0050	4.28	0.00	6.00	3.60	58.98	78.06
668	1.77	4.71	0.38	0.009	0.333	49.71	0.22	0	2.0	18.00	15.65	1.15	0.027	0.0050	4.28	0.00	8.29	4.98	104.67	138.54
745	1.77	4.71	0.38	0.009	0.333	49.71	0.25	0	2.0	18.00	15.65	1.15	0.027	0.0050	4.28	0.00	8.07	4.84	121.94	161.40

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

Table 1-2. 25-year, 24-hour Precipitation Event Nodal Areas, Peak Flow Rates, and Runoff Volumes for Landfill Area 1 and Landfill Area 2



Project: Winyah_2020 Simulation Run: Permit_Mod_25YR

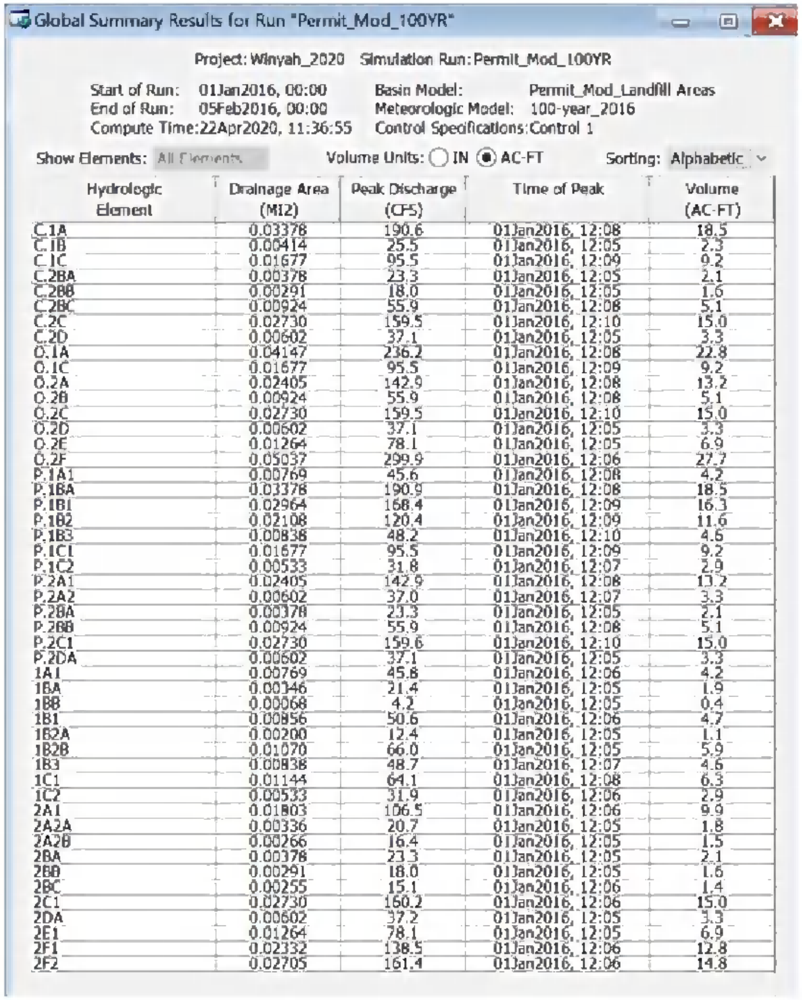
Start of Run: 01Jan2016, 00:00
End of Run: 05Feb2016, 00:00
Compute Time: 22Apr2020, 11:01:13

Basin Model: Permit_Mod_Landfill Areas
Meteorologic Model: 25-Year_2016
Control Specifications: Control 1

Show Elements: All Elements
Volume Units: ○ IN ● AC-FT
Sorting: Alphabetic

Hydrologic Element	Drainage Area (MI2)	Peak Discharge (CFS)	Time of Peak	Volume (AC-FT)
C.1A	0.03378	143.0	01Jan2016, 12:09	13.8
C.1B	0.00414	19.3	01Jan2016, 12:05	1.7
C.1C	0.01677	72.1	01Jan2016, 12:09	6.9
C.2BA	0.00378	17.6	01Jan2016, 12:05	1.5
C.2BB	0.00291	13.6	01Jan2016, 12:05	1.2
C.2BC	0.00924	42.2	01Jan2016, 12:08	3.8
C.2C	0.02730	120.2	01Jan2016, 12:10	11.2
C.2D	0.00602	28.0	01Jan2016, 12:05	2.5
O.1A	0.04147	177.2	01Jan2016, 12:08	17.0
O.1C	0.01677	72.1	01Jan2016, 12:09	6.9
O.2A	0.02405	107.8	01Jan2016, 12:08	9.9
O.2B	0.00924	42.2	01Jan2016, 12:08	3.8
O.2C	0.02730	120.2	01Jan2016, 12:10	11.2
O.2D	0.00602	28.0	01Jan2016, 12:05	2.5
O.2E	0.01264	59.0	01Jan2016, 12:05	5.2
O.2F	0.05037	226.6	01Jan2016, 12:06	20.6
P.1A1	0.00769	34.5	01Jan2016, 12:08	3.2
P.1BA	0.03378	143.0	01Jan2016, 12:09	13.8
P.1B1	0.02964	126.6	01Jan2016, 12:09	12.1
P.1B2	0.02108	90.5	01Jan2016, 12:09	8.6
P.1B3	0.00838	36.7	01Jan2016, 12:10	3.4
P.1C1	0.01677	72.2	01Jan2016, 12:09	6.9
P.1C2	0.00533	24.0	01Jan2016, 12:08	2.2
P.2A1	0.02405	107.8	01Jan2016, 12:08	9.9
P.2A2	0.00602	28.0	01Jan2016, 12:07	2.5
P.2BA	0.00378	17.6	01Jan2016, 12:05	1.5
P.2BB	0.00924	42.2	01Jan2016, 12:08	3.8
P.2C1	0.02730	120.4	01Jan2016, 12:10	11.2
P.2DA	0.00602	28.1	01Jan2016, 12:05	2.5
1A1	0.00769	34.6	01Jan2016, 12:06	3.1
1BA	0.00346	16.1	01Jan2016, 12:05	1.4
1BB	0.00068	3.2	01Jan2016, 12:05	0.3
1B1	0.00856	38.2	01Jan2016, 12:06	3.5
1B2A	0.00200	9.3	01Jan2016, 12:05	0.8
1B2B	0.01070	49.9	01Jan2016, 12:05	4.4
1B3	0.00838	36.8	01Jan2016, 12:07	3.4
1C1	0.01144	48.4	01Jan2016, 12:08	4.7
1C2	0.00533	24.1	01Jan2016, 12:06	2.2
2A1	0.01803	80.4	01Jan2016, 12:06	7.4
2A2A	0.00336	15.7	01Jan2016, 12:05	1.4
2A2B	0.00266	12.4	01Jan2016, 12:05	1.1
2BA	0.00378	17.6	01Jan2016, 12:05	1.5
2BB	0.00291	13.6	01Jan2016, 12:05	1.2
2BC	0.00255	11.4	01Jan2016, 12:06	1.0
2C1	0.02730	121.0	01Jan2016, 12:06	11.2
2DA	0.00602	28.1	01Jan2016, 12:05	2.5
2E1	0.01264	59.0	01Jan2016, 12:05	5.2
2F1	0.02332	104.7	01Jan2016, 12:06	9.6
2F2	0.02705	121.9	01Jan2016, 12:06	11.1

Table 1-3. P 100-year, 24-hour Precipitation Event Nodal Areas, Peak Flow Rates, and Runoff Volumes for Landfill Area 1 and Landfill Area 2



Project: Winyah_2020 Simulation Run: Permit_Mod_100YR

Start of Run: 01Jan2016, 00:00
End of Run: 05Feb2016, 00:00
Compute Time: 22Apr2020, 11:36:55
Basin Model: Permit_Mod_Landfill Areas
Meteorologic Model: 100-year_2016
Control Specifications: Control 1

Show Elements: All Elements
Volume Units: ○ IN ● AC-FT
Sorting: Alphabetic

Hydrologic Element	Drainage Area (MI2)	Peak Discharge (CFS)	Time of Peak	Volume (AC-FT)
C.1A	0.03378	190.6	01Jan2016, 12:08	18.5
C.1B	0.00414	25.5	01Jan2016, 12:05	2.3
C.1C	0.01677	95.5	01Jan2016, 12:09	9.2
C.2BA	0.00378	23.3	01Jan2016, 12:05	2.1
C.2BB	0.00291	18.0	01Jan2016, 12:05	1.6
C.2BC	0.00924	55.9	01Jan2016, 12:08	5.1
C.2C	0.02730	159.5	01Jan2016, 12:10	15.0
C.2D	0.00602	37.1	01Jan2016, 12:05	3.3
O.1A	0.04147	236.2	01Jan2016, 12:08	22.8
O.1C	0.01677	95.5	01Jan2016, 12:09	9.2
O.2A	0.02405	142.9	01Jan2016, 12:08	13.2
O.2B	0.00924	55.9	01Jan2016, 12:08	5.1
O.2C	0.02730	159.5	01Jan2016, 12:10	15.0
O.2D	0.00602	37.1	01Jan2016, 12:05	3.3
O.2E	0.01264	78.1	01Jan2016, 12:05	6.9
O.2F	0.05037	299.9	01Jan2016, 12:06	27.7
P.1A1	0.00769	45.6	01Jan2016, 12:08	4.2
P.1BA	0.03378	190.9	01Jan2016, 12:08	18.5
P.1B1	0.02964	168.4	01Jan2016, 12:09	16.3
P.1B2	0.02108	120.4	01Jan2016, 12:09	11.6
P.1B3	0.00838	48.2	01Jan2016, 12:10	4.6
P.1C1	0.01677	95.5	01Jan2016, 12:09	9.2
P.1C2	0.00533	31.8	01Jan2016, 12:07	2.9
P.2A1	0.02405	142.9	01Jan2016, 12:08	13.2
P.2A2	0.00602	37.0	01Jan2016, 12:07	3.3
P.2BA	0.00378	23.3	01Jan2016, 12:05	2.1
P.2BB	0.00924	55.9	01Jan2016, 12:08	5.1
P.2C1	0.02730	159.6	01Jan2016, 12:10	15.0
P.2DA	0.00602	37.1	01Jan2016, 12:05	3.3
1A1	0.00769	45.8	01Jan2016, 12:06	4.2
1BA	0.00346	21.4	01Jan2016, 12:05	1.9
1BB	0.00068	4.2	01Jan2016, 12:05	0.4
1B1	0.00856	50.6	01Jan2016, 12:06	4.7
1B2A	0.00200	12.4	01Jan2016, 12:05	1.1
1B2B	0.01070	66.0	01Jan2016, 12:05	5.9
1B3	0.00838	48.7	01Jan2016, 12:07	4.6
1C1	0.01144	64.1	01Jan2016, 12:08	6.3
1C2	0.00533	31.9	01Jan2016, 12:06	2.9
2A1	0.01803	106.5	01Jan2016, 12:06	9.9
2A2A	0.00336	20.7	01Jan2016, 12:05	1.8
2A2B	0.00266	16.4	01Jan2016, 12:05	1.5
2BA	0.00378	23.3	01Jan2016, 12:05	2.1
2BB	0.00291	18.0	01Jan2016, 12:05	1.6
2BC	0.00255	15.1	01Jan2016, 12:06	1.4
2C1	0.02730	160.2	01Jan2016, 12:06	15.0
2DA	0.00602	37.2	01Jan2016, 12:05	3.3
2E1	0.01264	78.1	01Jan2016, 12:05	6.9
2F1	0.02332	138.5	01Jan2016, 12:06	12.8
2F2	0.02705	161.4	01Jan2016, 12:06	14.8

HEC-HMS HYDROLOGIC MODEL INPUT PARAMETERS

```
 Basin: Permit_Mod_Landfill Areas
    Last Modified Date: 22 April 2020
    Last Modified Time: 15:37:24
    Version: 4.1
    Filepath Separator: \
    Unit System: English
    Missing Flow To Zero: No
    Enable Flow Ratio: No
    Compute Local Flow At Junctions: No

    Enable Sediment Routing: No

    Enable Quality Routing: No
End:

Subbasin: 2BA
    Last Modified Date: 9 March 2020
    Last Modified Time: 20:21:38
    Canvas X: 23003.054744325596
    Canvas Y: -3949.942720958053
    From Canvas X: 8895.376192653515
    From Canvas Y: -18515.221528144317
    Area: 0.00378
    Downstream: P.2BA

    Canopy: None
    Plant Uptake Method: None

    Surface: None

    LossRate: SCS
    Percent Impervious Area: 0.0
    Curve Number: 95

    Transform: SCS
    Lag: 3.60
    Unitgraph Type: STANDARD

    Baseflow: None
End:

Reach: P.2BA
    Last Modified Date: 31 March 2017
    Last Modified Time: 16:23:03
    Canvas X: 20677.44830580029
    Canvas Y: -2228.9304895720807
    From Canvas X: 23248.526410763978
    From Canvas Y: -2737.548829362001
    Downstream: C.2BA

    Route: Kinematic Wave
    Channel: Kinematic Wave
    Length: 100
    Energy Slope: 0.005
    Mannings n: 0.027
    Shape: Trapezoid
    Number of Subreaches: 2
    Width: 3
    Side Slope: 3
    Channel Loss: None
End:

Reach: C.2BA
    Last Modified Date: 31 March 2017
    Last Modified Time: 16:23:01
    Canvas X: 18997.809603640897
    Canvas Y: -2621.8256741307214
    From Canvas X: 20677.44830580029
    From Canvas Y: -2228.9304895720807
    Downstream: P.2BB

    Route: Kinematic Wave
    Channel: Kinematic Wave
    Length: 100
    Energy Slope: 0.005
    Mannings n: 0.012
    Shape: Circular
    Number of Subreaches: 2
    Width: 2
    Channel Loss: None
End:

Subbasin: 2BB
    Last Modified Date: 9 March 2020
    Last Modified Time: 20:21:38
    Canvas X: 21508.415534496355
    Canvas Y: -4834.058034482235
    From Canvas X: 8895.376192653515
    From Canvas Y: -18515.221528144317
    Area: 0.00291
    Downstream: C.2BB

    Canopy: None
    Plant Uptake Method: None

    Surface: None

    LossRate: SCS
    Percent Impervious Area: 0.0
    Curve Number: 95

    Transform: SCS
    Lag: 3.60
    Unitgraph Type: STANDARD

    Baseflow: None
End:

Reach: C.2BB
    Last Modified Date: 31 March 2017
    Last Modified Time: 16:22:17
    Canvas X: 18997.809603640897
    Canvas Y: -2621.8256741307214
    From Canvas X: 19774.14504443189
    From Canvas Y: -4404.683813152387
    Downstream: P.2BB

    Route: Kinematic Wave
    Channel: Kinematic Wave
    Length: 100
    Energy Slope: 0.005
    Mannings n: 0.012
```

```
     Shape: Circular
     Number of Subreaches: 2
     Width: 2
     Channel Loss: None
End:

Subbasin: 2BC
     Last Modified Date: 10 March 2020
     Last Modified Time: 20:10:53
     Canvas X: 18678.88461280165
     Canvas Y: -4541.540694056375
     From Canvas X: 8895.376192653515
     From Canvas Y: -18515.221528144317
     Area: 0.00255
     Downstream: P.2BB

     Canopy: None
     Plant Uptake Method: None

     Surface: None

     LossRate: SCS
     Percent Impervious Area: 0.0
     Curve Number: 95

     Transform: SCS
     Lag: 5.13
     Unitgraph Type: STANDARD

     Baseflow: None
End:

Reach: P.2BB
     Last Modified Date: 31 March 2017
     Last Modified Time: 16:22:56
     Canvas X: 16654.197216235632
     Canvas Y: -2801.102645308636
     From Canvas X: 18997.809603640897
     From Canvas Y: -2621.8256741307214
     Downstream: C.2BC

     Route: Kinematic Wave
     Channel: Kinematic Wave
     Length: 700
     Energy Slope: 0.005
     Mannings n: 0.027
     Shape: Trapezoid
     Number of Subreaches: 2
     Width: 3
     Side Slope: 3
     Channel Loss: None
End:

Reach: C.2BC
     Last Modified Date: 31 March 2017
     Last Modified Time: 16:23:14
     Canvas X: 15761.847071912121
     Canvas Y: -5121.1839390362575
     From Canvas X: 16654.197216235632
     From Canvas Y: -2801.102645308636
     Label X: -59.0
     Label Y: 2.0
     Downstream: O.2B

     Route: Kinematic Wave
     Channel: Kinematic Wave
     Length: 100
     Energy Slope: 0.005
     Mannings n: 0.012
     Shape: Circular
     Number of Subreaches: 2
     Width: 4
     Channel Loss: None
End:

Junction: O.2B
     Last Modified Date: 22 April 2020
     Last Modified Time: 15:37:24
     Canvas X: 15761.847071912121
     Canvas Y: -5121.1839390362575
     From Canvas X: 8895.376192653515
     From Canvas Y: -18515.221528144317
End:

Subbasin: 2DA
     Last Modified Date: 10 March 2020
     Last Modified Time: 20:10:53
     Canvas X: 20320.84178915117
     Canvas Y: -8767.493860325882
     From Canvas X: 8895.376192653515
     From Canvas Y: -18515.221528144317
     Area: 0.00602
     Downstream: P.2DA

     Canopy: None
     Plant Uptake Method: None

     Surface: None

     LossRate: SCS
     Percent Impervious Area: 0.0
     Curve Number: 95

     Transform: SCS
     Lag: 3.6
     Unitgraph Type: STANDARD

     Baseflow: None
End:

Reach: P.2DA
     Last Modified Date: 31 March 2017
     Last Modified Time: 16:18:48
     Canvas X: 20006.750464226818
     Canvas Y: -10834.623116141
     From Canvas X: 22051.566279892017
     From Canvas Y: -10374.11653943865
     Downstream: C.2D

     Route: Kinematic Wave
     Channel: Kinematic Wave
     Length: 100
```

```
     Energy Slope: 0.005
     Mannings n: 0.027
     Shape: Trapezoid
     Number of Subreaches: 2
     Width: 3
     Side Slope: 3
     Channel Loss: None
End:

Reach: C.2D
     Last Modified Date: 31 March 2017
     Last Modified Time: 16:18:48
     Canvas X: 16797.119428312264
     Canvas Y: -10822.998570815244
     From Canvas X: 20006.750464226818
     From Canvas Y: -10834.623116141
     Label X: -4.0
     Label Y: -18.0
     Downstream: O.2D

     Route: Kinematic Wave
     Channel: Kinematic Wave
     Length: 100
     Energy Slope: 0.005
     Mannings n: 0.012
     Shape: Circular
     Number of Subreaches: 2
     Width: 2
     Channel Loss: None
End:

Junction: O.2D
     Last Modified Date: 22 April 2020
     Last Modified Time: 15:37:24
     Canvas X: 16797.119428312264
     Canvas Y: -10822.998570815244
     From Canvas X: 8895.376192653515
     From Canvas Y: -18515.221528144317
     Label X: -55.0
     Label Y: 14.0
End:

Subbasin: 2F2
     Last Modified Date: 9 March 2020
     Last Modified Time: 20:21:38
     Canvas X: 17165.83334587702
     Canvas Y: -7700.389151252137
     From Canvas X: 8895.376192653515
     From Canvas Y: -18515.221528144317
     Area: 0.02705
     Downstream: O.2F

     Canopy: None
     Plant Uptake Method: None

     Surface: None

     LossRate: SCS
     Percent Impervious Area: 0.0
     Curve Number: 95

     Transform: SCS
     Lag: 4.84
     Unitgraph Type: STANDARD

     Baseflow: None
End:

Subbasin: 2F1
     Last Modified Date: 9 March 2020
     Last Modified Time: 20:21:38
     Canvas X: 17972.40072371894
     Canvas Y: -9537.570400780958
     From Canvas X: 8895.376192653515
     From Canvas Y: -18515.221528144317
     Area: 0.02332
     Downstream: O.2F

     Canopy: None
     Plant Uptake Method: None

     Surface: None

     LossRate: SCS
     Percent Impervious Area: 0.0
     Curve Number: 95

     Transform: SCS
     Lag: 4.98
     Unitgraph Type: STANDARD

     Baseflow: None
End:

Junction: O.2F
     Last Modified Date: 22 April 2020
     Last Modified Time: 15:37:24
     Canvas X: 15328.6520963482
     Canvas Y: -8955.049516784013
     From Canvas X: 8895.376192653515
     From Canvas Y: -18515.221528144317
End:

Subbasin: 1B2B
     Last Modified Date: 9 March 2020
     Last Modified Time: 20:21:38
     Canvas X: 14263.186468365408
     Canvas Y: -5060.234731025208
     From Canvas X: 19389.249277992683
     From Canvas Y: -18152.530062061167
     Label X: 0.0
     Label Y: 1.0
     Area: 0.01070
     Downstream: P.1B2

     Canopy: None
     Plant Uptake Method: None

     Surface: None

     LossRate: SCS
     Percent Impervious Area: 0.0
```

```
     Curve Number: 95

     Transform: SCS
     Lag: 3.66
     Unitgraph Type: STANDARD

     Baseflow: None
End:

Subbasin: 1B3
     Last Modified Date: 10 March 2020
     Last Modified Time: 20:10:53
     Canvas X: 12730.526611120333
     Canvas Y: -5691.329966361414
     From Canvas X: 8895.376192653515
     From Canvas Y: -18515.221528144317
     Area: 0.00838
     Downstream: P.1B3

     Canopy: None
     Plant Uptake Method: None

     Surface: None

     LossRate: SCS
     Percent Impervious Area: 0.0
     Curve Number: 95

     Transform: SCS
     Lag: 5.72
     Unitgraph Type: STANDARD

     Baseflow: None
End:

Reach: P.1B3
     Last Modified Date: 31 March 2017
     Last Modified Time: 16:17:13
     Canvas X: 12375.961346364431
     Canvas Y: -2555.952947269673
     From Canvas X: 12871.815426171663
     From Canvas Y: -4910.980340324205
     Downstream: P.1B2

     Route: Kinematic Wave
     Channel: Kinematic Wave
     Length: 800
     Energy Slope: 0.005
     Mannings n: 0.027
     Shape: Trapezoid
     Number of Subreaches: 2
     Width: 3
     Side Slope: 3
     Channel Loss: None
End:

Subbasin: 1B2A
     Last Modified Date: 9 March 2020
     Last Modified Time: 20:21:38
     Canvas X: 10987.501675429858
     Canvas Y: -5090.286885088837
     From Canvas X: 8895.376192653515
     From Canvas Y: -18515.221528144317
     Area: 0.00200
     Downstream: P.1B2

     Canopy: None
     Plant Uptake Method: None

     Surface: None

     LossRate: SCS
     Percent Impervious Area: 0.0
     Curve Number: 95

     Transform: SCS
     Lag: 3.60
     Unitgraph Type: STANDARD

     Baseflow: None
End:

Reach: P.1B2
     Last Modified Date: 31 March 2017
     Last Modified Time: 16:17:13
     Canvas X: 8839.266923751873
     Canvas Y: -2095.7621939451737
     From Canvas X: 12375.961346364431
     From Canvas Y: -2555.952947269673
     Label X: -18.0
     Label Y: 11.0
     Downstream: P.1B1

     Route: Kinematic Wave
     Channel: Kinematic Wave
     Length: 953
     Energy Slope: 0.005
     Mannings n: 0.027
     Shape: Trapezoid
     Number of Subreaches: 2
     Width: 3
     Side Slope: 3
     Channel Loss: None
End:

Subbasin: 1B1
     Last Modified Date: 10 March 2020
     Last Modified Time: 20:10:53
     Canvas X: 9755.36335882107
     Canvas Y: -5060.234731025208
     From Canvas X: 8895.376192653515
     From Canvas Y: -18515.221528144317
     Area: 0.00856
     Downstream: P.1B1

     Canopy: None
     Plant Uptake Method: None

     Surface: None

     LossRate: SCS
     Percent Impervious Area: 0.0
```

```
     Curve Number: 95

     Transform: SCS
     Lag: 5.15
     Unitgraph Type: STANDARD

     Baseflow: None
End:

Reach: P.1B1
     Last Modified Date: 31 March 2017
     Last Modified Time: 16:16:58
     Canvas X: 5967.055620542664
     Canvas Y: -1674.5485509712807
     From Canvas X: 8839.266923751873
     From Canvas Y: -2095.7621939451737
     Label X: -12.0
     Label Y: 12.0
     Downstream: P.1BA

     Route: Kinematic Wave
     Channel: Kinematic Wave
     Length: 215
     Energy Slope: 0.005
     Mannings n: 0.027
     Shape: Trapezoid
     Number of Subreaches: 2
     Width: 3
     Side Slope: 3
     Channel Loss: None
End:

Subbasin: 1BA
     Last Modified Date: 9 March 2020
     Last Modified Time: 20:21:38
     Canvas X: 7681.764728430675
     Canvas Y: -4849.869652579806
     From Canvas X: 8895.376192653515
     From Canvas Y: -18515.221528144317
     Area: 0.00346
     Downstream: C.1B

     Canopy: None
     Plant Uptake Method: None

     Surface: None

     LossRate: SCS
     Percent Impervious Area: 0.0
     Curve Number: 95

     Transform: SCS
     Lag: 3.60
     Unitgraph Type: STANDARD

     Baseflow: None
End:

Subbasin: 1BB
     Last Modified Date: 9 March 2020
     Last Modified Time: 20:21:38
     Canvas X: 5397.801019594877
     Canvas Y: -3798.0442603527936
     From Canvas X: 8895.376192653515
     From Canvas Y: -18515.221528144317
     Area: 0.00068
     Downstream: C.1B

     Canopy: None
     Plant Uptake Method: None

     Surface: None

     LossRate: SCS
     Percent Impervious Area: 0.0
     Curve Number: 95

     Transform: SCS
     Lag: 3.60
     Unitgraph Type: STANDARD

     Baseflow: None
End:

Reach: C.1B
     Last Modified Date: 31 March 2017
     Last Modified Time: 16:16:42
     Canvas X: 5967.055620542664
     Canvas Y: -1674.5485509712807
     From Canvas X: 6075.668158437937
     From Canvas Y: -2758.7492922041447
     Downstream: P.1BA

     Route: Kinematic Wave
     Channel: Kinematic Wave
     Length: 100
     Energy Slope: 0.005
     Mannings n: 0.012
     Shape: Circular
     Number of Subreaches: 2
     Width: 2
     Channel Loss: None
End:

Reach: P.1BA
     Last Modified Date: 31 March 2017
     Last Modified Time: 16:16:45
     Canvas X: 3644.8714328581973
     Canvas Y: -2212.727072916364
     From Canvas X: 5967.055620542664
     From Canvas Y: -1674.5485509712807
     Label X: -13.0
     Label Y: 14.0
     Downstream: C.1A

     Route: Kinematic Wave
     Channel: Kinematic Wave
     Length: 105
     Energy Slope: 0.005
     Mannings n: 0.027
     Shape: Trapezoid
     Number of Subreaches: 2
```

```
     Width: 3
     Side Slope: 3
     Channel Loss: None
End:

Reach: C.1A
     Last Modified Date: 31 March 2017
     Last Modified Time: 16:16:35
     Canvas X: 2302.4291510410985
     Canvas Y: -3016.6882546984416
     From Canvas X: 3644.8714328581973
     From Canvas Y: -2212.727072916364
     Label X: -3.0
     Label Y: 18.0
     Downstream: O.1A

     Route: Kinematic Wave
     Channel: Kinematic Wave
     Length: 100
     Energy Slope: 0.005
     Mannings n: 0.012
     Shape: Circular
     Number of Subreaches: 2
     Width: 4
     Channel Loss: None
End:

Subbasin: 1A1
     Last Modified Date: 10 March 2020
     Last Modified Time: 20:10:53
     Canvas X: 5818.5311764856815
     Canvas Y: -5360.7562716614975
     From Canvas X: 8895.376192653515
     From Canvas Y: -18515.221528144317
     Area: 0.00769
     Downstream: P.1A1

     Canopy: None
     Plant Uptake Method: None

     Surface: None

     LossRate: SCS
     Percent Impervious Area: 0.0
     Curve Number: 95

     Transform: SCS
     Lag: 4.92
     Unitgraph Type: STANDARD

     Baseflow: None
End:

Reach: P.1A1
     Last Modified Date: 31 March 2017
     Last Modified Time: 16:16:05
     Canvas X: 2302.4291510410985
     Canvas Y: -3016.6882546984416
     From Canvas X: 4144.4312883434895
     From Canvas Y: -5419.404425552808
     Downstream: O.1A

     Route: Kinematic Wave
     Channel: Kinematic Wave
     Length: 525
     Energy Slope: 0.005
     Mannings n: 0.027
     Shape: Trapezoid
     Number of Subreaches: 2
     Width: 3
     Side Slope: 3
     Channel Loss: None
End:

Junction: O.1A
     Last Modified Date: 22 April 2020
     Last Modified Time: 15:37:24
     Canvas X: 2302.4291510410985
     Canvas Y: -3016.6882546984416
     From Canvas X: 8895.376192653515
     From Canvas Y: -18515.221528144317
     Label X: -64.0
     Label Y: 12.0
End:

Subbasin: 2C1
     Last Modified Date: 10 March 2020
     Last Modified Time: 20:10:53
     Canvas X: 23530.332367961128
     Canvas Y: -8744.078342601737
     From Canvas X: 8895.376192653515
     From Canvas Y: -18515.221528144317
     Area: 0.02730
     Downstream: P.2C1

     Canopy: None
     Plant Uptake Method: None

     Surface: None

     LossRate: SCS
     Percent Impervious Area: 0.0
     Curve Number: 95

     Transform: SCS
     Lag: 5.32
     Unitgraph Type: STANDARD

     Baseflow: None
End:

Reach: P.2C1
     Last Modified Date: 31 March 2017
     Last Modified Time: 16:19:25
     Canvas X: 25559.082510168057
     Canvas Y: -10452.762155759388
     From Canvas X: 23697.096578980454
     From Canvas Y: -10382.476830461705
     Label X: -6.0
     Label Y: -14.0
     Downstream: C.2C
```

```
     Route: Kinematic Wave
     Channel: Kinematic Wave
     Length: 1350
     Energy Slope: 0.005
     Mannings n: 0.027
     Shape: Trapezoid
     Number of Subreaches: 2
     Width: 3
     Side Slope: 3
     Channel Loss: None
End:

Reach: C.2C
     Last Modified Date: 31 March 2017
     Last Modified Time: 16:19:27
     Canvas X: 26713.83693786458
     Canvas Y: -8852.916105675358
     From Canvas X: 25559.082510168057
     From Canvas Y: -10452.762155759388
     Downstream: O.2C

     Route: Kinematic Wave
     Channel: Kinematic Wave
     Length: 100
     Energy Slope: 0.005
     Mannings n: 0.012
     Shape: Circular
     Number of Subreaches: 2
     Width: 4
     Channel Loss: None
End:

Junction: O.2C
     Last Modified Date: 22 April 2020
     Last Modified Time: 15:37:24
     Canvas X: 26713.83693786458
     Canvas Y: -8852.916105675358
     From Canvas X: 8895.376192653515
     From Canvas Y: -18515.221528144317
End:

Subbasin: 2A1
     Last Modified Date: 10 March 2020
     Last Modified Time: 20:10:53
     Canvas X: 27961.051448809565
     Canvas Y: -6152.391400207119
     From Canvas X: 8895.376192653515
     From Canvas Y: -18515.221528144317
     Area: 0.01803
     Downstream: P.2A1

     Canopy: None
     Plant Uptake Method: None

     Surface: None

     LossRate: SCS
     Percent Impervious Area: 0.0
     Curve Number: 95

     Transform: SCS
     Lag: 5.16
     Unitgraph Type: STANDARD

     Baseflow: None
End:

Subbasin: 2A2A
     Last Modified Date: 9 March 2020
     Last Modified Time: 20:21:38
     Canvas X: 25914.170287235196
     Canvas Y: -6173.101796981193
     From Canvas X: 8895.376192653515
     From Canvas Y: -18515.221528144317
     Area: 0.00336
     Downstream: P.2A2

     Canopy: None
     Plant Uptake Method: None

     Surface: None

     LossRate: SCS
     Percent Impervious Area: 0.0
     Curve Number: 95

     Transform: SCS
     Lag: 3.60
     Unitgraph Type: STANDARD

     Baseflow: None
End:

Subbasin: 2A2B
     Last Modified Date: 9 March 2020
     Last Modified Time: 20:21:38
     Canvas X: 24615.649241001633
     Canvas Y: -4915.1595334424255
     From Canvas X: 8895.376192653515
     From Canvas Y: -18515.221528144317
     Area: 0.00266
     Downstream: P.2A2

     Canopy: None
     Plant Uptake Method: None

     Surface: None

     LossRate: SCS
     Percent Impervious Area: 0.0
     Curve Number: 95

     Transform: SCS
     Lag: 3.60
     Unitgraph Type: STANDARD

     Baseflow: None
End:

Reach: P.2A2
     Last Modified Date: 31 March 2017
     Last Modified Time: 16:20:54
```

```
    Canvas X: 28535.491595925763
    Canvas Y: -4400.863806752261
    From Canvas X: 26457.235003130787
    From Canvas Y: -4087.875881557411
    Downstream: P.2A1

    Route: Kinematic Wave
    Channel: Kinematic Wave
    Length: 690
    Energy Slope: 0.005
    Mannings n: 0.027
    Shape: Trapezoid
    Number of Subreaches: 2
    Width: 3
    Side Slope: 3
    Channel Loss: None
End:

Reach: P.2A1
    Last Modified Date: 31 March 2017
    Last Modified Time: 16:20:54
    Canvas X: 29768.736817670226
    Canvas Y: -6207.169744718049
    From Canvas X: 28535.491595925763
    From Canvas Y: -4400.863806752261
    Downstream: O.2A

    Route: Kinematic Wave
    Channel: Kinematic Wave
    Length: 600
    Energy Slope: 0.005
    Mannings n: 0.027
    Shape: Trapezoid
    Number of Subreaches: 2
    Width: 3
    Side Slope: 3
    Channel Loss: None
End:

Junction: O.2A
    Last Modified Date: 22 April 2020
    Last Modified Time: 15:37:24
    Canvas X: 29768.736817670226
    Canvas Y: -6207.169744718049
    From Canvas X: 8895.376192653515
    From Canvas Y: -18515.221528144317
End:

Subbasin: 1C1
    Last Modified Date: 10 March 2020
    Last Modified Time: 20:10:53
    Canvas X: 10481.609778654665
    Canvas Y: -6541.04394021408
    From Canvas X: 8895.376192653515
    From Canvas Y: -18515.221528144317
    Area: 0.01144
    Downstream: P.1C1

    Canopy: None
    Plant Uptake Method: None

    Surface: None

    LossRate: SCS
    Percent Impervious Area: 0.0
    Curve Number: 95

    Transform: SCS
    Lag: 6.88
    Unitgraph Type: STANDARD

    Baseflow: None
End:

Subbasin: 1C2
    Last Modified Date: 10 April 2020
    Last Modified Time: 19:49:36
    Canvas X: 13462.100862199468
    Canvas Y: -6598.028605270678
    From Canvas X: 8895.376192653515
    From Canvas Y: -18515.221528144317
    Area: 0.00533
    Downstream: P.1C2

    Canopy: None
    Plant Uptake Method: None

    Surface: None

    LossRate: SCS
    Percent Impervious Area: 0.0
    Curve Number: 95

    Transform: SCS
    Lag: 4.74
    Unitgraph Type: STANDARD

    Baseflow: None
End:

Reach: P.1C2
    Last Modified Date: 31 March 2017
    Last Modified Time: 16:13:25
    Canvas X: 10373.860234577145
    Canvas Y: -8170.956850765629
    From Canvas X: 13010.843942149506
    From Canvas Y: -8483.555281082447
    Label X: 2.0
    Label Y: 14.0
    Downstream: P.1C1

    Route: Kinematic Wave
    Channel: Kinematic Wave
    Length: 415
    Energy Slope: 0.005
    Mannings n: 0.027
    Shape: Trapezoid
    Number of Subreaches: 2
    Width: 3
    Side Slope: 3
    Channel Loss: None
End:
```

```
Reach: P.1C1
   Last Modified Date: 31 March 2017
   Last Modified Time: 16:13:25
   Canvas X: 7318.024026413357
   Canvas Y: -7835.279612895717
   From Canvas X: 10373.860234577145
   From Canvas Y: -8170.956850765629
   Downstream: C.1C

   Route: Kinematic Wave
   Channel: Kinematic Wave
   Length: 340
   Energy Slope: 0.005
   Mannings n: 0.027
   Shape: Trapezoid
   Number of Subreaches: 2
   Width: 3
   Side Slope: 3
   Channel Loss: None
End:

Reach: C.1C
   Last Modified Date: 31 March 2017
   Last Modified Time: 16:13:28
   Canvas X: 8694.584794293705
   Canvas Y: -8912.913464911355
   From Canvas X: 7318.024026413357
   From Canvas Y: -7835.279612895717
   Downstream: O.1C

   Route: Kinematic Wave
   Channel: Kinematic Wave
   Length: 100
   Energy Slope: 0.005
   Mannings n: 0.012
   Shape: Circular
   Number of Subreaches: 2
   Width: 6
   Channel Loss: None
End:

Junction: O.1C
   Last Modified Date: 22 April 2020
   Last Modified Time: 15:37:24
   Canvas X: 8694.584794293705
   Canvas Y: -8912.913464911355
   From Canvas X: 8895.376192653515
   From Canvas Y: -18515.221528144317
End:

Subbasin: 2E1
   Last Modified Date: 9 March 2020
   Last Modified Time: 20:21:38
   Canvas X: 23713.64404493401
   Canvas Y: -7373.647496731782
   From Canvas X: 8895.376192653515
   From Canvas Y: -18515.221528144317
   Area: 0.01264
   Downstream: O.2E

   Canopy: None
   Plant Uptake Method: None

   Surface: None

   LossRate: SCS
   Percent Impervious Area: 0.0
   Curve Number: 95

   Transform: SCS
   Lag: 3.60
   Unitgraph Type: STANDARD

   Baseflow: None
End:

Junction: O.2E
   Last Modified Date: 22 April 2020
   Last Modified Time: 15:37:24
   Canvas X: 26232.502654473094
   Canvas Y: -7373.647496731782
   From Canvas X: 8895.376192653515
   From Canvas Y: -18515.221528144317
End:

Basin Schematic Properties:
   Last View N: -5886.038098238729
   Last View S: -23932.82626360068
   Last View W: 112.26896901012151
   Last View E: 22946.90592082476
   Maximum View N: 417.93047210596296
   Maximum View S: -24691.817804820963
   Maximum View W: -2237.445293676677
   Maximum View E: 41947.30928408436
   Extent Method: Elements
   Buffer: 20
   Draw Icons: Yes
   Draw Icon Labels: Name
   Draw Map Objects: No
   Draw Gridlines: No
   Draw Flow Direction: No
   Fix Element Locations: No
   Fix Hydrologic Order: No
End::
```

Appendix 2

Drainage Feature Calculations

Design/Check: Trapezoidal/Triangular Channel

Methodology: Manning's Equation

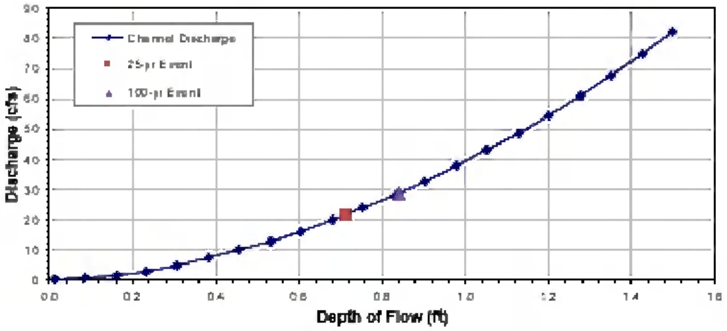
Project: Santee Cooper - Winyah Generating Station

Ditch ID: Side Slope Drainage Terrace

Peak Discharge, Q_{25} = 21.64 cfs (25-yr Event)
Peak Discharge, Q_{100} = 28.65 cfs (100-yr Event)
Bottom Width, B = 6.00 ft
Left Side Slope, Z_1 = 2.0 horizontal :1 vertical
Right Side Slope, Z_2 = 2.0 horizontal :1 vertical
Channel Depth, Y = 1.50 ft
Top Width, T = 12.0 ft
Manning's Roughness Coeff., n = 0.036
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	82.3	1.33	
0.72	5.32	9.20	0.58	4.06	21.60	0.72	Q (25-yr Event)
0.84	6.45	9.75	0.66	4.44	28.63	0.82	Q (100-yr Event)

Discharge versus Depth Relationship



Design/Check: Trapezoidal/Triangular Channel

Methodology: Manning's Equation

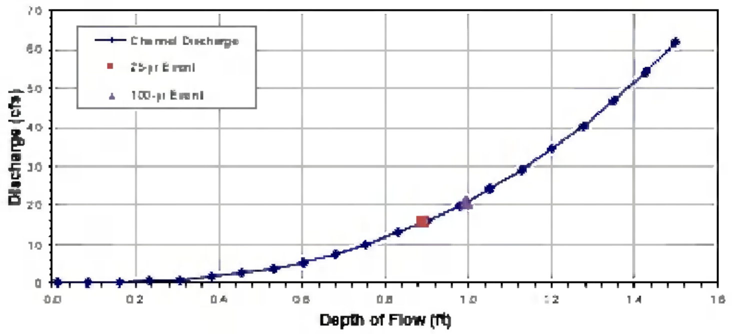
Project: Santee Cooper - Winyah Generating Station

Ditch ID: **Top Deck Drainage Terrace**

Peak Discharge, Q_{25} = 15.69 cfs (25-yr Event)
Peak Discharge, Q_{100} = 20.77 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	
0.89	9.20	20.74	0.44	1.70	15.66	0.14	Q (25-yr Event)
0.99	11.35	23.04	0.49	1.83	20.72	0.15	Q (100-yr Event)

Discharge versus Depth Relationship



Design/Check: Trapezoidal/Triangular Channel

Methodology: Manning's Equation

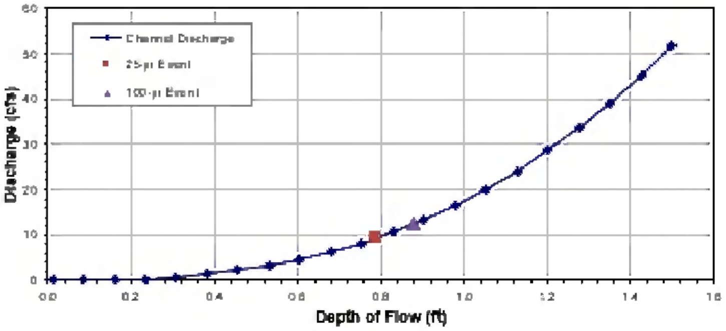
Project: Santee Cooper - Winyah Generating Station

Ditch ID: 1B2A Access Road Drainage Terrace

Peak Discharge, Q_{25} = 9.33 cfs (25-yr Event)
Peak Discharge, Q_{100} = 12.35 cfs (100-yr Event)
Bottom Width, B = 0.00 ft
Left Side Slope, Z_1 = 3.0 horizontal :1 vertical
Right Side Slope, Z_2 = 2.0 horizontal :1 vertical
Channel Depth, Y = 1.50 ft
Top Width, T = 7.5 ft
Manning's Roughness Coeff., n = 0.036
Longitudinal Channel Slope, S_o = 0.0800 ft/ft

Depth of Flow Y ft	Area of Flow A ft^2	Wetted Perimeter P ft	Hydraulic Radius R=A/P ft	Average Velocity V ft/s	Discharge (Flow Rate) Q=AV ft^3/s	Avg. Tractive Stress τ_0 lb/ft^2	Comments
0.01	0.00	0.05	0.00	0.32	0.0	0.02	
0.08	0.02	0.46	0.04	1.35	0.0	0.20	
0.16	0.06	0.86	0.07	2.05	0.1	0.37	
0.23	0.14	1.26	0.11	2.66	0.4	0.54	
0.31	0.24	1.66	0.14	3.19	0.8	0.71	
0.38	0.37	2.06	0.18	3.69	1.3	0.88	
0.46	0.52	2.47	0.21	4.16	2.2	1.06	
0.53	0.71	2.87	0.25	4.60	3.2	1.23	
0.61	0.92	3.27	0.28	5.02	4.6	1.40	
0.68	1.16	3.67	0.32	5.42	6.3	1.57	
0.76	1.43	4.08	0.35	5.81	8.3	1.75	
0.83	1.72	4.48	0.38	6.18	10.6	1.92	
0.90	2.04	4.88	0.42	6.55	13.4	2.09	
0.98	2.39	5.28	0.45	6.90	16.5	2.26	
1.05	2.77	5.68	0.49	7.25	20.1	2.43	
1.13	3.18	6.09	0.52	7.59	24.1	2.61	
1.20	3.61	6.49	0.56	7.92	28.6	2.78	
1.28	4.07	6.89	0.59	8.24	33.6	2.95	
1.35	4.56	7.29	0.63	8.56	39.1	3.12	
1.43	5.08	7.70	0.66	8.87	45.1	3.30	
1.50	5.63	8.10	0.69	9.18	51.6	3.47	
0.79	1.55	4.26	0.37	5.98	9.28	1.82	Q (25-yr Event)
0.88	1.92	4.73	0.41	6.41	12.30	2.03	Q (100-yr Event)

Discharge versus Depth Relationship



Design/Check: Trapezoidal/Triangular Channel

Methodology: Manning's Equation

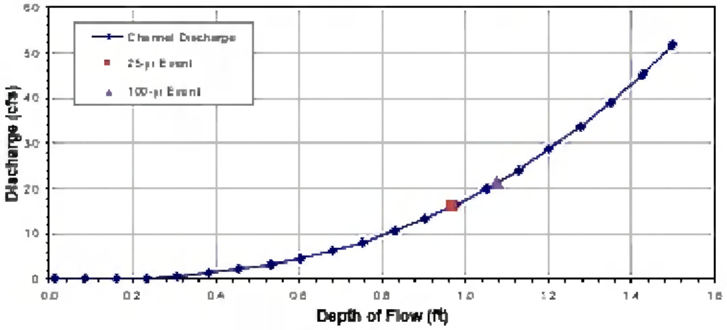
Project: Santee Cooper - Winyah Generating Station

Ditch ID: **1BA** **Access Road Drainage Terrace**

Peak Discharge, Q_{25} =	16.15	cfs (25-yr Event)
Peak Discharge, Q_{100} =	21.37	cfs (100-yr Event)
Bottom Width, B =	0.00	ft
Left Side Slope, Z_1 =	3.0	horizontal :1 vertical
Right Side Slope, Z_2 =	2.0	horizontal :1 vertical
Channel Depth, Y =	1.50	ft
Top Width, T =	7.5	ft
Manning's Roughness Coeff., n =	0.036	
Longitudinal Channel Slope, S_o =	0.0600	ft/ft

Depth of Flow Y ft	Area of Flow A ft^2	Wetted Perimeter P ft	Hydraulic Radius R=A/P ft	Average Velocity V ft/s	Discharge (Flow Rate) Q=AV ft^3/s	Avg. Tractive Stress τ_0 lb/ft^2	Comments
0.01	0.00	0.05	0.00	0.32	0.0	0.02	
0.08	0.02	0.46	0.04	1.35	0.0	0.20	
0.16	0.06	0.86	0.07	2.05	0.1	0.37	
0.23	0.14	1.26	0.11	2.66	0.4	0.54	
0.31	0.24	1.66	0.14	3.19	0.8	0.71	
0.38	0.37	2.06	0.18	3.69	1.3	0.88	
0.46	0.52	2.47	0.21	4.16	2.2	1.06	
0.53	0.71	2.87	0.25	4.60	3.2	1.23	
0.61	0.92	3.27	0.28	5.02	4.6	1.40	
0.68	1.16	3.67	0.32	5.42	6.3	1.57	
0.76	1.43	4.08	0.35	5.81	8.3	1.75	
0.83	1.72	4.48	0.38	6.18	10.6	1.92	
0.90	2.04	4.88	0.42	6.55	13.4	2.09	
0.98	2.39	5.28	0.45	6.90	16.5	2.26	
1.05	2.77	5.68	0.49	7.25	20.1	2.43	
1.13	3.18	6.09	0.52	7.59	24.1	2.61	
1.20	3.61	6.49	0.56	7.92	28.6	2.78	
1.28	4.07	6.89	0.59	8.24	33.6	2.95	
1.35	4.56	7.29	0.63	8.56	39.1	3.12	
1.43	5.08	7.70	0.66	8.87	45.1	3.30	
1.50	5.63	8.10	0.69	9.18	51.6	3.47	
0.97	2.35	5.23	0.45	6.86	16.13	2.24	Q (25-yr Event)
1.08	2.90	5.81	0.50	7.36	21.32	2.49	Q (100-yr Event)

Discharge versus Depth Relationship



Design/Check: Trapezoidal/Triangular Channel

Methodology: Manning's Equation

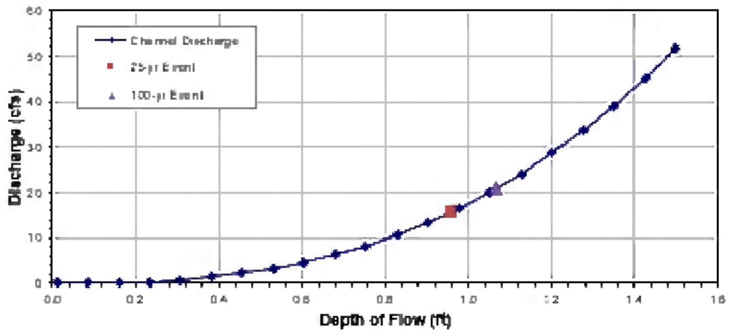
Project: Santee Cooper - Winyah Generating Station

Ditch ID: 2A2A Access Road Drainage Terrace

Peak Discharge, Q_{25} = 15.68 cfs (25-yr Event)
Peak Discharge, Q_{100} = 20.75 cfs (100-yr Event)
Bottom Width, B = 0.00 ft
Left Side Slope, Z_1 = 3.0 horizontal :1 vertical
Right Side Slope, Z_2 = 2.0 horizontal :1 vertical
Channel Depth, Y = 1.50 ft
Top Width, T = 7.5 ft
Manning's Roughness Coeff., n = 0.036
Longitudinal Channel Slope, S_o = 0.0800 ft/ft

Depth of Flow Y ft	Area of Flow A ft^2	Wetted Perimeter P ft	Hydraulic Radius R=A/P ft	Average Velocity V ft/s	Discharge (Flow Rate) Q=AV ft^3/s	Avg. Tractive Stress τ_0 lb/ft^2	Comments
0.01	0.00	0.05	0.00	0.32	0.0	0.02	
0.08	0.02	0.46	0.04	1.35	0.0	0.20	
0.16	0.06	0.86	0.07	2.05	0.1	0.37	
0.23	0.14	1.26	0.11	2.66	0.4	0.54	
0.31	0.24	1.66	0.14	3.19	0.8	0.71	
0.38	0.37	2.06	0.18	3.69	1.3	0.88	
0.46	0.52	2.47	0.21	4.16	2.2	1.06	
0.53	0.71	2.87	0.25	4.60	3.2	1.23	
0.61	0.92	3.27	0.28	5.02	4.6	1.40	
0.68	1.16	3.67	0.32	5.42	6.3	1.57	
0.76	1.43	4.08	0.35	5.81	8.3	1.75	
0.83	1.72	4.48	0.38	6.18	10.6	1.92	
0.90	2.04	4.88	0.42	6.55	13.4	2.09	
0.98	2.39	5.28	0.45	6.90	16.5	2.26	
1.05	2.77	5.68	0.49	7.25	20.1	2.43	
1.13	3.18	6.09	0.52	7.59	24.1	2.61	
1.20	3.61	6.49	0.56	7.92	28.6	2.78	
1.28	4.07	6.89	0.59	8.24	33.6	2.95	
1.35	4.56	7.29	0.63	8.56	39.1	3.12	
1.43	5.08	7.70	0.66	8.87	45.1	3.30	
1.50	5.63	8.10	0.69	9.18	51.6	3.47	
0.96	2.30	5.17	0.44	6.81	15.64	2.22	Q (25-yr Event)
1.07	2.84	5.75	0.49	7.31	20.72	2.46	Q (100-yr Event)

Discharge versus Depth Relationship



Design/Check: Trapezoidal/Triangular Channel

Methodology: Manning's Equation

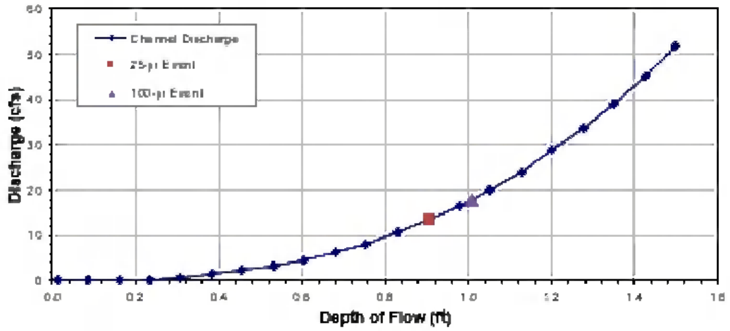
Project: Santee Cooper - Winyah Generating Station

Ditch ID: 2BB Access Road Drainage Terrace

Peak Discharge, Q_{25} =	13.58	cfs (25-yr Event)
Peak Discharge, Q_{100} =	17.97	cfs (100-yr Event)
Bottom Width, B =	0.00	ft
Left Side Slope, Z_1 =	3.0	horizontal :1 vertical
Right Side Slope, Z_2 =	2.0	horizontal :1 vertical
Channel Depth, Y =	1.50	ft
Top Width, T =	7.5	ft
Manning's Roughness Coeff., n =	0.036	
Longitudinal Channel Slope, S_o =	0.0800	ft/ft

Depth of Flow Y ft	Area of Flow A ft^2	Wetted Perimeter P ft	Hydraulic Radius R=A/P ft	Average Velocity V ft/s	Discharge (Flow Rate) Q=AV ft^3/s	Avg. Tractive Stress τ_0 lb/ft^2	Comments
0.01	0.00	0.05	0.00	0.32	0.0	0.02	
0.08	0.02	0.46	0.04	1.35	0.0	0.20	
0.16	0.06	0.86	0.07	2.05	0.1	0.37	
0.23	0.14	1.26	0.11	2.66	0.4	0.54	
0.31	0.24	1.66	0.14	3.19	0.8	0.71	
0.38	0.37	2.06	0.18	3.69	1.3	0.88	
0.46	0.52	2.47	0.21	4.16	2.2	1.06	
0.53	0.71	2.87	0.25	4.60	3.2	1.23	
0.61	0.92	3.27	0.28	5.02	4.6	1.40	
0.68	1.16	3.67	0.32	5.42	6.3	1.57	
0.76	1.43	4.08	0.35	5.81	8.3	1.75	
0.83	1.72	4.48	0.38	6.18	10.6	1.92	
0.90	2.04	4.88	0.42	6.55	13.4	2.09	
0.98	2.39	5.28	0.45	6.90	16.5	2.26	
1.05	2.77	5.68	0.49	7.25	20.1	2.43	
1.13	3.18	6.09	0.52	7.59	24.1	2.61	
1.20	3.61	6.49	0.56	7.92	28.6	2.78	
1.28	4.07	6.89	0.59	8.24	33.6	2.95	
1.35	4.56	7.29	0.63	8.56	39.1	3.12	
1.43	5.08	7.70	0.66	8.87	45.1	3.30	
1.50	5.63	8.10	0.69	9.18	51.6	3.47	
0.91	2.06	4.91	0.42	6.57	13.57	2.10	Q (25-yr Event)
1.01	2.54	5.44	0.47	7.05	17.92	2.33	Q (100-yr Event)

Discharge versus Depth Relationship



Down Drain Feature: 1A1
Flow Through Circular Pipe

Number of Pipes, N= 1
Diameter of pipe, D= 18 inches
Longitudinal Slope, So= 0.040 ft/ft
Manning's n= 0.009
Density of flowing liquid, rho= 1.94 slugs/ft^3
Peak Discharge, Q_{25}= 19.5 cfs
Peak Discharge, Q_{100}= 25.7 cfs

Theta radians	Theta degrees	Depth of Flow y inches	Area of Flow A ft^2	Wetted Perimeter P ft	Hydraulic Radius R ft	Average Velocity V ft/s	Discharge Q=A*V cfs	Force F lbf
0.00	0	0.0	0.000	0.00		0.0	0.00	0.0
0.25	14	0.1	0.001	0.19	0.00	0.8	0.00	0.0
0.50	29	0.3	0.006	0.38	0.02	2.0	0.01	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
3.47	199	10.5	1.066	2.60	0.41	18.3	19.46	689.6
3.99	229	12.7	1.334	2.99	0.45	19.3	25.76	965.4

Down Drain Feature: 1B1
Flow Through Circular Pipe

Number of Pipes, N=	2	
Diameter of pipe, D=	18	inches
Longitudinal Slope, So=	0.040	ft/ft
Manning's n=	0.009	
Density of flowing liquid, rho=	1.94	slugs/ft^3
Peak Discharge, Q_{25}=	38.2	cfs
Peak Discharge, Q_{100}=	50.5	cfs

Theta radians	Theta degrees	Depth of Flow y inches	Area of Flow A ft^2	Wetted Perimeter P ft	Hydraulic Radius R ft	Average Velocity V ft/s	Discharge Q=A*V cfs	Force F lbf
0.00	0	0.0	0.000	0.00		0.0	0.00	0.0
0.25	14	0.1	0.001	0.19	0.00	0.8	0.00	0.0
0.50	29	0.3	0.006	0.38	0.02	2.0	0.02	0.0
0.75	43	0.6	0.019	0.56	0.03	3.5	0.13	0.5
1.00	57	1.1	0.045	0.75	0.06	5.0	0.45	2.2
1.25	72	1.7	0.085	0.94	0.09	6.7	1.13	7.3
1.50	86	2.4	0.141	1.13	0.13	8.3	2.35	18.9
1.75	100	3.2	0.215	1.31	0.16	9.9	4.27	41.1
2.00	115	4.1	0.307	1.50	0.20	11.5	7.05	78.5
2.25	129	5.1	0.414	1.69	0.25	13.0	10.74	135.1
2.50	143	6.2	0.535	1.88	0.29	14.3	15.34	213.4
2.75	158	7.2	0.666	2.06	0.32	15.6	20.76	313.7
3.00	172	8.4	0.804	2.25	0.36	16.7	26.80	433.4
3.25	186	9.5	0.944	2.44	0.39	17.6	33.23	567.1
3.50	201	10.6	1.083	2.63	0.41	18.3	39.74	707.1
3.75	215	11.7	1.215	2.81	0.43	18.9	45.99	844.2
4.00	229	12.7	1.338	3.00	0.45	19.3	51.70	969.0
4.25	244	13.7	1.447	3.19	0.45	19.6	56.59	1073.2
4.50	258	14.7	1.541	3.38	0.46	19.6	60.46	1151.0
4.75	272	15.5	1.617	3.56	0.45	19.6	63.23	1199.0
5.00	286	16.2	1.676	3.75	0.45	19.3	64.86	1217.3
5.25	301	16.8	1.718	3.94	0.44	19.0	65.44	1208.8
5.50	315	17.3	1.745	4.13	0.42	18.7	65.12	1178.4
5.75	329	17.7	1.760	4.31	0.41	18.2	64.12	1132.7
6.00	344	17.9	1.766	4.50	0.39	17.7	62.67	1078.7
6.25	358	18.0	1.767	4.69	0.38	17.3	61.05	1022.9
3.44	197	10.3	1.050	2.58	0.41	18.2	38.19	673.7
3.95	226	12.5	1.314	2.96	0.44	19.3	50.60	945.2

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}= 28.5 cfs
Peak Discharge, Q_{100}= 37.8 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
3.07	176	8.7	0.842	2.30	0.37	16.9	28.52	468.7
3.42	196	10.3	1.042	2.57	0.41	18.1	37.79	665.1

Down Drain Feature: 1B3
Flow Through Circular Pipe

Number of Pipes, N= 2
Diameter of pipe, D= 18 inches
Longitudinal Slope, So= 0.040 ft/ft
Manning's n= 0.009
Density of flowing liquid, rho= 1.94 slugs/ft^3
Peak Discharge, Q_{25}= 30.3 cfs
Peak Discharge, Q_{100}= 40.1 cfs

Theta radians	Theta degrees	Depth of Flow y inches	Area of Flow A ft^2	Wetted Perimeter P ft	Hydraulic Radius R ft	Average Velocity V ft/s	Discharge Q=A*V cfs	Force F lbf
0.00	0	0.0	0.000	0.00		0.0	0.00	0.0
0.25	14	0.1	0.001	0.19	0.00	0.8	0.00	0.0
0.50	29	0.3	0.006	0.38	0.02	2.0	0.02	0.0
0.75	43	0.6	0.019	0.56	0.03	3.5	0.13	0.5
1.00	57	1.1	0.045	0.75	0.06	5.0	0.45	2.2
1.25	72	1.7	0.085	0.94	0.09	6.7	1.13	7.3
1.50	86	2.4	0.141	1.13	0.13	8.3	2.35	18.9
1.75	100	3.2	0.215	1.31	0.16	9.9	4.27	41.1
2.00	115	4.1	0.307	1.50	0.20	11.5	7.05	78.5
2.25	129	5.1	0.414	1.69	0.25	13.0	10.74	135.1
2.50	143	6.2	0.535	1.88	0.29	14.3	15.34	213.4
2.75	158	7.2	0.666	2.06	0.32	15.6	20.76	313.7
3.00	172	8.4	0.804	2.25	0.36	16.7	26.80	433.4
3.25	186	9.5	0.944	2.44	0.39	17.6	33.23	567.1
3.50	201	10.6	1.083	2.63	0.41	18.3	39.74	707.1
3.75	215	11.7	1.215	2.81	0.43	18.9	45.99	844.2
4.00	229	12.7	1.338	3.00	0.45	19.3	51.70	969.0
4.25	244	13.7	1.447	3.19	0.45	19.6	56.59	1073.2
4.50	258	14.7	1.541	3.38	0.46	19.6	60.46	1151.0
4.75	272	15.5	1.617	3.56	0.45	19.6	63.23	1199.0
5.00	286	16.2	1.676	3.75	0.45	19.3	64.86	1217.3
5.25	301	16.8	1.718	3.94	0.44	19.0	65.44	1208.8
5.50	315	17.3	1.745	4.13	0.42	18.7	65.12	1178.4
5.75	329	17.7	1.760	4.31	0.41	18.2	64.12	1132.7
6.00	344	17.9	1.766	4.50	0.39	17.7	62.67	1078.7
6.25	358	18.0	1.767	4.69	0.38	17.3	61.05	1022.9
3.14	180	9.0	0.881	2.35	0.37	17.2	30.31	505.8
3.52	201	10.7	1.092	2.64	0.41	18.4	40.17	716.7

Down Drain Feature: 1C1
Flow Through Circular Pipe

Number of Pipes, N=	2	
Diameter of pipe, D=	18	inches
Longitudinal Slope, So=	0.040	ft/ft
Manning's n=	0.009	
Density of flowing liquid, rho=	1.94	slugs/ft^3
Peak Discharge, Q_{25}=	44.0	cfs
Peak Discharge, Q_{100}=	58.3	cfs

Theta radians	Theta degrees	Depth of Flow y inches	Area of Flow A ft^2	Wetted Perimeter P ft	Hydraulic Radius R ft	Average Velocity V ft/s	Discharge Q=A*V cfs	Force F lbf
0.00	0	0.0	0.000	0.00		0.0	0.00	0.0
0.25	14	0.1	0.001	0.19	0.00	0.8	0.00	0.0
0.50	29	0.3	0.006	0.38	0.02	2.0	0.02	0.0
0.75	43	0.6	0.019	0.56	0.03	3.5	0.13	0.5
1.00	57	1.1	0.045	0.75	0.06	5.0	0.45	2.2
1.25	72	1.7	0.085	0.94	0.09	6.7	1.13	7.3
1.50	86	2.4	0.141	1.13	0.13	8.3	2.35	18.9
1.75	100	3.2	0.215	1.31	0.16	9.9	4.27	41.1
2.00	115	4.1	0.307	1.50	0.20	11.5	7.05	78.5
2.25	129	5.1	0.414	1.69	0.25	13.0	10.74	135.1
2.50	143	6.2	0.535	1.88	0.29	14.3	15.34	213.4
2.75	158	7.2	0.666	2.06	0.32	15.6	20.76	313.7
3.00	172	8.4	0.804	2.25	0.36	16.7	26.80	433.4
3.25	186	9.5	0.944	2.44	0.39	17.6	33.23	567.1
3.50	201	10.6	1.083	2.63	0.41	18.3	39.74	707.1
3.75	215	11.7	1.215	2.81	0.43	18.9	45.99	844.2
4.00	229	12.7	1.338	3.00	0.45	19.3	51.70	969.0
4.25	244	13.7	1.447	3.19	0.45	19.6	56.59	1073.2
4.50	258	14.7	1.541	3.38	0.46	19.6	60.46	1151.0
4.75	272	15.5	1.617	3.56	0.45	19.6	63.23	1199.0
5.00	286	16.2	1.676	3.75	0.45	19.3	64.86	1217.3
5.25	301	16.8	1.718	3.94	0.44	19.0	65.44	1208.8
5.50	315	17.3	1.745	4.13	0.42	18.7	65.12	1178.4
5.75	329	17.7	1.760	4.31	0.41	18.2	64.12	1132.7
6.00	344	17.9	1.766	4.50	0.39	17.7	62.67	1078.7
6.25	358	18.0	1.767	4.69	0.38	17.3	61.05	1022.9
3.67	210	11.4	1.175	2.75	0.43	18.8	44.10	802.7
4.36	250	14.2	1.490	3.27	0.46	19.6	58.44	1111.6

Down Drain Feature: 2A1
Flow Through Circular Pipe

Number of Pipes, N=	3	
Diameter of pipe, D=	18	inches
Longitudinal Slope, So=	0.040	ft/ft
Manning's n=	0.009	
Density of flowing liquid, rho=	1.94	slugs/ft^3
Peak Discharge, Q_{25}=	56.1	cfs
Peak Discharge, Q_{100}=	74.2	cfs

Theta radians	Theta degrees	Depth of Flow y inches	Area of Flow A ft^2	Wetted Perimeter P ft	Hydraulic Radius R ft	Average Velocity V ft/s	Discharge Q=A*V cfs	Force F lbf
0.00	0	0.0	0.000	0.00		0.0	0.00	0.0
0.25	14	0.1	0.001	0.19	0.00	0.8	0.00	0.0
0.50	29	0.3	0.006	0.38	0.02	2.0	0.04	0.0
0.75	43	0.6	0.019	0.56	0.03	3.5	0.20	0.5
1.00	57	1.1	0.045	0.75	0.06	5.0	0.67	2.2
1.25	72	1.7	0.085	0.94	0.09	6.7	1.69	7.3
1.50	86	2.4	0.141	1.13	0.13	8.3	3.52	18.9
1.75	100	3.2	0.215	1.31	0.16	9.9	6.41	41.1
2.00	115	4.1	0.307	1.50	0.20	11.5	10.57	78.5
2.25	129	5.1	0.414	1.69	0.25	13.0	16.11	135.1
2.50	143	6.2	0.535	1.88	0.29	14.3	23.01	213.4
2.75	158	7.2	0.666	2.06	0.32	15.6	31.13	313.7
3.00	172	8.4	0.804	2.25	0.36	16.7	40.21	433.4
3.25	186	9.5	0.944	2.44	0.39	17.6	49.85	567.1
3.50	201	10.6	1.083	2.63	0.41	18.3	59.61	707.1
3.75	215	11.7	1.215	2.81	0.43	18.9	68.99	844.2
4.00	229	12.7	1.338	3.00	0.45	19.3	77.55	969.0
4.25	244	13.7	1.447	3.19	0.45	19.6	84.88	1073.2
4.50	258	14.7	1.541	3.38	0.46	19.6	90.70	1151.0
4.75	272	15.5	1.617	3.56	0.45	19.6	94.84	1199.0
5.00	286	16.2	1.676	3.75	0.45	19.3	97.29	1217.3
5.25	301	16.8	1.718	3.94	0.44	19.0	98.16	1208.8
5.50	315	17.3	1.745	4.13	0.42	18.7	97.68	1178.4
5.75	329	17.7	1.760	4.31	0.41	18.2	96.17	1132.7
6.00	344	17.9	1.766	4.50	0.39	17.7	94.01	1078.7
6.25	358	18.0	1.767	4.69	0.38	17.3	91.58	1022.9
3.41	195	10.2	1.033	2.56	0.40	18.1	56.11	656.7
3.90	224	12.3	1.292	2.93	0.44	19.2	74.38	923.1

Down Drain Feature: 2A2
Flow Through Circular Pipe

Number of Pipes, N= 2
Diameter of pipe, D= 18 inches
Longitudinal Slope, So= 0.040 ft/ft
Manning's n= 0.009
Density of flowing liquid, rho= 1.94 slugs/ft^3
Peak Discharge, Q_{25} = 24.8 cfs
Peak Discharge, Q_{100} = 32.8 cfs

Theta radians	Theta degrees	Depth of Flow y inches	Area of Flow A ft^2	Wetted Perimeter P ft	Hydraulic Radius R ft	Average Velocity V ft/s	Discharge Q=A*V cfs	Force F lbf
0.00	0	0.0	0.000	0.00		0.0	0.00	0.0
0.25	14	0.1	0.001	0.19	0.00	0.8	0.00	0.0
0.50	29	0.3	0.006	0.38	0.02	2.0	0.02	0.0
0.75	43	0.6	0.019	0.56	0.03	3.5	0.13	0.5
1.00	57	1.1	0.045	0.75	0.06	5.0	0.45	2.2
1.25	72	1.7	0.085	0.94	0.09	6.7	1.13	7.3
1.50	86	2.4	0.141	1.13	0.13	8.3	2.35	18.9
1.75	100	3.2	0.215	1.31	0.16	9.9	4.27	41.1
2.00	115	4.1	0.307	1.50	0.20	11.5	7.05	78.5
2.25	129	5.1	0.414	1.69	0.25	13.0	10.74	135.1
2.50	143	6.2	0.535	1.88	0.29	14.3	15.34	213.4
2.75	158	7.2	0.666	2.06	0.32	15.6	20.76	313.7
3.00	172	8.4	0.804	2.25	0.36	16.7	26.80	433.4
3.25	186	9.5	0.944	2.44	0.39	17.6	33.23	567.1
3.50	201	10.6	1.083	2.63	0.41	18.3	39.74	707.1
3.75	215	11.7	1.215	2.81	0.43	18.9	45.99	844.2
4.00	229	12.7	1.338	3.00	0.45	19.3	51.70	969.0
4.25	244	13.7	1.447	3.19	0.45	19.6	56.59	1073.2
4.50	258	14.7	1.541	3.38	0.46	19.6	60.46	1151.0
4.75	272	15.5	1.617	3.56	0.45	19.6	63.23	1199.0
5.00	286	16.2	1.676	3.75	0.45	19.3	64.86	1217.3
5.25	301	16.8	1.718	3.94	0.44	19.0	65.44	1208.8
5.50	315	17.3	1.745	4.13	0.42	18.7	65.12	1178.4
5.75	329	17.7	1.760	4.31	0.41	18.2	64.12	1132.7
6.00	344	17.9	1.766	4.50	0.39	17.7	62.67	1078.7
6.25	358	18.0	1.767	4.69	0.38	17.3	61.05	1022.9
2.92	167	8.0	0.758	2.19	0.35	16.3	24.74	391.9
3.23	185	9.4	0.935	2.43	0.39	17.5	32.82	558.5

Down Drain Feature: 2C1
Flow Through Circular Pipe

Number of Pipes, N=	5	
Diameter of pipe, D=	18	inches
Longitudinal Slope, So=	0.040	ft/ft
Manning's n=	0.009	
Density of flowing liquid, rho=	1.94	slugs/ft^3
Peak Discharge, Q_{25}=	100.5	cfs
Peak Discharge, Q_{100}=	133.0	cfs

Theta radians	Theta degrees	Depth of Flow y inches	Area of Flow A ft^2	Wetted Perimeter P ft	Hydraulic Radius R ft	Average Velocity V ft/s	Discharge Q=A*V cfs	Force F lbf
0.00	0	0.0	0.000	0.00		0.0	0.00	0.0
0.25	14	0.1	0.001	0.19	0.00	0.8	0.00	0.0
0.50	29	0.3	0.006	0.38	0.02	2.0	0.06	0.0
0.75	43	0.6	0.019	0.56	0.03	3.5	0.33	0.5
1.00	57	1.1	0.045	0.75	0.06	5.0	1.12	2.2
1.25	72	1.7	0.085	0.94	0.09	6.7	2.82	7.3
1.50	86	2.4	0.141	1.13	0.13	8.3	5.86	18.9
1.75	100	3.2	0.215	1.31	0.16	9.9	10.69	41.1
2.00	115	4.1	0.307	1.50	0.20	11.5	17.62	78.5
2.25	129	5.1	0.414	1.69	0.25	13.0	26.85	135.1
2.50	143	6.2	0.535	1.88	0.29	14.3	38.35	213.4
2.75	158	7.2	0.666	2.06	0.32	15.6	51.89	313.7
3.00	172	8.4	0.804	2.25	0.36	16.7	67.01	433.4
3.25	186	9.5	0.944	2.44	0.39	17.6	83.08	567.1
3.50	201	10.6	1.083	2.63	0.41	18.3	99.34	707.1
3.75	215	11.7	1.215	2.81	0.43	18.9	114.99	844.2
4.00	229	12.7	1.338	3.00	0.45	19.3	129.25	969.0
4.25	244	13.7	1.447	3.19	0.45	19.6	141.47	1073.2
4.50	258	14.7	1.541	3.38	0.46	19.6	151.16	1151.0
4.75	272	15.5	1.617	3.56	0.45	19.6	158.07	1199.0
5.00	286	16.2	1.676	3.75	0.45	19.3	162.15	1217.3
5.25	301	16.8	1.718	3.94	0.44	19.0	163.60	1208.8
5.50	315	17.3	1.745	4.13	0.42	18.7	162.80	1178.4
5.75	329	17.7	1.760	4.31	0.41	18.2	160.29	1132.7
6.00	344	17.9	1.766	4.50	0.39	17.7	156.68	1078.7
6.25	358	18.0	1.767	4.69	0.38	17.3	152.63	1022.9
3.52	202	10.7	1.093	2.64	0.41	18.4	100.53	717.7
4.08	234	13.1	1.373	3.06	0.45	19.4	133.28	1004.0

Down Drain Feature: 2E1
Flow Through Circular Pipe

Number of Pipes, N= 3
Diameter of pipe, D= 18 inches
Longitudinal Slope, So= 0.040 ft/ft
Manning's n= 0.009
Density of flowing liquid, rho= 1.94 slugs/ft^3
Peak Discharge, Q_{25}= 51.0 cfs
Peak Discharge, Q_{100}= 67.5 cfs

Theta radians	Theta degrees	Depth of Flow y inches	Area of Flow A ft^2	Wetted Perimeter P ft	Hydraulic Radius R ft	Average Velocity V ft/s	Discharge Q=A*V cfs	Force F lbf
0.00	0	0.0	0.000	0.00		0.0	0.00	0.0
0.25	14	0.1	0.001	0.19	0.00	0.8	0.00	0.0
0.50	29	0.3	0.006	0.38	0.02	2.0	0.04	0.0
0.75	43	0.6	0.019	0.56	0.03	3.5	0.20	0.5
1.00	57	1.1	0.045	0.75	0.06	5.0	0.67	2.2
1.25	72	1.7	0.085	0.94	0.09	6.7	1.69	7.3
1.50	86	2.4	0.141	1.13	0.13	8.3	3.52	18.9
1.75	100	3.2	0.215	1.31	0.16	9.9	6.41	41.1
2.00	115	4.1	0.307	1.50	0.20	11.5	10.57	78.5
2.25	129	5.1	0.414	1.69	0.25	13.0	16.11	135.1
2.50	143	6.2	0.535	1.88	0.29	14.3	23.01	213.4
2.75	158	7.2	0.666	2.06	0.32	15.6	31.13	313.7
3.00	172	8.4	0.804	2.25	0.36	16.7	40.21	433.4
3.25	186	9.5	0.944	2.44	0.39	17.6	49.85	567.1
3.50	201	10.6	1.083	2.63	0.41	18.3	59.61	707.1
3.75	215	11.7	1.215	2.81	0.43	18.9	68.99	844.2
4.00	229	12.7	1.338	3.00	0.45	19.3	77.55	969.0
4.25	244	13.7	1.447	3.19	0.45	19.6	84.88	1073.2
4.50	258	14.7	1.541	3.38	0.46	19.6	90.70	1151.0
4.75	272	15.5	1.617	3.56	0.45	19.6	94.84	1199.0
5.00	286	16.2	1.676	3.75	0.45	19.3	97.29	1217.3
5.25	301	16.8	1.718	3.94	0.44	19.0	98.16	1208.8
5.50	315	17.3	1.745	4.13	0.42	18.7	97.68	1178.4
5.75	329	17.7	1.760	4.31	0.41	18.2	96.17	1132.7
6.00	344	17.9	1.766	4.50	0.39	17.7	94.01	1078.7
6.25	358	18.0	1.767	4.69	0.38	17.3	91.58	1022.9
3.28	188	9.6	0.961	2.46	0.39	17.7	51.04	584.2
3.71	213	11.5	1.195	2.78	0.43	18.8	67.59	823.8

Down Drain Feature: 2F1
Flow Through Circular Pipe

Number of Pipes, N= 5
Diameter of pipe, D= 18 inches
Longitudinal Slope, So= 0.040 ft/ft
Manning's n= 0.009
Density of flowing liquid, rho= 1.94 slugs/ft^3
Peak Discharge, Q_{25}= 95.2 cfs
Peak Discharge, Q_{100}= 126.0 cfs

Theta radians	Theta degrees	Depth of Flow y inches	Area of Flow A ft^2	Wetted Perimeter P ft	Hydraulic Radius R ft	Average Velocity V ft/s	Discharge Q=A*V cfs	Force F lbf
0.00	0	0.0	0.000	0.00		0.0	0.00	0.0
0.25	14	0.1	0.001	0.19	0.00	0.8	0.00	0.0
0.50	29	0.3	0.006	0.38	0.02	2.0	0.06	0.0
0.75	43	0.6	0.019	0.56	0.03	3.5	0.33	0.5
1.00	57	1.1	0.045	0.75	0.06	5.0	1.12	2.2
1.25	72	1.7	0.085	0.94	0.09	6.7	2.82	7.3
1.50	86	2.4	0.141	1.13	0.13	8.3	5.86	18.9
1.75	100	3.2	0.215	1.31	0.16	9.9	10.69	41.1
2.00	115	4.1	0.307	1.50	0.20	11.5	17.62	78.5
2.25	129	5.1	0.414	1.69	0.25	13.0	26.85	135.1
2.50	143	6.2	0.535	1.88	0.29	14.3	38.35	213.4
2.75	158	7.2	0.666	2.06	0.32	15.6	51.89	313.7
3.00	172	8.4	0.804	2.25	0.36	16.7	67.01	433.4
3.25	186	9.5	0.944	2.44	0.39	17.6	83.08	567.1
3.50	201	10.6	1.083	2.63	0.41	18.3	99.34	707.1
3.75	215	11.7	1.215	2.81	0.43	18.9	114.99	844.2
4.00	229	12.7	1.338	3.00	0.45	19.3	129.25	969.0
4.25	244	13.7	1.447	3.19	0.45	19.6	141.47	1073.2
4.50	258	14.7	1.541	3.38	0.46	19.6	151.16	1151.0
4.75	272	15.5	1.617	3.56	0.45	19.6	158.07	1199.0
5.00	286	16.2	1.676	3.75	0.45	19.3	162.15	1217.3
5.25	301	16.8	1.718	3.94	0.44	19.0	163.60	1208.8
5.50	315	17.3	1.745	4.13	0.42	18.7	162.80	1178.4
5.75	329	17.7	1.760	4.31	0.41	18.2	160.29	1132.7
6.00	344	17.9	1.766	4.50	0.39	17.7	156.68	1078.7
6.25	358	18.0	1.767	4.69	0.38	17.3	152.63	1022.9
3.44	197	10.3	1.048	2.58	0.41	18.2	95.24	671.6
3.94	226	12.5	1.311	2.96	0.44	19.3	126.18	942.5

Down Drain Feature: 2F2
Flow Through Circular Pipe

Number of Pipes, N= 6
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} = 117.4 cfs
Peak Discharge, Q_{100} = 155.4 cfs

Theta radians	Theta degrees	Depth of Flow y inches	Area of Flow A ft^2	Wetted Perimeter P ft	Hydraulic Radius R ft	Average Velocity V ft/s	Discharge Q=A*V cfs	Force F lbf
0.00	0	0.0	0.000	0.00		0.0	0.00	0.0
0.25	14	0.1	0.001	0.19	0.00	0.8	0.00	0.0
0.50	29	0.3	0.006	0.38	0.02	2.0	0.07	0.0
0.75	43	0.6	0.019	0.56	0.03	3.5	0.40	0.5
1.00	57	1.1	0.045	0.75	0.06	5.0	1.35	2.2
1.25	72	1.7	0.085	0.94	0.09	6.7	3.38	7.3
1.50	86	2.4	0.141	1.13	0.13	8.3	7.04	18.9
1.75	100	3.2	0.215	1.31	0.16	9.9	12.82	41.1
2.00	115	4.1	0.307	1.50	0.20	11.5	21.14	78.5
2.25	129	5.1	0.414	1.69	0.25	13.0	32.21	135.1
2.50	143	6.2	0.535	1.88	0.29	14.3	46.02	213.4
2.75	158	7.2	0.666	2.06	0.32	15.6	62.27	313.7
3.00	172	8.4	0.804	2.25	0.36	16.7	80.41	433.4
3.25	186	9.5	0.944	2.44	0.39	17.6	99.70	567.1
3.50	201	10.6	1.083	2.63	0.41	18.3	119.21	707.1
3.75	215	11.7	1.215	2.81	0.43	18.9	137.98	844.2
4.00	229	12.7	1.338	3.00	0.45	19.3	155.10	969.0
4.25	244	13.7	1.447	3.19	0.45	19.6	169.76	1073.2
4.50	258	14.7	1.541	3.38	0.46	19.6	181.39	1151.0
4.75	272	15.5	1.617	3.56	0.45	19.6	189.68	1199.0
5.00	286	16.2	1.676	3.75	0.45	19.3	194.58	1217.3
5.25	301	16.8	1.718	3.94	0.44	19.0	196.32	1208.8
5.50	315	17.3	1.745	4.13	0.42	18.7	195.36	1178.4
5.75	329	17.7	1.760	4.31	0.41	18.2	192.35	1132.7
6.00	344	17.9	1.766	4.50	0.39	17.7	188.02	1078.7
6.25	358	18.0	1.767	4.69	0.38	17.3	183.15	1022.9
3.48	199	10.5	1.071	2.61	0.41	18.3	117.49	694.8
4.01	230	12.8	1.341	3.00	0.45	19.3	155.52	972.2

Appendix 3

Perimeter Reach Calculations

Design/Check: Trapezoidal/Triangular Channel

Methodology: Manning's Equation

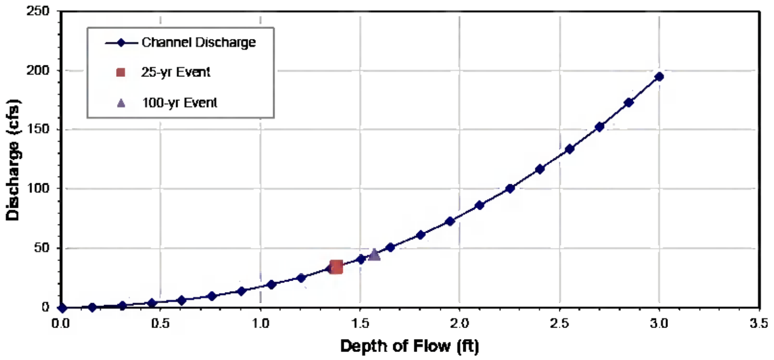
Project: Santee Cooper - Winyah Generating Station

Ditch ID: **Perimeter Reach 1A1** **Design**

Peak Discharge, Q_{25} =	34.49	cfs (25-yr Event)
Peak Discharge, Q_{100} =	45.62	cfs (100-yr Event)
Bottom Width, B =	3.00	ft
Left Side Slope, Z_1 =	3.00	horizontal : 1 vertical
Right Side Slope, Z_2 =	3.00	horizontal : 1 vertical
Channel Depth, Y =	3.00	ft
Top Width, T =	21.0	ft
Manning's Roughness Coeff., n =	0.027	
Longitudinal Channel Slope, S_o =	0.005	ft/ft

Depth of Flow Y ft	Area of Flow A ft^2	Wetted Perimeter P ft	Hydraulic Radius R=A/P ft	Average Velocity V ft/s	Discharge (Flow Rate) Q=AV ft^3/s	Avg. Tractive Stress τ_0 lb/ft^2	Comments
0.01	0.03	3.06	0.01	0.18	0.0	0.00	
0.16	0.55	4.01	0.14	1.04	0.6	0.04	
0.31	1.21	4.95	0.24	1.53	1.9	0.08	
0.46	2.01	5.90	0.34	1.90	3.8	0.11	
0.61	2.93	6.85	0.43	2.22	6.5	0.13	
0.76	3.99	7.79	0.51	2.50	10.0	0.16	
0.91	5.19	8.74	0.59	2.76	14.3	0.19	
1.06	6.52	9.68	0.67	3.00	19.5	0.21	
1.21	7.98	10.63	0.75	3.22	25.7	0.23	
1.36	9.58	11.57	0.83	3.44	32.9	0.26	
1.51	11.31	12.52	0.90	3.65	41.2	0.28	
1.65	13.18	13.46	0.98	3.85	50.7	0.31	
1.80	15.18	14.41	1.05	4.04	61.3	0.33	
1.95	17.31	15.36	1.13	4.23	73.2	0.35	
2.10	19.58	16.30	1.20	4.41	86.3	0.37	
2.25	21.98	17.25	1.27	4.59	100.8	0.40	
2.40	24.51	18.19	1.35	4.76	116.7	0.42	
2.55	27.18	19.14	1.42	4.93	134.1	0.44	
2.70	29.99	20.08	1.49	5.10	152.9	0.47	
2.85	32.93	21.03	1.57	5.26	173.3	0.49	
3.00	36.00	21.97	1.64	5.42	195.3	0.51	
1.38	9.89	11.75	0.84	3.48	34.41	0.26	Q (25-yr Event)
1.57	12.16	12.96	0.94	3.74	45.48	0.29	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}= 34.49 cfs
Peak Discharge, Q_{100}= 45.62 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
3.55	203	**21.6**	4.438	5.32	0.83	7.8	34.52	520.8
4.13	236	**26.5**	5.580	6.19	0.90	8.2	45.72	726.7

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_{25}= 177.21 cfs
Peak Discharge, Q_{100}= 236.19 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
3.01	172	**28.0**	8.968	7.51	1.19	19.8	177.18	6791.4
3.35	192	**33.2**	11.134	8.38	1.33	21.2	236.23	9723.6

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, Q_{25}=	126.58	cfs
Peak Discharge, Q_{100}=	168.37	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
3.97	227	**37.8**	11.908	8.93	1.33	10.6	126.67	2614.0
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/ft^3
Peak Discharge, Q_{25}=	143.04	cfs
Peak Discharge, Q_{100}=	190.87	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
3.65	209	**37.5**	12.929	9.13	1.42	11.1	143.19	3076.5
4.34	249	**47.0**	16.489	10.86	1.52	11.6	191.26	4303.7

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_{25}= 36.71 cfs
Peak Discharge, Q_{100}= 48.24 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
3.93	225	**20.8**	3.626	4.91	0.74	10.1	36.77	723.2
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}= 36.71 cfs
Peak Discharge, Q_{100}= 48.24 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
3.61	207	**25.9**	6.233	6.33	0.99	5.9	36.75	420.4
4.22	242	**31.8**	7.811	7.38	1.06	6.2	48.28	579.0

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/ft^3
Peak Discharge, Q_{25}= 90.46 cfs
Peak Discharge, Q_{100}= 120.36 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
3.92	225	**33.1**	9.245	7.84	1.18	9.8	90.60	1722.3
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

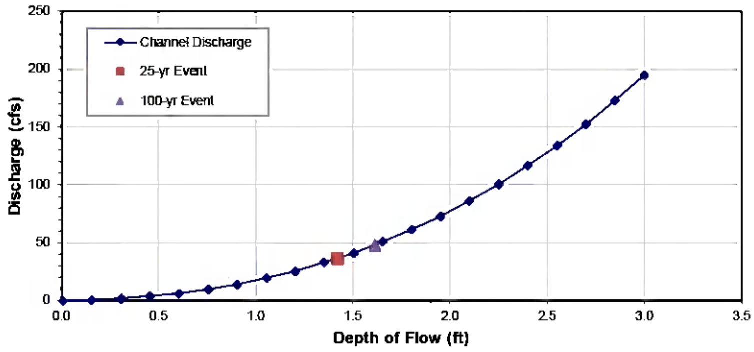
Project: Santee Cooper - Winyah Generating Station

Ditch ID: Perimeter Reach 1B3a Design

Peak Discharge, Q_{25} = 36.71 cfs (25-yr Event)
Peak Discharge, Q_{100} = 48.24 cfs (100-yr Event)
Bottom Width, B = 3.00 ft
Left Side Slope, Z_1 = 3.00 horizontal : 1 vertical
Right Side Slope, Z_2 = 3.00 horizontal : 1 vertical
Channel Depth, Y = 3.00 ft
Top Width, T = 21.0 ft
Manning's Roughness Coeff., n = 0.027
Longitudinal Channel Slope, S_o = 0.005 ft/ft

Depth, of Flow Y ft	Area of Flow A ft^2	Wetted Perimeter P ft	Hydraulic Radius R=A/P ft	Average Velocity V ft/s	Discharge (Flow Rate) Q=AV ft^3/s	Avg. Tractive Stress τ_0 lb/ft^2	Comments
0.01	0.03	3.06	0.01	0.18	0.0	0.00	
0.16	0.55	4.01	0.14	1.04	0.6	0.04	
0.31	1.21	4.95	0.24	1.53	1.9	0.08	
0.46	2.01	5.90	0.34	1.90	3.8	0.11	
0.61	2.93	6.85	0.43	2.22	6.5	0.13	
0.76	3.99	7.79	0.51	2.50	10.0	0.16	
0.91	5.19	8.74	0.59	2.76	14.3	0.19	
1.06	6.52	9.68	0.67	3.00	19.5	0.21	
1.21	7.98	10.63	0.75	3.22	25.7	0.23	
1.36	9.58	11.57	0.83	3.44	32.9	0.26	
1.51	11.31	12.52	0.90	3.65	41.2	0.28	
1.65	13.18	13.46	0.98	3.85	50.7	0.31	
1.80	15.18	14.41	1.05	4.04	61.3	0.33	
1.95	17.31	15.36	1.13	4.23	73.2	0.35	
2.10	19.58	16.30	1.20	4.41	86.3	0.37	
2.25	21.98	17.25	1.27	4.59	100.8	0.40	
2.40	24.51	18.19	1.35	4.76	116.7	0.42	
2.55	27.18	19.14	1.42	4.93	134.1	0.44	
2.70	29.99	20.08	1.49	5.10	152.9	0.47	
2.85	32.93	21.03	1.57	5.26	173.3	0.49	
3.00	36.00	21.97	1.64	5.42	195.3	0.51	
1.42	10.35	12.00	0.86	3.53	36.57	0.27	Q (25-yr Event)
1.62	12.68	13.22	0.96	3.80	48.13	0.30	Q (100-yr Event)

Discharge versus Depth Relationship



Design/Check: Trapezoidal/Triangular Channel

Methodology: Manning's Equation

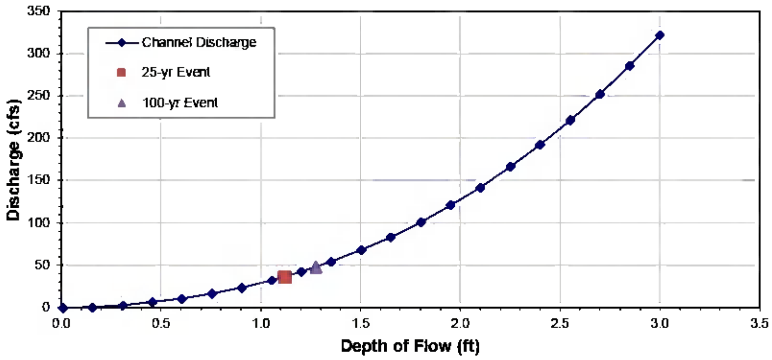
Project: Santee Cooper - Winyah Generating Station

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

Peak Discharge, Q_{25} =	36.71	cfs (25-yr Event)
Peak Discharge, Q_{100} =	48.24	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 ft²	Wetted Perimeter P ft	Hydraulic Radius R=A/P ft	Average Velocity V ft/s	Discharge (Flow Rate) Q=AV ft³/s	Avg. Tractive Stress τ_0 lb/ft²	Comments
0.01	0.03	3.06	0.01	0.30	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	
1.12	7.14	10.10	0.71	5.11	36.51	0.60	Q (25-yr Event)
1.28	8.74	11.09	0.79	5.49	48.02	0.67	Q (100-yr Event)

Discharge versus Depth Relationship



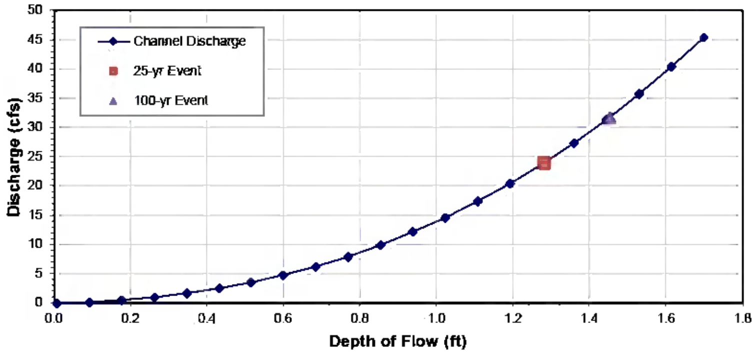
Design/Check: Trapezoidal/Triangular Channel

Methodology: Manning's Equation
Project: Santee Cooper - Winyah Generating Station
Ditch ID: **Perimeter Reach 1C1** **Design**

Peak Discharge, Q_{25} = 23.99 cfs (25-yr Event)
Peak Discharge, Q_{100} = 31.78 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 = 0.03
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	0.02	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	
1.28	7.50	10.11	0.74	3.20	23.98	0.23	Q (25-yr Event)
1.45	9.25	11.19	0.83	3.44	31.77	0.26	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_{25}= 23.99 cfs
Peak Discharge, Q_{100}= 31.78 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
3.76	216	**19.6**	3.397	4.71	0.72	7.1	24.00	329.0
4.58	263	**24.9**	4.355	5.73	0.76	7.3	31.85	451.9

PIPE 1C1b

Diameter of pipe, D=	48	inches
Longitudinal Slope, So=	0.008	ft/ft
Manning's n=	0.012	
Density of flowing liquid, rho=	1.94	slugs/ft^3
Peak Discharge, Q_{25}=	72.21	cfs
Peak Discharge, Q_{100}=	95.53	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
3.18	182	**24.5**	6.444	6.36	1.01	11.2	72.16	1567.8
3.58	205	**29.2**	7.994	7.15	1.12	12.0	95.62	2218.8

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}=	72.21	cfs
Peak Discharge, Q_{100}=	95.53	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
3.05	175	**22.9**	5.927	6.10	0.97	12.2	72.15	1704.1
3.40	195	**27.1**	7.328	6.81	1.08	13.0	95.56	2417.4

Design/Check: Trapezoidal/Triangular Channel

Methodology: Manning's Equation

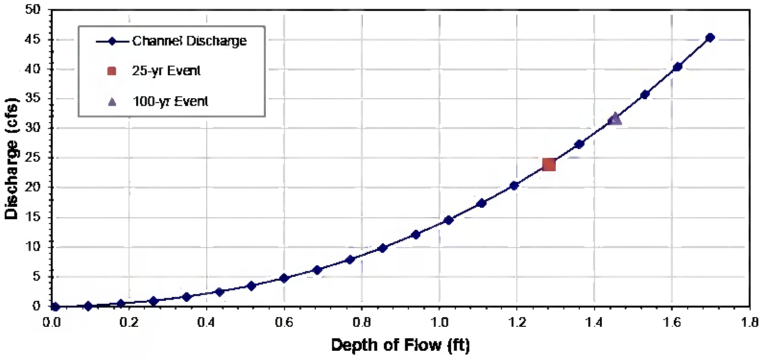
Project: Santee Cooper - Winyah Generating Station

Ditch ID: Perimeter Reach 1C2 Design

Peak Discharge, Q_{25} =	23.99	cfs (25-yr Event)
Peak Discharge, Q_{100} =	31.78	cfs (100-yr Event)
Bottom Width, B =	2.00	ft
Left Side Slope, Z_1 =	3.00	horizontal :1 vertical
Right Side Slope, Z_2 =	3.00	horizontal :1 vertical
Channel Depth, Y =	1.70	ft
Top Width, T =	12.2	ft
Manning's Roughness Coeff., n =	0.027	
Longitudinal Channel Slope, S_o =	0.005	ft/ft

Depth of Flow Y ft	Area of Flow A 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	
1.28	7.50	10.11	0.74	3.20	23.98	0.23	Q (25-yr Event)
1.45	9.25	11.19	0.83	3.44	31.77	0.26	Q (100-yr Event)

Discharge versus Depth Relationship



Design/Check: Trapezoidal/Triangular Channel

Methodology: Manning's Equation

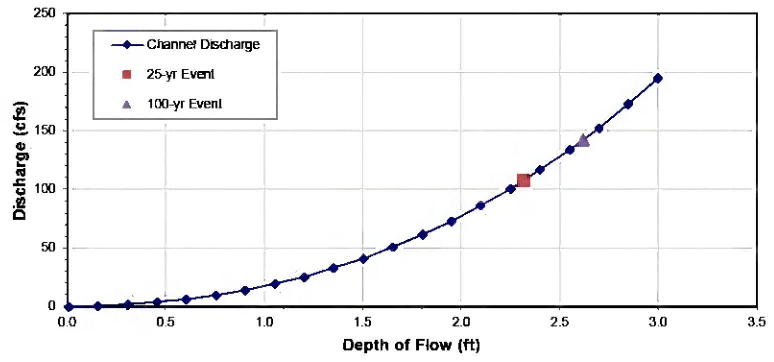
Project: Santee Cooper - Winyah Generating Station

Ditch ID: **Perimeter Reach 2A1** **Design**

Peak Discharge, Q_{25} =	107.76	cfs (25-yr Event)
Peak Discharge, Q_{100} =	142.91	cfs (100-yr Event)
Bottom Width, B =	3.00	ft
Left Side Slope, Z_1 =	3.00	horizontal :1 vertical
Right Side Slope, Z_2 =	3.00	horizontal :1 vertical
Channel Depth, Y =	3.00	ft
Top Width, T =	21.0	ft
Manning's Roughness Coeff., n =	0.027	
Longitudinal Channel Slope, S_o =	0.005	ft/ft

Depth of Flow Y ft	Area of Flow A ft^2	Wetted Perimeter P ft	Hydraulic Radius R=A/P ft	Average Velocity V ft/s	Discharge (Flow Rate) Q=AV ft^3/s	Avg. Tractive Stress τ_0 lb/ft^2	Comments
0.01	0.03	3.06	0.01	0.18	0.0	0.00	
0.16	0.55	4.01	0.14	1.04	0.6	0.04	
0.31	1.21	4.95	0.24	1.53	1.9	0.08	
0.46	2.01	5.90	0.34	1.90	3.8	0.11	
0.61	2.93	6.85	0.43	2.22	6.5	0.13	
0.76	3.99	7.79	0.51	2.50	10.0	0.16	
0.91	5.19	8.74	0.59	2.76	14.3	0.19	
1.06	6.52	9.68	0.67	3.00	19.5	0.21	
1.21	7.98	10.63	0.75	3.22	25.7	0.23	
1.36	9.58	11.57	0.83	3.44	32.9	0.26	
1.51	11.31	12.52	0.90	3.65	41.2	0.28	
1.65	13.18	13.46	0.98	3.85	50.7	0.31	
1.80	15.18	14.41	1.05	4.04	61.3	0.33	
1.95	17.31	15.36	1.13	4.23	73.2	0.35	
2.10	19.58	16.30	1.20	4.41	86.3	0.37	
2.25	21.98	17.25	1.27	4.59	100.8	0.40	
2.40	24.51	18.19	1.35	4.76	116.7	0.42	
2.55	27.18	19.14	1.42	4.93	134.1	0.44	
2.70	29.99	20.08	1.49	5.10	152.9	0.47	
2.85	32.93	21.03	1.57	5.26	173.3	0.49	
3.00	36.00	21.97	1.64	5.42	195.3	0.51	
2.32	23.07	17.66	1.31	4.66	107.59	0.41	Q (25-yr Event)
2.62	28.48	19.58	1.45	5.01	142.72	0.45	Q (100-yr Event)

Discharge versus Depth Relationship



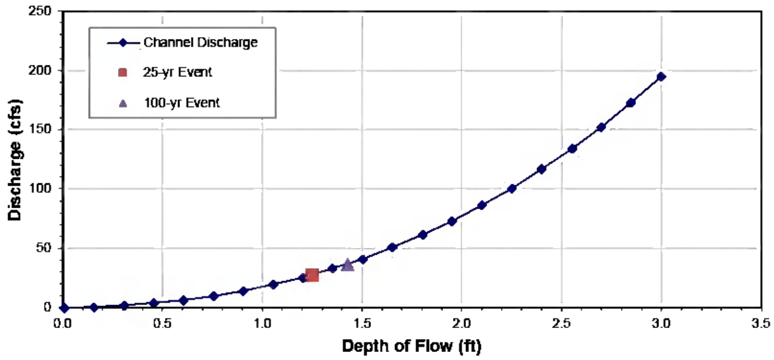
Design/Check: Trapezoidal/Triangular Channel

Methodology: Manning's Equation
Project: Santee Cooper - Winyah Generating Station
Ditch ID: **Perimeter Reach 2A2** **Design**

Peak Discharge, Q_{25} = 27.98 cfs (25-yr Event)
Peak Discharge, Q_{100} = 37.02 cfs (100-yr Event)
Bottom Width, B = 3.00 ft
Left Side Slope, Z_1 = 3.00 horizontal :1 vertical
Right Side Slope, Z_2 = 3.00 horizontal :1 vertical
Channel Depth, Y = 3.00 ft
Top Width, T = 21.0 ft
Manning's Roughness Coeff., n = 0.027
Longitudinal Channel Slope, S_o = 0.005 ft/ft

Depth of Flow Y ft	Area of Flow A ft^2	Wetted Perimeter P ft	Hydraulic Radius R=A/P ft	Average Velocity V ft/s	Discharge (Flow Rate) Q=AV ft^3/s	Avg. Tractive Stress τ_0 lb/ft^2	Comments
0.01	0.03	3.06	0.01	0.18	0.0	0.00	
0.16	0.55	4.01	0.14	1.04	0.6	0.04	
0.31	1.21	4.95	0.24	1.53	1.9	0.08	
0.46	2.01	5.90	0.34	1.90	3.8	0.11	
0.61	2.93	6.85	0.43	2.22	6.5	0.13	
0.76	3.99	7.79	0.51	2.50	10.0	0.16	
0.91	5.19	8.74	0.59	2.76	14.3	0.19	
1.06	6.52	9.68	0.67	3.00	19.5	0.21	
1.21	7.98	10.63	0.75	3.22	25.7	0.23	
1.36	9.58	11.57	0.83	3.44	32.9	0.26	
1.51	11.31	12.52	0.90	3.65	41.2	0.28	
1.65	13.18	13.46	0.98	3.85	50.7	0.31	
1.80	15.18	14.41	1.05	4.04	61.3	0.33	
1.95	17.31	15.36	1.13	4.23	73.2	0.35	
2.10	19.58	16.30	1.20	4.41	86.3	0.37	
2.25	21.98	17.25	1.27	4.59	100.8	0.40	
2.40	24.51	18.19	1.35	4.76	116.7	0.42	
2.55	27.18	19.14	1.42	4.93	134.1	0.44	
2.70	29.99	20.08	1.49	5.10	152.9	0.47	
2.85	32.93	21.03	1.57	5.26	173.3	0.49	
3.00	36.00	21.97	1.64	5.42	195.3	0.51	
1.25	8.46	10.92	0.78	3.29	27.87	0.24	Q (25-yr Event)
1.43	10.41	12.04	0.86	3.54	36.88	0.27	Q (100-yr Event)

Discharge versus Depth Relationship



Design/Check: Trapezoidal/Triangular Channel

Methodology: Manning's Equation

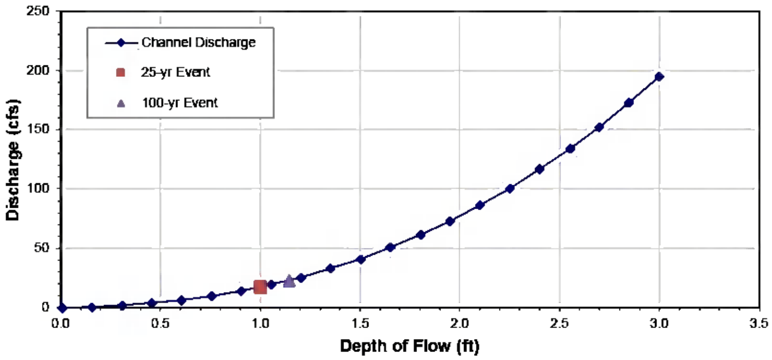
Project: Santee Cooper - Winyah Generating Station

Ditch ID: Perimeter Reach 2BA **Design**

Peak Discharge, Q_{25} = 17.62 cfs (25-yr Event)
Peak Discharge, Q_{100} = 23.32 cfs (100-yr Event)
Bottom Width, B = 3.00 ft
Left Side Slope, Z_1 = 3.00 horizontal :1 vertical
Right Side Slope, Z_2 = 3.00 horizontal :1 vertical
Channel Depth, Y = 3.00 ft
Top Width, T = 21.0 ft
Manning's Roughness Coeff., n = 0.027
Longitudinal Channel Slope, S_o = 0.005 ft/ft

Depth of Flow Y ft	Area of Flow A ft^2	Wetted Perimeter P ft	Hydraulic Radius R=A/P ft	Average Velocity V ft/s	Discharge (Flow Rate) Q=AV ft^3/s	Avg. Tractive Stress τ_0 lb/ft^2	Comments
0.01	0.03	3.06	0.01	0.18	0.0	0.00	
0.16	0.55	4.01	0.14	1.04	0.6	0.04	
0.31	1.21	4.95	0.24	1.53	1.9	0.08	
0.46	2.01	5.90	0.34	1.90	3.8	0.11	
0.61	2.93	6.85	0.43	2.22	6.5	0.13	
0.76	3.99	7.79	0.51	2.50	10.0	0.16	
0.91	5.19	8.74	0.59	2.76	14.3	0.19	
1.06	6.52	9.68	0.67	3.00	19.5	0.21	
1.21	7.98	10.63	0.75	3.22	25.7	0.23	
1.36	9.58	11.57	0.83	3.44	32.9	0.26	
1.51	11.31	12.52	0.90	3.65	41.2	0.28	
1.65	13.18	13.46	0.98	3.85	50.7	0.31	
1.80	15.18	14.41	1.05	4.04	61.3	0.33	
1.95	17.31	15.36	1.13	4.23	73.2	0.35	
2.10	19.58	16.30	1.20	4.41	86.3	0.37	
2.25	21.98	17.25	1.27	4.59	100.8	0.40	
2.40	24.51	18.19	1.35	4.76	116.7	0.42	
2.55	27.18	19.14	1.42	4.93	134.1	0.44	
2.70	29.99	20.08	1.49	5.10	152.9	0.47	
2.85	32.93	21.03	1.57	5.26	173.3	0.49	
3.00	36.00	21.97	1.64	5.42	195.3	0.51	
1.00	6.02	9.34	0.64	2.91	17.51	0.20	Q (25-yr Event)
1.15	7.40	10.26	0.72	3.14	23.20	0.22	Q (100-yr Event)

Discharge versus Depth Relationship



Design/Check: Trapezoidal/Triangular Channel

Methodology: Manning's Equation

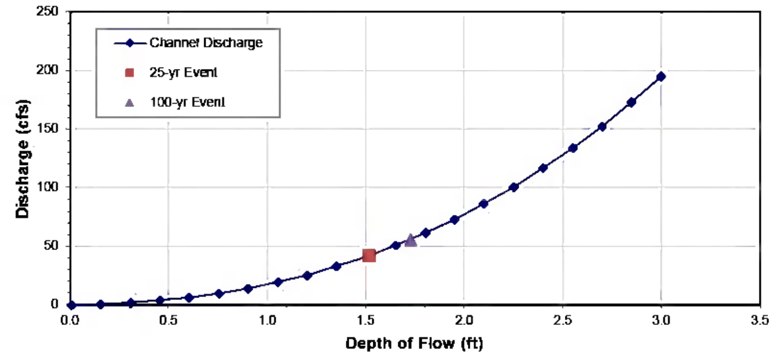
Project: Santee Cooper - Winyah Generating Station

Ditch ID: **Perimeter Reach 2BB** **Design**

Peak Discharge, Q_{25} =	42.20	cfs (25-yr Event)
Peak Discharge, Q_{100} =	55.89	cfs (100-yr Event)
Bottom Width, B =	3.00	ft
Left Side Slope, Z_1 =	3.00	horizontal :1 vertical
Right Side Slope, Z_2 =	3.00	horizontal :1 vertical
Channel Depth, Y =	3.00	ft
Top Width, T =	21.0	ft
Manning's Roughness Coeff., n =	0.027	
Longitudinal Channel Slope, S_o =	0.005	ft/ft

Depth of Flow Y ft	Area of Flow A ft^2	Wetted Perimeter P ft	Hydraulic Radius R=A/P ft	Average Velocity V ft/s	Discharge (Flow Rate) Q=AV ft^3/s	Avg. Tractive Stress τ_0 lb/ft^2	Comments
0.01	0.03	3.06	0.01	0.18	0.0	0.00	
0.16	0.55	4.01	0.14	1.04	0.6	0.04	
0.31	1.21	4.95	0.24	1.53	1.9	0.08	
0.46	2.01	5.90	0.34	1.90	3.8	0.11	
0.61	2.93	6.85	0.43	2.22	6.5	0.13	
0.76	3.99	7.79	0.51	2.50	10.0	0.16	
0.91	5.19	8.74	0.59	2.76	14.3	0.19	
1.06	6.52	9.68	0.67	3.00	19.5	0.21	
1.21	7.98	10.63	0.75	3.22	25.7	0.23	
1.36	9.58	11.57	0.83	3.44	32.9	0.26	
1.51	11.31	12.52	0.90	3.65	41.2	0.28	
1.65	13.18	13.46	0.98	3.85	50.7	0.31	
1.80	15.18	14.41	1.05	4.04	61.3	0.33	
1.95	17.31	15.36	1.13	4.23	73.2	0.35	
2.10	19.58	16.30	1.20	4.41	86.3	0.37	
2.25	21.98	17.25	1.27	4.59	100.8	0.40	
2.40	24.51	18.19	1.35	4.76	116.7	0.42	
2.55	27.18	19.14	1.42	4.93	134.1	0.44	
2.70	29.99	20.08	1.49	5.10	152.9	0.47	
2.85	32.93	21.03	1.57	5.26	173.3	0.49	
3.00	36.00	21.97	1.64	5.42	195.3	0.51	
1.52	11.49	12.61	0.91	3.67	42.15	0.28	Q (25-yr Event)
1.73	14.14	13.93	1.02	3.94	55.74	0.32	Q (100-yr Event)

Discharge versus Depth Relationship



Design/Check: Trapezoidal/Triangular Channel

Methodology: Manning's Equation

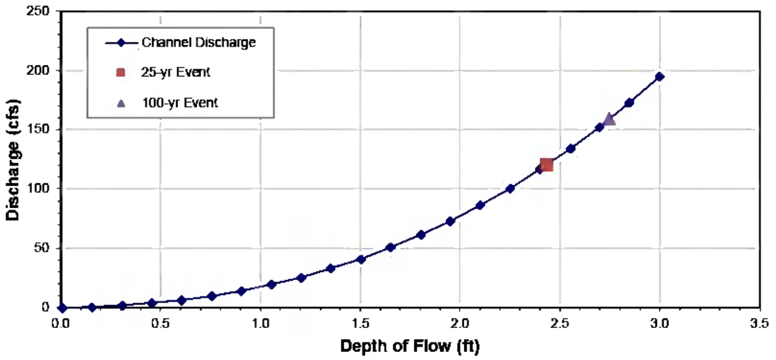
Project: Santee Cooper - Winyah Generating Station

Ditch ID: **Perimeter Reach 2C1** **Design**

Peak Discharge, Q_{25} =	120.38	cfs (25-yr Event)
Peak Discharge, Q_{100} =	159.56	cfs (100-yr Event)
Bottom Width, B =	3.00	ft
Left Side Slope, Z_1 =	3.00	horizontal :1 vertical
Right Side Slope, Z_2 =	3.00	horizontal :1 vertical
Channel Depth, Y =	3.00	ft
Top Width, T =	21.0	ft
Manning's Roughness Coeff., n =	0.027	
Longitudinal Channel Slope, S_o =	0.005	ft/ft

Depth of Flow Y ft	Area of Flow A ft^2	Wetted Perimeter P ft	Hydraulic Radius R=A/P ft	Average Velocity V ft/s	Discharge (Flow Rate) Q=AV ft^3/s	Avg. Tractive Stress τ_0 lb/ft^2	Comments
0.01	0.03	3.06	0.01	0.18	0.0	0.00	
0.16	0.55	4.01	0.14	1.04	0.6	0.04	
0.31	1.21	4.95	0.24	1.53	1.9	0.08	
0.46	2.01	5.90	0.34	1.90	3.8	0.11	
0.61	2.93	6.85	0.43	2.22	6.5	0.13	
0.76	3.99	7.79	0.51	2.50	10.0	0.16	
0.91	5.19	8.74	0.59	2.76	14.3	0.19	
1.06	6.52	9.68	0.67	3.00	19.5	0.21	
1.21	7.98	10.63	0.75	3.22	25.7	0.23	
1.36	9.58	11.57	0.83	3.44	32.9	0.26	
1.51	11.31	12.52	0.90	3.65	41.2	0.28	
1.65	13.18	13.46	0.98	3.85	50.7	0.31	
1.80	15.18	14.41	1.05	4.04	61.3	0.33	
1.95	17.31	15.36	1.13	4.23	73.2	0.35	
2.10	19.58	16.30	1.20	4.41	86.3	0.37	
2.25	21.98	17.25	1.27	4.59	100.8	0.40	
2.40	24.51	18.19	1.35	4.76	116.7	0.42	
2.55	27.18	19.14	1.42	4.93	134.1	0.44	
2.70	29.99	20.08	1.49	5.10	152.9	0.47	
2.85	32.93	21.03	1.57	5.26	173.3	0.49	
3.00	36.00	21.97	1.64	5.42	195.3	0.51	
2.43	25.07	18.39	1.36	4.80	120.26	0.43	Q (25-yr Event)
2.75	30.93	20.39	1.52	5.15	159.39	0.47	Q (100-yr Event)

Discharge versus Depth Relationship



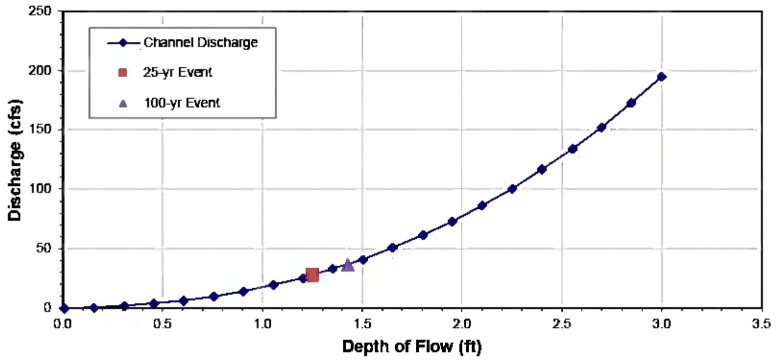
Design/Check: Trapezoidal/Triangular Channel

Methodology: Manning's Equation
Project: Santee Cooper - Winyah Generating Station
Ditch ID: **Perimeter Reach 2DA** **Design**

Peak Discharge, Q_{25} =	28.06	cfs (25-yr Event)
Peak Discharge, Q_{100} =	37.14	cfs (100-yr Event)
Bottom Width, B =	3.00	ft
Left Side Slope, Z_1 =	3.00	horizontal :1 vertical
Right Side Slope, Z_2 =	3.00	horizontal :1 vertical
Channel Depth, Y =	3.00	ft
Top Width, T =	21.0	ft
Manning's Roughness Coeff., n =	0.027	
Longitudinal Channel Slope, S_o =	0.005	ft/ft

Depth of Flow Y ft	Area of Flow A ft^2	Wetted Perimeter P ft	Hydraulic Radius R=A/P ft	Average Velocity V ft/s	Discharge (Flow Rate) Q=AV ft^3/s	Avg. Tractive Stress τ_0 lb/ft^2	Comments
0.01	0.03	3.06	0.01	0.18	0.0	0.00	
0.16	0.55	4.01	0.14	1.04	0.6	0.04	
0.31	1.21	4.95	0.24	1.53	1.9	0.08	
0.46	2.01	5.90	0.34	1.90	3.8	0.11	
0.61	2.93	6.85	0.43	2.22	6.5	0.13	
0.76	3.99	7.79	0.51	2.50	10.0	0.16	
0.91	5.19	8.74	0.59	2.76	14.3	0.19	
1.06	6.52	9.68	0.67	3.00	19.5	0.21	
1.21	7.98	10.63	0.75	3.22	25.7	0.23	
1.36	9.58	11.57	0.83	3.44	32.9	0.26	
1.51	11.31	12.52	0.90	3.65	41.2	0.28	
1.65	13.18	13.46	0.98	3.85	50.7	0.31	
1.80	15.18	14.41	1.05	4.04	61.3	0.33	
1.95	17.31	15.36	1.13	4.23	73.2	0.35	
2.10	19.58	16.30	1.20	4.41	86.3	0.37	
2.25	21.98	17.25	1.27	4.59	100.8	0.40	
2.40	24.51	18.19	1.35	4.76	116.7	0.42	
2.55	27.18	19.14	1.42	4.93	134.1	0.44	
2.70	29.99	20.08	1.49	5.10	152.9	0.47	
2.85	32.93	21.03	1.57	5.26	173.3	0.49	
3.00	36.00	21.97	1.64	5.42	195.3	0.51	
1.25	8.48	10.93	0.78	3.29	27.95	0.24	Q (25-yr Event)
1.43	10.44	12.05	0.87	3.55	37.00	0.27	Q (100-yr Event)

Discharge versus Depth Relationship



APPENDIX I-C

Calculations for Active Area Slope Run-On Controls

Written by: **C.Jordan** Date: **9/3/21** Reviewed by: **A. Smith** Date: **9/3/21**

Client: **Santee Cooper** Project: **Winyah Generating Station** Project No.: **GC8100** Phase No.: **05**

ACTIVE AREA RUN-ON CONTROL DESIGN

1 INTRODUCTION

The purpose of this calculation package is to:

- present an update to the originally permitted calculation from 2016 for compliance with the Federal CCR rules;
- present the analysis for the sizing of the temporary diversion berms to be utilized as necessary at the active face (i.e., diversions around areas of exposed waste) during development of the Class Three Landfill at the Winyah Generating Station (WGS) located in Georgetown County, South Carolina; and
- present the analysis for the sizing of the contact water management ditch located around the landfill perimeter, which will receive contact water generated from exterior-facing waste slopes and will convey the contact water runoff to temporary internal low points for collection and removal.

Diversion berms are temporary soil berms that will be constructed as necessary up-gradient from the active working face (exposed waste areas) to intercept storm water runoff flow before it comes in contact with waste. If these temporary diversion berms are used, they will route the non-contact (clean) storm water around active areas into the surface water management system and away from the active face. Contact water management ditches are temporary features along the landfill perimeter, which will intercept and convey contact water from exterior-facing uncovered waste slopes (i.e., prior to installation of intermediate or final cover).

To provide operator flexibility to adapt to differing conditions, rather than provide just one required design berm and ditch size for run-on control, this calculation package provides the sizing needed for a variety of cases. These are intended to capture the expected range of operational conditions, while providing operational flexibility to choose the appropriate minimum berm and ditch sizes based on the conditions that exist up-gradient from each specific active area at any point in time. As such, for these analyses, the maximum up-gradient drainage area which can be managed by each given diversion berm size and ditch size, and for the 25-year, 24-hour rainfall event, is calculated for run-on controls as required by the Federal CCR Rule.

Written by: **C.Jordan** Date: **9/3/21** Reviewed by: **A. Smith** Date: **9/3/21**

Client: **Santee Cooper** Project: **Winyah Generating Station** Project No.: **GC8100** Phase No.: **05**

2 ASSUMPTIONS AND PROCEDURES

The section discusses the assumptions and procedures for the design of the temporary diversion berms and contact water management ditch.

Active Face Run-On Diversion Berms

It is assumed that temporary diversion berms will be installed with flow line (longitudinal) slopes ranging from 0.5% to 2%. Temporary diversion berms will be placed as needed up-gradient from the active working face. The temporary diversion berms are assumed to be "tack-on" berms with a 2.5:1 side slope (see Figure 1 of this calculation package) to form a v-shaped channel. A channel depth of 2.0 feet was assumed (i.e., this is a fixed parameter of these calculations).

The Rational Method described in the Georgetown County *Storm Water Management Design Manual* (2006) is used to calculate the peak surface water discharge (this is the specified method for drainage areas of 20 acres or less). This approach is considered appropriate given this local recommendation, and because it is expected that drainage areas to a given diversion berm will be on the order of 20 acres or less. The channels were sized assuming they are flowing full, which is considered adequate since they are interior and temporary site features, and given other conservative selections of parameters as documented herein. The following steps were utilized to calculate the drainage areas that each diversion berm can accommodate.

1. Compute the discharge capacity of diversion berms with 0.5%, 1%, 1.5%, and 2% slopes using Manning's Equation for open channel flow.
2. Apply the Rational Method to compute the up-gradient drainage area that would produce the discharge capacity calculated in Step 1.

Manning's equation was used to estimate the peak discharge capacity of the vee-shaped channel created by a temporary diversion berm. Manning's equation (Chow, 1959) is expressed as:

Written by: **C.Jordan** Date: **9/3/21** Reviewed by: **A. Smith** Date: **9/3/21**

Client: **Santee Cooper** Project: **Winyah Generating Station** Project No.: **GC8100** Phase No.: **05**

$$Q = \frac{1.49}{n} AR^{2/3} S^{1/2} \qquad (1)$$

where:

Q = discharge (cfs),
n = Manning's roughness coefficient,
A = area of cross-section of flow (ft^2),
P = wetted perimeter (ft),
R = hydraulic radius = A/P (ft), and
S = longitudinal slope (ft/ft).

The peak discharge from the contributing drainage area by the Rational Method (Georgetown County, 2006) can be computed by:

$$Q = C \times C_f \times i \times A \qquad (2)$$

where:

Q = peak design discharge (cfs),
C = runoff coefficient (dimensionless),
C_f = frequency factor based on recurrence Interval
i = design rainfall intensity (in/hr), and
A = drainage area (acres).

For this 2021 update, the design rainfall intensity in Equation (2) was found using the National Oceanic and Atmospheric Association (NOAA) Point Precipitation Data Frequency Server information for the Winyah Generating Station. From this source, the 25-year intensity is 10.3 inches per hour for a storm with a 5-minute duration and is included as Attachment 1 of this package.

Equation (2) is rearranged, and the watershed drainage area was back-calculated for each potential flow line slope of a temporary diversion berm.

Contact Water Management Ditch

It is assumed that the contact water management ditches will be installed with flow line (longitudinal) slopes ranging from 0.5% to 2%. A schematic (conceptual) layout of the contact water management ditch and contributing areas is presented in Figure 2 of this calculation package. As shown, for calculation purposes the ditch is modeled as a vee-shaped channel. A minimum channel depth of 2.0 feet was assumed (i.e., this is a fixed parameter of these calculations), and since this ditch is positioned near the landfill perimeter, it is designed to have 0.5-ft of freeboard (i.e., a flow depth of 1.5 feet). Note that to route the ditch towards low points, the ditch depth will by definition increase as the flow line drops in elevation along the ditch alignment. To be conservative, the ditch is modeled with a constant depth using the minimum flow depth. Thus, if it is installed and operated according to the sizing and drainage area calculations presented herein, this will assure it is able to handle the design storm.

The Rational Method approach and methodology as described above for the diversion berms is used. The calculations are performed for ditches with 0.5%, 1%, 1.5%, and 2% slopes.

3 DESIGN PARAMETERS

This section discusses the justification behind the selected design parameters for the temporary diversion berms.

The Manning's roughness coefficient (*n*) for the diversion berm was selected as 0.02 for unlined, bare soil channels with flow depths larger than 0.5 ft, as shown in Table 1 (Georgetown County, 2006). This was also used for the ditch (considered an appropriate representation of a bare ground channel formed of exposed waste). The peak discharge flowing to the channel is calculated using the Rational Method.

A runoff coefficient of C = 0.60 was selected to model the diversion berm scenario, based on information provided by the *Storm Water Management Design Manual* (Georgetown County, 2006) for graded areas with no plant cover and clayey soils having an average slope of 5% to 10% as shown in Table 2. A runoff coefficient of C = 0.90 was selected to model the contact water management ditch scenario. This was increased as compared to the diversion berm scenario to account for graded bare ground waste material, which is judged likely to allow less infiltration and produce more runoff (similar, but not quite as much runoff as from paved streets). A runoff coefficient frequency factor of C_f = 1.1 was selected based on information provided by the *Storm Water Management Design Manual* (Georgetown County, 2006), as shown in Table 3.

For a conservative design approach, a minimum time of concentration of 5 minutes was used for the rainfall intensity by Equation (2).

4 RESULTS

Diversion Berms

The results of the temporary diversion berm calculations are summarized in Table 4 for each assumed flow line slope. The calculated drainage areas represent the maximum drainage area that each temporary diversion berm configuration can accommodate for the 25-year design rainfall event. It should be noted that if, during operations, a larger area than those calculated in Table 5 will be draining towards the active face, multiple diversion berms may be constructed to comply with the drainage area requirements presented herein for the given berm height and the selected flow line slope.

Contact Water Management Ditch

The results of the contact water management ditch calculations are summarized in Table 5 for each assumed flow line slope. The calculated drainage areas represent the maximum drainage area that each contact water management ditch configuration can accommodate for the 25-year design rainfall event. It should be noted that if, during operations, a larger area than those calculated in Table 5 will be draining towards the contact water management ditch, this indicates that either: (i) a new and separate contact water management ditch is needed (flowing to its own separate low area for collection and removal of contact water); (ii) up-gradient diversion berms/terraces on the exterior waste slopes can be used to divert contact water to other areas and thereby reduce the drainage area contributing to the contact water management ditch to be within acceptable area; or (iii) an enlarged ditch (e.g., with a trapezoidal bottom instead of the v-ditch assumed herein) is needed, which may be calculated using the methodology presented herein.

5 REFERENCES

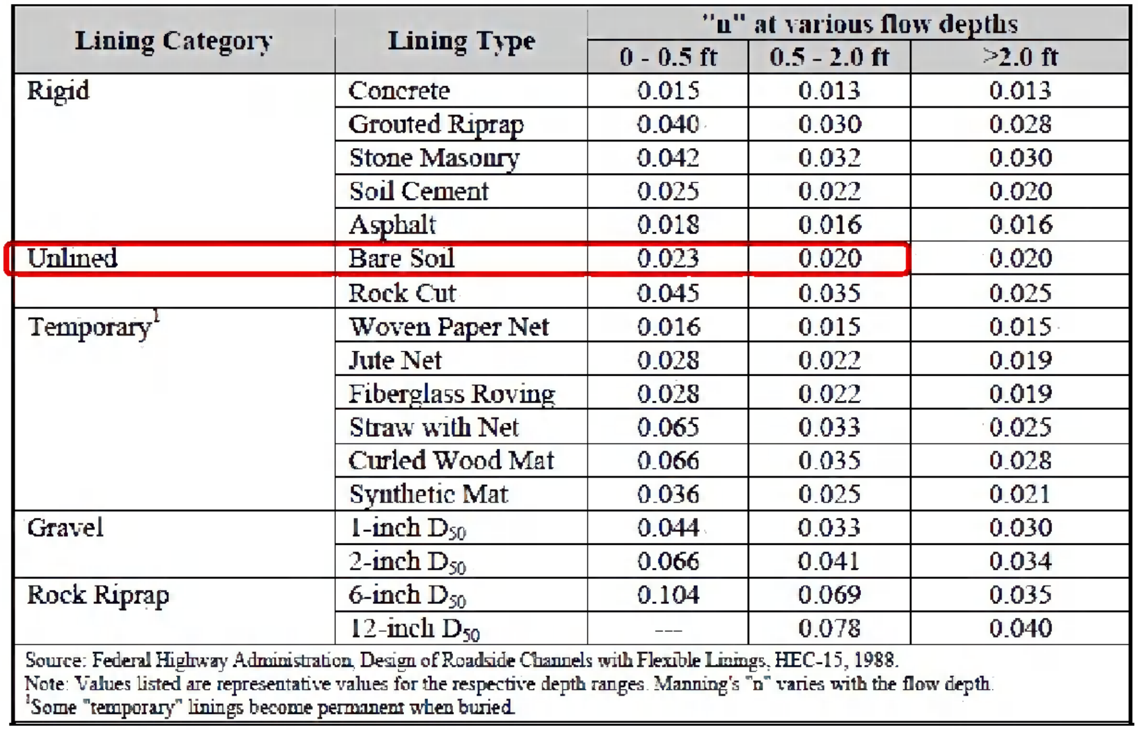
Chow, V.T. (1959), *Open Channel-Hydraulics,* McGraw-Hill.

Georgetown County (2006), *Storm Water Management Design Manual*, Georgetown County, South Carolina, revised 14 November 2006.

TABLES

Table 1. Manning's Roughness Coefficients for Artificial Channels

(from Georgetown County, 2006)

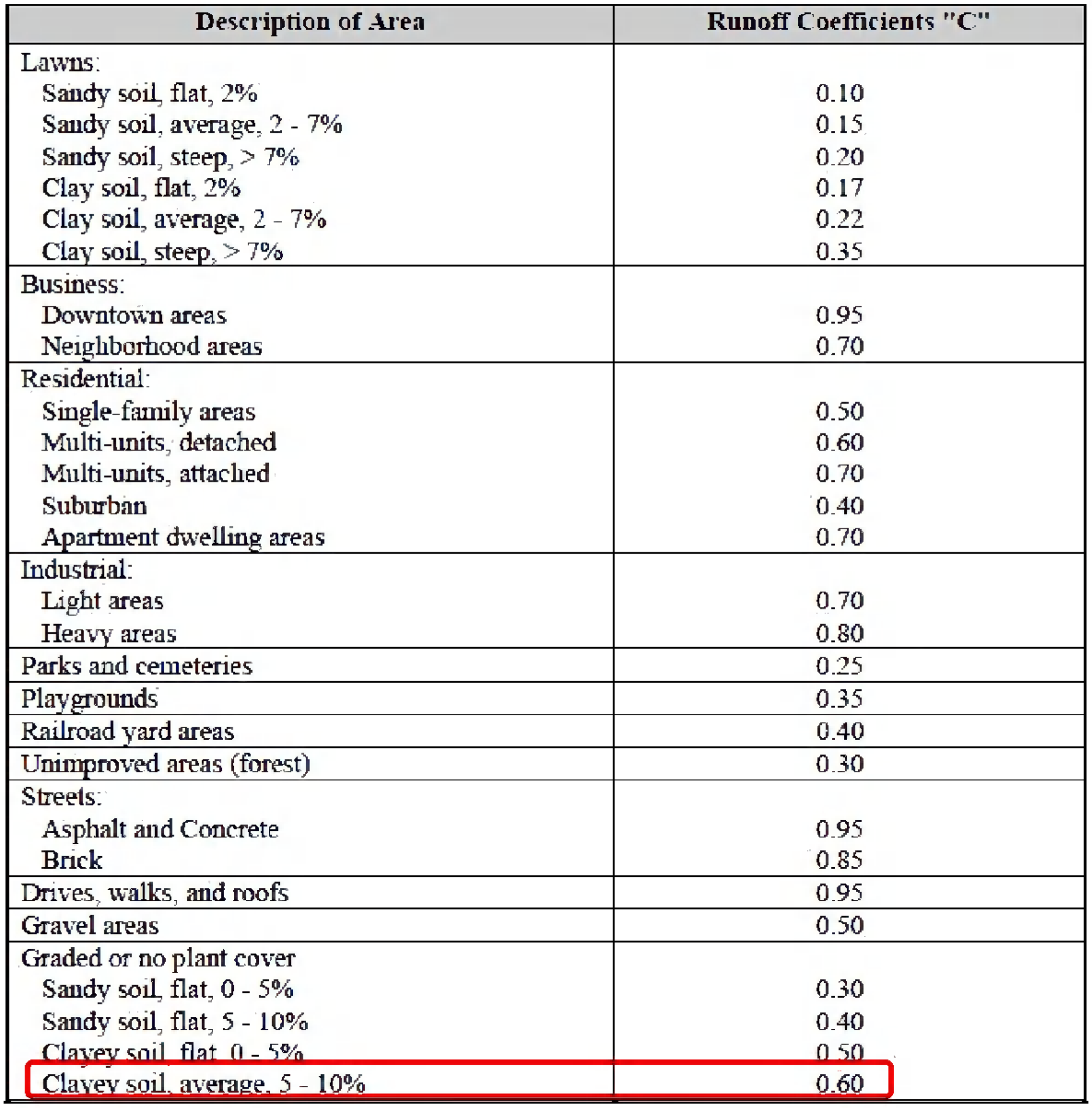


Lining Category	Lining Type	"n" at various flow depths		
		0 - 0.5 ft	0.5 - 2.0 ft	>2.0 ft
Rigid	Concrete	0.015	0.013	0.013
	Grouted Riprap	0.040	0.030	0.028
	Stone Masonry	0.042	0.032	0.030
	Soil Cement	0.025	0.022	0.020
	Asphalt	0.018	0.016	0.016
Unlined	Bare Soil	0.023	0.020	0.020
	Rock Cut	0.045	0.035	0.025
Temporary[1]	Woven Paper Net	0.016	0.015	0.015
	Jute Net	0.028	0.022	0.019
	Fiberglass Roving	0.028	0.022	0.019
	Straw with Net	0.065	0.033	0.025
	Curled Wood Mat	0.066	0.035	0.028
	Synthetic Mat	0.036	0.025	0.021
Gravel	1-inch D_{50}	0.044	0.033	0.030
	2-inch D_{50}	0.066	0.041	0.034
Rock Riprap	6-inch D_{50}	0.104	0.069	0.035
	12-inch D_{50}	---	0.078	0.040

Source: Federal Highway Administration, Design of Roadside Channels with Flexible Linings, HEC-15, 1988.
Note: Values listed are representative values for the respective depth ranges. Manning's "n" varies with the flow depth.
[1] Some "temporary" linings become permanent when buried.

Table 2. Runoff Coefficients for Rational Method

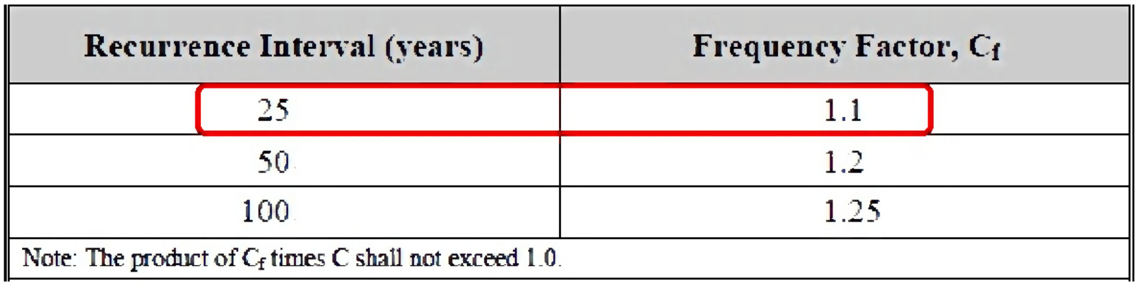
(from Georgetown County, 2006)



Description of Area	Runoff Coefficients "C"
Lawns:	
Sandy soil, flat, 2%	0.10
Sandy soil, average, 2 - 7%	0.15
Sandy soil, steep, > 7%	0.20
Clay soil, flat, 2%	0.17
Clay soil, average, 2 - 7%	0.22
Clay soil, steep, > 7%	0.35
Business:	
Downtown areas	0.95
Neighborhood areas	0.70
Residential:	
Single-family areas	0.50
Multi-units, detached	0.60
Multi-units, attached	0.70
Suburban	0.40
Apartment dwelling areas	0.70
Industrial:	
Light areas	0.70
Heavy areas	0.80
Parks and cemeteries	0.25
Playgrounds	0.35
Railroad yard areas	0.40
Unimproved areas (forest)	0.30
Streets:	
Asphalt and Concrete	0.95
Brick	0.85
Drives, walks, and roofs	0.95
Gravel areas	0.50
Graded or no plant cover	
Sandy soil, flat, 0 - 5%	0.30
Sandy soil, flat, 5 - 10%	0.40
Clayey soil, flat, 0 - 5%	0.50
Clayey soil, average, 5 - 10%	0.60

Table 3. Rational Method Runoff Coefficient Frequency Factors

(from Georgetown County, 2006)



Recurrence Interval (years)	Frequency Factor, C_f
25	1.1
50	1.2
100	1.25

Note: The product of C_f times C shall not exceed 1.0.

Table 4. Diversion Berm Drainage Area Sizing

Depth of Channel (ft)	Diversion Berm Flow Line Slope (%)	Maximum Predicted Flow Velocity (ft/s)	Maximum Predicted Flow Rate (cfs)	Maximum Drainage Area (ac)
2.0	0.5%	5.05	55.58	8.2
	1.0%	7.15	78.60	11.6
	1.5%	8.75	96.27	14.2
	2.0%	10.11	111.16	16.4

Note: The back-calculated maximum allowable drainage area for the channel dimensions (geometry and slope) given above, as calculated by the Rational Method, assumes that the channel created by the diversion berm is flowing full when conveying the peak discharge during the 25-year rainfall event and from the maximum contributing drainage area.

Table 5. Contact Water Management Ditch Drainage Area Sizing

Depth of Channel (ft)	Depth of Flow (ft)	Diversion Berm Flow Line Slope (%)	Maximum Predicted Flow Velocity (ft/s)	Maximum Predicted Flow Rate (cfs)	Maximum Drainage Area (ac)
2	1.5	0.5%	4.20	28.34	2.8
		1.0%	5.94	40.08	3.9
		1.5%	7.27	49.09	4.8
		2.0%	8.40	56.68	5.6

Note: As indicated in the above table, the contact water management ditch is designed to have 0.5-ft of freeboard. That is, the back-calculated maximum allowable drainage area for the channel dimensions (geometry and slope) given above, as calculated by the Rational Method, assumes that the 2-ft deep channel created by the contact water management ditch has a depth of flow of 1.5-ft when conveying the peak discharge during the 25-year rainfall event and from the maximum contributing drainage area.

FIGURES

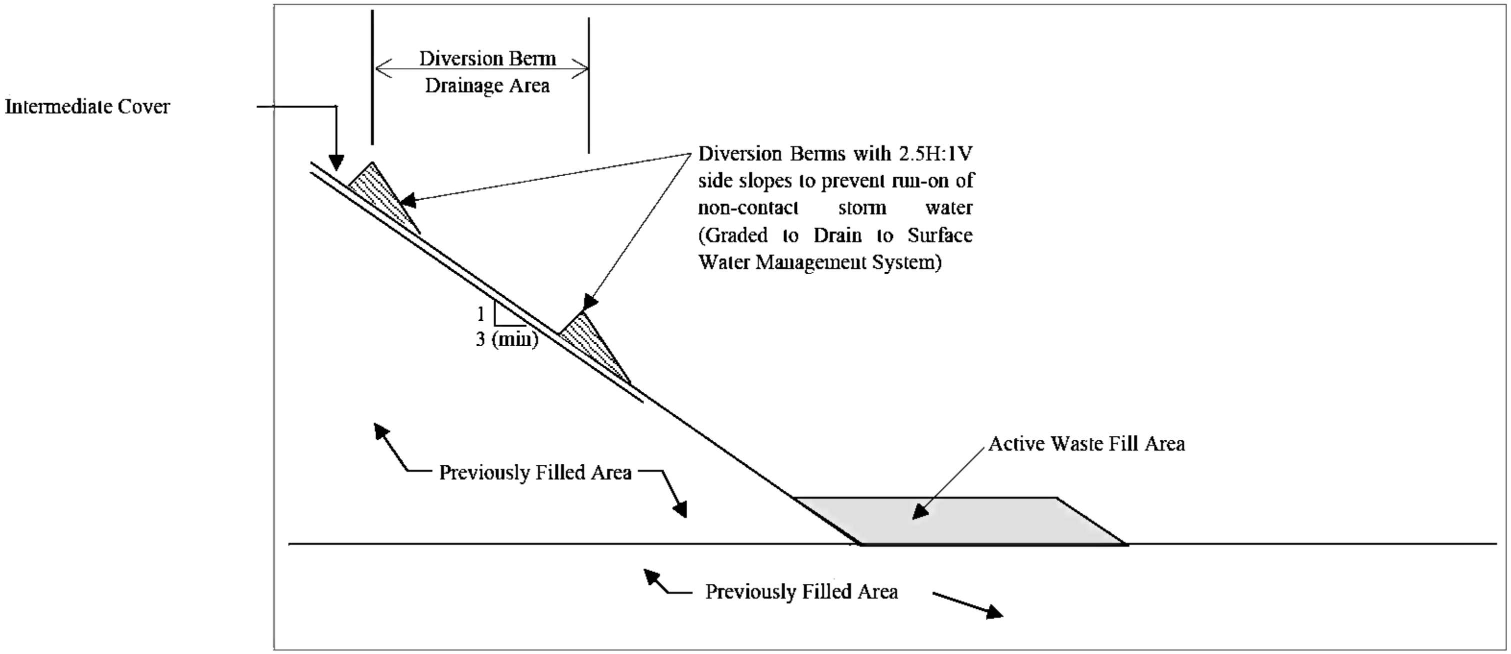


Figure 1. Typical/Schematic (Conceptual) of Phased Active Fill Area Section (portraying possible conditions where diversion berms may be used) (Not to Scale (NTS))

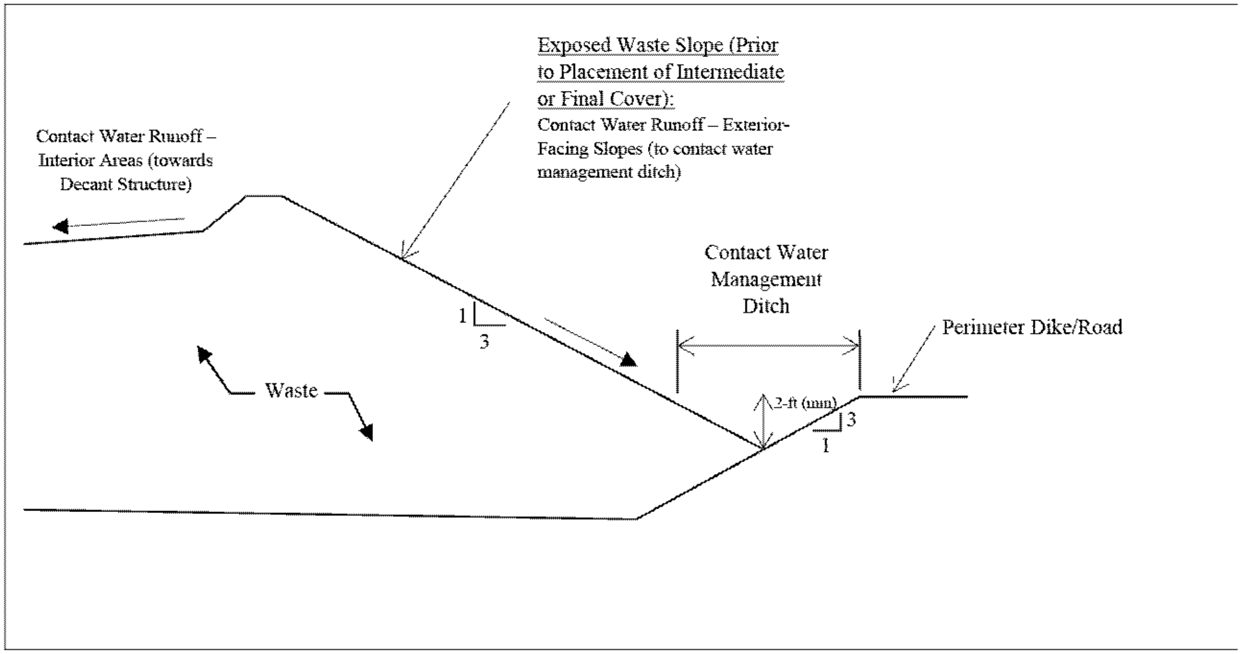


Figure 2. Typical/Schematic (Conceptual) of Exterior-Facing Waste Slope and Contact Water Management Ditch (Not to Scale (NTS))