Run-on and Run-off Control System Plan for Winyah Generating Station's New CCR Class 3 Landfill Area 2

40 CFR Part 257
Operating Criteria
§257.81 (c)

CONTENTS

1. INTRODUCTION

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

This document serves as certification that the run-on and run-off control system plan for the new CCR Landfill Area 2 ("CCR Unit") at Winyah Generating Station in Georgetown, South Carolina meets the requirements of §257.81. The design is documented in the Winyah Generating Station Class Three Landfill Permit Application approved by the South Carolina Department of Health and Environmental Control (DHEC) on 15 September 2017 (Permit #LF3-00042). The run-on and run-off control system meets the South Carolina solid waste management regulation R.61-107.19 as certified by the design engineer-of-record, Scott M. Graves, P.E., Geosyntec Consultants, Inc. DHEC issued a permit to construct on September 15, 2017 with an effective date of September 30, 2017. Construction Quality Assurance was documented in a report by Insight Group dated December 10, 2021.

2. DISCUSSION

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

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

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

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

The new CCR Unit is located within the footprint of Ash Pond A, which is currently undergoing closure. Run-on controls, such as perimeter and diversion berms, will be used to prevent stormwater flow onto the active portion of the landfill.

The CCR Unit is surrounded by a perimeter berm that prevents run-on from flowing onto the active portion of the landfill. The perimeter berm will intercept the stormwater run-off flow generated within the Ash Pond A footprint so that it does not come into contact with the landfill waste and allow this stormwater to be collected and managed approrpriately under the facility's existing NPDES permit.

Storm water run-on may also be directed away from the active working area using temporary diversion berms positioned if necessary, around/up-gradient from the active working area, to prevent flow onto the active portion of the landfill during the peak discharge from a 24-hour, 25-year storm. Calculations for sizing of the temporary diversion berms, depending on the up-gradient area contributing stormwater run-on, are presented in Appendix A. All facilities are designed to handle the peak discharge from a 24- hour, 25-year storm event.

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

Run-off controls will be used at active portions of the landfill to collect and control at least the water volume resulting from a 24-hour, 25-year storm. The new CCR Unit includes a decant structure (chimney drain) and attenuation basin located within the active portion of the landfill. The attenuation basin is designed to intercept stormwater runoff from the active face of the landfill and convey it directly to the NPDES permitted wastewater pond (industrial cooling pond) via the decant structure, gravity piping, and pumping. The purpose of the decant structure is to minimize leachate generation and to collect and control at least the water volume (and peak flow rate) resulting from a 24-hour, 25-year storm. The design documentation for the decant structure and the volumetric analysis for the industrial cooling pond to contain the 24-hour, 25-year storm are included in Appendix B.

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

Storm water run-off from active areas that has come in contact with waste will be managed as contact water. Contact water will be removed to prevent stagnant ponding of water and to minimize it from otherwise infiltrating through the waste to become leachate. This will be accomplished using pumps to remove contact water, as well as using decant structures to filter and convey contact water out of the cell. In both cases (pumps and decant structures), the contact water will be conveyed through piping that is independent from the rest of the leachate management system. Calculations for sizing of the decant structure to effectively manage the volume of water resulting from the design storm are included in Appendix B. All run-off generated from the active portion of the new CCR Unit is conveyed to an on-site NPDES permitted wastewater pond. The wastewater pond will contain the volume resulting from a 24-hour, 25-year storm. The effluent from the wastewater pond is ultimately discharged from the site through NPDES Permit #SC0022471 Outfall 002. All run-off from the active portion of the new CCR Unit is designed to be handled in accordance with the surface water requirements under §257.3-3.

This document satisfies the requirements of §257.81(c) by providing a run-on and run-off control system plan that demonstrates how the run-on and run-off control system for the new Class 3 Landfill Area 2 at the Winyah Generating Station has been designed and constructed and will be operated and maintained to meet the applicable requirements of this section and state regulations, including supporting engineering calculations.

3. CONCLUSIONS

This document presents the run-on and run-off control system plan for the new Class 3 Landfill Area 2 at Winyah Generating Station in Georgetown, SC. The run-on and run-off control system plan is in accordance with the requirements of Title 40 CFR §257.81 for CCR landfills.

4. CERTIFICATION

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

Federal CCR Rule: 40 CFR §257.81

CCR Unit: WGS Class Three Landfill Area 2 - New CCR Landfill

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

Seal and Signature: SOUTH CAROLINA LICENSED PROFESSIONAL ENGINEER No. 35800 AUBREE L. DECOTEAU Aubree R. Decoteau 12/16/21

Printed Name: Aubree L. Decoteau

P.E. License Number: 35800 State of South Carolina

APPENDIX A

Calculations for Run-on Control Systems

APPENDIX F-5

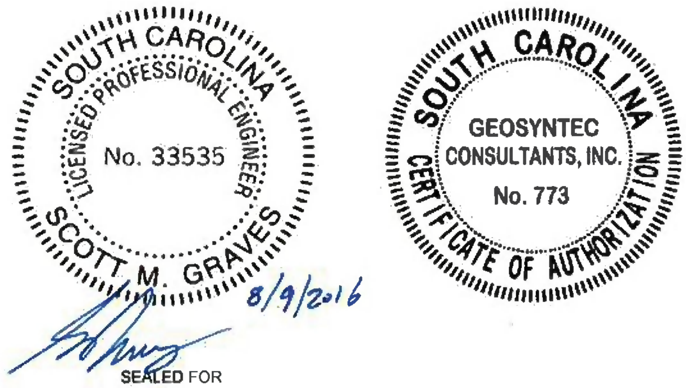
ACTIVE AREA RUN-ON CONTROL DESIGN

Written by: **A. Sivashanthan** Date: **6/15/2016** Reviewed by: **B. Klenzendorf** Date: **6/17/2016**

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

APPENDIX F-5

ACTIVE AREA RUN-ON CONTROL DESIGN



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CALCULATION PAGES F5-1
THROUGH F5-13

1 INTRODUCTION

The purpose of this calculation package is to present the analysis for the sizing of the temporary diversion berms to be utilized as necessary at the active face (i.e., diversions around areas of exposed waste) during development of the proposed Class Three Landfill at the Winyah Generating Station (WGS) located in Georgetown County, South Carolina.

Diversion berms are temporary soil berms that will be constructed as necessary up-gradient from the active working face (exposed waste areas) to intercept storm water runoff flow before it comes in contact with waste. If these temporary diversion berms are used, they will route the non-contact (clean) storm water around active areas into the surface water management system and away from the active face. To provide operator flexibility to adapt to differing conditions, rather than provide just one required design berm size, this calculation package provides the sizing needed for a variety of cases. These are intended to capture the expected range of operational conditions, while providing operational flexibility to choose the appropriate minimum berm size based on the conditions that exist up-gradient from each specific active area at any point in time. As such, for these analyses, the maximum up-gradient drainage area which can be managed by each given diversion berm size and for the 25-year rainfall event is calculated.

2 ASSUMPTIONS AND PROCEDURES

The section discusses the assumptions and procedures for the design of the temporary diversion berms.

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

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

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

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

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

where:

Q = discharge (cfs),

n = Manning's roughness coefficient,

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

P = wetted perimeter (ft),

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

S = longitudinal slope (ft/ft).

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

$$Q = C \times C_f \times i \times A \tag{2}$$

where:

Q = peak design discharge (cfs),

C = runoff coefficient (dimensionless),

C_f = frequency factor based on recurrence Interval

i = design rainfall intensity (in/hr), and

A = drainage area (acres).

The design rainfall intensity in Equation (2) is calculated using guidance in the Georgetown County *Storm Water Design Manual* (2006). The manual estimates the design rainfall intensity as shown in Equation (3):

$$i = \frac{a}{(b+t_c)^c} \tag{3}$$

where:

i = design rainfall intensity (in/hr),

t_c = time of concentration (minutes [min]), and

a, b, and c = coefficients for specific return period.

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

3 DESIGN PARAMETERS

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

The Manning's roughness coefficient (n) for the diversion berm was selected as 0.02 for unlined, bare soil channels with flow depths larger than 0.5 ft, as shown in Table 1 (Georgetown County, 2006). The peak discharge flowing to the channel is calculated using the Rational Method.

A runoff coefficient of C = 0.60 was selected based on information provided by the *Storm Water Management Design Manual* (Georgetown County, 2006) for graded areas with no plant cover and clayey soils having an average slope of 5% to 10% as shown in Table 2. A runoff coefficient frequency factor of C_f = 1.1 was selected based on information provided by the *Storm Water Management Design Manual* (Georgetown County, 2006), as shown in Table 3.

For a conservative design approach, a minimum time of concentration of 5 minutes was used to calculate the rainfall intensity by Equation (3). Because the drainage areas for this analysis are much smaller, a time of concentration of 5 minutes is used.

To calculate the design rainfall intensity using Equation (3), the coefficients a, b, and c were selected for a 25-year return period from Table 4 as 288.87, 29.41, and 0.996 respectively (Georgetown County, 2006). The corresponding rainfall intensity for a time of concentration of 5 minutes was selected as 8.51 in/hr for a 25-year storm event.

4 RESULTS

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

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. Rainfall Intensity Coefficient for Georgetown County

(from Georgetown County, 2006)

Frequency (years)	a	b	c	i (in/hr) t_c=5min	i (in/hr) t_c=10min	i (in/hr) t_c=15min
2	249.76	34.10	1.026	5.80	5.13	4.59
5	261.38	32.32	1.015	6.63	5.84	5.21
10	269.35	31.13	1.007	7.26	6.37	5.68
25	288.87	29.41	0.996	8.51	7.43	6.60
50	288.87	28.24	0.989	9.04	7.87	6.97
100	296.41	27.09	0.981	9.86	8.55	7.55
Source: South Carolina Department of Transportation, September 1997						

Table 5. Diversion Berm Drainage Area Sizing

Depth of Channel (ft)	Diversion Berm Flow Line Slope (%)	Maximum Predicted Flow Velocity (ft/s)	Maximum Predicted Flow Rate (cfs)	Maximum Drainage Area (ac)
2.0	0.5%	5.05	55.58	9.9
	1.0%	7.15	78.60	14.0
	1.5%	8.75	96.27	17.1
	2.0%	10.11	111.16	19.8

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

FIGURES

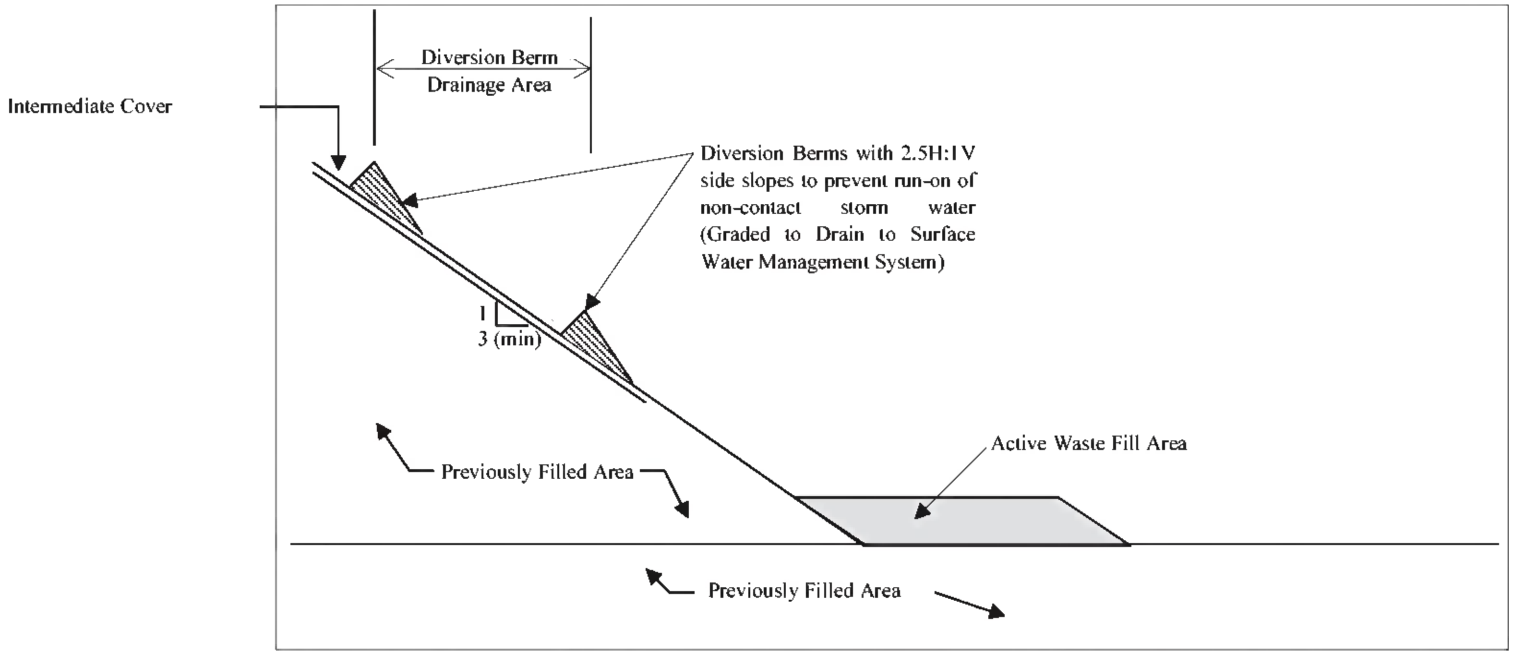

Figure 1. Typical/Schematic (Conceptual) of Active Fill Area Section

(portraying possible conditions where diversion berms may be used) (Not to Scale (NTS))

APPENDIX B

Calculations for Run-off Control Systems

APPENDIX E-5

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

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

APPENDIX E-5

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

SOUTH CAROLINA LICENSED PROFESSIONAL ENGINEER No. 33535 SCOTT M. GRAVES

8/9/2016

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

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

PURPOSE

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

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

METHOD OF ANALYSIS

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

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

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

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

DESIGN PARAMETERS

<u>Rainfall Event</u>

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

Contact Water Catchment Area

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

Contact Water Attenuation Basin and Outlet Structures

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

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

Leachate and Contact Water Storage Ponds

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

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

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

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

RESULTS

Attenuation Basin and Decant Structure

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

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

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

Leachate and Contact Water Storage Ponds

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

CONCLUSIONS

Based on calculation herein, the decant structure is designed to drain the 25-yr, 24-hr storm within approximately three days without overtopping. In addition, the decant structure is designed to drain the 100-yr, 24-hr storm within approximately three days without overtopping the containment berm. The Temporary Pond and Permanent Pond are designed to have adequate capacity to manage the contact water and direct rainfall generated by the design storm events, in addition to storing the peak monthly average discharge of leachate and contact water from the active area, and peak monthly average rainfall over the ponds.

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

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

PF tabular

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

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

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

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

Please refer to NOAA Atlas 14 document for more information.

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TABLE E-5.2 – SIZING OF ATTENUATION BASIN

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

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

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

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

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

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

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

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

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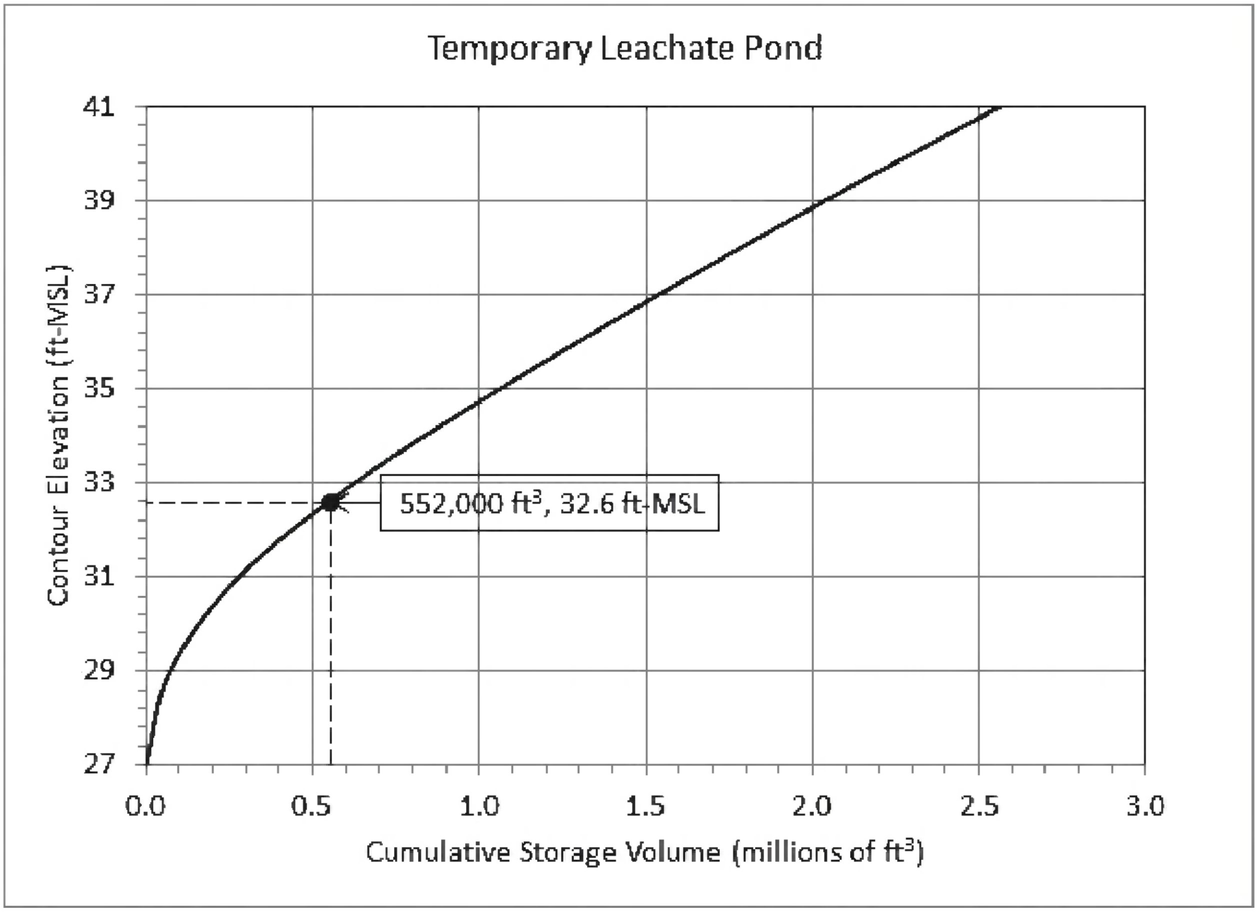

Figure E-5.1. Cumulative Storage Volume of Temporary Pond (Initial Pond Elevation Shown)

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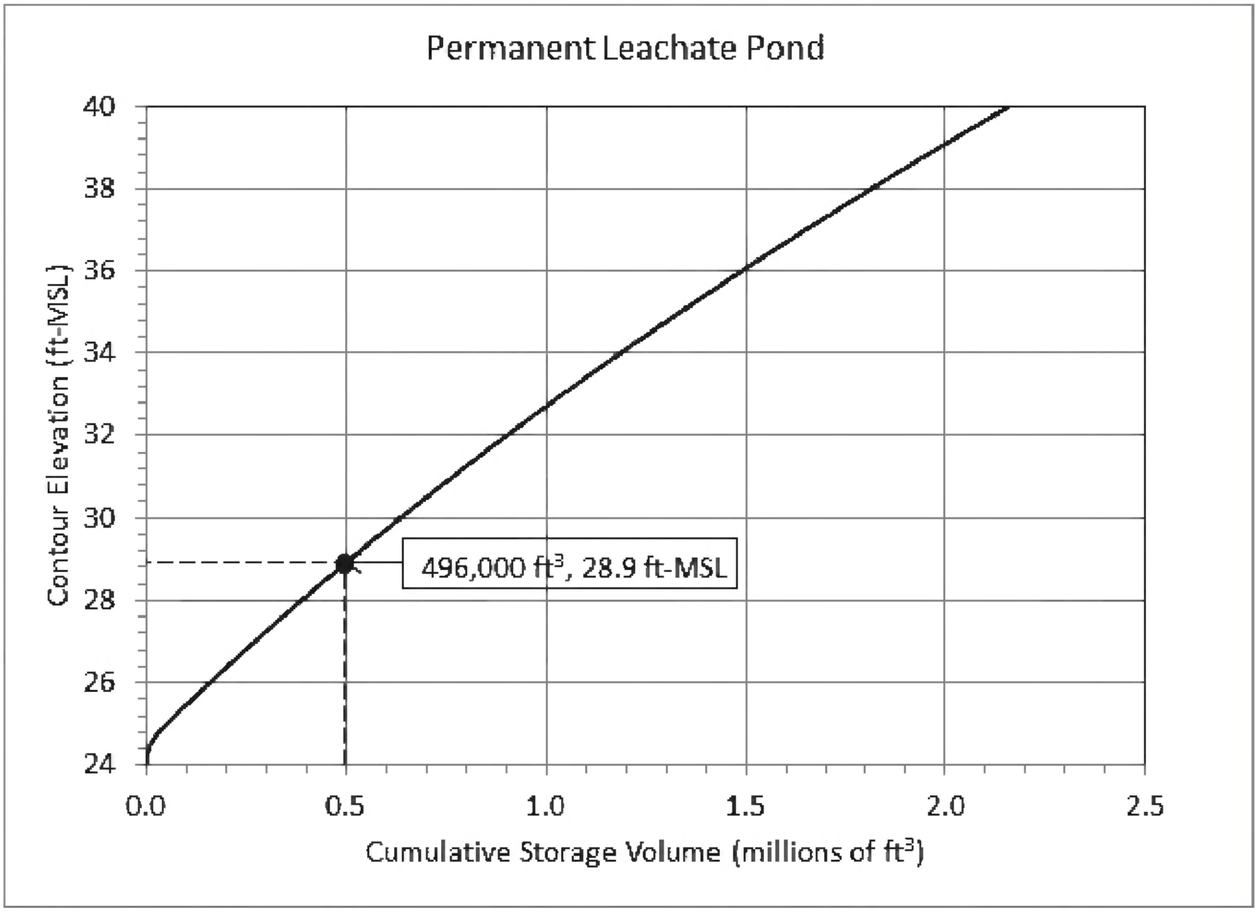

Figure E-5.2. Cumulative Storage Volume of Permanent Pond (Initial Pond Elevation Shown)

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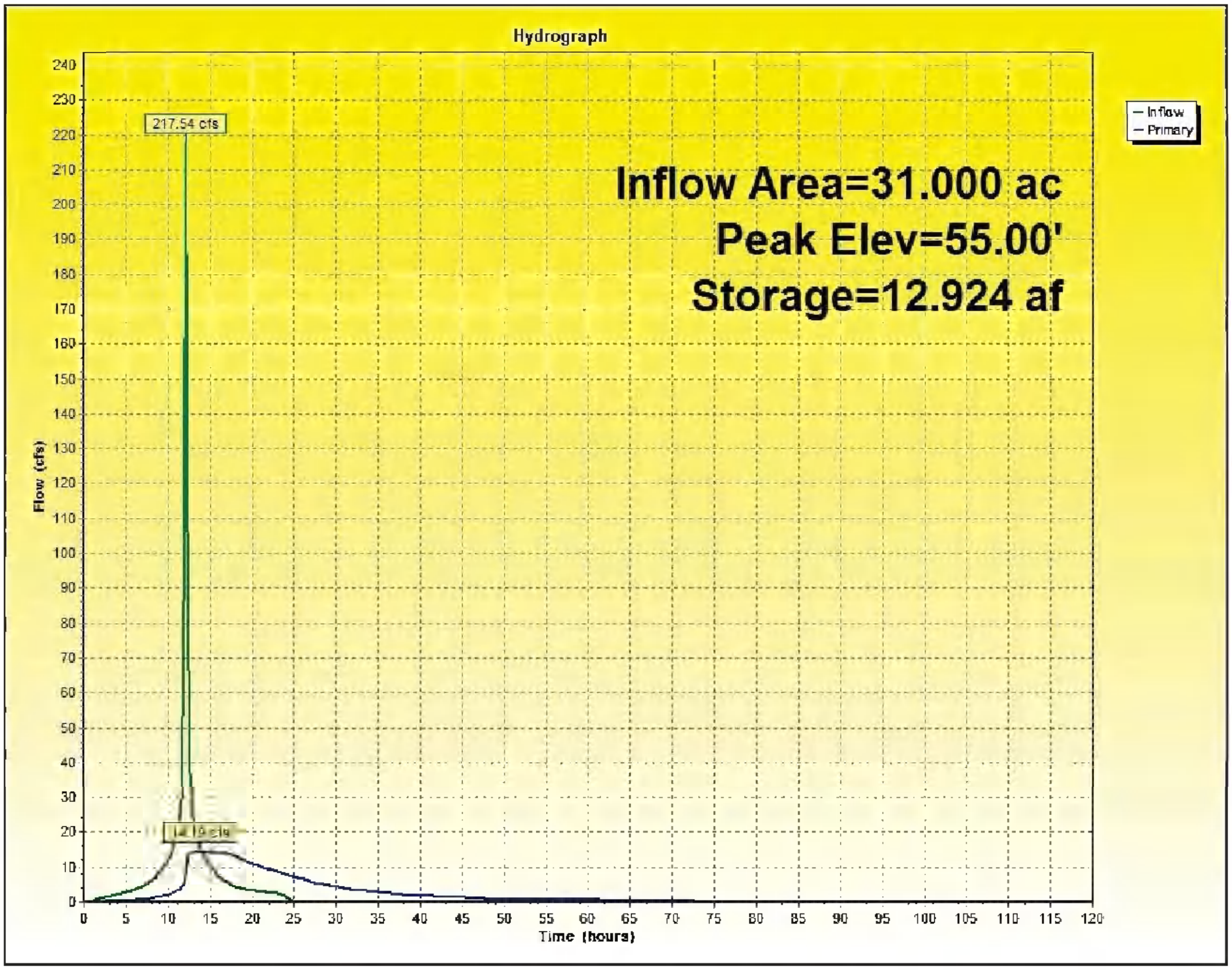

Figure E-5.3. Inflow and Outflow Hydrograph for 25-yr, 24-hr Rainfall Event

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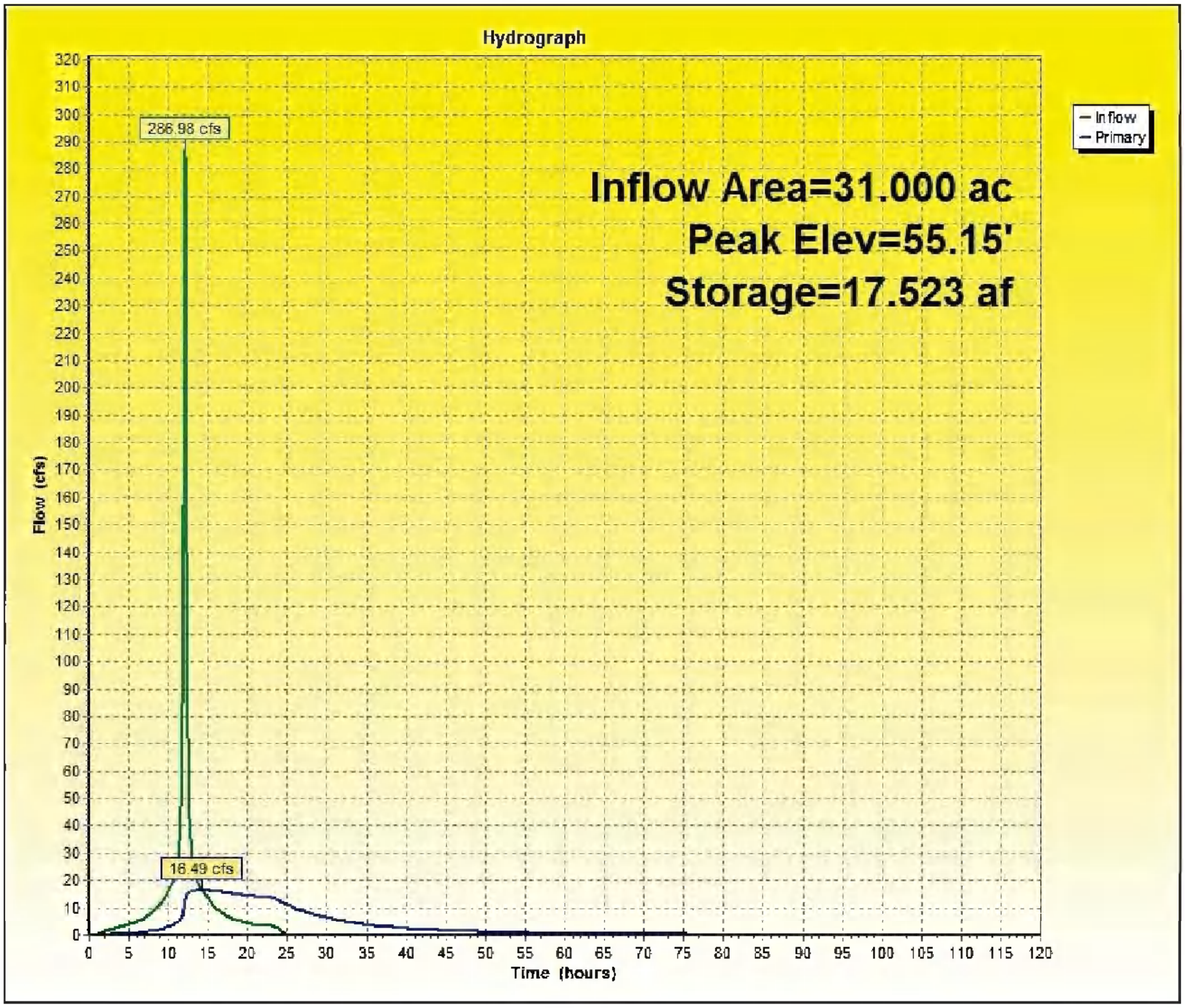

Figure E-5.4. Inflow and Outflow Hydrograph for 100-yr, 24-hr Rainfall Event

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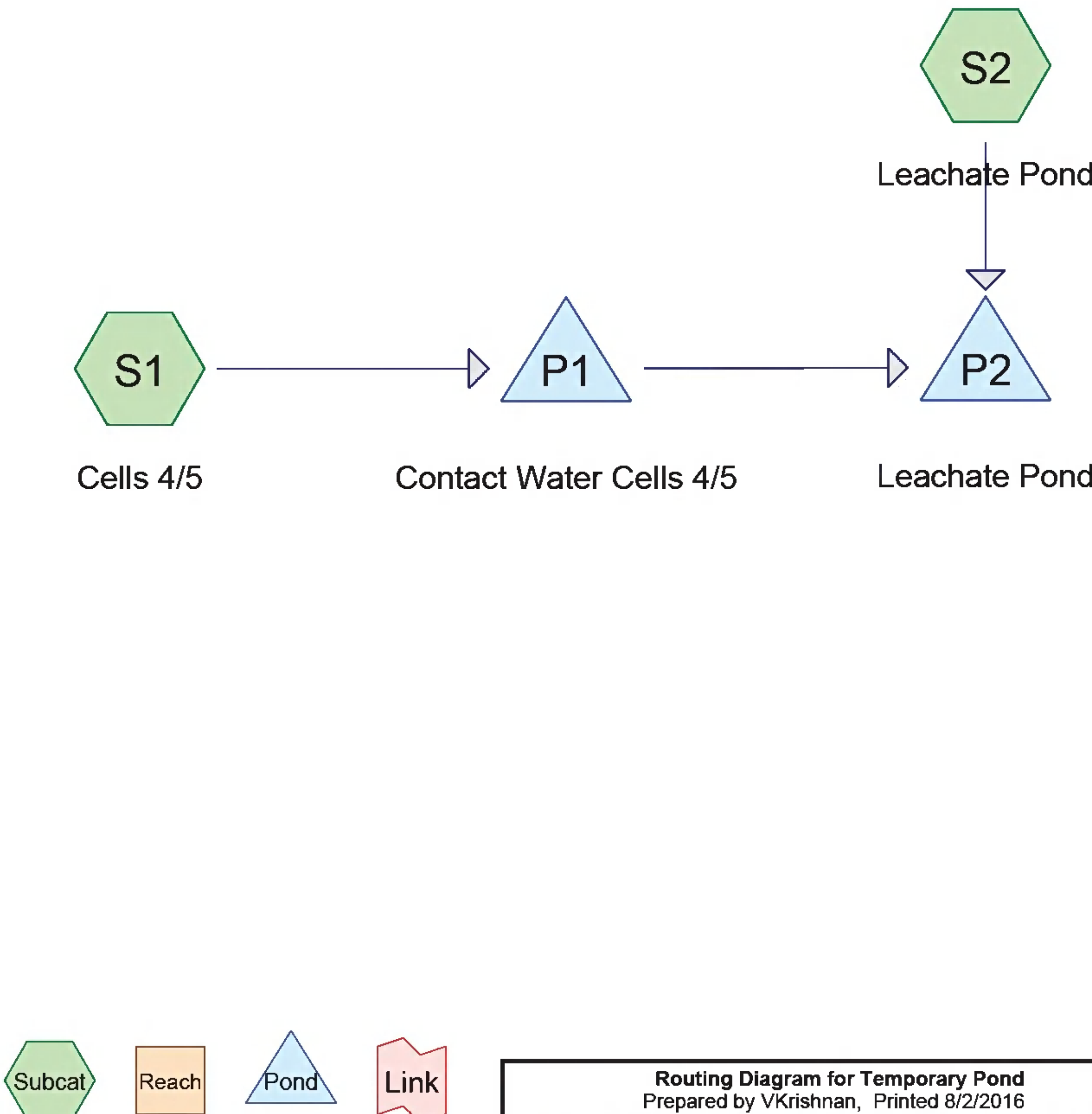
APPENDIX E-5.A

HYDROCAD REPORTS

TEMPORARY LEACHATE POND



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

Area Listing (all nodes)

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

Soil Listing (all nodes)

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

Ground Covers (all nodes)

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

Pipe Listing (all nodes)

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

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

Subcatchment S1: Cells 4/5	Runoff Area=31.000 ac 0.00% Impervious Runoff Depth=7.92" Tc=10.0 min CN=97 Runoff=217.54 cfs 20.460 af
Subcatchment S2: Leachate Pond	Runoff Area=268,880 sf 100.00% Impervious Runoff Depth=8.28" Tc=0.0 min CN=100 Runoff=57.55 cfs 4.259 af
Pond P1: Contact Water Cells 4/5	Peak Elev=55.00' Storage=12.924 af Inflow=217.54 cfs 20.460 af Outflow=14.19 cfs 20.460 af
Pond P2: Leachate Pond	Peak Elev=36.69' Storage=1,462,024 cf Inflow=64.22 cfs 24.719 af Outflow=0.78 cfs 7.004 af

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

Summary for Subcatchment S1: Cells 4/5

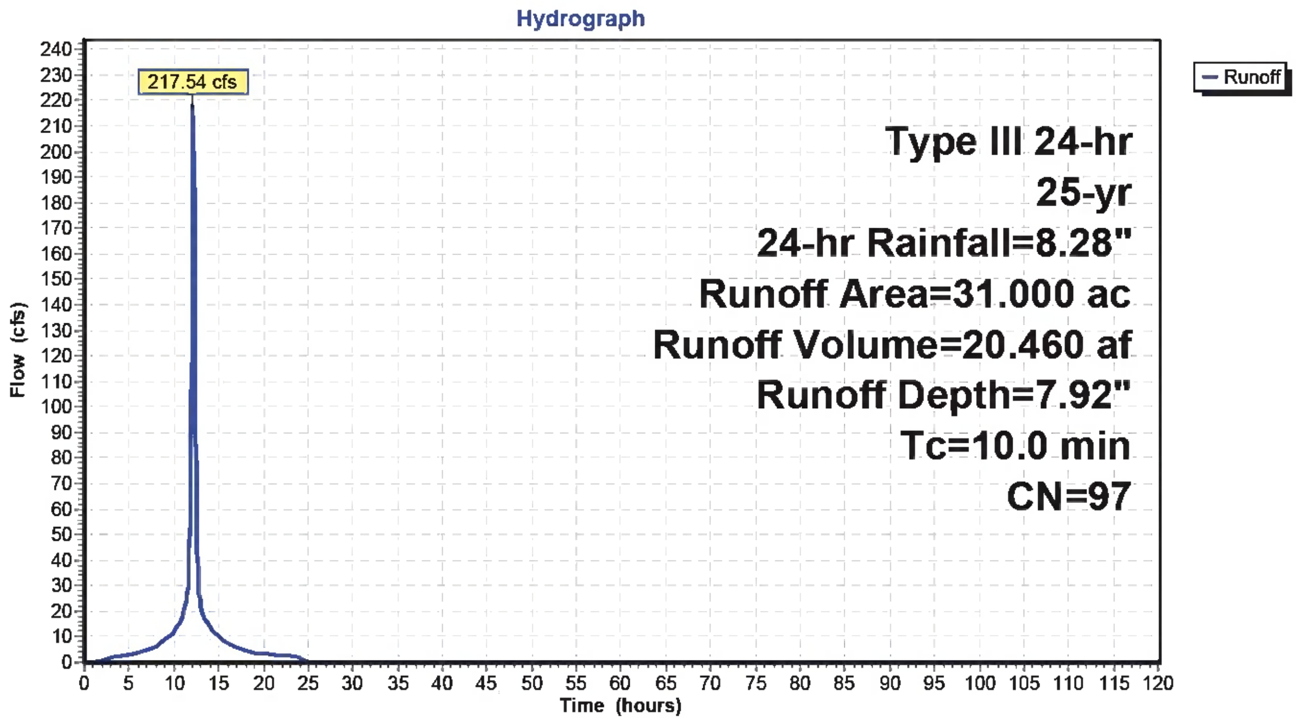
Runoff = 217.54 cfs @ 12.14 hrs, Volume= 20.460 af, Depth= 7.92"

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

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

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

Subcatchment S1: Cells 4/5



Summary for Subcatchment S2: Leachate Pond

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

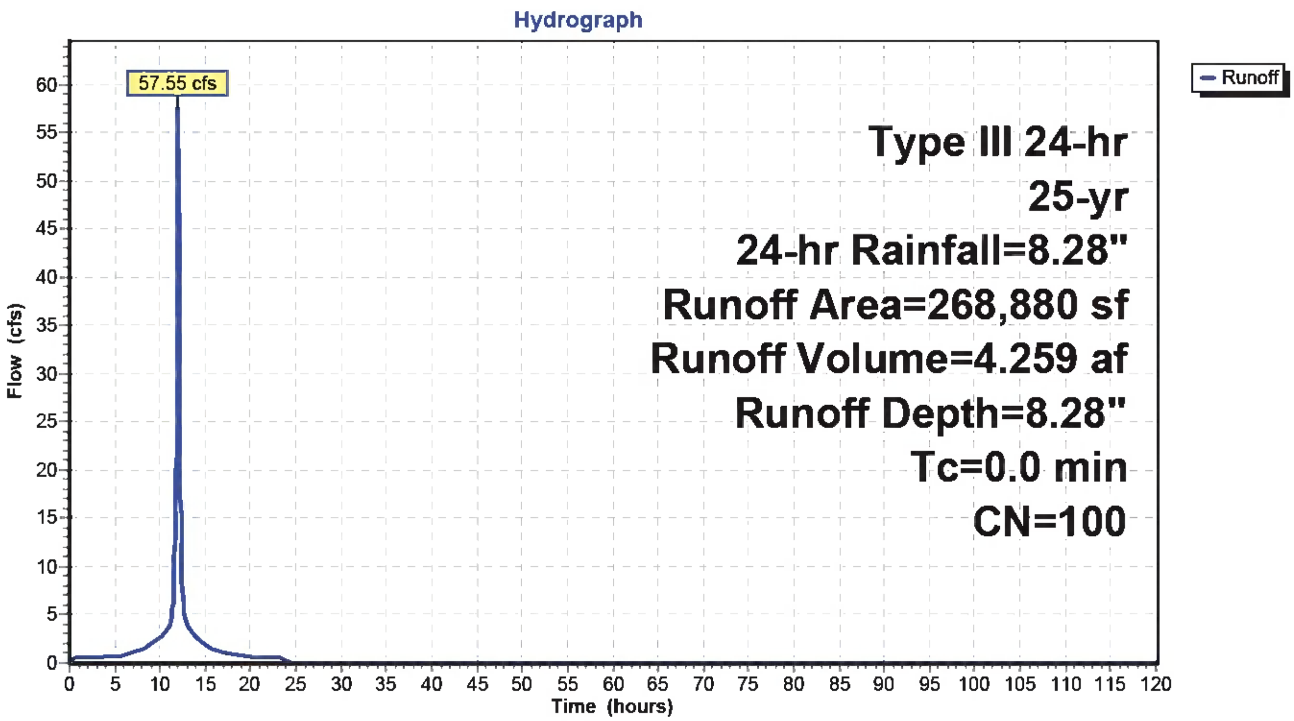
Runoff = 57.55 cfs @ 12.00 hrs, Volume= 4.259 af, Depth= 8.28"

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

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

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

Subcatchment S2: Leachate Pond



Summary for Pond P1: Contact Water Cells 4/5

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

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

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

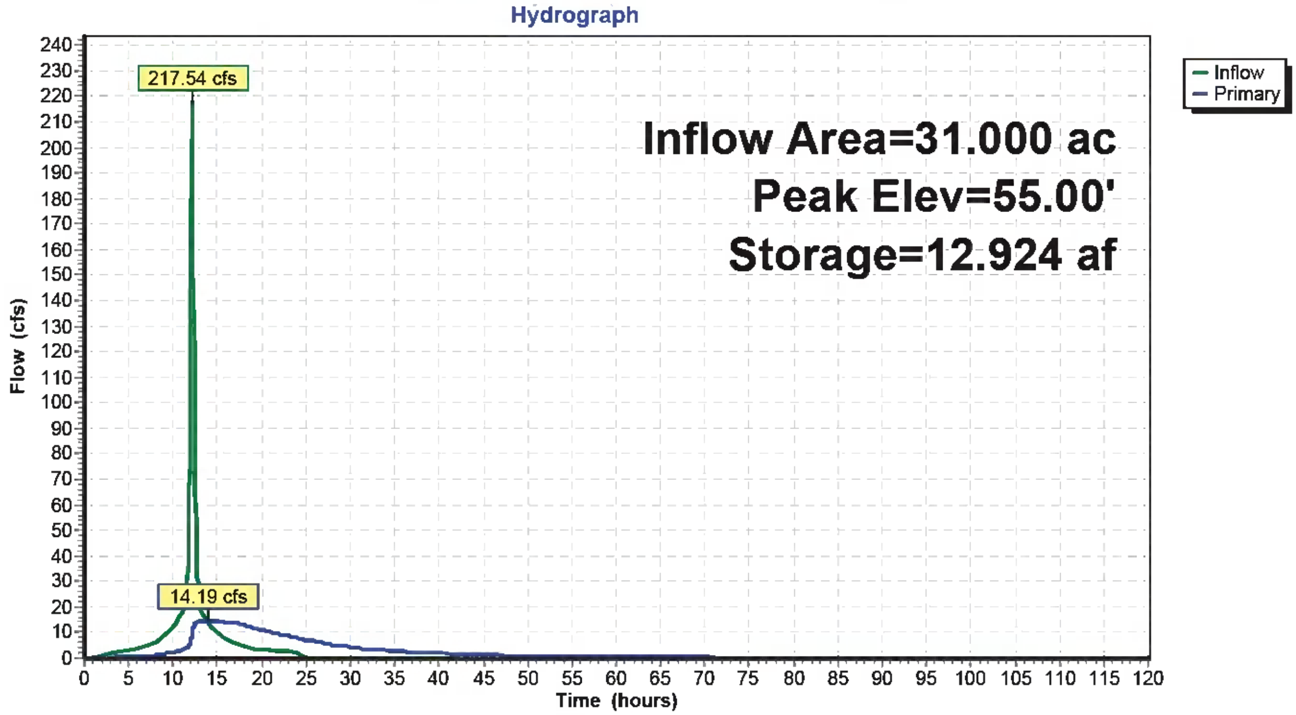
Volume	Invert	Avail.Storage	Storage Description
#1	51.00'	74.999 af	**Custom Stage Data (Conic)** Listed below (Recalc)

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

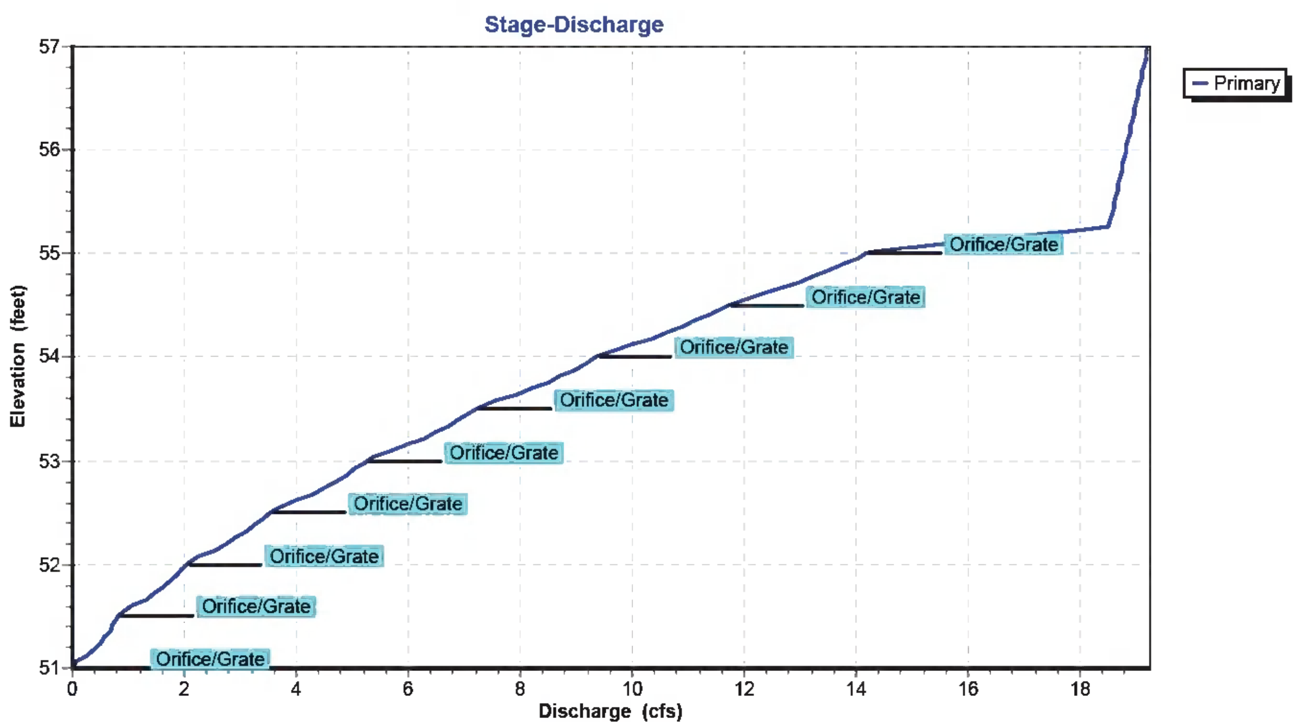
Device	Routin	Invert	Outlet Devices
#1	Primary	36.00'	**18.0" Round Culvert** L= 1,300.0' CPP, projecting, no headwall, Ke= 0.900 Inlet / Outlet Invert= 36.00' / 29.50' S= 0.0050 '/' Cc= 0.900 n= 0.009, Flow Area= 1.77 sf
#2	Device 1	51.00'	**2.0" Vert. Orifice/Grate X 12.00** C= 0.600
#3	Device 1	51.50'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

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

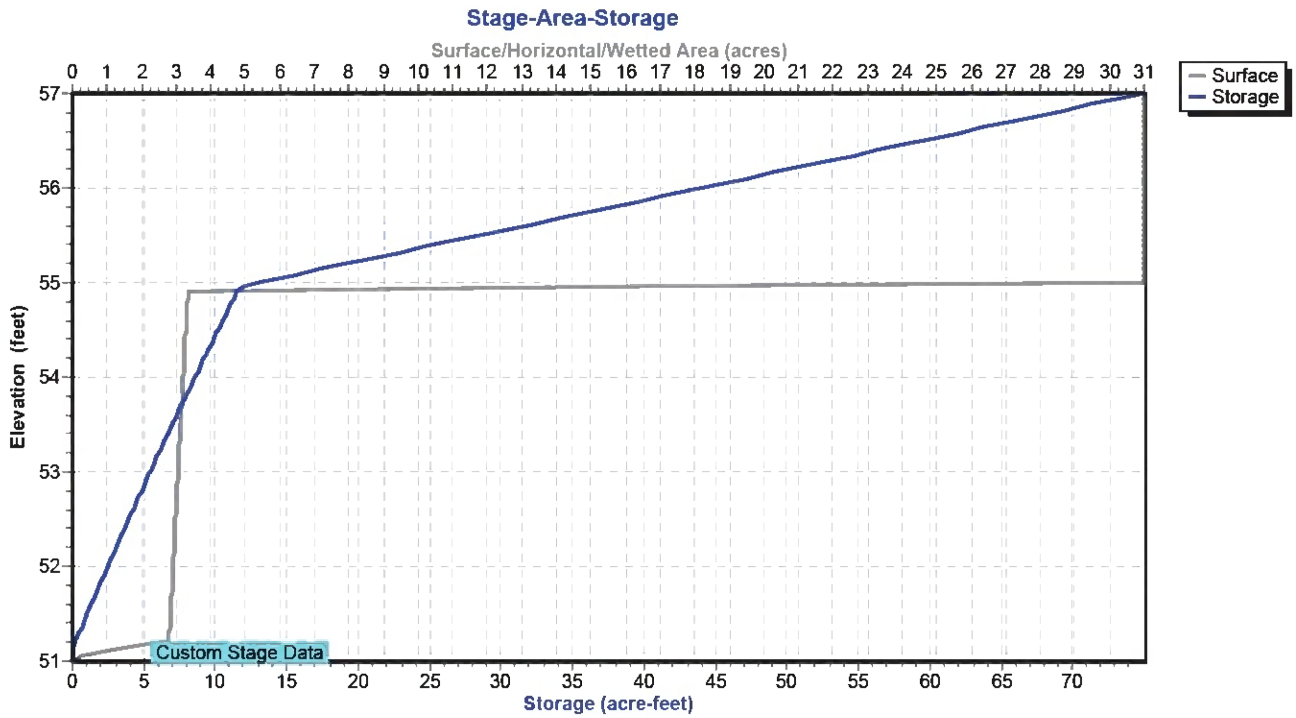
Pond P1: Contact Water Cells 4/5



Pond P1: Contact Water Cells 4/5



Pond P1: Contact Water Cells 4/5



Summary for Pond P2: Leachate Pond

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

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

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

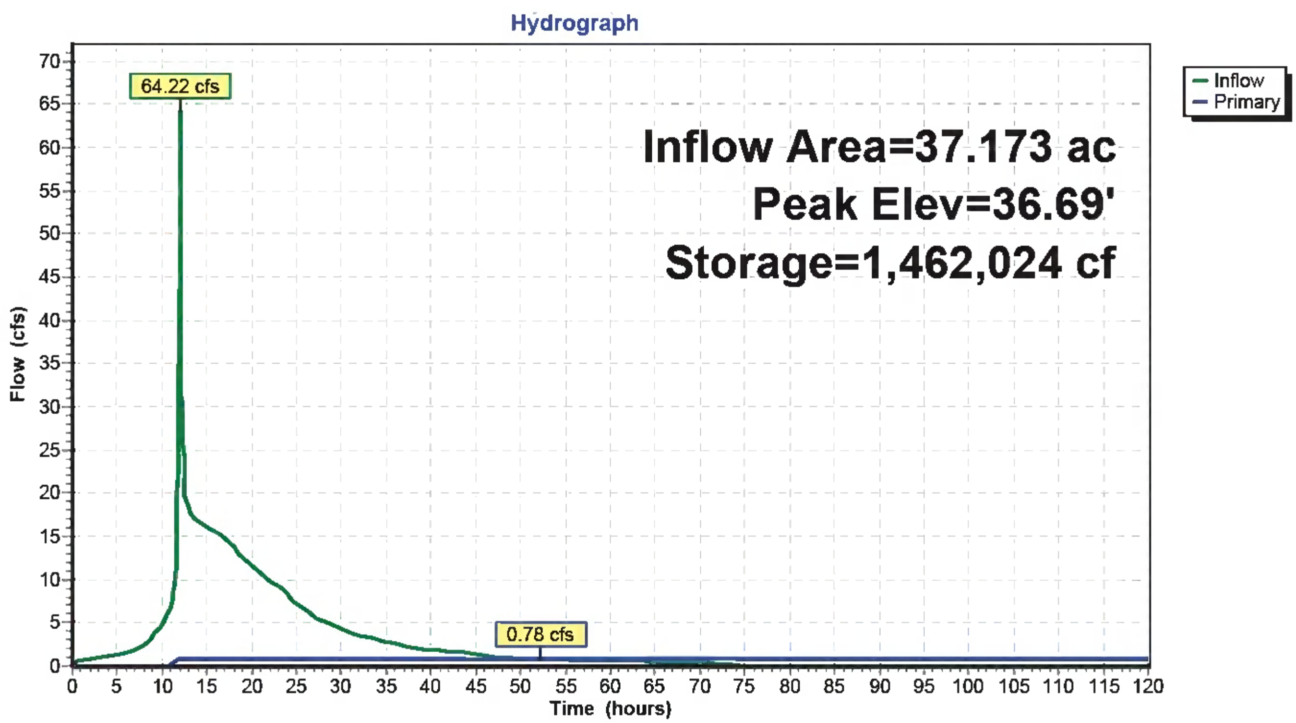
Volume	Invert	Avail.Storage	Storage Description
#1	27.00'	2,564,304 cf	**Custom Stage Data (Conic)**Listed below (Recalc)

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

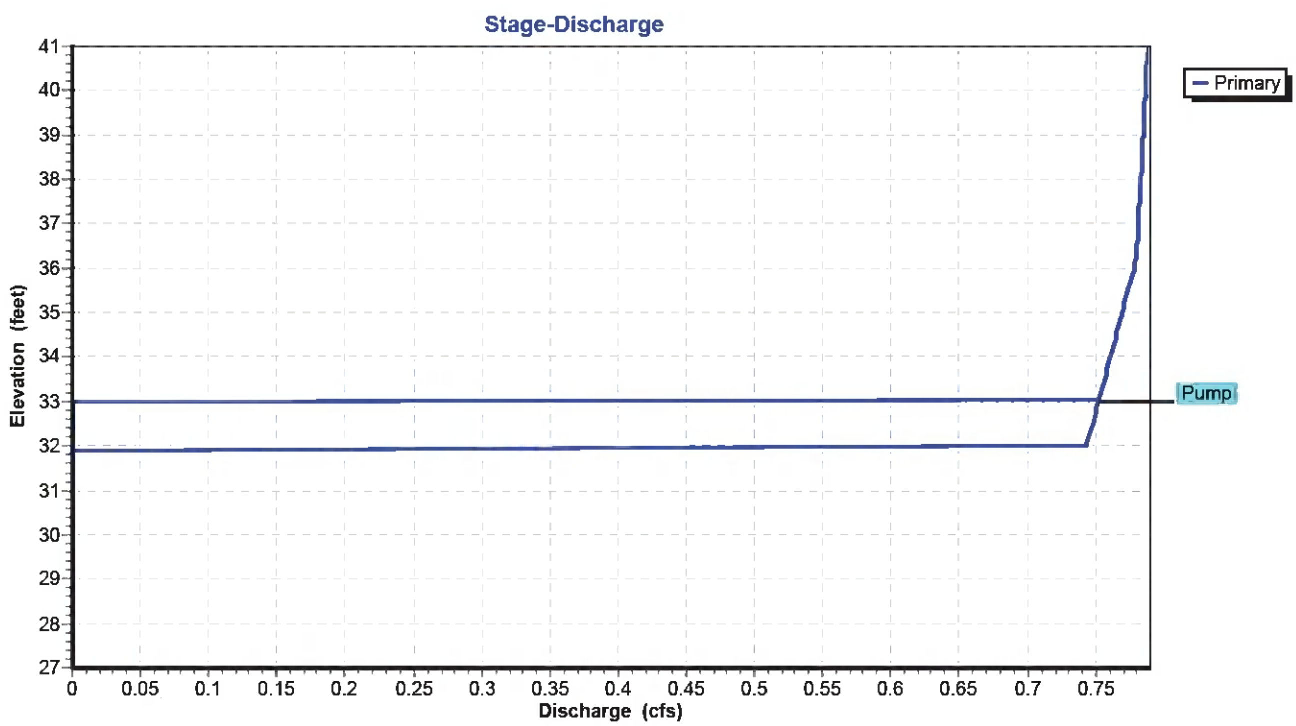
Device	Routin	Invert	Outlet Devices
#1	Primary	33.00'	**Pump** Discharges@41.00' Turns Off@32.00' 6.0" Diam. x 300.0' Long Discharge, Hazen-Williams C= 130 Flow (gpm)= 40.0 100.0 200.0 250.0 300.0 350.0 355.0 Head (feet)= 47.00 40.00 30.00 26.00 20.00 8.00 2.00 -Loss (feet)= 0.06 0.31 1.13 1.71 2.39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

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

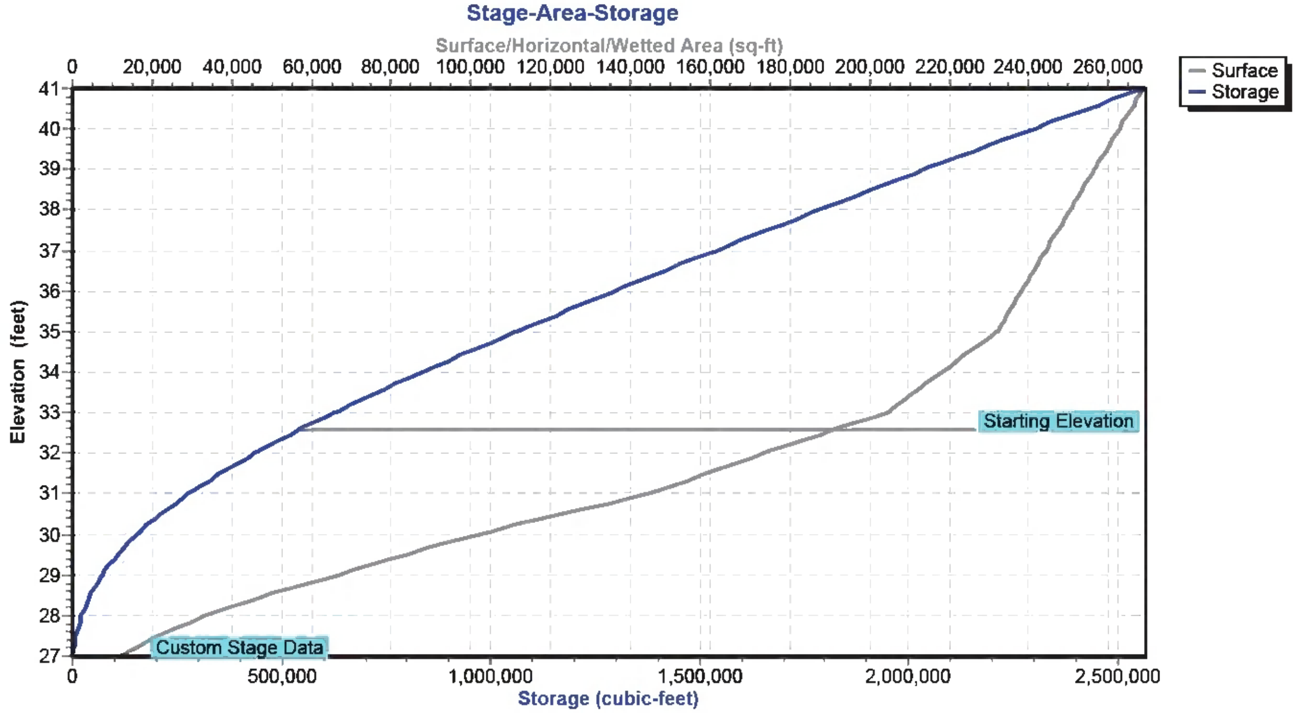
Pond P2: Leachate Pond



Pond P2: Leachate Pond



Pond P2: Leachate Pond



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

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

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

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

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

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

Summary for Subcatchment S1: Cells 4/5

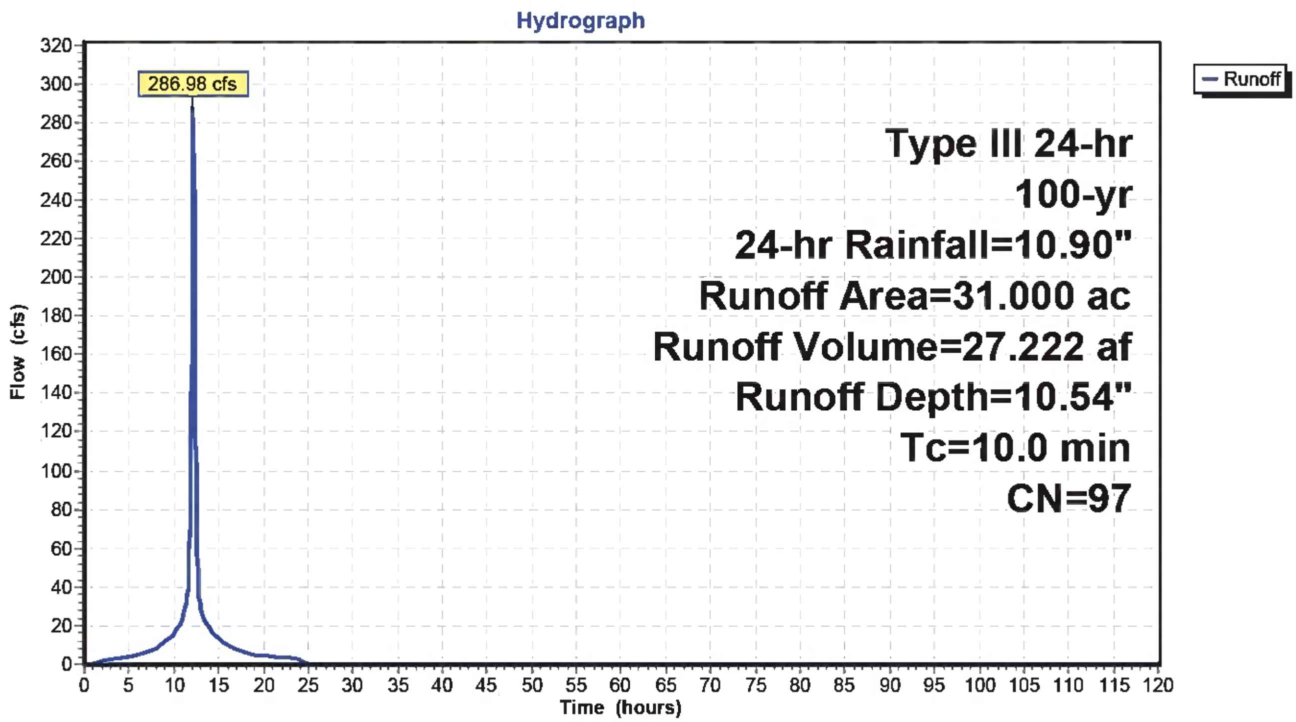
Runoff = 286.98 cfs @ 12.14 hrs, Volume= 27.222 af, Depth=10.54"

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

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

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

Subcatchment S1: Cells 4/5



Summary for Subcatchment S2: Leachate Pond

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

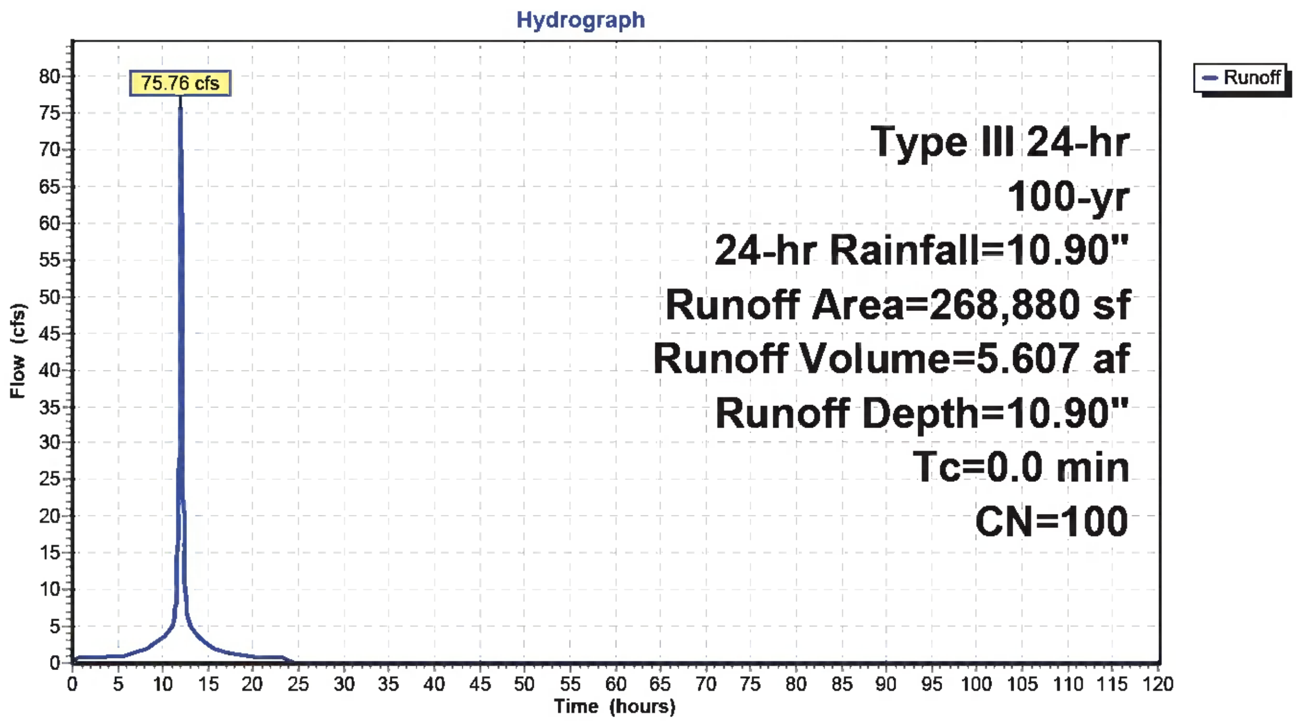
Runoff = 75.76 cfs @ 12.00 hrs, Volume= 5.607 af, Depth=10.90"

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

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

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

Subcatchment S2: Leachate Pond



Summary for Pond P1: Contact Water Cells 4/5

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

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

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

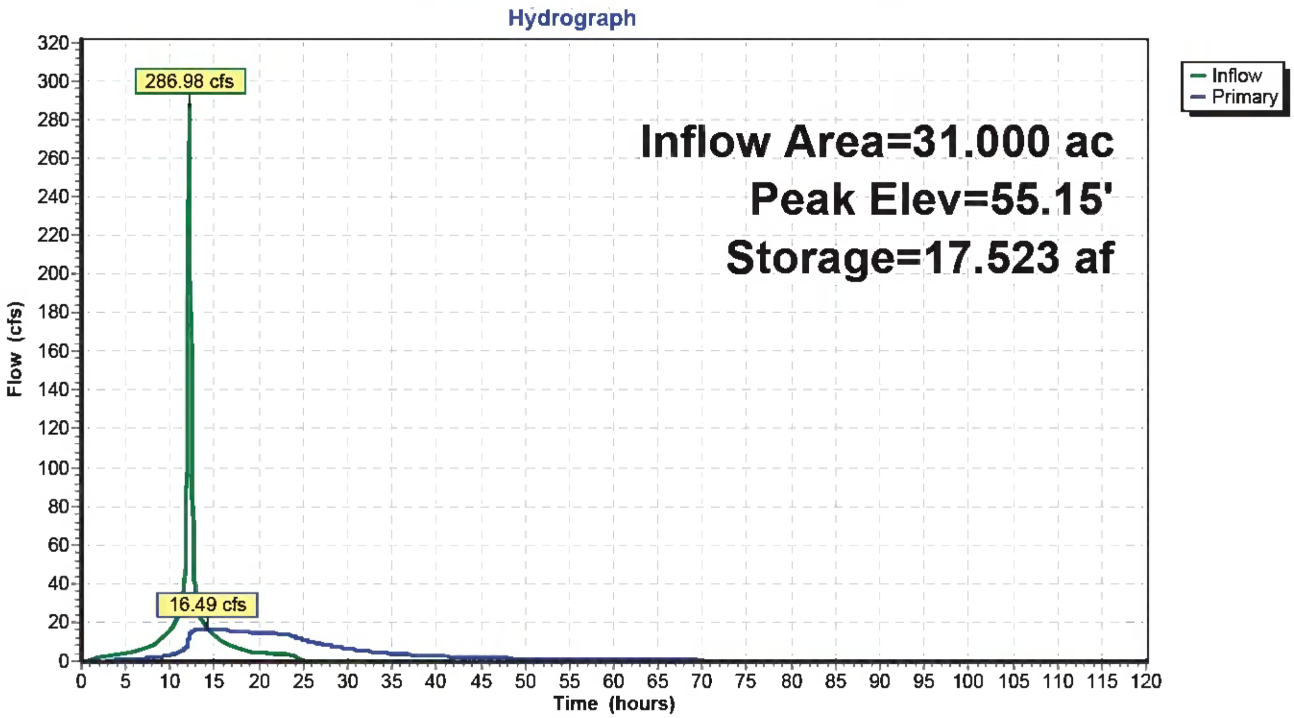
Volume	Invert	Avail.Storage	Storage Description
#1	51.00'	74.999 af	**Custom Stage Data (Conic)**Listed below (Recalc)

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

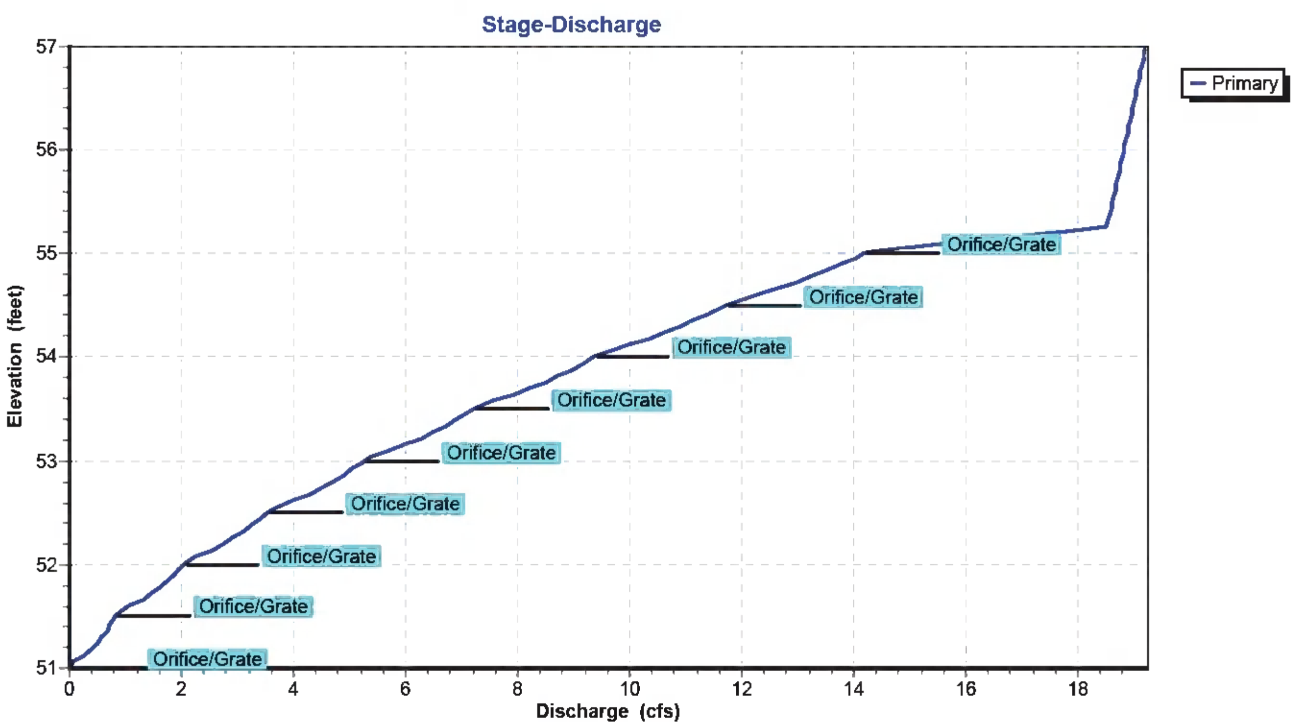
Device	Routin	Invert	Outlet Devices
#1	Primary	36.00'	**18.0" Round Culvert** L= 1,300.0' CPP, projecting, no headwall, Ke= 0.900 Inlet / Outlet Invert= 36.00' / 29.50' S= 0.0050 '/' Cc= 0.900 n= 0.009, Flow Area= 1.77 sf
#2	Device 1	51.00'	**2.0" Vert. Orifice/Grate X 12.00** C= 0.600
#3	Device 1	51.50'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

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

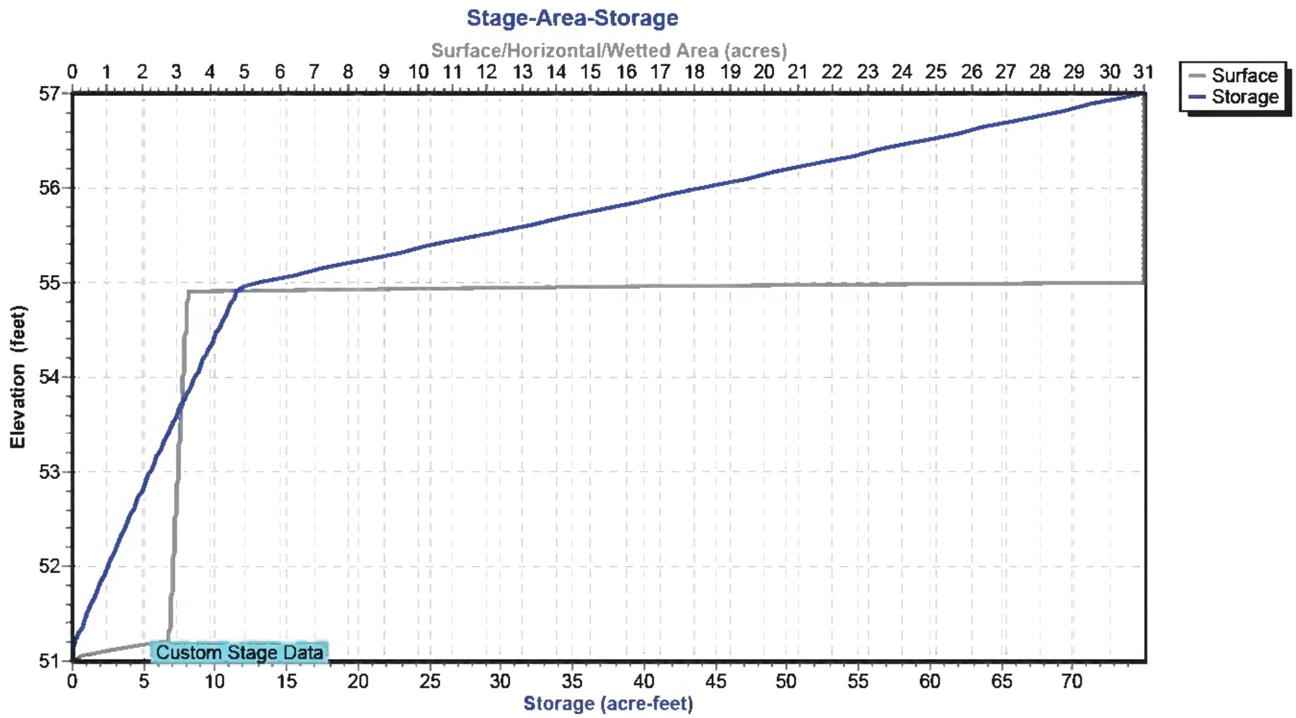
Pond P1: Contact Water Cells 4/5



Pond P1: Contact Water Cells 4/5



Pond P1: Contact Water Cells 4/5



Summary for Pond P2: Leachate Pond

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

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

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

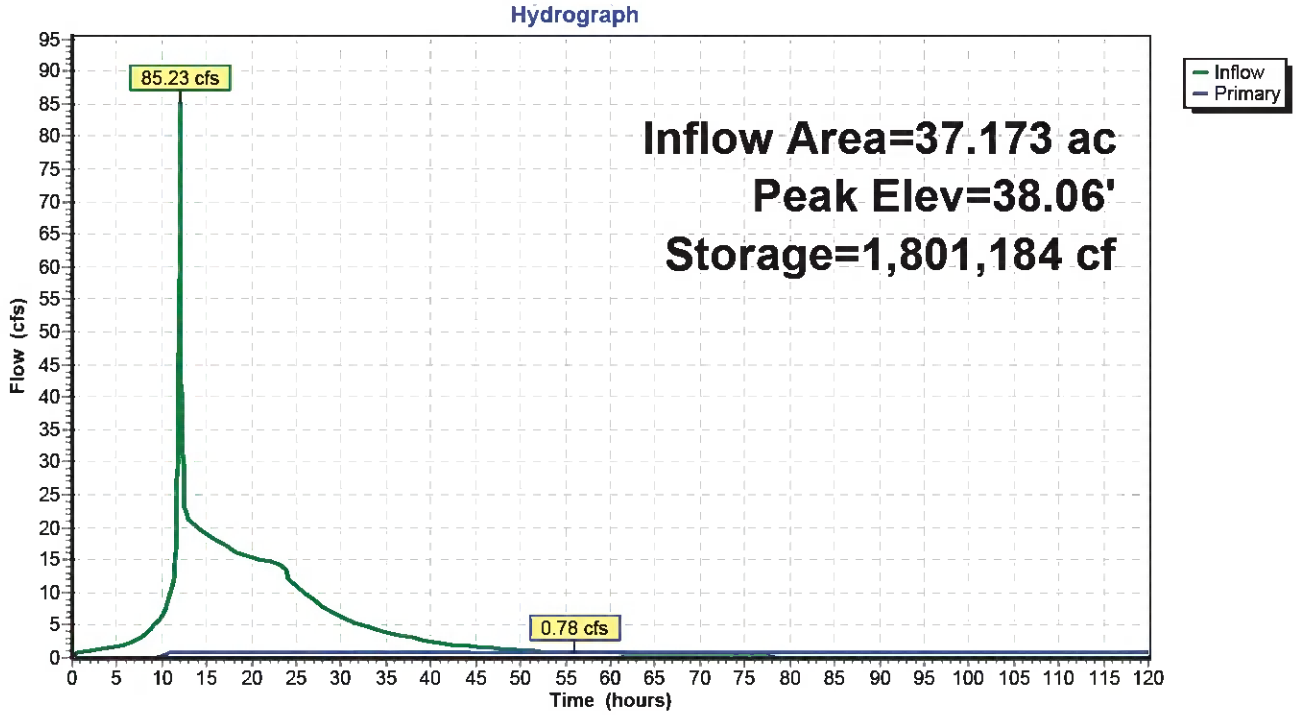
Volume	Invert	Avail.Storage	Storage Description
#1	27.00'	2,564,304 cf	**Custom Stage Data (Conic)**Listed below (Recalc)

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

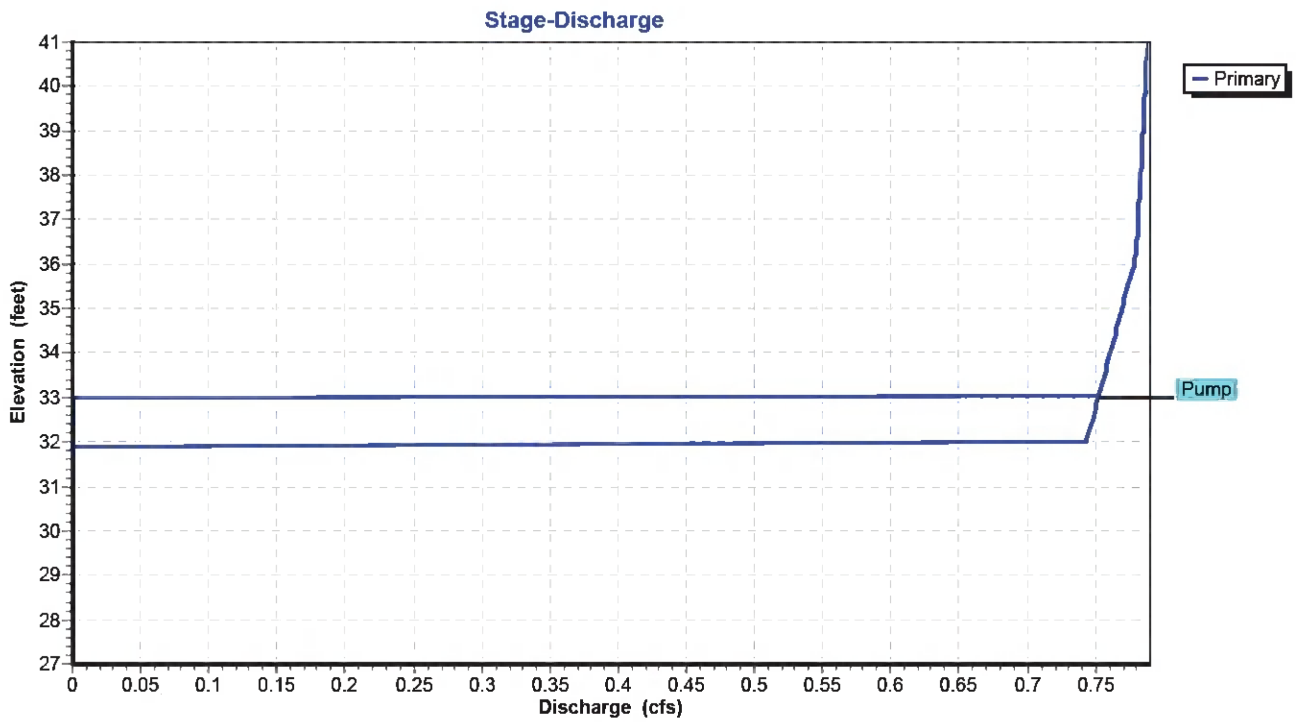
Device	Routin	Invert	Outlet Devices
#1	Primary	33.00'	**Pump** Discharges@41.00' Turns Off@32.00' 6.0" Diam. x 300.0' Long Discharge, Hazen-Williams C= 130 Flow (gpm)= 40.0 100.0 200.0 250.0 300.0 350.0 355.0 Head (feet)= 47.00 40.00 30.00 26.00 20.00 8.00 2.00 -Loss (feet)= 0.06 0.31 1.13 1.71 2.39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

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

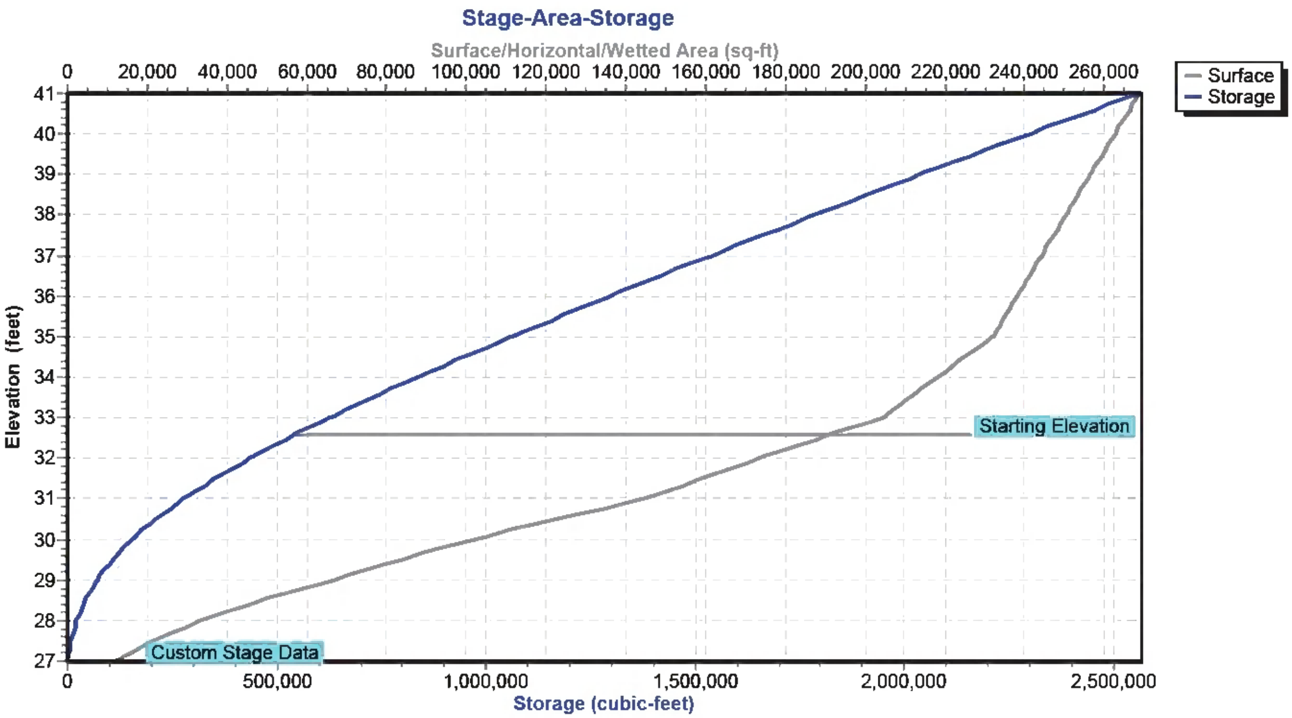
Pond P2: Leachate Pond



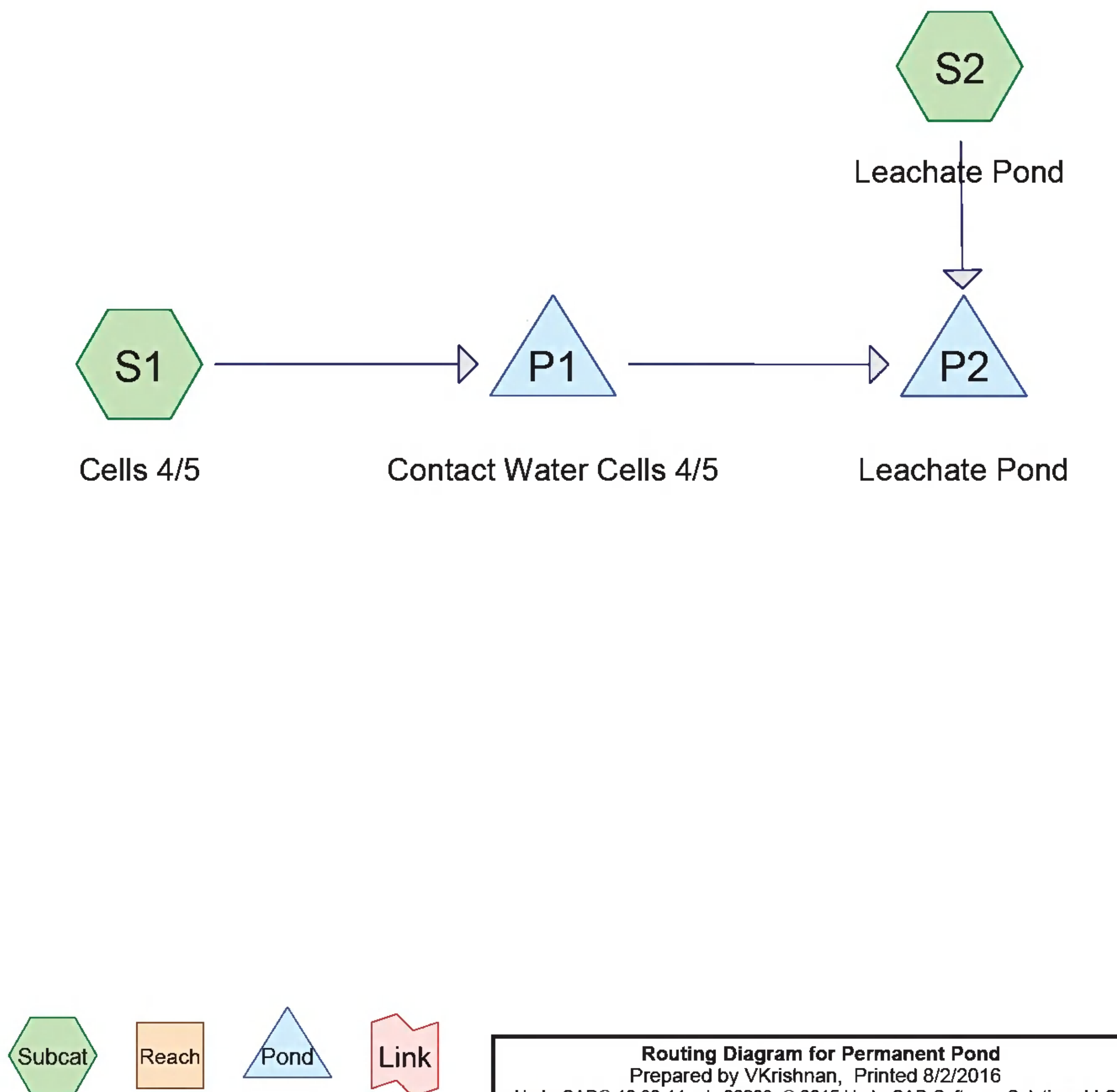
Pond P2: Leachate Pond



Pond P2: Leachate Pond



PERMANENT LEACHATE POND



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

Area Listing (all nodes)

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

Soil Listing (all nodes)

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

Ground Covers (all nodes)

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

Pipe Listing (all nodes)

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

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

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

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

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

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

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

Summary for Subcatchment S1: Cells 4/5

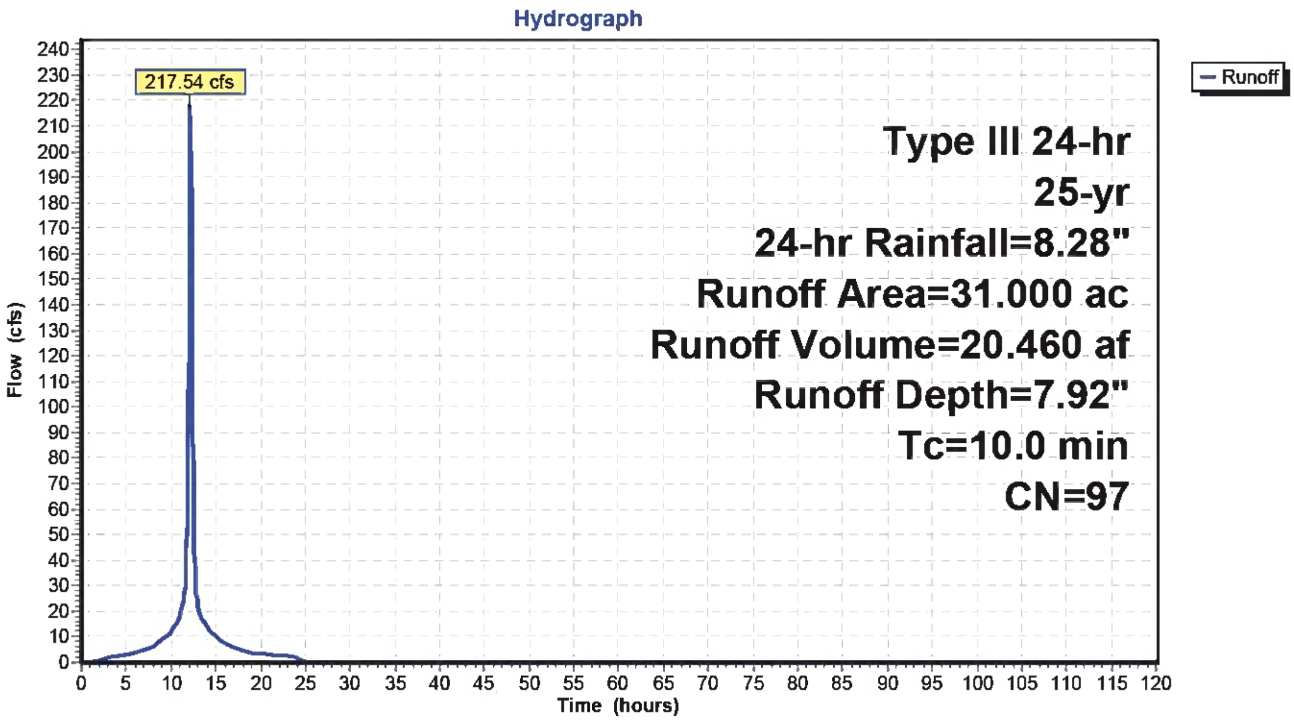
Runoff = 217.54 cfs @ 12.14 hrs, Volume= 20.460 af, Depth= 7.92"

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

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

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

Subcatchment S1: Cells 4/5



Summary for Subcatchment S2: Leachate Pond

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

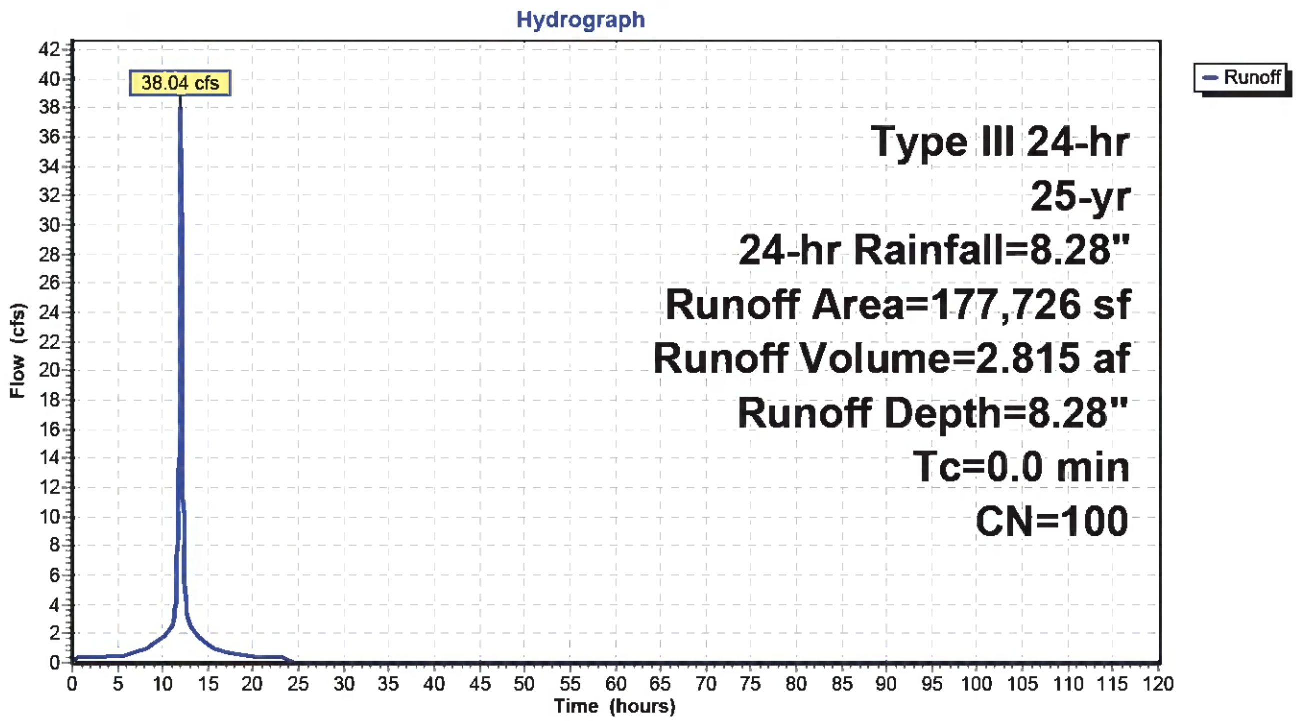
Runoff = 38.04 cfs @ 12.00 hrs, Volume= 2.815 af, Depth= 8.28"

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

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

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

Subcatchment S2: Leachate Pond



Summary for Pond P1: Contact Water Cells 4/5

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

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

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

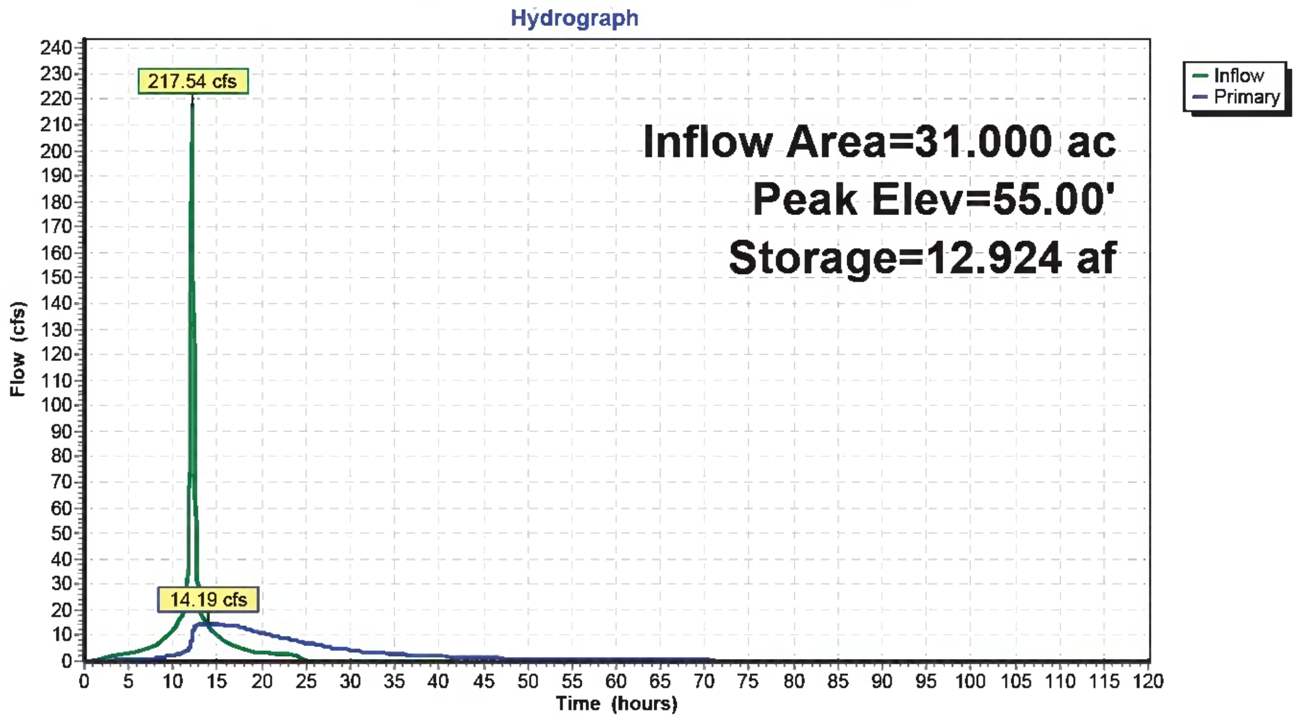
Volume	Invert	Avail.Storage	Storage Description
#1	51.00'	74.999 af	**Custom Stage Data (Conic)** Listed below (Recalc)

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

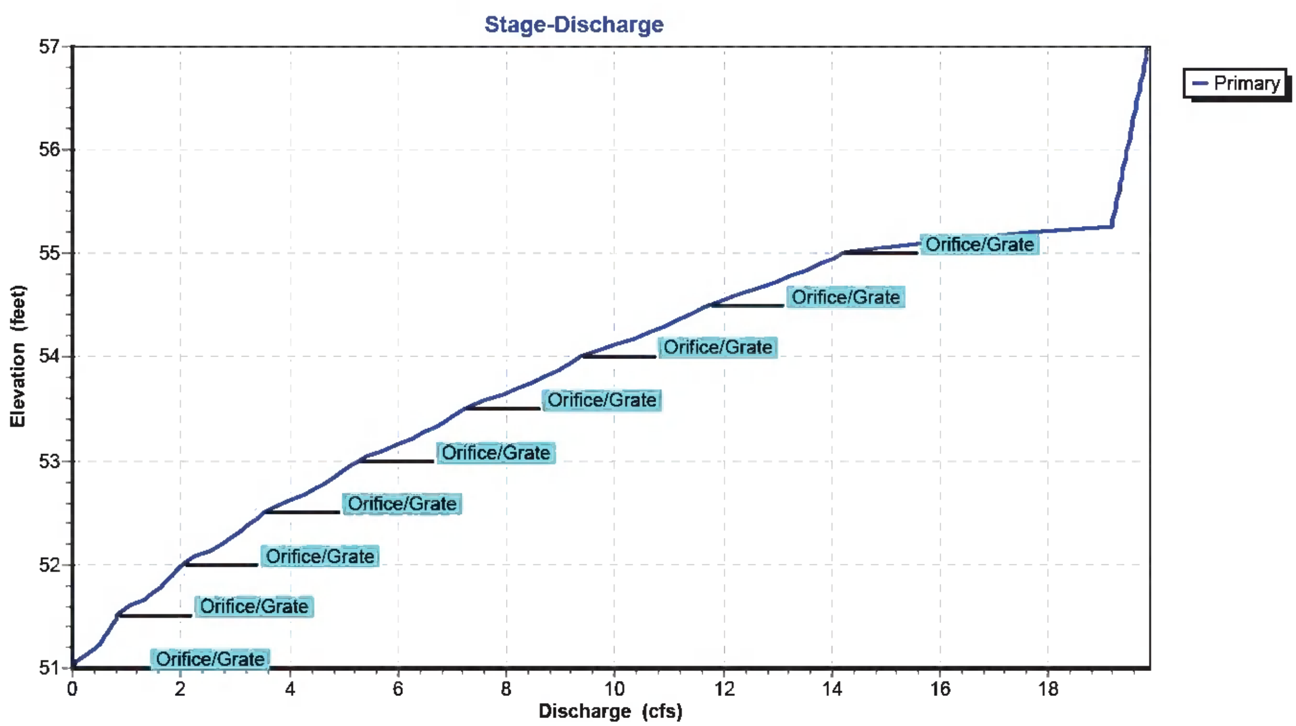
Device	Routing	Invert	Outlet Devices
#1	Primary	36.00'	**18.0" Round Culvert** L= 1,300.0' CPP, projecting, no headwall, Ke= 0.900 Inlet / Outlet Invert= 36.00' / 29.50' S= 0.0050 '/' Cc= 0.900 n= 0.009, Flow Area= 1.77 sf
#2	Device 1	51.00'	**2.0" Vert. Orifice/Grate X 12.00** C= 0.600
#3	Device 1	51.50'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

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

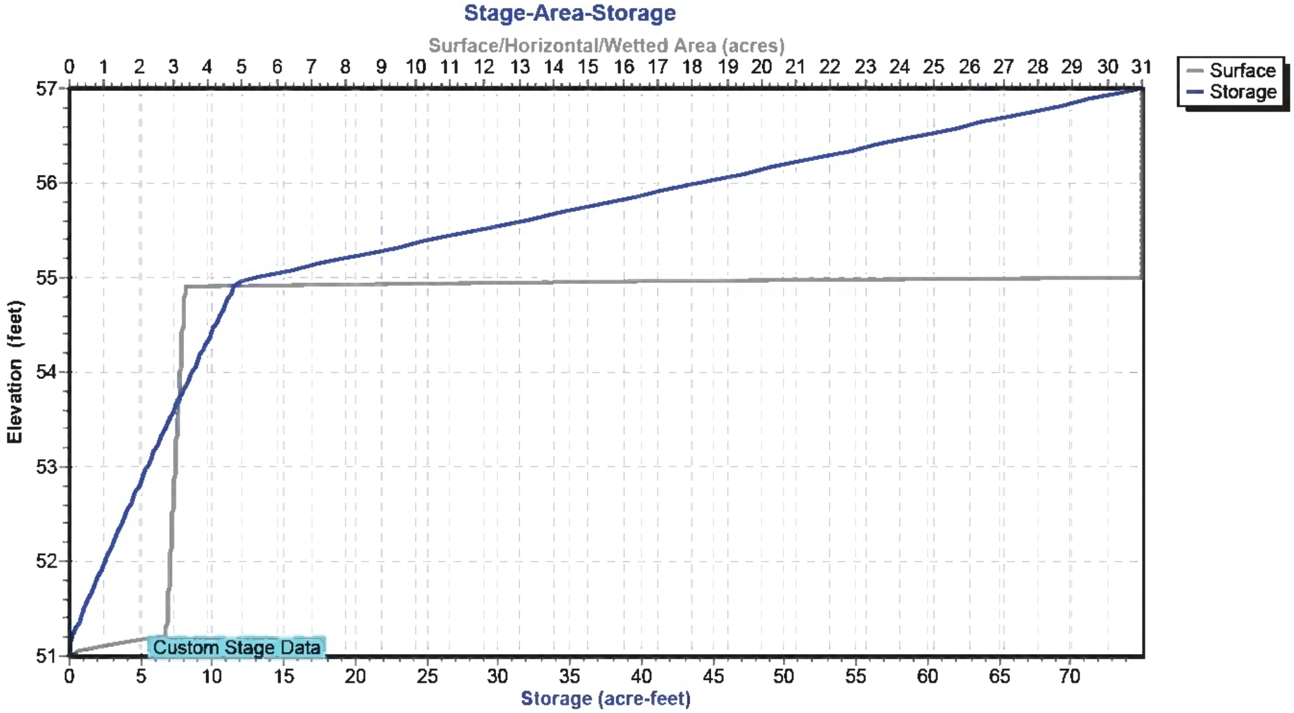
Pond P1: Contact Water Cells 4/5



Pond P1: Contact Water Cells 4/5



Pond P1: Contact Water Cells 4/5



Summary for Pond P2: Leachate Pond

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

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

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

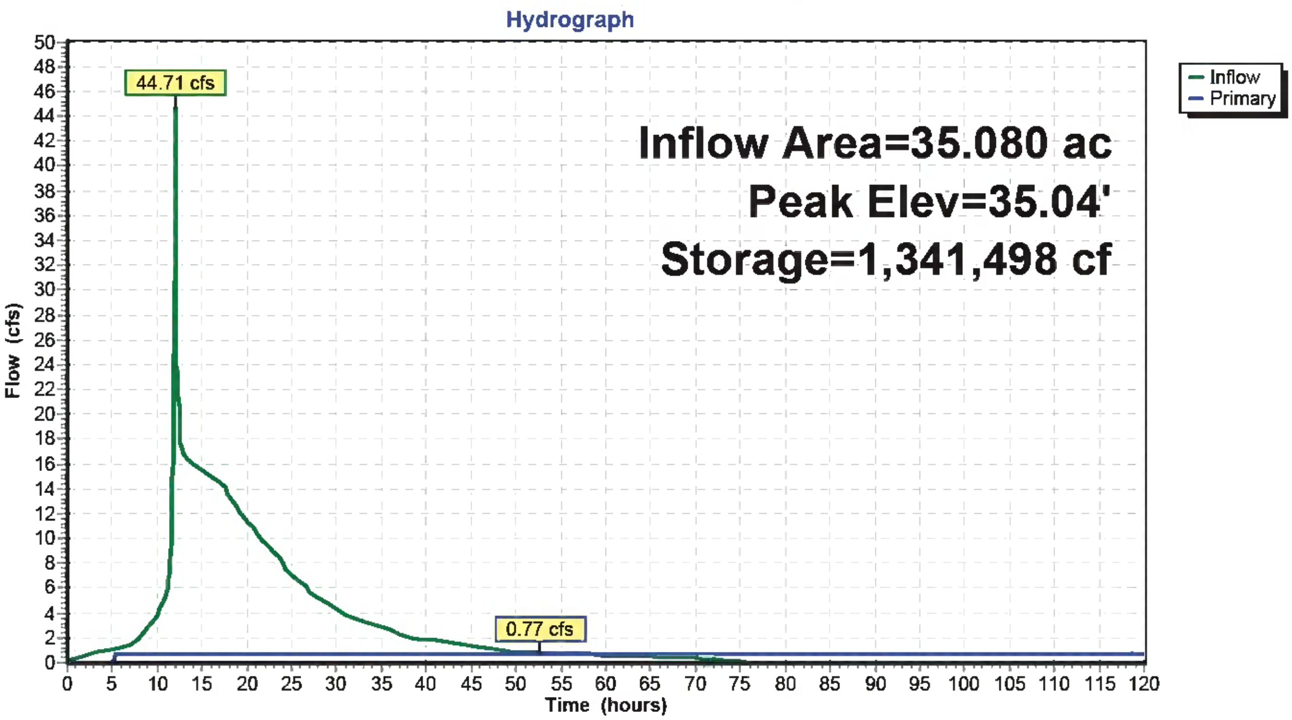
Volume	Invert	Avail.Storage	Storage Description
#1	24.00'	2,158,865 cf	**Custom Stage Data (Conic)**Listed below (Recalc)

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

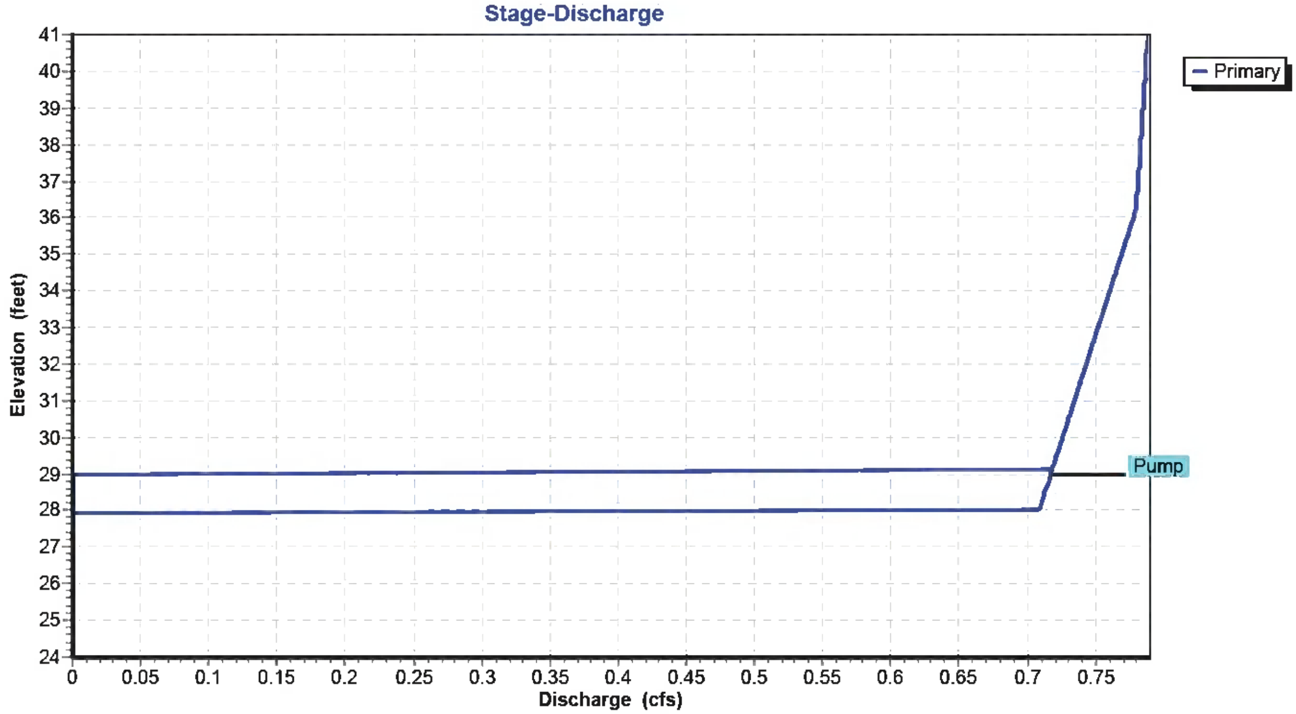
Device	Routing	Invert	Outlet Devices
#1	Primary	29.00'	**Pump** Discharges@41.00' Turns Off@28.00' 6.0" Diam. x 300.0' Long Discharge, Hazen-Williams C= 130 Flow (gpm)= 40.0 100.0 200.0 250.0 300.0 350.0 355.0 Head (feet)= 47.00 40.00 30.00 26.00 20.00 8.00 2.00 -Loss (feet)= 0.06 0.31 1.13 1.71 2.39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

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

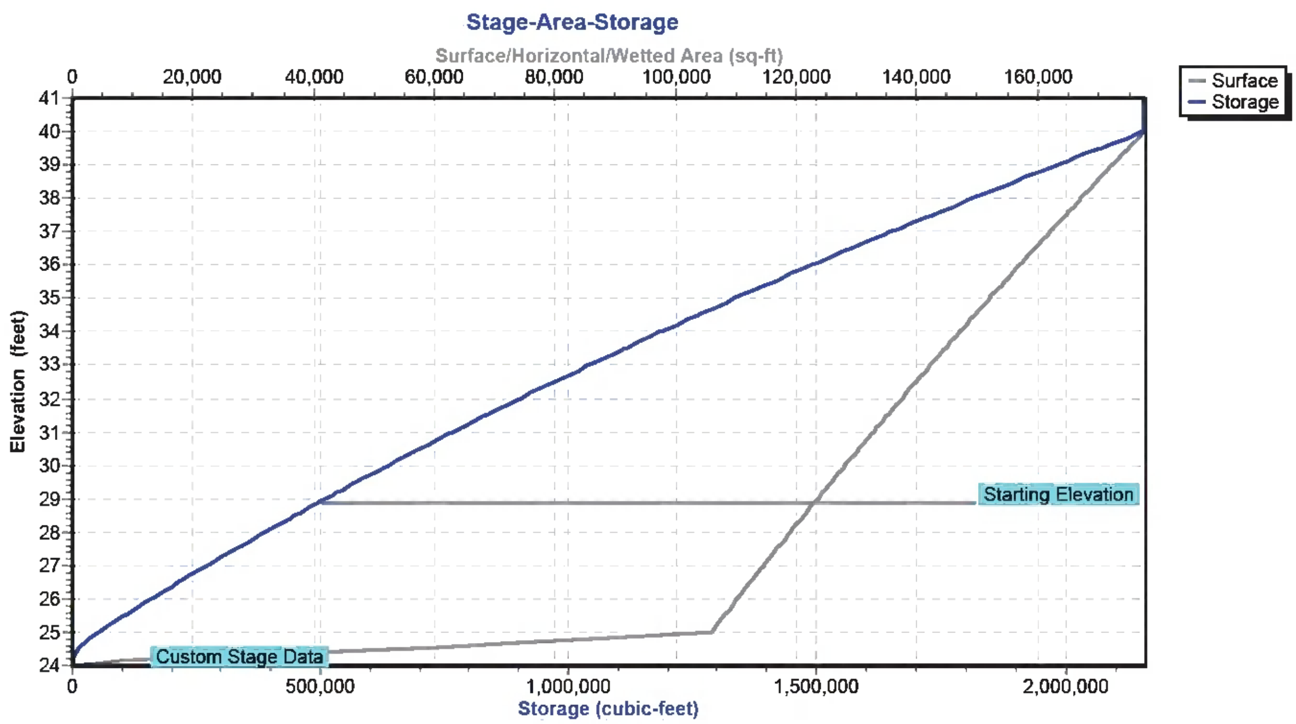
Pond P2: Leachate Pond



Pond P2: Leachate Pond



Pond P2: Leachate Pond



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

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

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

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

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

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

Summary for Subcatchment S1: Cells 4/5

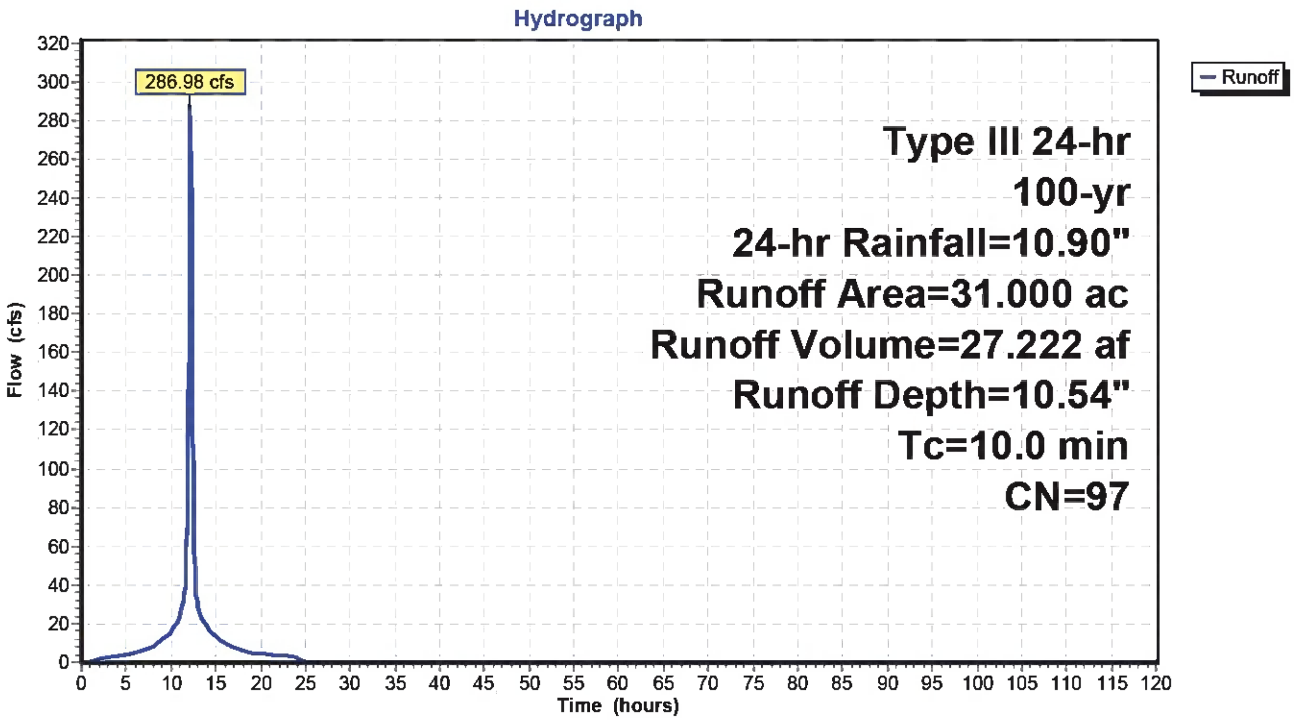
Runoff = 286.98 cfs @ 12.14 hrs, Volume= 27.222 af, Depth=10.54"

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

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

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

Subcatchment S1: Cells 4/5



Summary for Subcatchment S2: Leachate Pond

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

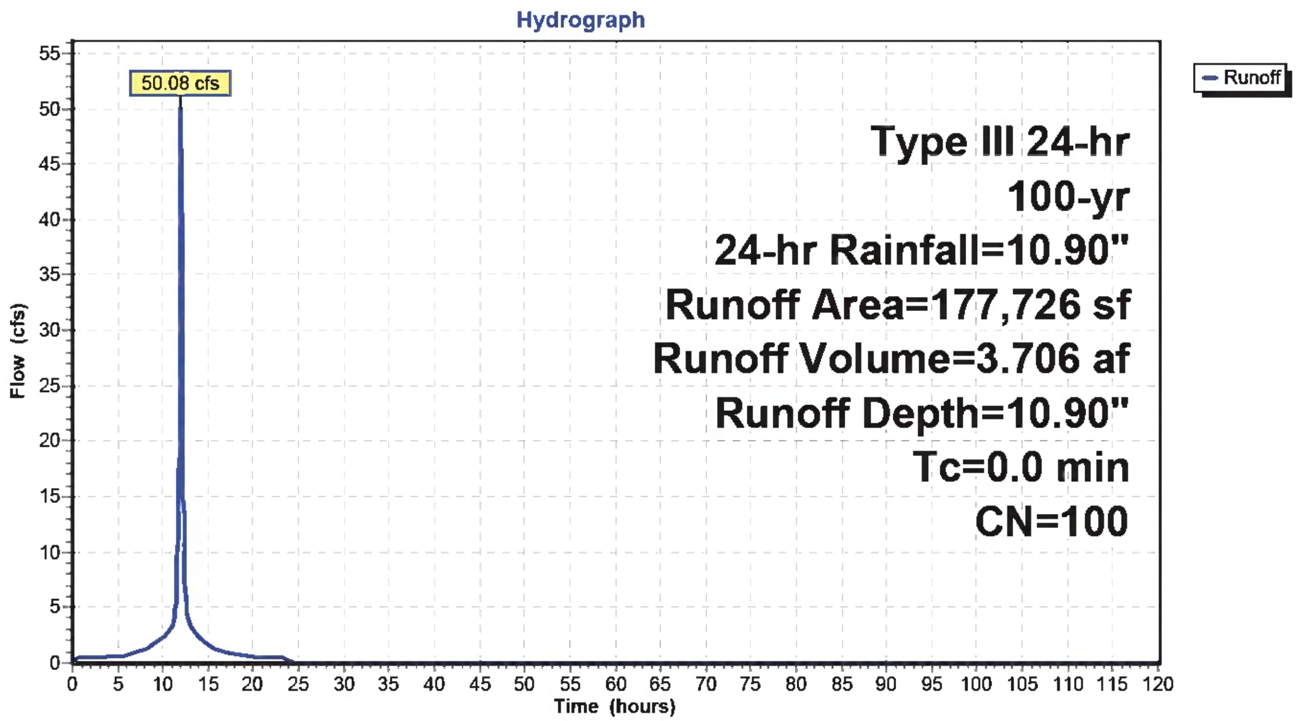
Runoff = 50.08 cfs @ 12.00 hrs, Volume= 3.706 af, Depth=10.90"

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

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

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

Subcatchment S2: Leachate Pond



Summary for Pond P1: Contact Water Cells 4/5

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

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

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

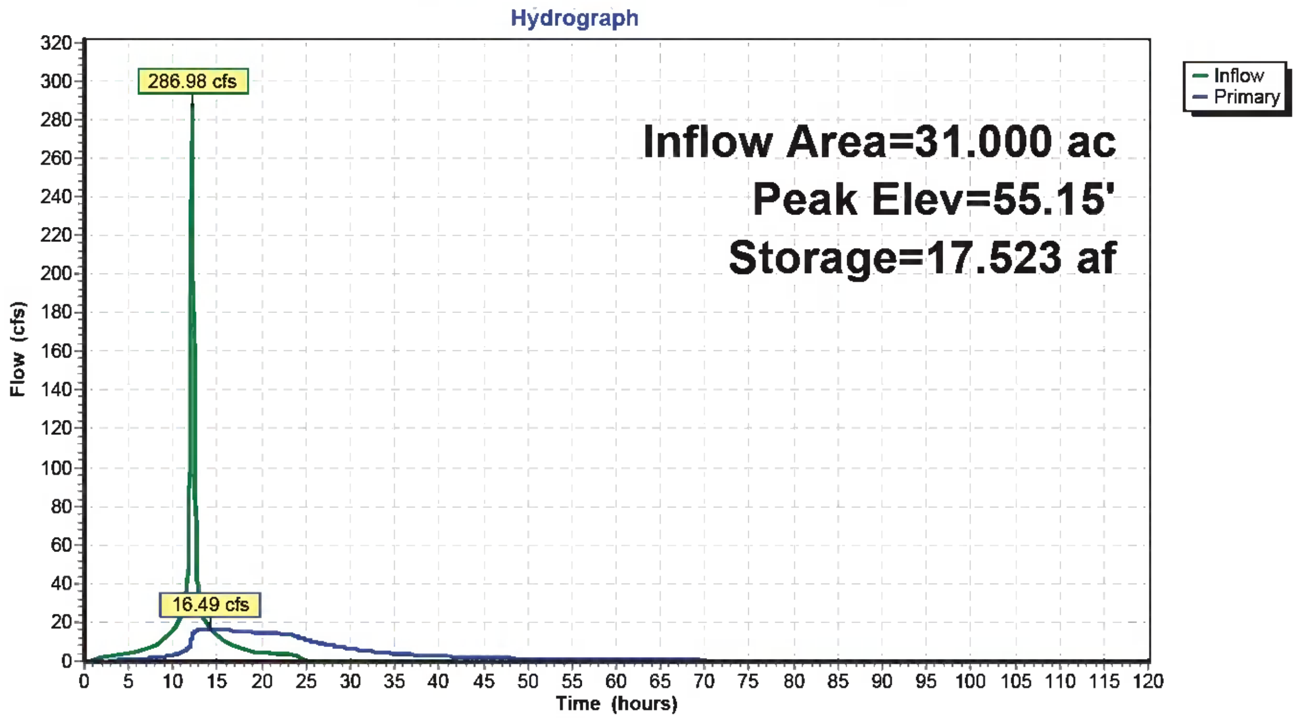
Volume	Invert	Avail.Storage	Storage Description
#1	51.00'	74.999 af	**Custom Stage Data (Conic)** Listed below (Recalc)

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

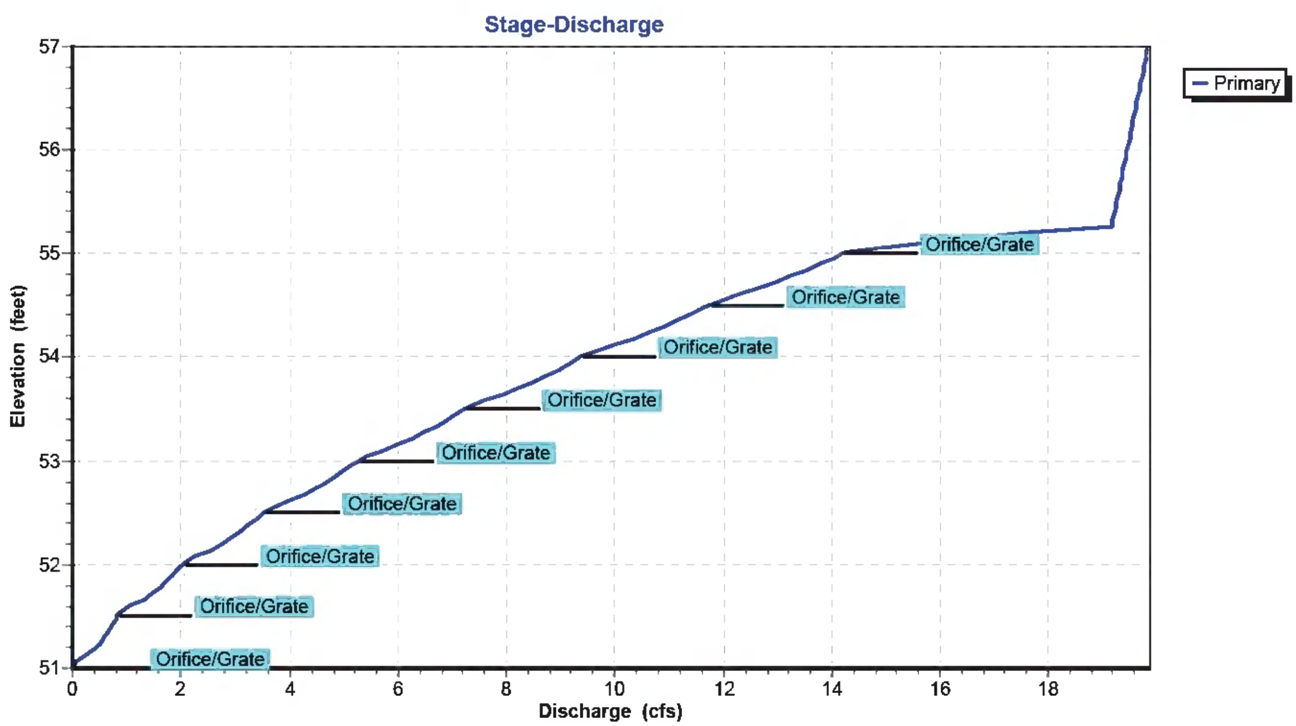
Device	Routing	Invert	Outlet Devices
#1	Primary	36.00'	**18.0" Round Culvert** L= 1,300.0' CPP, projecting, no headwall, Ke= 0.900 Inlet / Outlet Invert= 36.00' / 29.50' S= 0.0050 '/' Cc= 0.900 n= 0.009, Flow Area= 1.77 sf
#2	Device 1	51.00'	**2.0" Vert. Orifice/Grate X 12.00** C= 0.600
#3	Device 1	51.50'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

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

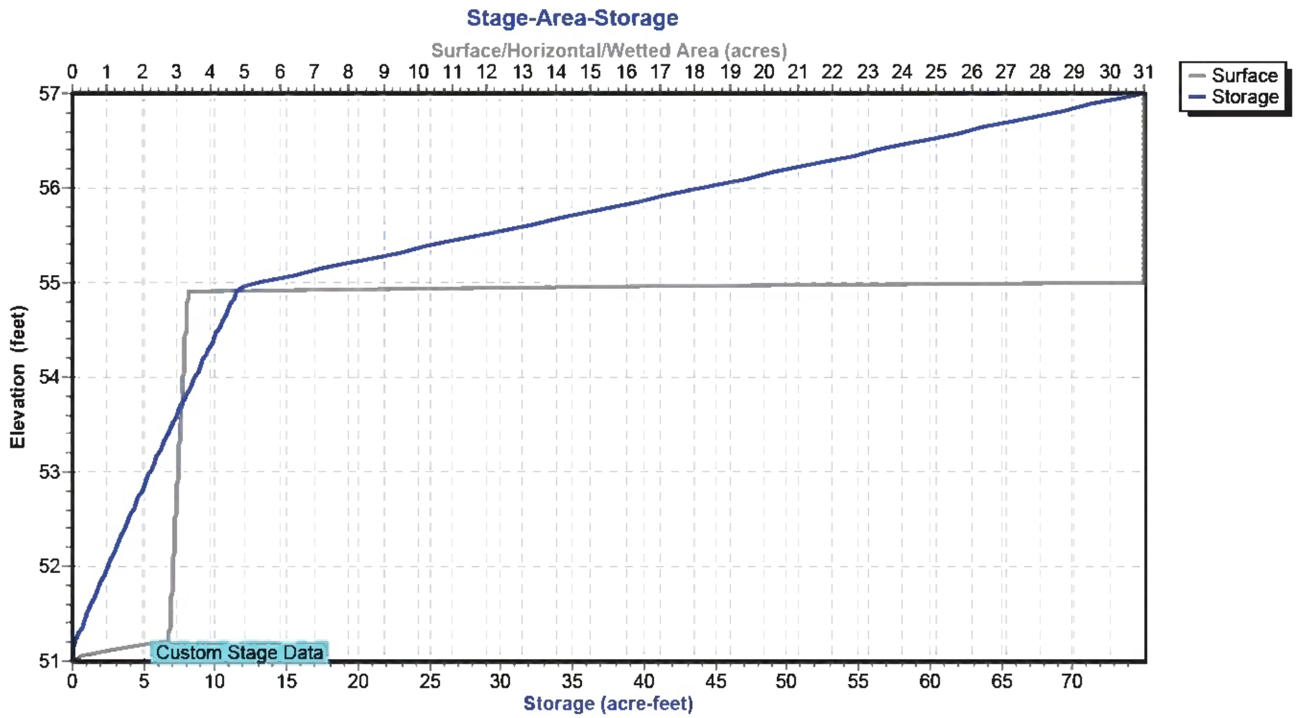
Pond P1: Contact Water Cells 4/5



Pond P1: Contact Water Cells 4/5



Pond P1: Contact Water Cells 4/5



Summary for Pond P2: Leachate Pond

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

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

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

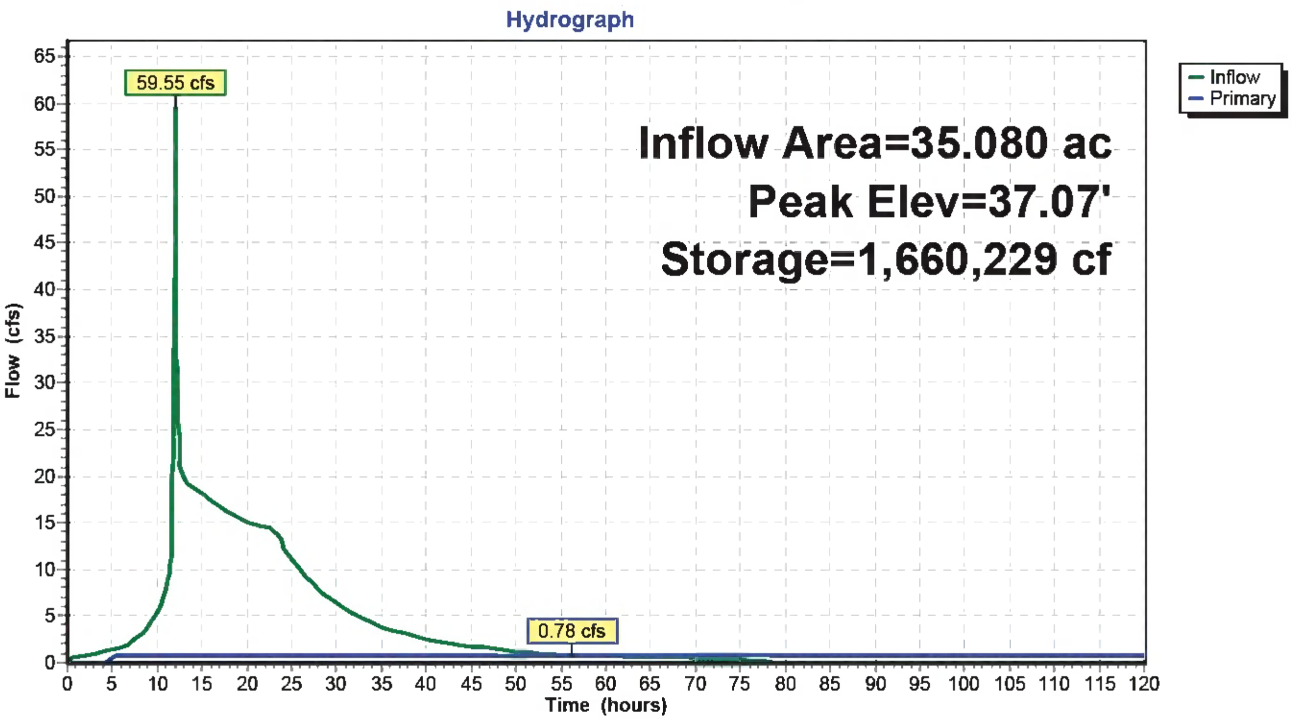
Volume	Invert	Avail.Storage	Storage Description
#1	24.00'	2,158,865 cf	**Custom Stage Data (Conic)**Listed below (Recalc)

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

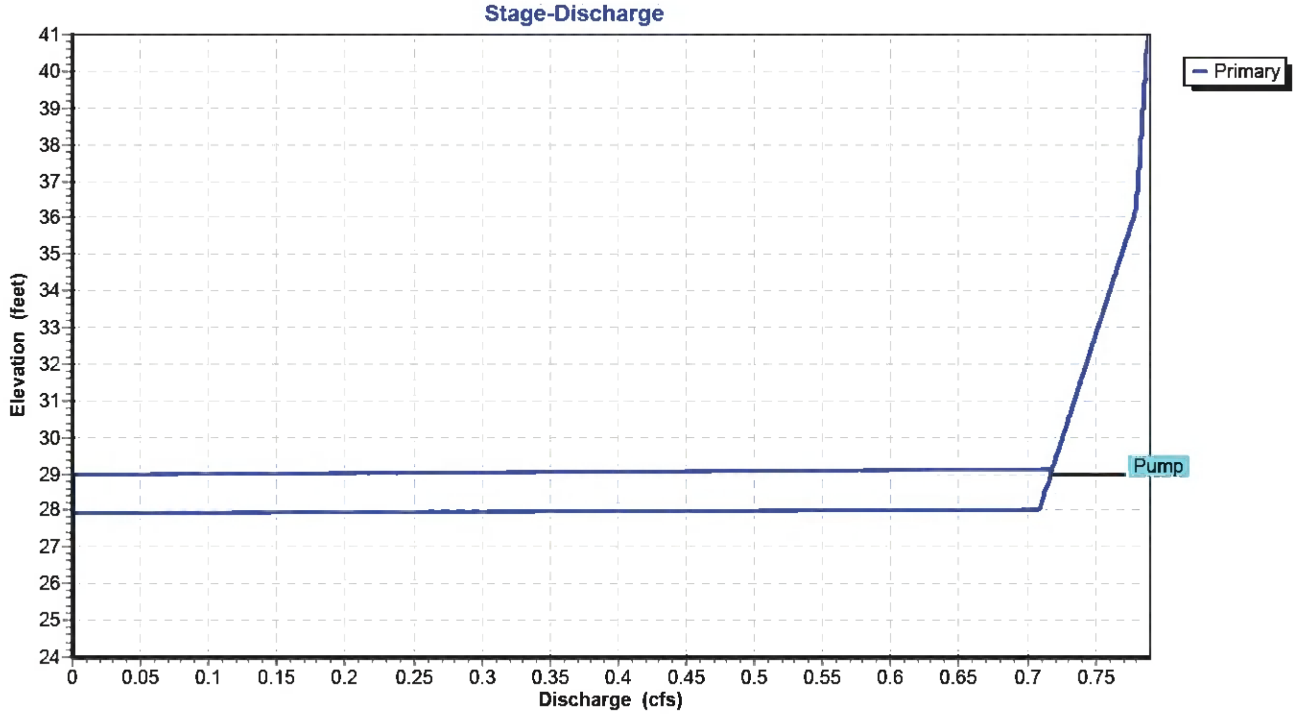
Device	Routing	Invert	Outlet Devices
#1	Primary	29.00'	**Pump** Discharges@41.00' Turns Off@28.00' 6.0" Diam. x 300.0' Long Discharge, Hazen-Williams C= 130 Flow (gpm)= 40.0 100.0 200.0 250.0 300.0 350.0 355.0 Head (feet)= 47.00 40.00 30.00 26.00 20.00 8.00 2.00 -Loss (feet)= 0.06 0.31 1.13 1.71 2.39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

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

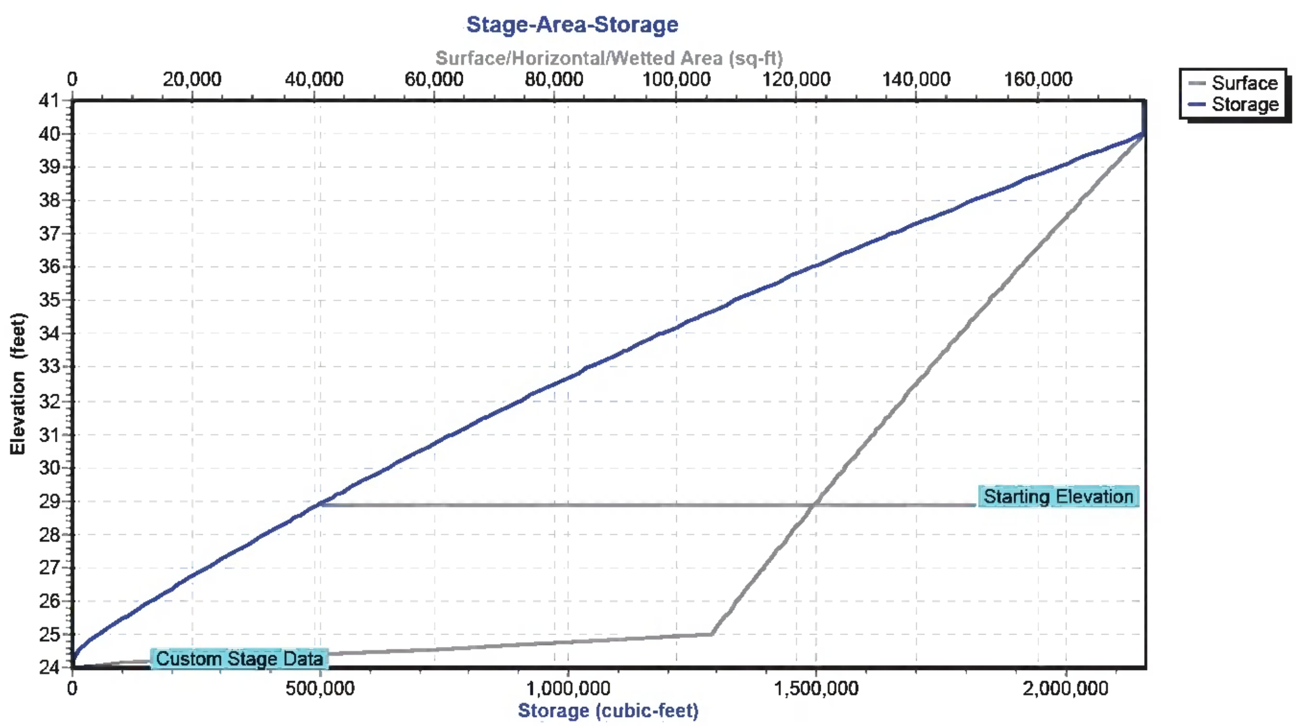
Pond P2: Leachate Pond



Pond P2: Leachate Pond



Pond P2: Leachate Pond



APPENDIX E-6

LEACHATE COLLECTION CORRIDOR AND CONTACT WATER PIPES - STRENGTH DESIGN

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

APPENDIX E-6

LEACHATE COLLECTION CORRIDOR AND CONTACT WATER PIPES - STRENGTH DESIGN

SOUTH CAROLINA LICENSED PROFESSIONAL ENGINEER No. 33535 SCOTT M. GRAVES

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

8/9/2016

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

PURPOSE

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

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

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

METHOD OF ANALYSES

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

Stresses on Pipes

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

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

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

where:

σ_{max} = stress on the pipe, psf;

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

D_p = thickness of the overburden materials, ft.

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

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

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

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

Wall Crushing

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

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

where:

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

σ_y = compressive yield strength of the pipe, psf;

SDR = standard dimension ratio of the pipe; and

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

Wall Buckling

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

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

where:

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

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

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

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

SDR = standard dimension ratio of the pipe.

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

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

where:

M_s = constrained modulus, psi;

E_s = Young's modulus, psi; and

ν = Poisson's ratio.

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

Bending Strain

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

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

where:

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

D_L = deflection lag factor;

K = bedding constant (Figure E-6.2);

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

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

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

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

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

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

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

where:

ε_b= bending strain, percent;

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

t = minimum wall thickness, in.;

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

D = mean pipe diameter, in.

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

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

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

CALCULATIONS

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

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

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

Winyah Generating Station - Pipe Strength Design

Performed by: V. Krishnan, 06/10/2016 Reviewed by: M. Christman, 07/01/2016

Input Parameters

Waste

d_c = 212 ft

γ_{avg} = 100 pcf

Pipe

SDR = 11

D_{od} = 6.625 in.

t_{min} = 0.602 in.

E = 30,841 psi

σ_y = 1500 psi

D_L = 1.25

K = 0.11

k = 1.5

Bedding Soil

E_s = 4700 psi

ν = 0.28

Calculated Parameters

σ_{max} = 147.2 psi

M_s = 6009 psi

E' = 9013 psi

W_c = 975 lb/in.

I = 0.01818 $in.^4/in.$

r_{mean} = 3.01 in.

SA = 736.1 psi

Strength Checks

Wall Crushing

$$FS_{WC} = \frac{2\sigma_y}{(SDR-1)\sigma_{max}}$$

FS_{WC} = **2.0 ≥ 1.5**

Wall Buckling

$$FS_{wb} = \frac{1.2}{\sigma_{max}}\left[\frac{E'E}{SDR^3}\right]^{1/2}$$

FS_{wb} = **3.7 ≥ 1.5**

Ring deflection (Modified Iowa Equation):

$$\Delta X = \frac{D_L K W_c}{(EI/r^3)+(0.061E')}$$

Change in diameter, ΔX = 0.235 in.

Ring deflection, $\Delta X\%$ = 3.55 %

Pipe wall bending strain, ε_b.

$$\varepsilon_b = 6\cdot\frac{t_{min}\cdot\Delta y}{D^2}$$

Δy = 0.235 in.

D = 6.02 in.

Bending strain, ε_b = 2.34 %

Allowable wall ring bending strain: from 4.2 to 8% (8% for 50 year design life) - [CPChem, 2002]

Variable Definition

d_c = maximum thickness of overlying materials, ft;

γ_{avg} = average unit weight of overlying materials (waste, liner and cover), pcf;

SDR = standard dimension ratio of the pipe;

D_{od} = outer diameter of pipe, in [CPChem, 2002];

t_{min} = minimum thickness of the pipe, in. [CPChem, 2002]

E = long-term modulus of elasticity of the pipe material [after 50 years based on S_A, Phillips 66, 1991], psi;

σ_y = compressive yield strength of the pipe;

D_L = deflection lag factor (assume 1.25) [Wilson-Fahmy and Koerner, 1994];

K = bedding constant (0° => 0.110) [Wilson-Fahmy and Koerner, 1994; Figure 2];

k = an empirically derived factor for calculating E' (ranges between 0.7 and 2.3, Selig, 1990);

E_s = Young's modulus of the bedding material, psi;

ν = Poisson's ratio of the bedding material;

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

M_s = constrained modulus of the bedding material;

E' = the modulus of soil reaction for pipe bedding material [Selig, 1990; Table 2], psi;

W_c = Marston's prism load per unit length of pipe, lb/in. [Wilson -Fahmy and Koerner, 1994] = $(\gamma_{avg})(d_c)(D_{od})$;

I = the moment of inertia of the pipe wall per unit length ($t_{min}^3/12$), $in.^4/in.$;

r_{mean} = mean radius = $(D_{od} - t_{min})/2$, in.

S_A = $(SDR-1)\sigma_{max}$ /2

FS_{WC} = factor of safety against wall crushing

FS_{wb} = factor of safety against wall buckling

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

$\Delta X\%$ = the ring deflection, %. = $100(\Delta X/D_{od})$

ε_b = Bending strain, %;

Δy = Vertical deflection, in. = ΔX;

D = diameter = Mean diameter $(D_{od}-t_{min})$, in.;

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

Winyah Generating Station - Pipe Strength Design

Performed by: V. Krishnan, 06/10/2016 Reviewed by: M. Christman, 07/01/2016

Input Parameters

Waste

d_c = 288 ft

γ_{avg} = 100 pcf

Pipe

SDR = 11

D_{od} = 6.625 in.

t_{min} = 0.602 in.

E = 28,292 psi

σ_y = 1500 psi

D_L = 1.25

K = 0.11

k = 1.5

Bedding Soil

E_s = 4700 psi

ν = 0.28

Calculated Parameters

σ_{max} = 199.9 psi

M_s = 6009 psi

E' = 9013 psi

W_c = 1,324 lb/in.

I = 0.01818 $in.^4/in.$

r_{mean} = 3.01 in.

SA = 999.4 psi

Strength Checks

Wall Crushing

$$FS_{WC} = \frac{2\sigma_y}{(SDR-1)\sigma_{max}}$$

FS_{WC} = **1.5 ≥ 1.5**

Wall Buckling

$$FS_{wb} = \frac{1.2}{\sigma_{max}}\left[\frac{E'E}{SDR^3}\right]^{1/2}$$

FS_{wb} = **2.6 ≥ 1.5**

Ring deflection (Modified Iowa Equation):

$$\Delta X = \frac{D_L K W_c}{(EI/r^3)+(0.061E')}$$

Change in diameter, ΔX = 0.320 in.

Ring deflection, ΔX% = 4.83 %

Pipe wall bending strain, ε_b.

$$\varepsilon_b = 6\cdot\frac{t_{min}\cdot\Delta y}{D^2}$$

Δy = 0.320 in.

D = 6.02 in.

Bending strain, ε_b = 3.19 %

Allowable wall ring bending strain: from 4.2 to 8% (8% for 50 year design life) - [CPChem, 2002]

Variable Definition

d_c = maximum thickness of overlying materials, ft;

γ_{avg} = average unit weight of overlying materials (waste, liner and cover), pcf;

SDR = standard dimension ratio of the pipe;

D_{od} = outer diameter of pipe, in [CPChem, 2002];

t_{min} = minimum thickness of the pipe, in. [CPChem, 2002]

E = long-term modulus of elasticity of the pipe material [after 50 years based on S_A, Phillips 66, 1991], psi;

σ_y = compressive yield strength of the pipe;

D_L = deflection lag factor (assume 1.25) [Wilson-Fahmy and Koerner, 1994];

K = bedding constant (0° => 0.110) [Wilson-Fahmy and Koerner, 1994; Figure 2];

k = an empirically derived factor for calculating E' (ranges between 0.7 and 2.3, Selig, 1990);

E_s = Young's modulus of the bedding material, psi;

ν = Poisson's ratio of the bedding material;

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

M_s = constrained modulus of the bedding material;

E' = the modulus of soil reaction for pipe bedding material [Selig, 1990; Table 2], psi;

W_c = Marston's prism load per unit length of pipe, lb/in. [Wilson -Fahmy and Koerner, 1994]

= $(\gamma_{avg})(d_c)(D_{od})$;

I = the moment of inertia of the pipe wall per unit length ($t_{min}^3/12$), $in.^4/in.$;

r_{mean} = mean radius = $(D_{od} - t_{min})/2$, in.

S_A = =(SDR-1)σ_{max} /2

FS_{WC} = factor of safety against wall crushing

FS_{wb} = factor of safety against wall buckling

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

ΔX% = the ring deflection, %.

= 100(ΔX/D_{od})

ε_b = Bending strain, %;

Δy = Vertical deflection, in. = ΔX;

D = diameter = Mean diameter (D_{od}-t_{min}), in.;

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24"ϕ SDR-11 HDPE Riser Pipe (design loading)

Winyah Generating Station - Pipe Strength Design

Performed by: V. Krishnan, 06/10/2016 Reviewed by: M. Christman, 07/01/2016

Input Parameters

Waste

d_c = 30 ft

γ_{avg} = 100 pcf

Pipe

SDR = 11

D_{od} = 24.000 in.

t_{min} = 2.182 in.

E = 38,558 psi

σ_y = 1500 psi

D_L = 1.25

K = 0.11

k = 1.5

Bedding Soil

E_s = 3300 psi

ν = 0.19

Calculated Parameters

σ_{max} = 20.8 psi

M_s = 3623 psi

E' = 5434 psi

W_c = 500 lb/in.

I = 0.86551 in.^{4}/in.

r_{mean} = 10.91 in.

SA = 104.2 psi

Strength Checks

Wall Crushing

$$FS_{WC} = \frac{2\sigma_y}{(SDR-1)\sigma_{max}}$$

FS_{WC} = 14.4 ≥ 1.5

Wall Buckling

$$FS_{wb} = \frac{1.2}{\sigma_{max}}\left[\frac{E'E}{SDR^3}\right]^{1/2}$$

FS_{wb} = 22.9 ≥ 1.5

Ring deflection (Modified Iowa Equation):

$$\Delta X = \frac{D_L K W_c}{(EI/r^3)+(0.061E')}$$

Change in diameter, ΔX = 0.192 in.

Ring deflection, ΔX% = 0.80 %

Pipe wall bending strain, ε_b.

$$\varepsilon_b = 6\cdot\frac{t_{min}\cdot\Delta y}{D^2}$$

Δy = 0.192 in.

D = 21.82 in.

Bending strain, ε_b = 0.53 %

Allowable wall ring bending strain: from 4.2 to 8% (8% for 50 year design life) - [CPChem, 2002]

Varaiable Definition

d_c = maximum thickness of overlying materials, ft;
γ_{avg} = average unit weight of overlying materials (waste, liner and cover), pcf;
SDR = standard dimension ratio of the pipe;
D_{od} = outer diameter of pipe, in [CPChem, 2002];
t_{min} = minimum thickness of the pipe, in. [CPChem, 2002]
E = long-term modulus of elasticity of the pipe material [after 50 years based on S_A, Phillips 66, 1991], psi;
σ_y = compressive yield strength of the pipe;
D_L = deflection lag factor (assume 1.25) [Wilson-Fahmy and Koerner, 1994];
K = bedding constant (0° => 0.110) [Wilson-Fahmy and Koerner, 1994; Figure 2];
k = an empirically derived factor for calculating E' (ranges between 0.7 and 2.3, Selig, 1990);
E_s = Young's modulus of the bedding material, psi;
ν = Poisson's ratio of the bedding material;
σ_{max} = maximum stress applied to the pipe, psi;
M_s = constrained modulus of the bedding material;
E' = the modulus of soil reaction for pipe bedding material [Selig, 1990; Table 2], psi;
W_c = Marston's prism load per unit length of pipe, lb/in. [Wilson -Fahmy and Koerner, 1994]
= (γ_{avg}) (d_c) (D_{od});
I = the moment of inertia of the pipe wall per unit length (t_{min}^3/12), in.^{4}/in.;
r_{mean} = mean radius = (D_{od} - t_{min})/2 , in.
S_A = =(SDR-1)σ_{max} /2
FS_{WC} = factor of safety against wall crushing
FS_{wb} = factor of safety against wall buckling
ΔX = maximum horizontal deflection or change in diameter, in;
ΔX% = the ring deflection, %.
= 100(ΔX/D_{od})
ε_b = Bending strain, %;
Δy = Vertical deflection, in. = ΔX;
D = diameter = Mean diameter (D_{od}-t_{min}), in.;

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24"ϕ SDR-11 HDPE Riser Pipe (design loading at Cell 5S)

Winyah Generating Station - Pipe Strength Design

Performed by: V. Krishnan, 06/10/2016 Reviewed by: M. Christman, 07/01/2016

Input Parameters

Waste

d_c = 158 ft

γ_{avg} = 100 pcf

Pipe

SDR = 11

D_{od} = 24.000 in.

t_{min} = 2.182 in.

E = 32,880 psi

σ_y = 1500 psi

D_L = 1.25

K = 0.11

k = 1.5

Bedding Soil

E_s = 4700 psi

v = 0.28

Calculated Parameters

σ_{max} = 109.7 psi

M_s = 6009 psi

E' = 9013 psi

W_c = 2,633 lb/in.

I = 0.86551 $in.^4/in.$

r_{mean} = 10.91 in.

SA = 548.6 psi

Strength Checks

Wall Crushing

$$\boxed{FS_{WC} = \frac{2\sigma_y}{(SDR-1)\sigma_{max}}}$$

FS_{WC} = **2.7 ≥ 1.5**

Wall Buckling

$$\boxed{FS_{wb} = \frac{1.2}{\sigma_{max}}\left[\frac{E'E}{SDR^3}\right]^{1/2}}$$

FS_{wb} = **5.2 ≥ 1.5**

Ring deflection (Modified Iowa Equation):

$$\boxed{\Delta X = \frac{D_L K W_c}{(EI/r^3) + (0.061\,E')}}$$

Change in diameter, ΔX = 0.633 in.

Ring deflection, ΔX% = 2.64 %

Pipe wall bending strain, ε_b.

$$\boxed{\varepsilon_b = 6 \cdot \frac{t_{min} \cdot \Delta y}{D^2}}$$

Δy = 0.633 in.

D = 21.82 in.

Bending strain, ε_b = 1.74 %

Allowable wall ring bending strain: from 4.2 to 8% (8% for 50 year design life) - [CPChem, 2002]

Varaiable Definition

d_c = maximum thickness of overlying materials, ft;

γ_{avg} = average unit weight of overlying materials (waste, liner and cover), pcf;

SDR = standard dimension ratio of the pipe;

D_{od} = outer diameter of pipe, in [CPChem, 2002];

t_{min} = minimum thickness of the pipe, in. [CPChem, 2002]

E = long-term modulus of elasticity of the pipe material [after 50 years based on S_A, Phillips 66, 1991], psi;

σ_y = compressive yield strength of the pipe;

D_L = deflection lag factor (assume 1.25) [Wilson-Fahmy and Koerner, 1994];

K = bedding constant (0° => 0.110) [Wilson-Fahmy and Koerner, 1994; Figure 2];

k = an empirically derived factor for calculating E' (ranges between 0.7 and 2.3, Selig, 1990);

E_s = Young's modulus of the bedding material, psi;

v = Poisson's ratio of the bedding material;

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

M_s = constrained modulus of the bedding material;

E' = the modulus of soil reaction for pipe bedding material [Selig, 1990; Table 2], psi;

W_c = Marston's prism load per unit length of pipe, lb/in. [Wilson -Fahmy and Koerner, 1994] = $(\gamma_{avg})(d_c)(D_{od})$;

I = the moment of inertia of the pipe wall per unit length ($t_{min}^3/12$), $in.^4/in.$;

r_{mean} = mean radius = $(D_{od} - t_{min})/2$, in.

S_A = $(SDR-1)\sigma_{max}$ /2

FS_{WC} = factor of safety against wall crushing

FS_{wb} = factor of safety against wall buckling

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

ΔX% = the ring deflection, %. = $100(\Delta X/D_{od})$

ε_b = Bending strain, %;

Δy = Vertical deflection, in. = ΔX;

D = diameter = Mean diameter $(D_{od}-t_{min})$, in.;

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24”ϕ SDR-11 HDPE Riser Pipe (maximum allowable loading)

Winyah Generating Station - Pipe Strength Design

Performed by: V. Krishnan, 06/10/2016 Reviewed by: M. Christman, 07/01/2016

Input Parameters

Waste

d_c = 288 ft

γ_{avg} = 100 pcf

Pipe

SDR = 11

D_{od} = 24.000 in.

t_{min} = 2.182 in.

E = 28,291 psi

σ_y = 1500 psi

D_L = 1.25

K = 0.11

k = 1.5

Bedding Soil

E_s = 4700 psi

ν = 0.28

Calculated Parameters

σ_{max} = 199.9 psi

M_s = 6009 psi

E' = 9013 psi

W_c = 4,797 lb/in.

I = 0.86551 in.^{4}/in.

r_{mean} = 10.91 in.

SA = 999.4 psi

Strength Checks

Wall Crushing

$$FS_{WC} = \frac{2\sigma_y}{(SDR-1)\sigma_{max}}$$

FS_{WC} = **1.5 ≥ 1.5**

Wall Buckling

$$FS_{wb} = \frac{1.2}{\sigma_{max}}\left[\frac{E'E}{SDR^3}\right]^{1/2}$$

FS_{wb} = **2.6 ≥ 1.5**

Ring deflection (Modified Iowa Equation):

$$\Delta X = \frac{D_L K W_c}{(EI/r^3) + (0.061E')}$$

Change in diameter, ΔX = 1.160 in.

Ring deflection, ΔX% = 4.83 %

Pipe wall bending strain, ε_b.

$$\varepsilon_b = 6 \cdot \frac{t_{min} \cdot \Delta y}{D^2}$$

Δy = 1.160 in.

D = 21.82 in.

Bending strain, ε_b = 3.19 %

Allowable wall ring bending strain: from 4.2 to 8% (8% for 50 year design life) - [CPChem, 2002]

Varaiable Definition

d_c = maximum thickness of overlying materials, ft;

γ_{avg} = average unit weight of overlying materials (waste, liner and cover), pcf;

SDR = standard dimension ratio of the pipe;

D_{od} = outer diameter of pipe, in [CPChem, 2002];

t_{min} = minimum thickness of the pipe, in. [CPChem, 2002]

E = long-term modulus of elasticity of the pipe material [after 50 years based on S_A, Phillips 66, 1991], psi;

σ_y = compressive yield strength of the pipe;

D_L = deflection lag factor (assume 1.25) [Wilson-Fahmy and Koerner, 1994];

K = bedding constant (0° => 0.110) [Wilson-Fahmy and Koerner, 1994; Figure 2];

k = an empirically derived factor for calculating E' (ranges between 0.7 and 2.3, Selig, 1990);

E_s = Young's modulus of the bedding material, psi;

ν = Poisson's ratio of the bedding material;

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

M_s = constrained modulus of the bedding material;

E' = the modulus of soil reaction for pipe bedding material [Selig, 1990; Table 2], psi;

W_c = Marston's prism load per unit length of pipe, lb/in. [Wilson -Fahmy and Koerner, 1994]

= (γ_{avg}) (d_c) (D_{od});

I = the moment of inertia of the pipe wall per unit length (t_{min}^3/12), in.^{4}/in.;

r_{mean} = mean radius = (D_{od} - t_{min})/2 , in.

S_A = (SDR-1)σ_{max} /2

FS_{WC} = factor of safety against wall crushing

FS_{wb} = factor of safety against wall buckling

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

ΔX% = the ring deflection, %.

= 100(ΔX/D_{od})

ε_b = Bending strain, %;

Δy = Vertical deflection, in. = ΔX;

D = diameter = Mean diameter (D_{od}-t_{min}), in.;

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18"ϕ SDR-11 HDPE Contact Water Discharge Pipe (design loading)

Winyah Generating Station - Pipe Strength Design

Performed by: V. Krishnan, 06/10/2016 Reviewed by: M. Christman, 07/01/2016

Input Parameters

Waste

d_c = 166 ft

γ_{avg} = 100 pcf

Pipe

SDR = 11

D_{od} = 18.000 in.

t_{min} = 1.636 in.

E = 32,565 psi

σ_y = 1500 psi

D_L = 1.25

K = 0.11

k = 1.5

Bedding Soil

E_s = 1000 psi

ν = 0.41

Calculated Parameters

σ_{max} = 115.3 psi

M_s = 2325 psi

E' = 3487 psi

W_c = 2,075 lb/in.

I = 0.36490 in.^{4}/in.

r_{mean} = 8.18 in.

SA = 576.4 psi

Strength Checks

Wall Crushing

$$FS_{WC} = \frac{2\sigma_y}{(SDR-1)\sigma_{max}}$$

FS_{WC} = **2.6 ≥ 1.5**

Wall Buckling

$$FS_{wb} = \frac{1.2}{\sigma_{max}}\left[\frac{E'E}{SDR^3}\right]^{1/2}$$

FS_{wb} = **3.0 ≥ 1.5**

Ring deflection (Modified Iowa Equation):

$$\Delta X = \frac{D_L K W_c}{(EI/r^3) + (0.061 E')}$$

Change in diameter, ΔX = 1.217 in.

Ring deflection, ΔX% = 6.76 %

Pipe wall bending strain, ε_b.

$$\varepsilon_b = 6 \cdot \frac{t_{min} \cdot \Delta y}{D^2}$$

Δy = 1.217 in.

D = 16.36 in.

Bending strain, ε_b = 4.46 %

Allowable wall ring bending strain: from 4.2 to 8% (8% for 50 year design life) - [CPChem, 2002]

Variable Definition

d_c = maximum thickness of overlying materials, ft;

γ_{avg} = average unit weight of overlying materials (waste, liner and cover), pcf;

SDR = standard dimension ratio of the pipe;

D_{od} = outer diameter of pipe, in [CPChem, 2002];

t_{min} = minimum thickness of the pipe, in. [CPChem, 2002]

E = long-term modulus of elasticity of the pipe material [after 50 years based on S_A, Phillips 66, 1991], psi;

σ_y = compressive yield strength of the pipe;

D_L = deflection lag factor (assume 1.25) [Wilson-Fahmy and Koerner, 1994];

K = bedding constant (0° => 0.110) [Wilson-Fahmy and Koerner, 1994; Figure 2];

k = an empirically derived factor for calculating E' (ranges between 0.7 and 2.3, Selig, 1990);

E_s = Young's modulus of the bedding material, psi;

ν = Poisson's ratio of the bedding material;

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

M_s = constrained modulus of the bedding material;

E' = the modulus of soil reaction for pipe bedding material [Selig, 1990; Table 2], psi;

W_c = Marston's prism load per unit length of pipe, lb/in. [Wilson -Fahmy and Koerner, 1994] = $(\gamma_{avg})(d_c)(D_{od})$;

I = the moment of inertia of the pipe wall per unit length ($t_{min}^3/12$), in.^{4}/in.;

r_{mean} = mean radius = $(D_{od} - t_{min})/2$, in.

S_A = $(SDR-1)\sigma_{max}$ /2

FS_{WC} = factor of safety against wall crushing

FS_{wb} = factor of safety against wall buckling

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

ΔX% = the ring deflection, %. = 100(ΔX/D_{od})

ε_b = Bending strain, %;

Δy = Vertical deflection, in. = ΔX;

D = diameter = Mean diameter (D_{od}-t_{min}), in.;

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

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

18”ϕ SDR-11 HDPE Contact Water Discharge Pipe (maximum allowable loading)

Winyah Generating Station - Pipe Strength Design

Performed by: V. Krishnan, 06/10/2016 Reviewed by: M. Christman, 07/01/2016

Input Parameters

Waste

d_c = 186 ft

γ_{avg} = 100 pcf

Pipe

SDR = 11

D_{od} = 18.000 in.

t_{min} = 1.636 in.

E = 31,807 psi

σ_y = 1500 psi

D_L = 1.25

K = 0.11

k = 1.5

Bedding Soil

E_s = 1000 psi

v = 0.41

Calculated Parameters

σ_{max} = 129.0 psi

M_s = 2325 psi

E' = 3487 psi

W_c = 2,322 lb/in.

I = 0.36490 $in.^4/in.$

r_{mean} = 8.18 in.

SA = 645.0 psi

Strength Checks

Wall Crushing

$$FS_{WC} = \frac{2\sigma_y}{(SDR-1)\sigma_{max}}$$

FS_{WC} = **2.33 ≥ 1.5**

Wall Buckling

$$FS_{wb} = \frac{1.2}{\sigma_{max}}\left[\frac{E'E}{SDR^3}\right]^{1/2}$$

FS_{wb} = **2.7 ≥ 1.5**

Ring deflection (Modified Iowa Equation):

$$\Delta X = \frac{D_L K W_c}{(EI/r^3) + (0.061 E')}$$

Change in diameter, ΔX = 1.365 in.

Ring deflection, ΔX% = 7.58 %

Pipe wall bending strain, ε_b.

$$\varepsilon_b = 6 \cdot \frac{t_{min} \cdot \Delta y}{D^2}$$

Δy = 1.365 in.

D = 16.36 in.

Bending strain, ε_b = 5.00 %

Allowable wall ring bending strain: from 4.2 to 8% (8% for 50 year design life) - [CPChem, 2002]

Variable Definition

d_c = maximum thickness of overlying materials, ft;

γ_{avg} = average unit weight of overlying materials (waste, liner and cover), pcf;

SDR = standard dimension ratio of the pipe;

D_{od} = outer diameter of pipe, in [CPChem, 2002];

t_{min} = minimum thickness of the pipe, in. [CPChem, 2002]

E = long-term modulus of elasticity of the pipe material [after 50 years based on S_A, Phillips 66, 1991], psi;

σ_y = compressive yield strength of the pipe;

D_L = deflection lag factor (assume 1.25) [Wilson-Fahmy and Koerner, 1994];

K = bedding constant (0° => 0.110) [Wilson-Fahmy and Koerner, 1994; Figure 2];

k = an empirically derived factor for calculating E' (ranges between 0.7 and 2.3, Selig, 1990);

E_s = Young's modulus of the bedding material, psi;

v = Poisson's ratio of the bedding material;

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

M_s = constrained modulus of the bedding material;

E' = the modulus of soil reaction for pipe bedding material [Selig, 1990; Table 2], psi;

W_c = Marston's prism load per unit length of pipe, lb/in. [Wilson -Fahmy and Koerner, 1994]

= (γ_{avg}) (d_c) (D_{od});

I = the moment of inertia of the pipe wall per unit length ($t_{min}^3/12$), $in.^4/in.$;

r_{mean} = mean radius = (D_{od} - t_{min})/2 , in.

S_A = =(SDR-1)σ_{max} /2

FS_{WC} = factor of safety against wall crushing

FS_{wb} = factor of safety against wall buckling

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

ΔX% = the ring deflection, %.

= 100(ΔX/D_{od})

ε_b = Bending strain, %;

Δy = Vertical deflection, in. = ΔX;

D = diameter = Mean diameter (D_{od}-t_{min}), in.;

SUMMARY AND CONCLUSIONS

<u>6"ϕ SDR-11 HDPE Leachate Corridor Pipes</u>

Under the design loading resulting from a total waste height of 212 ft on top of the leachate corridor pipes, the pipe strength evaluation is summarized as follows:

* Factor of safety against pipe wall crushing, FS_{wc} = 2.0 (OK)
* Factor of safety against pipe wall buckling, FS_{wb} = 3.7 (OK)
* Bending strain = 2.34 % (OK)

The back-calculated maximum height of waste over the corridor that would result in acceptable factors of safety and allowable strains is 288 ft. This is in excess of that proposed, but is provided here as an indication of the tallest waste height that the design could tolerate.

<u>24"ϕ SDR-11 HDPE Riser Pipes</u>

Under the design loading resulting from a total waste height of 30 ft on top of the riser pipes, the pipe strength evaluation is summarized as follows:

* Factor of safety against pipe wall crushing, FS_{wc} = 14.4 (OK)
* Factor of safety against pipe wall buckling, FS_{wb} = 22.9 (OK)
* Bending strain = 0.53 % (OK)

Under the design loading resulting from a total waste height of 158 ft on top of the riser pipe in Subcell 5S, the pipe strength evaluation is summarized as follows:

* Factor of safety against pipe wall crushing, FS_{wc} = 2.7 (OK)
* Factor of safety against pipe wall buckling, FS_{wb} = 5.2 (OK)
* Bending strain = 1.74 % (OK)

The back-calculated maximum height of waste over the riser pipes that would result in acceptable factors of safety and allowable strains is 288 ft. This is in excess of that proposed, but is provided here as an indication of the tallest waste height that the design could tolerate.

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

18"ϕ SDR-11 HDPE Contact Water Discharge Pipes

Under the design loading resulting from a total waste height of 166 ft on top of the contact water discharge pipe, the pipe strength evaluation is summarized as follows:

* Factor of safety against pipe wall crushing, FS_{wc} = 2.6 (OK)
* Factor of safety against pipe wall buckling, FS_{wb} = 3.0 (OK)
* Bending strain = 4.46% (OK)

The back-calculated maximum height of waste over the corridor that would result in acceptable factors of safety and allowable strains is 186 ft. This is in excess of that proposed, but is provided here as an indication of the tallest waste height that the design could tolerate.

Based on the above results, the specified pipes are anticipated to perform as designed.

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

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TABLES

Table E-6.1. Modulus of Soil Reaction for Pipe Bedding Material (from Selig, 1990)

Soil Type: SW, SP, GW, GP

Stress level psi (kPa)	95% D698 E_s	95% D698 B	95% D698 v_s	85% D698 E_s	85% D698 B	85% D698 v_s
1 (7)	1600 (11)	2800 (19)	0.40	1300 (9)	900 (6)	0.26
5 (34)	4100 (28)	3300 (23)	0.29	2100 (14)	1200 (8)	0.21
10 (70)	6000 (41)	3900 (27)	0.24	2600 (18)	1400 (10)	0.19
20 (140)	8600 (59)	5300 (37)	0.23	3300 (23)	1800 (12)	0.19
40 (280)	13000 (90)	8700 (60)	0.25	4100 (28)	2500 (17)	0.23
60 (410)	16000 (110)	13000 (90)	0.29	4700 (32)	3500 (24)	0.28

Soil Type: GM, SM, ML, and GC, SC with < 20% fines

Stress level psi (kPa)	95% D698 E_s	95% D698 B	95% D698 v_s	85% D698 E_s	85% D698 B	85% D698 v_s
1 (7)	1800 (12)	1900 (13)	0.34	600 (4)	400 (3)	0.25
5 (34)	2500 (17)	2000 (14)	0.29	700 (5)	450 (3)	0.24
10 (70)	2900 (20)	2100 (14)	0.27	800 (6)	500 (3)	0.23
20 (140)	3200 (22)	2500 (17)	0.29	850 (6)	700 (5)	0.30
40 (280)	3700 (25)	3400 (23)	0.32	900 (6)	1200 (8)	0.38
60 (410)	4100 (28)	4500 (31)	0.35	1000 (7)	1800 (12)	0.41

Soil Type: CL, MH, GC, SC

Stress level psi (kPa)	95% D698 E_s	95% D698 B	95% D698 v_s	85% D698 E_s	85% D698 B	85% D698 v_s
1 (7)	400 (3)	800 (6)	0.42	100 (1)	100 (1)	0.33
5 (34)	800 (6)	900 (6)	0.35	250 (2)	200 (1)	0.29
10 (70)	1100 (8)	1000 (7)	0.32	400 (3)	300 (2)	0.28
20 (140)	1300 (9)	1100 (8)	0.30	600 (4)	400 (3)	0.25
40 (280)	1400 (10)	1600 (11)	0.35	700 (5)	800 (6)	0.35
60 (410)	1500 (10)	2100 (14)	0.38	800 (6)	1300 (9)	0.40

Note: Units of E_s and B are psi (MPa).

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

FIGURES

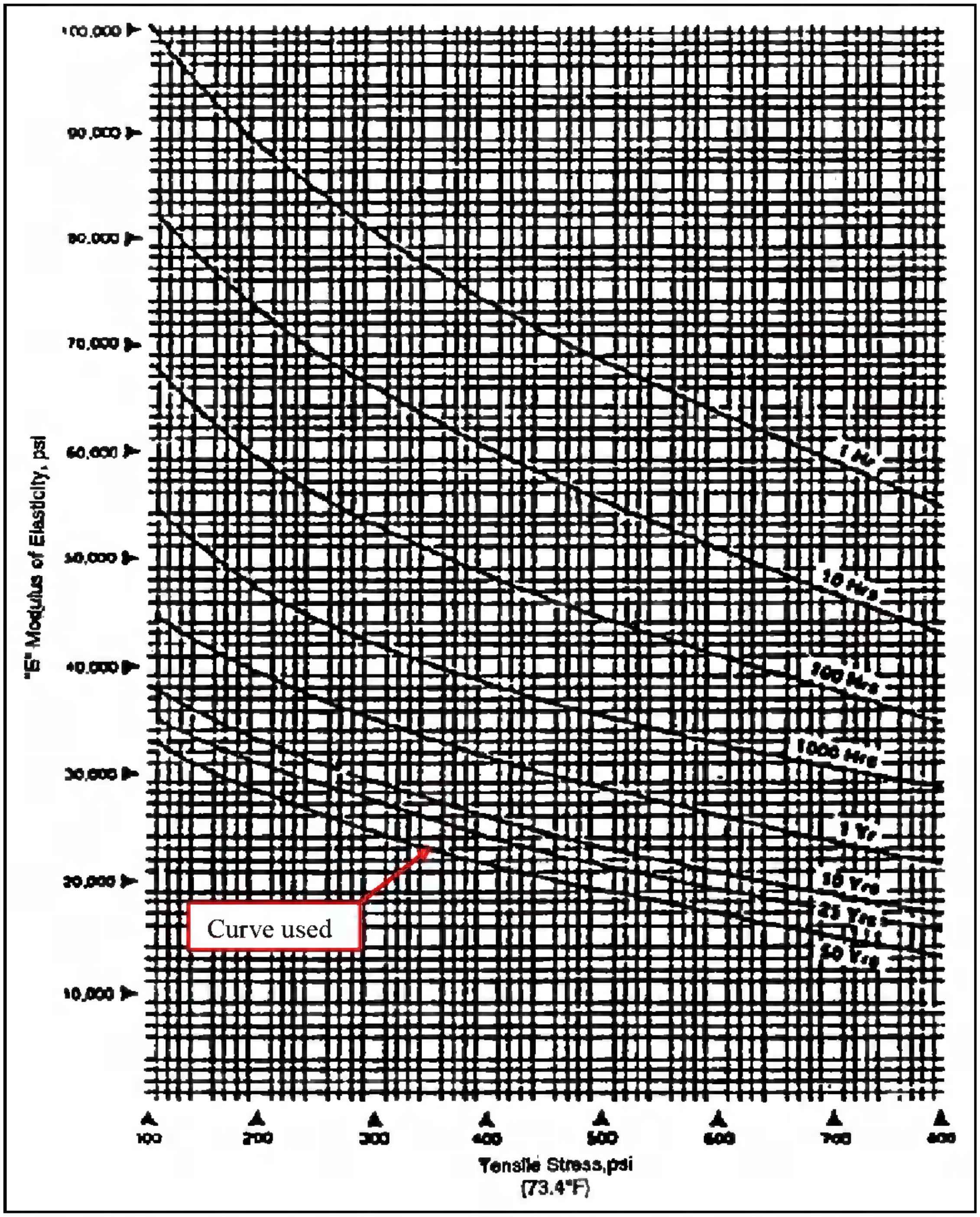

Figure E-6.1. Time Dependent Modulus of Elasticity for Polyethylene Pipe (from Phillips 66, 1991)

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

Values of Bedding Constant

Bedding Angle (degrees)	K
0	0.110
30	0.108
45	0.105
60	0.102
90	0.096
120	0.090
180	0.083

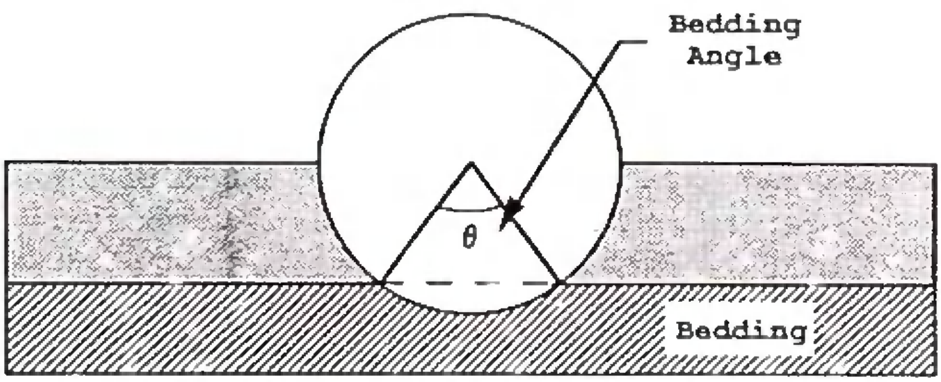

Figure E-6.2. Bedding Constant
(from Wilson-Fahmy and Koerner, 1994)

APPENDIX F-6

SITE WATERSHED ANALYSIS

Written by:	A. Sivashanthan	Date:	7/22/2016	Reviewed by:	B. Klenzendorf	Date:	8/4/2016
Client:	Santee Cooper	Project:	Winyah Generating Station	Project No.:	GSC5242	Phase No.:	01

APPENDIX F-6

SITE WATERSHED ANALYSIS

SOUTH CAROLINA LICENSED PROFESSIONAL ENGINEER No. 33535 SCOTT M. GRAVES

8/9/16

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

SEALED FOR
CALCULATION PAGES F6-1
THROUGH F6-54

1. INTRODUCTION

1.1 Purpose

The purpose of this calculation package is to evaluate the site watershed hydrology under pre-development and post-development conditions, for the portion of the Winyah Generating Station (WGS) site that includes the proposed Class Three Landfill areas (Landfill Area 1 and Landfill Area 2). The objectives of this site watershed hydrologic analysis are as follows:

- evaluate and demonstrate that sufficient capacity will exist in the on-site industrial cooling pond (on-site receiving water body) to contain the design storm event (25-year, 24-hour) following the proposed development of on-site landfill (i.e., Landfill Area 1 and Landfill Area 2); and

- compare the values of peak discharge, runoff volume, and cooling pond peak water surface elevation under post-development conditions to those that exist under pre-development existing conditions in order to demonstrate that the landfill design does not adversely alter, to any significant degree, the total quantity of runoff currently entering the cooling pond and the drainage patterns of the watershed in the vicinity of the cooling pond.

Note that the hydraulic design of the landfill surface water management features is presented in other calculation packages that accompany the Engineering Report.

1.2 Site Watershed Overview

WGS and the surface impoundments on site are located within the Sampit River watershed (ID: 03040207-01). The Sampit River Watershed occupies 105,260 acres in the Lower Coastal Plain and Coastal Zone deposits of South Carolina and consists primarily of the Sampit River and its tributaries, Pennyroyal Creek and Turkey Creek (Geosyntec, 2016). Pennyroyal Creek borders the western boundary of the WGS property and flows in a northeasterly direction. Turkey Creek is located east of the WGS property and flows in a northern direction towards Pennyroyal Creek where the two creeks meet about one-mile north of the WGS property line and one-half mile south of the Sampit River. The industrial cooling pond was formed by constructing a compacted earthen dam within the existing Turkey Creek. A portion of Turkey Creek was subsequently relocated to a man-made channel along the east side of the cooling pond to divert off-site runon areas to the south of the WGS site around the cooling pond.

The following definitions pertain to the two conditions analyzed in this package:

- Pre-Development Conditions – represent the existing conditions at the WGS site for drainage areas contributing runoff to the cooling pond.
- Post-Development Conditions – represent proposed conditions of the site once the landfills have been fully developed, with the final cover and permanent surface water management system installed; all drainage areas contributing runoff to the cooling pond outside of the proposed landfill areas were modeled as pre-development conditions for this scenario.

Figure 1 of this calculation package shows the pre-development drainage areas contributing runoff to the cooling pond.

2. METHODOLOGY

2.1 HEC-HMS Computer Model

Surface water discharges are estimated using the Hydrologic Modeling System (HEC-HMS) computer program Version 4.1 developed through the Hydraulic Engineering Center (HEC) of the United States Army Corps of Engineers (USACE). The program simulates natural and controlled precipitation-runoff and routing processes of a watershed. HEC-HMS is the successor to and replacement for the HEC-1 program (USACE, 2000). For precipitation-runoff-routing simulation, HEC-HMS provides the following components:

- Precipitation-specification options can describe an historical precipitation event, a frequency-based hypothetical precipitation event (i.e., design rainfall or storm event), or an event that represents the upper limit of precipitation possible at a given location. For this analysis, the hypothetical precipitation event used was the 25-year (4% annual chance), 24-hour duration event (herein referred to as the 25-year, 24-hour event).

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

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

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

- Hydraulic models of water-control measures such as surface water pond outfall structures.

HEC-HMS was used to model the pre-development conditions and the post-development conditions. More specifically, HEC-HMS modeling calculates surface water runoff volumes, peak flow rates, and flow characteristics for the drainage areas contributing runoff to the cooling pond.

2.2 Estimation of Time of Concentration for HEC-HMS SCS Curve Number Method

The time of concentration is defined as the time for runoff to flow from the most hydraulically remote point of the drainage area to the point under investigation. The time of concentration (T_c) is a summation of sheet flow travel time, shallow concentrated flow travel time, and open channel flow travel time.

The method to estimate the sheet flow travel time was obtained from the U.S. Department of Agriculture (USDA) document *Urban Hydrology for Small Watersheds, Technical Release 55 (TR-55)* (USDA, 1986). Manning's kinematic solution is used for estimating travel time for sheet flow for flow distances less than 300 ft (USDA, 1986):

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

where:

	T_t	= travel time for overland sheet flow (hr);
	n	= roughness coefficient for sheet flow;
	L	= flow length (ft);
	$P_{2\text{-}24}$	= 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 is shown in Appendix 1 of this calculation package for the pre-development scenario. Characteristics for the post-development scenario are provided in Appendix F-1 for the proposed landfills whereas other subcatchment areas (outside the landfill areas) maintain pre-development conditions in the post-development model; the pre-development Unit 2 Slurry Pond and Ash Pond A are proposed to be converted to Area 1 Landfill and Area 2 Landfill, respectively for post-development conditions. To estimate sheet flow travel time (T_t), a roughness coefficient (n) of 0.40 for woods with light underbrush and a roughness coefficient of 0.05 for fallow (no residue) was selected for pre-development conditions as shown in Table 1 (USDA, 1986). Maximum flow lengths (L) were selected for each drainage area based on the existing site topography. As shown in Table 2 (NOAA, 2015), the rainfall depth for the 2-year, 24-hour frequency ($P_{2\text{-}24}$) at the site is 4.42 inches.

The method used to estimate shallow concentrated flow was obtained from the Upland Method (USDA, 1986) using the equation below.

$$V = K_v \sqrt{S}$$

where: V = average velocity (ft/sec),

K_v = shallow concentrated flow velocity factor (ft/sec), and

S = land slope (ft/ft).

The shallow concentrated flow velocity factor (K_v) is selected from Table 3 (HydroCAD, 2011) based on surface type. A velocity factor of K_v = 10.0 ft/sec was selected for developed areas and impoundment areas based on a nearly bare surface description. The land slopes were estimated from the topographic maps of existing conditions.

The method selected to estimate the shallow concentrated flow and open channel flow travel time is based on guidance provided in TR-55 (USDA, 1986). Travel time for shallow concentrated flow and open channel flow is estimated by dividing the longest drainage path by the velocity of runoff:

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

where: T_t = travel time (min);

L = flow length (ft); and

V = average velocity (ft/sec).

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

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

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

n = Manning's roughness coefficient;

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

A = cross sectional area (ft^2);

P = wetted perimeter (ft); and

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

To estimate open channel flow travel time (T_t), a Manning's roughness coefficient (n) value of 0.028 was selected to represent a dragline-excavated or dredged channel with no vegetation as shown in Table 4 (Chow, 1959).

The velocities and times of concentration used in the design are presented in Appendix 1. A minimum time of concentration of six minutes was used as recommended by TR-55 (USDA, 1986) because small areas with exceedingly short times of concentration could result in design rainfall intensities that are unrealistically high. The lag times calculated for each drainage area are presented in Appendix 1 for use in the SCS Curve Number Method and HEC-HMS software. The lag time is estimated as 0.6 times the time of concentration (USDA, 1986).

3. DESIGN PARAMETERS

The following data and assumptions were utilized in selecting engineering parameters to estimate surface water runoff.

3.1 Rainfall

- The rainfall depths corresponding to 24-hour duration hypothetical precipitation event and 25-year frequency return periods for the site is 8.28 inches as shown in Table 2 (NOAA, 2015). The design storm hyetograph is defined using a SCS Type III rainfall distribution, which is selected based on Figure 2 (USDA, 1986).

3.2 Drainage Areas and Reaches

- Drainage Areas – The contributing site watershed for the pre-development model is divided into multiple subbasins (subcatchments). Subbasins are modeled based on the management of on-site surface water drainage and are delineated for the following areas: developed area (i.e., WGS Plant Area) and existing surface impoundments areas (i.e., Ash Pond A, Ash Pond B, Slurry Pond 3 and 4 plus West Ash Pond, South Ash Pond, and Unit 2 Slurry Pond). The SCS Curve Number Loss Model was used to estimate the volume of runoff from a given subbasin. The SCS Unit Hydrograph Model was used to estimate the direct runoff flow rates from each subbasin. Each subbasin is assigned a curve number representing the type of ground cover for a given soil for the area. The subbasin area, curve number, and SCS Unit Hydrograph lag time input parameters associated with pre-development conditions are included in the HEC-HMS output in Appendix 1.

- Hydrologic Soil Groups (HSGs) – Figure 3 shows a soil map of the WGS site area, taken from the Web Soil Survey tool operated by the USDA Natural Resources Conservation Service (USDA, 2016) for Georgetown County. The predominant soil type at the site are Udorthents, loamy (map unit symbol 58) and Eulonia loamy fine sand (map unit symbol 26A). The on-site soil types have HSG designations as shown in Table 5 (USDA, 2016). To be conservative, both pre-development and post-development soils are assumed to be a HSG type D soil, which generally provides the highest calculated runoff volumes.

- Curve Number (CN) – Curve numbers are obtained from the TR-55 (USDA, 1986) and are based on the predominant HSG of the drainage area. Table 6 summarizes the CNs chosen for the analysis performed for pre-development conditions documented within this calculation package. A composite weighted CN value was calculated for drainage areas with surface water ponds. Table 7 summarizes the area of each cover type selected to calculate the composite CN value for the pre-development subbasins. Areas covered in standing water or exposed geomembrane are assumed to have a large CN value in order to convert the majority of the rainfall into runoff; a CN value of 98 which corresponds to impervious cover (paved areas) was selected for these areas.

3.3 Pre-Development Condition

Figure 1 delineates existing drainage areas contributing runoff to the cooling pond for pre-development conditions. Existing topographic information was compiled via LIDAR survey data (obtained from the Georgetown County, South Carolina) collected in 2005. The pre-development drainage area of 1,257 acres includes the proposed 120-acre landfill footprints as well as additional areas draining to the cooling pond. The total drainage area for pre-development conditions matches the post-development drainage area which allows for direct comparison between the two scenarios.

The Ash Pond A drainage area of approximately 83 acres is bounded by the intake canal to the north, the cooling pond to the east, Ash Pond B to the south, and the discharge canal to the west. Ash Pond A currently receives low volume wastewater from the existing coal-fired electric generating units and contact wastewater from Unit 2 Slurry Pond via a pump with a maximum capacity of 5.79 cfs. Ash Ponds A and B are hydraulically connected through a 30-inch diameter corrugated metal pipe (CMP), a 48-inch diameter smooth steel pipe, and a 24- inch diameter HDPE pipe, and an emergency spillway at elevation 37 ft (Thomas and Hutton, 2016; Thomas and Hutton, 2012). The Ash Pond A area will be converted to the Area 2 Landfill for post-development conditions.

The Unit 2 Slurry Pond drainage area of approximately 37 acres is bounded by the intake canal to the south, the WGS industrial plant to the west, and undeveloped private properties to the north and east. The Unit 2 Slurry Pond discharges to Ash Pond A via a pump with a maximum capacity of 5.79 cfs. The Unity 2 Slurry Pond area will be converted to the Area 1 Landfill for post-development conditions.

The Ash Pond B drainage area of approximately 74 acres is bounded by Ash Pond A to the north, the cooling pond to the east and south, and the discharge canal to the west. A 24-inch diameter smooth interior, corrugated exterior high density polyethylene pipe culvert conveys water from the riser structure to the discharge canal of the cooling pond (Santee Cooper, 2012; Thomas and Hutton, 2016).

The South Ash Pond drainage area of approximately 76 acres is bounded by the railroad line and coal pile to the north, the discharge canal to the east, and a forested area to the south and west. The South Ash Pond currently receives low volume wastewater (from Units 3 and 4) and other process water inflows (from Unit 3 and 4 fly ash sluice and blowdown from the SEFA Star Facility). Water is discharged from the South Ash Pond through a riser structure and outlet pipe to the discharge canal (Lockwood Greene, 1978).

The Slurry Pond 3 & 4 plus West Ash Pond drainage area of approximately 172 acres is bounded by Pennyroyal Creek to the west, the WGS industrial plant to the east, undeveloped land and the South Ash Pond to the south, and undeveloped land to the north. Slurry Pond 3 & 4 is separated from the West Ash Pond by a dike. Slurry Pond 3 & 4 receives process water and stormwater from the plant areas. Slurry Pond 3 & 4 discharges to the discharge canal via a two pumps with a combined maximum capacity of 3,075 gpm. Slurry Pond 3 & 4 has additional pump stations which discharge to the discharge canal during normal operating conditions but become inundated during larger rainfall events (i.e., the 100-year, 24-hour storm event). Slurry Pond 3 & 4 is designed to contain stormwater runoff for a 25-year, 24-hour storm event; rainfall events greater than the 25-year, 24-hour event will discharge via an emergency spillway to Pennyroyal Creek and the two pumps to the discharge canal. For this analysis, only the discharges to the discharge canal were considered. Therefore, the two pumps that are not inundated during extreme events were the only flows considered from the Slurry Pond 3 & 4. This conservatively estimates the flow from Slurry Pond 3 & 4 during extreme events as additional flow will discharge via the emergency spillway to Pennyroyal Creek. Portions of the coal pile drain to a pump station that can route water to Slurry Pond 3 & 4; however, for the purposes of this analysis, it was assumed that these pumps are not operational during extreme rainfall events and runoff from the coal pile flows directly into the discharge canal.

The West Ash Pond is approximately 64 acres (of the total 172 acres of the drainage area) and is located immediately south of Slurry Pond 3 & 4. The West Ash Pond gravity drains to Slurry Pond 3 & 4 through two 36 inch diameter culverts, four 22 inch diameter culverts, and an emergency spillway. The West Ash Pond and Slurry Pond 3 & 4 were modeled as a single drainage area and the hydraulic connection between the two ponds was not considered since the ultimate discharge to the cooling pond is controlled by the two pumps.

The developed drainage area of approximately 266 acres consists of the WGS power blocks, paved roadways, coal pile area, and associated impervious cover and infrastructure. The developed area drains directly into the intake canal or discharge canal. On-site stormwater infrastructure, such as smaller ponds, were not explicitly considered in this analysis; these ponds were conservatively considered to be completely full for modeling purposes.

The undeveloped area located in the southern portion of the WGS property to the west of the cooling pond and south of the South Ash Pond was not considered in this analysis. The undeveloped area consists of a combination of grass-covered land and wetland areas which drain to infrastructure immediately west of the cooling pond, in the form of long pond areas that intercept runoff from the undeveloped area and route it to the south and around the cooling pond via the Turkey Creek bypass channel (immediately east of the cooling pond) according to a Santee Cooper report (Carter, 1995). All of the undeveloped area was assumed to drain to the Turkey Creek bypass channel.

The cooling pond drainage area of approximately 550 acres is conservatively assumed to be entirely standing water. The outlet structure for the cooling pond consists of an emergency spillway at elevation 20.93 ft. An existing concrete riser structure is also located in the cooling pond but has been removed from operation and was not considered in this analysis; the emergency spillway was modeled in this analysis as the only outlet structure for the cooling pond. The elevation-storage curve for the cooling pond in Appendix 1 was derived from the Santee Cooper report (Carter, 1995).

The all the pre-development drainage areas drain directly to either the cooling pond or the intake canal or discharge canal which are hydraulically connected to the cooling pond.

3.4 Post-Development Condition

Figure 1 and Figure 2 of Appendix F-1 shows the delineation of drainage areas contributing to each drainage feature. Also, the final configuration of the proposed surface water management system design for the landfill is shown on the Engineering Drawings that accompany the permit application. The proposed surface water management system will utilize drainage terraces, downdrain pipes, perimeter drainage channels, and culverts to control surface water runoff from the site. The facility area associated with the proposed landfills (i.e., post-development conditions) is approximately 120.2 acres (37.2 acres for Area 1 Landfill and 83.0 acres for Area 2 Landfill). All contributing areas draining to the cooling pond outside of the landfill areas remain unchanged for post-development conditions.

3.5 Industrial Cooling Pond

The on-site cooling pond is incorporated in the pre-development and post-development condition analyses. The cooling pond is analyzed to verify containment of the 25-year, 24-

hour rainfall event for both pre-development and post-development runoff. The cooling pond is accounted for in the HEC-HMS program as "reservoir" node. The elevation-storage relationship is input for the cooling pond to describe the volume of storage provided by the cooling pond, which is computed based on the pond geometry (Carter, 1995). WGS National Pollutant Discharge Elimination System (NPDES) permit (Permit No. SC0022471) includes two outfalls for the cooling pond: Outfall 001 discharges to Turkey Creek and Outfall 002 discharges to the North Santee River. Outfall 001 consists of the emergency spillway, whereas Outfall 002 is operated by a pump station. For the purposes of the site watershed analysis, only Outfall 001 is utilized and the pump station at Outfall 002 was not considered. Input and output files for the site watershed analysis are provided in Appendix 1.

3.6 Nodal Network Diagrams

Nodal network diagrams used in HEC-HMS for the site watershed analysis are discussed below and correspond to the output files included in Appendix 1.

- Pre-Development Nodal Network – Figure 4 of this calculation package presents the nodal network drawing for the pre-development conditions. The pre-development nodal network diagram shows the subbasins and discharge locations on Figure 1.

- Post-Development Nodal Network – Figure 5 presents the nodal network drawing for the post-development conditions. The post-development nodal network diagram shows the landfill subbasins, reaches, and discharge locations together with the pre-development areas outside of the landfill areas draining to the cooling pond.

4. RESULTS

A summary of the results are presented in Table 8 of this calculation package, and detailed modeling results are in the appendix included with this calculation package. Table 8 summarizes analysis results for the pre- and post-development peak discharges and total runoff volumes from the site, as well as water surface elevations in the cooling pond. Inspection of Table 8 reveals that both pre-development and post-development flows result in zero-discharge from the cooling pond for the 25-year, 24-hour event.

Thus, the cooling pond is adequate to retain the runoff from the contributing site watershed (including developed landfill areas), and further; the landfill is not anticipated to adversely affect or significantly alter the drainage patterns in the vicinity of the site.

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TABLES

Table 1. Roughness Coefficient for Sheet Flow

(from USDA, 1986)

Surface description	n [1]
Smooth surfaces (concrete, asphalt, gravel, or bare soil)	0.011
Fallow (no residue)	0.05
Cultivated soils:	
Residue cover ≤20%	0.06
Residue cover >20%	0.17
Grass:	
Short grass prairie	0.15
Dense grasses [2]	0.24
Bermudagrass	0.41
Range (natural)	0.13
Woods: [3]	
Light underbrush	0.40
Dense underbrush	0.80

1 The n values are a composite of information compiled by Engman (1986).

2 Includes species such as weeping lovegrass, bluegrass, buffalo grass, blue grama grass, and native grass mixtures.

3 When selecting n , consider cover to a height of about 0.1 ft. This is the only part of the plant cover that will obstruct sheet flow.

Table 2. Precipitation Frequency for Georgetown, South Carolina (from NOAA, 2015)

NOAA Atlas 14, Volume 2, Version 3
Location name: Georgetown, South Carolina, US*
Latitude: 33.3343°, Longitude: -79.3551°
Elevation: 26 ft*
* source: Google Maps

POINT PRECIPITATION FREQUENCY ESTIMATES

G.M. Bonnin, D. Martin, B. Lin, T. Parzybok, M.Yekta, and D. Riley
NOAA, National Weather Service, Silver Spring, Maryland

PF tabular | PF graphical | Maps & aerials

PF tabular

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

Duration	Average recurrence interval (years)									
	1	2	5	10	25	50	100	200	500	1000
5-min	0.502 (0.468-0.543)	0.586 (0.545-0.636)	0.668 (0.619-0.722)	0.761 (0.703-0.823)	0.857 (0.789-0.927)	0.942 (0.863-1.02)	1.02 (0.929-1.10)	1.10 (0.993-1.19)	1.19 (1.07-1.29)	1.28 (1.14-1.39)
10-min	0.803 (0.748-0.868)	0.938 (0.872-1.02)	1.07 (0.992-1.16)	1.22 (1.13-1.32)	1.37 (1.26-1.48)	1.50 (1.38-1.62)	1.62 (1.48-1.75)	1.74 (1.57-1.88)	1.89 (1.69-2.04)	2.02 (1.79-2.19)
15-min	1.00 (0.935-1.09)	1.18 (1.10-1.28)	1.35 (1.25-1.46)	1.54 (1.42-1.67)	1.73 (1.59-1.87)	1.90 (1.74-2.05)	2.05 (1.87-2.22)	2.19 (1.99-2.37)	2.37 (2.13-2.57)	2.53 (2.25-2.75)
30-min	1.38 (1.28-1.49)	1.63 (1.51-1.77)	1.92 (1.78-2.08)	2.23 (2.06-2.41)	2.57 (2.36-2.77)	2.86 (2.62-3.09)	3.14 (2.86-3.39)	3.42 (3.09-3.69)	3.78 (3.38-4.09)	4.10 (3.64-4.45)
60-min	1.72 (1.60-1.85)	2.04 (1.90-2.22)	2.46 (2.29-2.67)	2.90 (2.68-3.14)	3.42 (3.14-3.69)	3.88 (3.55-4.19)	4.32 (3.94-4.67)	4.79 (4.34-5.18)	5.42 (4.86-5.87)	5.98 (5.32-6.50)
2-hr	2.08 (1.92-2.24)	2.49 (2.30-2.69)	3.07 (2.83-3.31)	3.66 (3.37-3.96)	4.37 (4.00-4.72)	5.01 (4.56-5.40)	5.63 (5.10-6.07)	6.27 (5.64-6.76)	7.10 (6.33-7.68)	7.85 (6.93-8.51)
3-hr	2.23 (2.06-2.42)	2.66 (2.45-2.91)	3.30 (3.03-3.59)	3.97 (3.64-4.33)	4.79 (4.37-5.21)	5.55 (5.02-6.03)	6.31 (5.66-6.84)	7.10 (6.33-7.71)	8.19 (7.20-8.91)	9.18 (7.98-10.0)
6-hr	2.66 (2.44-2.91)	3.19 (2.91-3.49)	3.94 (3.60-4.32)	4.75 (4.33-5.20)	5.76 (5.21-6.30)	6.69 (6.01-7.31)	7.61 (6.79-8.32)	8.62 (7.62-9.41)	9.97 (8.71-10.9)	11.2 (9.69-12.3)
12-hr	3.11 (2.84-3.44)	3.72 (3.39-4.11)	4.63 (4.21-5.12)	5.62 (5.10-6.20)	6.85 (6.17-7.54)	8.00 (7.15-8.78)	9.17 (8.12-10.0)	10.4 (9.15-11.4)	12.2 (10.5-13.4)	13.8 (11.8-15.2)
24-hr	3.64 (3.33-3.99)	4.42 (4.06-4.85)	5.71 (5.22-6.26)	6.77 (6.17-7.41)	8.28 (7.50-9.05)	9.53 (8.59-10.4)	10.9 (9.74-11.9)	12.3 (11.0-13.5)	14.4 (12.7-15.8)	16.1 (14.1-17.7)

Table 3. Upland Method Velocity Factors for Shallow Concentrated Flow (from HydroCAD, 2011)

Surface Description	K_v [ft/sec]	K_v [m/sec]
Paved	20.33	6.2
Unpaved	16.13	4.92
Grassed Waterway	15.0	4.57
Nearly Bare & Untilled	10.0	3.05
Cultivated Straight Rows	9.0	2.74
Short Grass Pasture	7.0	2.13
Woodland	5.0	1.52
Forest w/Heavy Litter	2.5	0.76

Table 4. Manning's n Values for Open Channels (from Chow, 1959)

Type of channel and description	Minimum	Normal	Maximum
C. EXCAVATED OR DREDGED			
a. Earth, straight and uniform			
1. Clean, recently completed	0.016	0.018	0.020
2. Clean, after weathering	0.018	0.022	0.025
3. Gravel, uniform section, clean	0.022	0.025	0.030
4. With short grass, few weeds	0.022	0.027	0.033
b. Earth, winding and sluggish			
1. No vegetation	0.023	0.025	0.030
2. Grass, some weeds	0.025	0.030	0.033
3. Dense weeds or aquatic plants in deep channels	0.030	0.035	0.040
4. Earth bottom and rubble sides	0.028	0.030	0.035
5. Stony bottom and weedy banks	0.025	0.035	0.040
6. Cobble bottom and clean sides	0.030	0.040	0.050
c. Dragline-excavated or dredged			
1. No vegetation	0.025	0.028	0.033
2. Light brush on banks	0.035	0.050	0.060
d. Rock cuts			
1. Smooth and uniform	0.025	0.035	0.040
2. Jagged and irregular	0.035	0.040	0.050
e. Channels not maintained, weeds and brush uncut			
1. Dense weeds, high as flow depth	0.050	0.080	0.120
2. Clean bottom, brush on sides	0.040	0.050	0.080
3. Same, highest stage of flow	0.045	0.070	0.110
4. Dense brush, high stage	0.080	0.100	0.140

Table 5. Hydrologic Soil Groups for On-Site Soils (from USDA, 2016)

Hydrologic Soil Group— Summary by Map Unit — Georgetown County, South Carolina (SC043)				
Map unit symbol	Map unit name	Rating	Acres in AOI	Percent of AOI
10	Leon sand, 0 to 2 percent slopes	A/D	198.2	4.3%
12A	Yauhannah loamy fine sand, 0 to 2 percent slopes	B/D	638.1	14.0%
13	Bladen loam	C/D	336.6	7.4%
18	Cape Fear loam	C/D	15.1	0.3%
20	Centenary fine sand	A	186.7	4.1%
24B	Chisolm sand, 0 to 4 percent slopes	A	55.0	1.2%
25A	Wakulla sand, 0 to 2 percent slopes	A	36.3	0.8%
26A	Eulonia loamy fine sand, 0 to 2 percent slopes	C/D	551.9	12.1%
27	Rutlege sand	A/D	89.1	2.0%
28	Echaw sand	A	503.0	11.0%
33	Hobonny muck	A/D	55.0	1.2%
34	Johnston loam	A/D	131.5	2.9%
36B	Lakeland fine sand, 0 to 6 percent slopes	A	5.4	0.1%
50	Lynn Haven sand	A/D	42.2	0.9%
54A	Chipley fine sand, 0 to 2 percent slopes	A	4.2	0.1%
55	Witherbee fine sand	A/D	447.4	9.8%
57	Grifton loamy fine sand	B/D	80.2	1.8%
58	Udorthents, loamy	B/D	410.4	9.0%
59	Wahee fine sandy loam	C/D	484.4	10.6%
61	Yemassee loamy fine sand	B/D	39.0	0.9%
W	Water		254.9	5.6%
Totals for Area of Interest			4,564.7	100.0%

Table 6. Summary of Runoff Curve Numbers (from USDA, 1986)

Table 2-2a Runoff curve numbers for urban areas [1]

Cover description: Cover type and hydrologic condition	Average percent impervious area [2]	Curve numbers for hydrologic soil group: A	B	C	D
Fully developed urban areas (vegetation established)					
Open space (lawns, parks, golf courses, cemeteries, etc.) [3]:					
Poor condition (grass cover < 50%)		68		86	89
Fair condition (grass cover 50% to 75%)		49	69	79	84
Good condition (grass cover > 75%)		39	61	74	80
Impervious areas:					
Paved parking lots, roofs, driveways, etc. (excluding right-of-way)		98	98	98	98
Streets and roads:					
Paved; curbs and storm sewers (excluding right-of-way)		98	98	98	98
Paved; open ditches (including right-of-way)		83	89	92	93
Gravel (including right-of-way)		76	85	89	91
Dirt (including right-of-way)		72	82	87	89
Western desert urban areas:					
Natural desert landscaping (pervious areas only) [4]		63	77	85	88
Artificial desert landscaping (impervious weed barrier, desert shrub with 1- to 2-inch sand or gravel mulch and basin borders)		96	96	96	96
Urban districts:					
Commercial and business	85	89	92	94	95
Industrial	72	81	88	91	93

Table 7. Composite Curve Numbers

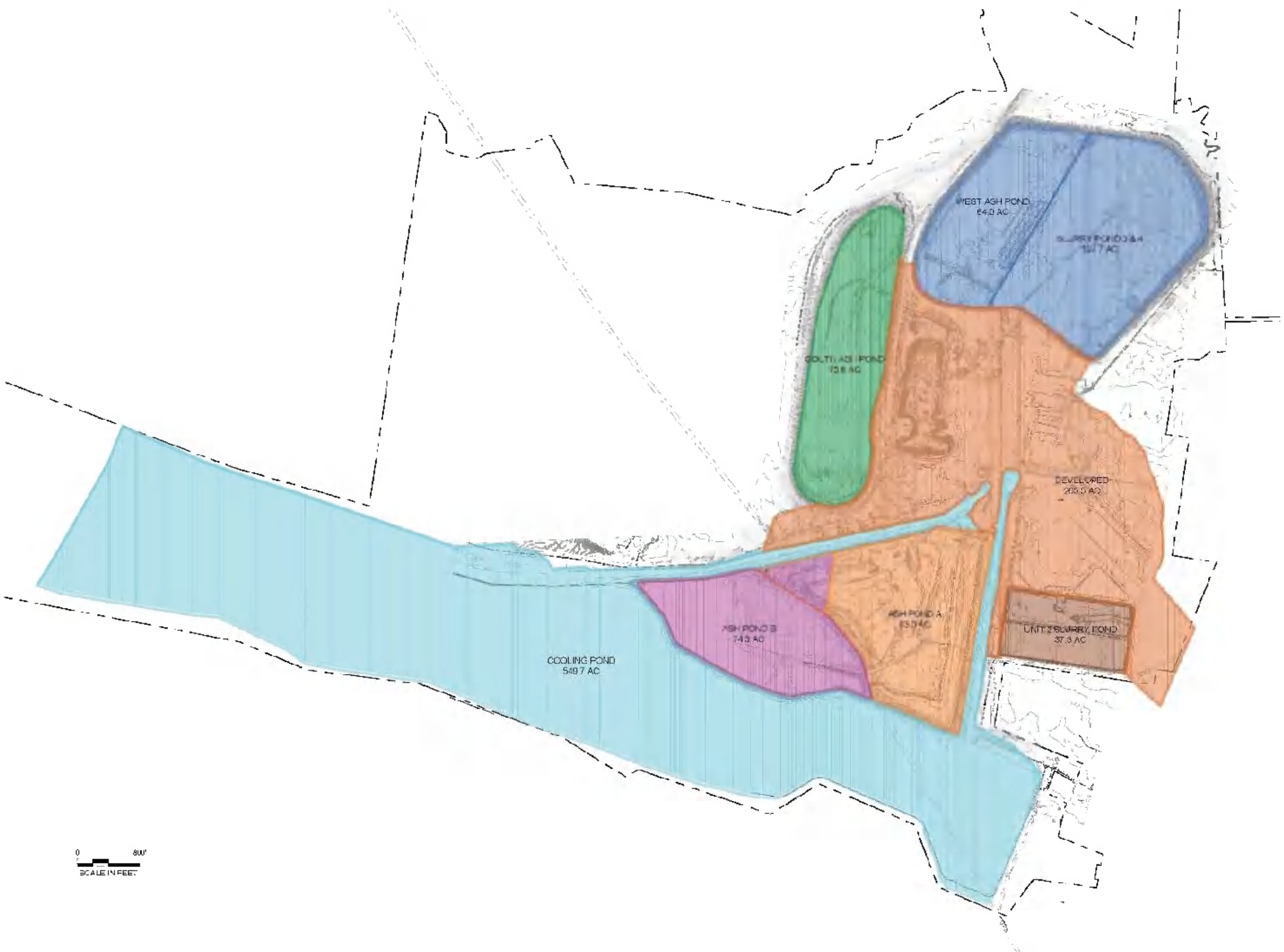
Area Designation	Cover Type 1	Cover Type 1 Area (ac)	Cover Type 1 CN	Cover Type 2	Cover Type 2 Area (ac)	Cover Type 2 CN	Composite CN
Developed	Industrial	265.5	93	N/A	N/A	N/A	93
Unit 2 Slurry Pond	Exposed Geomembrane	37.3	98	N/A	N/A	N/A	98
Slurry Pond 3&4 plus West Ash Pond	Open space with grass cover <50%	96.7	89	Water or Exposed Geomembrane	75.0	98	92.9
South Ash Pond	Open space with grass cover <50%	64.0	89	Water	11.6	98	90.4
Ash Pond A	Open space with grass cover <50%	83.0	89	N/A	N/A	N/A	89
Ash Pond B	Open space with grass cover <50%	67.3	89	Water	7.0	98	89.8
Cooling Pond	Water	549.7	98	N/A	N/A	N/A	98

Table 8. Summary of Peak Discharge and Total Discharge Volumes at Cooling Pond Outfall

	Pre-Development 25-Year, 24-Hour Event	Post-Development 25-Year, 24-Hour Event
Cooling Pond Peak Outflow (cfs)	0.0	0.0
Cooling Pond Peak Inflow (cfs)	4,273	4,855
Cooling Pond Inflow Volume (ac-ft)	704.7	691.0
Cooling Pond Water Surface Elevation (ft)	20.9	20.8

FIGURES

Figure 1. Pre-Development Drainage Plan



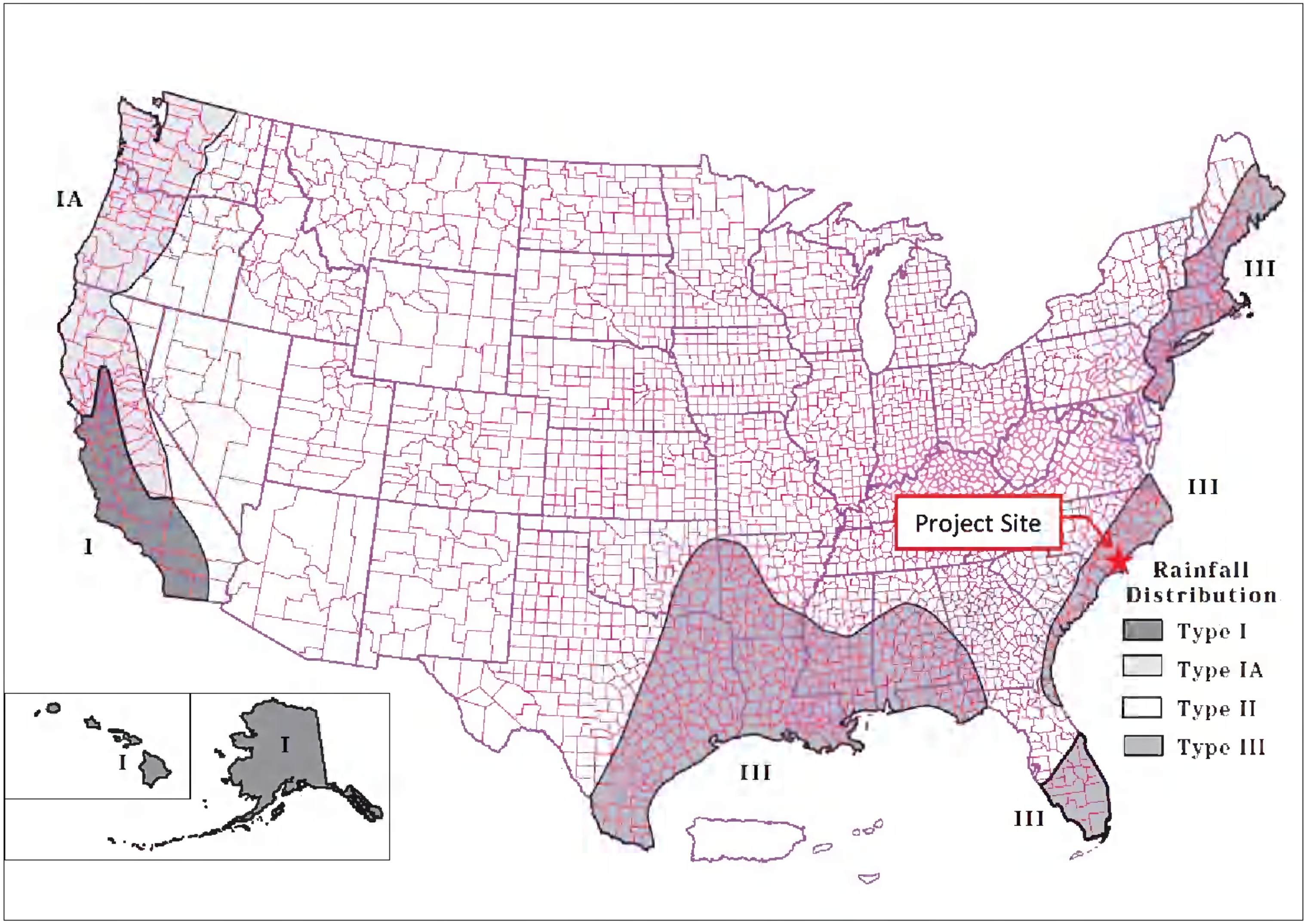


Figure 2. SCS Rainfall Distributions (from USDA, 1986)

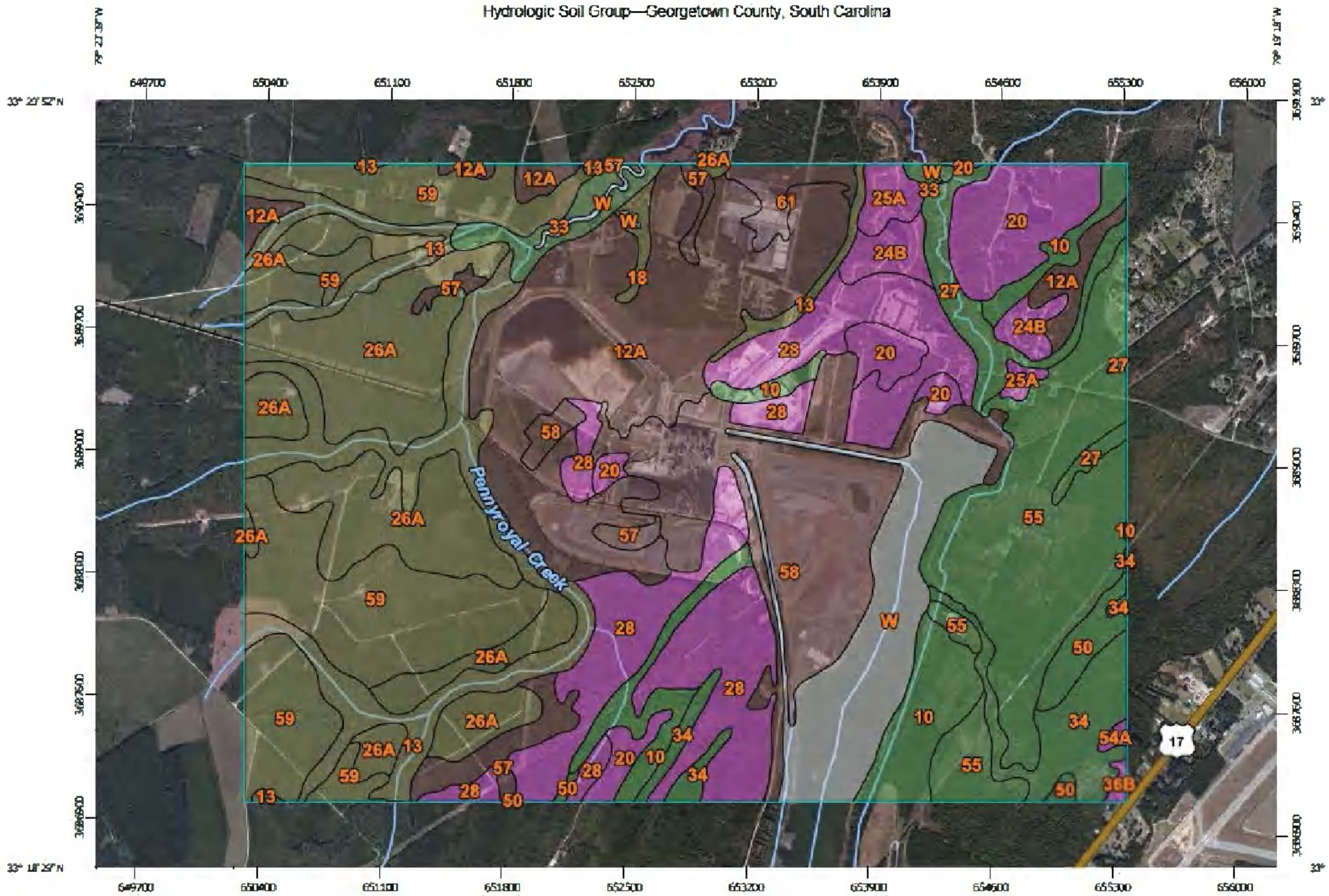


Figure 3. Soil Survey Map

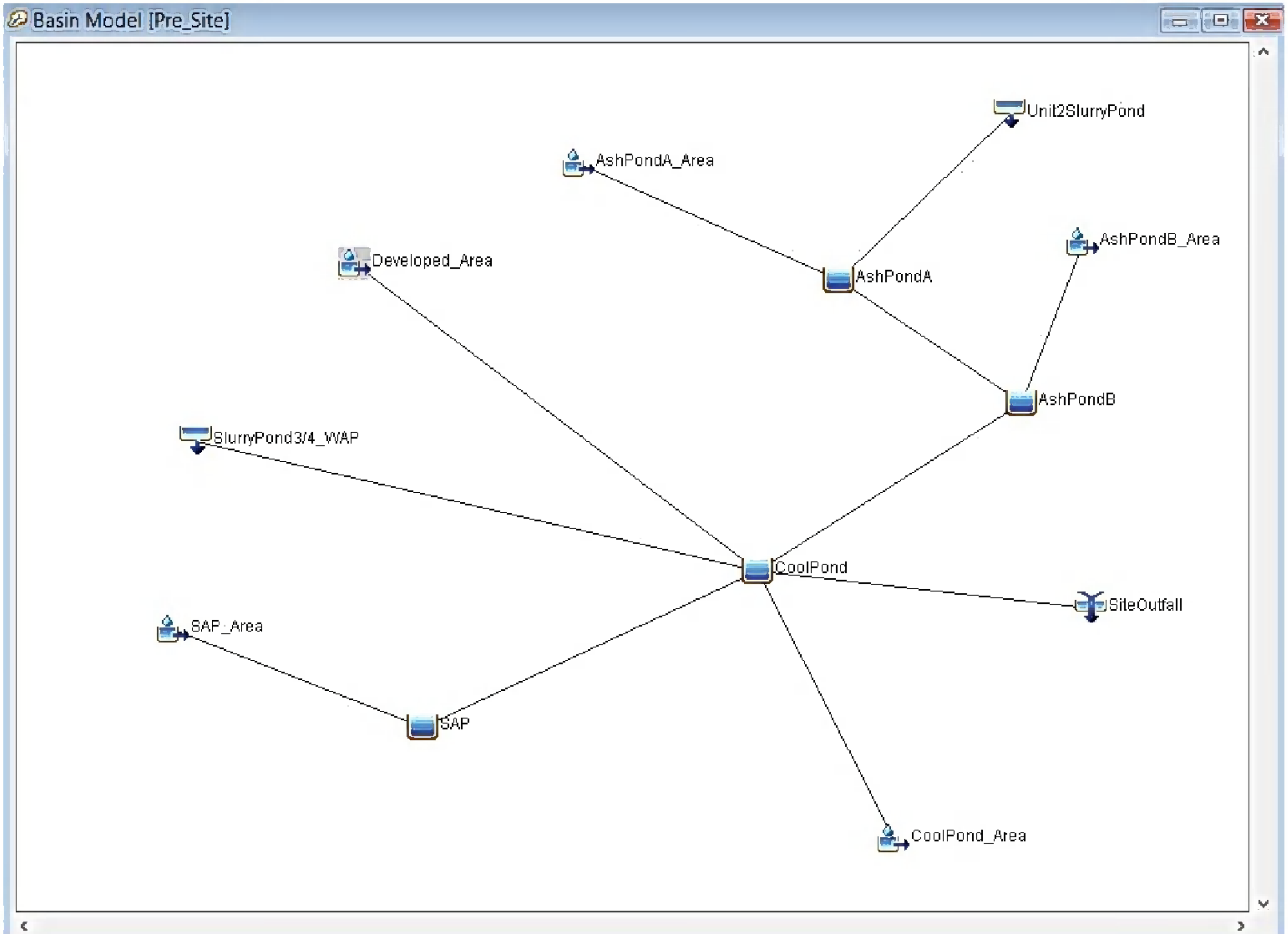


Figure 4. Pre-Development HEC-HMS Nodal Network

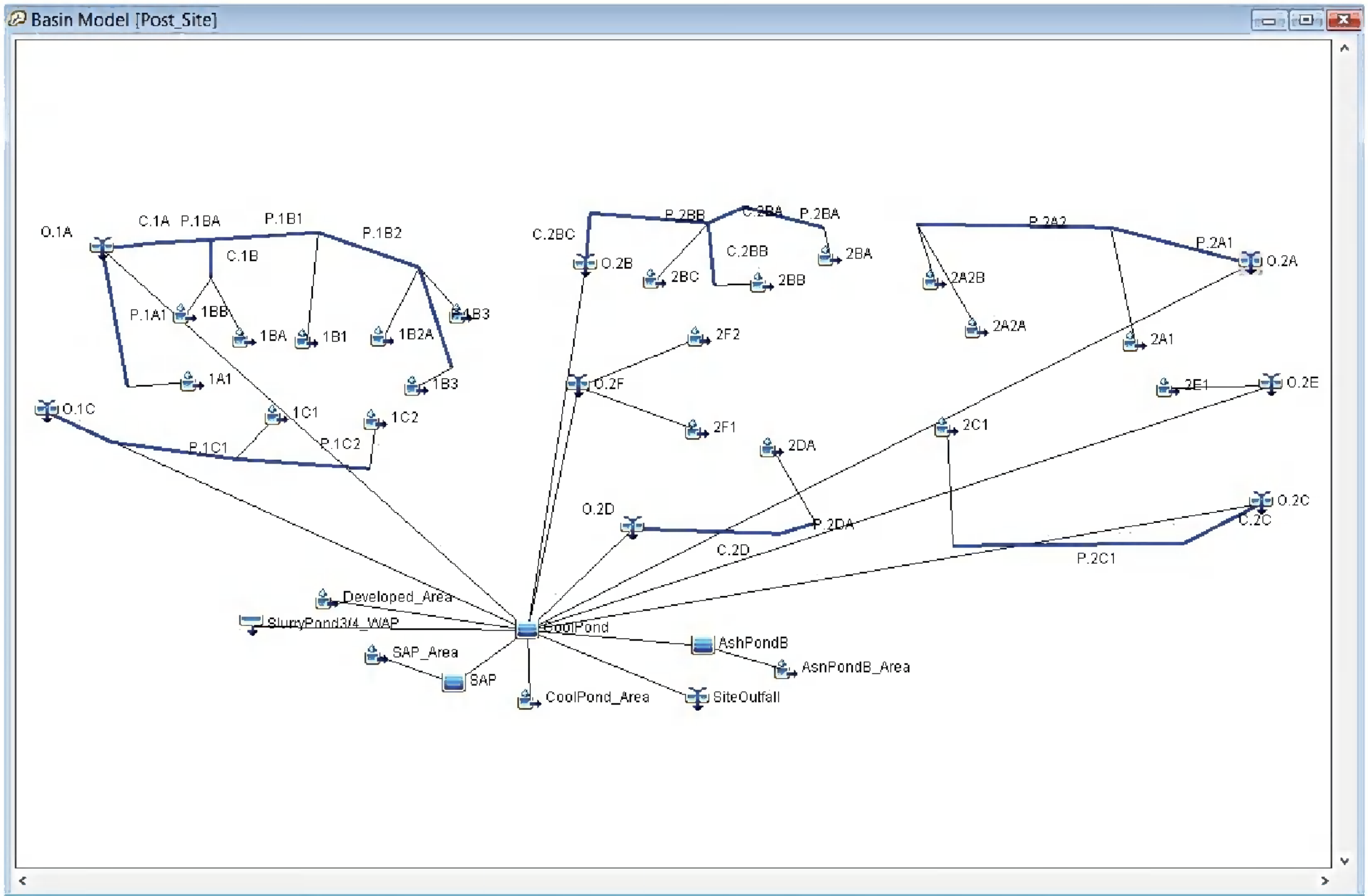


Figure 5. Post-Development HEC-HMS Nodal Network

APPENDIX 1

HEC-HMS HYDROLOGIC MODEL PARAMETERS

Table 1-1. Pre-Development 25-year, 24-hour Precipitation Event Nodal Areas, Peak Flow Rates, and Runoff Volumes

Global Summary Results for Run "Pre_Site_25-Year"

Project: Winyah Landfills_Areas 1 an   Simulation Run: Pre_Site_25-Year

Start of Run: 01Jan2016, 00:00   Basin Model: Pre_Site
End of Run: 06Jan2016, 00:00   Meteorologic Model: 25-Year
Compute Time: 04Aug2016, 18:21:34   Control Specifications: Control 1

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

Hydrologic Element	Drainage Area (MI2)	Peak Discharge (CFS)	Time of Peak	Volume (AC-FT)
AshPondA	0.18785	414.1	01Jan2016, 12:16	61.4
AshPondA_Area	0.12963	481.4	01Jan2016, 12:11	48.1
AshPondB	0.30388	57.8	01Jan2016, 15:19	113.5
AshPondB_Area	0.11603	245.7	01Jan2016, 12:38	43.7
CoolPond	1.96398	0.0	01Jan2016, 00:00	0.0
CoolPond_Area	0.85885	4052.2	01Jan2016, 12:05	368.3
Developed_Area	0.41491	578.1	01Jan2016, 13:17	164.6
SAP	0.11812	137.7	01Jan2016, 13:29	44.9
SAP_Area	0.11812	198.0	01Jan2016, 12:56	44.9
SiteOutfall	1.96398	0.0	01Jan2016, 00:00	0.0
SlurryPond3/4_WAP	0.26822	6.8	01Jan2016, 00:00	13.4
Unit2SlurryPond	0.05822	5.8	01Jan2016, 00:00	11.5

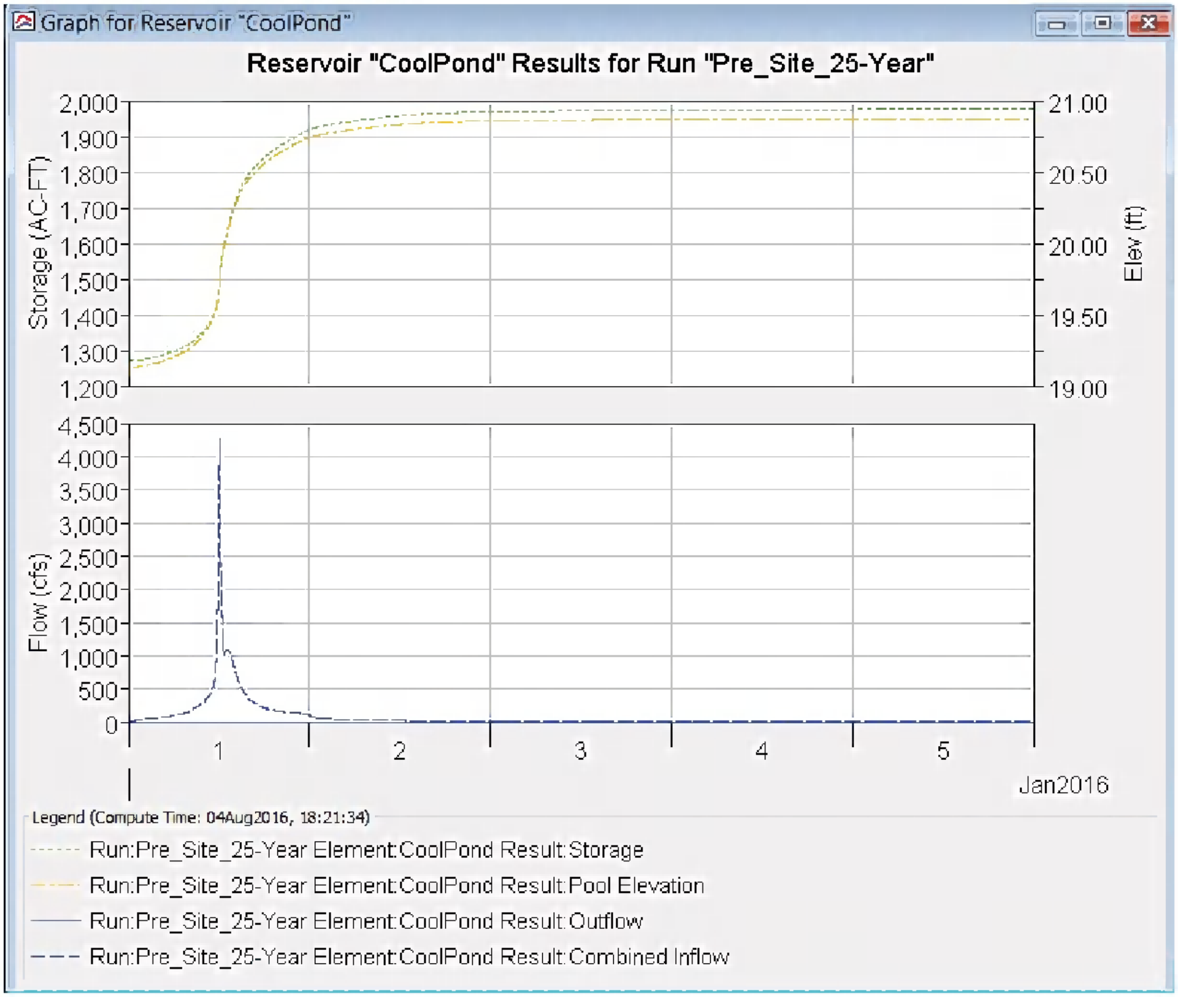


Figure 1-1. Pre-Development 24-year, 24-hour Precipitation Event Cooling Pond Surface Water Pond Hydrograph and Elevation/Storage Relationships

Table 1-2. Cooling Pond Elevation-Storage Relationship

Paired Data | Table | Graph

Elevation (FT)	Storage (AC-FT)
9.63	0.00001
11.63	30.00000
13.63	105.00000
15.63	290.00000
17.63	735.00000
19.63	1450.00000
20.63	1870.00000
20.93	2000.00000
21.63	2300.00000
23.63	3000.00000

Table 1-3. Ash Pond A Elevation-Area Relationship

Paired Data | Table | Graph

Elevation (FT)	Area (AC)
28.0	0.015
30.0	0.162
32.0	0.273
34.0	0.869
36.0	1.525
38.0	3.293
38.8	8.432

Table 1-4. Ash Pond B Elevation-Area Relationship

Paired Data | Table | Graph

Elevation (FT)	Area (AC)
34.00	0.006
36.00	30.021
38.00	59.773
39.68	62.211

Table 1-5. South Ash Pond Elevation-Area Relationship

Paired Data | Table | Graph

Elevation (FT)	Area (AC)
12.0	0.001
14.0	0.007
16.0	0.033
18.0	0.314
20.0	2.071
22.0	3.080
24.0	4.152
26.0	5.225
28.0	6.456
30.0	10.286
32.0	20.794
34.0	41.028
36.0	56.316
36.9	64.301

Table 1-6. Post-Development 25-year, 24-hour Precipitation Event Nodal Areas, Peak Flow Rates, and Runoff Volumes

Global Summary Results for Run "Post_Site_25-Year"

Project: Winyah Landfills_Areas 1 an Simulation Run: Post_Site_25-Year

Start of Run: 01Jan2016, 00:00 Basin Model: Post_Site
End of Run: 06Jan2016, 00:00 Meteorologic Model: 25-Year
Compute Time: 04Aug2016, 18:23:52 Control Specifications: Control 1

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

Hydrologic Element	Drainage Area (MI2)	Peak Discharge (CFS)	Time of Peak	Volume (AC-FT)
AshPondB	0.11603	29.8	01Jan2016, 14:56	40.7
AshPondB_Area	0.11603	245.7	01Jan2016, 12:38	43.7
CoolPond	1.96399	0.0	01Jan2016, 00:00	0.0
CoolPond_Area	0.85885	4052.2	01Jan2016, 12:05	368.3
C.1A	0.03378	115.8	01Jan2016, 12:10	10.6
C.1B	0.00414	16.3	01Jan2016, 12:05	1.3
C.1C	0.01677	59.5	01Jan2016, 12:10	5.3
C.2BA	0.00378	14.9	01Jan2016, 12:06	1.2
C.2BB	0.00291	11.5	01Jan2016, 12:05	0.9
C.2BC	0.00924	35.1	01Jan2016, 12:09	2.9
C.2C	0.02730	102.7	01Jan2016, 12:10	8.6
C.2D	0.00602	23.6	01Jan2016, 12:06	1.9
Developed_Area	0.41491	578.1	01Jan2016, 13:17	164.6
O.1A	0.04147	143.9	01Jan2016, 12:10	13.0
O.1C	0.01677	59.5	01Jan2016, 12:10	5.3
O.2A	0.02405	89.5	01Jan2016, 12:09	7.6
O.2B	0.00924	35.1	01Jan2016, 12:09	2.9
O.2C	0.02730	102.7	01Jan2016, 12:10	8.6
O.2D	0.00602	23.6	01Jan2016, 12:06	1.9
O.2E	0.01264	49.0	01Jan2016, 12:06	4.0
O.2F	0.05037	187.1	01Jan2016, 12:07	15.8
P.1A1	0.00769	28.1	01Jan2016, 12:09	2.4
P.1BA	0.03378	115.9	01Jan2016, 12:09	10.6
P.1B1	0.02964	102.7	01Jan2016, 12:10	9.3
P.1B2	0.02108	73.4	01Jan2016, 12:10	6.6
P.1B3	0.00838	29.6	01Jan2016, 12:11	2.6
P.1C1	0.01677	59.6	01Jan2016, 12:10	5.3
P.1C2	0.00533	20.1	01Jan2016, 12:08	1.7

Table 1-6 (Continued). Post-Development 25-year, 24-hour Precipitation Event Nodal Areas, Peak Flow Rates, and Runoff Volumes

Global Summary Results for Run "Post_Site_25-Year"

Project: Winyah Landfills_Areas 1 an   Simulation Run: Post_Site_25-Year

Start of Run: 01Jan2016, 00:00   Basin Model: Post_Site
End of Run: 06Jan2016, 00:00   Meteorologic Model: 25-Year
Compute Time: 04Aug2016, 18:23:52   Control Specifications: Control 1

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

Hydrologic Element	Drainage Area (MI2)	Peak Discharge (CFS)	Time of Peak	Volume (AC-FT)
P.2A1	0.02405	89.5	01Jan2016, 12:09	7.6
P.2A2	0.00602	23.7	01Jan2016, 12:08	1.9
P.2BA	0.00378	14.9	01Jan2016, 12:05	1.2
P.2BB	0.00924	35.2	01Jan2016, 12:08	2.9
P.2C1	0.02730	102.8	01Jan2016, 12:10	8.6
P.2DA	0.00602	23.6	01Jan2016, 12:05	1.9
SAP	0.11812	137.7	01Jan2016, 13:29	44.9
SAP_Area	0.11812	198.0	01Jan2016, 12:56	44.9
SiteOutfall	1.96399	0.0	01Jan2016, 00:00	0.0
SlurryPond3/4_WAP	0.26822	6.8	01Jan2016, 00:00	13.4
1A1	0.00769	28.3	01Jan2016, 12:07	2.4
1BA	0.00346	13.7	01Jan2016, 12:05	1.1
1BB	0.00068	2.7	01Jan2016, 12:05	0.2
1B1	0.00856	32.3	01Jan2016, 12:07	2.7
1B2A	0.00200	7.9	01Jan2016, 12:05	0.6
1B2B	0.01070	41.1	01Jan2016, 12:06	3.4
1B3	0.00838	29.8	01Jan2016, 12:08	2.6
1C1	0.01144	39.7	01Jan2016, 12:09	3.6
1C2	0.00533	20.1	01Jan2016, 12:06	1.7
2A1	0.01803	66.0	01Jan2016, 12:08	5.7
2A2A	0.00336	13.3	01Jan2016, 12:05	1.1
2A2B	0.00266	10.5	01Jan2016, 12:05	0.8
2BA	0.00378	14.9	01Jan2016, 12:05	1.2
2BB	0.00291	11.5	01Jan2016, 12:05	0.9
2BC	0.00255	9.3	01Jan2016, 12:08	0.8
2C1	0.02730	103.3	01Jan2016, 12:06	8.6
2DA	0.00602	23.7	01Jan2016, 12:05	1.9
2E1	0.01264	49.0	01Jan2016, 12:06	4.0
2F1	0.02332	87.2	01Jan2016, 12:07	7.3
2F2	0.02705	100.0	01Jan2016, 12:07	8.5

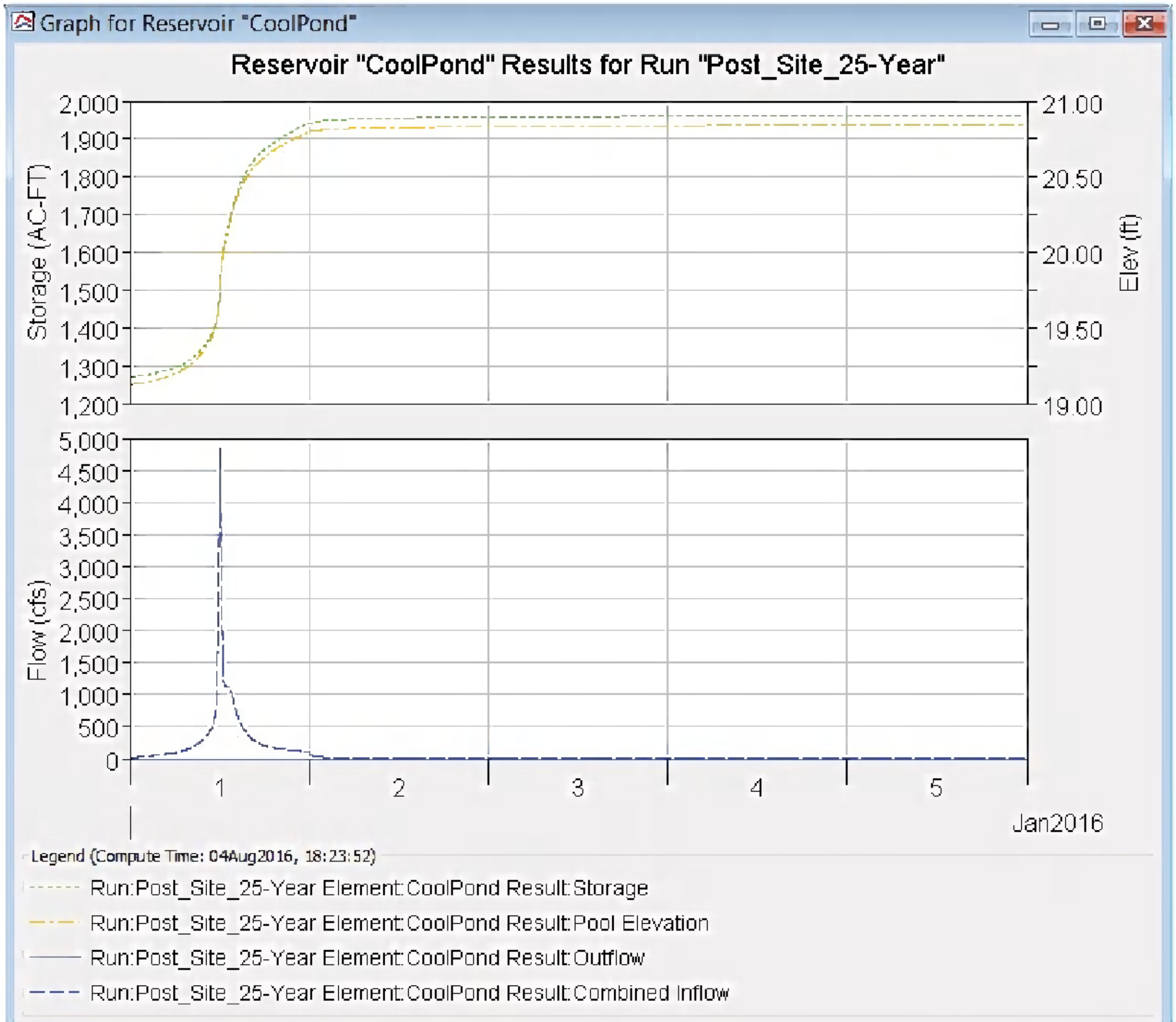


Figure 1-2. Post-Development 24-year, 24-hour Precipitation Event Cooling Pond Surface Water Pond Hydrograph and Elevation/Storage Relationships

HEC-HMS PRE-DEVELOPMENT HYDROLOGIC MODEL INPUT PARAMETERS

Pre-Development HEC-HMS Basin Input Parameters for Kinematic Wave Model

PRE-DEVELOPMENT CONDITIONS

			Watershed Characterization			Sheet Flow				Shallow Concentrated Flow					Open Channel Flow											
Subcatchment Designation	Area A (mi^2)	Area A (acres)	Initial Abstraction (in)	Curve Number	Impervious Cover (%)	Flow Length (ft)	Manning's n	Slope (ft/ft)	Time T_t (min)	Flow Length (ft)	Velocity Factor (ft/s)	Slope (ft/ft)	Average Velocity (ft/s)	Time T_t (min)	Flow Length (ft)	Depth d (ft)	Area A (ft^2)	Wetted P (ft)	Hydraulic Radius (ft)	Manning's n	Slope (ft/ft)	Velocity (ft/s)	Time T_t (min)	Design T_c (min)	SCS Lag Time (min)	HMS 25-yr Flow (cfs)
Developed	0.41491	265.5	0.15	93.0	0.00	300	0.40	0.010	57.5	2777	10.00	0.005	0.71	65.46										123	73.8	578.1
Slurry Pond 3&4 +WAP	0.26822	171.7	0.15	92.9	0.00	218	0.05	0.018	6.6	3036	10.00	0.0003	0.18	278.84	1475	2.0	62.00	37.65	1.65	0.028	0.005	5.25	4.69	290	174.1	6.8
SAP	0.11812	75.6	0.21	90.4	0.00	300	0.05	0.007	12.8	3223	10.00	0.005	0.71	75.98										89	53.3	198.0
Ash Pond A	0.12963	83.0	0.25	89.0	0.00	151	0.05	0.013	5.6						3152	2.0	62.00	37.65	1.65	0.028	0.005	5.25	10.01	16	9.4	481.4
Ash Pond B	0.11603	74.3	0.23	89.8	0.00	300	0.05	0.005	14.4	1737	10.00	0.005	0.71	40.94	1000	2.0	22.00	17.65	1.25	0.028	0.005	4.36	3.82	59	35.5	245.7
Unit 2 Slurry Pond	0.05822	37.3	0.04	98.0	0.00	142	0.05	0.014	5.2	3261	10.00	0.005	0.68	80.15										85	51.2	5.8

2-year, 24-hour Design Rainfall Depth = 4.50 inches

Perimeter Channel Left Side Slope = 3.0 H:V
Perimeter Channel Right Side Slope = 3.0 H:V
Perimeter Channel Bottom Width = 25.0 ft
Interior Channel Bottom Width = 5.0 ft

Notes:

1) *Curve number = 93 represents industrial or fully developed urban areas for hydrologic soil group D (USDA, 1986).*
2) *Curve number = 89 represents open space with no grass for hydrologic soil group D (USDA, 1986).*
3) *Curve number = 90 represents 10 % of water with open space with no grass for hydrologic soil group D (USDA, 1986).*
4) *Curve number = 73 represents brush-weed grass mixture with good cover for hydrologic soil group D (USDA, 1986).*
5) *Manning's roughness coefficient: n = 0.40 represents woods with light underbrush for sheet flow (USDA, 1984).*
6) *Manning's roughness coefficient: n = 0.05 represents fallow with no residue for sheet flow (USDA, 1984).*
7) *Manning's roughness coefficient: n = 0.028 represents a dragline-excavated or dredged channel with no vegetation (Chow, 1959).*
8) *Travel Time (T_t) is calculated using Manning's kinematic solutions for sheet flow (USDA, 1986).*

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

9) *Velocity factor of 10.0 ft/s corresponds to a value for nearly bare and untilled surfaces from the Upland Method as reported by HydroCAD v.10 Owner's Manual.*
10) *Open channel flow velocity is calculated using Manning's equation (USDA, 1986).*

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

Basin: Pre_Site
Last Modified Date: 1 August 2016
Last Modified Time: 13:47:27
Version: 4.1
Filepath Separator: \
Unit System: English
Missing Flow To Zero: No
Enable Flow Ratio: No
Compute Local Flow At Junctions: No

Enable Sediment Routing: No

Enable Quality Routing: No
End:

Subbasin: CoolPond_Area
Last Modified Date: 30 July 2016
Last Modified Time: 22:00:38
Canvas X: 16949.950824572217
Canvas Y: -22428.927249820517
Area: 0.85885
Downstream: CoolPond

Canopy: None
Plant Uptake Method: None

Surface: None

LossRate: SCS
Percent Impervious Area: 0.0
Curve Number: 98

Transform: SCS
Lag: 3.6
Unitgraph Type: STANDARD

Baseflow: None
End:

Subbasin: Developed_Area
Last Modified Date: 3 August 2016
Last Modified Time: 17:50:42
Canvas X: 5771.395371927307
Canvas Y: -10492.754390167414
Area: 0.41491
Downstream: CoolPond

Canopy: None
Plant Uptake Method: None

Surface: None

LossRate: SCS
Percent Impervious Area: 0.0
Curve Number: 93

Transform: SCS
Lag: 73.8
Unitgraph Type: STANDARD

Baseflow: None
End:

Subbasin: AshPondA_Area
Last Modified Date: 3 August 2016
Last Modified Time: 17:50:12
Canvas X: 10414.849851559466
Canvas Y: -8423.35429603773
Area: 0.12963
Downstream: AshPondA

Canopy: None
Plant Uptake Method: None

Surface: None

LossRate: SCS
Percent Impervious Area: 0.0
Curve Number: 89

Transform: SCS
Lag: 9.4
Unitgraph Type: STANDARD

Baseflow: None
End:

Source: Unit2SlurryPond
Last Modified Date: 3 August 2016
Last Modified Time: 17:50:11
Canvas X: 19371.13211305606
Canvas Y: -7389.937112018892
Area: 0.05822
Downstream: AshPondA

Flow Method: GAGE_FLOW
Flow Gage: SlurryPond2_DG
End Flow Method:
End:

Reservoir: AshPondA
Last Modified Date: 3 August 2016
Last Modified Time: 17:50:09
Canvas X: 15811.584034768952
Canvas Y: -10834.661058748352
Downstream: AshPondB

Route: Controlled Outflow
Routing Curve: Elevation-Area
Initial Outflow Equals Inflow: Yes
Elevation-Area Table: Ash Pond A_EA
Adaptive Control: On
Main Tailwater Condition: None
Auxiliary Tailwater Condition: None

Conduit: Culvert
Conduit Outlet: Main
Culvert Shape: Circular
Chart Number: 1
Scale Number: 1
Solution Control: Automatic
Diameter: 2.5
Number Barrels: 1
Culvert Length: 40.8
Entrance Loss Coefficient: 0.5
Exit Loss Coefficient: 1
Top Manning's n: 0.025
Inlet Invert Elevation: 37.5
Outlet Invert Elevation: 36.52
End Conduit:

Conduit: Culvert
Conduit Outlet: Main
Culvert Shape: Circular

Chart Number: 1
Scale Number: 1
Solution Control: Automatic
Diameter: 4
Number Barrels: 1
Culvert Length: 30.9
Entrance Loss Coefficient: 0.5
Exit Loss Coefficient: 1
Top Manning's n: 0.012
Inlet Invert Elevation: 35.49
Outlet Invert Elevation: 35.28
End Conduit:

Conduit: Culvert
Conduit Outlet: Main
Culvert Shape: Circular
Chart Number: 1
Scale Number: 1
Solution Control: Automatic
Diameter: 3.5
Number Barrels: 1
Culvert Length: 24.6
Entrance Loss Coefficient: 0.5
Exit Loss Coefficient: 1
Top Manning's n: 0.012
Inlet Invert Elevation: 36.2
Outlet Invert Elevation: 35.7
End Conduit:

Spillway: Broad-Crested Spillway
Spillway Outlet: Main
Spillway Crest Length: 100
Spillway Crest Elevation: 37
Spillway Coefficient: 2.6
End Spillway:

Evaporation Method: Zero Evaporation
End Evaporation:
End:

Subbasin: AshPondB_Area
Last Modified Date: 3 August 2016
Last Modified Time: 17:50:35
Canvas X: 20878.91005805196
Canvas Y: -10038.703402333067
Area: 0.11603
Downstream: AshPondB

Canopy: None
Plant Uptake Method: None

Surface: None

LossRate: SCS
Percent Impervious Area: 0.0
Curve Number: 89.9

Transform: SCS
Lag: 35.5
Unitgraph Type: STANDARD

Baseflow: None
End:

Reservoir: AshPondB
Last Modified Date: 3 August 2016
Last Modified Time: 17:49:29
Canvas X: 19606.460413120556
Canvas Y: -13380.19146382773
Downstream: CoolPond

Route: Controlled Outflow
Routing Curve: Elevation-Area
Initial Outflow Equals Inflow: Yes
Elevation-Area Table: Ash Pond B_EA
Adaptive Control: On
Main Tailwater Condition: None
Auxiliary Tailwater Condition: None

Conduit: Culvert
Conduit Outlet: Main
Culvert Shape: Circular
Chart Number: 1
Scale Number: 1
Solution Control: Automatic
Diameter: 1.8
Number Barrels: 1
Culvert Length: 113.3
Entrance Loss Coefficient: 0.5
Exit Loss Coefficient: 1
Top Manning's n: 0.013
Inlet Invert Elevation: 34
Outlet Invert Elevation: 17.99
End Conduit:

Spillway: Broad-Crested Spillway
Spillway Outlet: Main
Spillway Crest Length: 4
Spillway Crest Elevation: 34.9
Spillway Coefficient: 3
End Spillway:

Evaporation Method: Zero Evaporation
End Evaporation:
End:

Source: SlurryPond3/4_WAP
Last Modified Date: 3 August 2016
Last Modified Time: 17:50:40
Canvas X: 2469.2063694956887
Canvas Y: -14166.439655372591
Area: 0.26822
Downstream: CoolPond

Flow Method: GAGE_FLOW
Flow Gage: SlurryPond-WAP_DG
End Flow Method:
End:

Subbasin: SAP_Area
Last Modified Date: 3 August 2016
Last Modified Time: 17:50:39
Canvas X: 2015.1553816613414
Canvas Y: -18087.789095760138
Area: 0.11812
Downstream: SAP

Canopy: None
Plant Uptake Method: None

Surface: None

LossRate: SCS
Percent Impervious Area: 0.0

Curve Number: 90.4

Transform: SCS
Lag: 53.3
Unitgraph Type: STANDARD

Baseflow: None
End:

Reservoir: SAP
Last Modified Date: 3 August 2016
Last Modified Time: 17:50:37
Canvas X: 7174.825697960743
Canvas Y: -20110.379859749504
Downstream: CoolPond

Route: Controlled Outflow
Routing Curve: Elevation-Area
Initial Outflow Equals Inflow: Yes
Elevation-Area Table: SAP_EA
Adaptive Control: On
Main Tailwater Condition: None
Auxiliary Tailwater Condition: None

Spillway: Broad-Crested Spillway
Spillway Outlet: Main
Spillway Crest Length: 30
Spillway Crest Elevation: 28.73
Spillway Coefficient: 3
End Spillway:

Evaporation Method: Zero Evaporation
End Evaporation:
End:

Reservoir: CoolPond
Last Modified Date: 5 August 2016
Last Modified Time: 01:22:36
Canvas X: 14164.3532470499
Canvas Y: -16841.59376915017
Downstream: SiteOutfall

Route: Controlled Outflow
Routing Curve: Elevation-Storage
Initial Elevation: 19.13
Elevation-Storage Table: ES_Cool_Pond
Adaptive Control: On
Main Tailwater Condition: None
Auxiliary Tailwater Condition: None

Spillway: Broad-Crested Spillway
Spillway Outlet: Main
Spillway Crest Length: 63.29
Spillway Crest Elevation: 20.93
Spillway Coefficient: 3
End Spillway:

Evaporation Method: Zero Evaporation
End Evaporation:
End:

Junction: SiteOutfall
Last Modified Date: 3 August 2016
Last Modified Time: 17:50:33
Canvas X: 21044.01950817354
Canvas Y: -17633.73810792579
End:

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

HEC-HMS POST-DEVELOPMENT HYDROLOGIC MODEL INPUT PARAMETERS

Basin: Post_Site
Last Modified Date: 30 July 2016
Last Modified Time: 21:57:03
Version: 4.1
Filepath Separator: \
Unit System: English
Missing Flow To Zero: No
Enable Flow Ratio: No
Compute Local Flow At Junctions: No

Enable Sediment Routing: No

Enable Quality Routing: No
End:

Subbasin: CoolPond_Area
Last Modified Date: 3 August 2016
Last Modified Time: 17:52:03
Canvas X: 2821.678332635842
Canvas Y: -16678.674874247386
Area: 0.85885
Downstream: CoolPond

Canopy: None
Plant Uptake Method: None

Surface: None

LossRate: SCS
Percent Impervious Area: 0.0
Curve Number: 98

Transform: SCS
Lag: 3.6
Unitgraph Type: STANDARD

Baseflow: None
End:

Subbasin: Developed_Area
Last Modified Date: 3 August 2016
Last Modified Time: 17:51:48
Canvas X: -9203.552192078438
Canvas Y: -10741.21730266971
Area: 0.41491
Downstream: CoolPond

Canopy: None
Plant Uptake Method: None

Surface: None

LossRate: SCS
Percent Impervious Area: 0.0
Curve Number: 93

Transform: SCS
Lag: 73.8
Unitgraph Type: STANDARD

Baseflow: None
End:

Source: SlurryPond3/4_WAP
Last Modified Date: 3 August 2016
Last Modified Time: 17:51:50
Canvas X: -13637.855948066828
Canvas Y: -12319.528809038457
Area: 0.26822
Downstream: CoolPond

Flow Method: GAGE_FLOW
Flow Gage: SlurryPond-WAP_DG
End Flow Method:
End:

Subbasin: SAP_Area
Last Modified Date: 3 August 2016
Last Modified Time: 17:52:01
Canvas X: -6272.402251679334
Canvas Y: -14048.15569696614
Area: 0.11812
Downstream: SAP

Canopy: None
Plant Uptake Method: None

Surface: None

LossRate: SCS
Percent Impervious Area: 0.0
Curve Number: 90.4

Transform: SCS
Lag: 53.3
Unitgraph Type: STANDARD

Baseflow: None
End:

Reservoir: SAP
Last Modified Date: 3 August 2016
Last Modified Time: 17:52:00
Canvas X: -1687.783114132013
Canvas Y: -15701.62489411435
Downstream: CoolPond

Route: Controlled Outflow
Routing Curve: Elevation-Area
Initial Outflow Equals Inflow: Yes
Elevation-Area Table: SAP_EA
Adaptive Control: On
Main Tailwater Condition: None
Auxiliary Tailwater Condition: None

Spillway: Broad-Crested Spillway
Spillway Outlet: Main
Spillway Crest Length: 30
Spillway Crest Elevation: 28.73
Spillway Coefficient: 3
End Spillway:

Evaporation Method: Zero Evaporation
End Evaporation:
End:

Subbasin: AshPondB_Area
Last Modified Date: 3 August 2016
Last Modified Time: 17:52:13
Canvas X: 18003.53187008762
Canvas Y: -14874.890295540245
Area: 0.11603
Downstream: AshPondB

Canopy: None
Plant Uptake Method: None

Surface: None

LossRate: SCS
Percent Impervious Area: 0.0
Curve Number: 89.9

Transform: SCS
Lag: 35.5
Unitgraph Type: STANDARD

Baseflow: None
End:

Reservoir: AshPondB
Last Modified Date: 3 August 2016
Last Modified Time: 17:52:09
Canvas X: 13118.281969422442
Canvas Y: -13446.894170730426
Downstream: CoolPond

Route: Controlled Outflow
Routing Curve: Elevation-Area
Initial Outflow Equals Inflow: Yes
Elevation-Area Table: Ash Pond B_EA
Adaptive Control: On
Main Tailwater Condition: None
Auxiliary Tailwater Condition: None

Conduit: Culvert
Conduit Outlet: Main
Culvert Shape: Circular
Chart Number: 1
Scale Number: 1
Solution Control: Automatic
Diameter: 1.8
Number Barrels: 1
Culvert Length: 113.3
Entrance Loss Coefficient: 0.5
Exit Loss Coefficient: 1
Top Manning's n: 0.013
Inlet Invert Elevation: 34
Outlet Invert Elevation: 17.99
End Conduit:

Spillway: Broad-Crested Spillway
Spillway Outlet: Main
Spillway Crest Length: 4
Spillway Crest Elevation: 34.9
Spillway Coefficient: 3
End Spillway:

Evaporation Method: Zero Evaporation
End Evaporation:
End:

Subbasin: 2F2
Last Modified Date: 1 July 2016
Last Modified Time: 16:18:23
Canvas X: 12885.616843649492
Canvas Y: 4794.7598030265435
Area: 0.02705
Downstream: O.2F

Canopy: None
Plant Uptake Method: None

Surface: None

LossRate: SCS
Percent Impervious Area: 0.0
Curve Number: 80

Transform: SCS
Lag: 5.77
Unitgraph Type: STANDARD

Baseflow: None
End:

Subbasin: 2F1
Last Modified Date: 1 July 2016
Last Modified Time: 16:18:23
Canvas X: 12744.106111734043
Canvas Y: -677.804127648571
Area: 0.02332
Downstream: O.2F

Canopy: None
Plant Uptake Method: None

Surface: None

LossRate: SCS
Percent Impervious Area: 0.0
Curve Number: 80

Transform: SCS
Lag: 5.44
Unitgraph Type: STANDARD

Baseflow: None
End:

Junction: O.2F
Last Modified Date: 30 July 2016
Last Modified Time: 21:57:46
Canvas X: 5692.872341359882

Canvas Y: 1847.8327345456855
Downstream: CoolPond
End:

Subbasin: 1B2B
Last Modified Date: 1 July 2016
Last Modified Time: 16:16:48
Canvas X: -1231.8523918511928
Canvas Y: 6143.726782000704
From Canvas X: 10493.873085339166
From Canvas Y: 362.69146608314986
Label X: 0.0
Label Y: 1.0
Area: 0.01070
Downstream: P.1B2

Canopy: None
Plant Uptake Method: None

Surface: None

LossRate: SCS
Percent Impervious Area: 0.0
Curve Number: 80

Transform: SCS
Lag: 4.58
Unitgraph Type: STANDARD

Baseflow: None
End:

Subbasin: 1B3
Last Modified Date: 30 July 2016
Last Modified Time: 20:36:01
Canvas X: -3887.0462606038272
Canvas Y: 1894.7386703252268
Area: 0.00838
Downstream: P.1B3

Canopy: None
Plant Uptake Method: None

Surface: None

LossRate: SCS
Percent Impervious Area: 0.0
Curve Number: 80

Transform: SCS
Lag: 6.93
Unitgraph Type: STANDARD

Baseflow: None
End:

Reach: P.1B3
Last Modified Date: 29 July 2016
Last Modified Time: 12:57:46
Canvas X: -3713.78669946232
Canvas Y: 8938.33764631685
From Canvas X: -1811.6481445504396
From Canvas Y: 3056.9616153151255
Downstream: P.1B2

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

Subbasin: 1B2A
Last Modified Date: 29 July 2016
Last Modified Time: 12:56:14
Canvas X: -5879.428452015083
Canvas Y: 4800.296032799975
Area: 0.00200
Downstream: P.1B2

Canopy: None
Plant Uptake Method: None

Surface: None

LossRate: SCS
Percent Impervious Area: 0.0
Curve Number: 80

Transform: SCS
Lag: 3.79
Unitgraph Type: STANDARD

Baseflow: None
End:

Reach: P.1B2
Last Modified Date: 29 July 2016
Last Modified Time: 12:57:00
Canvas X: -9744.809909356078
Canvas Y: 11051.39687314094
From Canvas X: -3713.78669946232
From Canvas Y: 8938.33764631685
Label X: -18.0
Label Y: 11.0
Downstream: P.1B1

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

End:

Subbasin: 1B1
Last Modified Date: 30 July 2016
Last Modified Time: 20:36:35
Canvas X: -10405.140917738609
Canvas Y: 4668.197125443163
Area: 0.00856
Downstream: P.1B1

Canopy: None
Plant Uptake Method: None

Surface: None

LossRate: SCS
Percent Impervious Area: 0.0
Curve Number: 80

Transform: SCS
Lag: 5.15
Unitgraph Type: STANDARD

Baseflow: None
End:

Reach: P.1B1
Last Modified Date: 5 July 2016
Last Modified Time: 14:12:06
Canvas X: -16128.009657053855
Canvas Y: 10567.154133660419
From Canvas X: -9744.809909356078
From Canvas Y: 11051.39687314094
Label X: -12.0
Label Y: 12.0
Downstream: P.1BA

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

Subbasin: 1BA
Last Modified Date: 1 July 2016
Last Modified Time: 16:16:48
Canvas X: -14102.994564680768
Canvas Y: 4712.219192668666
Area: 0.00346
Downstream: C.1B

Canopy: None
Plant Uptake Method: None

Surface: None

LossRate: SCS
Percent Impervious Area: 0.0
Curve Number: 80

Transform: SCS
Lag: 3.60
Unitgraph Type: STANDARD

Baseflow: None
End:

Subbasin: 1BB
Last Modified Date: 1 July 2016
Last Modified Time: 16:16:48
Canvas X: -17580.73787549542
Canvas Y: 6164.947411110228
Area: 0.00068
Downstream: C.1B

Canopy: None
Plant Uptake Method: None

Surface: None

LossRate: SCS
Percent Impervious Area: 0.0
Curve Number: 80

Transform: SCS
Lag: 3.60
Unitgraph Type: STANDARD

Baseflow: None
End:

Reach: C.1B
Last Modified Date: 15 June 2016
Last Modified Time: 15:02:49
Canvas X: -16128.009657053855
Canvas Y: 10567.154133660419
From Canvas X: -16128.009657053859
From Canvas Y: 8278.006637934319
Downstream: P.1BA

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

Reach: P.1BA
Last Modified Date: 5 July 2016
Last Modified Time: 14:12:03
Canvas X: -19693.79710231951
Canvas Y: 10347.04379753291

From Canvas X: -16128.009657053855
From Canvas Y: 10567.154133660419
Label X: -13.0
Label Y: 14.0
Downstream: C.1A

Route: Kinematic Wave
Channel: Kinematic Wave
Length: 105
Energy Slope: 0.005
Mannings n: 0.027
Shape: Trapezoid
Number of Subreaches: 2
Width: 3
Side Slope: 3
Channel Loss: None
End:

Reach: C.1A
Last Modified Date: 29 July 2016
Last Modified Time: 13:27:37
Canvas X: -22482.613380442213
Canvas Y: 10030.299285254518
From Canvas X: -19693.79710231951
From Canvas Y: 10347.04379753291
Label X: -3.0
Label Y: 18.0
Downstream: O.1A

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

Subbasin: 1A1
Last Modified Date: 1 July 2016
Last Modified Time: 16:16:48
Canvas X: -17184.539270465903
Canvas Y: 2202.9613608150576
Area: 0.00769
Downstream: P.1A1

Canopy: None
Plant Uptake Method: None

Surface: None

LossRate: SCS
Percent Impervious Area: 0.0
Curve Number: 80

Transform: SCS
Lag: 5.92
Unitgraph Type: STANDARD

Baseflow: None
End:

Reach: P.1A1
Last Modified Date: 22 June 2016
Last Modified Time: 16:17:54
Canvas X: -22482.613380442213
Canvas Y: 10030.299285254518
From Canvas X: -21014.459119084568
From Canvas Y: 1850.7848230110412
Downstream: O.1A

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

Junction: O.1A
Last Modified Date: 30 July 2016
Last Modified Time: 21:57:39
Canvas X: -22482.613380442213
Canvas Y: 10030.299285254518
Label X: -64.0
Label Y: 12.0
Downstream: CoolPond
End:

Subbasin: 2C1
Last Modified Date: 1 July 2016
Last Modified Time: 16:16:48
Canvas X: 27481.82337413488
Canvas Y: -577.4551005548856
Area: 0.02730
Downstream: P.2C1

Canopy: None
Plant Uptake Method: None

Surface: None

LossRate: SCS
Percent Impervious Area: 0.0
Curve Number: 80

Transform: SCS
Lag: 5.00
Unitgraph Type: STANDARD

Baseflow: None
End:

Reach: P.2C1
Last Modified Date: 5 July 2016
Last Modified Time: 14:12:56

Canvas X: 41571.21717786729
Canvas Y: -7368.745639044617
From Canvas X: 27887.27355553725
From Canvas Y: -7571.470729745801
Label X: -6.0
Label Y: -14.0
Downstream: C.2C

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

Reach: C.2C
Last Modified Date: 29 July 2016
Last Modified Time: 13:27:37
Canvas X: 46137.8514921873
Canvas Y: -5041.691204269195
From Canvas X: 41571.21717786729
From Canvas Y: -7368.745639044617
Downstream: O.2C

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

Junction: O.2C
Last Modified Date: 30 July 2016
Last Modified Time: 21:57:58
Canvas X: 46137.8514921873
Canvas Y: -5041.691204269195
Downstream: CoolPond
End:

Subbasin: 2A1
Last Modified Date: 1 July 2016
Last Modified Time: 16:18:23
Canvas X: 38631.70336270011
Canvas Y: 4490.672166974764
Area: 0.01803
Downstream: P.2A1

Canopy: None
Plant Uptake Method: None

Surface: None

LossRate: SCS
Percent Impervious Area: 0.0
Curve Number: 80

Transform: SCS
Lag: 6.03
Unitgraph Type: STANDARD

Baseflow: None
End:

Subbasin: 2A2A
Last Modified Date: 1 July 2016
Last Modified Time: 16:16:48
Canvas X: 29306.349190445544
Canvas Y: 5301.572529779507
Area: 0.00336
Downstream: P.2A2

Canopy: None
Plant Uptake Method: None

Surface: None

LossRate: SCS
Percent Impervious Area: 0.0
Curve Number: 80

Transform: SCS
Lag: 3.60
Unitgraph Type: STANDARD

Baseflow: None
End:

Subbasin: 2A2B
Last Modified Date: 1 July 2016
Last Modified Time: 16:16:48
Canvas X: 26772.285556680727
Canvas Y: 8139.723799596108
Area: 0.00266
Downstream: P.2A2

Canopy: None
Plant Uptake Method: None

Surface: None

LossRate: SCS
Percent Impervious Area: 0.0
Curve Number: 80

Transform: SCS
Lag: 3.60
Unitgraph Type: STANDARD

Baseflow: None
End:

Reach: P.2A2
Last Modified Date: 15 June 2016

Last Modified Time: 15:06:19
Canvas X: 37212.6277277918
Canvas Y: 11281.962705464492
From Canvas X: 25758.77291482666
From Canvas Y: 11546.252075256181
Downstream: P.2A1

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

Reach: P.2A1
Last Modified Date: 3 August 2016
Last Modified Time: 17:52:28
Canvas X: 45467.788634463366
Canvas Y: 9198.417880803721
From Canvas X: 37212.6277277918
From Canvas Y: 11281.962705464492
Downstream: O.2A

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

Junction: O.2A
Last Modified Date: 3 August 2016
Last Modified Time: 17:52:28
Canvas X: 45467.788634463366
Canvas Y: 9198.417880803721
Downstream: CoolPond
End:

Subbasin: 1C1
Last Modified Date: 30 July 2016
Last Modified Time: 20:36:01
Canvas X: -12166.023606758685
Canvas Y: 133.92420121646865
Area: 0.01144
Downstream: P.1C1

Canopy: None
Plant Uptake Method: None

Surface: None

LossRate: SCS
Percent Impervious Area: 0.0
Curve Number: 80

Transform: SCS
Lag: 7.69
Unitgraph Type: STANDARD

Baseflow: None
End:

Subbasin: 1C2
Last Modified Date: 30 July 2016
Last Modified Time: 20:36:35
Canvas X: -6294.508075225764
Canvas Y: -97.64352108602907
Area: 0.00533
Downstream: P.1C2

Canopy: None
Plant Uptake Method: None

Surface: None

LossRate: SCS
Percent Impervious Area: 0.0
Curve Number: 80

Transform: SCS
Lag: 5.02
Unitgraph Type: STANDARD

Baseflow: None
End:

Reach: P.1C2
Last Modified Date: 29 July 2016
Last Modified Time: 13:20:42
Canvas X: -14675.281438612294
Canvas Y: -2375.333630637142
From Canvas X: -6684.710789452354
From Canvas Y: -2980.3543311221183
Label X: 2.0
Label Y: 14.0
Downstream: P.1C1

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

Reach: P.1C1
Last Modified Date: 15 June 2016
Last Modified Time: 15:04:00

Canvas X: -21960.93356443286
Canvas Y: -1384.8371180633476
From Canvas X: -14675.281438612294
From Canvas Y: -2375.333630637142
Downstream: C.1C

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

Reach: C.1C
Last Modified Date: 29 July 2016
Last Modified Time: 13:27:37
Canvas X: -25834.875480277027
Canvas Y: 376.04557095673226
From Canvas X: -21960.93356443286
From Canvas Y: -1384.8371180633476
Downstream: O.1C

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

Junction: O.1C
Last Modified Date: 30 July 2016
Last Modified Time: 21:57:32
Canvas X: -25834.875480277027
Canvas Y: 376.04557095673226
Downstream: CoolPond
End:

Subbasin: 2E1
Last Modified Date: 3 August 2016
Last Modified Time: 17:52:22
Canvas X: 40625.996794706356
Canvas Y: 1829.2509956677422
Area: 0.01264
Downstream: O.2E

Canopy: None
Plant Uptake Method: None

Surface: None

LossRate: SCS
Percent Impervious Area: 0.0
Curve Number: 80

Transform: SCS
Lag: 4.21
Unitgraph Type: STANDARD

Baseflow: None
End:

Junction: O.2E
Last Modified Date: 3 August 2016
Last Modified Time: 17:52:20
Canvas X: 46713.76974784296
Canvas Y: 1979.5663772266707
Downstream: CoolPond
End:

Subbasin: 2BA
Last Modified Date: 1 July 2016
Last Modified Time: 16:16:48
Canvas X: 20589.395913598288
Canvas Y: 9600.133674793733
Area: 0.00378
Downstream: P.2BA

Canopy: None
Plant Uptake Method: None

Surface: None

LossRate: SCS
Percent Impervious Area: 0.0
Curve Number: 80

Transform: SCS
Lag: 3.60
Unitgraph Type: STANDARD

Baseflow: None
End:

Reach: P.2BA
Last Modified Date: 29 July 2016
Last Modified Time: 13:24:10
Canvas X: 15538.680105655912
Canvas Y: 12520.77702451859
From Canvas X: 20225.430884358313
From Canvas Y: 11302.987275198375
Downstream: C.2BA

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

Reach: C.2BA
Last Modified Date: 29 July 2016
Last Modified Time: 13:25:58
Canvas X: 13413.084311893013
Canvas Y: 11607.068275270633
From Canvas X: 15538.680105655912
From Canvas Y: 12520.77702451859
Downstream: P.2BB

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

Subbasin: 2BB
Last Modified Date: 1 July 2016
Last Modified Time: 16:16:48
Canvas X: 16575.52671264449
Canvas Y: 8018.912474417995
Area: 0.00291
Downstream: C.2BB

Canopy: None
Plant Uptake Method: None

Surface: None

LossRate: SCS
Percent Impervious Area: 0.0
Curve Number: 80

Transform: SCS
Lag: 3.60
Unitgraph Type: STANDARD

Baseflow: None
End:

Reach: C.2BB
Last Modified Date: 29 July 2016
Last Modified Time: 13:23:07
Canvas X: 13413.084311893013
Canvas Y: 11607.068275270633
From Canvas X: 13717.16531196527
From Canvas Y: 7897.280074389091
Downstream: P.2BB

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

Subbasin: 2BC
Last Modified Date: 1 July 2016
Last Modified Time: 16:16:48
Canvas X: 10250.641911141538
Canvas Y: 8262.1772744758
Area: 0.00255
Downstream: P.2BB

Canopy: None
Plant Uptake Method: None

Surface: None

LossRate: SCS
Percent Impervious Area: 0.0
Curve Number: 80

Transform: SCS
Lag: 6.09
Unitgraph Type: STANDARD

Baseflow: None
End:

Reach: P.2BB
Last Modified Date: 29 July 2016
Last Modified Time: 13:24:06
Canvas X: 6358.405110216641
Canvas Y: 12154.414075400697
From Canvas X: 13413.084311893013
From Canvas Y: 11607.068275270633
Downstream: C.2BC

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

Reach: C.2BC
Last Modified Date: 29 July 2016
Last Modified Time: 13:27:37
Canvas X: 6115.140310158829
Canvas Y: 9052.78787466367
From Canvas X: 6358.405110216641
From Canvas Y: 12154.414075400697
Label X: -59.0
Label Y: 2.0
Downstream: O.2B

Route: Kinematic Wave

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

Junction: O.2B
Last Modified Date: 30 July 2016
Last Modified Time: 21:57:53
Canvas X: 6115.140310158829
Canvas Y: 9052.78787466367
Downstream: CoolPond
End:

Subbasin: 2DA
Last Modified Date: 1 July 2016
Last Modified Time: 16:16:48
Canvas X: 17198.998598498627
Canvas Y: -1757.9620139287435
Area: 0.00602
Downstream: P.2DA

Canopy: None
Plant Uptake Method: None

Surface: None

LossRate: SCS
Percent Impervious Area: 0.0
Curve Number: 80

Transform: SCS
Lag: 3.70
Unitgraph Type: STANDARD

Baseflow: None
End:

Reach: P.2DA
Last Modified Date: 15 June 2016
Last Modified Time: 15:05:07
Canvas X: 17658.926938518853
Canvas Y: -6838.545244216715
From Canvas X: 19772.49226240484
From Canvas Y: -6157.806019961932
Downstream: C.2D

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

Reach: C.2D
Last Modified Date: 5 July 2016
Last Modified Time: 14:13:10
Canvas X: 8897.406134285062
Canvas Y: -6572.885643172613
From Canvas X: 17658.926938518853
From Canvas Y: -6838.545244216715
Label X: -4.0
Label Y: -18.0
Downstream: O.2D

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

Junction: O.2D
Last Modified Date: 30 July 2016
Last Modified Time: 21:57:42
Canvas X: 8897.406134285062
Canvas Y: -6572.885643172613
Label X: -55.0
Label Y: 14.0
Downstream: CoolPond
End:

Reservoir: CoolPond
Last Modified Date: 3 August 2016
Last Modified Time: 17:51:46
Canvas X: 2671.3629510769133
Canvas Y: -12545.001881376855
Downstream: SiteOutfall

Route: Controlled Outflow
Routing Curve: Elevation-Storage
Initial Elevation: 19.13
Elevation-Storage Table: ES_Cool_Pond
Adaptive Control: On
Main Tailwater Condition: None
Auxiliary Tailwater Condition: None

Spillway: Broad-Crested Spillway
Spillway Outlet: Main
Spillway Crest Length: 63.29
Spillway Crest Elevation: 20.93
Spillway Coefficient: 3
End Spillway:

Evaporation Method: Zero Evaporation
End Evaporation:
End:

Junction: SiteOutfall

Last Modified Date: 3 August 2016
Last Modified Time: 17:52:11
Canvas X: 12742.493515525119
Canvas Y: -16678.674874247386
End:

Basin Schematic Properties:
Last View N: 13250.763321987739
Last View S: -17408.661171716536
Last View W: -27692.344123826642
Last View E: 50321.33890525724
Maximum View N: 13250.763321987739
Maximum View S: -17408.661171716536
Maximum View W: -27692.344123826642
Maximum View E: 50321.33890525724
Extent Method: Elements
Buffer: 5
Draw Icons: Yes
Draw Icon Labels: Name
Draw Map Objects: No
Draw Gridlines: No
Draw Flow Direction: No
Fix Element Locations: No
Fix Hydrologic Order: No
End: