October 7, 2016

American Electric Power
1 Riverside Plaza, 22nd Floor
Columbus, Ohio 43215

Attention: Mr. Brian G. Palmer

Reference: **Ash Pond Complex CCR Surface Impoundment**
Inflow Design Flood Assessment
Conesville Power Plant
Conesville, Ohio
S&ME Project No. 7217-14-009H
Ohio PE Firm License No. 03530

Dear Mr. Palmer:

In accordance with our proposal dated June 21, 2016 and your request for additional work, S&ME, Inc. (S&ME) has completed an Inflow Design Flood Assessment of the ash pond complex at the Conesville Power Plant in Conesville, Ohio. The Assessment was performed to fulfill the requirements of 40 CFR § 257.82(c)(1), *Hydrologic and hydraulic capacity requirements for CCR surface impoundments-Inflow design flood control system plan,* of the USEPA Rules for Disposal of Coal Combustion Residuals from Electric Facilities.

❖ Background

The Conesville Generating Plant is located in Coshocton County, Ohio, east of the Muskingum River, across from the town of Conesville. Adjacent to the plant is an approximate 100 acre ash pond complex within which are a number of individual ponds or lagoons: Three fly ash ponds denoted as Fly Ash Pond A, B and C, a bottom ash pond, and a Clearwater pond. The ponds are isolated from exterior surface water inflow. The ponds are regularly dredged and the recovered fly ash and bottom ash is either beneficially re-used or taken to the nearby Abandoned Mine Lands reclamation projects or the nearby residual waste landfill. The separator dike between each lagoon is constructed of fly ash and bottom ash. The separator dikes convey flow either through a pipe or through outlet structures. Because the ponds are interconnected with pipes and outlet structures which equalize the water surface to a common normal pool, the entire pond complex was treated as one hydraulic unit. For the purposes of this study, the interior dikes and outlet devices within the pond complex were not included (or hydraulically modelled) and a single maximum permitted water surface elevation was assumed across all ponds. This assumption is consistent with the Ohio Dam Safety Program dam inventory record for the Conesville Ash Pond Complex.

❖ Previous Analyses

In 2010, BBC&M Engineering, Inc. performed a hydrologic and hydraulic (H&H) analyses of the Ash Pond Complex which was used as a basis for the H&H study included in Attachment A.

❖ Inflow Design Flood 257.82(a)(3)

The Ash Pond Complex is regulated by the Ohio Department of Natural Resources (ODNR) as a Class II dam. An H&H study was previously performed in 2010 and a few inputs made in the previous analysis have changed requiring updated analyses as described in Attachment A.

The objective of this study is to demonstrate that the Ash Pond Complex can safely pass the 50% probable maximum flood (PMF) as required by ODNR. Per rule 257.82(A)(3)(ii), the Ash Pond is classified as a significant hazard potential CCR surface impoundment which requires that the impoundment safely pass the 1000-year flood. NOAA Point precipitation data for a location near the site is included in Attachment B. A comparison of the 50% Probable Maximum Precipitation (PMP) rainfall depths and the corresponding 1000 year depth estimates from NOAA Point precipitation data is included in Table 1 below:

Storm Duration	ODNR 50% PMP Estimate	NOAA 1,000-year Depth Estimate
6-HR	9.375 Inches	6.98 Inches
24-HR	13.25 Inches	9.30 Inches

As the 50% PMP represents a more severe event that the 1,000 year precipitation, the inflow design flood used for this study was based on the 50% PMP. The 50% Probable Maximum Precipitation (PMP) was used to estimate the 50% PMF based on revised rainfall depth estimates for the state of Ohio from a statewide PMP study released by ODNR in 2013. The H&H study included as Attachment A demonstrates that the facility has the capacity to manage the inflow design flood, as defined above, in a manner which meets the requirements of 257.82(A)(3)(ii).

❖ Flood Control Plan 257.82(c)

All stormwater runoff from the watershed and all plant inflows are currently discharged through the Clearwater Pond outlet structure to a holding pond that feeds plant operations and ultimately discharges to the Muskingum River. The Ash Pond Complex is designed to safely pass the 50% PMF without overtopping the ash pond embankment, El. 770, via the Clearwater Pond outlet structure with normal pool set to the maximum permitted operating pool elevation, El. 762. The H&H study included as Attachment A suggests that the facility has the capacity to manage the inflow design flood in a manner which meets the requirements of 257.82(A)(3)(ii).

Closing

S&ME certifies that this assessment meets the requirements of paragraphs (a)(3) and (c) of Part 257.82 for the hydrologic and hydraulic capacity requirements for CCR surface impoundments.

Sincerely,

S&ME, Inc.

Matthew R. Marquis, P.E.
Project Engineer
Registration No. 80696

Michael G. Rowland, P.E.
Senior Engineer
Registration No. 65559

Attachments

- Attachment A – Hydrologic and Hydraulic Study
- Attachment B – NOAA Point Precipitation Data

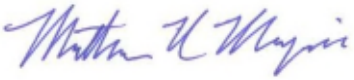
Attachment A – Hydrologic and Hydraulic Study

COVER SHEET

S&ME

Project Name: Conesville Ash Pond Complex H&H	
Subject: Hydrologic and Hydraulic Study	
Project No. 7217-14-009H	Calc. No.
Discipline Water Resources	Sheet 1 of 6

Ash Pond Complex
Hydrologic and Hydraulic Study

Computations By:	Signature: 	Date 10-7-2016
	Name: Matthew R. Marquis, P.E.	
	Title: Project Engineer	
Reviewed By:	Signature:	Date 10-7-2016
	Name: Stephen J. Loskota, P.E.	
	Title: Senior Project Engineer	

0	First Issue	MRM	SJL	10/7/2016
No.	Description	By	Review	Date

Table of Contents

Objective 3

Project Background 3

Hydrologic Study 3

Hydrology 3

Storage 4

Rainfall 4

Hydraulic Study 4

Ash Pond Outlet 4

Plant Inflows 5

Results / Discussion 6

References 6

Appendices 6

Appendix A 6

List of Tables

Table 1 – Ash Pond Complex Routing Results 6

Objective

The ash pond complex at the Conesville Generating Plant is a Class II dam according to the Ohio Department of Natural Resources (ODNR) Division of Soil and Water. A Hydrologic and Hydraulic (H&H) study was performed in 2010 and a few assumptions made in the previous analysis have changed requiring updated analyses as described below:

1. The previous analysis identified low spots along the exterior dike which have been repaired. The top of dike used in this study is elevation 770.

2. The PMF was updated to reflect current ODNR PMP Guidance.

3. The outlet pipe was originally assumed to be smaller than the actual 36-inch pipe as discovered by a recent pipe inspection. The pipe size was updated in the outlet structure rating curve used in this study.

The objective of this study is to demonstrate that the Ash Pond Complex can safely pass the 50% probable maximum precipitation (PMP) as required by ODNR.

Project Background

The Conesville Generating Plant is located in Coshocton County, Ohio, east of the Muskingum River, across from the town of Conesville. Adjacent to the plant is an approximate 100 acre ash pond complex within which are a number of individual ponds or lagoons: Three fly ash ponds denoted as Fly Ash Pond A, B and C, a bottom ash pond, and a Clearwater pond. The ponds are isolated from exterior surface water inflow. The ponds are regularly dredged and the recovered fly ash and bottom ash is either being beneficially re-used or taken to the nearby Abandoned Mine Lands reclamation projects or the nearby residual waste landfill. The separator dike between each lagoon is constructed of fly ash and bottom ash. The separator dikes convey flow either through a pipe or through outlet structures. Because the ponds are interconnected with pipes and outlet structures which equalize the water surface to a common normal pool, the entire pond complex was treated as one hydraulic unit. For the purposes of this study, the interior dikes and outlet devices within the pond complex were not included (or hydraulically modelled) and a single maximum permitted water surface elevation was assumed across all ponds. This assumption is consistent with the Ohio Dam Safety Program dam inventory record for the Conesville Ash Pond Complex.

Hydrologic Study

S&ME used the SCS method to estimate the peak discharge during the design storm event. A Hydrologic and hydraulic analysis was conducted using the HydroCAD v.10 computer program to model the Ash Pond Complex drainage area using the TR-20 methodology.

Hydrology

The ponds are completely isolated from exterior surface water inflow. The previous analysis performed in 2010 (BBCM, 2010) assumed a drainage area of approximately 128.64 Acres. The Curve number used in

this study was based on the value used in the previous hydrologic analysis performed in 2010 (BBCM, 2010). The Curve number was assumed to be 93 since the majority of the drainage area is open water. The time of concentration of 30 minutes was obtained from the previous analysis (BBCM,2010).

Storage

The storage within the Ash Pond Complex was obtained from the previous analysis (BBCM, 2010). Storage and surface area data is included in Appendix A, A.2.

Rainfall

The Ash Pond Complex has been identified as a Class II dam by ODNR, and as such must safely pass 50% of the Probable Maximum Flood (PMF) in accordance with OAC Rule 1501:21-13-02. The Probable Maximum Precipitation (PMP) was used to estimate the PMF based on revised rainfall depth estimates for the state of Ohio from a statewide PMP study released by ODNR in 2013. Historic PMP values were conservatively high to account for a higher level of uncertainty associated with the predicted values. When the results of the latest ODNR study were found to reduce the precipitation values, new guidelines were released on the appropriate use of the values in the hydrologic and hydraulic analysis of dams. Specifically, the new guidelines require the evaluation of two separate rainfall distributions: a 24-hour SCS Type II distribution commonly used in the Midwest but modified slightly by ONDR; and a 6-hour PMP distribution developed by ODNR using techniques from Hydrometeorlogical Report No. 52. The distributions are presented in Appendix A, A.3-A.6. The time step (which influences peak duration and intensity) used in each distribution has been defined by ODNR. The more conservative event (the one resulting in the higher peak water surface elevation) is used as the design event. Maps provided in Appendix A, A.7-A.8, were used to estimate the total rainfall volumes applied to each of the two specified rainfall distributions.

Hydraulic Study

The Ash Pond Complex outlet device was modeled using HydroCAD and hydraulic components used in the study are described in more detail in the following sections. The HydroCAD reports for each rainfall distribution scenario are included in Appendix A, A.15-A.28.

Ash Pond Outlet

The Ash Pond outlet device is a concrete structure with stop logs that control the normal pool and an outlet pipe at the base of the structure to discharge flow from the Ash Pond. The stop logs are 4 feet long and the logs are placed to set the normal pool to elevation 762 which corresponds to the maximum permitted operating pool. The 36-inch concrete outlet pipe inside inlet elevation sits at approximately elevation 750 and the outlet invert sits at elevation 730. The outlet pipe is approximately 300 feet long. Each component of the outlet structure was evaluated separately using accepted engineering equations and a composite rating curve was developed for the entire structure based on the limiting flow at each incremental headwater elevation. The composite rating curve was used in HydroCAD to model the outflow from the pond. A photo of the outlet structure is included below:

Date: 7/7/2015

Photographer: JDR

Location / Orientation	Clear Water Pond Outlet
Remarks	Concrete Stop Log Structure

The stop log structure rating curve was developed using a composite of a sharp-crested weir equation with end contractions to simulate the concrete sidewalls and a broad-crested weir equation as the headwater elevation climbs outside of the applicable range for the sharp-crested equation. The length of the weir is the stop log length, 4 feet, and the height of the weir is the difference between the normal pool elevation and the bottom of the structure, 12 feet. As the water falls over the weir and begins to fill the void space within the structure, the flow transitions to orifice flow into the outlet pipe at the base of the structure. The outlet pipe orifice flow was modeled using the orifice equation with the centroid of the opening set to 1.5-feet above the invert of the pipe, elevation 751.5. As the headwater elevation rises, the flow transitions to pipe pressure flow and the capacity of the pipe controls the outflow. The pipe pressure flow was modeled using a simplified energy equation by solving for velocity and multiplying by the cross sectional area to obtain the estimated flow capacity at incremental headwater elevations. The manning's roughness coefficient for the pipe was assumed to be 0.011 based on the HydroCD manual entry for a clean straight concrete pipe. The final rating curve for the outlet structure was developed by choosing the controlling flow at each incremental headwater elevation from all three primary flow types and the calculations used to develop the curve are included in Appendix A, A.10-14.

Plant Inflows

Plant inflows to the Ash Pond Complex were obtained from a current water balance diagram obtained from AEP. The diagram suggests that average daily flows to the Ash Pond total approximately 27.4 million gallons per day (MGD) which approximately equals 42.4 cfs. The plant inflow was applied as a constant base flow across the entire storm duration for both the 24-hour and 6-hour scenarios. A water balance diagram used to estimate plant flows is included in the Appendix, A.9.

Results / Discussion

The results of the study are included in Table 1 below:

Table 1 – Ash Pond Complex Routing Results

Scenario	HydroCAD WSEL	Top of exterior Dike
ODNR 6-HR Dimensionless Distribution	763.3	770.0
ODNR 24-HR SCS Type-II Modified Distribution	764.1	770.0

The predicted maximum water surface elevation (WSEL) determined using HydroCad is lower than the elevation of the exterior dike which suggests that the Ash Pond Complex can safely pass the 50% PMF in accordance with the requirements of OAC Rule 1501:21-13-02.

References

BBCM Engineering, Inc. (2010). "Addendum to Bottom Ash Pond Investigation." *American Electric Power Service Corp.*, December, 2010.

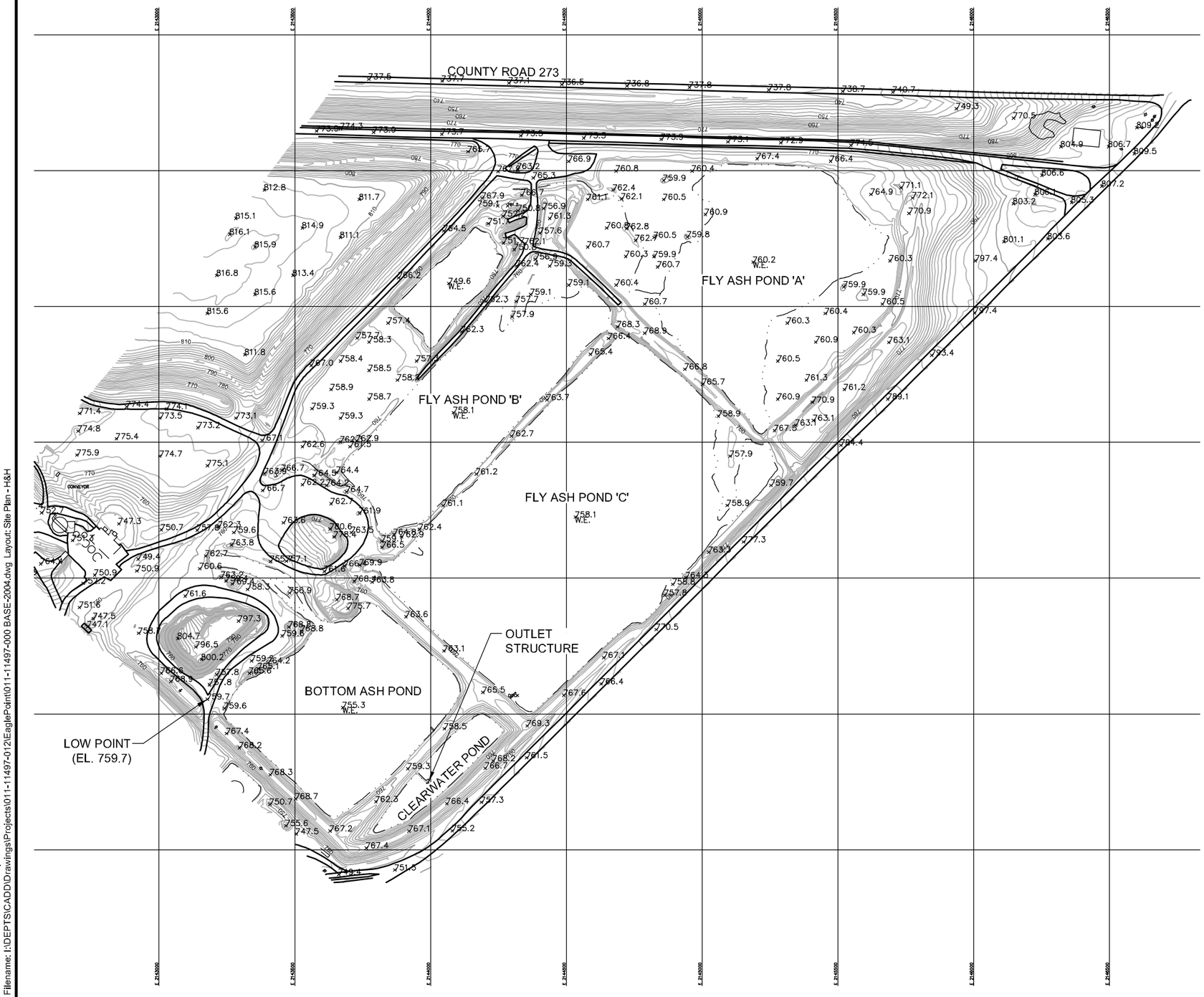
Chow, Ven Te. "Chapter 5 - Development of Uniform Flow and Its Formulas." *Open-channel Hydraulics*. New York: McGraw-Hill, 1959. Print.

HydroCAD User Manual. Vers. 10.00. n.p.: HydroCAD Software Solutions LLC, Program documentation.

Appendices

Appendix A

- Site Plan [A.1]
- Ash Pond Complex Storage [A.2]
- ODNR PMP Application Guidelines [A.3-A.6]
- PMP Charts [A.7-A.8]
- Conesville Water Balance Diagram [A.9]
- Outlet Structure Rating Curve Calculations [A.10-A.14]
- HydroCAD Output Reports [A.15-A.28]



REFERENCE: AEP DRAWING '41798_Conesville_rev1'

Approximate Pool Elevations

Location	Typical Operating Pool Elevations (feet)	Pool Elevations on October 22, 2009 (feet)
Bottom Ash Pond	755 to 760	754.69
Fly Ash Pond	758 to 764	757.78
Clearwater Pond	755	753.39
Fly Ash Storage	760	N/A

NOTE: TABLE TAKEN FROM CHA FINAL REPORT DATED FEBRUARY 15, 2010.

0 400 800

SCALE IN FEET

SITE PLAN

Conesville Station
Ash Pond Investigation
Conesville, Ohio

Project: 011-11497-012	Drawn By: AJS	
Drawing Date: 4-14-2009	Approved By: MGR	
Last Updated: 12-17-2010	Scale: 1" = 400'	1:1

BBC&M SOLUTIONS TO BUILD ON
Columbus (614) 793-2226
Cleveland (216) 901-1000
Cincinnati (513) 771-8471
Dayton (937) 424-1011

PLATE 1

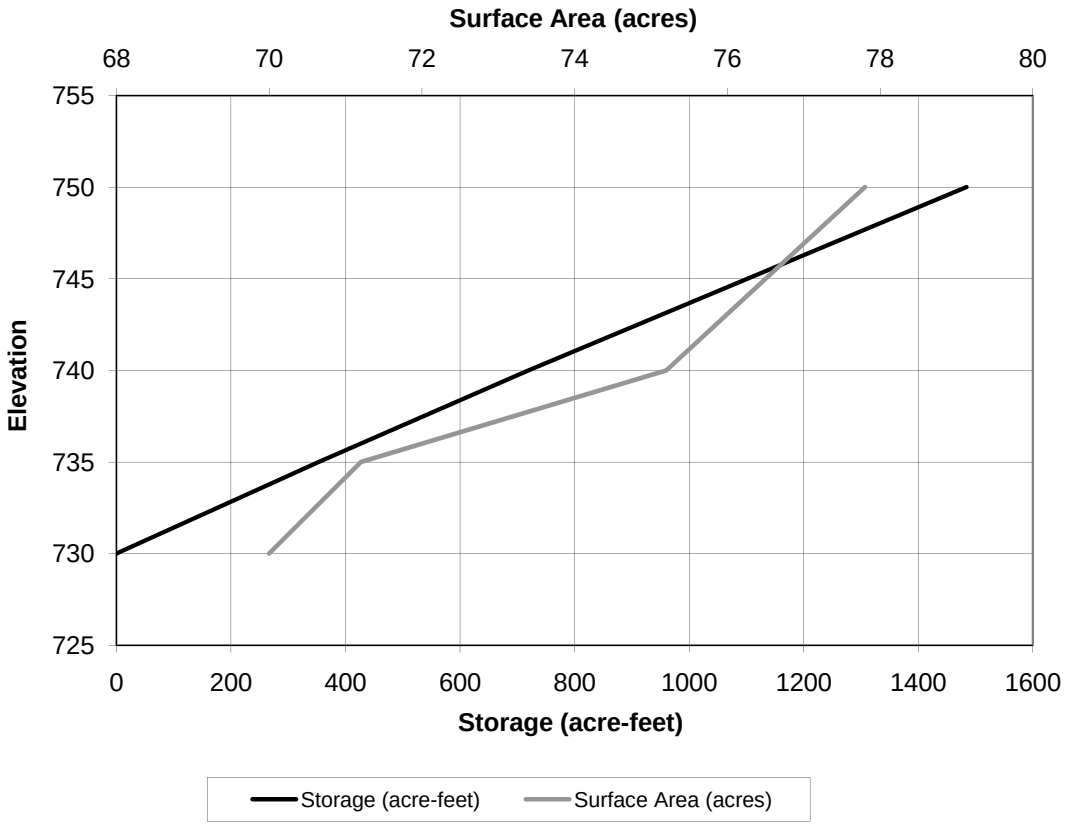
Images: ~BBCM CV Ash Pond Borings.jpg ~Conesville Aerial.jpg ~Scan68856 1.JPG ~Scan68857.JPG ~Section A - 1974.tif
Xrefs: ~41798 Conesville rev1.dwg
File Last Updated: Dec 17, 2010
Plot Info: 12-17-2010 @ 10:18am By: asmith
BBC&M Filename: I:\DEPTS\CADD\Drawings\Projects\011-11497-012\EaglePoint\011-11497-000 BASE-2004.dwg Layout: Site Plan - H&H

Conesville Bottom Ash Ponds - Storage Volume Computations					
Elevation	Area	Avg Area	Distance	Volume	Cum Vol
	acres	acres	feet	ac-ft	ac-ft
730	70.00				0
		70.60	5	353.00	
735	71.20				353.0
		73.20	5	366.00	
740	75.20				719.0
		76.50	10	765.00	
750	77.80				1484.0
		81.50	10	815.00	
760	85.20				2299.0
		85.50	6	513.00	
766	85.80				2812.0
		87.75	4	351.00	
770	89.70				3163.0

Values in area column are estimated to match volume data provided by AEP in previous HEC-1 analyses and are not considered precise.

Bottom Ash Ponds - Surface Area/Storage/Elevation

Note: Surface Area Values are Estimated



Probable Maximum Precipitation Application Guidelines

August 23, 2013

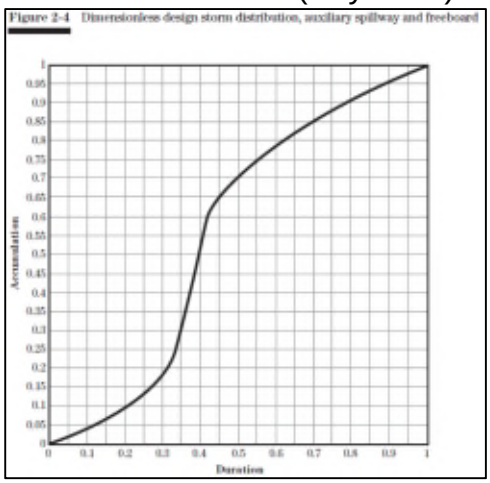
"Probable Maximum Precipitation Study for the State of Ohio" (April 2013, Applied Weather Associates, LLC) is acceptable and appropriate for determining Probable Maximum Precipitation (PMP) values under Ohio Administrative Code Rule 1501:21-13-02. This study shall supersede Hydrometeorological Report No. 51 (HMR51) by the National Weather Service for the purpose of determining design floods for dams within the division's jurisdiction.

"Probable Maximum Flood" or "PMF" means the flood that may be expected from the most severe combination of critical meteorologic and hydrologic conditions that are reasonably possible in the drainage basin under study. The "PMF" is derived from the PMP and is determined by using a hydrologic model to simulate the drainage basin's response to those critical conditions which produce the most severe flood runoff. **Because each basin is different, it is necessary to simulate multiple flood events using different PMP durations to determine the most severe flood. The most severe flood would then be called the PMF.** The table below provides guidance for application of PMP values for determining the PMF for a given drainage basin.

Drainage Basin Size	Storm Duration	Rainfall Depth and Spatial Extent	Temporal Distribution
<1 mi^2	6 hours	Obtain the rainfall depth from the 1 mi^2 PMP chart. The spatial extent of the storm is 1 mi^2.	Dimensionless Design Storm Distribution[A]
	24 hours	Obtain the rainfall depth from the 1 mi^2 PMP chart. The spatial extent of the storm is 1 mi^2.	SCS Type II at 1.5-hour time step[B]
1 mi^2 - 10 mi^2	6 hours	Determine the rainfall depth by interpolation between 6-hour, 1 mi^2 and 6-hour, 10 mi^2 PMP rainfall depths based on the drainage basin size.[C] The spatial extent of the storm is equal to the drainage basin size.	Dimensionless Design Storm Distribution[A]
	24 hours	Determine the rainfall depth by interpolation between 24-hour, 1 mi^2 and 24-hour, 10 mi^2 PMP rainfall depths based on the drainage basin size.[C] The spatial extent of the storm is equal to the drainage basin size.	SCS Type II at 1.5-hour time step[B]
>10 mi^2	24 hours	Use HMR52 (computer program) to generate a 72-hour basin-specific storm (the time step should be set to 1 hour); extract the most severe 24 hours from within the 72-hour storm[D].	Use HMR52 (computer program) to generate a 72-hour basin-specific storm (the time step should be set to 1 hour); extract the most severe 24 hours from within the 72-hour storm[D]
	72 hours	Use HMR52 (computer program) to generate a 72-hour basin-specific storm (the time step should be set to 1.5 hours)[D].	Use HMR52 (computer program) to generate a 72-hour basin-specific storm (the time step should be set to 1.5 hours)[D]

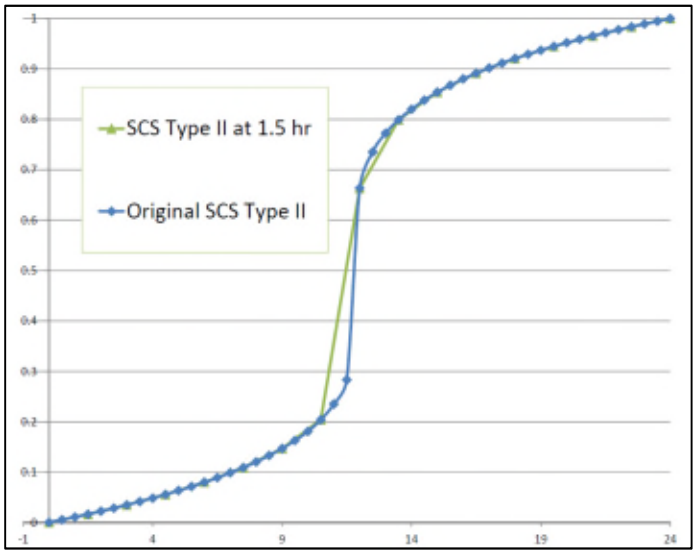
[A] The Dimensionless Design Storm Distribution is from NRCS, Technical Release 60 (July 2005).

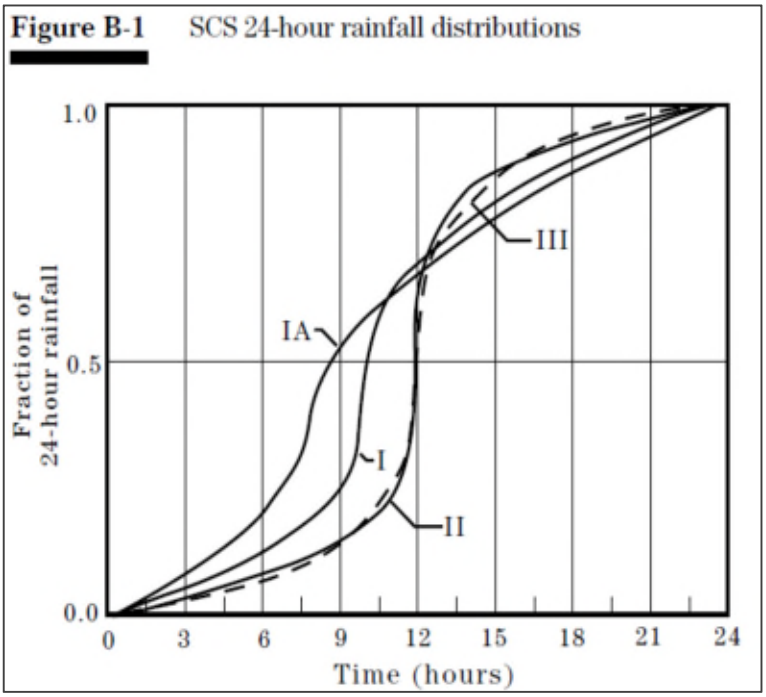
Hr.	Cum.	Incr	Hr.	Cum.	Incr	Hr.	Cum.	Incr
0	0.000	0	2.2	0.379	0.150	4.2	0.856	0.020
0.2	0.013	0.013	2.4	0.530	0.150	4.4	0.875	0.019
0.4	0.026	0.013	2.6	0.625	0.095	4.6	0.893	0.018
0.6	0.041	0.015	2.8	0.670	0.045	4.8	0.910	0.017
0.8	0.058	0.017	3	0.705	0.035	5	0.927	0.016
1	0.077	0.019	3.2	0.736	0.031	5.2	0.942	0.016
1.2	0.098	0.021	3.4	0.764	0.028	5.4	0.957	0.015
1.4	0.121	0.023	3.6	0.790	0.026	5.6	0.972	0.015
1.6	0.146	0.025	3.8	0.814	0.024	5.8	0.986	0.014
1.8	0.179	0.033	4	0.836	0.022	6	1.000	0.014
2	0.229	0.050						



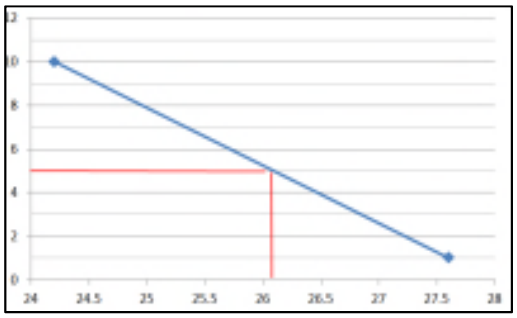
[B] The SCS Type II distribution at a 1.5-hour time step is shown below. The time step has been modified from the original distribution to provide peak intensities that are appropriate for Ohio storms. Please note that computer programs that use the original SCS Type II distribution will likely generate a flood peak that is inappropriately high.

Hr.	Cum.	Incr	Hr.	Cum.	Incr
0	0	0	12	0.663	0.459
1.5	0.016	0.016	13.5	0.799	0.136
3	0.035	0.018	15	0.854	0.055
4.5	0.056	0.021	16.5	0.891	0.038
6	0.080	0.024	18	0.921	0.029
7.5	0.109	0.029	19.5	0.945	0.024
9	0.147	0.038	21	0.965	0.021
10.5	0.204	0.058	22.5	0.984	0.018
12	0.663	0.459	24	1	0.016

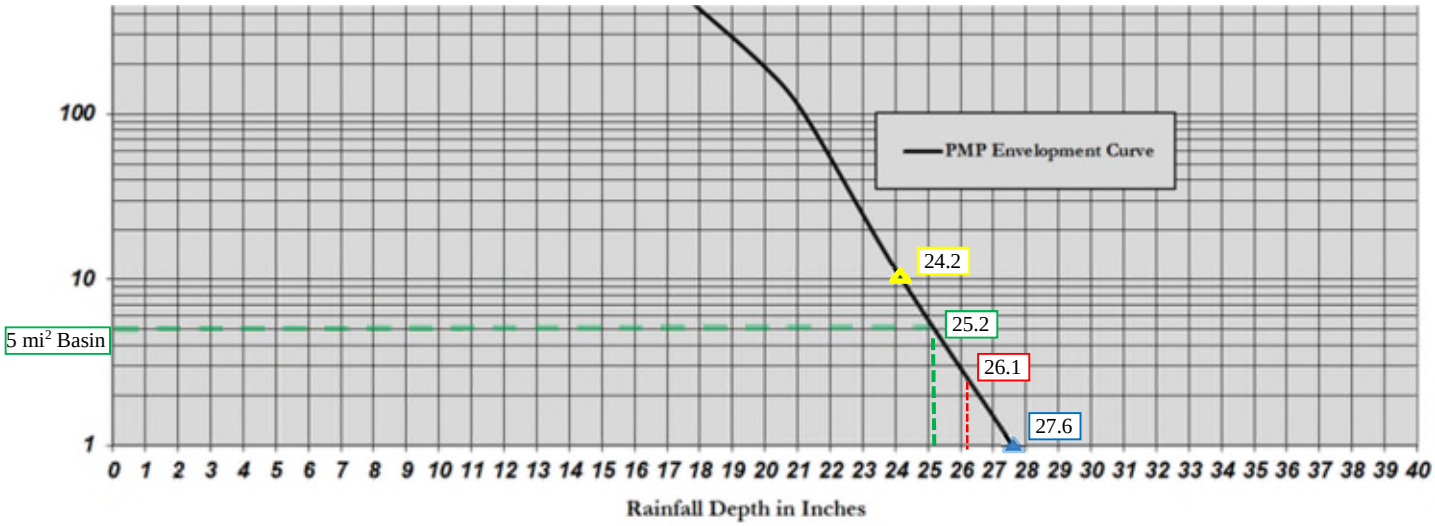




[C] HMR51 does not provide PMP rainfall depths for areas less than 10 mi^2. The 2013 PMP Study provides PMP rainfall depths for areas 1.0 mi^2 and greater. PMP rainfall depths for drainage basins between 1 mi^2 and 10 mi^2 generally fit a straight line when the drainage basin/storm area is taken as log10. For the example shown below, the PMP rainfall depth for a 10 mi^2 basin (storm area) is 24.2 inches, and the PMP rainfall depth for 1 mi^2 basin is 27.6 inches. For a 5 mi^2 basin, a straight line interpolation (no log10) would calculate the PMP to be 26.1 inches, which is incorrect. The correct value for the 5 mi^2 PMP rainfall depth is 25.2 inches.



PMP Rainfall Depth for a 5 mi^2 basin using linear interpolation (incorrect)



PMP Rainfall Depth for a 5 mi^2 basin using interpolation with basin size as log10 (correct)

[D] Hydrometeorological Report No. 52 (HMR52) describes application of PMP values east of the 105th Meridian. The report is available through the National Weather Service: http://www.nws.noaa.gov/oh/hdsc/studies/pmp.html. The US Army Corps of Engineers developed a computer program for applying this report (the computer program is also called HMR52). The computer program is available at http://www.dodson-hydro.com/download.

Comments:
In past analyses, there was confidence that the PMP values from HMR51 and the rainfall distributions were very conservative. The new PMP values and distributions no longer provide an additional safety factor; therefore, it is important to assure that appropriate, justified hydrologic parameters are used. Important items that need to be specifically addressed and documented in a hydrologic study for a dam are listed below:

- More detailed investigation of the time of concentration/lag time
- Justification for unit hydrograph duration
- When assessing infiltration/runoff using curve numbers, specifically address connected impervious, and the “Quality Cover” should be no higher than “fair” unless specifically justified.

Important References:

Natural Resources Conservation Service, National Engineering Handbook
Part 628 Dams, Chapter 50 Earth Spillway Design (1997)
Part 630 Hydrology, Chapter 9 Hydrologic Soil-Cover Complexes (2004), Chapter 15 Time of Concentration (2010), Chapter 16 Hydrographs (2007)

Natural Resources Conservation Service, Conservation Engineering Division
Earth Dams and Reservoirs, TR–60, July 2005

US Army Corps of Engineers, Hydrologic Engineering Center
HEC-1, Flood Hydrograph Package, User’s Manual (1998)

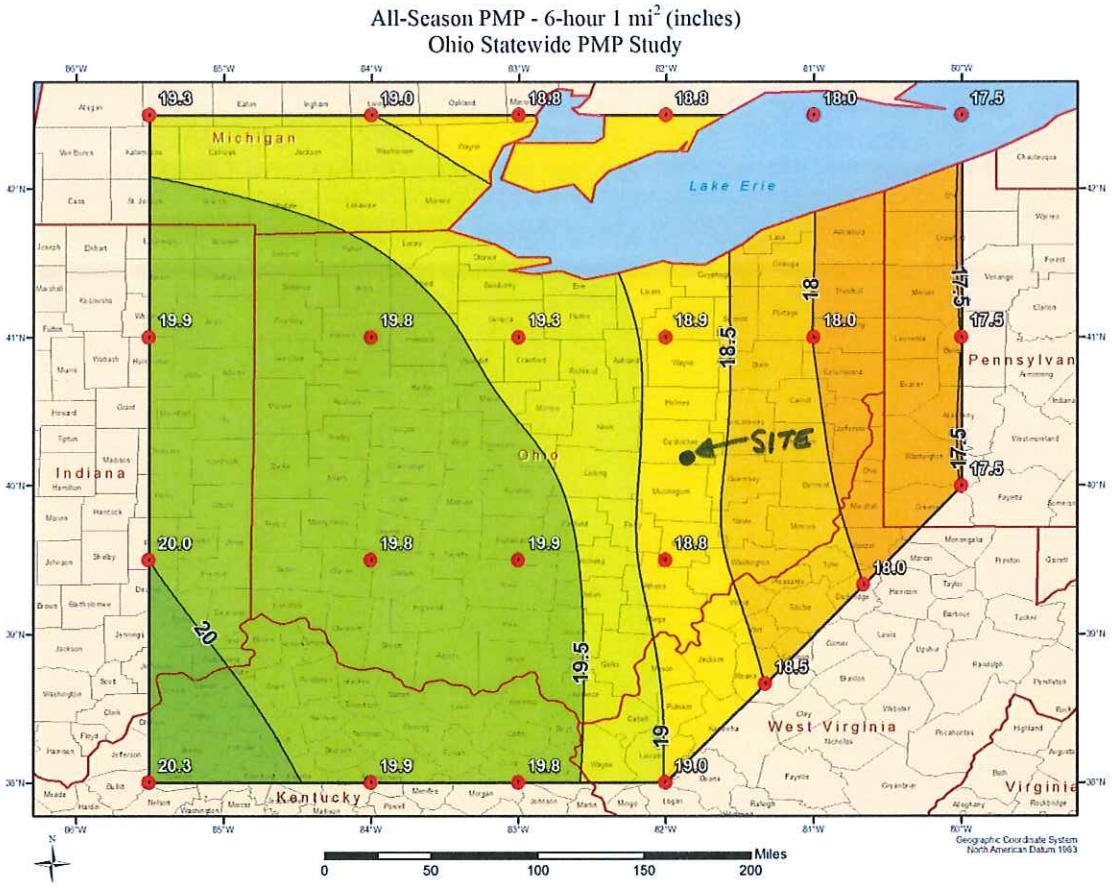


Figure 11.1 All-season PMP (inches) for 6-hour, 1-square mile

SITE ≈ 18.75 IN (6-HR)

50% PMP ≈ 9.375 IN

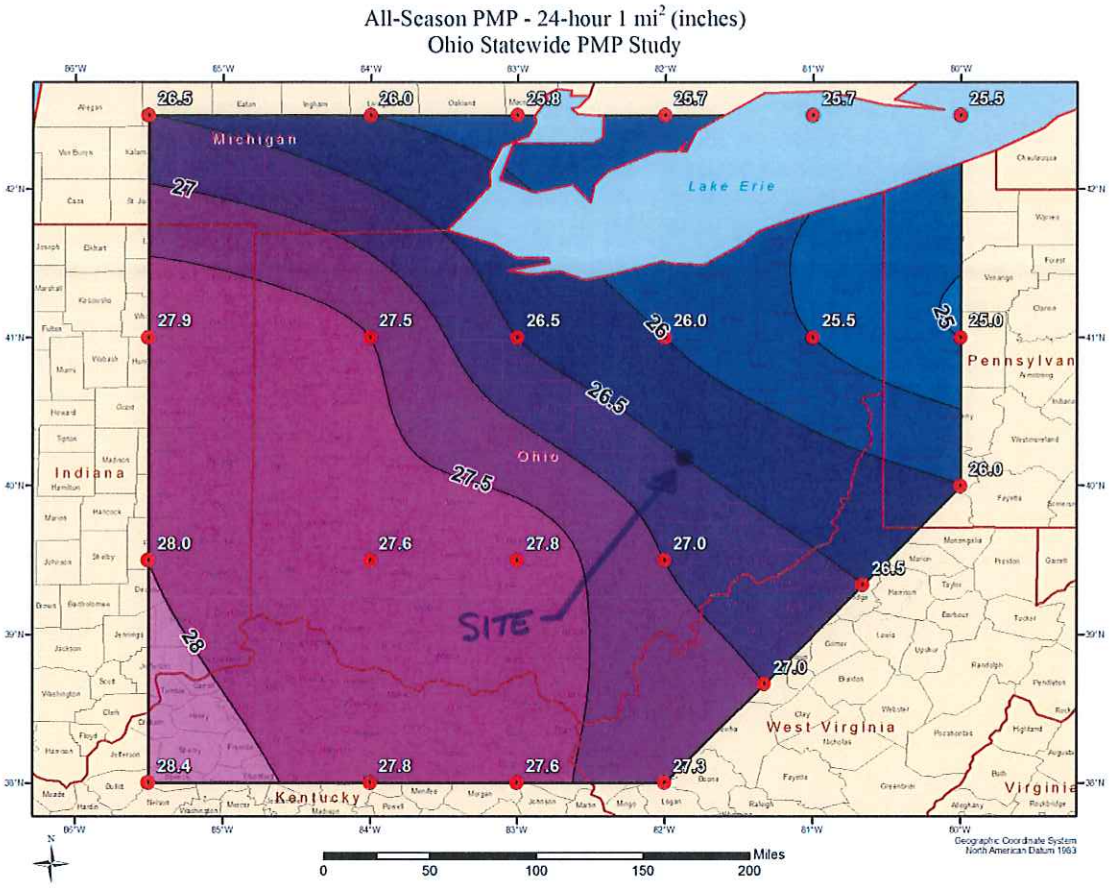


Figure 11.3 All-season PMP (inches) for 24-hour, 1-square mile

SITE ≈ 26.50 IN (24-HR)

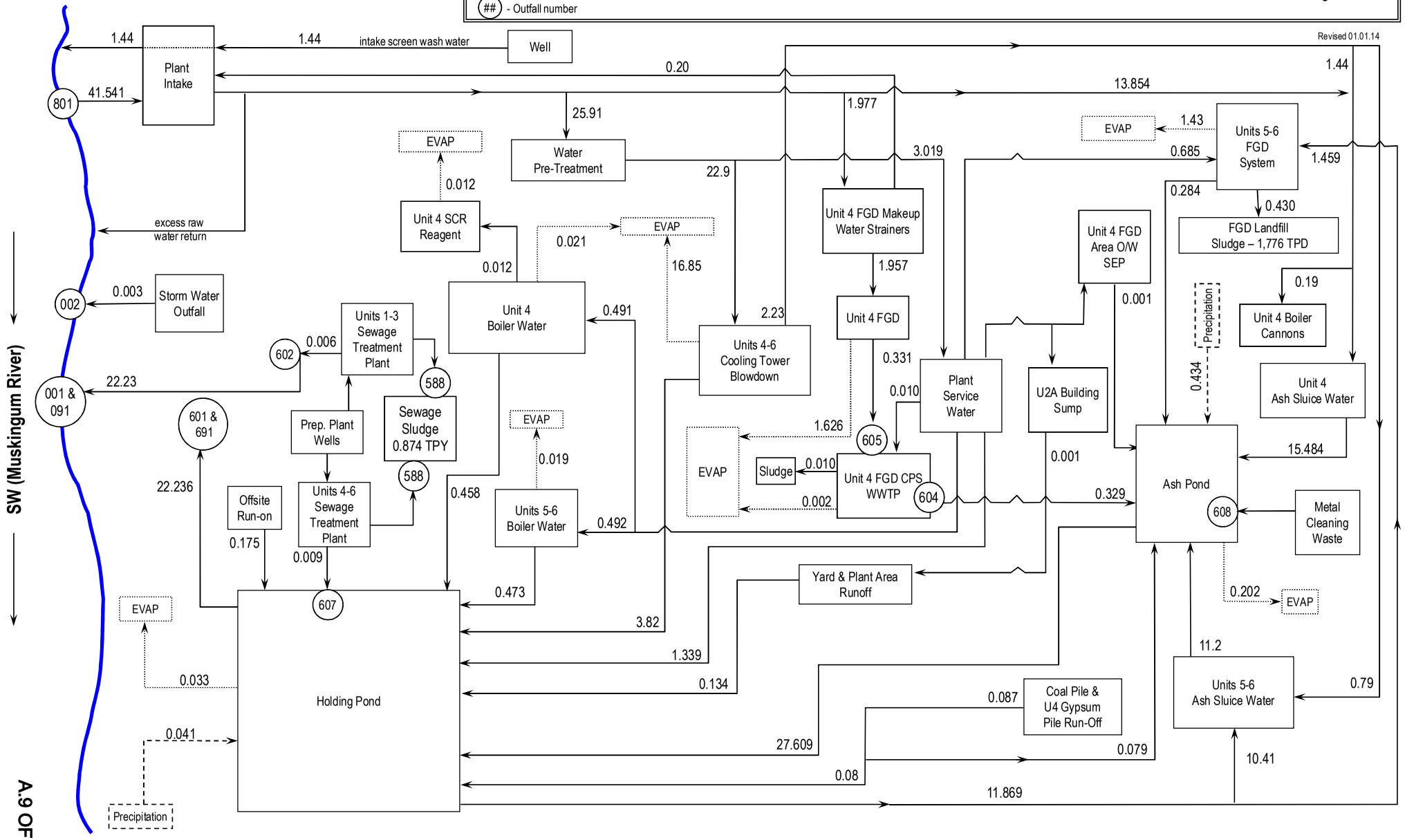
50% PMP ≈ 13.25 IN

Water & Ecological Resource Services

NOTES:
Flows are based on design flows in MGD.
Precipitation is assuming 39"/year.
Units 1/2 circulation water are not included since they have been retired.
(##) - Outfall number

AEP Generation Resources Inc.
Conesville Plant
Water Balance Diagram

Revised 01.01.14



Project: Conesville Ash Pond Complex H&H
Location: Coshocton, Ohio
Project Number: 7217-14-009H

Clearwater Pond - Sharp Crested Weir Flow

Calculated By: MRM **Date:** 10/3/2016

Reviewed By: **Date:**

Source: Hydraulic Engineering Circular No. 22, Third Edition
Urban Drainage Design Manual (Rev. 2013)

Sharp Crested Weirs

Typical sharp crested weirs are illustrated in Figure 8-13. Equation 8-19 provides the discharge relationship for **sharp crested weirs** with no end contractions (illustrated in Figure 8-13a).

$$Q = C_{SCW} L H^{1.5} \quad (8\text{-}19)$$

where:

Q = Discharge, m^3/s (ft^3/s)
L = Horizontal weir length, m (ft)
H = Head above weir crest excluding velocity head, m (ft)
C_{SCW} = 1.81 + 0.22 (H/H_c) [3.27 + 0.4 (H/H_c) in English units]

As indicated above, the value of the coefficient C_{SCW} is known to vary with the ratio H/H_c (see Figure 8-13c for definition of terms). For values of the ratio H/H_c less than 0.3, a constant C_{SCW} of 1.84 (3.33 in English units) is often used.

Equation 8-20 provides the discharge equation for **sharp-crested weirs with end contractions** (illustrated in Figure 8-13(b)). As indicated above, the value of the coefficient C_{SCW} is known to vary with the ratio H/H_c (see Figure 8-13c for definition of terms). For values of the ratio H/H_c less than 0.3, a constant C_{SCW} of 1.84 (3.33 in English units) is often used.

$$Q = C_{SCW} (L - 0.2 H) H^{1.5} \quad (8\text{-}20)$$

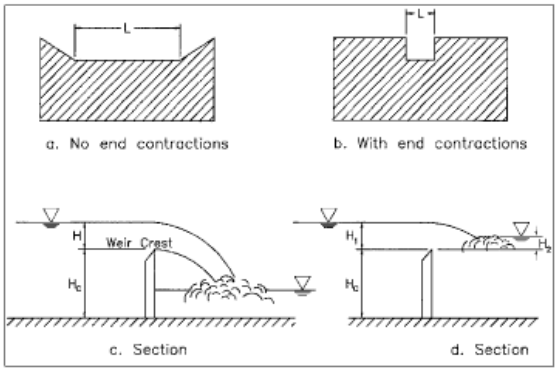


Figure 8-13. Sharp crested weirs.

	Value	Units	Description
Pool Elevation =	762	Feet MSL	Maximum Normal Operating Pool
Length of Weir =	4.0	Feet	From 2010 Analysis
Height of Weir =	12.0	Feet	From 2010 Analysis

Headwater Elevation (FT)	L, Length (feet)	Hc, Feet	H/Hc	Cscw, Coefficient*	Q, Orifice Discharge (CFS) [Use EQ 8-20]
762.00	4.0	12.00	0.00	3.33	*0.0*
763.00	4.0	12.00	0.08	3.33	*12.7*
764.00	4.0	12.00	0.17	3.33	*33.9*
765.00	4.0	12.00	0.25	3.33	*58.8*
766.00	4.0	12.00	0.33	3.40	*87.1*
767.00	4.0	12.00	0.42	3.44	*115.3*
768.00	4.0	12.00	0.50	3.47	*142.8*
769.00	4.0	12.00	0.58	3.50	*168.7*
770.00	4.0	12.00	0.67	3.54	*192.1*
771.00	4.0	12.00	0.75	3.57	*212.1*
772.00	4.0	12.00	0.83	3.60	*227.9*

* Cscw = 3.33 when H/Hc < 0.3

Project: **Conesville Ash Pond Complex H&H**
Location: **Coshocton, Ohio**
Project Number: **7217-14-009H**

Clearwater Pond - Broad Crested Weir Flow

Calculated By: MRM **Date:** 10/3/2016

Reviewed By: **Date:**

Source: Hydraulic Engineering Circular No. 22, Third Edition
Urban Drainage Design Manual (Rev. 2013)

Broad-Crested Weir

The equation typically used for a broad-crested weir is:[49]

$$Q = C_{BCW} L H^{1.5} \quad (8\text{-}22)$$

where:

Q = Discharge, m^3/s (ft^3/s)
C_{BCW} = Broad-crested weir coefficient, 1.35 – 1.83 (2.34 to 3.32)
L = Broad-Crested weir length, m (ft)
H = Head above weir crest, m (ft)

Table 8-1. English Units - Broad-Crested Weir Coefficient C Values as a Function of Weir Crest.

Broad-Crested Weir Coefficient C Values as a Function of Weir Crest Breadth and Head (coefficient has units of $ft^{0.5}/sec$)[1]

Head[2] (ft)	Breadth of Crest of Weir (ft)										
	0.50	0.75	1.00	1.5	2.0	2.50	3.00	4.00	5.00	10.00	15.00
0.2	2.80	2.75	2.69	2.62	2.54	2.48	2.44	2.38	2.34	2.49	2.68
0.4	2.92	2.80	2.72	2.64	2.61	2.60	2.58	2.54	2.50	2.56	2.70
0.6	3.08	2.89	2.75	2.64	2.61	2.60	2.68	2.69	2.70	2.70	2.70
0.8	3.30	3.04	2.85	2.68	2.60	2.60	2.67	2.68	2.68	2.69	2.64
1.0	3.32	3.14	2.98	2.75	2.66	2.64	2.65	2.67	2.68	2.68	2.63
1.2	3.32	3.20	3.08	2.86	2.70	2.65	2.64	2.67	2.66	2.69	2.64
1.4	3.32	3.26	3.20	2.92	2.77	2.68	2.64	2.65	2.65	2.67	2.64
1.6	3.32	3.29	3.28	3.07	2.89	2.75	2.68	2.66	2.65	2.64	2.63
1.8	3.32	3.32	3.31	3.07	2.88	2.74	2.68	2.66	2.65	2.64	2.63
2.0	3.32	3.31	3.30	3.03	2.85	2.76	2.72	2.68	2.65	2.64	2.63
2.5	3.32	3.32	3.31	3.28	3.07	2.89	2.81	2.72	2.67	2.64	2.63
3.0	3.32	3.32	3.32	3.32	3.20	3.05	2.92	2.73	2.66	2.64	2.63
3.5	3.32	3.32	3.32	3.32	3.32	3.19	2.97	2.76	2.68	2.64	2.63
4.0	3.32	3.32	3.32	3.32	3.32	3.32	3.07	2.79	2.70	2.64	2.63
4.5	3.32	3.32	3.32	3.32	3.32	3.32	3.32	2.88	2.74	2.64	2.63
5.0	3.32	3.32	3.32	3.32	3.32	3.32	3.32	3.07	2.79	2.64	2.63
5.5	3.32	3.32	3.32	3.32	3.32	3.32	3.32	3.32	2.88	2.64	2.63

(1) Table is taken from Reference 49.

	Value	Units	Description
Pool Elevation =	762	Feet MSL	Maximum Normal Operating Pool
Width =	4.0	Feet	From 2010 Analysis

Headwater Elevation (FT)	Cbcw, Coefficient	Ao (SF)	Ho (FT)	Q, Discharge (CFS)
762.00	3.32	0.00	0.00	0.0
763.00	3.32	4.00	1.00	13.3
764.00	3.32	8.00	2.00	37.6
765.00	3.32	12.00	3.00	69.0
766.00	3.32	16.00	4.00	106.2
767.00	3.32	20.00	5.00	148.5
768.00	3.32	24.00	6.00	195.2
769.00	3.32	28.00	7.00	245.9
770.00	3.32	32.00	8.00	300.5
771.00	3.32	36.00	9.00	358.6
772.00	3.32	40.00	10.00	420.0

Project: Conesville Ash Pond Complex H&H
Location: Coshocton, Ohio
Project Number: 7217-14-009H

Clearwater Pond - Pipe Inlet Flow

Calculated By: MRM **Date:** 10/3/2016

Reviewed By: **Date:**

Source: Hydraulic Engineering Circular No. 22, Third Edition
Urban Drainage Design Manual (Rev. 2013)

8.4.4.1 Orifices

For a single orifice as illustrated in Figure 8-12 (a), orifice Flow can be determined using Equation 8-17.

$$Q = C_o A_o (2 g H_o)^{0.5} \quad (8\text{-}17)$$

where:

Q = Orifice flow rate, m^3/s (ft^3s)
C_o = Discharge coefficient (0.40 – 0.60)
Ao = Area of orifice, m^2 (ft^2)
H_o = Effective head on the orifice measured from the centroid of the opening, m (ft)
g = Gravitational acceleration, 9.81 m/s^2 (32.2 ft/s^2 for English units)

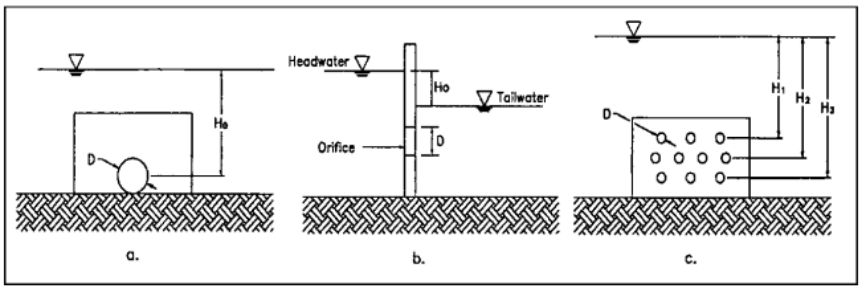


Figure 8-12. Definition sketch for orifice flow.

	Value	Units	Description
Pool Elevation =	762	Feet MSL	Maximum Normal Operating Pool
Diameter =	36.0	Inches	Inside Diameter of Orifice
Orifice Elevation =	751.50	Feet MSL	Controid Elevation of Orifice (750 + 1.5ft)
Co =	0.6	(0.4-0.6)	Use 0.6 for circular pipe with square edges

Headwater Elevation (FT)	Co, Coefficient	Ao (SF)	Ho (FT)	g (FT/S^2)	Q, Orifice Discharge (CFS)
762.00	0.6	7.07	10.50	36.2	*116.9*
763.00	0.6	7.07	11.50	37.2	*124.1*
764.00	0.6	7.07	12.50	38.2	*131.1*
765.00	0.6	7.07	13.50	39.2	*138.0*
766.00	0.6	7.07	14.50	40.2	*144.8*
767.00	0.6	7.07	15.50	41.2	*151.6*
768.00	0.6	7.07	16.50	42.2	*158.3*
769.00	0.6	7.07	17.50	43.2	*164.9*
770.00	0.6	7.07	18.50	41.2	*165.6*
771.00	0.6	7.07	19.50	42.2	*172.1*
772.00	0.6	7.07	20.50	43.2	*178.5*

Project: Conesville Ash Pond Complex H&H
Location: Coshocton, Ohio
Project Number: 7217-14-009H

Clearwater Pond - Pipe Pressure Flow
36" Concrete Outlet Pipe

Calculated By: MRM **Date:** 10/3/2016

Reviewed By: **Date:**

Sources: Hydraulic Engineering Circular No. 22, Third Edition, Urban Drainage Design Manual (Rev. 2013)
Oho Department of Transportation, Hydraulics Manual, Chapter 8, Appendix A

	Value	Units	Description
Manning's n =	**0.011**		Based on Concrete Pipe, straight, (Chow, 1959)
Inlet Invert =	**750**	Feet msl	
Outlet Invert (z2) =	**730**	Feet msl	
Entrance Coefficent ($K_{e)}$ =	**0.5**		
Expansion Coefficent ($K_{o)}$ =	**1**		
Bend Coefficent ($K_{b)}$ =	**0**		
Pipe Diameter (D) =	**3**	Feet	From Pipe Inspection
Pipe Length (L) =	**300**	Feet	
Darcy-Weisbach (*f*) =	0.016		Relate Manning's n to Darcy-Weisbach friction factor
Gravity Constant (g) =	32.2	Feet/sec^2	

$$f = \frac{185\ n^2}{D^{\frac{1}{3}}}$$

Headwater Elevation (z1) (FT)	Velocity (FPS)	Area (SF)	Flow (CFS)
762.00	**22.55**	**7.07**	159.4
763.00	**22.90**	**7.07**	161.9
764.00	**23.25**	**7.07**	164.3
765.00	**23.59**	**7.07**	166.7
766.00	**23.92**	**7.07**	169.1
767.00	**24.25**	**7.07**	171.4
768.00	**24.58**	**7.07**	173.7
769.00	**24.90**	**7.07**	176.0
770.00	**25.21**	**7.07**	178.2
771.00	**25.53**	**7.07**	180.4
772.00	**25.84**	**7.07**	182.6

ENERGY EQUATION

$$\frac{p_1}{\gamma}+\frac{v_1^2}{2g}+z_1=\frac{p_2}{\gamma}+\frac{v_2^2}{2g}+z_2+\sum h_L$$

HEAD LOSS EQUATION

$$\sum h_L=\frac{v^2}{2g}\left(f\frac{L}{D}+K_e+K_o+K_b\right)$$

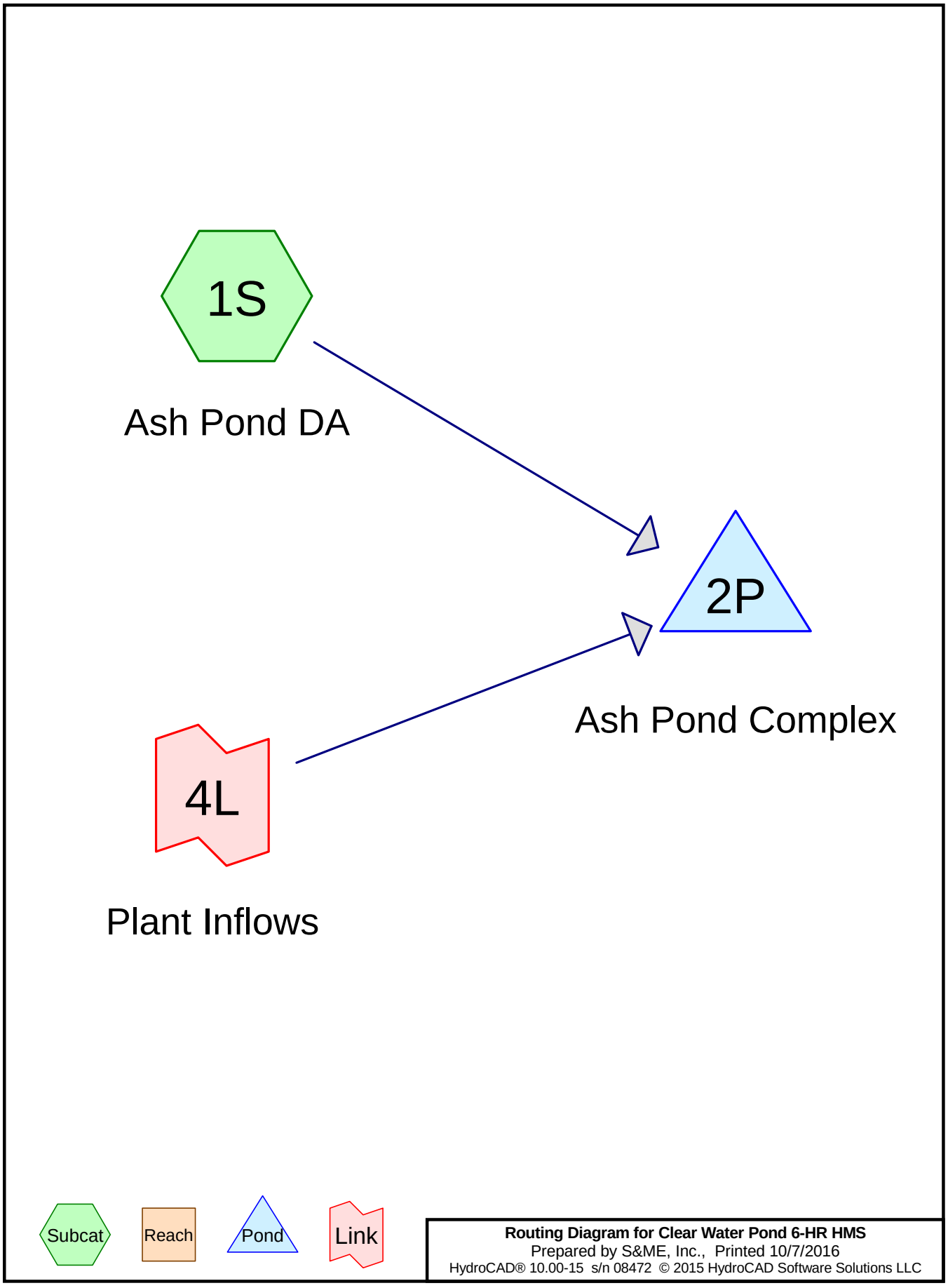
SOLVE FOR VELOCITY

$$v=\sqrt{\frac{2g(z_1-z_2)}{\left(1+\left(f\frac{L}{D}+K_e+K_o+K_b\right)\right)}}$$

$$Q = VA$$

Clearwater Pond Outlet Structure Capacity - Case 4							
Lake Elevation	Stop Log Flow Sharp Weir*	Stop Log Flow Broad Weir	Pipe Inlet Flow	Pressure Pipe Flow	**Control Flow**	***Total Outflow***	Control Type
feet	cfs		cfs	cfs	**cfs**	***MGD***	
762.00	0.0	0.0	116.9	159.4	**0.0**	***0.0***	Stop Log Flow Sharp Weir*
763.00	12.7	13.3	124.1	161.9	**12.7**	***8.2***	Stop Log Flow Sharp Weir*
764.00	33.9	37.6	131.1	164.3	**33.9**	***21.9***	Stop Log Flow Sharp Weir*
765.00	58.8	69.0	138.0	166.7	**69.0**	***44.6***	Stop Log Flow Broad Weir
766.00	87.1	106.2	144.8	169.1	**106.2**	***68.7***	Stop Log Flow Broad Weir
767.00	115.3	148.5	151.6	171.4	**148.5**	***96.0***	Stop Log Flow Broad Weir
768.00	142.8	195.2	158.3	173.7	**158.3**	***102.3***	Pipe Inlet Flow
769.00	168.7	245.9	164.9	176.0	**164.9**	***106.6***	Pipe Inlet Flow
770.00	192.1	300.5	165.6	178.2	**165.6**	***107.0***	Pipe Inlet Flow
771.00	212.1	358.6	172.1	180.4	**172.1**	***111.2***	Pipe Inlet Flow
772.00	227.9	420.0	178.5	182.6	**178.5**	***115.4***	Pipe Inlet Flow

*Equation [8-20] from HEC-22 only applicable for range of values when L>3H. After elevation 764 the broad-crested weir equation is more appropriate.


1S
Ash Pond DA
2P
Ash Pond Complex
4L
Plant Inflows
Subcat
Reach
Pond
Link
Routing Diagram for Clear Water Pond 6-HR HMS
Prepared by S&ME, Inc., Printed 10/7/2016
HydroCAD® 10.00-15 s/n 08472 © 2015 HydroCAD Software Solutions LLC

Area Listing (all nodes)

Area (acres)	CN	Description (subcatchment-numbers)
128.640	93	(1S)

Summary for Subcatchment 1S: Ash Pond DA

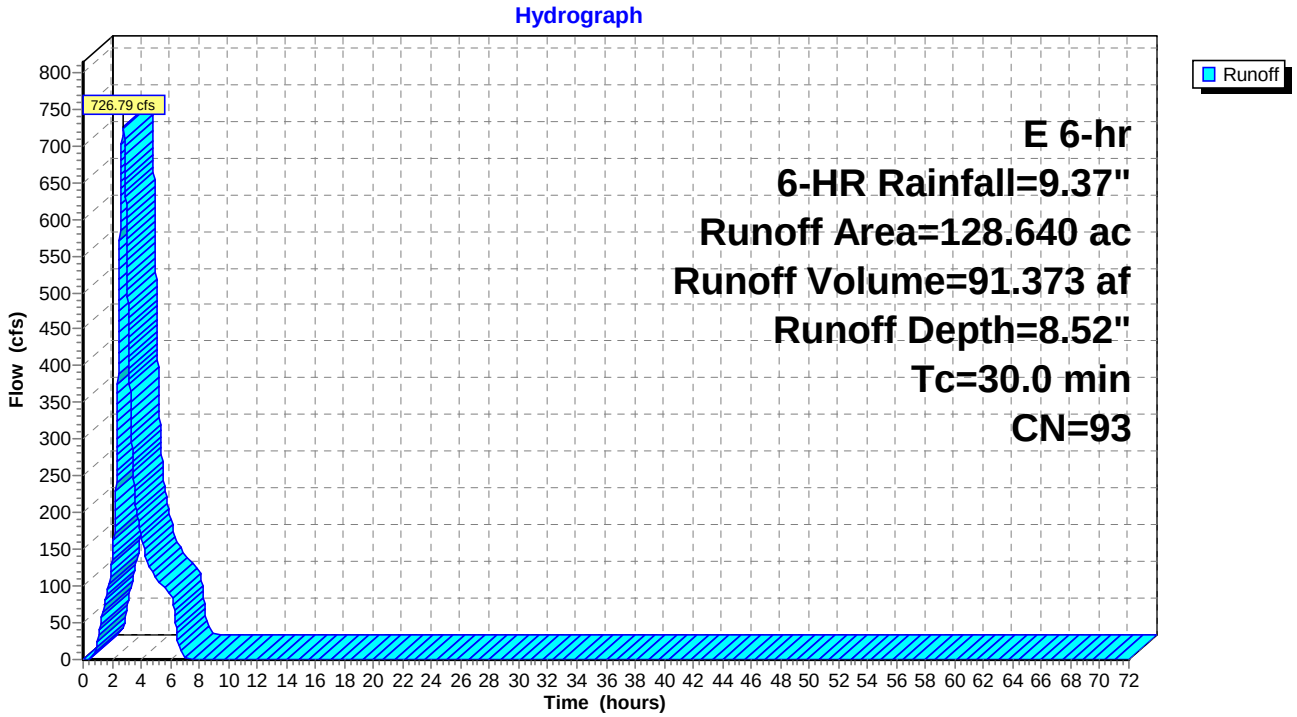
Runoff = 726.79 cfs @ 2.76 hrs, Volume= 91.373 af, Depth= 8.52"

Runoff by SCS TR-20 method, UH=SCS, Weighted-CN, Time Span= 0.00-72.00 hrs, dt= 0.01 hrs
E 6-hr 6-HR Rainfall=9.37"

	Area (ac)	CN	Description
*	128.640	93	
	128.640		100.00% Pervious Area

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

Subcatchment 1S: Ash Pond DA



Summary for Pond 2P: Ash Pond Complex

Inflow Area = 128.640 ac, 0.00% Impervious, Inflow Depth = 10.49" for 6-HR event
Inflow = 769.19 cfs @ 2.76 hrs, Volume= 112.433 af
Outflow = 18.14 cfs @ 6.60 hrs, Volume= 64.893 af, Atten= 98%, Lag= 230.2 min
Primary = 18.14 cfs @ 6.60 hrs, Volume= 64.893 af

Routing by Dyn-Stor-Ind method, Time Span= 0.00-72.00 hrs, dt= 0.01 hrs / 3
Starting Elev= 762.00' Surf.Area= 0.000 ac Storage= 2,470.000 af
Peak Elev= 763.26' @ 6.60 hrs Surf.Area= 0.000 ac Storage= 2,577.429 af (107.429 af above start)
Flood Elev= 770.00' Surf.Area= 0.000 ac Storage= 3,163.000 af (693.000 af above start)

Plug-Flow detention time= (not calculated: initial storage exceeds outflow)
Center-of-Mass det. time= 1,734.8 min (1,932.7 - 197.9)

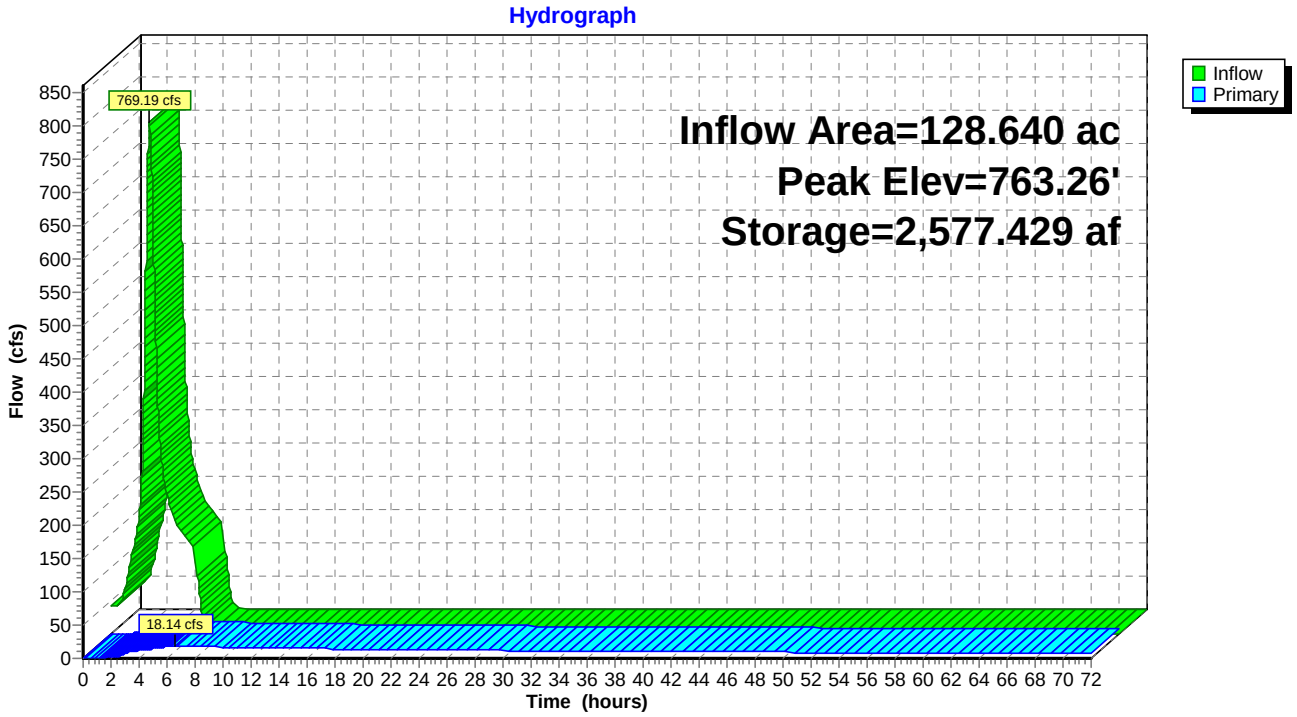
Volume	Invert	Avail.Storage	Storage Description
#1	730.00'	3,300.000 af	**Custom Stage Data** Listed below

Elevation (feet)	Cum.Store (acre-feet)
730.00	0.000
735.00	353.000
740.00	719.000
750.00	1,484.000
760.00	2,299.000
766.00	2,812.000
770.00	3,163.000
771.00	3,200.000
772.00	3,300.000

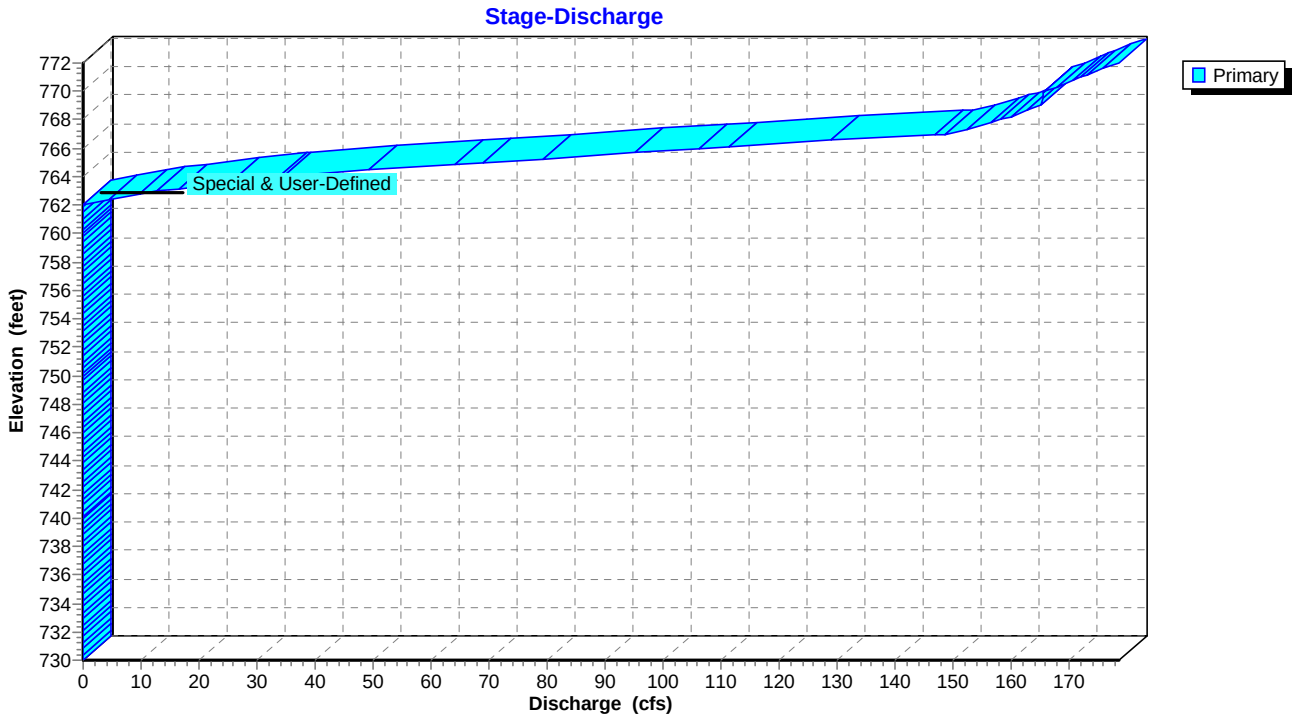
Device	Routing	Invert	Outlet Devices
#1	Primary	762.00'	**Special & User-Defined** Elev. (feet) 762.00 763.00 764.00 765.00 766.00 767.00 768.00 769.00 770.00 771.00 772.00 Disch. (cfs) 0.000 12.700 33.900 69.000 106.200 148.500 158.300 164.900 165.600 172.100 178.500

Primary OutFlow Max=18.14 cfs @ 6.60 hrs HW=763.26' (Free Discharge)
↑—**1=Special & User-Defined** (Custom Controls 18.14 cfs)

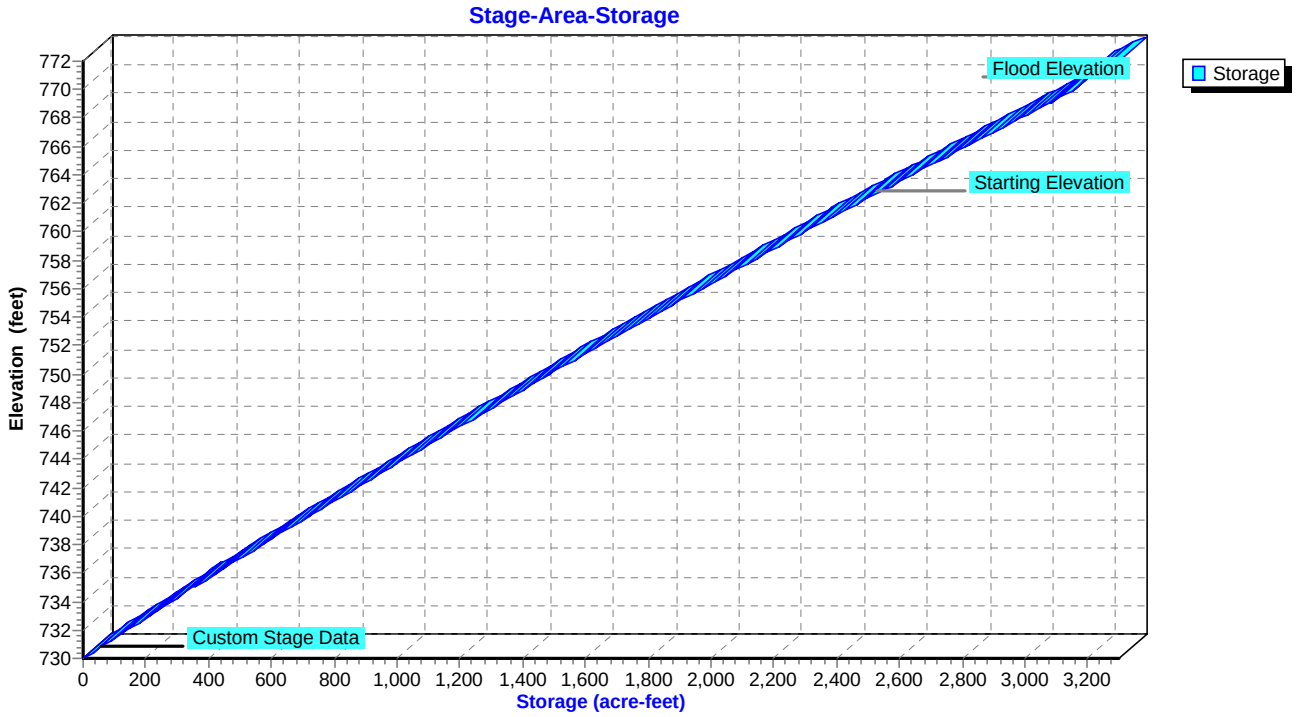
Pond 2P: Ash Pond Complex



Pond 2P: Ash Pond Complex



Pond 2P: Ash Pond Complex



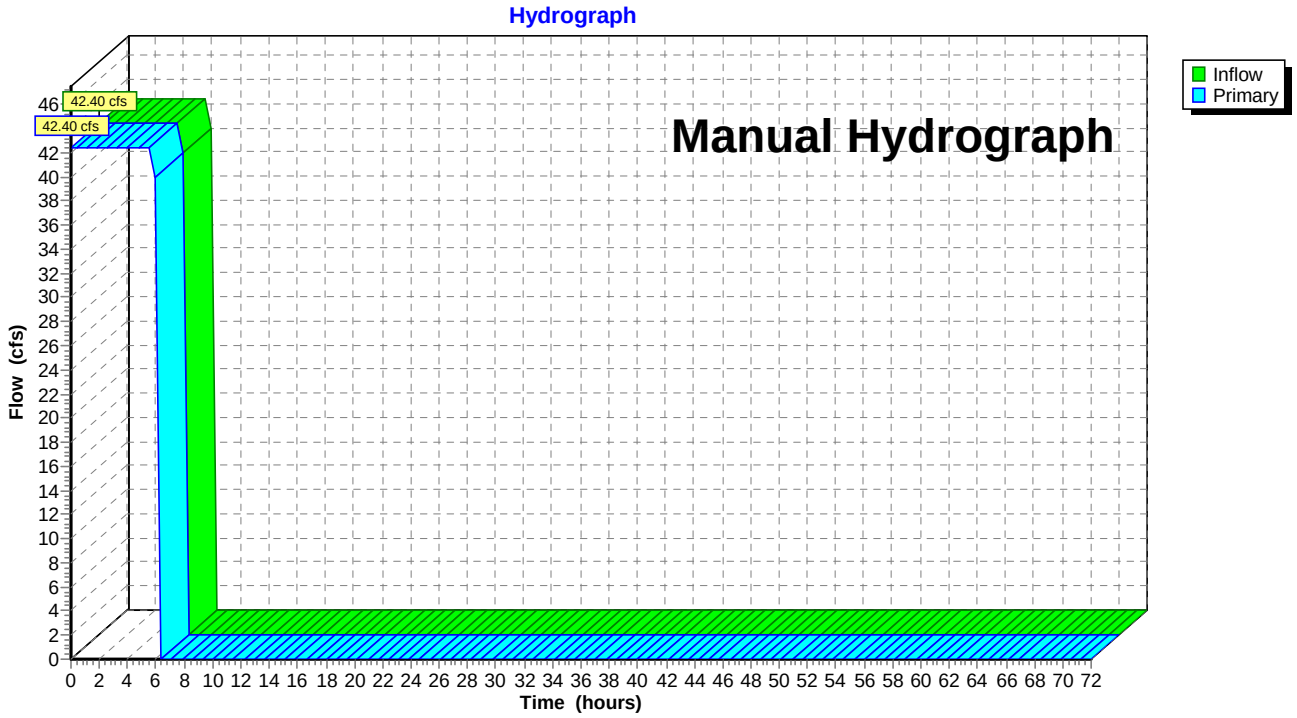
Summary for Link 4L: Plant Inflows

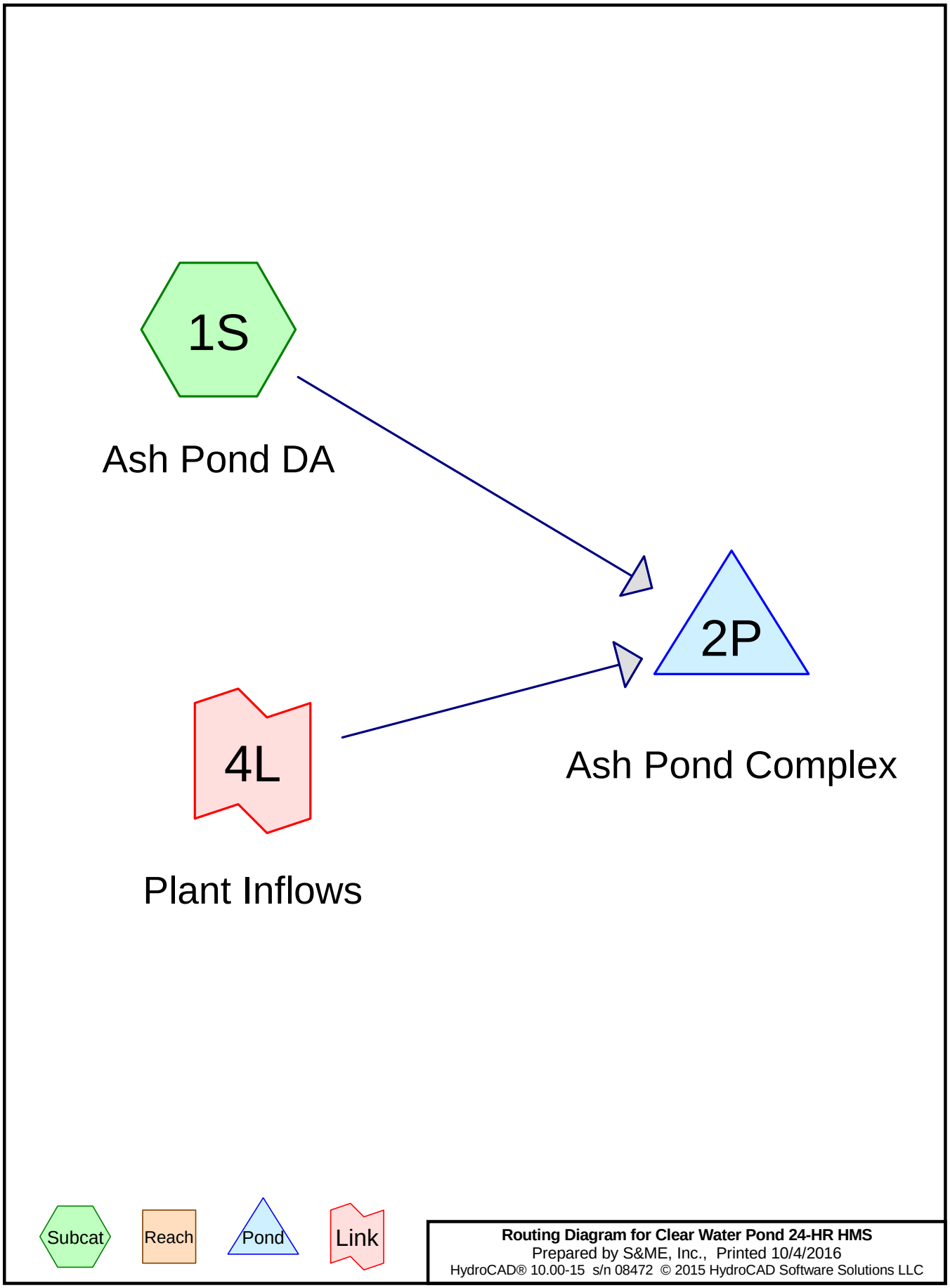
Inflow = 42.40 cfs @ 0.00 hrs, Volume= 21.060 af
Primary = 42.40 cfs @ 0.00 hrs, Volume= 21.060 af, Atten= 0%, Lag= 0.0 min

Primary outflow = Inflow, Time Span= 0.00-72.00 hrs, dt= 0.01 hrs

2 Point manual hydrograph, To= 0.00 hrs, dt= 6.00 hrs, cfs =
42.40 42.40

Link 4L: Plant Inflows




1S
Ash Pond DA
2P
4L
Ash Pond Complex
Plant Inflows
Subcat
Reach
Pond
Link
Routing Diagram for Clear Water Pond 24-HR HMS
Prepared by S&ME, Inc., Printed 10/4/2016
HydroCAD® 10.00-15 s/n 08472 © 2015 HydroCAD Software Solutions LLC

Area Listing (all nodes)

Area (acres)	CN	Description (subcatchment-numbers)
128.640	93	(1S)

Summary for Subcatchment 1S: Ash Pond DA

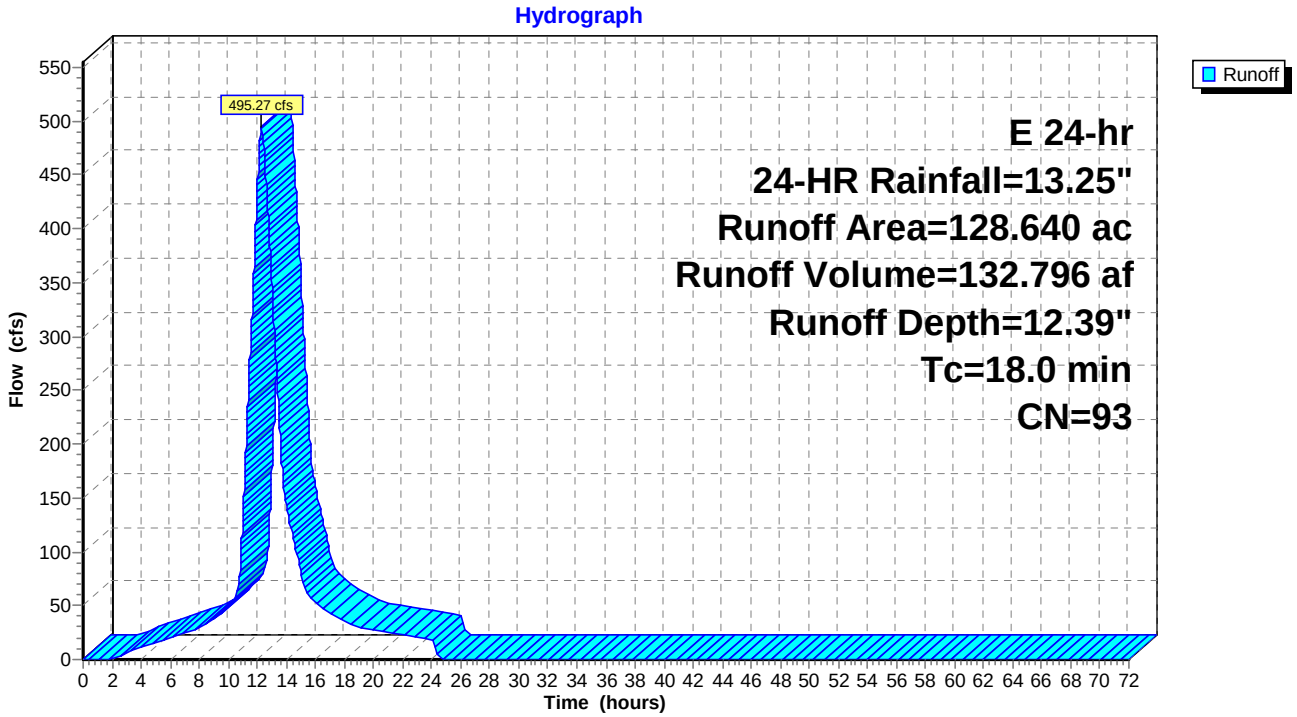
Runoff = 495.27 cfs @ 12.26 hrs, Volume= 132.796 af, Depth=12.39"

Runoff by SCS TR-20 method, UH=SCS, Weighted-CN, Time Span= 0.00-72.00 hrs, dt= 0.01 hrs
E 24-hr 24-HR Rainfall=13.25"

	Area (ac)	CN	Description
*	128.640	93	
	128.640		100.00% Pervious Area

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

Subcatchment 1S: Ash Pond DA



Summary for Pond 2P: Ash Pond Complex

Inflow Area = 128.640 ac, 0.00% Impervious, Inflow Depth = 20.24" for 24-HR event
Inflow = 537.67 cfs @ 12.26 hrs, Volume= 216.930 af
Outflow = 38.28 cfs @ 24.01 hrs, Volume= 127.475 af, Atten= 93%, Lag= 704.6 min
Primary = 38.28 cfs @ 24.01 hrs, Volume= 127.475 af

Routing by Dyn-Stor-Ind method, Time Span= 0.00-72.00 hrs, dt= 0.01 hrs / 3
Starting Elev= 762.00' Surf.Area= 0.000 ac Storage= 2,470.000 af
Peak Elev= 764.12' @ 24.01 hrs Surf.Area= 0.000 ac Storage= 2,651.670 af (181.670 af above start)
Flood Elev= 770.00' Surf.Area= 0.000 ac Storage= 3,163.000 af (693.000 af above start)

Plug-Flow detention time= (not calculated: initial storage exceeds outflow)
Center-of-Mass det. time= 1,445.7 min (2,208.1 - 762.4)

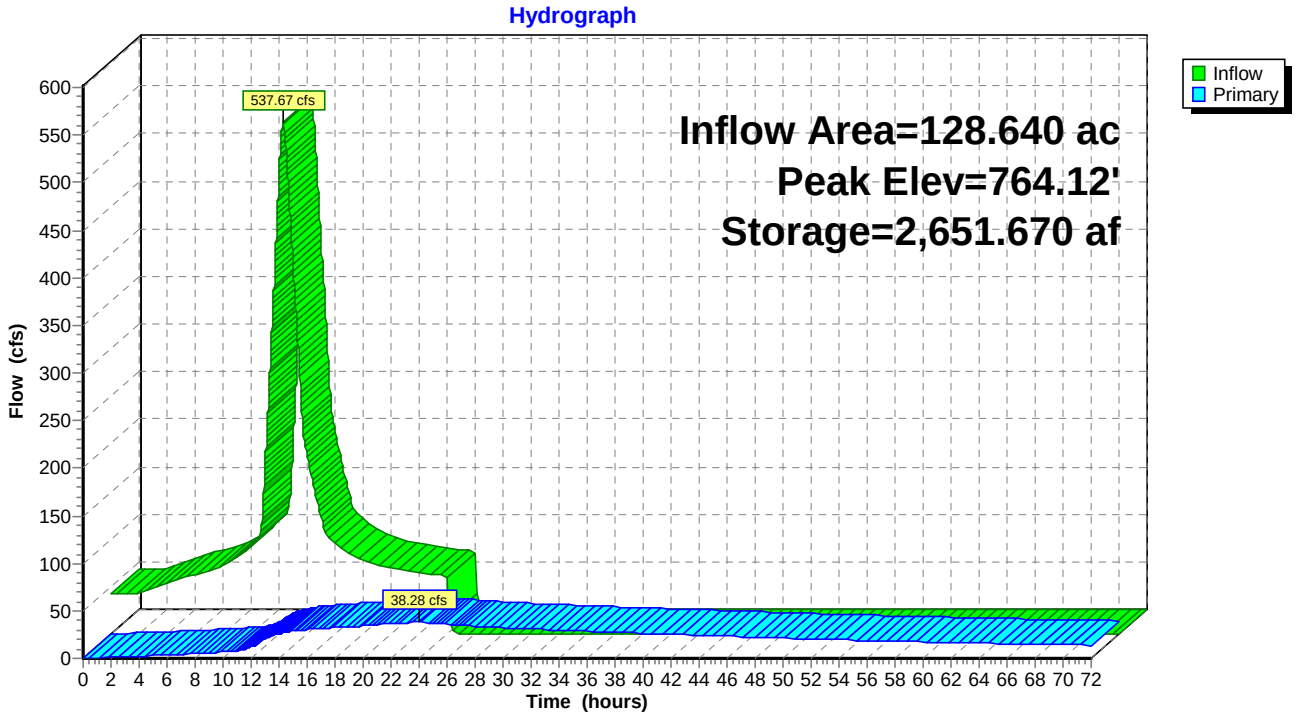
Volume	Invert	Avail.Storage	Storage Description
#1	730.00'	3,300.000 af	**Custom Stage Data** Listed below

Elevation (feet)	Cum.Store (acre-feet)
730.00	0.000
735.00	353.000
740.00	719.000
750.00	1,484.000
760.00	2,299.000
766.00	2,812.000
770.00	3,163.000
771.00	3,200.000
772.00	3,300.000

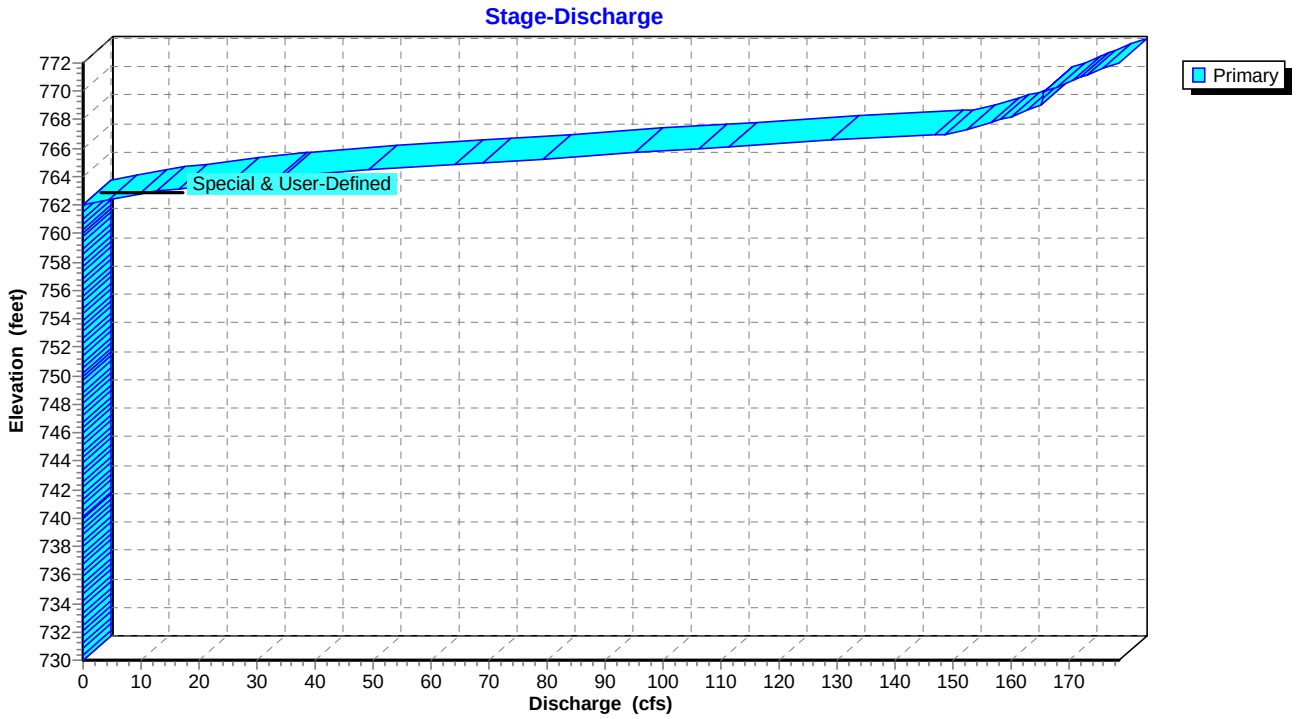
Device	Routing	Invert	Outlet Devices
#1	Primary	762.00'	**Special & User-Defined** Elev. (feet) 762.00 763.00 764.00 765.00 766.00 767.00 768.00 769.00 770.00 771.00 772.00 Disch. (cfs) 0.000 12.700 33.900 69.000 106.200 148.500 158.300 164.900 165.600 172.100 178.500

Primary OutFlow Max=38.28 cfs @ 24.01 hrs HW=764.12' (Free Discharge)
↑—**1=Special & User-Defined** (Custom Controls 38.28 cfs)

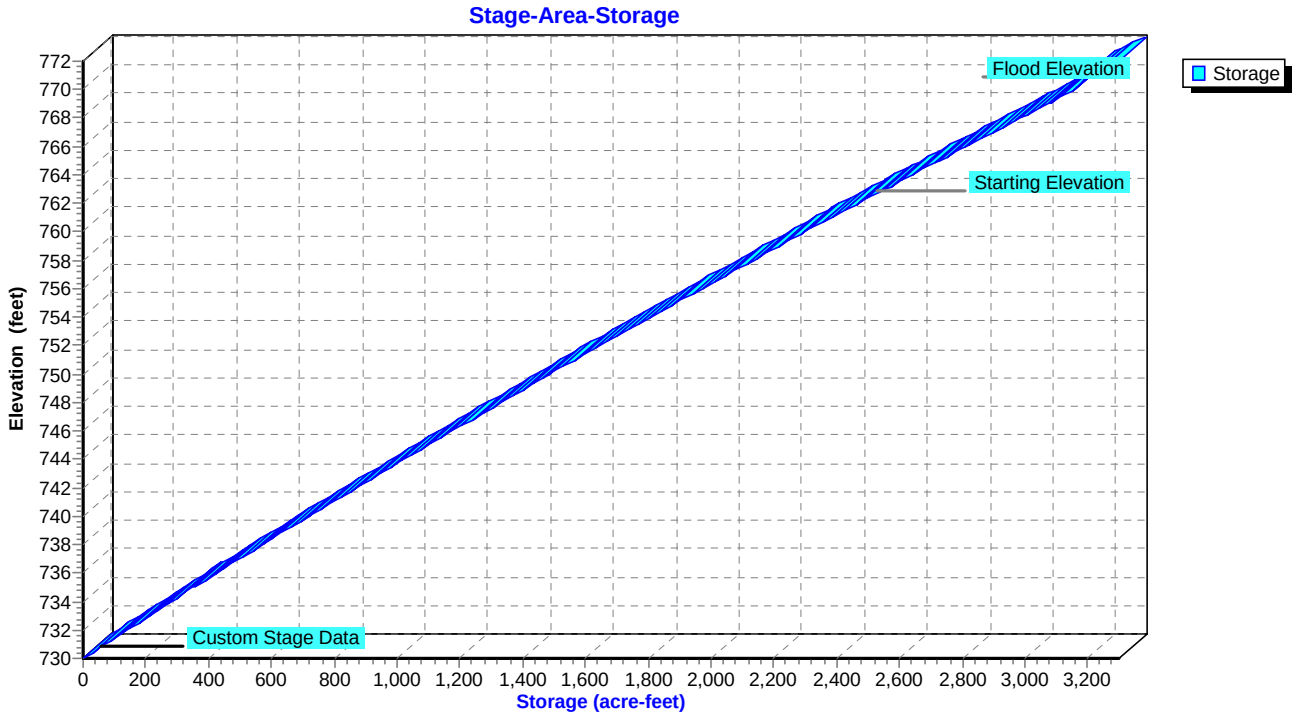
Pond 2P: Ash Pond Complex



Pond 2P: Ash Pond Complex



Pond 2P: Ash Pond Complex



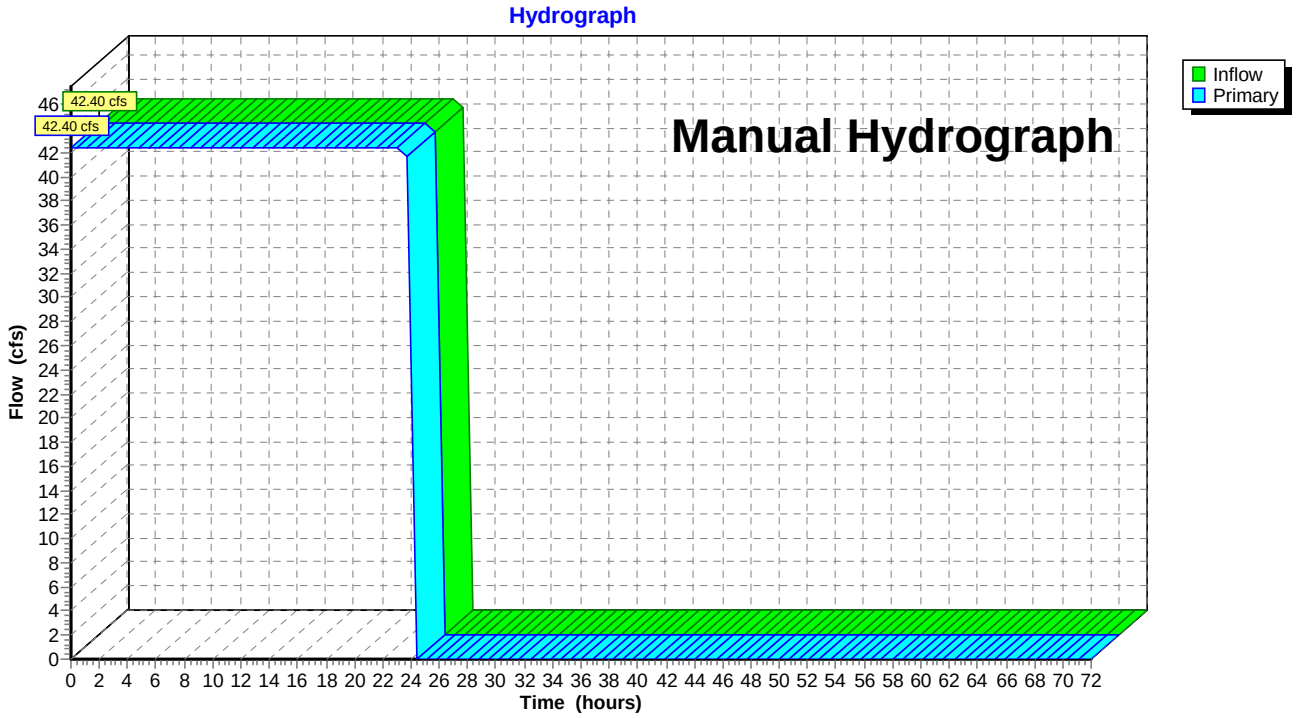
Summary for Link 4L: Plant Inflows

Inflow = 42.40 cfs @ 0.00 hrs, Volume= 84.134 af
Primary = 42.40 cfs @ 0.00 hrs, Volume= 84.134 af, Atten= 0%, Lag= 0.0 min

Primary outflow = Inflow, Time Span= 0.00-72.00 hrs, dt= 0.01 hrs

2 Point manual hydrograph, To= 0.00 hrs, dt= 24.00 hrs, cfs =
42.40 42.40

Link 4L: Plant Inflows



Attachment B – NOAA Point Precipitation Data

NOAA Atlas 14, Volume 2, Version 3
Location name: Conesville, Ohio, USA*
Latitude: 40.1885°, Longitude: -81.8707°
Elevation: 755.2 ft**
* source: ESRI Maps
** source: USGS

POINT PRECIPITATION FREQUENCY ESTIMATES

G.M. Bonnin, D. Martin, B. Lin, T. Parzybok, M.Yekta, and D. Riley

NOAA, National Weather Service, Silver Spring, Maryland

PF_tabular | PF_graphical | Maps_&_aerials

PF tabular

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

Duration	Average recurrence interval (years)									
	1	2	5	10	25	50	100	200	500	1000
5-min	0.338 (0.306–0.374)	0.405 (0.366–0.448)	0.490 (0.442–0.541)	0.555 (0.499–0.612)	0.638 (0.573–0.703)	0.701 (0.627–0.771)	0.762 (0.679–0.838)	0.826 (0.733–0.908)	0.910 (0.802–1.00)	0.972 (0.852–1.07)
10-min	0.525 (0.475–0.582)	0.633 (0.572–0.700)	0.762 (0.687–0.841)	0.857 (0.771–0.945)	0.975 (0.876–1.08)	1.06 (0.950–1.17)	1.15 (1.02–1.26)	1.23 (1.09–1.36)	1.34 (1.18–1.47)	1.42 (1.24–1.56)
15-min	0.644 (0.582–0.713)	0.774 (0.699–0.856)	0.935 (0.844–1.03)	1.05 (0.949–1.16)	1.21 (1.08–1.33)	1.32 (1.18–1.45)	1.43 (1.27–1.57)	1.53 (1.36–1.69)	1.67 (1.47–1.83)	1.77 (1.55–1.95)
30-min	0.852 (0.771–0.943)	1.04 (0.936–1.15)	1.28 (1.16–1.41)	1.46 (1.32–1.61)	1.70 (1.53–1.87)	1.88 (1.68–2.07)	2.06 (1.83–2.26)	2.24 (1.99–2.46)	2.47 (2.18–2.72)	2.65 (2.33–2.92)
60-min	1.04 (0.941–1.15)	1.27 (1.15–1.41)	1.61 (1.45–1.77)	1.86 (1.68–2.05)	2.21 (1.98–2.43)	2.48 (2.22–2.73)	2.75 (2.45–3.03)	3.04 (2.69–3.34)	3.42 (3.02–3.76)	3.73 (3.27–4.10)
2-hr	1.20 (1.09–1.34)	1.47 (1.33–1.63)	1.88 (1.70–2.10)	2.22 (2.00–2.47)	2.72 (2.44–3.01)	3.13 (2.80–3.46)	3.58 (3.18–3.95)	4.06 (3.60–4.48)	4.77 (4.20–5.25)	5.37 (4.69–5.90)
3-hr	1.29 (1.16–1.44)	1.56 (1.41–1.75)	2.01 (1.81–2.25)	2.37 (2.12–2.65)	2.90 (2.59–3.24)	3.35 (2.98–3.72)	3.84 (3.40–4.27)	4.38 (3.86–4.85)	5.17 (4.52–5.72)	5.85 (5.07–6.46)
6-hr	1.53 (1.38–1.70)	1.84 (1.66–2.06)	2.34 (2.11–2.61)	2.77 (2.49–3.07)	3.39 (3.03–3.75)	3.93 (3.49–4.34)	4.52 (4.00–4.98)	5.18 (4.54–5.68)	6.14 (5.34–6.73)	6.98 (6.02–7.62)
12-hr	1.81 (1.65–2.01)	2.18 (1.98–2.41)	2.74 (2.48–3.03)	3.22 (2.91–3.55)	3.94 (3.55–4.34)	4.57 (4.09–5.01)	5.26 (4.69–5.76)	6.04 (5.34–6.59)	7.20 (6.31–7.83)	8.20 (7.12–8.90)
24-hr	2.10 (1.95–2.28)	2.51 (2.33–2.74)	3.14 (2.91–3.41)	3.68 (3.40–3.99)	4.50 (4.13–4.86)	5.21 (4.74–5.61)	5.99 (5.40–6.44)	6.86 (6.13–7.37)	8.17 (7.18–8.77)	9.30 (8.06–9.98)
2-day	2.44 (2.26–2.64)	2.91 (2.71–3.15)	3.59 (3.34–3.89)	4.18 (3.87–4.52)	5.06 (4.66–5.46)	5.81 (5.31–6.26)	6.63 (6.01–7.15)	7.53 (6.75–8.12)	8.87 (7.82–9.58)	10.0 (8.70–10.8)
3-day	2.61 (2.44–2.81)	3.12 (2.91–3.36)	3.83 (3.57–4.11)	4.43 (4.12–4.76)	5.31 (4.91–5.71)	6.06 (5.57–6.51)	6.88 (6.27–7.38)	7.76 (7.01–8.34)	9.05 (8.06–9.75)	10.1 (8.92–11.0)
4-day	2.79 (2.61–2.99)	3.32 (3.11–3.56)	4.06 (3.80–4.34)	4.67 (4.36–5.00)	5.57 (5.17–5.95)	6.32 (5.83–6.75)	7.12 (6.53–7.62)	7.99 (7.26–8.55)	9.24 (8.30–9.92)	10.3 (9.13–11.1)
7-day	3.38 (3.18–3.61)	4.01 (3.77–4.29)	4.86 (4.56–5.20)	5.57 (5.21–5.95)	6.59 (6.14–7.03)	7.43 (6.89–7.93)	8.33 (7.67–8.90)	9.28 (8.48–9.92)	10.6 (9.61–11.4)	11.8 (10.5–12.6)
10-day	3.89 (3.65–4.15)	4.61 (4.34–4.92)	5.52 (5.19–5.89)	6.27 (5.88–6.68)	7.31 (6.83–7.79)	8.15 (7.59–8.69)	9.03 (8.36–9.63)	9.94 (9.15–10.6)	11.2 (10.2–12.0)	12.2 (11.0–13.1)
20-day	5.44 (5.13–5.76)	6.43 (6.06–6.82)	7.58 (7.15–8.05)	8.48 (7.99–9.01)	9.69 (9.10–10.3)	10.6 (9.94–11.3)	11.5 (10.8–12.3)	12.5 (11.6–13.3)	13.7 (12.6–14.6)	14.6 (13.4–15.6)
30-day	6.83 (6.48–7.21)	8.05 (7.63–8.50)	9.39 (8.89–9.92)	10.4 (9.86–11.0)	11.8 (11.1–12.4)	12.8 (12.0–13.5)	13.8 (12.9–14.5)	14.7 (13.8–15.6)	16.0 (14.9–16.9)	16.9 (15.6–17.9)
45-day	8.75 (8.30–9.23)	10.3 (9.74–10.8)	11.8 (11.2–12.5)	13.0 (12.3–13.7)	14.4 (13.7–15.2)	15.5 (14.7–16.4)	16.6 (15.6–17.5)	17.6 (16.5–18.6)	18.8 (17.6–19.9)	19.7 (18.4–20.9)
60-day	10.5 (10.0–11.1)	12.3 (11.8–13.0)	14.1 (13.4–14.8)	15.4 (14.6–16.2)	17.0 (16.1–17.9)	18.1 (17.2–19.1)	19.2 (18.2–20.2)	20.2 (19.1–21.3)	21.4 (20.2–22.6)	22.3 (21.0–23.6)

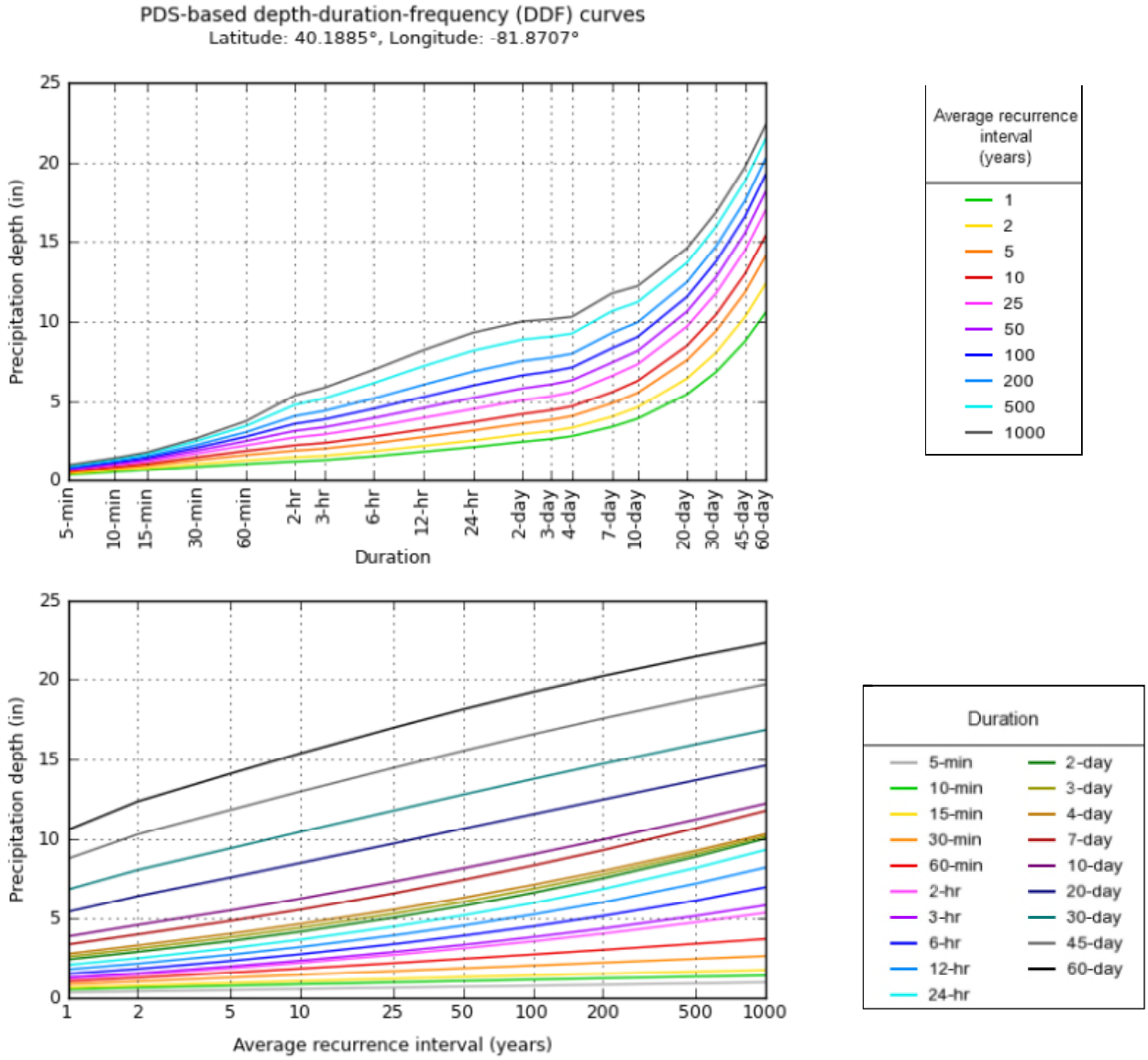
[1] Precipitation frequency (PF) estimates in this table are based on frequency analysis of partial duration series (PDS).

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.

Back to Top

PF graphical



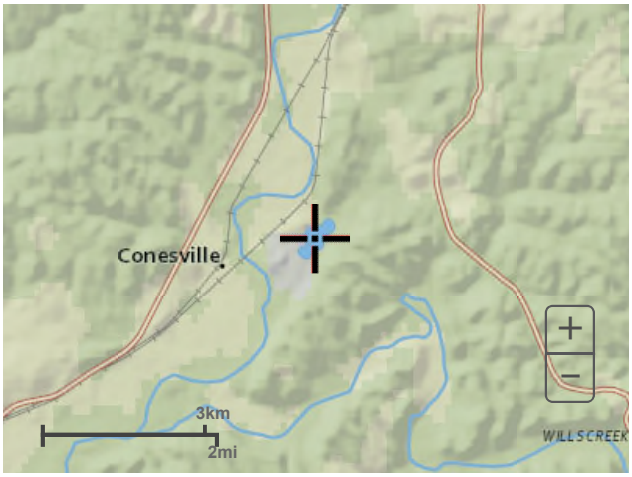
NOAA Atlas 14, Volume 2, Version 3

Created (GMT): Tue Oct 4 15:51:40 2016

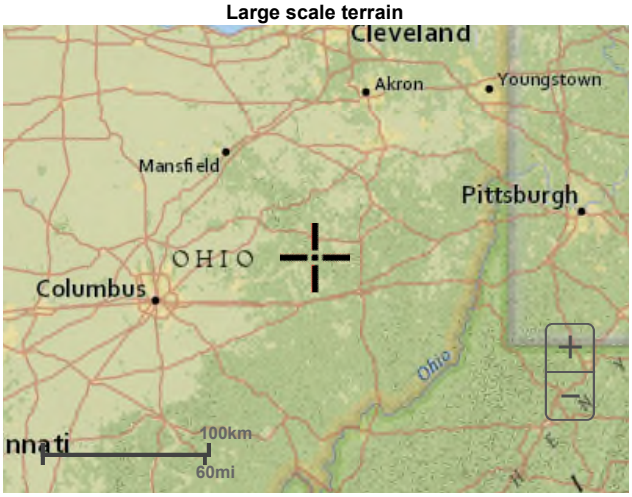
Back to Top

Maps & aerials

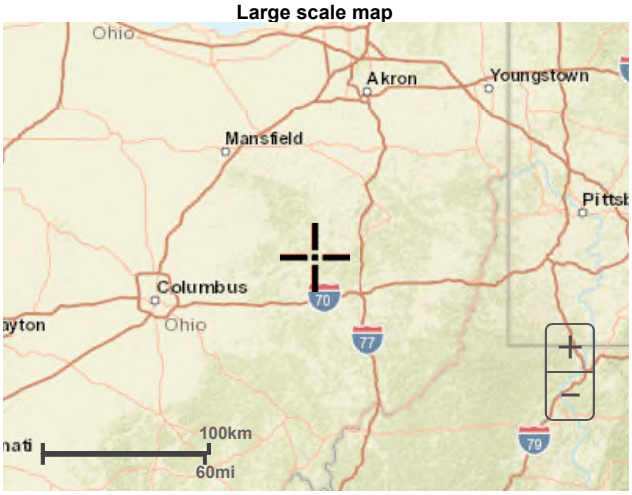
Small scale terrain



Large scale terrain



Large scale map



Large scale aerial

Back to Top

US Department of Commerce
National Oceanic and Atmospheric Administration
National Weather Service
National Water Center
1325 East West Highway
Silver Spring, MD 20910
Questions?: HDSC.Questions@noaa.gov

Disclaimer