DATE: September 13, 2024

DEPARTMENT OF GENERAL SERVICES
BUREAU OF CAPITAL PROJECT DESIGN MANAGEMENT
1800 HERR STREET
HARRISBURG, PENNSYLVANIA

ADDENDUM NO. 3

on

PROJECT NO. DGS C-0960-0086 PHASE 001
PROJECT TITLE - Marquette Lake Dam - Rehabilitation
PROFESSIONAL:
D'Appolonia Engineering Division of Ground Technology, Inc.
701 Rodi Road, Floor 2
Pittsburgh, PA, 15235

If you submitted a bid prior to this Addendum being issued, your bid has been discarded and you must re-submit your bid(s) prior to the bid opening date and time.

ADMINISTRATIVE CHANGES – ALL CONTRACTS

Item 1- The bid period will be extended by (2) weeks. Revised dates are as follows:

- Deadline to Submit RFI's: 9/24/24 @ 3:00 PM
- Deadline to Submit Addenda: 10/1/24
- Bid Due Date: 10/8/24 @ 2:00 PM

Item 2 – Pre-Final Hydrologic and Hydraulic Design Report dated March 2021 that includes hydrologic data used during design has been provided for reference during bidding. Use of report does not change contractor responsibility for design of Control of Water and Diversion of surface water plans. Refer to attached report.

SPECIFICATION CHANGES – ALL CONTRACTS

Item 1 – Section 033000 Paragraph 2.3.A: REPLACE Paragraph 2.3.A to read "Structural Concrete Mix: PennDOT Class AAAP Concrete according to Section 704 of the PennDOT Standard Specifications, except a minimum 25% of the cementitious material (by weight) shall consist of fly ash. Maximum water-cement ratio by weight is 0.42. The requirements of this Specification supersede the requirements of the PennDOT Standard Specifications where a conflict may exist. All concrete shall be Structural Concrete unless otherwise indicated."

Item 2 – Section 010100 Paragraph 1.4: CLARIFCATION - The contract duration of 730 days incorporates and assumes limited construction activities during the winter due to poor weather such that winter shutdown will not add days to schedule.

Item 3 – Section 31 23 20 Paragraph 3.4.A: CLARIFICATION – Paragraph 3.4.A includes process and requirements for subgrade inspection, preparation and repair of depressions/holes/cracks by backfilling with PennDOT Class C concrete as directed by Professional. The intent is that Cass C Backfill Concrete provided at unit cost under Section 010250 be used to backfill overexcavation required to meet subgrade preparation requirements beyond the bearing levels and strata shown on the plans.

Item 4 – Section 31 23 20 Paragraph 3.4.A: REPLACE Paragraph 3.4.A to read "Flood Protection: The Contractor shall provide flood protection of the work area to the requirements of this section. A higher level of protection may be provided at the Contractor's discretion and expense. The Contractor agrees to the following provisions."

Item 5 – Section 31 23 20: CLARIFICATION - Per 31 23 20 3.4.C.2, a cofferdam, diversion and/or bypass shall be installed to protect the spillway work areas for at least the 10-year frequency storm with 1-foot of freeboard. The intent of this requirement is to protect work areas from inundation/flooding up to 10-year storm. Per 31 23 20 1.5.A, the control of water plan will demonstrate protection of the embankment soils, abutment soils, and "active" work area for storm events up to the 100-yr/24-hr. The intent of this requirement is to allow for flows from storms larger than 10-yr/24-hr and up to 100-yr/24-hr, to be passed through inactive work areas provided that the embankment soils, abutment soils, and "active" work areas are protected.

DRAWING CHANGES – ALL CONTRACTS

Item 1 - Drawing G-3: CLARIFICATION - Security requirements for access to the Base by truck and equipment operators are addressed under Section 016350. There are no requirements to bond roads on Fort Indiantown Gap. Maintenance requirements are limited to keeping the roads clean and removing any dirt tracked onto roads due to construction, earthmoving and hauling operations. Trucks and truck drivers should comply with all Pennsylvania traffic laws, regulations, and requirements to use public roads including legal weight limits and following posted speed limits and other regulatory traffic signs.

Item 2 – Drawing C-303: CLARIFICATION - The box culvert replacing the existing 17th street bridge includes a structure mounted guiderail along headwalls of culvert in accordance with BD-632M and BC-706M. Wingwalls are intended to be precast to limit duration 17th Street is closed during installation. Precast wingwalls and endwall section shall be in accordance with PennDOT requirements in BD-600M.

Item 3 – Drawing C-114: CLARIFICATION - For Bidding purposes, Elevation "E" on the Standpipe Piezometer Schedule shall be El. 478.0'. Piezometers are not nested and will be single standpipe piezometer in each drill hole.

Pennsylvania Department of General Services
Lebanon County, Pennsylvania

Marquette Lake Dam
(DEP ID No. 38-078)
Pre-Final Hydrologic and Hydraulic Design Report

March 2021

Prepared By:

Gannett Fleming

207 Senate Avenue
Camp Hill, PA 17011

PENNSYLVANIA DEPARTMENT OF GENERAL SERVICES

MARQUETTE LAKE DAM
(DEP ID NO. 38-078)
PRE-FINAL HYROLOGIC AND HYDRAULIC ANALYSIS REPORT

March 2021

Prepared For:

D'Appolonia Engineering Division of Ground Technology, Inc.
701 Rodi Road, Floor 2
Pittsburgh, PA 15235-4559

Prepared By:

GANNETT FLEMING, INC.
207 SENATE AVENUE
CAMP HILL, PA 17011
Phone Number: (717) 763-7212

Gannett Fleming Job Number: 067868

EXECUTIVE SUMMARY

Marquette Lake Dam (DEP ID No. 38-078) is situated on Indiantown Run in East Hanover Township, Lebanon County, Pennsylvania on the lands of Indiantown Gap Military Reservation. This 27-foot high, 1,180-foot long, zoned earth embankment, Class C-1 dam was constructed in the early 1940's in conjunction with the wartime expansion of the military complex. The facility is owned and operated by the Pennsylvania Department of Military and Veterans Affairs (DMVA).

On January 7, 2011, the DMVA received a letter from PA DEP Division of Dam Safety stating that repairs were needed due to multiple deficiencies, including inadequate spillway capacity.

A Spillway Capacity and Incremental Dam Breach Analysis for Marquette Lake Dam, prepared by Greenman-Pedersen, Inc., dated April 24, 2014, was completed and showed that the Spillway Design Flood (SDF) was the Probable Maximum Flood (PMF). This study also determined that the existing principal spillway only passed approximately 50% of the PMF and therefore did not have adequate capacity and would likely fail during the SDF. In the Fall 2018/Spring 2019, Pennsylvania released new direction on development of the Probable Maximum Storm (PMS) in the Commonwealth, including updated precipitation depths and distributions.

A new, independent hydrologic model was created using the new PA PMP study data as well as updated estimates of hydrologic parameters. Compared to the previous hydrologic model, the watershed area remained unchanged and the hydrologic losses and lag time increased. The PMP 24-hour depth decreased from 26.5 inches to 25.89 inches, and the PMF inflow estimate decreased to 18,529 cubic feet per second (cfs); a 36% reduction compared to the 2014 Greenman-Peterson estimate of 29,002 cfs. An independent evaluation of spillway discharge capacity confirmed that the dam cannot pass the SDF of the PMF without overtopping the dam embankment. The analyses show that during the PMF the dam will be overtopped for 1.5 hours by a maximum depth of 1.3 feet.

The outlet works were assessed to determine their capacity to draw down the top 2 feet of the reservoir, assuming an inflow of 70% of the areally-adjusted maximum mean monthly flow (13.5 cfs). The proposed outlet works can lower the pool 2.06 feet in 2 hours and meets PADEP criteria.

MARQUETTE LAKE DAM REHABILITATION
HYDROLOGIC AND HYDRAULIC DESIGN REPORT
TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

APPENDICES

1. PROJECT PURPOSE AND BACKGROUND

Marquette Lake Dam (DEP ID No. 38-078) is situated on Indiantown Run in East Hanover Township, Lebanon County, Pennsylvania on the lands of Indiantown Gap Military Reservation. The dam is a 27-foot high, 1,180-foot long, zoned earth embankment with a 100-foot long concrete ogee principal spillway which includes an access bridge that spans the spillway. An intake tower and associated low-level outlet conduit allow draining of the reservoir. The reservoir formed by the dam is approximately one mile in length, has a surface area of approximately 16 acres at normal pool and impounds approximately 90 acre-feet of water at normal pool. The contributing drainage area to the reservoir is approximately 5.9 square miles. The size classification of the dam is Category C, as the impoundment storage is less than 1,000 acre-feet and the dam is less than 40 feet high. The dam is a Hazard Potential Category 1 structure due to development immediately downstream of the dam and along Indiantown Run.

Marquette Lake Dam was constructed in the early 1940's in conjunction with the wartime expansion of the military complex. The facility is owned and operated by the Pennsylvania Department of Military and Veterans Affairs (DMVA). On January 7, 2011, the DMVA received a letter from PA DEP Division of Dam Safety stating that repairs were needed due to multiple deficiencies, including inadequate spillway capacity.

As a Hazard Potential Category 1 structure, the spillway design flood (SDF) is established by an incremental damage assessment including floods up to the Probable Maximum Flood (PMF). A Spillway Capacity and Incremental Dam Breach Analysis for Marquette Lake Dam, prepared by Greenman-Pedersen, Inc., dated April 24, 2014, showed that the SDF was the Probable Maximum Flood (PMF). This study also determined that the existing 100-foot ogee spillway only passed approximately 50% of the PMF and therefore did not have adequate capacity and would likely fail by overtopping during the SDF.

In the Fall 2018/Spring 2019, Pennsylvania released new direction on development of the Probable Maximum Storm (PMS) in the Commonwealth, including updated precipitation depths and distributions. Gannett Fleming was commissioned by D'Appolonia Engineering Division of Ground Technology, Inc. to provide professional engineering services to determine the updated PMF to establish the design flood for the spillway, and to complete hydraulic analysis of the low-level outlet works to verify adequate draw-down capacity in conformance with PA Code § 105.96. Outlet works. This submission is part of DGS Project No. C-0960-0086 Ph001 for the construction of Marquette Lake Dam – Improvements to Marquette Lake Dam, Task 1, Programming Design Submission, Tasks a & b.

This study documents the updated hydrologic analyses using the PA PMP precipitation depths and distributions and updated hydrologic parameters (watershed delineations, precipitation loss, watershed lag time, and basin routing calculations). This study also documents analyses to evaluate reservoir drawdown capacity for conformance with PA Code.

2. HYDROLOGIC MODELING AND RESERVOIR ROUTING

The Greenman-Pedersen Marquette Lake 2014 HEC-1 hydrologic model used Hydrometeorological Report 51 (HMR 51) precipitation depths and United States Army Corps EM 1110-2-1411 temporal distributions. As part of the current study, Gannett Fleming used GIS with current survey, LiDAR terrain data, land cover data, and soil data to develop an independent hydrologic model for Marquette Lake Watershed and Reservoir using the USACE Hydrologic Engineering Center (HEC) Hydrologic Modeling System (HEC-HMS) software. PMP estimates were calculated using the PADEP PMP ArcGIS Tool and Distribution Spreadsheet. All elevations referenced in this report are in the North American Vertical Datum of 1988 (NAVD88).

2.1. Watershed Delineation

Gannett Fleming used a 2008, 1-meter digital elevation model (DEM) obtained from the PAMAP program by PA Department of Conservation and Natural Resources (DCNR), along with the ArcGIS Desktop Suite's Spatial Analyst Toolbox, to delineate a watershed boundary for Marquette Lake. Due to the watershed size and homogeneity, no subwatersheds were delineated. A comparison of the computed drainage area and the previous Greenman-Peterson study's subwatersheds are provided in Table 2.1 Figure 2.1 depicts the watershed delineation.

Compared to the 2014 Greenman-Peterson delineation (Figure 2-2), the sum of the previous study's watersheds compares well to the current study area of 5.88 square miles (see Table 2.1 for comparison).

Table 2.1 – Drainage Area

Subwatershed	Current Study Drainage Area (mi^2)	Greenman-Peterson 2014 Drainage Area (mi^2)
IGAP	-	0.127
PGAP	-	5.334
IMARQ	-	0.0281
PMARQ	-	0.380
Overall Watershed DA	**5.88**	**5.87**

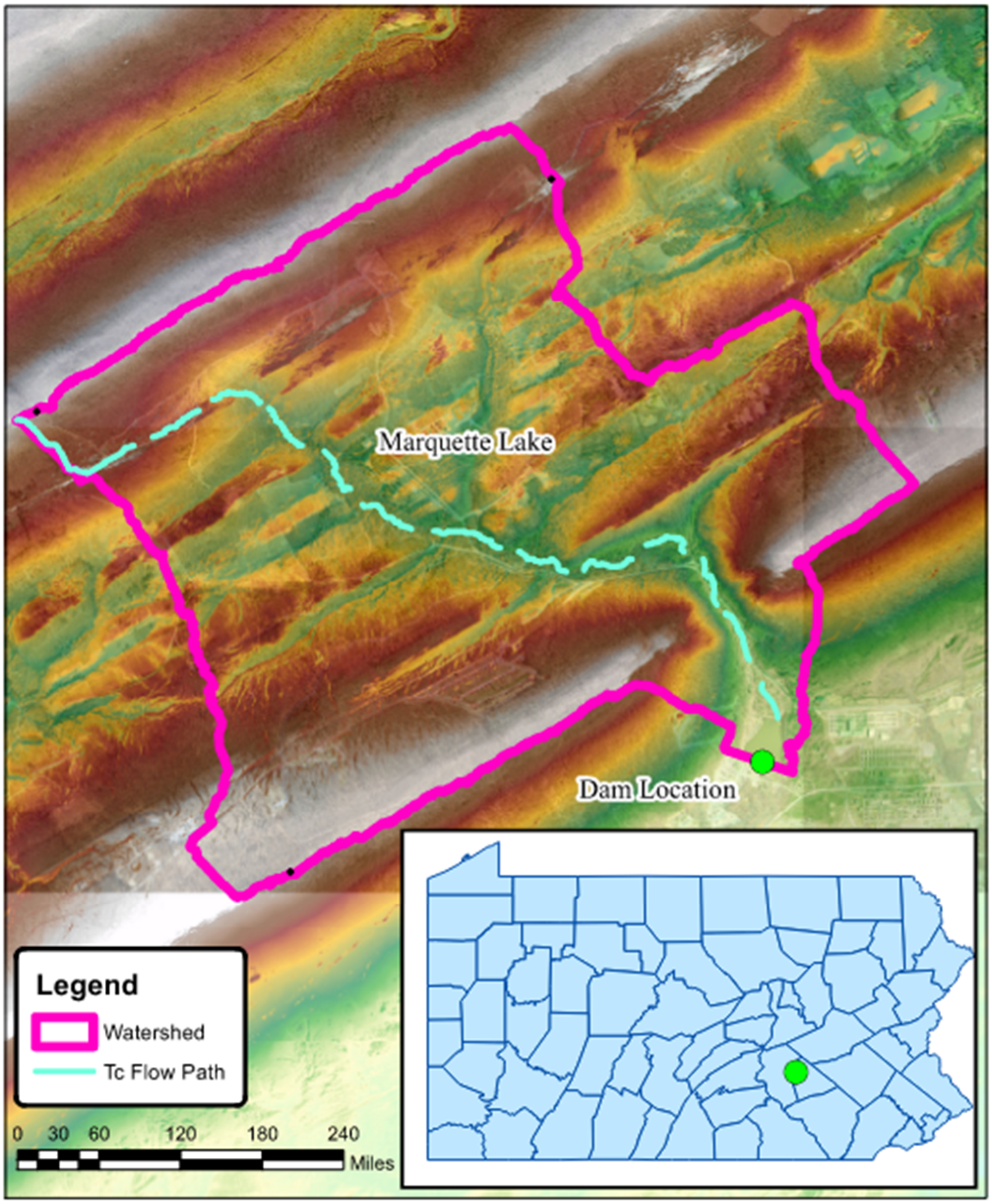


Figure 2-1– Current Study Marquette Lake Dam Watershed

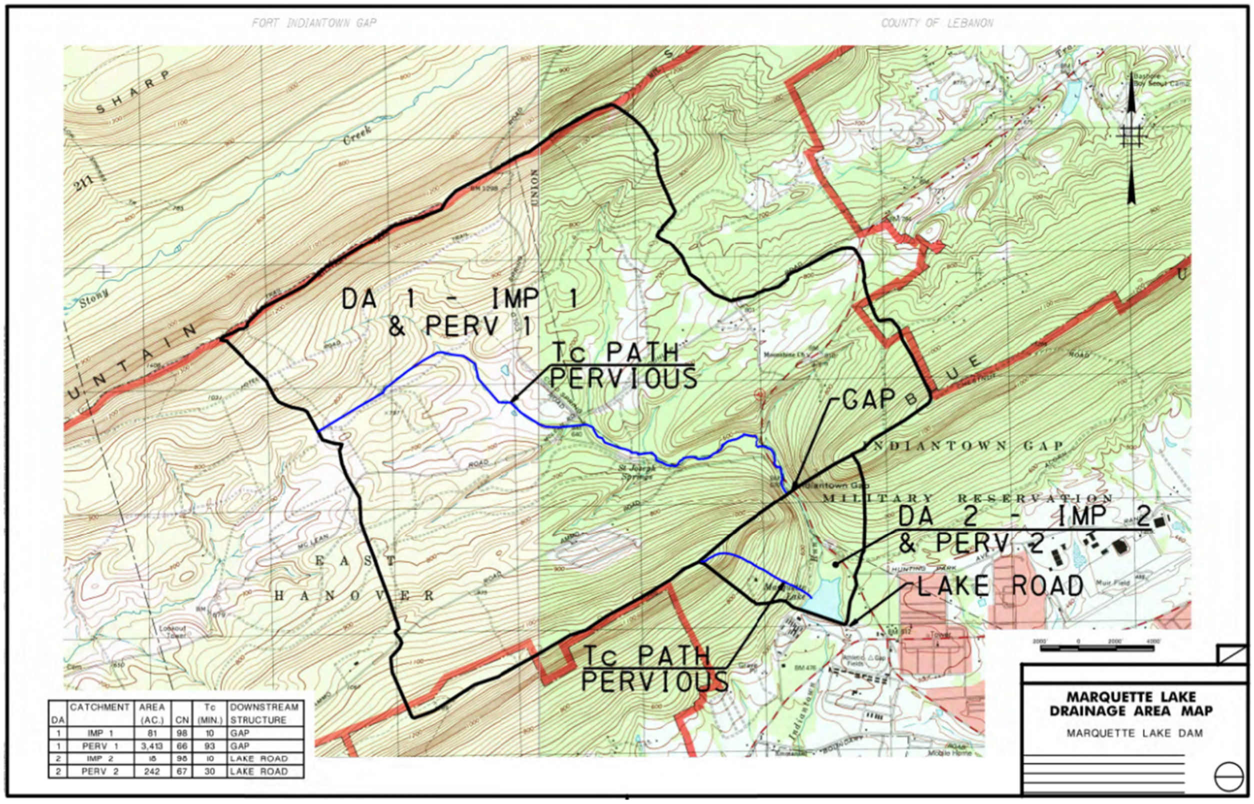


Figure 2-2– 2014 Greenman-Peterson Marquette Lake Dam Subwatersheds

2.2. <u>Hydrologic Loss Method</u>

The SCS Runoff Curve Number (RCN) method as presented in the Natural Resources Conservation Service (NRCS) *National Engineering Handbook* (*NEH*) *Chapter 10: Estimation of Direct Runoff from Storm Rainfall* was used to estimate direct runoff from each storm. The RCN is a runoff coefficient that relates runoff potential to land cover and hydrologic soil group (HSG) type. RCN values for combinations of HSG and land cover have been published in the NEH. The major factors in estimating the RCN are HSG, land cover type, and hydrologic condition. The HSGs are classified into four classes (A, B, C, and D) based on the soil's runoff potential, with HSG "A" having the smallest runoff potential and HSG "D" having the greatest.

The RCN for the Marquette Lake watershed was calculated within a GIS environment using soil and land cover data in conjunction with the delineated watershed. Land cover data were obtained from the Multi-Resolution Land Characteristics (MRLC) Consortium's National Land Cover Database (NLCD) 2016 dataset. Soil data were obtained from a 2019 study published in the NRCS Web Soil Survey Geographic Database (SSURGO). The NLCD and NRCS data were then merged using GIS for each subwatershed to relate HSG and land cover. The RCN was then calculated using the merged HSG and land cover feature as discussed in *NEH Chapter 9: Hydrologic Soil-Cover Complexes*. The RCN is given in Table 2.2; soil, land cover, and RCN maps are given in Appendix A.

In Greenman-Peterson's model, hydrologic losses were also estimated using the RCN method. Differences were noted between the Greenman-Peterson RCN and GF RCN and are reported in Table 2.2.

Table 2.2– Loss Rate

Subwatershed	**Current Study RCN**	**Greenman-Peterson 2014 RCN**
IGAP	-	98
PGAP	-	66
IMARQ	-	98
PMARQ	-	67
Overall Watershed Weighted	**58**	**67**

The difference between the current study's RCN and the Greenman-Peterson RCN is attributed to NRCS updates to published Hydrologic Soil Groups (HSGs). Major modifications to the NRCS methods for assigning map unit components to Hydrologic Soil Groups were published in Chapter 7 of the National Engineering Handbook Part 630 in 2007 and 2009.

Since the updated methods were put into effect, updated soils data have been published in many areas. This data is available for download via the Web Soil Survey. For Dauphin and Lebanon Counties, Web Soil Survey data was last updated in September 2019. According to the NRCS, no further changes or updates to Hydrologic Soil Group assignments or assessments are planned.

The Greenman Peterson RCNs were generated prior to updates to the soil data in 2019. The RCNs computed as part of this study are based on best-available land cover and soils data and are considered to be the best estimates of the Marquette Lake watershed's hydrologic losses. Although the newly computed RCN varies significantly from the prior study, tor large precipitation events, such as the PMF, the inflow becomes less sensitive to the RCN due to the relatively small ratio of initial abstraction and losses to the total runoff. A comparison between previous and current computed Marquette Lake runoff volumes is given in Section 2.7.

2.3. Hydrologic Transform Method

To convert excess precipitation into runoff, the SCS Unit Hydrograph Transform Method was used. This method requires a calculated lag time (L), defined as the time between the centroid of rainfall distribution and peak runoff, which is estimated as 60% of the time of concentration (T_c). T_c is defined as the time required for the hydraulically most distant runoff to reach the watershed outlet. To calculate T_c and then L, the segmental velocity approach was applied using GIS flow path tracing tools. This methodology is outlined in *NEH Chapter 15: Time of*

Concentration and directly calculates the average velocity, in three separate flow segments: sheet, shallow concentrated, and channelized flow.

Sheet flow velocity was estimated based on slope and Manning's roughness coefficient for a total length not exceeding 100 feet where it is assumed to transition to shallow concentrated type flow regime. Shallow concentrated flow computations were applied from this point to where the topography becomes channelized. For the remainder of the stream course, open channel hydraulics was used to compute flow velocities and travel times. The total T_c is the summation of the travel times for each individual segment. The current study and Greenman-Peterson 2014 Study time of concentration and lag times are presented in

Table 2.3. The longest flow path is presented in Figure 2-1 and calculations used for each flow segment are presented in Appendix B.

Table 2.3 – Unit Hydrograph Parameters

	Current Study		Greenman-Peterson 2014 Study	
Subwatershed	**T_c (hr)**	***Lag* (hr)**	**L_c (mi)**	**T_c (hr)**
IGAP	-	-	0.167	0.1
PGAP	-	-	1.55	0.93
IMARQ	-	-	0.167	0.1
PMARQ	-	-	0.5	0.3
Overall Watershed	2.33	1.4	-	-

Comparing the current study to the 2014 Study, the individual subwatershed times of concentration appear to be substantially less than the current study. Because the 2014 study broke the dam's watershed into multiple subwatersheds, there is not direct comparison of times, however the 2014 'PGAP' subwatershed constitutes the largest subwatershed and is approximately 67% of the current study's lag time. The Greenman-Peterson shallow concentrated flow land covers were limited to 'paved' or 'unpaved', which did not account for the different land covers that are present at the site. The updated expanded land covers, which result in longer travel times, account for some of the differences in the lag time between the two studies.

2.4. Precipitation

Precipitation data from the PADEP PMP tool and the National Oceanic and Atmospheric Administration (NOAA) Atlas 14 Point Precipitation Frequency Estimates were applied to each subwatershed to develop inflow hydrographs.

2.4.1. Probable Maximum Precipitation

Gannett Fleming used PADEP's new ArcGIS PMP Tool and associated spreadsheet to develop PMP depth-duration estimates and temporal distributions. The depth-duration data for Marquette Lake Watershed is shown in Table 2.4; values in **bold** represent the maximum depth for a given duration. Temporal distributions for the storm events considered, as directed by PADEP, are given in Table 2.5 and Appendix C. The depths represent a basin average of gridded rainfall data which is applied uniformly over the entire drainage area. For comparison, the 24-hour HMR 51 PMP index used in the 2014 study was 26.5 inches. Compared to the HMR 51 precipitation depth, the Local 24-hour depth of 25.89 inches is a decrease in precipitation of 2%.

Table 2.4 –PMP Depth-Duration (controlling storms bolded)

Storm	Duration and Depth (inches)							
	1-hr	2-hr	3-hr	4-hr	5-hr	6-hr	12-hr	24-hr
Local	**10.20**	**11.70**	**13.20**	**14.47**	**16.90**	**22.97**	**25.00***	**25.89**
Tropical	9.16	9.16	11.40	12.16	14.16	16.52	**25.66***	25.66
General	3.60	5.86	9.89	9.89	9.89	14.00	16.22	16.42

*See discussion below for discussion of controlling storm

Table 2.5 – Storm-Specific Temporal Distributions

Storm Duration	Controlling Temporal Distribution
3 hr Storm Specific	1547_1
6 hr Storm Specific	1406_1
12 hr Storm Specific	1491_1*
24 hr Storm Specific	1406_1
2 hr Synthetic	-

In general, the Local storm distributions showed a larger total precipitation depth for the various durations, except for the 12-hour duration. For the 12-hour duration, the tropical storm resulted in a greater precipitation depth, and both local and tropical storms were run in HEC-HMS to determine which event produced the highest reservoir elevation.

2.4.2. Frequency Storms

Precipitation data for the 2-, 10-, 50-, 100-, 200-, and 500-year storm events were obtained from the NOAA Precipitation Frequency Data Server (PFDS) interface developed to facilitate use of NOAA Atlas 14 precipitation frequency estimates. The NOAA Atlas 14 estimates are shown in Appendix D. The depth-duration data are reported in Table 2.6.

Table 2.6 – Frequency Storm Data from NOAA Atlas 14

Duration	5 min	15 min	1 hr	2 hr	3 hr	6 hr	12 hr	24 hr
2-year Event	0.404	0.797	1.36	1.60	1.76	2.18	2.68	3.08
10-year Event	0.520	1.03	1.90	2.31	2.55	3.16	3.93	4.56
50-year Event	0.623	1.23	2.43	3.13	3.48	4.37	5.55	6.56
100-year Event	0.668	1.31	2.67	3.53	3.95	4.99	6.40	7.63
200-year Event	0.708	1.38	2.91	3.97	4.46	5.68	7.36	8.87
500-year Event	0.762	1.48	3.24	4.61	5.21	6.71	8.84	10.80

2.5. Stage-Storage and Stage-Discharge Relationships

The stage-storage relationship reported in the 2014 Greenman Peterson study was used in the current study. Their study states that the pond volume was computed from topographic mapping. Because no bathymetric data is available, the reasonableness of these values could not be assessed. The stage-storage relationship used in the model is reported in Table 2.6

Stage-discharge relationships for the dam were also developed in the 2014 Greenman-Peterson study. An independent analysis of the principal ogee spillway discharge relationship was conducted as part of this study.

The discharge rating curve for Marquette uncontrolled ogee spillway structure was developed using a custom excel spreadsheet which is based upon calculations outlined in the United States Bureau of Reclamation Design of Small Dams. The spillway is comprised of 3 uncontrolled ogee crest sections separated by pointed-nose bridge piers (see Figure 2.2-3). The total active length of the weir is 98.9 feet, which accounts for the ineffective areas of the piers. The bottom chord elevation of the bridge (518.75 feet) is slightly lower than the lowest top of dam elevation (519.23 feet).

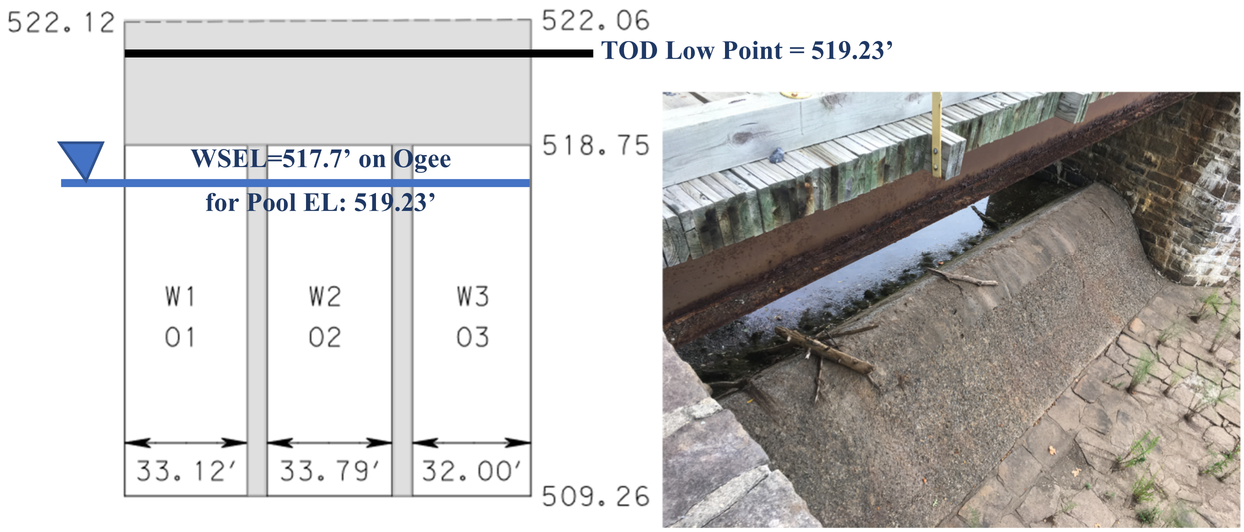


Figure 2.2-3 – Elevation & Isometric Views of the Uncontrolled Ogee Crest

As water passes over the ogee weir crest, the water surface profile will contract and the water surface elevation directly over the ogee crest is expected to be somewhat lower than the nominal reservoir elevation as depicted in Figure 2.2-4. The expected drawdown for the maximum available head (±10 feet) is approximately 1.5 feet. Therefore, for a pool elevation of 519.23 feet, the expected water surface elevation upstream of the ogee crest will be approximately 517.7 feet (see Figure 2.2-3), approximately one foot below the low chord of the spillway bridge. Therefore, for all flows approaching overtopping, interference in weir flow over the ogee weir is not expected. For pool elevations exceeding the top of dam, the 1,000+-foot-long dam crest would act as a weir, conveying flow that would quickly dwarf the ogee weir discharge. For the purposes of this study, potential interference by the bridge low chord and superstructure was not considered.

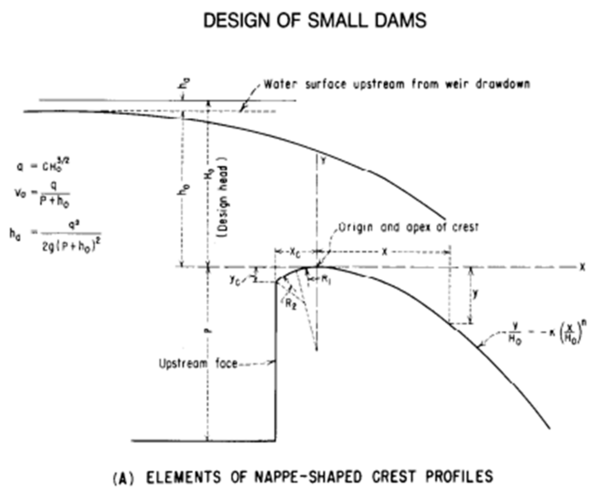


Figure 2.2-4 – Variables Used in Determining Ogee Crest Geometry

No detailed cross-sectional geometric data of the ogee shape was available at the time of this report, so the approximate geometry of the spillway was estimated from site photos. Based on a design head (Ho) of 5.5 feet, Figure 2.2-5 depicts both the estimated actual weir shape, and the ogee shape predicted by the compound curve ogee equation. As can be seen, the calculated ogee profile based upon 5.5 feet of design head closely follows the estimated actual existing weir profile. For development of the spillway discharge rating curve, a design head of 5.5 feet was assumed. Approach depth was estimated to be 4 feet.

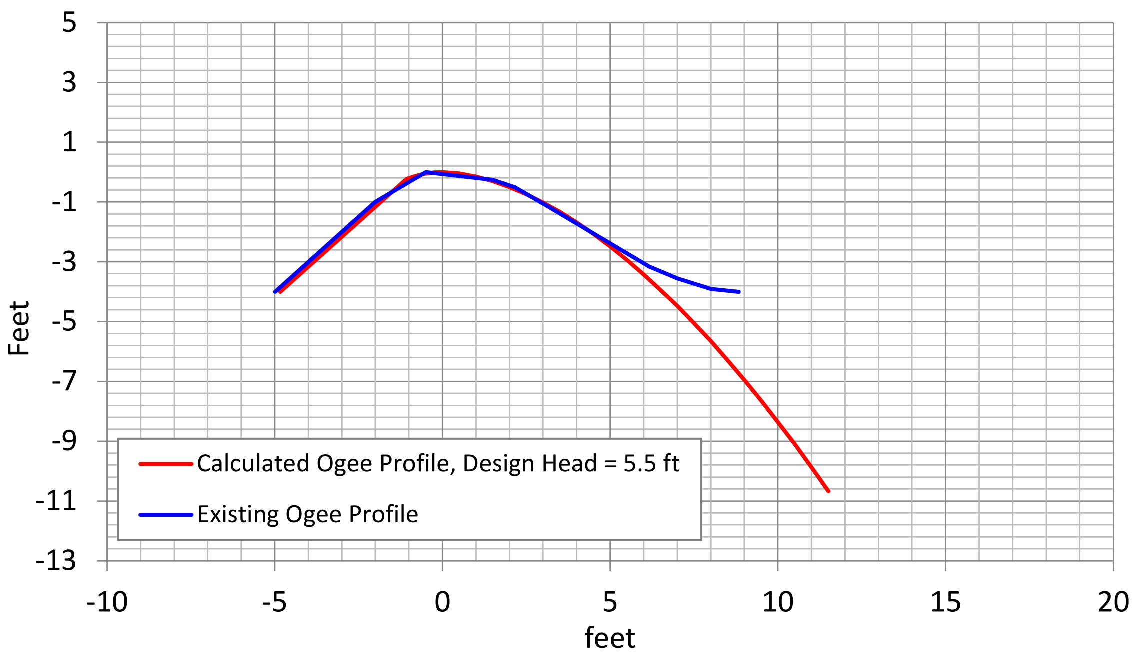


Figure 2.2-5 – Comparison of Calculated Ogee Profile to Existing Ogee Profile

Assuming that the water surface profile does not impact the low chord of the bridge, for water surface elevations exceeding 519.23 feet, contribution to discharge from dam overtopping was estimated using the broad-crested weir equation with a discharge coefficient of 2.6 and an effective length of 1,000 feet.

The total discharge rating curve for the principal ogee spillway and overtopping discharge is given in Table 2.7 and Figure 2-6. More information regarding dam hydraulics is given in Section 3.2.

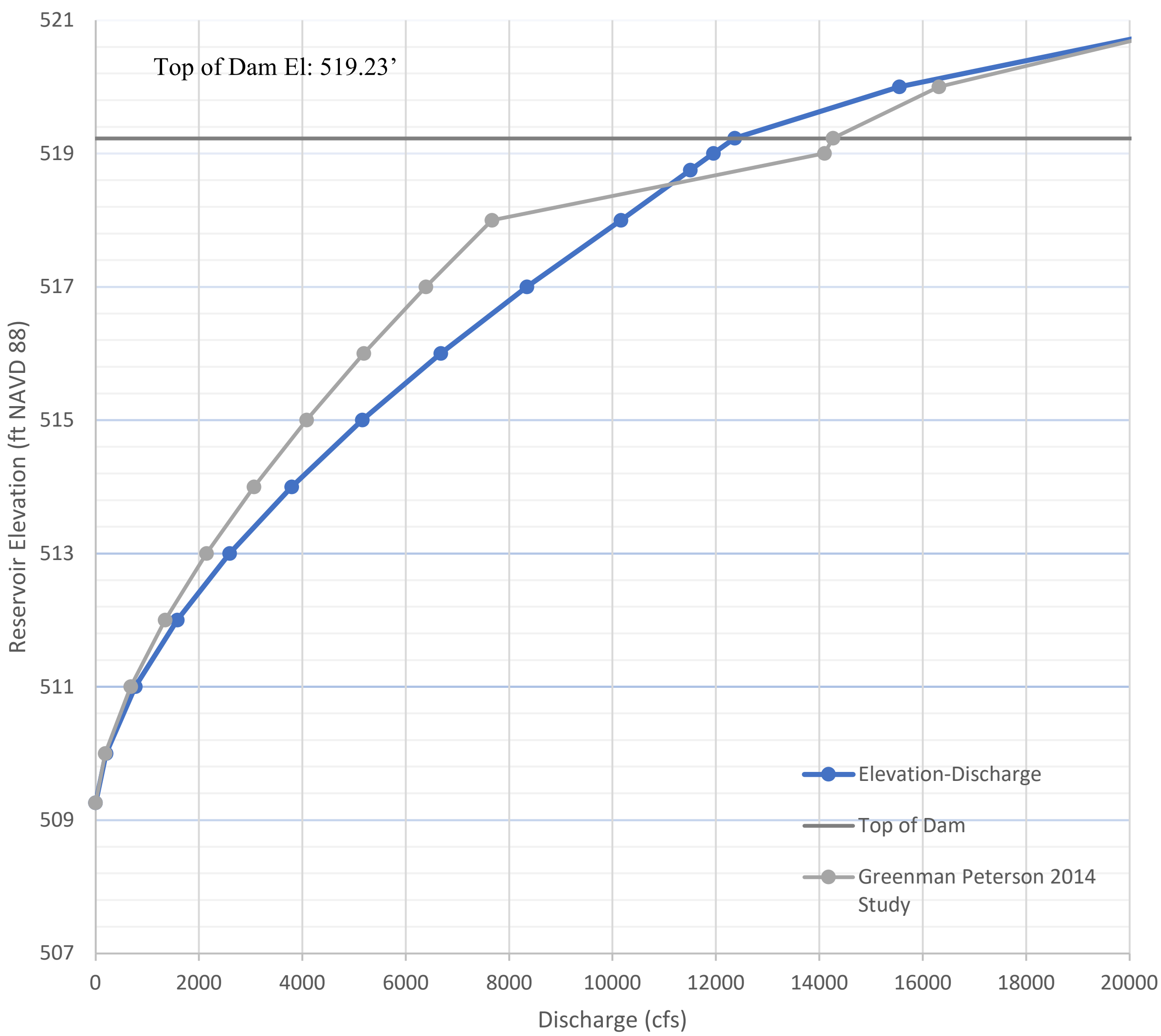


Figure 2-6 – Marquette Lake Dam Total Discharge Rating Curve

Table 2.7 – Marquette Lake Stage-Storage-Discharge Relationship

Stage (feet NAVD 88)	Ogee Discharge (cfs)	Overtopping Discharge (cfs)	Ogee & Overtopping Discharge (cfs)***	Storage (Acre-feet)
492.00	-	-	-	0.00
493.00	-	-	-	5.24
494.00	-	-	-	10.48
495.00	-	-	-	15.72
496.00	-	-	-	20.96
497.00	-	-	-	26.20
498.00	-	-	-	31.44
499.00	-	-	-	36.68
500.00	-	-	-	41.92
501.00	-	-	-	47.16
502.00	-	-	-	52.40
503.00	-	-	-	57.64
504.00	-	-	-	62.88
505.00	-	-	-	68.12
506.00	-	-	-	73.36
507.00	-	-	-	78.60
508.00	-	-	-	83.84
509.00	-	-	-	89.08
509.26*	0	-	0	90.44
510.00	204	-	204	102.36
511.00	770	-	770	119.83
512.00	1580	-	1580	137.30
513.00	2596	-	2596	157.09
514.00	3795	-	3795	176.88
515.00	5161	-	5161	199.35
516.00	6681	-	6681	221.82
517.00	8346	-	8346	246.81
518.00	10166	-	10166	271.80
518.75	11505	-	11505	292.40
519.00	11950		11950	299.27
519.23**	12364	0	12364	305.59
520.00	13779	1766	15545	326.74
521.00	15681	6129	21801	356.34
522.00	17651	11991	28392	385.94
523.00	19685	19036	36068	417.16
524.00	21779	27090	44731	448.37
525.00	23929	36039	54269	481.22
526.00	26132	45802	64602	514.07
527.00	28385	56315	75669	548.57
528.00	30684	67529	87420	583.08
529.00	33027	79402	99817	619.25
530.00	35412	91899	112824	655.42

*Crest of Principal spillway Ogee
**Low point on Top of Dam
***Influence of spillway bridge beams on ogee hydraulics were not evaluated

The Greenman-Peterson Study accounted for 3 separate Ogee weir sections of equivalent length but used a constant weir coefficient of 3.0 for all heads. At the point where the low chord elevation of the bridge intersected the reservoir elevation, the discharge calculation was shifted to the orifice equation with an orifice coefficient of 0.6. This method does not account for the drawdown of the pool in the vicinity of the dam, incorrectly forces orifice flow at the reservoir low chord elevation which causes a sudden increase in discharge capacity, and therefore underestimates the ogee efficiency.

2.6. Hydrologic Model Results

Inflow and outflow hydrographs were created using the USACE HEC-HMS version 4.6.1 computer software and the watershed and reservoir routing parameters documented previously. At the beginning of the simulation, Marquette Lake was assumed to start at normal pool level for each flood scenario. No reach routing was necessary as the hydrologic model contained only one watershed.

2.6.1. PMP Analysis

Based on the guidance in the 2019 PA PMP study, the 3, 6, 12, and 24-hour Storm Specific storms, and 2-hour synthetic storms must all be evaluated to determine the controlling Probable Maximum Storm (PMS). The PMP storm duration that results in the highest discharge at Marquette Lake is then considered to be the PMF. Peak PMF inflows and outflows for the various PMSs at Marquette Lake are summarized in Table 2.8. Although the 12-hour Tropical storm had the greatest depth for that duration, due to the storm intensity (controlled by the required temporal distribution) the 12-hour local storm resulted in a larger outflow. Overall, the 24-hr Local PMS event governed as the PMF. Outflow is nearly equivalent to inflow for all events modeled; the reservoir provides little attenuation. The inflow, outflow, and stage hydrographs for the PMF are given in Figure 2-7.

Table 2.8 – PMS Results

PMP Duration and Distribution	Inflow (cfs)	Outflow (cfs)	Maximum Water Surface Elevation (feet NAVD 88)
2-hour-synthetic	11,010	10,728	518.3’
Local 3-hour	12,266	11,981	519.0’
Local 6-hour	17,951	17,914	520.4’
Tropical 12-hour	7,516	7,490	516.5’
Local 12-hour	18,393	18,341	520.4’
Local 24-hour	**18,529**	**18,475**	**520.5’**

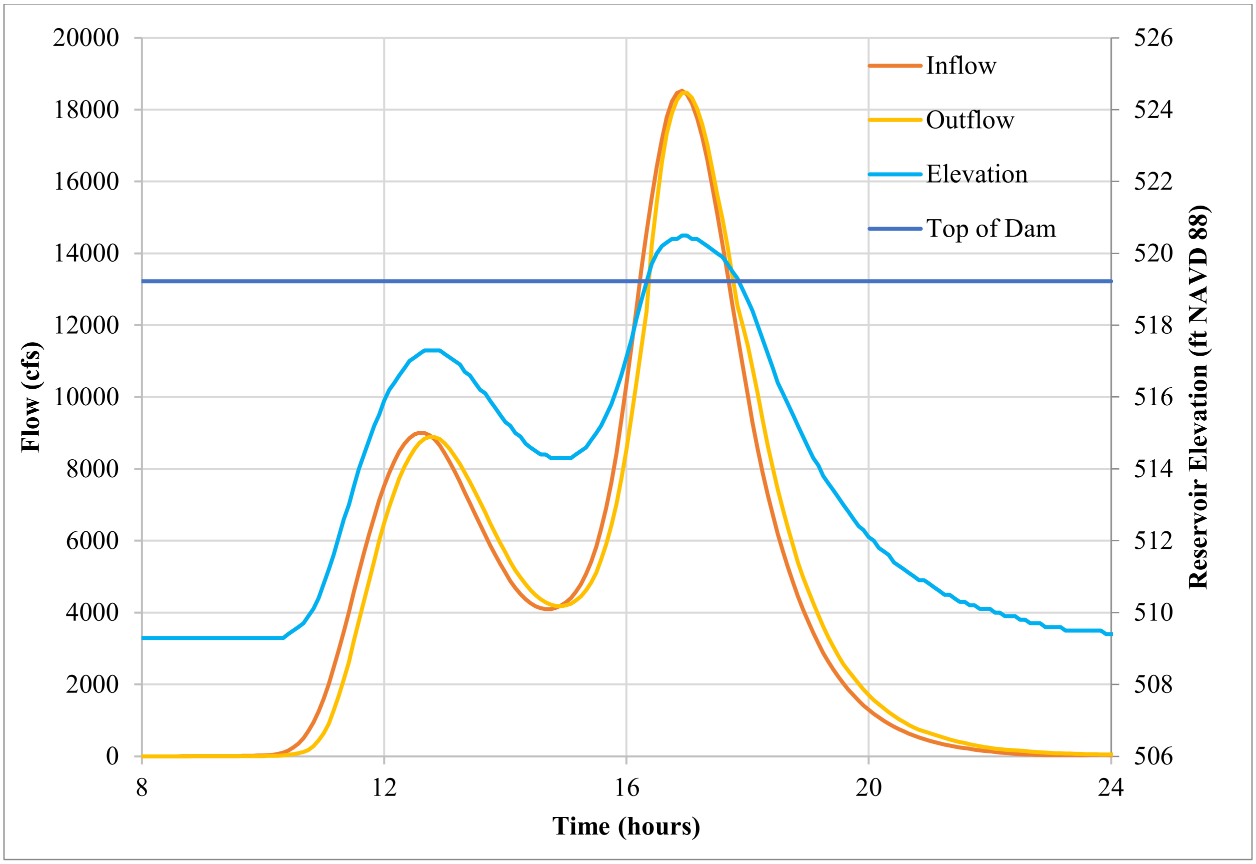


Figure 2-7 – PMF Inflow, Outflow and Elevation

The results show that the dam overtops by a maximum of 1.3 feet during the PMF for a duration of 1.5 hours, confirming that Marquette Lake Dam does not have adequate spillway capacity. Comparison with results from t the Greenman-Peterson study are provided in Table 2.10.

Table 2.9 – Marquette Lake PMF Results & Comparison with Previous Study

PMF Results	Current Study	Greenman-Peterson 2014 Study
PMF Inflow (cfs)	18,529	29,002
PMF Outflow (cfs)	18,475	28,924
Max. Reservoir Elevation (feet NAVD 88)	520.5	521.96
Max. Overtopping Depth (feet)	1.3	2.73
Overtopping Duration (hours)	1.5	2.6

The current PMF reduces the peak inflow and outflow by approximately 36% resulting in a reduction in the depth and duration of overtopping.

The difference in results between the current study and the 2014 Greenman-Peterson study is attributed to changes in watershed parameters and changes in the temporal distributions. As noted previously, the total 24-hour precipitation depth is comparable between the two studies.

The Greenman-Peterson 2014 analysis uses the HEC-1 Probable Maximum Precipitation card (PM) to apply a temporal distribution to the PMP. The report incorrectly states that "HMR52 method" is used. The PM card within HEC-1 applies the outdated Hydrometeorological Report No. 33 (HMR 33) methodology, which has been superseded by HMR 52. HMR 33 uses a United States Army Corps of Engineers' EM 1110-2-1411 temporal distribution. This temporal distribution is more intense than the current study storm distribution prescribed by PADEP. In addition to the change in temporal distribution, the current study computed a 13% increase in RCN (see Section 2.2) due to updated soil data. While this study's hydrologic model may predict a less conservative PMF compared to previous models, the inputs used are based on the latest hydrologic methodologies and Commonwealth requirements and are recommended for design of modifications to Marquette Lake Dam.

2.6.2. Recurrence-Interval Storm Analysis

Recurrence-interval storms simulations were conducted in a HEC-HMS model for the 2 through 1000-year storm events. The precipitation was uniformly distributed over the entire Marquette Lake Watershed area and distributed temporally in the HEC-HMS hydrologic model as a 24-hour duration frequency storm. Results for all frequency events are reported in

Table 2.10 (extended results are included in Appendix D). The existing dam can pass the 1000-year flood event without overtopping the embankment.

Table 2.10 – Frequency Storm Results

Storm Event	Current Study Peak Inflow (cfs)	StreamStats Regression Peak Flow (cfs)	Peak WSEL (feet NAVD 88)
2-year	196	286	509.9
10-year	807	726	511.0
50-year	1946	1310	512.3
100-year	2606	1620	512.9
200-year	3371	1960	513.6
500-year	4549	2490	514.5
1000-year	5571	-	515.2

USGS gaging station No. 01572950 on Indiantown Run near Harper Tavern, PA, is located just upstream of Marquette Lake. With a drainage are of 5.48 mi2, this gaging station records streamflow from 93% of the contributing area to the lake. Between 2003 and 2014 this gage recorded an instantaneous peak discharge of 2,520 cfs on September 18, 2004. The flood-frequency results presented in this study are believed to constitute conservative estimates of the peak inflows that could be expected at the site.

3. HYDRAULIC ANALYSIS

3.1. Tailwater Analysis

Tailwater was evaluated by developing a 2D hydraulic model of the reach immediately downstream of the dam using the U.S. Army Corps of Engineers HEC-RAS 5.0.7 model. A 2D analysis was selected to account for the significant out-of-bank and multi-directional flows downstream of the dam. The terrain used in the model was created using the aforementioned 1-meter DEM. The 2D flow area extends from upstream of Marquette Lake Dam downstream nearly to Memorial Lake and includes the flow constrictions at Lake Road, 17th street, and Clement Avenue. The 2D flow area generally consists of 20-foot square cells with smaller cell sizes in the vicinity of roads, embankments, and other abrupt changes in terrain to obtain more detailed hydraulic information. The model geometry is shown in Figure 3-1.

The PMF outflow hydrograph generated in the HEC-HMS model was routed into the 2D flow at a boundary condition at the downstream toe of the spillway at an energy grade slope of 0.07 based on average slope of the terrain. A normal depth boundary condition at the downstream model extent with an energy grade slope of 0.015 was used in the model, based on the average slope of the terrain.

The model used the full momentum flow equations with a 10 second computation interval for all discharges.

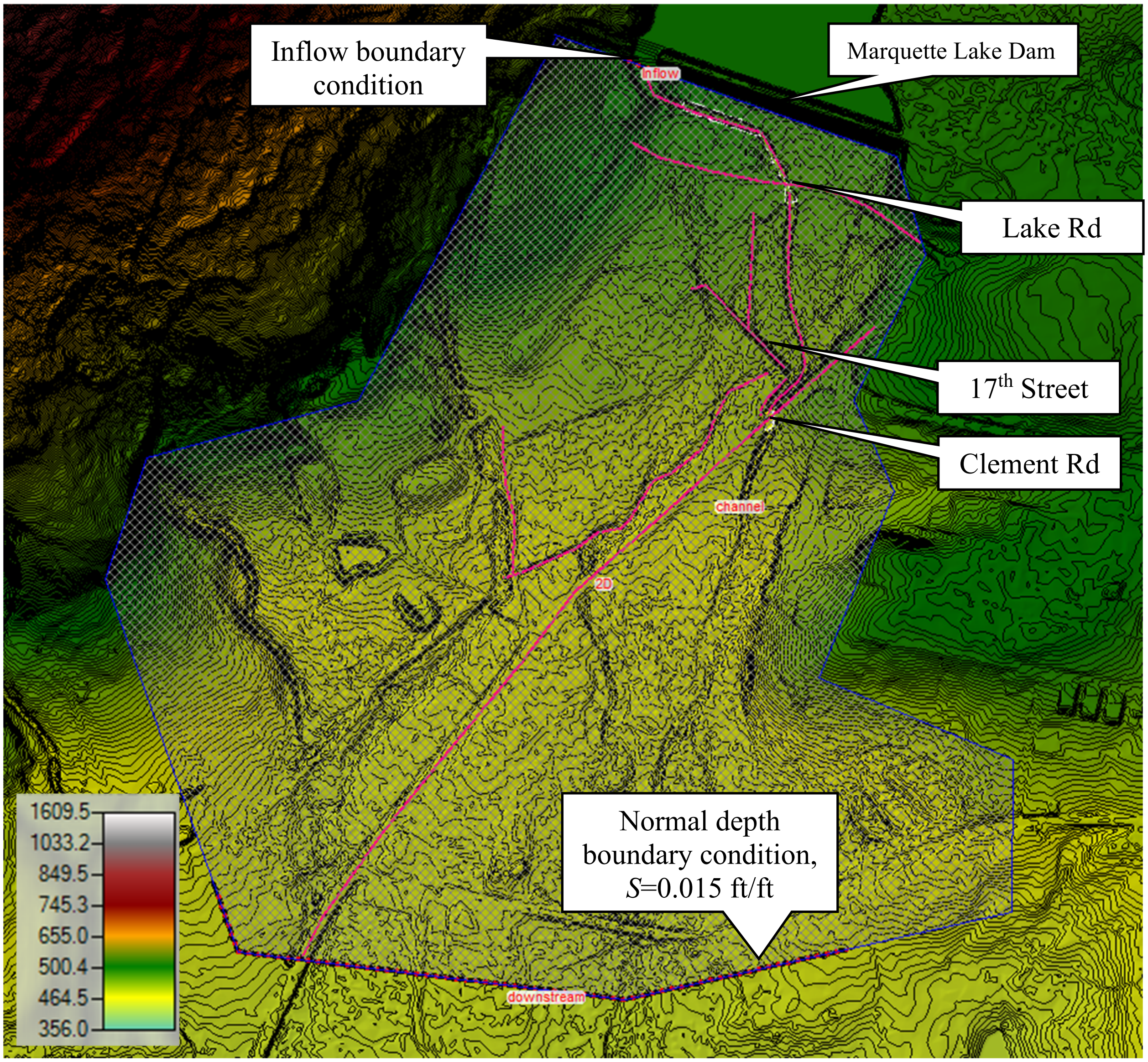


Figure 3-1 – 2D Hydraulic model geometry with 1-foot contours

There is substantial backwater present at the dam due to the topography downstream of the dam. A tailwater rating curve was developed at a location just downstream of the stilling basin to best represent water surface elevations in the downstream reach. The tailwater sampling location is shown in Figure 3-2 for water surface elevations approximating the 0.75 PMF event. This stage-discharge relationship is used in the stilling basin design calculations and is presented in Figure 3-3.

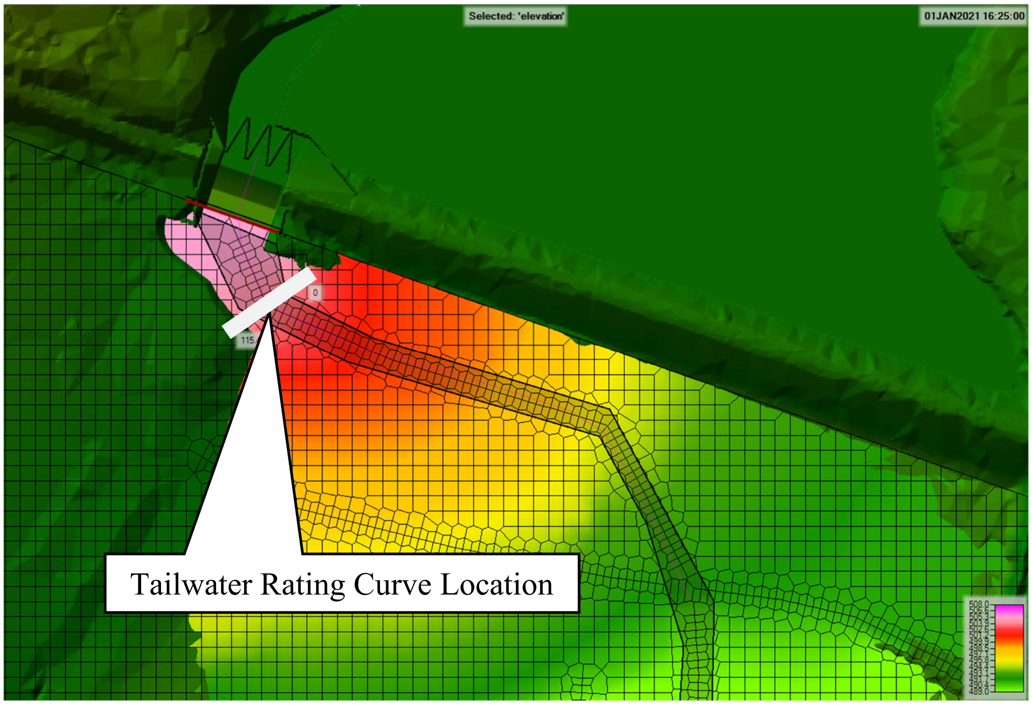


Figure 3-2 – Tailwater Rating Curve Location

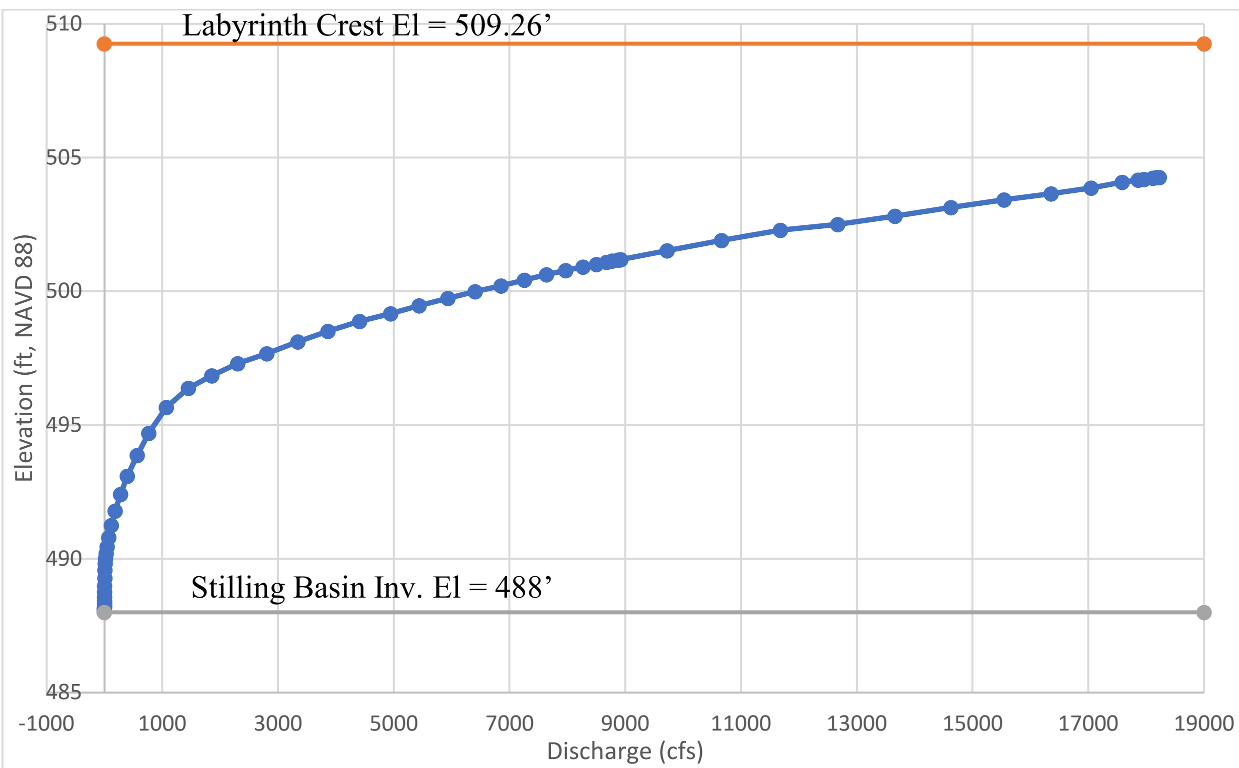


Figure 3-3 – Tailwater Rating Curve

3.2. Labyrinth Spillway Weir Design

A new labyrinth spillway has been designed in the footprint of the existing concrete ogee weir. The discharge capacity of the labyrinth weir has been optimized based on the cycle length, cycle width, weir height, crest shape, reservoir approach conditions, and bedrock location. The existing spillway weir, training walls, chute, and exit channel will all be demolished and replaced with a new, cast-in-place labyrinth spillway weir, training walls, chute, stilling basin, and exit channel. The labyrinth weir was proportioned using discharge coefficient data for a half-round crest shape, obtained from physical model studies performed at Utah State University's Utah Water Research Laboratory (Crookston and Tullis, 2012).

A 92-foot-wide by 51-foot-long, 3-cycle labyrinth spillway is proposed. An isometric view of the labyrinth spillway has been included in Figure 3-5. The existing crest elevation will be set to the same elevation as the ogee weir (509.26 feet). An approach depth of approximately 10 feet will be required to develop the full discharge capacity for a 10-foot design head. The discharge capacity of the labyrinth spillway is nearly 18,500 cfs at the maximum reservoir elevation. Once developed, the labyrinth spillway rating curve (shown in Figure 3-4) was incorporated into the HEC-HMS reservoir routing model to ensure the discharge capacity was adequate to safely pass the PMF without overtopping the dam.

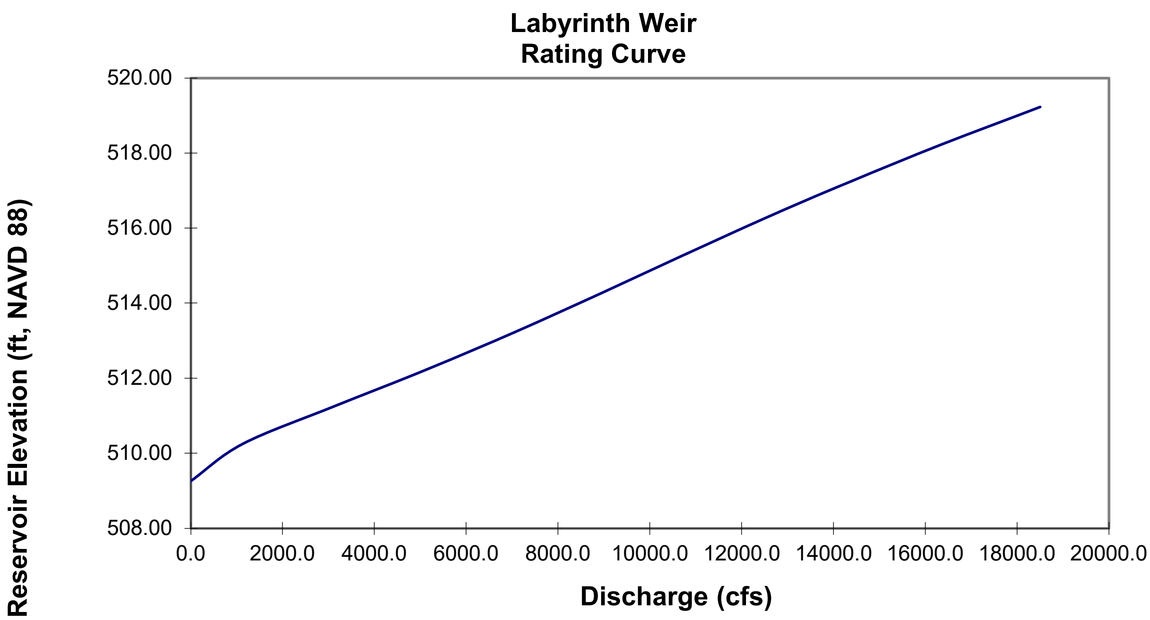


Figure 3-4 – Labyrinth Weir Rating Curve

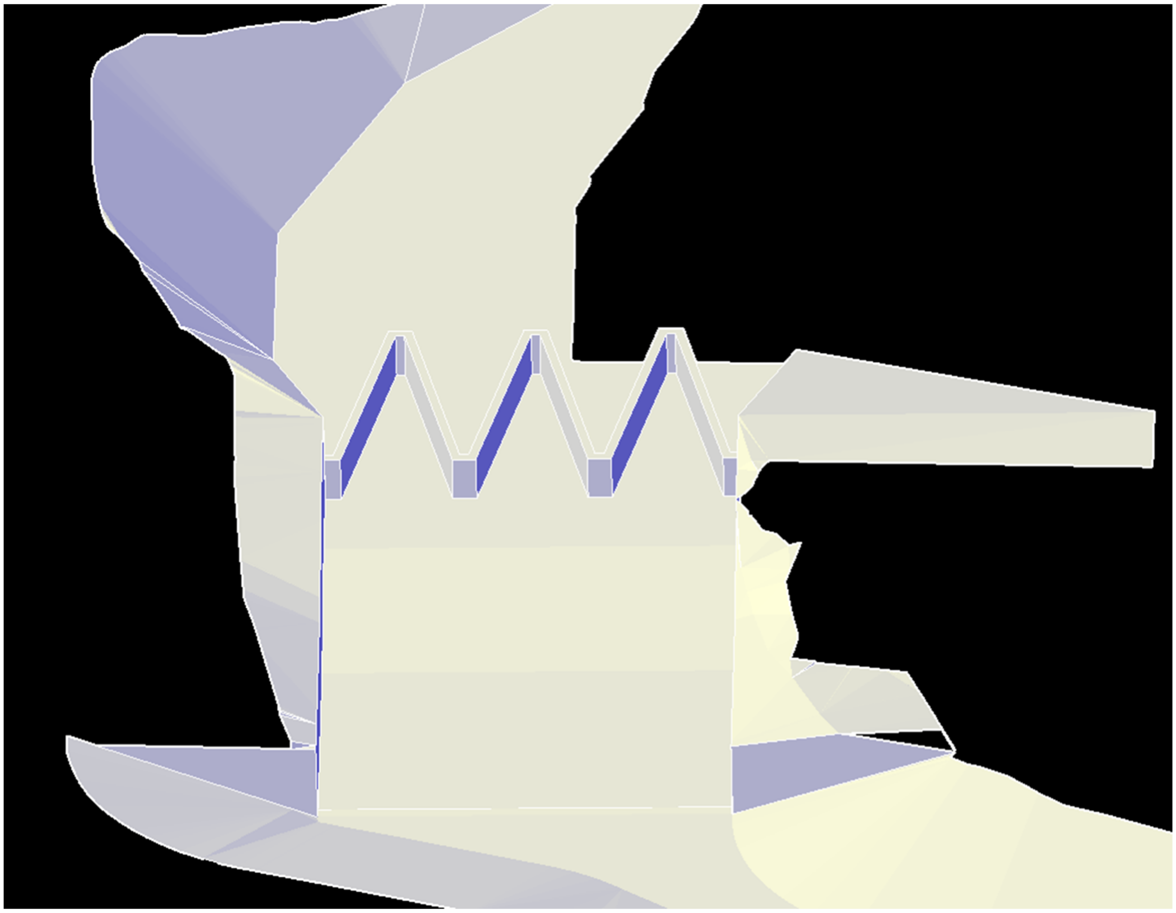


Figure 3-5 - Isometric View of Labyrinth Spillway (looking upstream)

The left labyrinth spillway training wall was placed at the same location as the existing ogee left spillway training wall. Based upon limited geotechnical investigations to date, conducted by GTS Technologies (*Subsurface Profile Drawings, March 2014*), near the left spillway training wall bedrock is at approximate elevation 505 feet. Moving 50 feet left along the centerline of the dam, the approximate rock surface elevation lowers to 479 feet; see Figure 3-6. To minimize the size and cost of the labyrinth spillway foundation, the labyrinth spillway was placed within the area of shallower bedrock. Additional geotechnical investigations to better locate the exact location of bedrock may allow the spillway to move further to the left to minimize reservoir approach grading.

At the time of this design, the labyrinth spillway was placed to accommodate a bridge across the spillway opening. The labyrinth spillway width was also minimized to serve two functions: to minimize the bridge span effectively minimizing the required girder size for the bridge, and to also minimize the amount of approach grading necessary to reduce approach energy losses to the proposed labyrinth spillway. Detailed labyrinth spillway calculations are included in Appendix E.

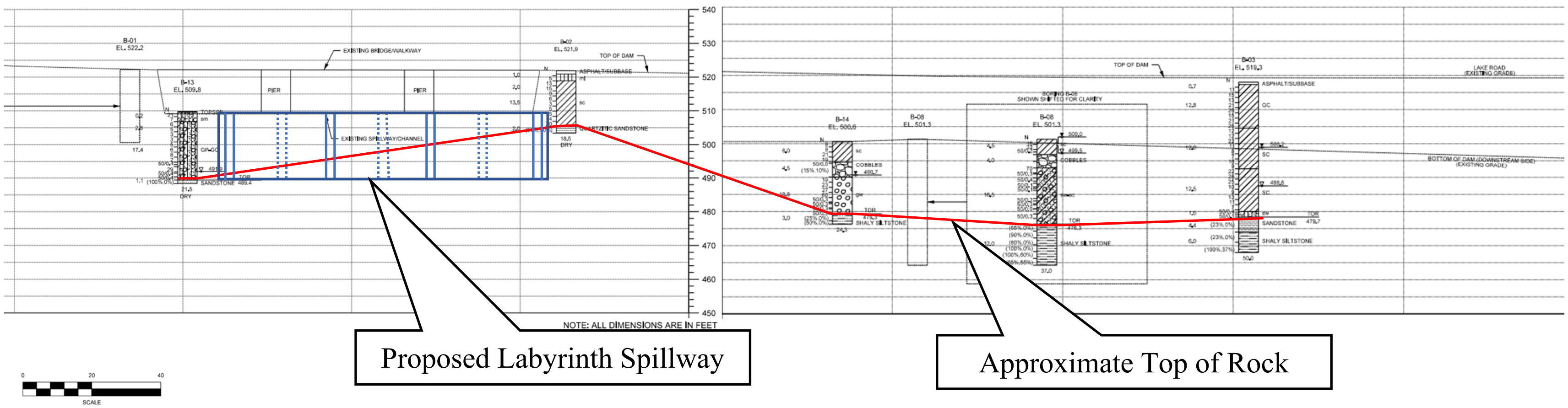


Figure 3-6 – Profile View of Approximate Top of Rock and Proposed Labyrinth Spillway Location at Dam Centerline

3.3. Spillway Approach Design

A second two-dimensional HEC-RAS model was developed to assess the approach losses and flow patterns within the reservoir near labyrinth spillway and within the spillway chute.

There is an existing knoll within the reservoir, near the right dam abutment, that could cause an impediment to free flow to the labyrinth spillway. This knoll was eliminated with proposed grading. The revised approach geometry was modeled within HEC-RAS with a two-dimensional flow area with a grid size of 5 feet. The upstream boundary condition routes in a range of flows, approximately up to the PMF, and the downstream boundary condition was set to a normal depth with an energy grade slope of 0.017 based on the slope of the downstream terrain. Manning's roughness coefficients of 0.012, 0.08, and 0.045 were used for the spillway concrete, stilling basin, and channel, respectively. A 0.2 second computation interval was selected for routing calculations.

The labyrinth weir was modeled as an inline structure with a user-inputted stage-discharge relationship. For detailed discharge characteristics of the labyrinth weir, refer to Section 3.2.

The resultant models showed negligible approach losses for the range of expected flows, up to the approximate PMF, as can be seen in Figure 3-7. Future modifications will be made in final design to further refine the approach grading to minimize the required excavation. The final spillway geometry will be verified using a 3-dimensional computational fluid dynamics (CFD) model.

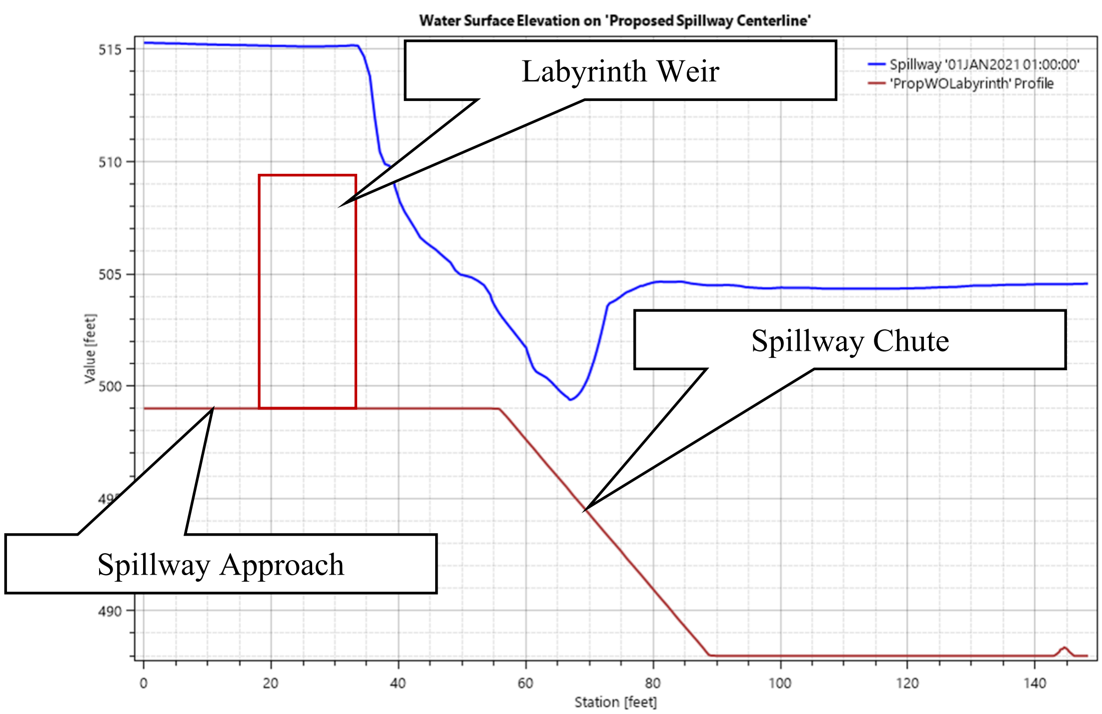


Figure 3-7 – Hydraulic Profile of Labyrinth Approach and Spillway Chute

3.4. Spillway Chute Design

A concrete slab and spillway training walls have been designed to convey the labyrinth weir flows into the downstream stilling basin. The concrete slab has been positioned so that it will be founded in or on rock.

The two-dimensional HEC-RAS model, described in Section 3.3, was used to assess the spillway chute hydraulics. The height of the spillway training walls were principally governed by the height of the adjacent embankment flattening as the water within the chute is entirely supercritical with a maximum depth of approximately 5 feet, allowing for 15 feet of freeboard. Chute walls were chosen to be non-converging as discussed in detail in Section 3.5. The chute training wall height was designed to safely contain the maximum PMF flow. Figure 3-7 contains a water surface profile through the spillway chute.

3.5. Stilling Basin Design

The spillway stilling basin was designed in accordance with procedures outlined the United States Bureau of Reclamation's Design of Small Dams, Chapter 9.E (USBR, 1987). A spreadsheet was developed to assess the potential use of different types of hydraulic-jump energy dissipators. These types of stilling basins serve to dissipate energy and transfer supercritical flow to subcritical flow. A range of inflows was inputted into the spreadsheet, up to the PMF, along with the labyrinth rating curve (developed in Section 3.2) and tailwater rating curve (developed in Section 3.1) to determine spillway chute conjugate depths.

The design flow selected for the stilling basin was 0.75 PMF. It is not practical to design a stilling basin for an event as infrequent as the PMF. Additionally, the peak PMF flow occurs only at one instant; the 0.75 PMF design is better suited for a wider range of discharges. Some damage downstream is acceptable during a PMF event, so long as the integrity of the dam is not compromised. The stilling basin will be founded in rock, so the potential for undermining will be eliminated.

There is an existing parking lot to the south of Indiantown Run, which serves as a point of constriction for the grading necessary to accommodate the stilling basin. Maintaining the existing edge of this parking lot and grading down to the stilling basin (at 2.5H:1V) results in a maximum allowable stilling basin length of approximately 60 feet (see Figure 3-8).

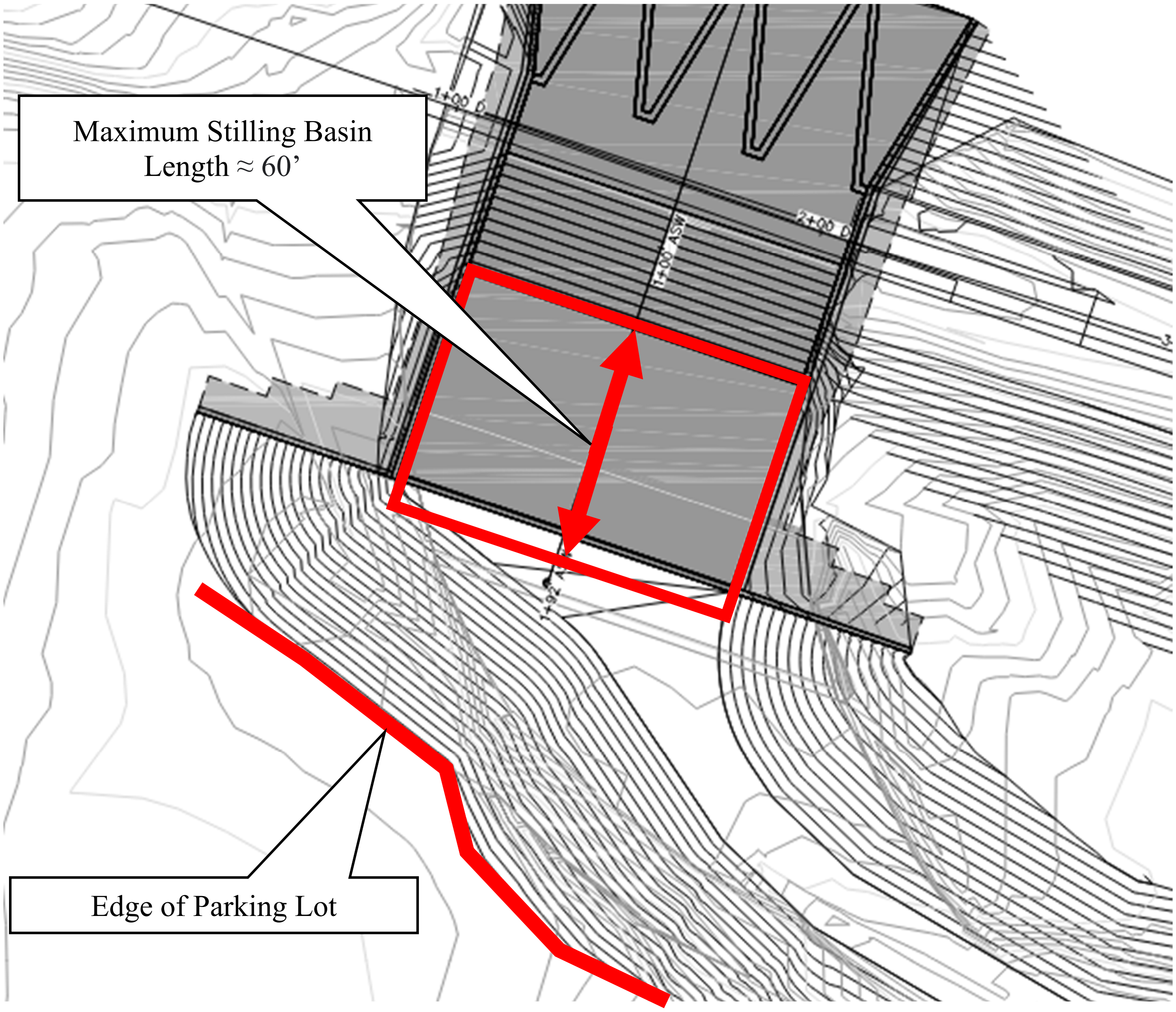


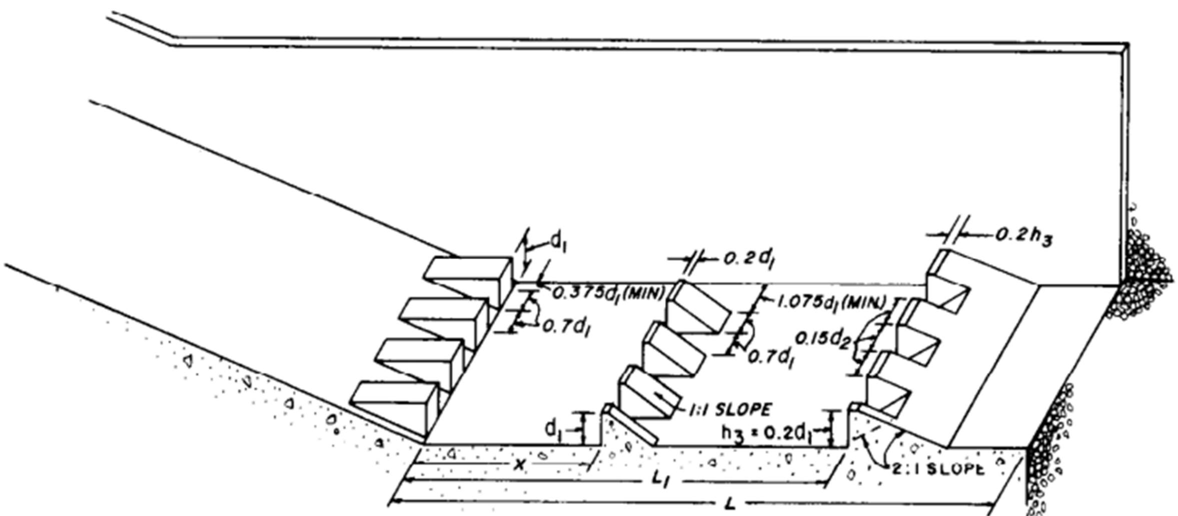
Figure 3-8 – Maximum Stilling Basin Length Due to Existing Parking Lot

This design constriction eliminates several types of stilling basin designs, including USBR Type I and Type IV, as they require basin lengths in excess of 60 feet.

For design flows from the 50-year event up to 0.25 PMF, Froude numbers are between 4.5 and 9, which is represent hydraulic conditions where a stable, well-formed hydraulic jump occurs. Above 0.25 PMF, Froude numbers are below 4.5. This condition is not entirely suitable for a USBR Type IV basin, as a hydraulic jump will not completely form and additional means of wave suppression would be required downstream.

USBR Type II basins eliminate the baffle blocks but are more susceptible to sweep out. USBR therefore requires that the tailwater depth be 5 percent greater than the computed conjugate depth. This requirement is only met for 0.25 PMF. Additionally, Type II basins are only recommended for Froude numbers greater than 4.5, therefore, a Type II basin cannot be used.

As USBR Types I through IV basins cannot be used, the Alternative Low Froude Number basin was chosen (see Figure 3-9). The chute length is short due to the limited drop in elevation from the labyrinth slab to the invert of the stilling basin. Converging the chute walls further increased the necessary length of the stilling basin and simultaneously decreased the Froude number. Both of these conditions are unfavorable, due to the limited available length for the stilling basin, and the already-low Froude numbers. As such, the spillway chute was chosen to be non-converging.



(A) DIMENSIONS FOR ALTERNATIVE LOW FROUDE NUMBER BASIN

Figure 3-9 – Alternative Low Froude Number Stilling Basin Schematic

The final stilling basin geometry will be verified using a 3-dimensional computational fluid dynamics (CFD) model. Detailed stilling basin dimensions, including chute block, baffle block and dentated end sill dimensions have been included in Appendix F.

Based on best available data, and preliminary hydraulic analysis, an exit channel has been graded from the end of the stilling basin to approximate proposed stream channel station 3+80 DC. Approximate hydraulic analysis results indicate the need for potential armoring of this segment of the channel. For the purposes of schematic design, R-6 riprap lining has been selected within the limits of channel grading. Detailed analysis of the channel hydraulics and appropriate lining material and limits will be evaluated in preliminary design.

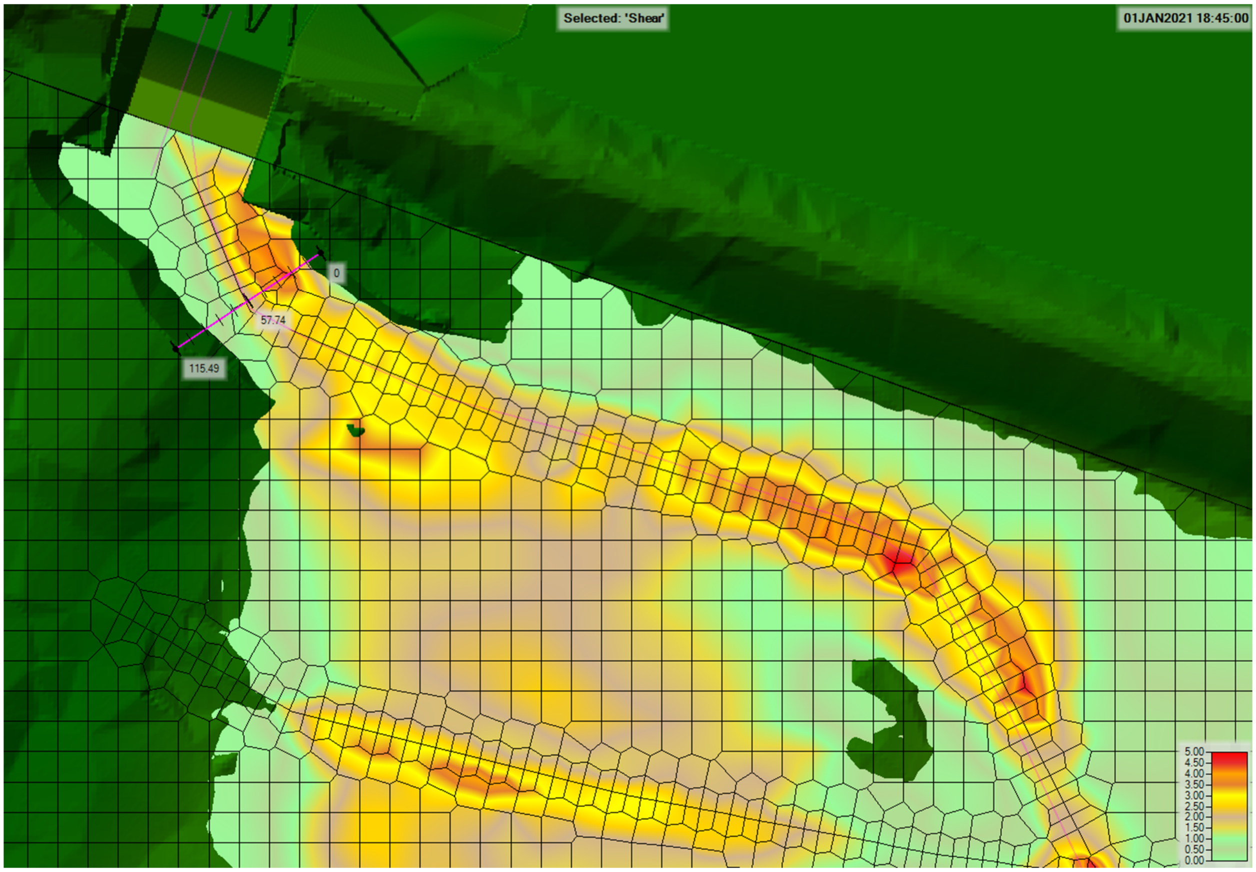


Figure 3-10 – Approximate Shear Stresses During a 1000-year Recurrence Interval Storm Peak Flow

4. DRAWDOWN ANALYSIS

PADEP requires the low-level outlet works to be able to lower the top 2 feet of the reservoir pool within 24 hours, assuming an initial elevation at normal pool level and an inflow of 70% of the maximum mean monthly discharge.

As part of this analysis, the tailwater model, developed in Section 3.1, was used to evaluate the potential submergence of the outlet conduit.

Photographs of the outlet works and downstream area are shown in Figure 4-1.

For a 2-year storm, water surface elevations were shown to be well above the crest of the discharge conduit. Inspection of site photographs taken by D'Appolonia in October 2020 show that even during dry conditions, the crest of the discharge conduit is submerged. Based on these analyses, in developing the discharge rating curve for the outlet works, it was assumed that tailwater was present up to the crown elevation of the pipe (483.0 feet).

Figure 4-1– Photographs of Low-Level Outlet Works Discharge Conduit Outlet

4.1. Lake Drain Analysis

The maximum mean monthly discharge into Marquette Lake was estimated from the USGS gaging station No. 01572950 on Indiantown Run near Harper Tavern, PA, just upstream of Marquette Lake. With a drainage are of 5.48 mi^2, this gaging station records streamflow from 93% of the contributing area to the lake. The maximum mean monthly flow for the period of record (12 years) occurs in March at 18 cfs. Transposed to Marquette Lake's drainage area of 5.88 mi^2 using a ratio of drainage areas the maximum mean monthly flow is 19.3 cfs. The required inflow for the analysis (70%) is 13.5 cfs.

Limited information is available to describe the exact physical configuration of the outlet works. The 1980 Phase I Inspection study reports an upstream invert of 482 feet (MSL) and a diameter of 36 inches. Elevations reported in the Phase I report in MSL datum were compared to known elevation in NAVD 88. Because the elevations compared well, no further adjustments were made, and the upstream invert elevation was assumed to be 482 feet (NAVD 88). There was no data available for the upstream intake tower opening, so it was assumed to be 36-inchx36-inch square. Both the tower opening, and the 36-inch DIP discharge conduit are equipped with sluice gates. The downstream discharge conduit invert was also not known but was estimated to be at elevation 480 feet based on LiDAR returns of the water surface elevation of the submerged outlet channel and on field observations.

A discharge rating curve for the 36-inch-diameter pipe outlet was developed using a custom excel spreadsheet. The discharge rating curve assumes that both sluice gates are completely open.

Using level pool routing and with the assumed inflow conditions, the low-level outlet works can lower the pool 2.06 feet in 2 hours if the two sluice gates are completely opened. See Appendix G for detailed calculations. No modifications to the existing outlet works are needed to meet PADEP criteria.

5. REFERENCES

1. AEG, *Marquette Lake Dam Geophysics Report, Fort Indiantown Gap Dam Project*. July 2017.

2. GAI Consultants, Inc, *Phase I Inspection Report – Reservation Dam NDI ID No. PA-00014 PENNDER ID No. 38-78*. January 1980.

3. Greenman-Pedersen, Inc., *Spillway Capacity and Incremental Dam Breach Analysis for Marquette Lake Dam.* April 24, 2014

4. HEC, (2018), *Hydrologic Modeling system HEC-HMS Version 4.6.1*. U.S. Army Corps of Engineers, Hydrologic Engineering Center, Davis, CA.

5. HEC, (2019), *River Analysis System HEC-RAS Version 5.0.7*. U.S. Army Corps of Engineers, Hydrologic Engineering Center, Davis, CA.

6. NOAA. (2006). NOAA Atlas 14: Precipitation-Frequency Atlas of the United States. U.S. Department of Commerce, National Oceanic and Atmospheric Administration, National Weather Service.

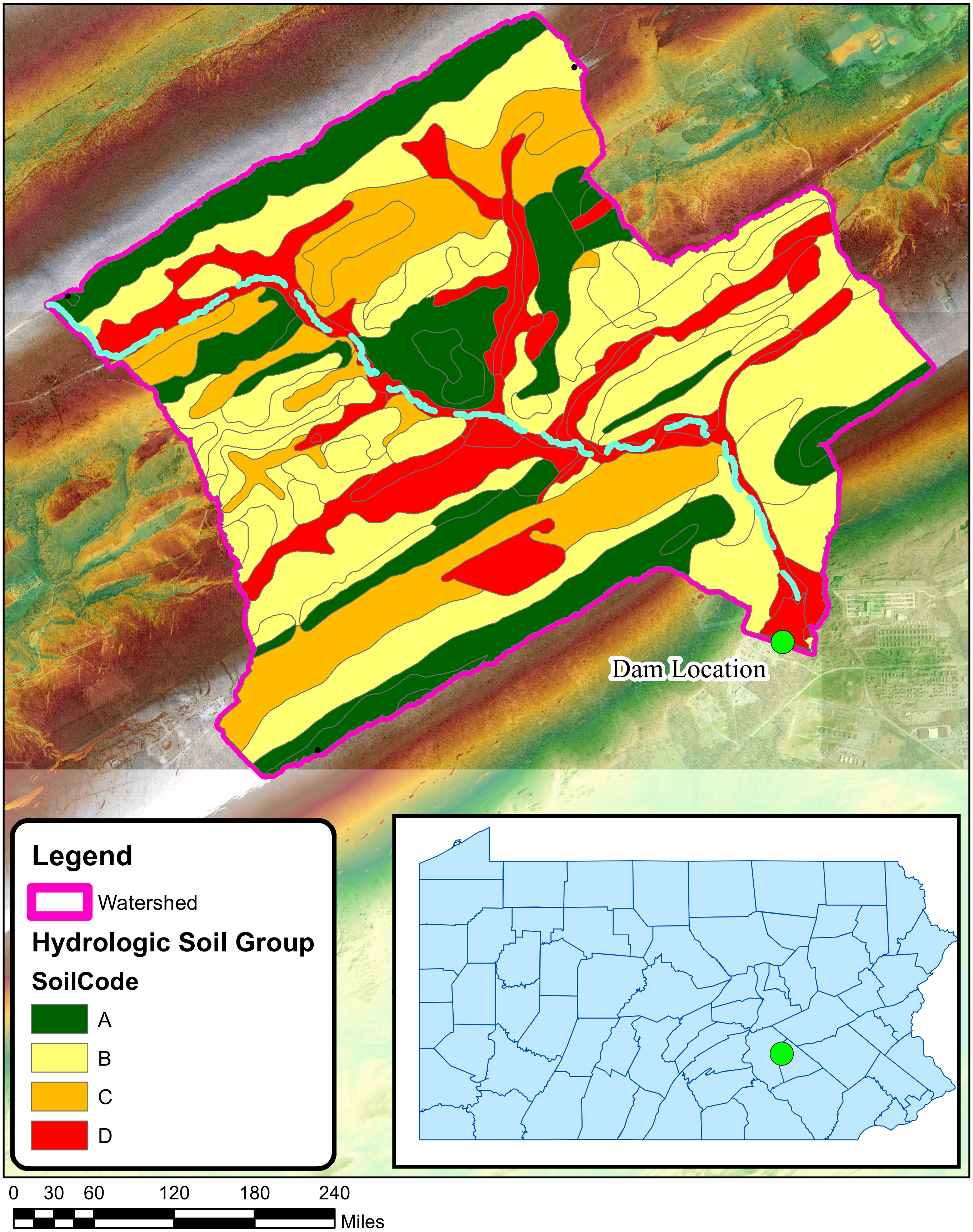
7. Roland, M.A., and M.H. Stuckey, (2008). "Regression equations for estimating flood flows at selected recurrence intervals for ungaged streams in Pennsylvania." *U.S. Geological Survey Scientific Investigations Report 2008-5102*, 57p. (http://pubs.usgs.gov/sir/2008/5102/). Accessed on October 1, 2019.

8. U.S. Department of the Interior USBR (1987). *Design of small dams*. Third Edition, Washington D.C.

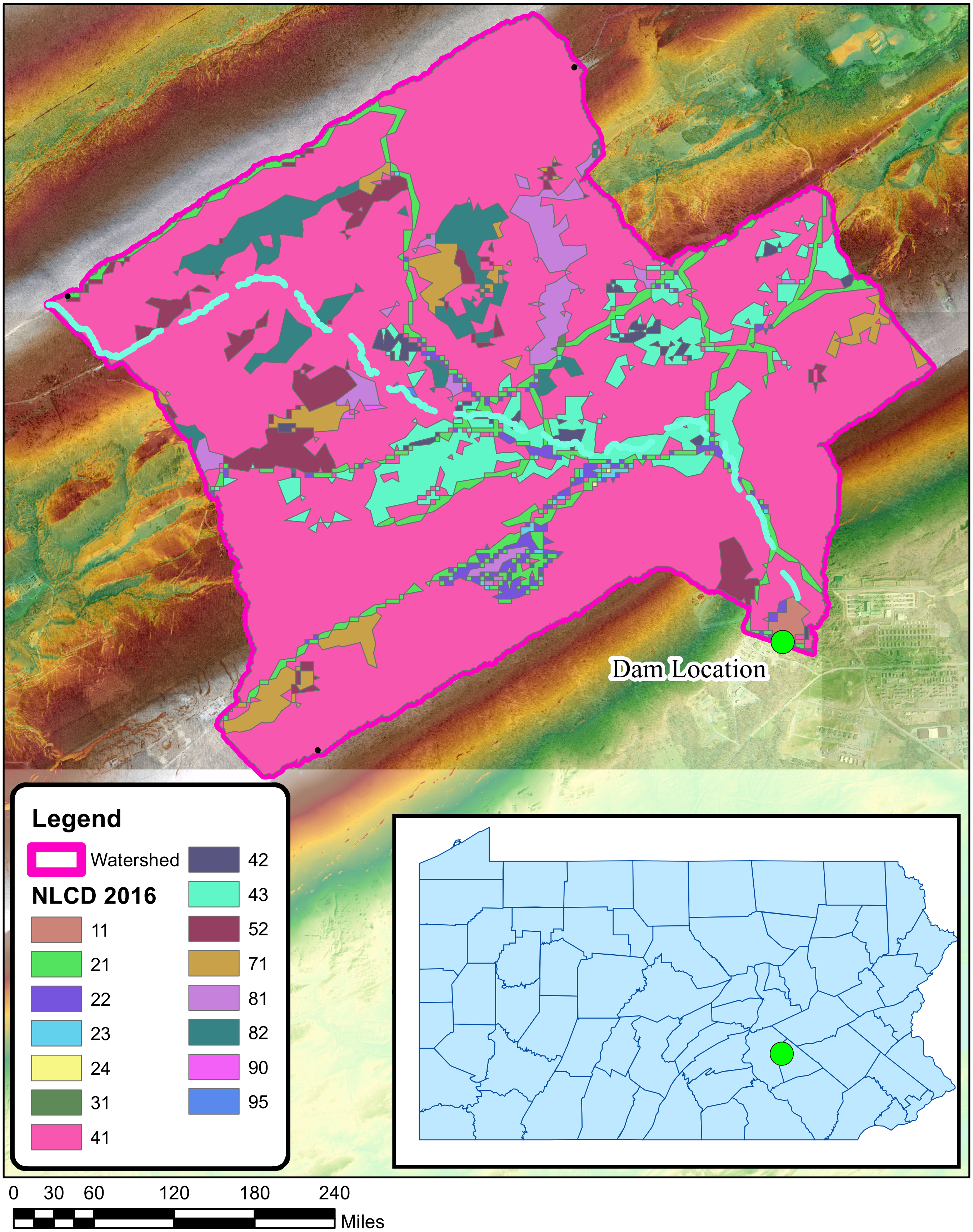
9. U.S. Geological Survey, 2017, The StreamStats program, online at http://streamstats.usgs.gov.

10. U.S. Weather Bureau, 1955, *Climatological Data: National summary*, Vol. 6.

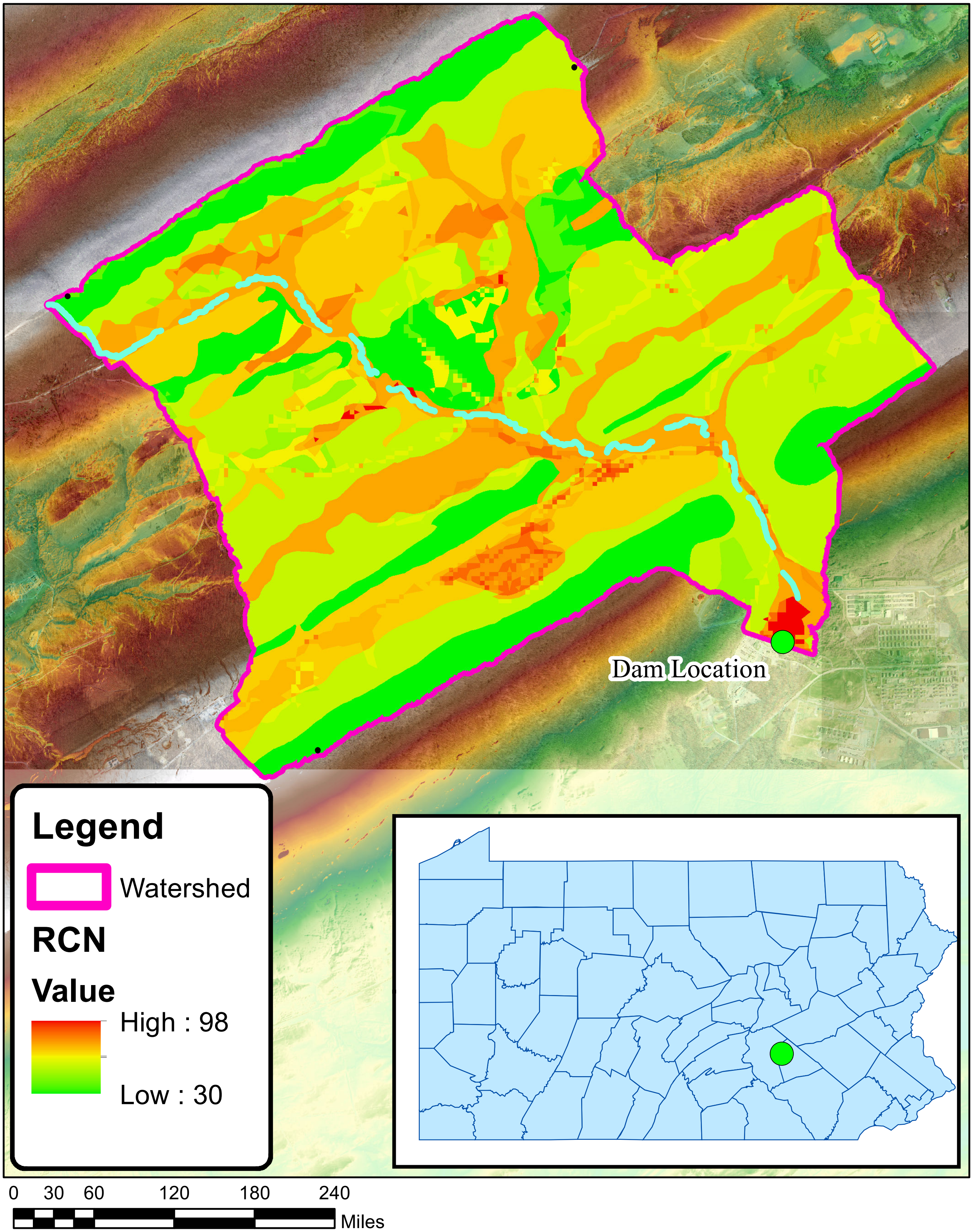
Appendix A – Soil, Land Cover, and Runoff Curve Number Maps



Hydrologic Soil Group



2016 NLCD Data



Runoff Curve Number

Appendix B – Time of Concentration Calculations

Project: Marquette Dam
Location: Lebanon County, PA
Subwatershed:

By: NCC Date: 10/8/2020
Checked: SDT Date: 10/27/2020

Flowpath Description:

Sheet Flow Properties

Segment ID	SH-1
1. Surface description	Woods - Light Underbrush
2. Manning's roughness coefficient, n (NRCS NEH,Table 15-1)	0.400
3. Flow length, L (ft)	100
4. Two-year 24-hour rainfall, P2 (in) (NOAA HDSC PFDS)	3.08
5. Land slope, s (ft/ft)	0.14500

$$T_t = \frac{0.007\,(nL)^{0.8}}{(P2)^{0.5}(s)^{0.4}}$$

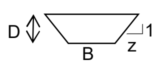
T_t = 0.17 hours

Shallow Concentrated Flow Properties

Segment ID	SC-1	SC-2	SC-3
6. Surface description (NRCS NEH, Table 15-3)	Forest with heavy ground litter and hay meadows	Forest with heavy ground litter and hay meadows	Grassed waterways
7. Flow length, L (ft)	797	1364	5461
8. Watercourse slope, s (ft/ft)	0.26000	0.05000	0.06000
9. Average velocity, V (NRCS NEH, Table 15-3)	1.283	0.563	3.952
	V=2.516(s)^0.5	V=2.516(s)^0.5	V=16.135(s)^0.5
T_t =	0.17 hours	0.67 hours	0.38 hours

$$T_t = \frac{L}{3600V}$$

Open Channel Flow Properties



Segment ID	CH-1	CH-2
10. Bottom width, B (ft)	3	5
11. Depth flowing full, D (ft)	2	3.5
12. Channel side slope, z H : 1 V	2	2
13. Cross sectional area, A (sq ft)	14	42
14. Wetted perimeter, P (ft)	11.94	20.65
15. Hydraulic radius, r = A/P (ft)	1.17	2.03
16. Channel slope, s (ft/ft)	0.01200	0.01000
17. Manning's roughness coefficient, n	0.040	0.040
18. Velocity, V (ft/sec)	4.54	5.98
Q (cfs) =	63.507328	251.12716
19. Flow length, L (ft)	6,140	12,167
Tt =	0.38 hours	0.57 hours

$$V = \frac{1.49(r^{2/3})(s^{1/2})}{n}$$

$$T_t = \frac{L}{3600V}$$

Results

Time of Concentration =	**2.34**	**hours**	**=**	**140.17**	**minutes**
Lag Time =	**1.40**	**hours**	**=**	**84.10**	**minutes**

Appendix C – PADEP PMP Distributions

Input the rainfall data for the Local, Tropical, and General Storm directly from the PMP tool.
This data is available on the PMP_Basin_Average.csv file
which is located in the CSV_ folder for the analyzed watershed.

	1 HR	2 HR	3 HR	4 HR	5 HR	6 HR	12 HR	24 HR
Local	10.2	11.7	13.2	14.47	16.9	22.97	25	25.89
Tropical	9.16	9.16	11.4	12.16	14.16	16.52	25.66	25.66
General	3.6	5.86	9.89	9.89	9.89	14	16.22	16.42

The green highlighted values in the table above are the controlling PMP values for the specified durations.
The Yellow highlighted Storm type below is the controlling storm for the specific duration.
- Use GIS program to view PMP_Points for your watershed to determine the controlling storm at each duration.
- If Local controls at all durations, only the Local_PMP_Points will need to be used.
- If other storms (General, Tropical) control at certain durations, make sure to use the correct PMP_Points file.
- If multiple storms control at a specific duration, i.e. more than one Local storm, try all distributions and choose the most conservative answer.

Select the appropriate storm from the red highlighted dropdown for each duration.

3 HR	
Local	1547_1

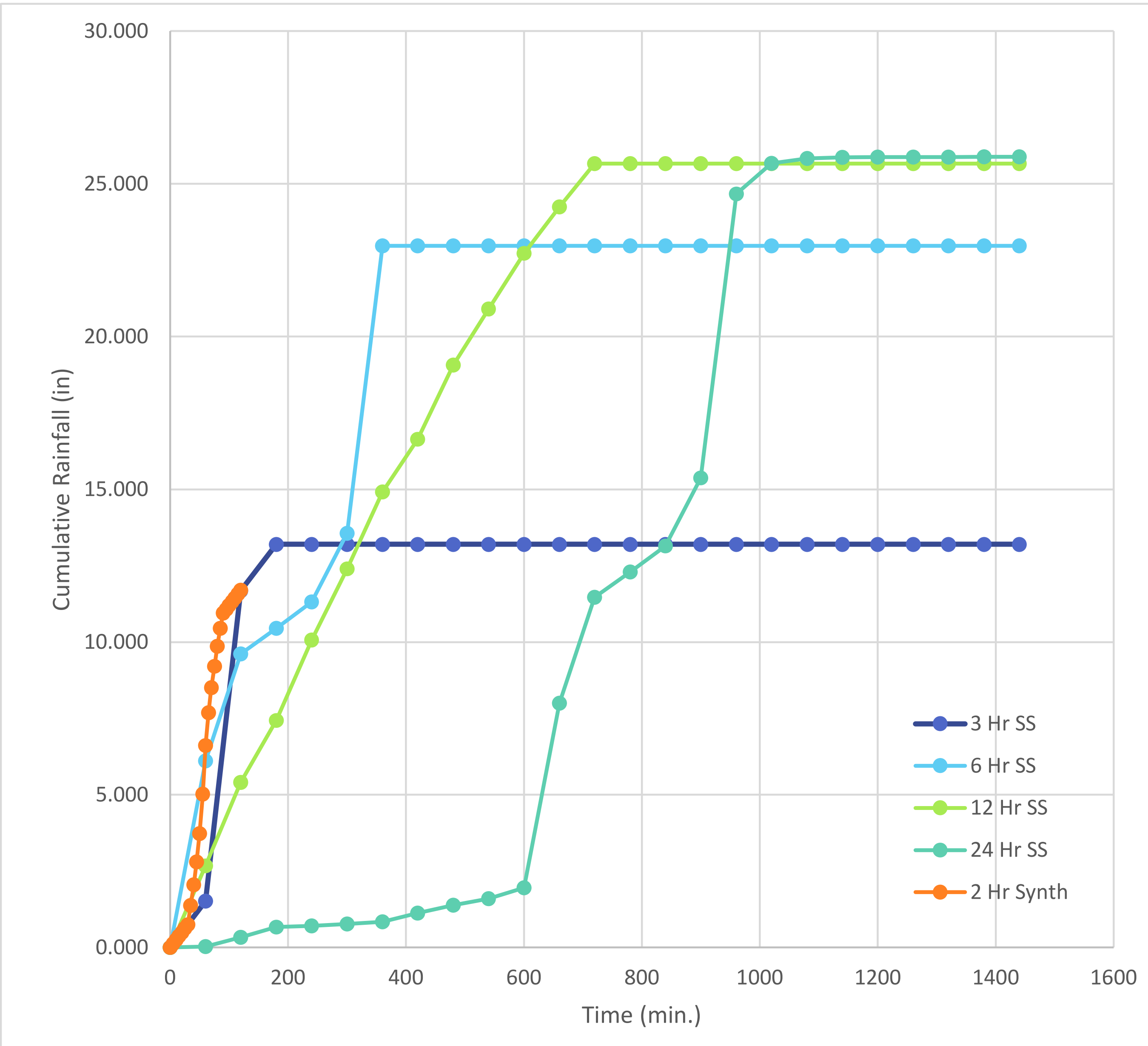
6 HR	
Local	1406_1

12 HR	
Tropical	1491_1

24 HR	
Local	1406_1

The storm specific distributions for use in HEC-HMS or other
hydraulic routing programs will be available to the right.
The rainfall distributions are given in 1-hour increments.
A 5-minute timestep should be used in the hydraulic routing program to capture the peak of the storm.

STORM SPECIFIC DISTRIBUTION								2 HR Synth	
3 HR		6 HR		12 HR		24 HR			
1547_1		1406_1		1491_1		1406_1			
MIN	INC	MIN	INC	MIN	INC	MIN	INC	MIN	INC
0	0.000	0	0.000	0	0.000	0	0.000	*0*	0.000
60	1.523	60	6.109	60	2.670	60	0.025	*5*	0.125
120	10.154	120	3.506	120	2.737	120	0.306	*10*	0.125
180	1.523	180	0.836	180	2.024	180	0.338	*15*	0.125
240	0.000	240	0.859	240	2.633	240	0.041	*20*	0.125
300	0.000	300	2.252	300	2.328	300	0.056	*25*	0.125
360	0.000	360	9.408	360	2.528	360	0.071	*30*	0.125
420	0.000	420	0.000	420	1.720	420	0.284	*35*	0.629
480	0.000	480	0.000	480	2.433	480	0.270	*40*	0.679
540	0.000	540	0.000	540	1.825	540	0.212	*45*	0.745
600	0.000	600	0.000	600	1.825	600	0.352	*50*	0.920
660	0.000	660	0.000	660	1.521	660	6.042	*55*	1.298
720	0.000	720	0.000	720	1.416	720	3.468	*60*	1.592
780	0.000	780	0.000	780	0.000	780	0.827	*65*	1.077
840	0.000	840	0.000	840	0.000	840	0.850	*70*	0.812
900	0.000	900	0.000	900	0.000	900	2.227	*75*	0.705
960	0.000	960	0.000	960	0.000	960	9.306	*80*	0.659
1020	0.000	1020	0.000	1020	0.000	1020	0.996	*85*	0.580
1080	0.000	1080	0.000	1080	0.000	1080	0.164	*90*	0.501
1140	0.000	1140	0.000	1140	0.000	1140	0.029	*95*	0.125
1200	0.000	1200	0.000	1200	0.000	1200	0.007	*100*	0.125
1260	0.000	1260	0.000	1260	0.000	1260	0.000	*105*	0.125
1320	0.000	1320	0.000	1320	0.000	1320	0.000	*110*	0.125
1380	0.000	1380	0.000	1380	0.000	1380	0.018	*115*	0.125
1440	0.000	1440	0.000	1440	0.000	1440	0.001	*120*	0.125



30.000
25.000
20.000
15.000
10.000
5.000
0.000
Cumulative Rainfall (in)
0
200
400
600
800
1000
1200
1400
1600
Time (min.)
3 Hr SS
6 Hr SS
12 Hr SS
24 Hr SS
2 Hr Synth

Appendix D – NOAA Atlas 14 Precipitation Frequency Estimates

NOAA Atlas 14, Volume 2, Version 3
Location name: Annville, Pennsylvania, USA*
Latitude: 40.4463°, Longitude: -76.6066°
Elevation: 621.5 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	0.340 (0.308-0.375)	0.404 (0.366-0.446)	0.471 (0.426-0.520)	0.520 (0.470-0.574)	0.580 (0.523-0.640)	0.623 (0.559-0.686)	0.668 (0.598-0.737)	0.708 (0.631-0.781)	0.762 (0.675-0.844)	0.807 (0.710-0.895)
10-min	0.536 (0.486-0.591)	0.640 (0.580-0.707)	0.746 (0.675-0.825)	0.822 (0.743-0.908)	0.910 (0.820-1.00)	0.975 (0.876-1.07)	1.04 (0.930-1.15)	1.10 (0.979-1.21)	1.17 (1.04-1.30)	1.23 (1.08-1.37)
15-min	0.666 (0.603-0.734)	0.797 (0.723-0.881)	0.935 (0.846-1.03)	1.03 (0.930-1.14)	1.14 (1.03-1.26)	1.23 (1.10-1.35)	1.31 (1.17-1.44)	1.38 (1.23-1.52)	1.48 (1.31-1.63)	1.54 (1.36-1.71)
30-min	0.903 (0.818-0.995)	1.09 (0.987-1.20)	1.31 (1.19-1.45)	1.47 (1.33-1.62)	1.67 (1.50-1.84)	1.81 (1.63-2.00)	1.96 (1.75-2.16)	2.10 (1.87-2.32)	2.29 (2.03-2.54)	2.43 (2.14-2.70)
60-min	1.12 (1.01-1.23)	1.36 (1.23-1.50)	1.67 (1.51-1.85)	1.90 (1.72-2.10)	2.20 (1.98-2.42)	2.43 (2.18-2.68)	2.67 (2.39-2.95)	2.91 (2.60-3.21)	3.24 (2.87-3.59)	3.51 (3.09-3.89)
2-hr	1.33 (1.20-1.47)	1.60 (1.45-1.78)	2.00 (1.81-2.21)	2.31 (2.08-2.56)	2.76 (2.47-3.05)	3.13 (2.79-3.46)	3.53 (3.13-3.91)	3.97 (3.49-4.40)	4.61 (4.01-5.12)	5.15 (4.44-5.75)
3-hr	1.45 (1.31-1.62)	1.76 (1.59-1.97)	2.19 (1.98-2.45)	2.55 (2.29-2.84)	3.05 (2.73-3.40)	3.48 (3.09-3.87)	3.95 (3.48-4.40)	4.46 (3.90-4.96)	5.21 (4.50-5.82)	5.87 (5.00-6.57)
6-hr	1.80 (1.62-2.02)	2.18 (1.96-2.44)	2.71 (2.43-3.04)	3.16 (2.82-3.53)	3.81 (3.38-4.25)	4.37 (3.85-4.87)	4.99 (4.37-5.56)	5.68 (4.92-6.33)	6.71 (5.74-7.50)	7.61 (6.42-8.53)
12-hr	2.22 (1.99-2.50)	2.68 (2.40-3.02)	3.36 (3.00-3.78)	3.93 (3.49-4.41)	4.79 (4.22-5.36)	5.55 (4.85-6.19)	6.40 (5.55-7.15)	7.36 (6.31-8.22)	8.84 (7.45-9.87)	10.2 (8.43-11.4)
24-hr	2.55 (2.33-2.84)	3.08 (2.81-3.42)	3.87 (3.52-4.29)	4.56 (4.13-5.04)	5.61 (5.05-6.19)	6.56 (5.85-7.21)	7.63 (6.75-8.37)	8.87 (7.75-9.70)	10.8 (9.30-11.8)	12.5 (10.6-13.7)
2-day	2.97 (2.70-3.32)	3.58 (3.25-4.00)	4.50 (4.08-5.02)	5.29 (4.77-5.89)	6.49 (5.81-7.20)	7.54 (6.70-8.37)	8.74 (7.70-9.68)	10.1 (8.82-11.2)	12.2 (10.5-13.5)	14.1 (11.9-15.6)
3-day	3.14 (2.87-3.47)	3.77 (3.45-4.18)	4.74 (4.32-5.24)	5.56 (5.06-6.15)	6.82 (6.16-7.51)	7.93 (7.11-8.72)	9.18 (8.17-10.1)	10.6 (9.35-11.6)	12.8 (11.1-14.1)	14.8 (12.7-16.2)
4-day	3.30 (3.04-3.63)	3.97 (3.66-4.37)	4.97 (4.57-5.46)	5.84 (5.35-6.40)	7.15 (6.51-7.81)	8.31 (7.51-9.06)	9.63 (8.63-10.5)	11.1 (9.88-12.1)	13.5 (11.8-14.6)	15.5 (13.4-16.8)
7-day	3.90 (3.61-4.26)	4.68 (4.33-5.11)	5.80 (5.36-6.34)	6.77 (6.24-7.38)	8.24 (7.53-8.95)	9.53 (8.66-10.3)	11.0 (9.91-11.9)	12.6 (11.3-13.6)	15.1 (13.3-16.4)	17.4 (15.1-18.8)
10-day	4.49 (4.18-4.86)	5.37 (5.00-5.81)	6.58 (6.12-7.11)	7.61 (7.05-8.20)	9.12 (8.40-9.81)	10.4 (9.56-11.2)	11.9 (10.8-12.7)	13.5 (12.2-14.4)	15.9 (14.2-17.0)	17.9 (15.8-19.2)
20-day	6.12 (5.75-6.53)	7.25 (6.81-7.73)	8.63 (8.10-9.20)	9.76 (9.14-10.4)	11.4 (10.6-12.1)	12.7 (11.9-13.6)	14.2 (13.1-15.1)	15.8 (14.5-16.8)	18.0 (16.5-19.2)	19.9 (18.0-21.2)
30-day	7.60 (7.17-8.07)	8.96 (8.45-9.51)	10.5 (9.87-11.1)	11.7 (11.0-12.4)	13.5 (12.6-14.3)	14.9 (13.9-15.8)	16.4 (15.3-17.4)	18.0 (16.7-19.1)	20.3 (18.7-21.5)	22.1 (20.2-23.5)
45-day	9.55 (9.06-10.1)	11.2 (10.6-11.8)	12.9 (12.2-13.6)	14.2 (13.5-15.0)	16.1 (15.2-16.9)	17.5 (16.5-18.4)	19.0 (17.9-20.0)	20.5 (19.3-21.6)	22.6 (21.2-23.9)	24.3 (22.6-25.6)
60-day	11.4 (10.9-12.0)	13.4 (12.7-14.1)	15.3 (14.5-16.0)	16.8 (15.9-17.6)	18.8 (17.8-19.7)	20.3 (19.2-21.3)	21.9 (20.7-23.0)	23.5 (22.1-24.7)	25.7 (24.1-27.0)	27.4 (25.6-28.9)

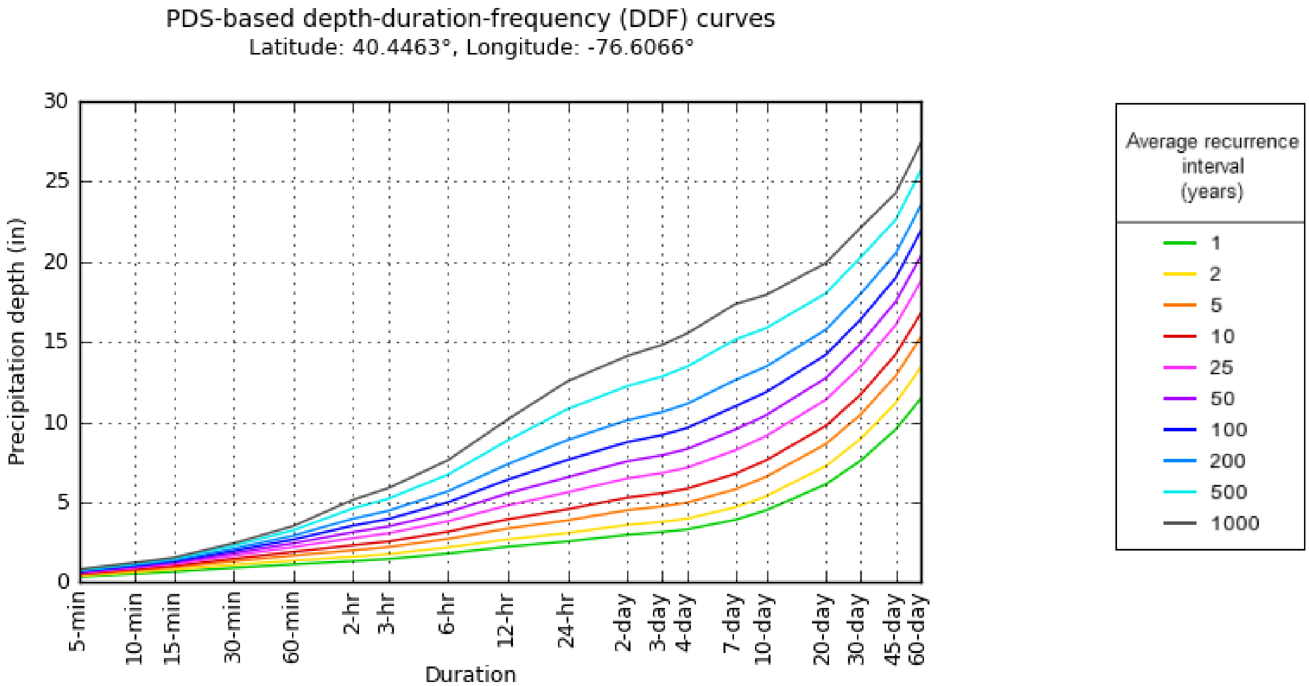
[1] Precipitation frequency (PF) estimates in this table are based on frequency analysis of partial duration series (PDS).

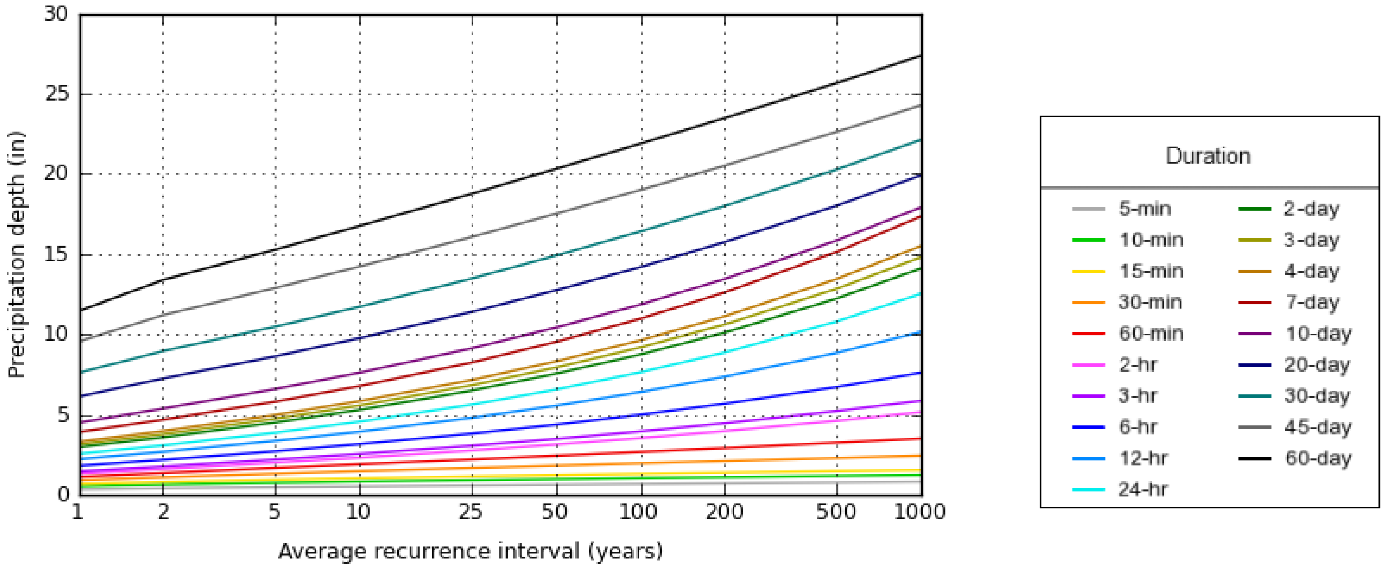
Numbers in parenthesis are PF estimates at lower and upper bounds of the 90% confidence interval. The probability that precipitation frequency estimates (for a given duration and average recurrence interval) will be greater than the upper bound (or less than the lower bound) is 5%. Estimates at upper 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



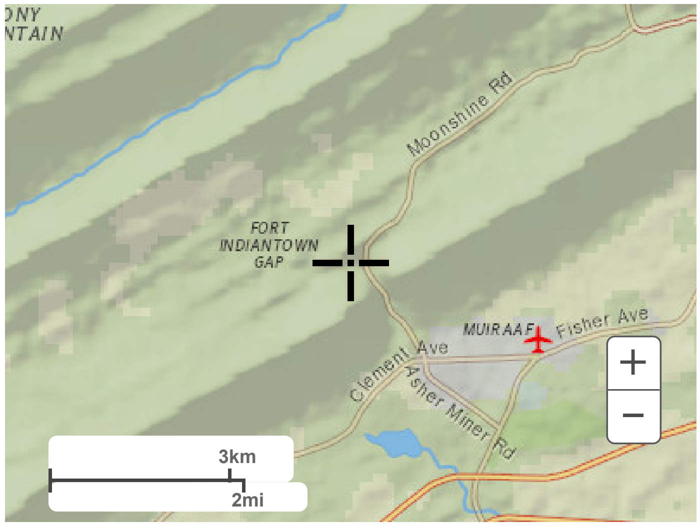


NOAA Atlas 14, Volume 2, Version 3 Created (GMT): Thu Oct 8 18:17:35 2020

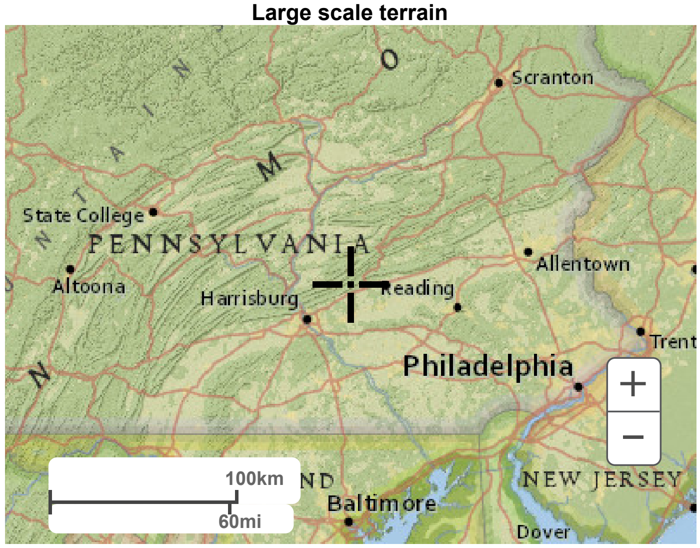
Back to Top

Maps & aerials

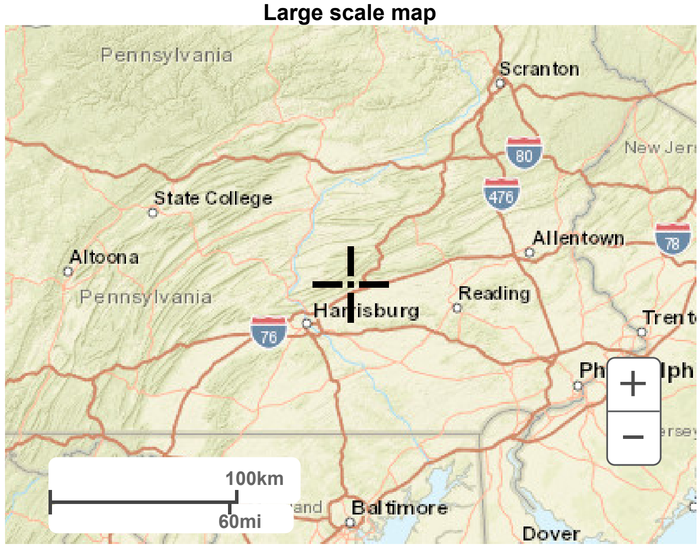
Small scale terrain



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]

Appendix E – Labyrinth Spillway Design Calculations

LABYRINTH WEIR DESIGN

PROJECT:	Marquette	**DATE:**	18-Jan-21
PROJECT NO.	067868	**DESIGN:**	GLR
FLOOD CRITERIA:		**CHECK:**	NCC

HYDRAULIC CONDTIONS - INPUT DATA

Design Flow	Q_{design}	18,500.0	**cfs**	
Design Flow WSE	H	519.23	**ft**	
Crest Elevation	H_{crest}	509.26	**ft**	
Approach Channel Elevation	H_{apron}	499.0	**ft**	
Unsubmerged Total Upstream Head	H_t	9.97	**ft**	(assuming no approach velocity)
Total Downstream Head	H_d	0.0	**ft**	(assuming zero submergence)

LABYRINTH WEIR GEOMETRY - INPUT DATA

Wall Angle	α	14.6	**deg**	**Angle is okay**
Number of Cycles	N	3.0	-	
Thickness of Wall	T_w	2.0	**ft**	
Thickness of Slab	T_s	2.0	**ft**	
Inside Apex Width	A	2.00	**ft**	
Sheet Pile Cutoff Depth	D_s	0.0	**ft**	
Concrete Wall Cutoff Depth	D_c	4.0	**ft**	
Crest Shape	-	Half-Round	-	
Aeration Device	-	None	-	(Breakers, Vents, or None)

CALCULATED DATA

Crest Height	P	10.26	**ft**	
Headwater Ratio	H_t/P	0.97	-	**Ratio is okay**
Labyrinth Weir Discharge Coefficient	C_d	0.36	-	
Total Centerline Length of Weir	L_c	302.50	**ft**	
Centerline Length of Sidewall	l_c	46.87	**ft**	
Length of Apron (parallel to flow)	B	47.363554461	**ft**	
Cycle Width	w	30.67	**ft**	
Outside Apex Width	D	5.09	**ft**	
Labyrinth Width (normal to flow)	W	92.00	**ft**	
Magnification Ratio	L/W	3.29	-	**Ratio is okay**
Cycle Width Ratio	w/P	2.99	-	**Ratio is okay**
Apex Ratio	A/w	0.065	-	**Ratio is okay**
Cycle Efficiency	ε'	1.19	-	
Efficacy	ε	1.70	-	
Linear Weir Discharge Coefficient	$C_d(90)$	0.70	-	
Length of Linear Weir for Same Flow	$L_c(90)$	156.23	**ft**	

Appendix F – Stilling Basin Design Calculations

STILLING BASIN DESIGN - MARQUETTE LAKE LABYRINTH SPILLWAY

PROJECT:	Marquette	**DESIGN:**	NCC	**CHECK:**	SDT
PROJECT NO.	67868	**DATE:**	19-Jan-21	**DATE:**	23-Feb-21

SPILLWAY GEOMETRY DATA

Stilling Basin Elevation	488.00	*ft*	
Spillway Width (PSW)	92.00	*ft*	Spillway Width at bottom of stilling basin
Weir Length (PSW)	50.86	*ft*	
Crest Elevation (PSW)	509.26	*ft*	
Number of Cycles (PSW)	3		
Ratio of tailwater to d_2	0.85		(Minimum of 0.85 for Type III Basin)
			(minimum of 1.1 for Type IV basin for Froude 2.5 to 4.5)
			(minimum of 1.05 for type II Basin)

PRINCIPAL SPILLWAY LABYRINTH WEIR

	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19
	Event	**Reservoir Elevation**	**Discharge**	**Unit Discharge**	**Head over Weir**	**Tailwater Elevation**	**E_1**	**E_1 chute**	**7-8**	**D_1**	**TW/D_1**	**Vel Head**	**V_1**	**Froude Number**	**Elev2**	**D_2**	**Tailwater Depth**	**TW/D_2**	**Tailwater sufficient?**
	-	*ft NAVD*	*cfs*	*cfs/ft*	*ft*	*ft*	*ft*	*ft*	*ft*	*ft*	-	*ft*	*ft/sec*	-	*ft*	*ft*	*ft*	-	-
1	PMF	519.1041	18240.0	198.26	9.84	504.27	31.10	31.10	0.00	4.82	3.38	26.29	41.14	3.30	508.23	20.23	16.27	0.80	No
2	0.75 PMF	516.7344	13680.0	148.70	7.47	502.82	28.73	28.73	0.00	3.70	4.00	25.03	40.15	3.68	505.49	17.49	14.82	0.85	Yes
3	0.50 PMF	513.8701	9120.0	99.13	4.61	501.27	25.87	25.87	0.00	2.56	5.19	23.31	38.75	4.27	502.22	14.22	13.27	0.93	Yes
4	0.25 PMF	511.4268	4560.0	49.57	2.17	498.96	23.43	23.43	0.00	1.31	8.34	22.11	37.74	5.80	498.14	10.14	10.96	1.08	Yes
5	500-YR	511.4217	4549.0	49.45	2.16	498.95	23.42	23.42	0.00	1.31	8.36	22.11	37.74	5.81	498.13	10.13	10.95	1.08	Yes
6	100-YR	510.6374	2606.0	28.33	1.38	497.52	22.64	22.64	0.00	0.75	12.62	21.88	37.54	7.62	495.76	7.76	9.52	1.23	Yes
7	50-yr	510.3794	1946.0	21.15	1.12	496.93	22.38	22.38	0.00	0.56	15.83	21.82	37.48	8.79	494.74	6.74	8.93	1.33	Yes
8	10-yr	509.7527	807.0	8.77	0.49	494.81	21.75	21.75	0.00	0.24	28.92	21.52	37.22	13.51	492.39	4.39	6.81	1.55	Yes
								Solver Cell	0.00										

	Type I		Type II		Type III					Type IV		Alt. Low Froude # Basin		
	L/D_2	**Jump Length**	**L/D_2**	**Jump Length**	**L/D_2**	**Jump Length**	**h_3**	**h_4**	**Froude # Check**	**L/D_2**	**Jump Length**	**L/D_2**	**Jump Length**	**h_3**
	-	*ft*	-	*ft*	-	*ft*	*ft*	*ft*		-	*ft*	-	*ft*	*ft*
1	5.41	109.47	3.39	68.47	2.05	41.42	5.47	5.86	Not OK	5.50	111.34	3.20	64.80	0.96
2	5.63	98.41	3.50	61.22	2.12	37.05	4.44	4.58	Not OK	5.79	101.30	3.15	55.14	0.74
3	5.89	83.79	3.66	52.07	2.22	31.58	3.32	3.25	Not OK	6.02	85.58	3.06	43.47	0.51
4	6.27	63.62	3.98	40.36	2.44	24.73	2.04	1.77	OK	6.33	64.24	2.54	25.77	0.26
5	6.27	63.55	3.98	40.33	2.44	24.71	2.04	1.77	OK	6.34	64.18	2.54	25.71	0.26
6	6.37	49.41	4.20	32.58	2.62	20.29	1.40	1.09	OK	6.76	52.46	1.48	11.49	0.15
7	6.33	42.67	4.27	28.79	2.69	18.13	1.16	0.85	OK	7.08	47.75	0.77	5.22	0.11
8	5.91	25.93	4.28	18.78	2.76	12.11	0.67	0.42	OK	8.39	36.82	-2.06	-9.03	0.05

Note: Max unit discharge for Type III Basin is 200 cfs/ft
Note: Max velocities for Type III Basin are 50-60 ft/sec
Design Froude # Below Recommended Value of 4 for Types I, II, III, IV, Use Alt Low Froude # Basir

STILLING BASIN DESIGN - MARQUETTE LAKE LABYRINTH SPILLWAY

PROJECT:	Marquette Dam	**DESIGN:**	NCC	**CHECK:**	SDT
PROJECT NO.	67868	**DATE:**	19-Jan-21	**DATE:**	23-Feb-21

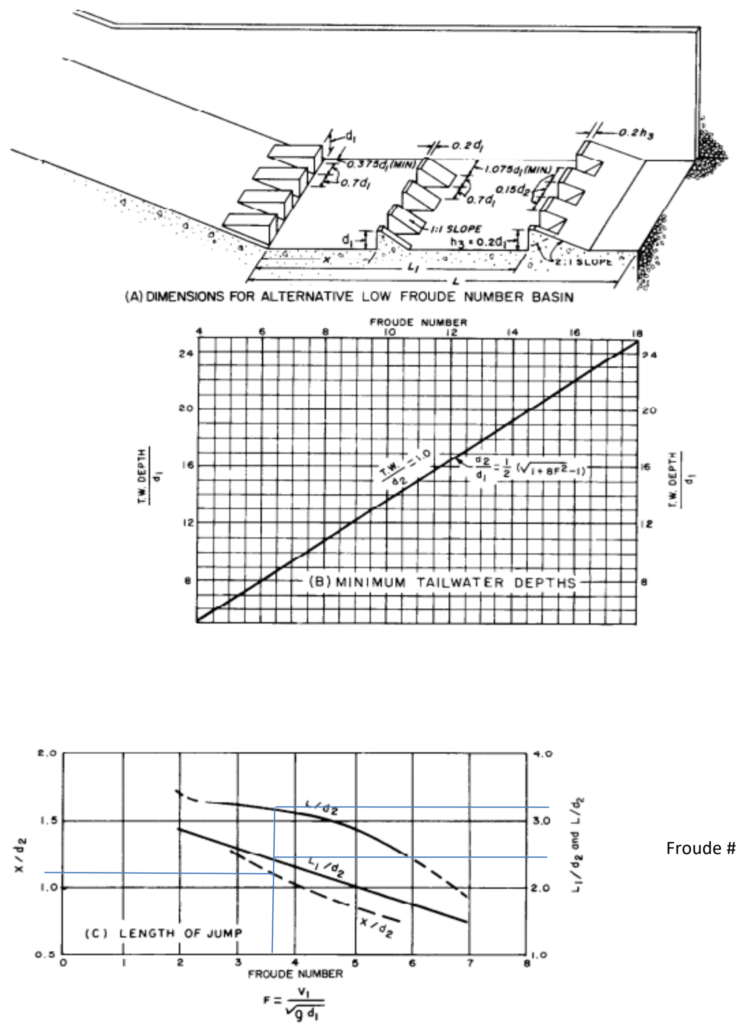
BASIN LENGTHS		
L =	55.00	ft
$0.8*D_2$ =	14.00	ft

CHUTE AND CHUTE BLOCKS		
Length of Chute =	33.00	ft
Chute Slope =		
D_1 =	4.00	ft
$0.375*D_1$ =	2.00	ft
$0.7*D_1$ =	3.00	ft
x	20.00	ft
L_1 =	42.00	ft
L =	56.00	ft
Width @Bottom	92.00	ft
# Chute Blocks =	15.00	

BAFFLE BLOCKS		
D_1 =	4.00	ft
$0.7*D_1$ =	3.00	ft
$1.075*D_1$ =	4.00	ft
$0.2*D_1$ =	1.00	ft
# Baffle Blocks =	15.00	

DENTATED END SILL		
$0.2*H_3$ =	1.00	ft
$0.15*D_2$ =	3.00	ft
H_3 =	1.00	ft
# Dentates =	18.00	

392 DESIGN OF SMALL DAMS



Froude # X/D2

Figure 9-40. Characteristics for alternative low Froude number stilling basins. 103 D 1876.

Appendix G – Lake Drain Calculations

Marquette Lake Dam TIME-STEP DRAWDOWN CALCULATION	Done By: NCC 11/3/2020 Check By: WJK 11/5/2020

Constant inflow =	13.5	cfs			
Starting El. =	509.26				
Time Step =	0.1	hr =	360	sec	

Target Water Remaining =	88%	
Target Water Remaining =	3,482,577	ft^3
Time to Drain =	0.1	days
"Drained" WSEL =	507.22	ft
Water Remaining =	3,440,873	ft^3

Note: All elevations reference the NAVD 88 vertical datum.

Time		Start Volume	Reservoir El.	Discharge	End Volume		Pool Drop	Drop Rate
(Hours)	(Days)	(ft^3)	(ft)	(cfs)	(ft^3)		(ft)	(ft/day)
0	0.00	3,939,566	509.26	108.0	3,905,564	Max =	0.15	35.75
0.1	0.00	3,905,564	509.11	107.7	3,871,669		0.15	35.75
0.2	0.01	3,871,669	508.96	107.4	3,837,883		0.15	35.64
0.3	0.01	3,837,883	508.81	107.0	3,804,205		0.15	35.53
0.4	0.02	3,804,205	508.67	106.7	3,770,635		0.15	35.41
0.5	0.02	3,770,635	508.52	106.5	3,737,173		0.15	35.30
0.6	0.03	3,737,173	508.37	106.1	3,703,819		0.15	35.18
0.7	0.03	3,703,819	508.23	105.8	3,670,574		0.15	35.07
0.8	0.03	3,670,574	508.08	105.5	3,637,436		0.15	34.96
0.9	0.04	3,637,436	507.94	105.3	3,604,406		0.15	34.84
1.0	0.04	3,604,406	507.79	104.9	3,571,484		0.14	34.73
1.1	0.05	3,571,484	507.65	104.7	3,538,670		0.14	34.62
1.2	0.05	3,538,670	507.50	104.4	3,505,963		0.14	34.50
1.3	0.05	3,505,963	507.36	104.1	3,473,364		0.14	34.39
1.4	0.06	3,473,364	507.22	103.8	3,440,873		0.14	34.28

Click to hideNews Bulletins

- **NOTICE** November 1, 2020 7:45 am ET: We are investigating some real-time data currently behind on the web.
  - **UPDATE** November 1, 7:15 pm ET: Real-time data delivery to NWISWeb has been restored at this time. We are continuing to monitor the situation for any further issues.
- Explore the **NEW** USGS National Water Dashboard to access real-time data from over 13,500 stations nationwide.
- Full News

USGS Surface-Water Monthly Statistics for the Nation

The statistics generated from this site are based on approved daily-mean data and may not match those published by the USGS in official publications. The user is responsible for assessment and use of statistics from this site. For more details on why the statistics may not match, click here.

USGS 01572950 Indiantown Run near Harper Tavern, PA

Available data for this site: Time-series: Monthly statistics GO

	Output formats
Lebanon County, Pennsylvania Hydrologic Unit Code 02050305 Latitude 40°26'20", Longitude 76°35'55" NAD27 Drainage area 5.48 square miles	HTML table of all data Tab-separated data Reselect output format

00060, Discharge, cubic feet per second,												
YEAR	Monthly mean in ft3/s (Calculation Period: 2002-09-01 -> 2014-09-30)											
	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
2002									1.92	6.04	12.7	16.7
2003	14.5	6.73	28.6	16.8	12.6	22.3	5.11	10.4	13.2	15.9	18	23.9
2004	10.1	9.16	12.4	22.6	10.5	7.18	7.9	12.5	37.5	7.47	14.4	26.3
2005	24.2	10.1	19.8	23.7	5.61	5.15	11.8	2.81	1.73	12.9	5.4	11.9
2006	25.1	22.2	5.4	6.82	7.66	22.1	8.73	2.84	6.22	9.96	19.8	9.45
2007	15.5	5.07	22.1	17.4	5.28	2.87	2.11	2.88	1.54	2.27	4.73	11.3
2008	7.66	24.6	34.6	10	14.5	4.34	2.32	1.37	2.45	4.94	5.31	27.1
2009	8.7	5.57	4.27	17.5	14.7	14.6	4.61	10.5	7.97	14.6	10.6	18.9

2010	14.8	8.48	20.2	9.97	9.79	2.84	3.37	1.69				
2013										8.94	5.37	10.7
2014	12.9	6.74	14.2	22.9	18	9.97	3.37	3.44	2.17			
Mean of monthly Discharge	15	11	18	16	11	10	5.5	5.4	8.3	9.2	11	17
** No Incomplete data have been used for statistical calculation												

Questions about sites/data?
Feedback on this web site
Automated retrievals
Help
Data Tips
Explanation of terms
Subscribe for system changes
News