SOUTH FORK LICKING RIVER

Flood Damage Reduction Planning Study

Prepared for:

South Licking Watershed Conservancy District

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EMH&T

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ABBREVIATIONS

BCA – Benefit Cost Analysis
BCR – Benefit to Cost Ratio
DEM – Digital Elevation Model
EIS – Environmental Impact Statement
EPA – Environmental Protection Agency
FEMA – Federal Emergency Management Agency
FIRM – Flood Insurance Rate Map
FIS – Flood Insurance Study
FMSM – Full Mossbarger Scott & May
HUC – Hydrologic Unit Code
Mcf – Million Cubic Feet
MWCD - Muskingum Watershed Conservancy District
NRCS – Natural Resources Conservation Service
NWS – National Weather Service
ODNR – Ohio Department of Natural Resources
ODOT – Ohio Department of Transportation
PMF – Probable Maximum Flood
PMP – Probably Maximum Precipitation
PWM – Partners in Watershed Management
RCN – Runoff Curve Number
RMSE – Root Mean Square Error
SCS – Soil Conservation Service
SFLR - South Fork Licking River
SLWCD - South Licking Watershed Conservancy District
Sq. Mi. – Square Miles
SSA – Storm Sanitary Analysis (model)
SWCD – Soil and Water Conservation District
SWMM – Stormwater Management Model
Tc – Times of Concentration
USACE – U.S. Army Corps of Engineers
USGS – U.S. Geological Survey

EXECUTIVE SUMMARY

The South Fork Licking River watershed has a history of repeated and extreme flooding. Flooding in 1959 partially inundated the recently completed I-70 roadway between State Route 37 and 79. Area-wide flooding throughout the watershed has occurred on multiple occasions over the past three years. Prior studies have investigated flood damage reduction solutions with various outcomes, including studies by the Soil Conservation Service (SCS) and then the Natural Resources Conservation Service (NRCS), working collaboratively with the South Licking Watershed Conservancy District (SLWCD). The SCS study resulted in a Watershed Work Plan for implementation by the SLWCD, but the process of implementation was not completed due to resistance to the associated property owner assessments.

The NRCS study proposed to significantly change SLWCD's Watershed Work Plan, but the study was not finalized due to constructability concerns with the proposed flood damage reduction measures. Other studies have been completed related to developing flood damage reduction solutions, including a recently completed report issued by ms consultants working on behalf of the Licking County Commissioners. The study documented in this report is a collaborative effort with ms consultants to develop hydrologic and hydraulic models for the South Fork Licking River (SFLR) watershed, using information from the past studies and updated with best available data.

The hydrologic modeling developed as part of this study is separated between the portion of the SFLR watershed to the U.S. Geological Survey (USGS) stream gauge at Kirkersville, and the portion of the watershed that discharges to and through Buckeye Lake. There are unique elements of those watershed areas that required the use of different hydrologic modeling platforms. These models have been used to provide hydrologic inputs to a 2D HEC-RAS model developed by ms consultants, which then provides calculated flood elevations and flood inundation areas along SFLR between the Village of Heath and the Village of Kirkersville. All of the modeling is complex in nature and represents the inclusion of a significant amount of data representing the SFLR watershed and the river channel through the study area.

The current focus in flood damage reduction solutions is the use of dry dams to create reginal stormwater detention basins along SFLR and its major tributaries. Dry dams capture flood flows from the upstream watershed area and control the rate of flow through the dam to the downstream watershed. Strategic locations for dry dams were evaluated based on watershed topography and disbursement of the dry dams thought the watershed, in locations that would capture a significant watershed area. Eight dry dams were originally identified and one of those was eliminated from consideration due to minimal incremental benefits. The remaining seven dry dams require additional optimization to potentially eliminate some while refining others to maintain the flood damage reduction benefits while reducing construction costs.

A preliminary analysis of flood damage reduction benefits and construction costs determined that a Benefit to Cost Ratio (BCR) of 1.0 or higher was not achieved for the seven dry dams. This can be partially attributed to the need for optimization and refinement of the dry dams as alluded to above, but is also attributed to the fact the majority of the flooded areas along SFLR are agricultural fields, as opposed to buildings and other similar assets. Determining and including 'indirect' benefits not specifically associated with flooded land and buildings would also improve the BCR outcome.

This study also includes information pertaining to a Channel Maintenance Plan for SLWCD. The purpose of this plan is to provide SLWCD with a program for identifying and prioritizing log jams for removal from the SFLR and major tributary channels. The program includes a score card spreadsheet for evaluating and comparing individual log jams based on both desktop and field-based data. The program also includes mapping of an easement corridor along 3rd through 5th order channels throughout the SFLR and Raccoon Creek watersheds, for the purpose of allowing the SLWCD to inspect and maintain these channels on an annual basis.

The goal of this study was to set in motion a process for the SLWCD to update the original Watershed Work Plan based on a program for flood damage reduction measures within the SFLR watershed. Providing a similar study for the Raccoon Creek watershed would be necessary to have a complete and updated Watershed Work Plan. The current findings have determined that a work plan based only on the use of dry dams for flood damage reduction may not be practical, without further analysis to both increase benefits and reduce costs. However, the development of the complex hydrologic and hydraulic models resulting from the study and the work completed by ms consultants offers SLWCD and other watershed stakeholders with a powerful tool to understand the impacts of changes in the watershed on area-wide flooding.

1.0 INTRODUCTION

This report provides a summary of the study process to identify and evaluate measures to reduce the risk of flooding and the associated damages within the South Fork Licking River (SFLR) watershed, as depicted on Figure 1-1. The watershed study area does not include the Raccoon Creek watershed, which confluences with the SFLR just upstream of where it merges with the North Fork Licking River. The total watershed size included within this study area is 185 square miles (sq. mi.), including 44 sq. mi. that is directly tributary to Buckeye Lake. The watershed area includes the 12-digit Hydrologic Unit Codes (HUC) listed below.

- 050400060401 – Muddy Fork Sub-Watershed
- 050400060402 – Headwaters South Fork Licking River
- 050400060403 - South Fork Licking River (Kirkersville)
- 050400060404 – Buckeye Lake Reservoir Feeder (Feeder Canal)
- 050400060405 – Buckeye Lake
- 050400060406 – South Fork Licking River (Bell Run)
- 050400060407 – Ramp Creek
- 050400060408 – Dutch Fork
- 050400060409 – South Fork Licking River (Beaver Run)

The SFLR watershed lies within Licking and Fairfield Counties, and a small portion of Perry County. The service area for the South Licking Watershed Conservancy District (SLWCD) includes the SFLR watershed described above, plus the adjacent Racoon Creek watershed. This study was prepared in cooperation with the SLWCD and the Licking County Soil and Water Conservation District (Licking County SWCD). The study was partially funded by a Partners in Watershed Management (PWM) grant from the Muskingum Watershed Conservancy District (MWCD). The study process began in March of 2022, and included the major components listed below:

- Watershed stakeholder involvement, which included public meetings to initially describe the goals of the study and then to present the results and conclusions of the study.
- Development of a Channel Maintenance Plan for the major watercourses in both the SFLR and Raccoon Creek watershed areas.
- Development of a hydrologic model for the SFLR watershed and evaluation of flood damage mitigation alternatives, also utilizing a 2D HEC-RAS model of a portion of the SFLR, as prepared by ms consultants.

The study process has included extensive coordination with individuals and agencies with knowledge of the SFLR watershed and previous similar flood mitigation studies. Namely, the U.S. Geological Survey (USGS) provided previously developed models for the SFLR and data from gauging stations located along that watercourse. The process of calibrating the models developed as part of the current study was closely coordinated with USGS staff. Other contributors were the SLWCD Board of Directors, the National Weather Service (NWS) and representatives of the Fairfield and Perry County Soil and Water Conservation Districts (SWCDs), as well as the Natural Resource Conservation Service (NRCS). The Ohio Department of Natural Resources (ODNR) provided information regarding the rehabilitation and operation of the Buckeye Lake Dam.

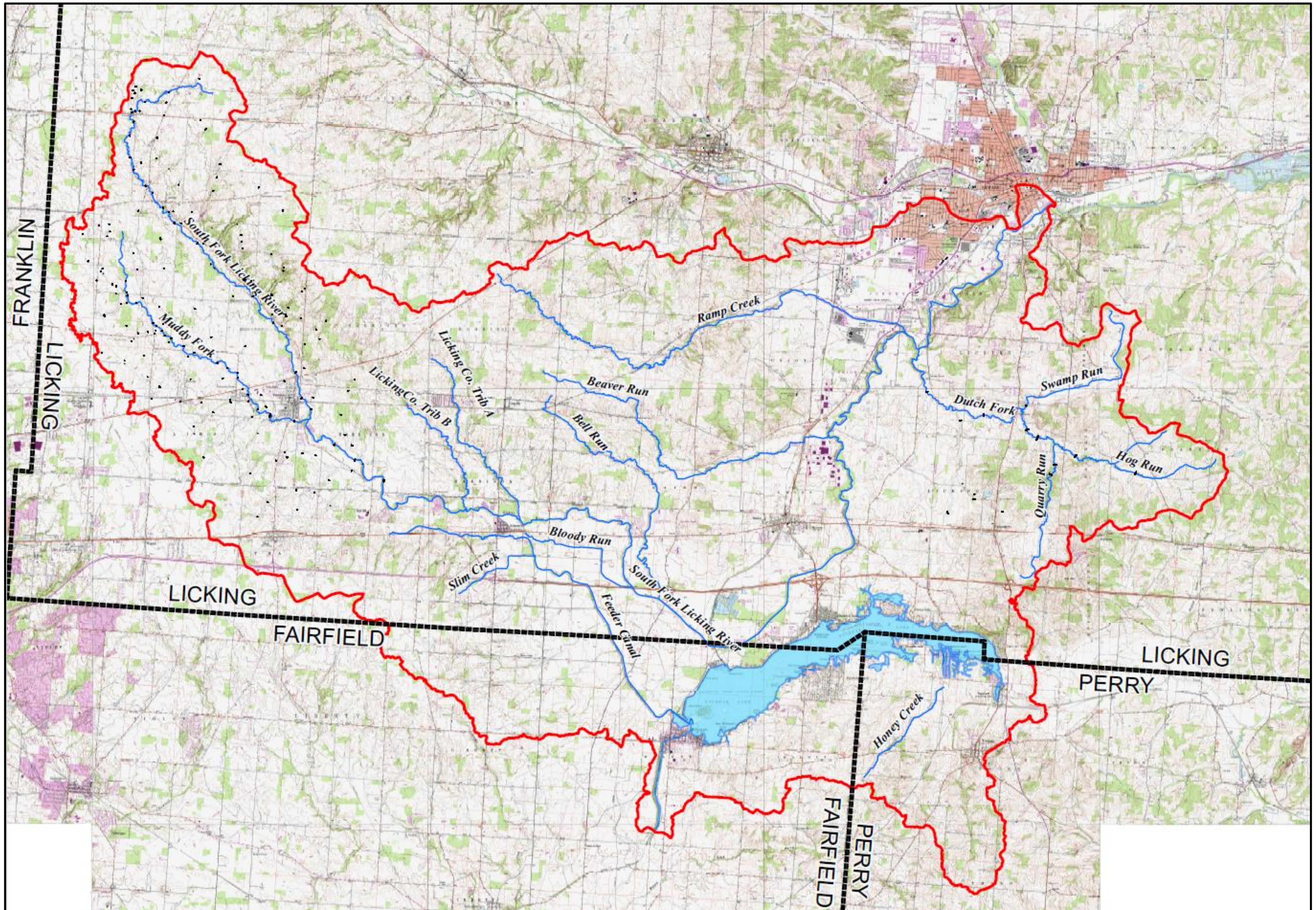


FIGURE 1-1
South Fork Licking River Watershed

There have been previous studies of the South Fork Licking River for the purpose of identifying existing flood hazard conditions, and identifying flood damage reduction measures, some of which are summarized below. Report documentation from these past studies is provided in an electronic archive accompanying the report.

- Soil Conservation Service (1980 – 1983): Working in coordination with SLWCD, the Soil Conservation Service (SCS) issues their initial Environmental Impact Statement (EIS) and Watershed Work Plan in 1980; which identifies three "Floodwater Retarding Reservoirs" in the SFLR watershed, as well several channel improvements projects, including a Bypass Channel along SFLR north of I-70. This study included other recommended flood damage reductions measures specific to the Raccoon Creek watershed. There were subsequent addendums to this study issued in 1980, 1981 and 1983. Based on information provided by Dan Blatter (SLWCD Board of Directors), the Watershed Water Plan was not implemented due to opposition to the assessment process required to fund construction of the watershed improvements. The modeling associated with this study was not found as part of the data collection process.

  Refer to the SCS report entitled *Watershed Plan and Environmental Impact Statement for South Fork Licking River,* June 1980; and subsequent addendum documents, 1980, 1981, 1983.

- Full Mossbarger Scott & May (FMSM) (2003 – 2004): Working in coordination with ODNR, FMSM performed a study of the SFLR and Raccoon Creek watersheds, using the HEC-HMS and HEC-RAS model platforms, using some of the model data from the SCS 1980 study, and updating the modeling to represent existing conditions. The modeling was calibrated to a 1997 flood event using limited available data from that flood event. The modeling had several purposes, including an analysis of the flood impacts of the I-70 bridges and roadway on flooding along SFLR, and an analysis of the flood impacts of the improvements to the Sellers Point spillway at Buckeye Lake (ODNR). The FMSM study evaluated several channel improvements projects (and modifications to the outlets from Buckeye Lake), with a focus on reducing the flood impacts of the Sellers Point spillway improvements. The potential improvements evaluated by FMSM included the I-70 Bypass Channel previously considered in the SCS study, and adding flood carrying capacity along SFLR. The latter improvements, which included a constructed floodplain bench extending between the Buckeye Lake emergency spillway channel and downstream of I-70, has previously been implemented by ODNR. The FMSM study determined the implementation of the I-70 By-pass Channel would increase flood elevations downstream along SFLR extending to the confluence with the North Fork Licking River in Newark. The modeling associated with this study was not found as part of the data collection process.

  Refer to the FMSM report entitled *South Fork Licking River Watershed Initiative, project #DNR 736 736-98-011, Final Report,* May 2003.

- Fuller Mossbarger Scott & May (2005): Working on behalf of the Federal Emergency Management Agency (FEMA), FMSM developed hydrologic (HEC-HMS) and hydraulic (HEC-RAS) models for the purpose of updating the published Flood Insurance Rate Map (FIRM) and Flood Insurance Study (FIS) for Licking County and incorporated areas. EMH&T had previously obtained these models from Licking County. They appear to be the basis for the currently published flood hazard data for the SFLR and Raccoon Creek. The two models could not be executed using current versions of the same software and the support mapping information lacked sufficient spatial references; however, some of the hydrologic parameters in the HEC-HMS model were applied to the current study, for the eastern-most sub-watershed areas.

- Natural Resources Conservation Service (2009 – 2010): Working in coordination with SLWCD, and with the cooperation and support of the Licking, Fairfield, and Perry County SWCD's, the Natural Resources Conservation Service (NRCS) prepared a draft update to the 1980 EIS and Watershed Work Plan. It appears the NRCS largely relied on the FMSM models (2003 – 2004) to develop watershed-scale flood damage reduction solutions. The initial recommendation of this study was a large-scale regional floodwater detention structure on-line with SFLR and Bell Run (Swamp Road Floodwater Detention Structure), a dry dam facility located immediately upstream of the western 1-70 bridge over SFLR. The recommended floodwater detention structure was paired with the I-70 Overflow (By-pass) Channel referenced previously, which resulted in a significant reduction in flood hazards downstream of the western I-70 bridge and extending to the eastern I-70 bridge along SFLR. Another channel improvement along SFLR near the Village of Hebron was also recommended. Subsequently, the NRCS performed a geotechnical investigation of in-situ soils at the location of the proposed dam (for the regional detention basin) and along the By-pass Channel corridor, which resulted in a finding the soils would compromise the construction and long-term stability of the flood damage reduction measures.

  The NRCS study process continued from that point with additional investigations considering other flood damage reduction measures, including a levee system along the south and east side of the SFLR to protect areas between SFLR and Buckeye Lake from flooding, mainly the lakeside area and along SR 360 west of No Name Creek. The July 2009 draft report prepared by NRCS documents the aforementioned regional detention basin and 1-70 By-Bass Channel. This document was not updated and finalized by NRCS. The modeling associated with this study was not found as part of the data collection process.

  Refer to the NRCS report entitled *Draft Supplemental Watershed Work Plan, Supplemental Environmental Impact Statement, South Fork Licking River,* July 2009.

- USGS (2012): Working in coordination with the NWS, the USGS prepared detailed HEC-RAS models along portions of SFLR and Raccoon Creek. These models were integrated into a flood prediction modeling system developed by the NWS to enhance the rating curves developed for the stream gauges along both watercourses that are also a component of the NWS flood prediction system. The HEC-RAS model for the SFLR was utilized in the development of the new hydrologic modeling prepared as part of this study (validation of the calibration process) and the 2D HEC-RAS model prepared by ms consultants.

  Refer to the USGS report entitled *Development of a Flood Warning System and Flood Inundation Mapping in Licking County, Ohio* (SIR 2012-5137).

- Tetra Tech and Gannett-Fleming: Working in coordination with ODNR related to improvements to the Buckeye Lake dam, various models were developed to evaluate and document the hydrology of the Buckeye Lake watershed and the larger SFLR watershed.

Models obtained during the data reconnaissance phase are a XPSWMM model, HEC-1 model and a HEC-RAS model. These models were reviewed and determined to be either non-working using current versions of the same computer programs or of insufficient detail. As described in Section 5 of this report, some of the hydrologic parameters associated with the Buckeye Lake sub-watersheds for the current study were derived from the HEC-1 model developed by Tetra Tech.

Refer to the Tetra Tech report entitled *Buckeye Lake Dam South Fork Licking River Watershed Technical Report* (ODNR File No. 9723-004), dated October 2015; and the Gannett Fleming Report entitled *Final Hydraulic and Hydrologic Report, Buckeye Lake Dam* (project Number DNR 150080), dated March 2018.

The study process completed by EMH&T focuses on developing hydrologic models for the SFLR watershed, with the purpose of providing hydrologic inputs in the form of calculated hydrographs to a 2D HEC-RAS model prepared by ms consultants. The combined hydrologic and hydraulic models provide an accurate representation of existing flood hazard conditions and also allow for the evaluation of flood damage reduction alternatives. The goal of this evaluation is to identify solutions with flood damage reduction benefits exceeding the cost of constructing recommended improvements. Based on the previous studies, the development of flood damage reduction alternatives focuses on hydrologic solutions in the form of regional detention basins, either along the SFLR main stem or major tributary watercourses, along with targeted bridge improvements. The regional detention basins can provide watershed-scale flood protection, while channel improvements only address flooding conditions in immediate proximity to those improvements.

The channel maintenance plan developed as part of this study serves as a complement to the development of flood damage reduction measures in that it addresses recurring channel blockages (e.g., log jams) that have been documented to contribute to localized flooding and channel erosion. Licking County SWCD completed a desk top investigation of log jam locations within the SFLR watershed in 2020. The current study builds on their effort by updating the findings of the 2020 investigation and developing tools for SLWCD to evaluate and prioritize the removal of log jams on an on-going basis, as well as identifying watercourse easements to allow SLWCD to access channels for the purpose of annual inspections and maintenance activities.

This report represents a preliminary investigation of potential flood damage reduction solutions for the SFLR watershed, with an emphasis on regional stormwater detention basins. The investigation of alternatives did not consider alterations to the I-70 bridges or roadway to address recurring flooding, as that is the focus of an on-going study authorized by the Ohio Department of Transportation (ODOT). Further study efforts will be required to refine the current analysis of flood damage reduction solutions in the SFLR watershed and possibly consider combinations of regional detention basins and improvements to I-70 to optimize the Benefit Cost Ratio (BCR).

2.0 BACKGROUND

The SLWCD seeks to use this study and further studies to identify channel maintenance activities and structural flood damage reduction solutions as part of a future update to the original EIS and Watershed Work Plan. Adoption of the updated plan will provide SLWCD with actionable items that could be implemented in phases as funding becomes available.

2.1 Problem Statement

The draft supplemental EIS developed by the NRCS contained the problem statement below:

Flooding is a major problem in the South Fork Licking River watershed, causing extensive damage to the Villages of Buckeye Lake and Hebron. The 100-year flood will inundate over 50 percent of the Village of Buckeye Lake and 15 percent of Hebron. Approximately 4,346 acres and 471 buildings (barns/garages, homes, trailers, commercial businesses) are affected by a 100-year flood event on the south side of U.S. 40 in the watershed. Floods in the area also adversely impact transportation facilities. I-70 lanes in this area and/or ramps to State Route (SR) 79 have recently been blocked by floods in 1997, 2004, twice in 2005, and twice in 2008. In the last 40 years, I-70 in the Buckeye Lake area has been closed or the exit ramps to SR 79 have been blocked 11 times.

More recently, I-70 has been flooded in 2020 and 2021, and as recently as May 5, 2022. The extensive nature of the floodplain along the portion of SFLR south of US 40, represented in Figure 2-1, has a significant impact on the land use potential for this area. The Villages of Hebron and Heath have separately considered their own flood hazard mitigation plans to address recurring flooding in their communities. Flooding of agricultural fields can occur frequently and for an extended period of time. The direct economic losses due to flooding of farm land has not been determined as part of this study; however, the benefits of reduced flooding to all open land has been considered.

Addressing wide-spread flooding on a watershed scale requires a comprehensive area-wide model to determine the sources of and the cause of flooding, which is often a combination of excessive stream flows combined with capacity limitations in the watercourse. In the SFLR watershed, where the extent of flooding is the greatest south of US 40, there is a convergence of flows from the upstream watershed, overflows from the Buckeye Lake Feeder Canal and from the Buckeye Lake spillways; combined with a very low channel gradient along the SFLR watercourse. The low and broad floodplain along this reach serves as storage for the flood flows that exceed the capacity of the SFLR channel, also impacted by hydraulic restrictions at some of the roadway bridges. The nature of flooding in this area is consistent with the historical nature of the Buckeye Lake area as a post-glacial lake and then as a large swamp.

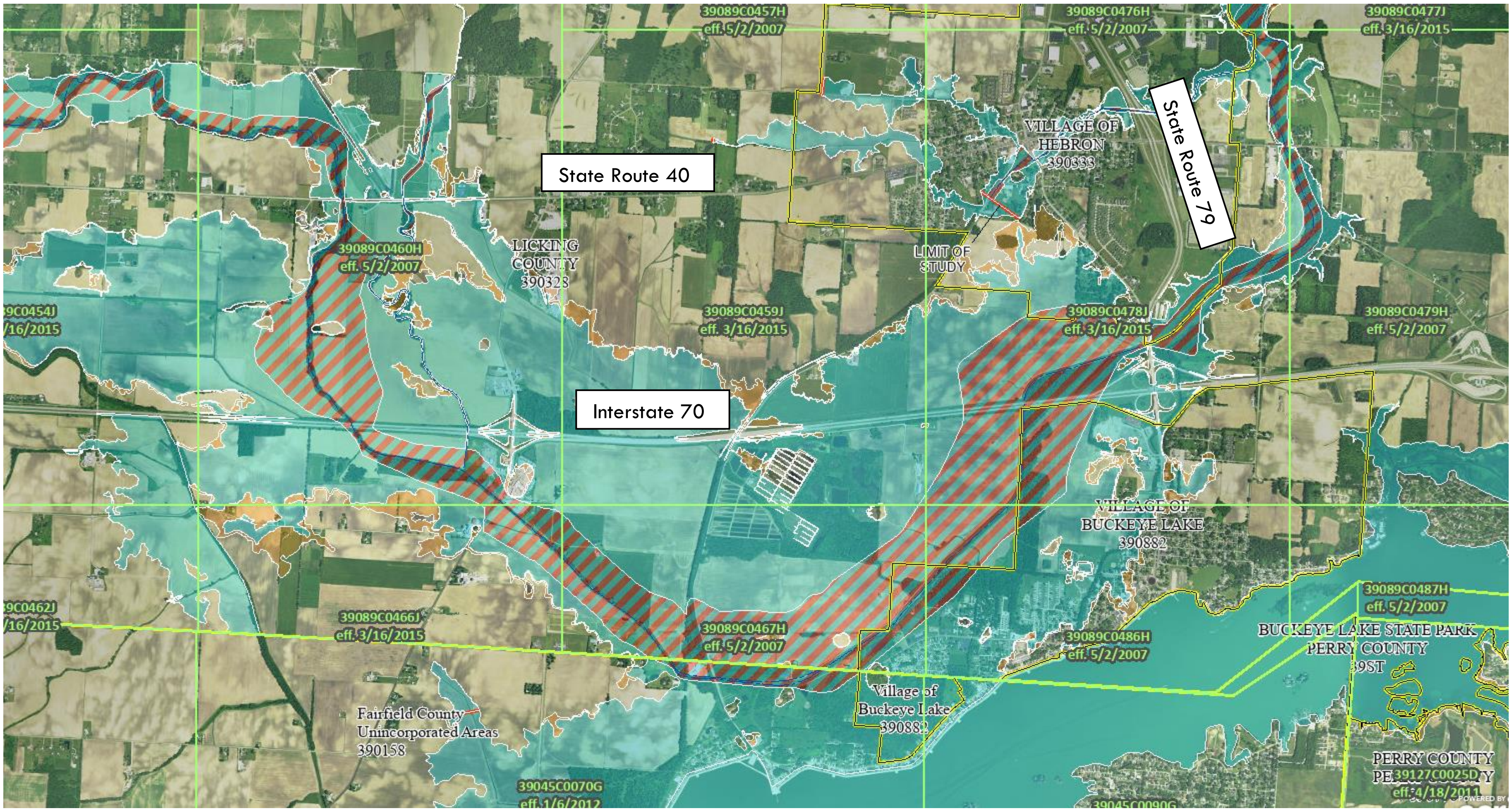


FIGURE 1-2
Published Flood Hazard information

Photo: I-70 Flooding (1959); Source – SCS Original EIS and Watershed Work Plan

Photo: I-70 at SR 79 Flooding (2005);
Source – NRCS Draft Work Plan Update

Photo: Village of Buckeye Lake Flooding
Source - NRCS Draft Work Plan Update

Photo: Flooded Agricultural Field
Source - NRCS Draft Work Plan Update

Photo: Flooded Residential Property
Source - NRCS Draft Work Plan Update

Channel maintenance to remove log jams and other debris blockages in the SFLR channel and other major tributaries is a key concern of the SLWCD and other watershed stakeholders. Past and current log jams have caused channel erosion, sometimes to the extent of causing a significant shift in the channel alignment. The most impactful log jam is located along SFLR near the Village of Hebron (downstream of US 40). This log jam has existed in some form for over a decade and has significantly altered the watercourse. Flooding associated with log jams is most significant when in proximity to a bridge opening; otherwise, the flood impacts may be less obvious in areas of low, broad floodplains adjacent to the affected channel.

2.2 South Licking Watershed Conservancy District

SLWCD was originally established in 1968 and was formed under Section 6101 of the Ohio Revised Code. The conservancy district is presided over by a Conservancy Court, consisting of a judge representing each of the three counties included within the district's boundary – Licking, Fairfield and Perry Counties. SLWCD is managed by an appointed Board of Directors, consisting of three members. The boundary of the conservancy district includes the watersheds of the main stem of SFLR and Raccoon Creek, to the confluence with North Fork Licking River. Within this boundary are numerous townships, Buckeye Lake and surrounding communities (Village of Buckeye Lake, Thornville and Millersport), the Cities of Heath, Johnstown, Pataskala, and portions of New Albany and Newark, and the Villages of Alexandria, Granville, Hebron and Kirkersville. Figure 2-1 indicates the various incorporated communities relative to the boundary of the SLWCD.

As a corporation of the State of Ohio, SLWCD has the ability to implement projects on a watershed scale for the benefit of all residents and communities within their jurisdiction. With the adoption of a Watershed Work Plan, approved by the Conservancy Court, the conservancy district can acquire land and easements, and levy an assessment, to implement flood damage reduction measures recommended by the Plan.

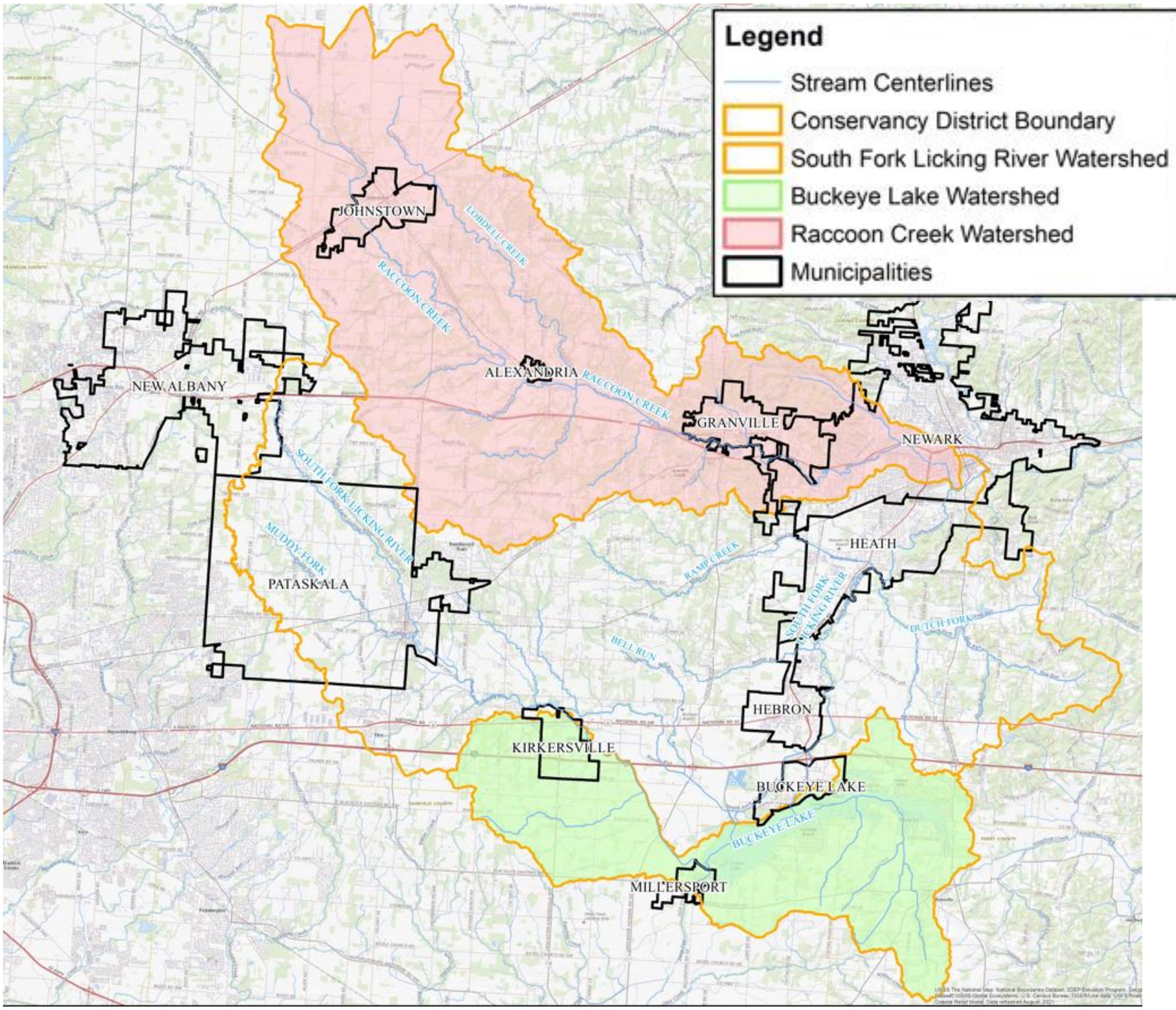


FIGURE 2-1
Incorporated Communities in SLWCD Boundary

3.0 DATA COLLECTION

The effort of data collection focused on the retrieval of previous hydrologic and hydraulic models for the SFLR watershed, and the data supporting those models. As documented in the introduction, numerous previous studies have been performed, but the models associated with most of those studies were not found during the data collection process. The models prepared by FMSM for their 2003 – 2004 studies and then subsequently reapplied for the NRCS studies in 2009-2010 appear to be the most relevant to the current study. Separate attempts by EMH&T and ms consultants were not successful in finding those models and related electronic data files. Other data collection efforts are described below.

3.1 Licking, Fairfield, and Perry County GIS Data

Working on a watershed basis requires mapping data best suited for compilation within an ArcGIS mapping and database platform. GIS-based data was sought and obtained from the Licking, Fairfield, and Perry County Auditor's departments. This data served as the foundation for a majority of the modeling and other study elements, including the development of project alternatives related to flood damage reduction.

3.1.1 Parcel Data

The Licking, Fairfield, and Perry County Auditor's Offices maintain county-wide mapping and a database of parcel data to facilitate tax assessments. The mapping includes a digital representation of property lines, legal ownership, buildings, and property and building valuations. This information was used for multiple purposes, including to identify property owners impacted by the proposed flood damage reduction measures, as well as to determine the cost of land required for the channel maintenance plan and the flood damage reduction measures, and the valuation of benefits to individual parcels and buildings associated with the reduction in flood hazard areas.

3.1.2 Elevation Data

A description of the elevation data obtained for the SFLR watershed area, including portions of Licking, Fairfield, and Perry Counties, is provided below for each county.

Licking County: LiDAR collected by Woolpert in April 2015. The documented horizontal accuracy is 1.418 feet (+/-) at 95% confidence interval and a vertical accuracy Root Mean Square Error (RMSE) of 0.202 feet and under the fundamental vertical accuracy guidelines is 0.396 feet (NAVD '88). The LiDAR was used to produce a one-meter DEM in August of 2015 and two-foot and four-foot interval contours in 2016.

Fairfield County: LiDAR collected by Woolpert in 2015 and appears to have the same metadata as the Licking County mapping. The LiDAR data was used to generate two-foot interval contours.

Perry County: OSIP LiDAR data collected by Woolpert in 2007. The captured LiDAR had a +/- 1-foot vertical accuracy at 95% confidence interval using NAVD '88; two-foot interval contours were created from the LiDAR data.

The elevation data obtained for the three counties and covering the area of the SFLR watershed was used to refine the sub-watershed area boundaries and to develop Root mean square error boundaries for the purpose of the hydrologic modeling, and to develop other hydrologic parameters. The elevation data was also used to identify the extents of the 100-year floodplain and the properties and buildings within the floodplain. The elevation data was essential to the development of flood damage alternatives; the existing topographic contours were used to develop preliminary grading plans for dry dam locations described in Section 6 of this report.

3.1.3 Planimetric Data

Planimetric data, including building footprints, road centerlines, railroads, rivers, creeks, and other bodies of water, were also provided within the GIS data obtained from the three counties. This planimetric data helps to identify potential flood damages to buildings and roads based on the floodplain modeling. To be able to determine property and building flood damages and benefits, valuation information from the Auditors database for parcels was paired with the parcels and buildings. For parcels that only had one building, this process was straight forward, and the improvement value in the database was assigned to the building polygon. For parcels that had multiple buildings, values for each building needed to be determined or refined from what the existing database provided. Accessory structures, such as garages and small outbuildings were assumed to have no value.

3.1.4 Political Boundary Data

Political boundary data includes municipal corporate limits and township boundaries. This type of data was used to identify municipal entities affected by existing flooding and the impacts of the flood damage reduction measures to those entities.

3.2 Bridge Data and Record Plans

The Licking County Engineer’s Office maintains the county roads and bridges within the portion of the county outside of the incorporated communities. There may also be agreements for the County Engineer to maintain county routes within an incorporated community. As part of this project, the Licking County Engineer’s Office provided a list of bridges with an overall condition rating and indication of if they were known to have had issues with log jams. The list contained 17 bridges within the limits of the 2D model area SFLR watershed, and most were in good to excellent condition. Three were reported to have had previous log jams, including: 1) Outville Road at SFLR; 2) Canal Road at an unnamed tributary to SFLR (south of I-70); and 3) Gale Road at SFLR. In addition, bridge inspection reports were obtained from the Ohio Department of Transportation (ODOT). The intent of obtaining this information was to identify bridges along the SFLR and major tributaries that may be targeted for improvements due to their overall condition and the opportunity to mitigate localized flooding attributed to an under-sized bridge or one that may be prone to capturing woody debris. Ultimately, the focus of this study is on the local and county-maintained bridges for the purpose of evaluating flood damage reduction measures.

Record plans for various bridges and culverts throughout the SFLR watershed were also obtained from the Licking County Engineer’s Office and other entities (e.g., City of Pataskala, ODOT). The plans were evaluated as part of the process of determining whether to add incidental storage to the new hydrologic modeling prepared as part of this study.

3.3 Model and Related Data

The Introduction of this report provides a summary of previous studies that would have generated hydrologic and/or hydraulic modeling for the SFLR watershed. The previous models developed by the SCS (1980) and then the NRCS (2009-2010) were not found as part of this effort, with the latter being related to a FMSM study (2003-2004). The HEC-1 modeling prepared by Tetra Tech under contract to ODNR was a reference in developing hydrologic parameters for the Buckeye Lake sub-watershed, and a HEC-HMS model prepared by FMSM (2005) under contract to FEMA was a reference in developing hydrologic parameters for the eastern-most sub-watersheds in the SFLR watershed.

The USGS (2012) unsteady state model was utilized by ms consultants in their preparation of the 2D HEC-RAS model for a portion of the SFLR watercourse. The USGS model provided supplemental geometric data for the SFLR channel, including bridges. The USGS model was also used as part of the current study to validate the hydrologic model calibration to the USGS Kirkersville stream gauge, as described in Section 5 of this report. Other studies revealed through the data collection process are listed below.

1. A Hydrologic and Hydraulic Floodplain Analysis for the Village of Hebron, dated March 2022, prepared by V3 Companies. This study was completed as part of an effort to identify the causes of and remedies for flooding along four tributary streams through the Village of Hebron. This study represents a more detailed evaluation of the subcatchment areas within and surrounding the Village of Hebron. The portion of the SFLR watershed lies within the 2D HEC-RAS model area (ms consultants).

2. Various HEC-RAS models obtained from ODNR prepared in support of delineating Approximate (Zone A) 100-year floodplain boundaries on the FIRM for Licking County and incorporated areas.

3.4 Utility Data

The study process did not include a comprehensive investigation of utilities throughout the SFLR watershed. However, major underground pipeline and overhead (transmission line) utility owners were contacted to acquire information about their utility locations and consider those in the location and alignment of flood damage reduction measures. More comprehensive utility investigations would be necessary as part of future advanced studies to identify utility locations and potential conflicts. Some of the utility owners contacted as part of this process are Marathon Pipe Line, AT&T Transmission, The Energy Cooperative (NGO Gas). Information from AEP Transmission and Columbus Gas was not obtained. In addition, information regarding the future potential solar farm located in Harrison Township (Union Ridge Solar Field) was obtained through the Ohio Power Siting Board.

3.5 Soil and Water Conservation District Data

3.5.1 Licking County SWCD

The Licking County SWCD has been a continuous source of data and support throughout the study process. Notably, the LICKING COUNTY SWCD provided documentation from their 2020 log jam assessment that became the foundation for the channel maintenance plan documented in this report. In addition, the LICKING COUNTY SWCD provided drone imagery for the large log jam along SFLR at the Village of Hebron and photos of past flooding events in the SFLR and Raccoon Creek watersheds. The LICKING COUNTY SWCD also provided access to supplementary information associated with the original EIS prepared by the SCS (1980) study to develop a flood damage reduction plan for the SLWCD.

3.5.2 Fairfield County SWCD

The Fairfield County SWCD provided information specific to the Buckeye Lake sub-watershed, including providing guidance on the hydrologic conditions surrounding the lake and the Feeder Canal. The Fairfield County SWCD also provided supplemental information associated with the NRCS (2009-2010) draft study to update the original EIS and Watershed Work Plan, as well as tile mapping for specific areas within Fairfield County. The supplemental data to the NRCS study included correspondence pertaining to other flood damage reduction measures than were documented in the draft report.

3.5.3 Perry County SWCD

The Perry County SWCD also provided information specific to the Buckeye Lake sub-watershed. Specifically, they were consulted on the history and current condition of the Thornport outlet from the lake.

3.5 NRCS Data

Through Dan Blatter (SLWCD, Board of Directors), an extensive amount of data was acquired pertaining to the NRCS's draft update to the 1980 EIS and Watershed Work Plan. Additional coordination with the local NRCS office also occurred in an effort to obtain the hydrologic and hydraulic models used as part of that study. The eventual conclusion was that these models were performed by FMSM and were not in the possession of NRCS. The information that was obtained included the draft report, supporting technical documentation and miscellaneous calculations, mapping and correspondence files, as well as documentation pertaining to the establishment of project costs and benefits. Of significance is a geologic report prepared to document soils and groundwater conditions along and underneath the location of recommended flood damage reduction measures. This undated soils report is believed to have led to the decision to not finalize the updated EIS and Watershed Work Plan due to the determination of unsuitable soils conditions.

3.6 ODNR Data

ODNR was provided a public information request to retrieve reports and model data related to the Tetra Tech and Gannet Fleming studies pertaining to the Buckeye Lake watershed and the dam.

The reports and models previously referenced were obtained as a result of this request. In addition, discussions with ODNR staff familiar with the operation of the dam spillway occurred to better understand the seasonal operation changes and to interpret the Buckeye Lake gauge data used to calibrate the new hydrologic model for that sub-watershed. A separate Tetra Tech report, entitled *Kirkersville Feeder Canal Study and Report Verification*, DNR-19005, dated November 2020, was also obtained from ODNR. This report is very specific to identifying improvements to address physical impairments with the Feeder Canal. There is no apparent modeling associated with the Tetra Tech study; however, there appears to be field survey data for the channel and Bloody Run spillway which was not obtained as part of the data collection process.

4.0 CHANNEL MAINTENANCE PLAN

The main objectives of the Channel Protection Plan include: identification of woody debris fields (logjams) and design of scoring criteria guidelines to apply to the more significant of these locations, and identification of a long-term plan for the management and removal of logjams. In 2020, the Licking County SWCD completed an investigation of the location and severity of logjams along numerous watercourses throughout the SFLR watershed. The current effort expands upon the 2020 study to further develop the Channel Protection Plan. The items completed in support of the Channel Protection Plan are listed below.

1. A desktop analysis of aerial imagery to identify logjam sites.
2. Field reconnaissance of targeted logjam sites identified by the desktop analysis for the purpose of data collection and documentation.
3. Development of scoring criteria to apply to individual log-jams for the purpose of documenting severity and prioritizing logjam removal locations.
4. Mapping of an easement buffer along major watercourses to provide a maintenance access corridor, and identify easement acreages required from individual property owners along those watercourses.
5. Development of cost estimates for acquiring the identified watercourse easements.

Unlike the flood damage reduction study component of this project, the effort to develop a Channel Maintenance Plan for the SLWCD includes both the SFLR and Raccoon Creek watersheds.

4.1 Data Resources

The *South Fork Licking River Watershed Land Use Evaluation and Woody Debris Mapping* report was prepared by LICKING COUNTY SWCD in 2020. The woody debris (logjams) were mapped using Licking County Auditor aerial imagery from March of 2019. The original mapping of these logjams indicated a range of severity, from minor blockages to total blockages. The mapping area included both the SFLR and Raccoon Creek watersheds. The LSCWD report and other data, as outlined in Table 4-1, have been collected and applied to the analysis and development of this Channel Protection Plan.

TABLE 4-1
Data Resources

Document/Resource	Project Relevance and Use
South Fork Licking River Watershed Land Use Evaluation and Woody Debris Mapping report (Licking County SWCD, 2020)	Woody debris points used as base data to conduct additional logjam investigation for the SFLR watershed
Licking County Aerial (2021)	Stream digitization and stream order identification
Fairfield County Aerial (2020)	Stream digitization and stream order identification
Perry County Aerial (2019)	Stream digitization and stream order identification
National Hydrography Dataset (2020)	Stream polyline layer for stream digitization and stream order identification

TABLE 4-1
Data Resources

Document/Resource	Project Relevance and Use
South Licking Watershed Conservancy District Watershed HUC-12 Boundaries	Determine project boundaries and HUC for each logjam location
Licking County Auditor Parcel Data (December 2022, shapefile)	Shapefile boundary and parcel information to determine estimated easement area and cost
Fairfield County Auditor Parcel Data (December 2022, shapefile)	Shapefile boundary and parcel information to determine estimated easement area
Fairfield County Auditor Parcel Data (December 2022, online)	Parcel information to determine estimated easement costs
FEMA National Floodplain Hazard Layer (2015)	Determine flood zone of each log jam location

4.2 Desktop Review and Mapping

The stream centerline mapping from the 2020 National Hydrography Dataset was updated by digitizing new stream centerlines using the most current aerial imagery for the three counties as referenced in Table 4-1, refer Figure 4-1 for an example. Once stream centerlines were updated, each stream was reviewed for evidence of logjams using the aerial imagery. EMH&T established and documented the categories described below.

- *Dam*: visible man-made dam.
- *Logjam Small:* represents locations where woody debris accounts for approximately 10% to 40% of the stream channel.
- *Logjam Large*: represents locations where woody debris accounts for approximately 41% to 70% of the stream channel.
- *Logjam Very Large*: represents locations where woody debris accounts for approximately 71-90% of the stream channel; woody debris appeared to span the entire width of the stream channel.
- *Logjam Total Blockage*: represents locations where woody debris accounts for approximately 91-100% of the stream channel; woody debris appeared to span the width and a significant amount of length along the channel.

Point data was created for a total of 945 dam and logjam locations throughout the SFLR watershed. Table 4-2 summarizes the results in terms of the watersheds and the defined categories. These locations are represented in GIS shapefile data provided with this report.

EMH&T also determined and documented the stream order of the watercourses within the SFLR and Raccoon Creek watersheds. When determining stream order, 1st order streams are the smallest mapped headwater streams, progressing to larger, downstream portions of the SFLR and Raccoon Creek determined to be 4th and 5th order streams. 2nd order streams are identified where two 1st order streams confluence. Two 2nd order streams confluencing results in a 3rd order stream, and two 3rd order streams confluencing results in a 4th order stream. Finally, two 4th order streams confluencing results in a 5th order stream. A polyline GIS shapefile of ordered streams was created for the two watershed areas.

TABLE 4-2
Summary of Observed Logjam Locations

Category	Watercourse						
	SFLR	Ramp Creek	Beaver Run	Dutch Run	Muddy Fork	Bell Run	Feeder Canal
Man-made Dam	4	0	0	0	1	1	0
Logjam – Small	183	116	63	81	112	48	19
Logjam – Large	67	41	18	28	27	30	16
Logjam – Very Large	16	22	8	8	3	8	5
Total Blockage	4	7	3	0	1	4	1
TOTALS =	274	186	92	117	144	91	41

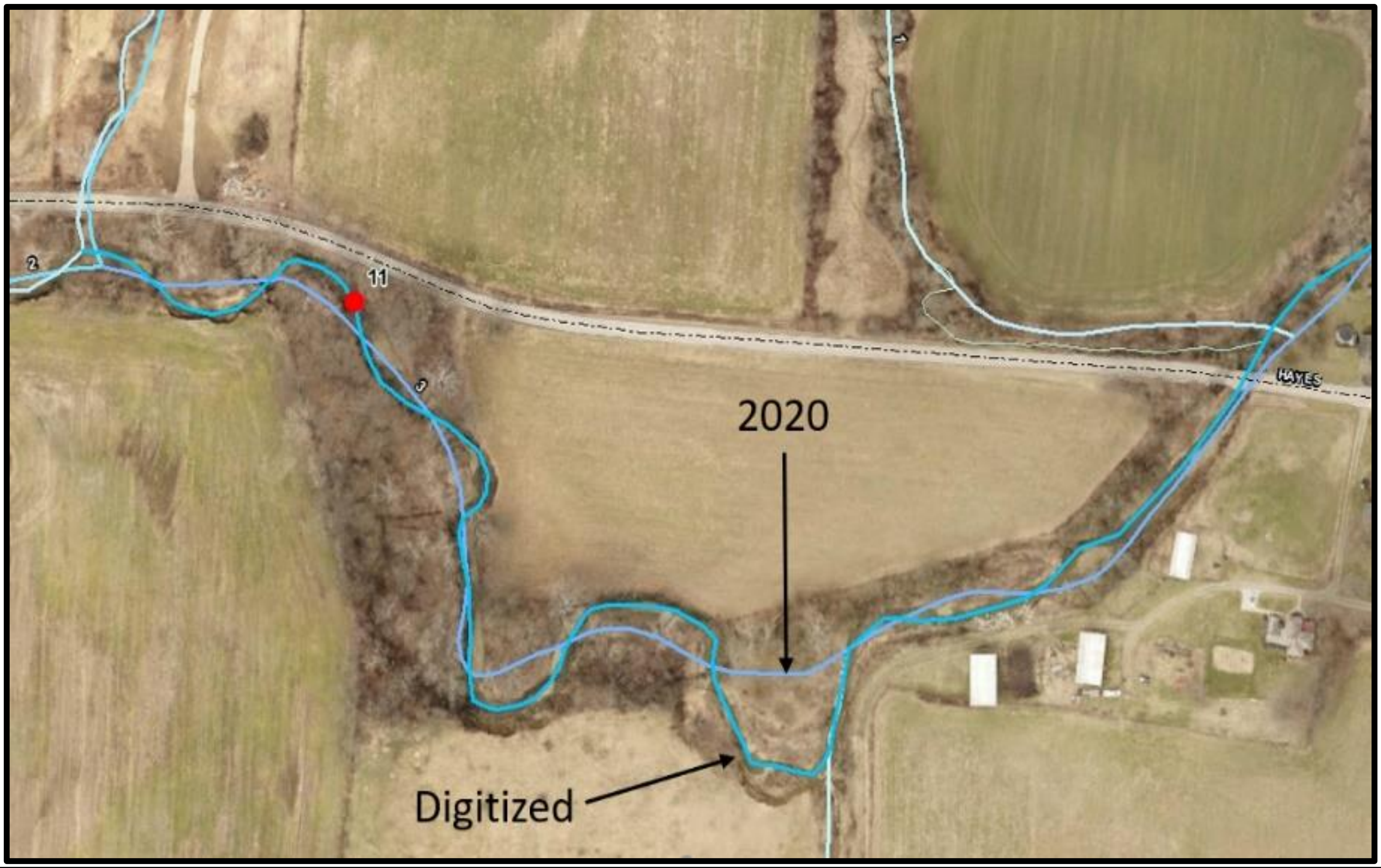


FIGURE 4-1
Example of Digitizing Stream Centerline to Conform to Aerial Imagery

4.3 Field Reconnaissance

To establish a reasonable number of log jam sites for field reconnaissance, we focused on *Log Jam Large* and *Total Blockage* locations along the 3rd order through 5th order streams. These larger watercourses are more likely to be the focus of a channel maintenance plan. EMH&T selected 31 log jam locations for field reconnaissance. A general location map with each of the 31 sites can be found on the Overall Site Visit Location Exhibit in Appendix A.

4.3.1 Field Score Cards

Score cards were developed for use during the site field reconnaissance. Information on the cover sheet was populated prior to the site visit to assist field staff in locating the mapped site. Pre-populated information included an inspection ID, coordinates, stream name, property address, property owner, parcel number, watershed, FEMA floodplain information, logjam size determined during desktop mapping, drainage area, and date of last rain event prior to site visit.

The second page of the field score card provided a place to record data collected in the field. Data collected in the field included evidence of overtopping banks, erosion of banks, presence of scour holes, stream flow, channel substrate, riparian corridor land use, logjam width, logjam height, logjam length, bank height, bankfull height, and bankfull width. Figure 4-2 depicts an example of a completed Field Score Card.

4.3.2 Logjam Field Investigation

A total of 31 sites were selected for field reconnaissance by EMH&T staff. After the appropriate property notification was completed in coordination with the LICKING COUNTY SWCD, the sites were visited during November of 2022. Of the 31 sites that were investigated, 24 sites were determined to have active logjams present at the time of the field visit. Four of the sites noted as having logjams during the desktop review were determined to be free of woody debris through the field investigation. The logjams identified during the desktop review for Sites 8, 9 and 20 appeared to have been washed out during previous large storm events, with large logs and debris observed on the stream banks. The field reconnaissance of Site 1 showed evidence of cut logs along the bank, suggesting that the debris may have been cut and moved through human intervention. Sites 4 and 29 were inaccessible at the time of the field investigations. Table 4-3 summarizes the sites where logjams were present upon field verification.

The 944 logjam locations identified through the initial desk top investigation are represented in the GIS shapefiles provided with this report. The 31 locations identified for field reconnaissance are represented in the Google Earth (kmz) files provided with this report.

EMH&T

Engineers, Surveyors, Planners, Scientists

SLWCD Channel Maintenance
Field Score Card
Site 11

Site Information:

ID #:	11
Latitude:	40.02107565
Longitude:	-82.53015772
County:	
Stream:	UT to Ramp Creek
Address:	1053 HAYES RD, GRANVILLE, OH
Property Owner:	KSP INFRA USA LLC
Parcel:	071-326400-00.000
Watershed:	Ramp Creek
Floodplain:	A
Logjam Category:	Log Jam Very Large
Drainage Area:	4.22 Square Miles
Last Rain Event:	11/14/2022

Site Visit Information:

Date: 11/15/2022

Evidence of overtopping banks: YES (NO)

Severely eroded banks: YES (NO) LEFT RIGHT

Scour Holes: YES (NO)

Stream Flow: (FLOWING) ISOLATED POOLS DRY

Substrate: BOULDER/SLAB (COBBLE) GRAVEL

Riparian corridor:

Left Bank: Wooded, agricultural field

Right Bank: Wooded, roadway

Log Jam Information:

Verification of Log Jam Size (circle one):

SMALL (10-40%) LARGE (41-70%) (VERY LARGE (71-90%)) TOTAL BLOCKAGE (90-100%)

Density of Log Jam Size (circle one):

Coarse (Water Can Flow Through) (Intermediate) Fine (No or Minimal Flow Through)

Channel Stability:

Bank Height= 6 ft. Bank Length= 105ft.

Erosion: Left Bank (Right Bank)

Bankfull Width (measured) = 43.2 ft.

Length of Logjam (measured) = 16 ft. parallel, 33 ft. perpendicular

Height of Logjam (measured) = 4 ft.,

Access Notes:

Right bank erosion, 105 ft.

FIGURE 4-2
Field Score Card

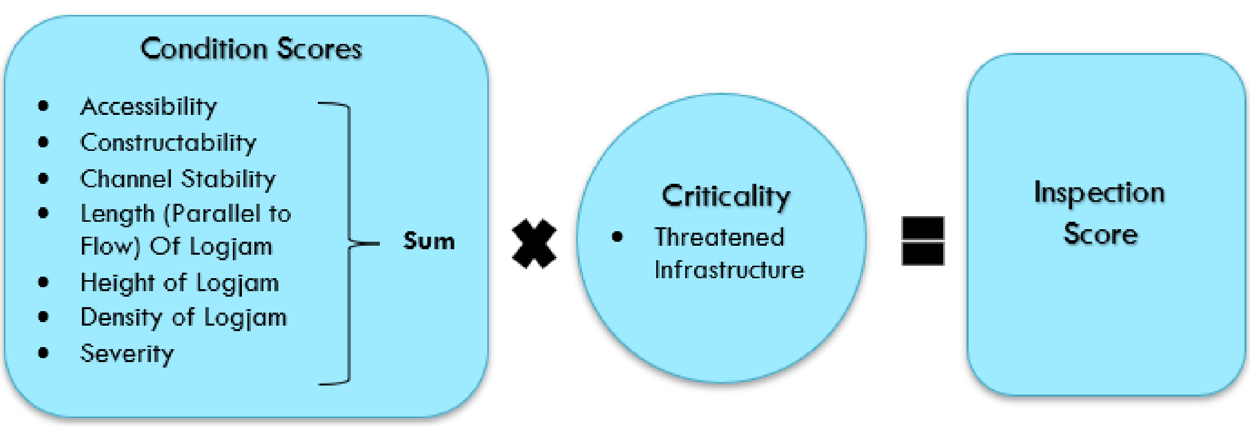
TABLE 4-3
Logjam Field Investigation Sites

Site ID	Logjam Present	Site ID	Logjam Present
0	Yes	16	Yes
1	No	17	Yes
2	Yes	18	Yes
3	Yes	19	Yes
4	Inaccessible at time of visit	20	No
5	Yes	21	Yes
6	Yes	22	Yes
7	Yes	23	Yes
8	No	24	Yes
9	No	25	Yes
10	Yes	26	Yes
11	Yes	27	Yes
12	Yes	28	Yes
13	Yes	29	Inaccessible at time of visit
14	Yes	30	Yes
15	Yes		

4.3.3 Individual Logjam Location Scoring

A risk-based scoring system was developed to evaluate the severity and opportunity for addressing each of the inspection locations. By applying a risk-based scoring system, decisions are proactive and the highest risks are addressed first. The scoring system is comprised of three components: condition ratings, asset criticality, and an inspection score. Each inspection point was assigned an inspection score using the formula presented below. Each component is further discussed herein.

Channel Protection Plan Scoring Formula



Condition Scores

There is a total of seven criteria comprising the condition score. The seven condition criteria are: accessibility, constructability, channel stability, length (parallel to flow) of logjam, height of logjam, density of logjam, and the observed severity of the existing condition related to bank erosion and flooding potential. The scoring system for each of the criteria varied based on the level of importance assigned to the criteria. For example, the range of scores for the 'severity' criteria (1 to 10) reflect the higher importance of that item in terms of determining the impact of the logjam on surrounding properties and prioritizing individual locations for removal.

Channel stability, length of logjam, height of logjam, and density of log jam were the second highest-weighted scores because these measurements are directly related to the current condition of the site condition and are correlated to the severity score. For example, removing logjam that is blocking a majority of the channel provides the greatest potential for improvement and thus receives the highest score. These scores ranged from 1 to 5.

The two remaining criteria, access and constructability, have the lowest range of scores (1 to 3) and primarily relate to the level of effort to address the issue. A greater level of effort relates to a lower score and thus reduces the overall inspection score. A high total inspection score indicates a high priority log jam location. Figure 4-3 is a list of the condition criteria and assigned scoring system. Score under each of the criteria were assigned to each logjam based on data recorded within the field score card and a desktop review of aerial photographs, easements, and parcel data.

Criticality Scores

Criticality scores were incorporated into the overall inspection score to help prioritize the logjam locations for removal or other measures. The criticality score is assigned based on the threatened infrastructure type, giving a higher criticality score to the higher priority assets such as private homes and highways. Criticality scores ranged from 1 to 5. For example, if there was a debris blockage at a culvert or bridge crossing of a 2-lane road, the criticality score would be based on the 1 to 4 lane road category (4); if an eroding bank is threatening a residential building, the criticality score would be based on the residential single-family home category (4). The threatened infrastructure type was identified as part of the field reconnaissance effort. If was determined that the log jam was not a threat to existing infrastructure, then the "Open Space" category was assigned to that location. The criticality categories and scores are presented in Figure 4-4.

Conditional Scores

Access (1-3)

Score	Access Determination
1	Requires work agreement from multiple property owners
2	Requires work agreement from a single property owner
3	Located entirely on City-owned property/drainage easement

Constructability (1-3)

Score	Constructability Determination
1	Requires extensive land disturbance/vegetation clearing
2	Requires moderate land disturbance/vegetation clearing
3	Requires minimal land disturbance/vegetation clearing

Channel Stability/Capacity - Relative Potential Improvement (1-5)

Score	Channel Stability
1	Bank erosion area <1000 square feet
3	Bank erosion area between 1000 to 3500 square feet
5	Bank erosion area >3500 square feet

Length (Parallel to Flow) of Logjam

Score	Constructability Determination
1	Channel blockage <1x bankfull width
3	Channel blockage between 1xand 4x bankfull width
5	Channel blockage >4x bankfull width

Height of Logjam

Score	Constructability Determination
1	Channel blockage <40% of bankfull width
3	Channel blockage between 40% to 70% of bankfull width
5	Channel blockage >70% of bankfull width

Density of Logjam

Score	Constructability Determination
1	Coarse (water can/will flow through logjam)
3	Intermediate
5	Fine (no or minimal flow through logjam)

Severity Score (1-10)

	Determination	
Score	Bank Erosion	Blockage
1	No threat anticipated	No adverse impacts to flooding
5	Threat anticipated within 2-5 years	Potential future flooding concerns
10	Threat anticipated within 0-2 years	Increased flooding threat to infrastructure

FIGURE 4-3
Summary of Condition Score Criteria

Criticality

Threatened Infrastructure Criticality (1-5)

Assign a score based on Asset Type

Threatened Infrastructure Structure Type	Asset Type	Criticality
Railroad	Transportation	5
Highway	Transportation	5
1-4 Lane Road	Transportation	4
Parking Lots	Transportation	3
Driveway	Transportation	3
Multi-Use Pathways (trails, golf course path, sidewalk, footbridge, etc.)	Transportation	2
Multiple Occupancy Building (hospital, apartment building, office building/business, strip mall, etc.)	Buildings	5
Residential Single-Family Home	Buildings	4
Other Non-Occupied	Buildings	2
Utility	Utility	3
Open Space	Open Space	1

FIGURE 4-4
Summary of Criticality Categories and Scores

Inspection Scoring Summary

The scoring process and individual inspection scores are presented within the Inspection Scoring Spreadsheet located within Appendix A. The inspection scores ranged from as low as 8 to a high score of 92. The resulting scores can be used by the SLWCD as a tool in comparing individual log jam locations and determining priority maintenance needs. Higher scores indicate a higher potential risk to infrastructure due to erosion or flooding.

4.4 Channel Maintenance Easement Corridor

The channel maintenance easement corridor would provide access for SLWCD to inspect and maintain watercourses within their jurisdiction. The maintenance would primarily be to find and remove log jams, but could also include addressing channel bank erosion where it threatens homes and public infrastructure. The extent of the channel maintenance easement corridor was determined based on a 50-foot buffer from the updated stream centerlines for the 3rd through 5th order streams within the SFLR and Raccoon Creek watersheds. County Auditor parcel shapefiles were used to identify individual property owners along the selected watercourses, and to determine the area and value of the channel maintenance easement on each parcel. Figure 4-5 demonstrates an example of an easement buffer, highlighted in yellow, along Ramp Creek and along an unnamed tributary of Ramp Creek, indicating the parcels overlapping the easement buffer. The mapped channel maintenance easements do not include railroad and roadway right-of-way. The mapping of the channel easement corridors and correlating impacted parcels is included within the GIS shapefiles provided along with this report. Accessing the shapefiles provides information on the impacted property owners, which are also identified as part of the easement cost analysis, described below.

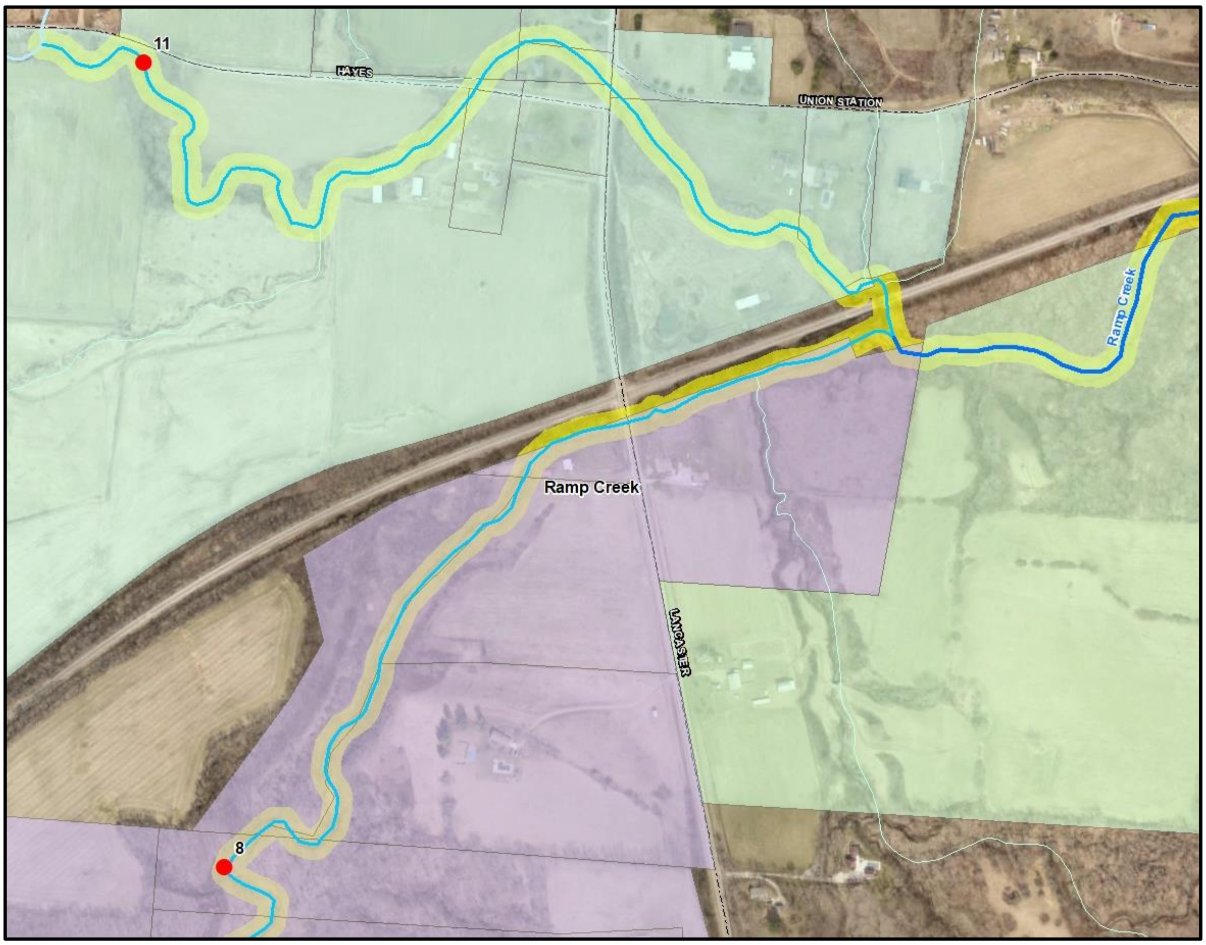


FIGURE 4-5
Example of Mapped Channel Maintenance Easement (Ramp Creek) and Overlapping Parcels

4.4.1 Easement Cost Analysis

To calculate potential costs for purchasing the stream easement corridor from each property owner, the County Auditor parcel data, including property valuations, were applied to the mapped easement corridors. An Easement Value Spreadsheet was created using the property attribute data acquired from the Licking County Auditor. The spreadsheet is organized by stream and then stream order. For estimating the value of the easement area on each impacted parcel, we applied the rules described below:

- The property value assigned to each parcel by the auditor was increased by 50% to estimate the market value of that property. The adjusted property values were then used to determine a value/acre of that parcel.
- The value of the land within the channel easement was determined by multiplying the easement area by the calculated value/acre.
- The cost of the easement is 30% of the calculated land value within the easement area on each parcel. This cost was then increased by 30% to include administrative costs (preparation of an appraisal and other documents).

The estimated total costs for each stream and stream order was calculated by rounding the total cost of each easement for each parcel up to the nearest one-hundred dollars. Note that any easement acreage from a single parcel under 0.01 acre was not calculated into the total easement cost. The multipliers referenced in the above rules comes from consultation with O.R. Colan, who provide real estate acquisition services for public infrastructure projects. The Easement Value Spreadsheet, provided with this report, includes the total easement costs, acreages and stream lengths for the SFLR and Raccoon Creek watersheds. This information is further broken down to 5th order, 4th order and 3rd order streams. The results of the easement cost analysis are provided in Table 4-4 (SFLR) and Table 4-5 (Raccoon Creek).

TABLE 4-4
Channel Maintenance Easement Costs (SFLR)

	Stream Length	Acreage	Land	Administrative	Total Cost
Total 5th Order =	30,308.00	51.13	$167,000.00	$50,100.00	$217,100.00
Total 4th Order =	188,668.00	412.35	$2,262,400.00	$678,720.00	$2,941,120.00
Total 3rd Order =	145,754.00	252.89	$1,416,100.00	$424,830.00	$1,840,930.00
TOTALS =	**364,730.00**	**716.37**	**$3,845,500.00**	**$1,153,650.00**	**$4,999,150.00**

TABLE 4-5
Channel Maintenance Easement Costs (Raccoon Creek)

	Stream Length	Acreage	Land	Administrative	Total Cost
Total 5th Order =	73,359.00	145.32	$867,800	$260,340	$1,128,140
Total 4th Order =	21,832.00	48.64	$180,500	$54,150	$234,650
Total 3rd Order =	180,707.00	404.71	$2,351,300	$705,390	$3,056,690
TOTALS =	**275,898.00**	**598.66**	**$3,399,600**	**$1,019,880**	**$4,419,480**

5.0 HYDROLOGIC STUDY

The hydrologic study prepared by EMH&T is complementary to the 2D HEC-RAS (hydraulic) model prepared by ms consultants for the SFLR watershed. The hydrologic study provides computed hydrographs representing peak flood discharge values and flow volumes for specific flood frequencies, for the portion of the watershed beyond the 2D model area. The computed hydrographs are input into the HEC-RAS model at the interface of the hydrologic and hydraulic model boundaries. The 2D HEC-RAS model utilizes 'rain-on-grid' to generate storm event hydrology within the limits of that modeling; therefore, the 2D model area is excluded from the hydrologic modeling described in this report. For the reasons described below, the hydrologic study was separated into the two components described below.

Figure 5-1 depicts the extent of the hydrologic modeling of the SFLR with respect to the 2D HEC-RAS model area. The SFLR watershed shown on this figure is based on the published HUC-12 sub-watershed boundaries and may not match the watershed and sub-watershed boundaries represented in the hydrologic modeling.

- HEC-HMS Model: EMH&T created a hydrologic model representing the 113.86 sq. mi. watershed directly tributary to the SFLR 2D mesh area, not including the watershed area tributary to Buckeye Lake, using the HEC-HMS version 4.9 computer program. The HEC-HMS model represents all or portions of the following HUC-12 sub-watersheds: Ramp Creek, Dutch Fork, Kirkersville, Muddy Fork, and Headwaters. Portions of the Bell Run and Beaver Run HUC-12 boundaries are within the limits of the 2D mesh created for the HEC-RAS model; therefore, they are not included in the HEC-HMS model. Otherwise, the areas within these HUC-12 sub-watersheds and outside of the mesh are included in the HEC-HMS model. One of the key interface points between the 2D HEC-RAS model and the HEC-HMS model is along the SFLR at the USGS stream gauge in Kirkersville, at Outville Road. The portion of the HEC-HMS model tributary to this point was calibrated to USGS Gauge No. 03144816.

- AutoCAD Storm-Sanitary Analysis (SSA) Model: The SSA model represents the following HUC-12 sub-watersheds: Buckeye Lake and Buckeye Lake Reservoir Feeder. EMH&T created a hydrologic model representing 45.07 sq. mi. of watershed area associated with Buckeye Lake and the Feeder Canal using the 2021 version of the SSA computer program. SSA is published by Autodesk and uses a similar computational engine to EPA's Stormwater Management Model (SWMM). The SSA model was developed for this portion of the overall SFLR watershed due to the enhanced ability to route flows through the Feeder Canal, represent overflows/diversions, reverse flows and the level-pool (storage) analysis through the lake. Four different SSA model runs representing existing conditions were created to represent the different operating conditions at Buckeye Lake under summer and winter conditions, and the calibration events, as described in Table 5-1. The starting water surface elevations listed in the table are explained later in this section of the report.

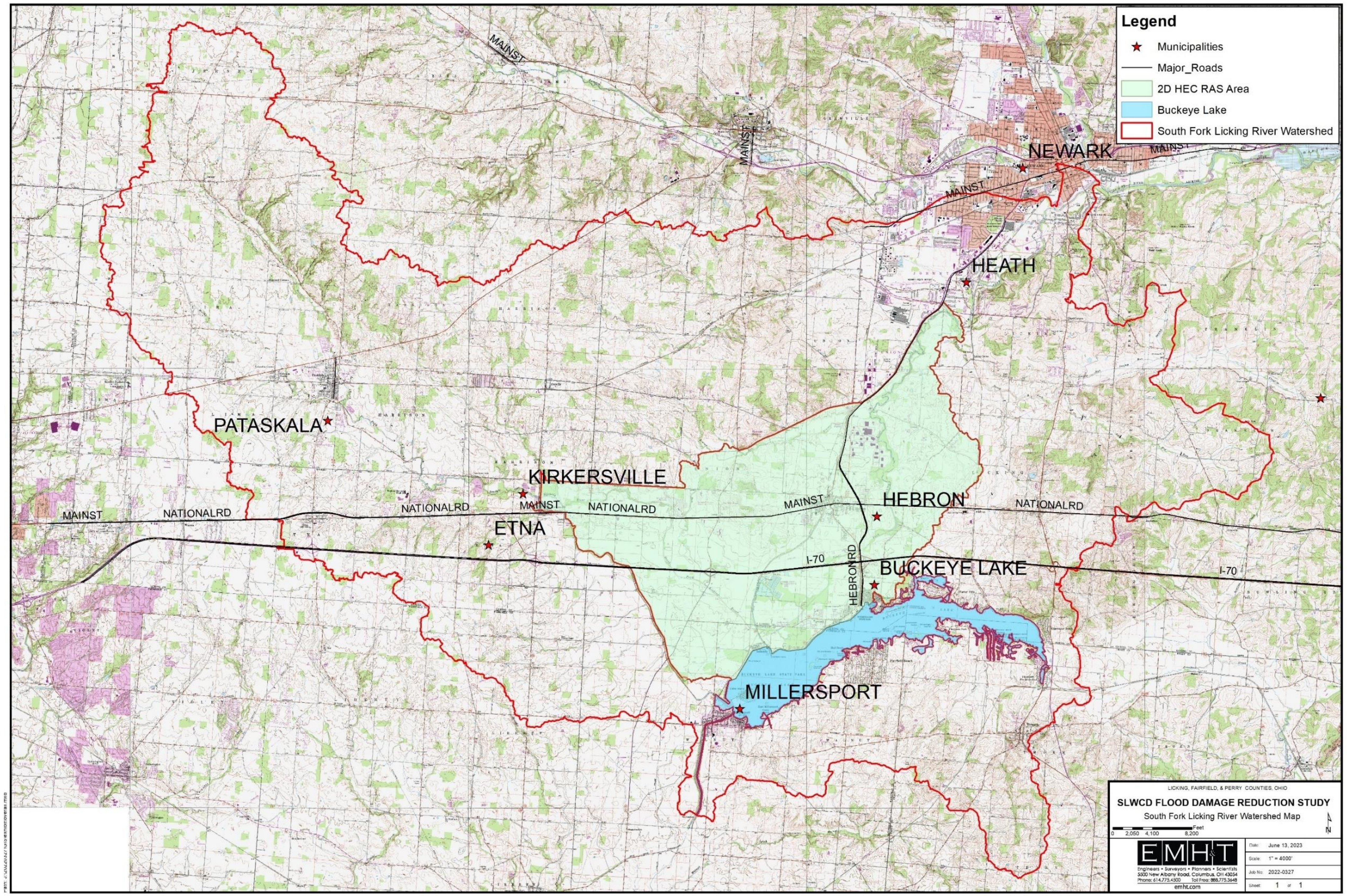


FIGURE 5-1
South Fork Licking River Watershed and 2D HEC-RAS Model Area

TABLE 5-1
Summary of HEC-HMS and SSA Models

Model Filename	Season	Amil Gate Operation	Starting Water Surface Elevation (NAVD '88)	Model Purpose
SFLR_MasterModel	N/A	N/A	N/A	Model Calibration, Design & Synthetic Storms
Buckeye Lake Summer	March 1st to November 15th	Down	890.97	Design & Synthetic Storms
Buckeye Lake Winter	November 15th to March 1st	Up	887.97	Design & Synthetic Storms
Buckeye Lake March 2020	n/a	Down	889.89	Model Calibration
Buckeye Lake July 2017	n/a	unknown	890.13	Model Calibration

5.1 Hydrology Overview

A detailed map of the SFLR watershed, sub-watersheds and subcatchments for both the HEC-HMS and SSA models is provided on Exhibit 1 within Appendix B. This exhibit also depicts the location of rain and river gauges referenced in this section of the report. The study area includes nine USGS 12-digit HUC (HUC-12) sub-watershed areas that are tributary to South Fork Licking River, including Buckeye Lake, listed below. The Buckeye lake and Buckeye Lake Reservoir Feeder sub-watershed areas discharge to and through the Buckeye Lake spillways and to the SFLR watercourse.

- 050400060401 – Muddy Fork Sub-Watershed
- 050400060402 – Headwaters South Fork Licking River
- 050400060403 - South Fork Licking River (Kirkersville)
- 050400060404 – Buckeye Lake Reservoir Feeder (Feeder Canal)
- 050400060405 – Buckeye Lake
- 050400060406 – South Fork Licking River (Bell Run)
- 050400060407 – Ramp Creek
- 050400060408 – Dutch Fork
- 050400060409 – South Fork Licking River (Beaver Run)

The boundary of each HUC-12 sub-watershed was verified using the available topography. These sub-watersheds were further divided into subcatchments to improve model detail and accuracy in representing local infrastructure. In many cases, the individual subcatchments represent the watershed subarea to designed or incidental (natural) detention storage locations.

5.2 Rainfall

The selected design storm duration is the 24-hour event. Other rainfall durations were tested in the SSA and HEC-HMS models to determine which would produce the highest peak flood discharge values for the watershed. The 24-hour rainfall duration produced a slightly higher peak flood discharge value in the SSA model of the Buckeye Lake sub-watershed, but the HEC-HMS model peak flood discharge values coincided with the 24-hour storm duration; therefore the 24-hour duration was selected for this study. The Illinois Water Survey Bulletin 71 (Huff and Angel)[1] rainfall distributions have been used in lieu of the NRCS Type II distribution, mainly due to the size of the watershed being studied. The specific rainfall distribution used in the SSA and HEC-HMS modeling is the 3rd quartile, 50% curve, 10-50 sq. mi., 24-hour distribution. Recorded rainfall data from the March 2020 storm event was used for calibration event, discussed later this this section of the report. For dam safety design purposes, the Ohio[2] specific distributions were used.

5.2.1 Design Storms

The source for design storm rainfall depths is NOAA Atlas 14. The Atlas 14 rainfall depths vary slightly throughout the SFLR watershed depending on the location selected. The selected rainfall depths are shown below (Atlas 14 location centered on Kirkersville), which coincides with the values used by ms consultants as part of the 2D HEC-RAS model.

1-year 24-hour Depth:	2.19 inches
2-year 24-hour Depth:	2.62 inches
5-year 24-hour Depth:	3.26 inches
10-year 24-hour Depth:	3.80 inches
25-year 24-hour Depth:	4.60 inches
50-year 24-hour Depth:	5.28 inches
100-year 24-hour Depth:	5.99 inches
200-year 24-hour Depth:	6.83 inches

5.2.2 Dam Design Rainfall Events

The proposed flood control dams described in Section 6 of this report would be considered either Class I or II based on volume, height, and hazard, based on State of Ohio dam safety regulations. Class I and II dams are required to analyze the all-season 6-hour and 24-hour Probable Maximum Precipitation (PMP) events. The rainfall depth is pro-rated based on basin drainage area. The PMP depths used in this study for the proposed dams are summarized in Table 5-2.

TABLE 5-2
Proposed Dam Probable Maximum Precipitation Depths

Proposed Dam	Drainage Area	6-hour PMP Depth (in)	24-hour PMP Depth (in)
Muddy Fork	10.67	17.65	23.65
Bell Run	2.70	18.70	25.70
SF Tributary A	5.22	18.20	24.60
SF Tributary B	3.17	18.55	25.40
Feeder Canal	5.85	18.10	24.50
Kirkersville	47.2	16.20	21.75
Headwaters	7.25	17.90	24.20

[1] Rainfall Frequency Atlas of the Midwest, Bulletin 71, Floyd A. Huff, and James R. Angel, Illinois State Water Survey, 1992.
[2] Technical Guidance for Dam Break Studies, Version 1.0, Ohio Department of Natural Resources, January 2023.

5.3 Runoff Method

Watersheds were delineated using the topographic data described under Section 3. Sub-watershed areas were delineated to coincide with the HUC-12 sub-watershed areas. The GIS toolset within HEC-HMS was used to perform an automated delineation of these sub-watersheds using the existing topography and up to a maximum area of 0.75 sq. mi. These sub-watershed delineations were then manually adjusted to provide breaks at roadways and other natural storage areas.

The NRCS Runoff Curve Number (RCN) method was used for this analysis in both the HEC-HMS and AutoCAD SSA models to predict runoff volume and peak flood discharge values. The RCN was based on an evaluation of land use and soil types within each of the delineated subcatchments. For this study we used the RCN values published in the City of Columbus'[3] Stormwater Drainage Manual and assumed that all agricultural land was in good hydrologic condition with conservation treatment. For example, agricultural land in Type C soils was assigned an RCN of 78. Composite RCNs were used in most cases with the exception of a subcatchment coinciding with the water surface of Buckeye Lake, which was modeled using an RCN of 100. Times of concentration (Tc) calculations were performed using the NRCS method with sheet flow, shallow concentrated flow, and channel flow components. The GIS tools within HEC-HMS was used for the majority of the subcatchments to determine the Tc flowpath. Calculated Tc values were converted to lag times by multiplying those values by 0.6. These values apply only to the HEC-HMS model area; hydrologic parameters for the Buckeye Lake sub-watersheds are discussed separately later in this section of the report.

5.4 Existing Watershed Storage Areas

For larger rainfall events, runoff can be temporarily attenuated in stormwater detention basins, behind roadway culverts, and at inline weirs and inline channel storage, such as along the Buckeye Lake feeder canal. The representation of the designed and incidental storage within the watershed increases the model accuracy and has helped with the calibration process. Otherwise calculated peak flood discharge values and Buckeye Lake water levels may be overestimated, making model calibration more difficult. Not every existing stormwater detention basin or storage area was modeled, but an effort was made to model the larger detention basins and roadway culverts where it is possible to capture and store a significant volume of runoff before roadway overtopping. The majority of the natural storage in the model was upstream of the Kirkersville gauge since this was our key calibration point. Also, the majority of the larger detention basins in the watershed are in the headwaters area upstream of Pataskala. The HEC-HMS model includes 95 storage nodes, of which 31 are larger existing stormwater detention basins associated with development sites within the watershed, and 64 are associated with incidental storage areas upstream of roadway culverts.

For the SSA model of the feeder canal and Buckeye Lake, a total of 35 storage areas were identified as potential areas where attenuation of flows may occur. The majority of these were upstream of highway culverts in the feeder canal watershed or along the feeder canal itself due to the shallow slope of the canal and adjacent land to the west. For the HEC-HMS model there are a total of 95 storage areas.

[3] City of Columbus Stormwater Drainage Manual, Division of Sewerage and Drainage, December 2022.

5.5 Buckeye Lake Sub-Watershed Hydrology & Hydraulics

A separate discussion of the hydrologic and hydraulic analysis of the sub-watershed areas tributary to Buckeye Lake has been provided due the unique complexities of developing the model analysis of this portion of the SFLR watershed. Considerable effort has been put forth to collect relevant data from multiple sources and determine how best to use the data when understanding how flow is directed to the lake, and the operation of the lake spillways that control the lake level and the release of flow to SFLR depending on seasonal operation conditions.

The Buckeye Lake watersheds include two HUC-12 sub-watersheds: 050400060404 – Buckeye Lake Reservoir Feeder (Feeder Canal) and 050400060405 – Buckeye Lake. The later sub-watershed is directly tributary to the lake and includes the lake area, Honey Creek and the 'Deep Cut' channel. The former is tributary to the Feeder Canal and includes the Bloody Run weir overflow to Pigeon Swamp Ditch. The sub-watershed boundaries and subcatchment areas were determined and refined using the auditor's topography previously referenced. The total sub-watershed area directly tributary to the lake (050400060405) is 28.29 sq. mi. The total sub-watershed area associated with the Feeder Canal is 16.78 sq. mi., for a total of 45.07 sq. mi. within the Buckeye Lake watershed. Refer to Exhibit 1 for a graphical depiction of the buckeye Lake sub-watersheds and subcatchments.

The nomenclature used for the subcatchments generally follows the 2015 Tetra Tech[4] report for the sub-watershed area directly tributary to the lake. Subarea 33, which is the lake itself, was divided into two areas: 1) the water surface area of the lake with an RCN of 100; and 2) the islands and other open space areas in or near the lake's edge. Isolating the lake surface produces more runoff volume then including it with a larger subcatchment area and developing a composite RCN. The lake area was determined using the most recent aerial imagery and determined a lake area of 2,910 acres. The Tetra Tech report indicates a lake area of 2,831.4 acres. RCN and Tc values used in this study are the same as those provided in the Tetra Tech report.

The 2015 Tetra Tech report indicated a drainage area of 44.1 sq. mi. to the lake compared to the 45.07 sq. mi. determined as part of this study. One difference between the Tetra Tech study and this study is the refinement of the subcatchment area associated with the Deep Cut channel, at the Village of Millersport, which is a remnant of the canal system. Most of the Deep Cut channel appears outside of the HUC-12 sub-watershed boundary. This study has revised the sub-watershed boundary to include the channel and the immediately surrounding areas as being tributary to Buckeye Lake, adding approximately 1 sq. mi. of drainage area. Additionally, this study provides a detailed evaluation of a complex hydrologic condition along the Feeder Canal at the Bloody Run weir, described in more detail later in this report.

5.5.1 SSA Model Selection

Due to the relatively flat slope in the canal, multiple outlet points, the potential for reverse flow during a large storm event, and large areas of farmland west of the canal that flood, the Feeder Canal portion of the Buckeye Lake sub-watershed could not be modeled as accurately using a traditional 1Dhydrologic or hydraulic model. A more dynamic unsteady-state model was needed to simulate the more complex flow conditions. The AutoCAD SSA model was selected as it has the ability to perform more complicated routing between storage areas and channels, using a similar

[4] Buckeye Lake Dam, South For Licking River Watershed Model Technical Report, ODNR File No. 9723-004, October 2015.

computational engine to the Environmental Protection Agency (EPA) Stormwater Management Model (SWMM). The AutoCAD SSA model utilizes the RCN method for calculating rainfall runoff and allows for user inputs of rainfall depths and distributions. In this regard, the SSA model has the same computational method for estimating runoff as the HEC-HMS model being used for the remainder of the SFLR watershed. The SSA model is not a public domain program, but is readily available as part of the AutoCAD Civil 3D software package.

The SSA model prepared for this study includes both of the HUC-12 sub-watershed areas associated with Buckeye Lake, to allow for a model simulation that captures flow through the Feeder Canal to the lake and the complexities of level pool routing through the lake given the varying operations of the two spillway structures.

5.5.2 Feeder Canal Diversions to SFLR

As noted above, the Feeder Canal overflows toward SFLR at the Bloody Run weir at elevation 904.4 feet and the adjacent embankment at an elevation of approximately 907.0 feet. During larger flood events the eastern embankment of the canal overflows to Pigeon Swamp Ditch just north of Palmer Road at an elevation of approximately 904.9. There are field tiles serving farm fields west of the Feeder Canal that go under the canal and discharge to Pigeon Swamp Ditch directly downstream of the Bloody Run Weir; however, that condition has not been accounted for in the SSA model.

Bloody Run Weir

To determine the crest elevation of the Bloody Run weir, for purposes of modeling, we have relied on a 1980 ditch petition plot (refer to Figure 5-3), which shows an elevation of 905.4 (NGVD, 1929). Field survey of the weir was going to be performed as part of ms consultant's modeling efforts, but multiple attempts were unsuccessful due to weather and flow conditions in the channel. Converting the elevation from the ditch petition plot to NAVD, 1988 (by subtracting 0.6) results in a weir crest elevation of 904.8. This elevation was further reduced to 904.4 based on field observations of the condition of the weir crest. Refer to Figure 5-2 for a depiction of the diversion of flow locations in the upper portion of the Feeder Canal at the Bloody Run weir. The model parameters representing the Bloody Run weir are listed below.

- Weir Crest Elevation = 904.4 (estimated)
- Weir Length = 40 feet (estimated)
- Weir Coefficient = 3.0 (sharp-crested weir)

The photos below provide more information regarding the Bloody Run weir overflow from the Feeder Canal to Pigeon Swamp Ditch and then to SFLR. The photos from December 2022 represent less than one-inch of rainfall in the upstream watershed and demonstrate that most of the flow in the northern portion of the Feeder Canal all going over the Blood Run weir, and not toward Buckeye Lake. A more recent photo during dry conditions shows the water level below the lip of the weir, but it was observed to be leaking through a hole in the Bloody Run weir structure, which is in very poor condition, and flowing into Pigeon Swamp Ditch.

Photo: Looking North at Bloody Run Weir, December 2022

Photo: Looking at Downstream Side of Weir

Photo credited to Dan Blatter

Photo: Looking North along Feeder Canal South of Bloody Run Weir, December 2022.

Photo: Looking N.E. at Bloody Run Weir, July 2023 Water was heard and observed leaking through weir below the waterline

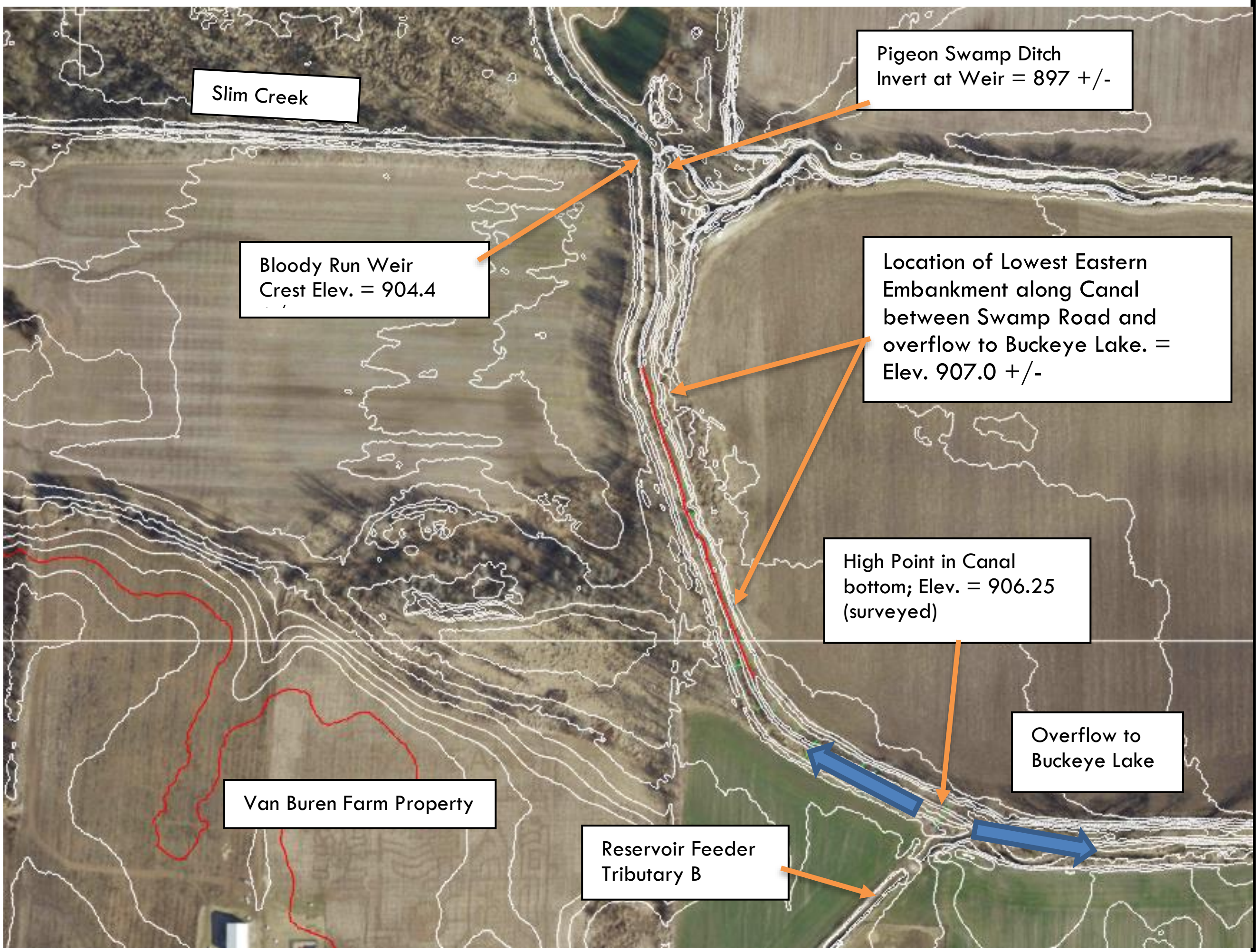


FIGURE 5-2
Overflow Locations from the Upper Feeder Canal to SFLR and Buckeye Lake

The Feeder Canal channel immediately south of the weir on the Van Buren Farm appears to have been filled, possibly by natural deposition of sediments, which creates a watershed divide along the canal. The watershed divide is at an elevation of 906.25 per field survey on the Van Buren Farm, see Figure 5-2. Therefore, it takes approximately 2 feet of hydraulic head on the Bloody Run Weir for water to begin to flow south into the portion of the Feeder Canal that has direct access to Buckeye Lake. Additional discussion of the movement of water through the Feeder Canal is provided below.

<u>Overflow to Buckeye Lake</u>

The Feeder Canal bottom profile was field surveyed by ms consultants as part of their study, between the location of the Bloody Run weir south towards the Feeder Tributary B as shown on Figure 5-2. The field survey and 2-foot contour interval topography confirmed the canal flows north from a high point elevation of 906.25 just north of Feeder Tributary B to the Bloody Run weir. Just south of this high point in the canal is the confluence of the Feeder Tributary B channel that is a few feet lower in elevation than the canal high point. Therefore, base flow from Feeder Tributary B flows to Buckeye Lake. There is a small embankment along the north side of Reservoir Feeder B, which keeps flow in Feeder Tributary B flowing east and toward Buckeye Lake, which can overflow during large flood events and flow north toward the Blood Run weir creating a reverse flow condition. In the SSA model, during the larger simulated rainfall events, flow in Feeder Tributary B was shown to partially flow north due to a lack of capacity in the receiving portion of the Feeder Canal to the south. Discussions with local property owners and others indicated this flow condition has been observed during past flooding events. The SSA model represents the Feeder Canal and Tributary B channel topography based on the field survey and supplemented by the area-wide topography referenced previously.

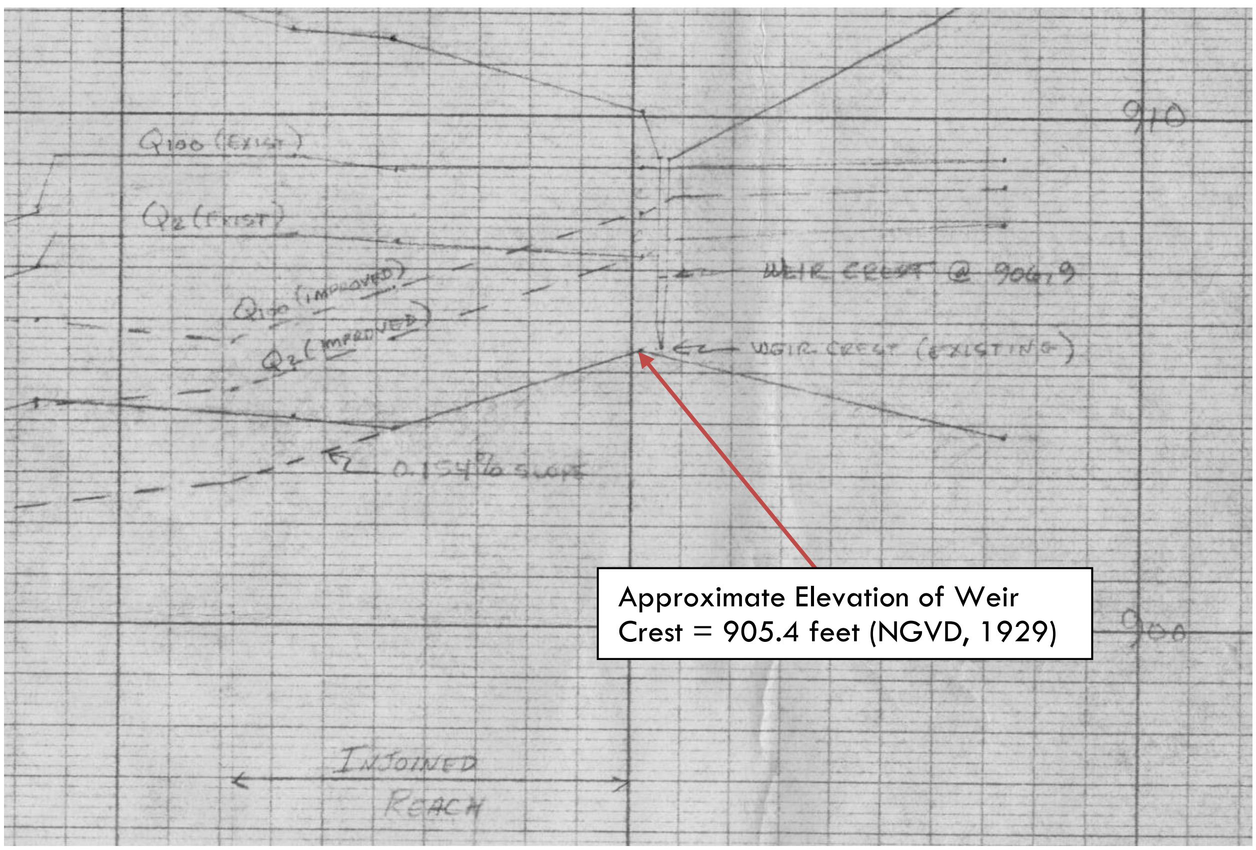


FIGURE 5-3
Ditch Petition Plot (1980) at the Bloody Run Weir

Earthen Embankment Overflow South of Bloody Run Weir

The earthen embankment along the east side of the Feeder Canal drops down to an elevation of approximatley 907.0 south of the Bloody Run weir, location depicted on Figure 5-2. This elevation was determined from the Licking County Digital Elevation Model (DEM). The embankment length used in the model is 670-feet long based on the County's 2-foot contours. Otherwise, the top of the eastern embankment appears to stay above elevation 910 north of I-70 to Swamp Road. The model parameters representing the Bloody earthen embankment overflow are listed below.

- Assymetrical Weir
- Invert Elevation = 907.0 (estimated)
- Weir Coefficient 2.6 (broad crested weir)
- Length 670 feet (estimated)

Palmer Road Overflow

According to the Licking County DEM and associated 2-foot contour interval topography, the eastern embankment along the Feeder Canal drops down to elevation 904.9 +/- south of I-70, refer to location in Figure 5-4. We have estimated 1,310 feet of embankment below an elevation of 907.0 between Palmer Road and I-70. The canal channel bottom in this area is at approximate elevation 898.5, providing 5 to 6 feet of flow depth before overtopping occurs. The photo below depicts overtopping of the Feeder Canal channel during the March 2020 event. The model parameters representing the earthen embankment overflow at Palmer Road are listed below.

- Assymetrical Weir
- Invert Elevation = 904.90 (estimated)
- Weir Coefficient 2.6 (broad crested weir)
- Length 1,310 feet (estimated)

Photo: Looking north along Feeder Canal Embankment just north of Palmer Road during March 2020 flood event

Photo credited to Dan Blatter

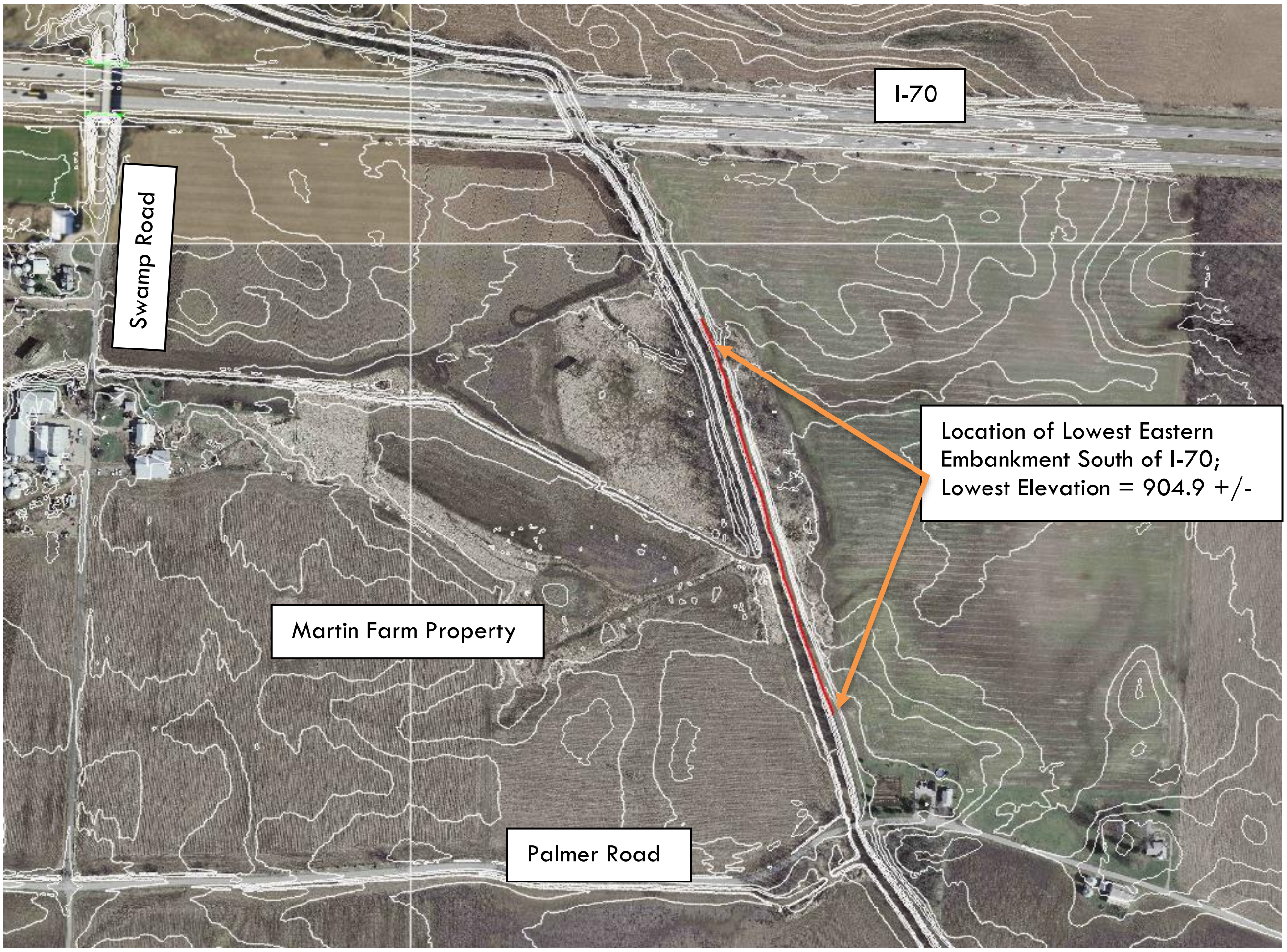


FIGURE 5-4
Overflow Locations from the Upper Feeder Canal to SFLR and Buckeye Lake

5.5.3 Buckeye Lake Spillways

Buckeye Lake currently has two spillways: 1) the primary spillway located along the north side of the lake near the intersection of SR 79 and Rosebraugh Circle; and 2) an emergency spillway constructed in the early 1990s (Seller's Point spillway). Refer to Figure 5-5 for a depiction of the location of the primary spillway and photos of this structure. The primary spillway is two outlets in series with the upstream outlet being a 33.5-foot long adjustable stop log structure. The downstream outlet is an Amil Gate. Previous Buckeye Lake dam reports were reviewed to determine normal lake operating conditions followed by a field visit to the two spillways with the ODNR dam tender.

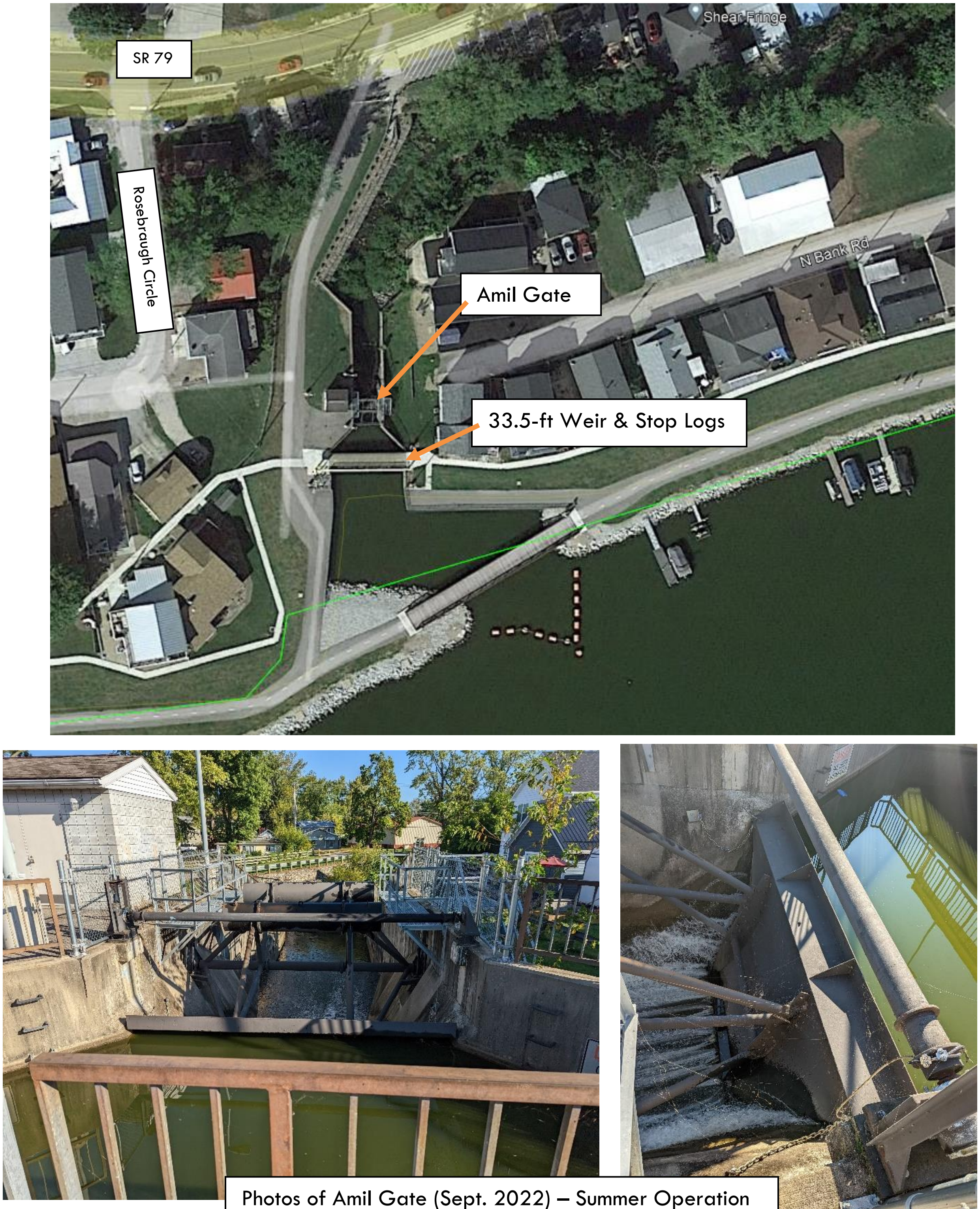


FIGURE 5-5
Location and Photos of the Primary Dam Spillway (Amil Gate)

Lake Normal Pool Elevation

The normal pool of the lake changes from winter to summer operations with a 3-foot +/- drawdown starting November 15th and then filling back up starting March 1st. The drawdown in the fall is accomplished by a partial opening of the 60-inch lake drains at both spillways to achieve a relatively slow drawdown rate to lessen the stress on the dam. When the lake is in summer operation, the lake drains are left closed. The summer pool elevation is considered to be 890.97 (NAVD ’88) per a conversation with the ODNR dam tender. During a site visit in September 2022, the lake was just above normal pool at an elevation of 891.15, according to the USGS gauge. Some water was flowing out through the bottom of the Amil Gate, as shown in Figure 5-5. The Amil Gate may not completely seal itself to the concrete channel when the lake reaches the normal pool elevation and some leakage may be occurring. We learned from the dam tender that the stop logs are no longer being used as part of the summer operation; therefore, the normal pool elevation is now completely controlled by the Amil Gate.

Lake Surface Area

Various developments around the shoreline have increased the total lake area over time. Several different normal pool areas were found in the various reports pertaining to Buckeye Lake, referenced in Table 5-3. To rectify the differences, the most recent aerial photos were analyzed in GIS to determine lake area at or near summer pool. The measured area is 2,910 acres, excluding islands. This included area associated with the ‘deep cut’ canal near the southwest corner of the lake that appeared to be missing from the watershed maps of other studies.

TABLE 5-3
Buckeye Lake Normal Pool Surface Area Comparison

Source	Summer Pool Surface Area (acres)	Notes
USACE 2015[5]	2,800	
Tetra Tech 2015[6]	2,831.4	
Gannett Fleming 2018[7]	2,560	Rounded to 4 sq. mi. in report
EMH&T 2022	2,910	

Amil Gate Spillway – Summer Operation

Photos of the gate during summer operations were taken on September 20th, 2022 at 9:34 AM, refer to Figure 5-5. During summer operations, the gate is in its down (closed) position. The Amil Gate, to the best of our knowledge, is a Waterman Type C (Model C-17) constant upstream level gate. The photos show water coming out of the bottom and sides of the gate. At the time of the photo, the lake was at an elevation of 891.15 (NAVD ‘88).

[5] Buckeye Lake Dam, Review of Past Reports and Existing Conditions and Recommendations for Interim Risk Reduction Measures, Operation, and Maintenance, USACE Huntington District, Great Lakes and Ohio River Division, March 2015.
[6] Buckeye Lake Dam South Fork Licking River Watershed Model Technical Report, ODNR File No. 9723-004, October 2015, Tetra Tech.
[7] Final Hydraulic and Hydrologic Report, Buckeye Lake Dam, Ohio, Project Number DNR 150080, Gannett Fleming, March 2018.

During summer operation, the counter-balance of the gate keeps the gate closed with approximately 6.78 feet of water held back on the upstream side at normal pool. As the water level rises, the gate rotates up at an unknown rate due to the buoyancy box and counter-weight setup. All of the water flows through the bottom and sides of the gate similar to a radial gate. The gate is designed to not be overtopped at any lake level. The Amil Gate has been analyzed by at least three previous studies. The reports obtained as part of this study are the 2015 report from Tetra Tech and the Gannett Fleming report from 2018[8]. The Tetra Tech report refers to a Dodson-Lindblom Associates, Inc. report from 1994 that was not obtained as part of this study. The previous studies either tried to model the Amil gate as weir or use the maximum capacity from the manufacturer's brochure (612 cfs) as a constant flow rate. After discussing this issue with the manufacturer, we were not confident in applying either method in this study, as the use of this style of gate is mainly used in canals with some form of water level on both sides of the gate. In this application the ballast would have been adjusted with counterweight added or removed to keep the gate closed with over 6 feet of water head on the lake side and no water on the downstream side. Under this condition, the manufacturer agreed the flow through the gate would not be a constant value.

For this study, the outflow rate for the Amil gate was estimated by analyzing the lake drawdown from near the emergency spillway crest elevation of 891.48 down to normal lake pool elevation of 890.97, assuming the Amil Gate is in its summer (closed) position. Using data from the USGS lake gauge located near the primary spillway, 14 different storms were identified, dating back to 1997, appearing to have a uniform drawdown from 891.48 to approximately normal pool. Storms with rainfall during the drawdown period skewed the readings and were not used. Figure 5-6 shows a graph of lake water level drawdown over time. The steeper the curve, the faster the outflow rate through the Amil Gate outlet. In addition to the drawdown graphs, the estimated outflow rate curves from previous reports are shown.

Based on the evaluation of the drawdown data, we offer the following observations:

1. The Amil gate outflow estimate of 612 cfs from previous reports is too high and does not match the observed drawdown data.

2. Many events did not reach normal pool. For older events this may have been due to maintenance issues or operational changes to the stop logs or Amil gate ballast. This also may be due to baseflow into the lake exceeding the outflow rate from the Amil Gate.

3. The fastest outflow rate was from May of 2002. The nuances in the operating condition of the lake are not well known during that time.

4. The most representative drawdown curve for the Amil gate spillway based on the current operating condition of the lake is believed to be from the June 17-22, 2022 event. This drawdown curve is one of the few to make it close to normal pool within a reasonable amount of time, and the Amil gate would not have been affected by the upstream stop log structure (removed at that time).

[8] Final Hydraulic and Hydrologic Report, Buckeye Lake Dam, Ohio. DNR 150080, Gannett Fleming, March 2018.

Amil Gate – Winter Operation

The capacity of the primary spillway was evaluated during winter operating conditions to determine the peak flood discharge values through this spillway. During winter operations, Buckeye Lake is drained down to an elevation of 887.97 feet (NAVD '88). The flow to the Amil Gate is controlled by the upstream 33.5-foot weir which has been permanently set to an elevation of 887.97 feet. During the winter, the Amil gate is manually opened and held open with cables; the gate is not opened until the lake has been drained down to the winter pool elevation using the 60-inch lake drains mentioned previously. At this point, baseflow overtopping the upstream weir discharges through the concrete trapezoidal channel with a bottom width of 9 feet 2¼ inches and a side slope of 1:1, see photos below. These measurements were taken from the Amil Gate standard detail. The Amil Gate concrete channel was modeled as a broad-crested weir with a length of 10.6 feet, which resulted in a weir coefficient that varied from 2.51 to 2.64. The invert of the channel at the gate is 884.19 feet (NAVD '88) from an as-built elevation on the record plans. (Note: EMH&T was not provided with the Record Plans for the Buckeye Lake dam and spillways, but this information was communicated to us by someone with access to those plans).

The analysis of the primary spillway capacity under the winter operation condition is based on applying the sharp created weir equation to the 33.5-foot long weir upstream of the Amil Gate, and applying a tailwater condition using the broad-crested weir analysis of the Amil Gate in the fully open position. This analysis assumes the Amil Gate in its open position does not impede flow through the primary spillway, even during larger flood events. The winter operation rating table is shown on Table 5-4.

TABLE 5-4
Primary Spillway Winter Operation Calculated Peak Flood Discharge Values

Lake Elevation (ft., NAVD '88)	Peak Flood Discharge Value (cfs)
887.97	0.00
888.25	16
888.50	42
889.00	114
890.00	313
891.00	567
892.00	865
893.00	1,122
894.00	1,383

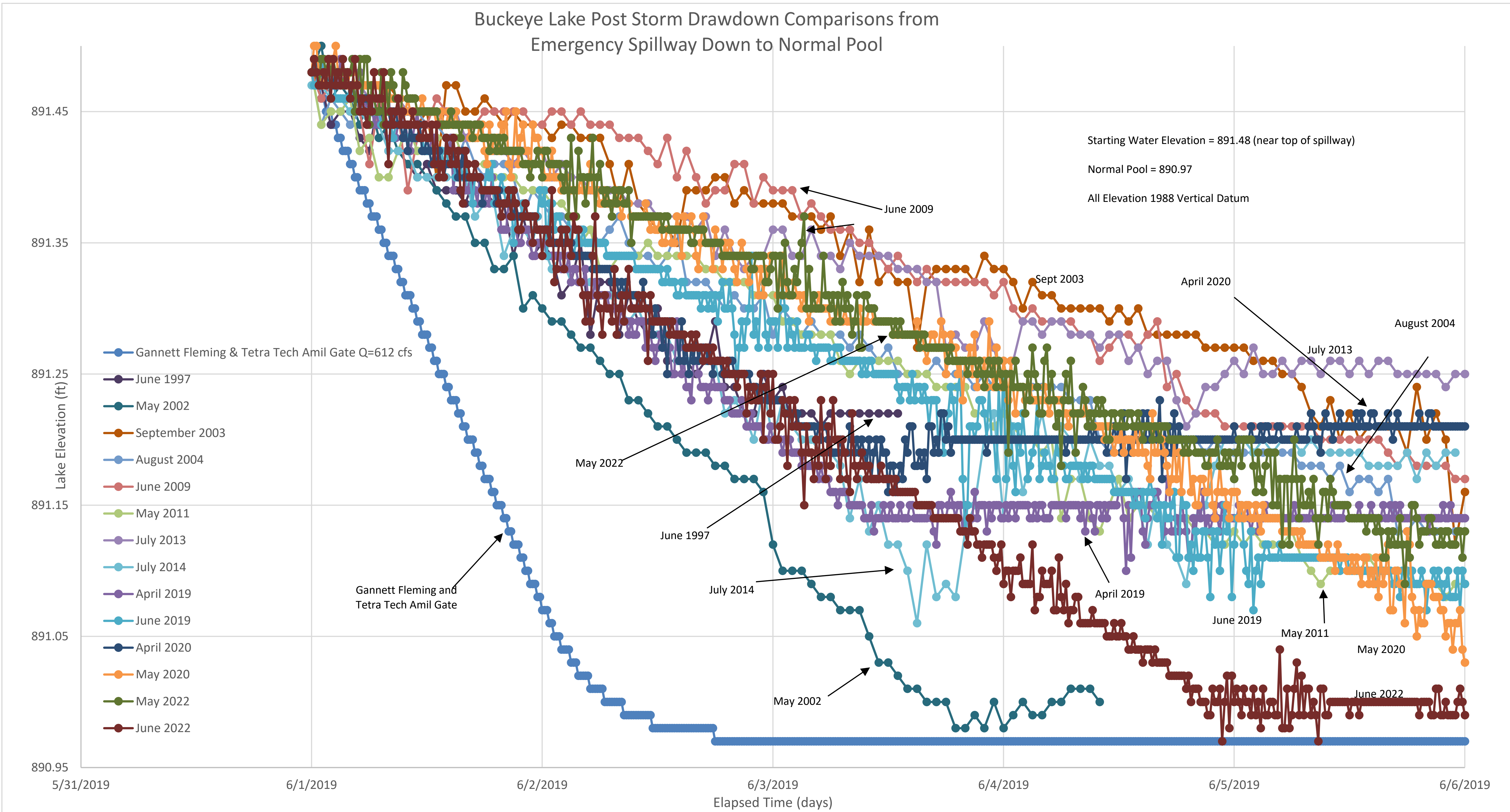
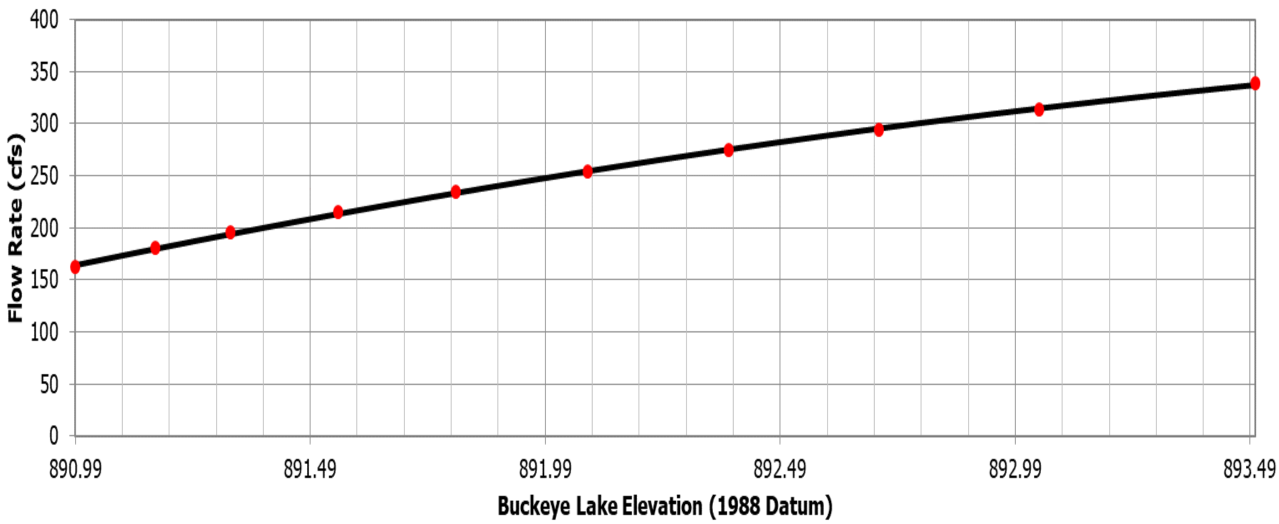


FIGURE 5-6

Recorded Drawdown Curves at the Buckeye Lake Amil Gate (Emergency Spillway Crest to Normal Pool Elevation



$$y = -7833288.5 + 17490.477x + -9.7628751x^2$$

Where x = Buckeye Lake Elevation in Feet

FIGURE 5-7
Calculated Outlet Rating Curve for Buckeye Lake Amil Gate (Summer Operating Condition)

Photos: Amil Gate During Winter Operation; sourced from Gannett Fleming 2018 Report (left)
Buckeye Lake Beacon (right)

Emergency Spillway (Sellers Point)

The emergency spillway is at Sellers Point, near the intersection of SR 360 and 2nd Street, as shown on Figure 5-8. The spillway is a "U" shaped ogee spillway with a length of 465 feet. The crest of the spillway is at an elevation of approximately 891.48 feet (NAVD '88), which is an average of the elevations shown in the 2015 Tetra Tech report and 2018 Gannett Fleming report. The weir elevation is only 0.51 feet above the lake normal pool elevation during the summer operation months. The lower 891.42-foot elevation was used in the SSA model. The concrete pad at the bottom of the ogee spillway is at an elevation of 878.25 feet (NAVD '88). The downstream channel is approximately 2,000-feet long before it empties into the South Fork Licking River channel. The slope of the channel downstream of the spillway is relatively flat with the channel having an invert elevation of approximately 874.0 feet at the confluence with South Fork Licking River, per field survey data.

The emergency spillway is an Ogee shaped spillway that has been modeled consistently at 472-feet long in previous reports. However, when measured using recent aerial imagery the length appears to be closer to 465 feet. The spillway has not been surveyed and the plans have not been available to review to confirm the plan or as-built length or crest elevation. The Tetra Tech report indicates a surveyed spillway crest elevation of 891.54 feet (NAVD '88) while the Gannett Fleming report indicates a surveyed spillway crest elevation of 891.42 feet. It is possible that both elevations existing along the extended length of the Sellers Point spillway.

The previous studies for Buckeye Lake state an ogee weir coefficient of 3.95. However, a calculation method was found online from Professor Victor Ponce at San Diego State University for ogee spillways along with an online rating table calculator at the following web address:

http://ponce.sdsu.edu/onlineogeerating.php

The key variables the online worksheet asks for are spillway length, design head H_D, lake level, and approach velocity. For the Seller's Point spillway, the spillway length is 465 feet with an approach velocity of basically 0. The design head was unknown until we were able to review the spillway detail. Based on the ogee weir equations and review of the record plans, we believe the design head for the spillway was calculated to be 3.23 feet but we have not been given access to the plans or calculations to confirm. Using the online spillway rating curve tool using a design head (Ho) of 3.23 feet, a dam height (P) of 6.62 feet, and a length of 465 feet yields the spillway rating curve shown on Table 5-5 and graphed on Figure 5-9.

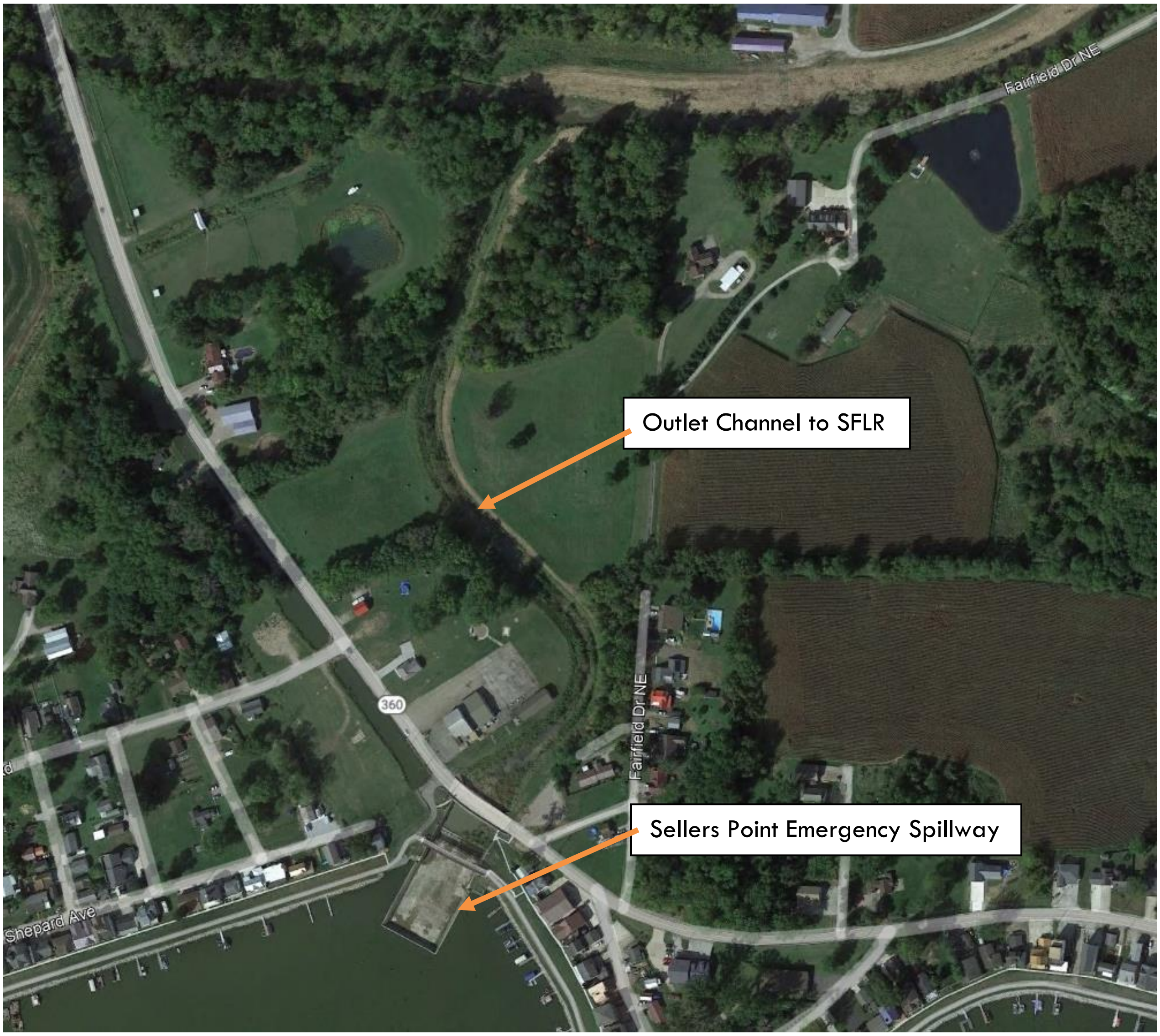


FIGURE 5-8
Buckeye Lake Emergency Spillway at Sellers Point

TABLE 5-5
Emergency Spillway Outflow Rating Comparison

Head (ft)	Buckeye Lake Elevation (ft., NAVD '88)	Ogee Calculator Outflow (cfs)	Weir Outflow C=3.95 (cfs)
0.00	891.42	0.00	0.00
0.323	891.74	272	416
0.646	892.07	799	1,203
0.969	892.39	1,515	2,193
1.292	892.71	2,403	3,364
1.615	893.04	3,434	4,734
1.938	893.36	4,612	6,204
2.261	893.68	5,911	7,801
2.584	894.00	7,344	9,515
2.907	894.33	8,913	11,397
3.23	894.65	10,614	13,328

Other Lake Outlets

There is a historical outlet from Buckeye Lake at the far eastern edge of the lake, referred to as the Thornport Outlet. Based on a discussion with David Snider of the Perry County SWCD, this outlet is no longer active and is not a factor developing a hydrologic model for the lake. There is a 30-inch pipe controlled by a valve to feed water from the lake into a canal and to the fish hatchery north of the lake. The valve is located on the western side of the emergency spillway. The valve is only opened when needed and not a factor in the model analysis of calibration and design storm events.

Buckeye Lake Composite Outlet Rating Curve

A composite outlet rating curve for the Buckeye Lake primary and emergency spillways has been developed for both winter and summer operation and compared to the two previous reports prepared by Tetra Tech and Gannett Fleming as shown on Figure 5-10. The peak flood discharge value from the lake is dominated by the flow through the emergency spillway as the lake elevation approaches 892.0 feet.

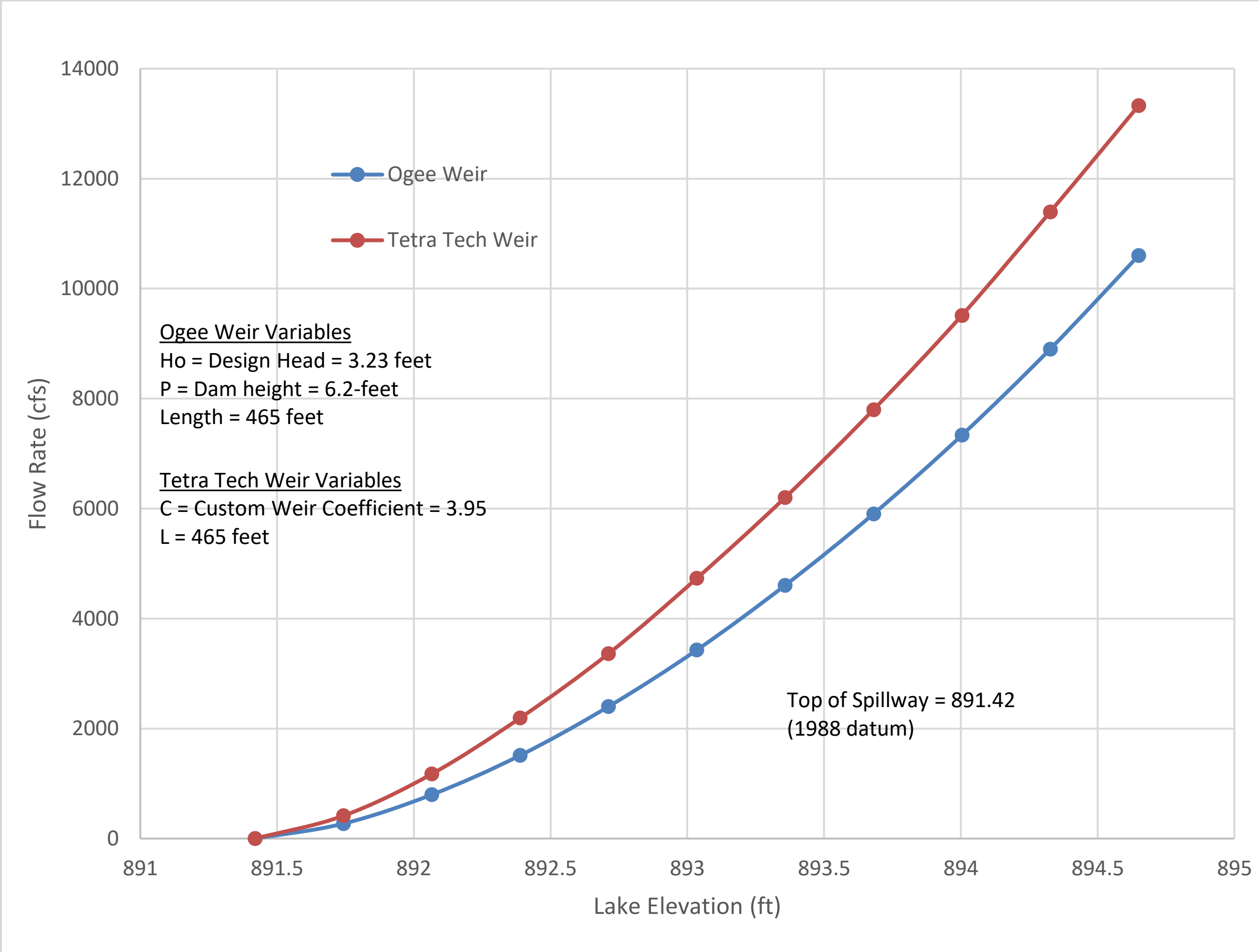


FIGURE 5-9
Peak Flood Discharge Value Rating Curve at Sellers Point Emergency Spillway

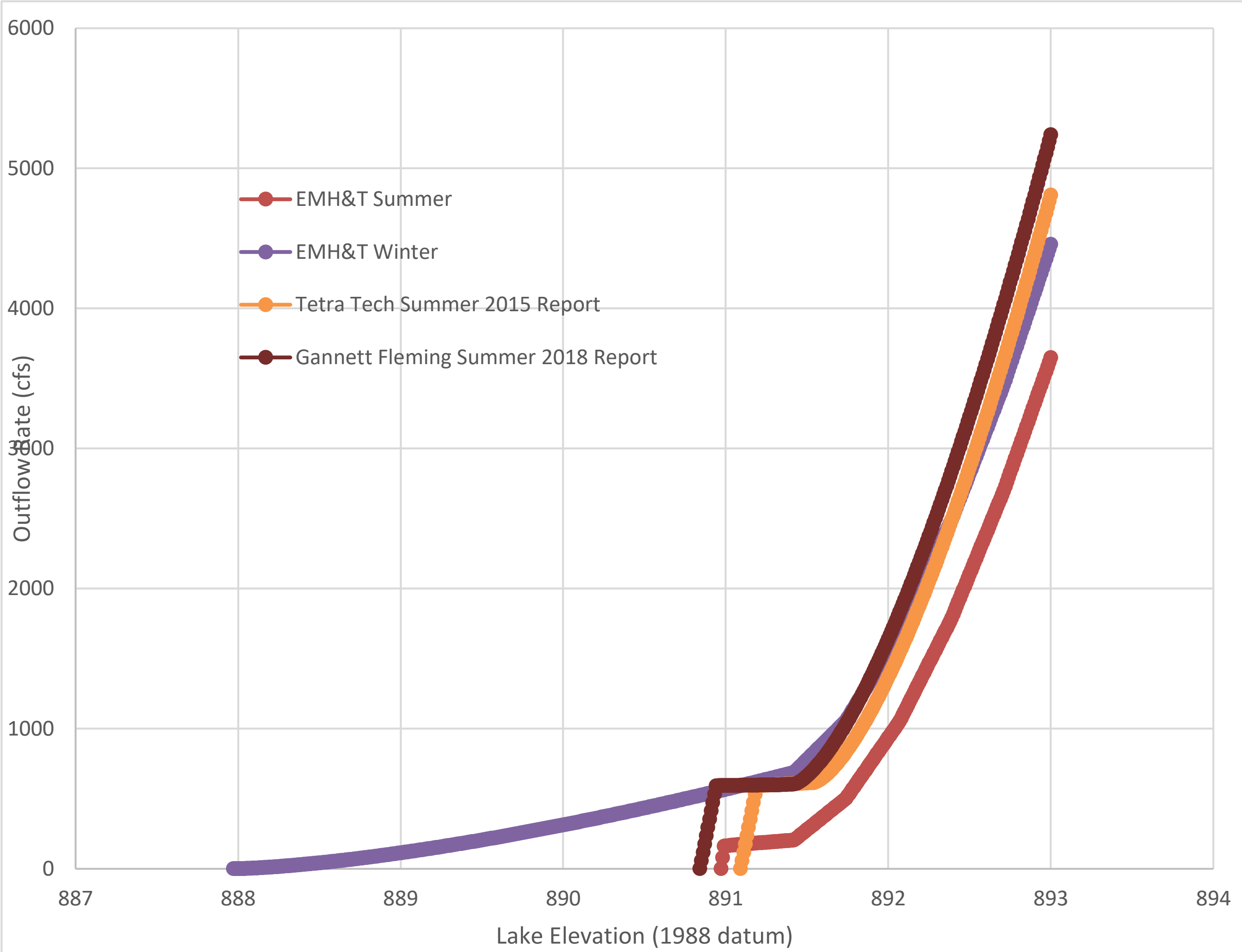


FIGURE 5-10
Composite Rating Curve for Peak Flood Discharge Values Discharging from Buckeye Lake Spillways

5.5.4 Hydrologic Model Results for Buckeye Lake Sub-Watershed

Tables 5-6 through 5-10 summarize the model results of the hydrologic model analysis of the Buckeye Lake sub-watershed, including the Feeder Canal and Buckeye Lake, during both summer and winter operating conditions at the lake. Design storms between the 1-year and 200-year recurrence intervals were considered in the modeling and documented in the tables below. The March 2020 and July 2017 storms were also modeled as calibration events and documented in these tables; however, the July 2017 results at Buckeye Lake are impacted by non-standard operating conditions at the time due to repairs to the dam. Refer to following sections of the report for a more detailed discussion of model calibration.

Observations regarding the outcome of the hydrologic modeling of the Buckeye Lake sub-watershed are provided below:

1. Under winter operating conditions, the Buckeye Lake emergency spillway (Sellers Point) does not activate for any of the design storm events. This reduces the peak flood discharge from the lake to the SFLR from 3,000 cfs. (summer) to 445 cfs. (winter), during the 100-year design storm event.

2. The Buckeye Lake normal pool levels do not have an impact on the Feeder Canal, where there are overflows from the canal toward SFLR.

3. The overflows from the Feeder Canal at the Bloody Run weir and the Palmer Road overflow are significantly larger than the peak flood discharge values from the canal to Buckeye Lake.

4. The March 2020 (calibration) rainfall event appears to be comparable to a 50-year design storm event at Buckeye Lake.

5. Buckeye Lake water surface elevations do not fluctuate significantly, even for the 200-year design storm event the increase in calculated lake level is less than 1.5 feet above the normal summer pool level.

TABLE 5-6
Existing Conditions Feeder Canal Outflow Summary

Storm Event (Recurrence Interval)	Rainfall Depth (in.)[1]	Calculated Peak Flood Discharge Values (cfs.) and Flow Volume (ac-ft.)							
		Through Bloody Run Weir & Adjacent Overflow		Overflow North of Palmer Road		To Buckeye Lake from Feeder Canal		To Buckeye Lake Total[4]	
		Q_{peak}	Vol.	Q_{peak}	Vol.	Q_{peak}	Vol.	Q_{peak}	Vol.
1-year	2.19	228	155	0.0	0.0	192	392	2,868	3,316
2-year	2.62	331	234	0.0	0.0	252	558	3,734	4,016
5-year	3.26	504	367	0.0	0.0	320	827	5,051	5,118
10-year	3.80	664	489	0.0	0.0	378	1,070	6,178	6,090
25-year	4.60	888	676	0.0	0.0	462	1,457	7,864	7,584
50-year	5.28	1,080	839	166	68	526	1,738	9,305	8,825
100-year	5.99	1,370	1,019	513	254	580	1,924	10,789	10,025
200-year	6.83	1,712	1,242	880	522	638	2,101	12,549	11,417
March 2020[2]	5.12	987	792	18	3	479	1,727	10,651	8,983
July 2017[3]	5.59	1842	915	483	152	506	1,742	16,223	9,356

[1]24-Hour rainfall distribution, Huff 3rd Quartile, 50% curve, 10-50 square mile watershed

[2]Starting WSE for Buckeye Lake is 889.89

[3]Starting WSE for Buckeye Lake is 891.74

[4]Includes the sub-watershed area directly to Buckeye Lake

TABLE 5-7
Existing Conditions Peak Flood Elevations Along Feeder Canal and Buckeye Lake (Summer Pool)

Storm Event (Recurrence Interval)	Bloody Run Weir	North Side of I-70	Overflow North of Palmer	Upstream Side of SR 37	Buckeye Lake[1]
1-year	905.93	902.37	901.85	899.26	891.56
2-year	906.37	902.63	902.51	900.46	891.73
5-year	907.00	903.44	903.41	901.60	891.95
10-year	907.52	904.01	903.98	902.46	892.13
25-year	908.23	904.83	904.81	903.57	892.39
50-year	908.81	905.36	905.32	904.31	892.59
100-year	909.17	905.68	905.61	904.85	892.81
200-year	909.41	905.89	905.80	905.39	893.06
March 2020[1]	908.59	905.10	905.07	903.87	892.06
July 2017[2]	909.49	905.67	905.59	904.39	

[1]Starting WSE for Buckeye Lake was 889.89

[2]Starting WSE for Buckeye Lake was 891.74

TABLE 5-8
Existing Conditions Peak Flood Elevations Along Feeder Canal and Buckeye Lake (Winter Pool)

Storm Event (Recurrence Interval)	Bloody Run Weir	North Side of I-70	Overflow North of Palmer	Upstream Side of SR 37	Buckeye Lake[1]
1-year	905.93	902.37	901.85	899.26	889.89
2-year	906.37	902.63	902.51	900.46	889.02
5-year	907.00	903.44	903.41	901.60	889.31
10-year	907.52	904.01	903.98	902.46	889.57
25-year	908.23	904.83	904.81	903.57	889.95
50-year	908.81	905.36	905.32	904.31	890.25
100-year	909.17	905.68	905.61	904.85	890.52
200-year	909.41	905.89	905.80	905.39	890.92
March 2020[1]	908.59	905.10	905.07	903.87	890.32

[1]Starting WSE for Buckeye Lake was 889.89

TABLE 5-9
Existing Conditions Buckeye Lake Outflow (Summer Pool)

Storm Event (Recurrence Interval)	Calculated Peak Flood Discharge Values (cfs.) and Flow Volume (ac-ft.)			
	Sellers Point Spillway[1]		Primary Spillway (Amil Gate)[1]	
	Q_{peak}	Volume	Q_{peak}	Volume
1-year	122	342	216	1,938
2-year	257	854	229	2,008
5-year	608	1,777	245	2,088
10-year	942	2,640	258	2,143
25-year	1,506	3,999	275	2,231
50-year	2,073	5,147	288	2,262
100-year	2,720	6,280	301	2,303
200-year	3,542	7,606	315	2,344
March 2020[2]	788	2,618	253	1,673
July 2017[2]				

[1]Only includes the volume released up to 5-days after start of storm
[2]Starting WSE for Buckeye Lake was 891.74

TABLE 5-10
Existing Conditions Buckeye Lake Outflow (Winter Pool)

Storm Event (Recurrence Interval)	Calculated Peak Flood Discharge Values (cfs.) and Flow Volume (ac-ft.)			
	Sellers Point Spillway[1]		Primary Spillway (Amil Gate)[1]	
	Q_{peak}	Volume	Q_{peak}	Volume
1-year	0.00	0.00	98	1,036
2-year	0.00	0.00	119	1,345
5-year	0.00	0.00	176	1,930
10-year	0.00	0.00	227	2,450
25-year	0.00	0.00	303	3,245
50-year	0.00	0.00	377	3,948
100-year	0.00	0.00	445	4,605
200-year	0.00	0.00	546	5,558

[1]Only includes the volume released up to 5-days after start of storm

5.6 Hydrologic Model Calibration

The initial HEC-HMS and SSA models developed for their respective sub-watershed areas, have been adjusted to reflect calibration to available USGS stream gauge records along the SFLR and separate lake gauge information for Buckeye Lake. The goal was to calibrate the HEC-HMS model to the USGS gauge at Kirkersville, the SSA model to the Buckeye Lake gauge, and the combined hydrologic and hydraulic (2D HEC-RAS) models to the USGS stream gauge at the eastern crossing of SFLR under I-70, and near Hebron. Also, due to documentation of recent flooding of I-70 near the SR-79 interchange, the results of the calibrated 2D HEC-RAS model can be validated against observed flooding at that location. The most recent documentation of I-70 flooding is derived from a drone video from March 21, 2020 at the SR-79 interchange that can be found at the following YouTube link: I-70 @ State Route 79. Refer to the ms consultants' report for further documentation of the 2D HEC-RAS model calibration process.

The March 2020 event was used as the calibration event for this study as it has the best available data and Buckeye Lake was in a normal operating condition. The rainfall from that event occurred March 18 through 20 (total duration of 48 hours) and the distribution of that rainfall was fairly broad across the watershed. The rainfall depth and intensity suggested this storm was a 50-year recurrence interval. Reliable rainfall and stream/lake gauge data was relatively plentiful throughout the watershed for this storm event.

There have been significant rainfall and flood events impacting the SFLR watershed prior to 2020; however, some of the older events preceded improvements to the Buckeye Dam spillway, while others preceded the installation of some of the gauges now available throughout the watershed. A significant rainfall and flood event occurred within the SFLR watershed in July 2017; however, the USGS stream gauge at Kirkersville failed during this event. Furthermore, the operation of the Buckeye Lake spillways was altered from normal operation due to reconstruction of the lake dam. The July 2017 event was captured at the USGS stream gauge near Hebron and was used by ms consultants in their calibration of the 2D HEC-RAS model. Calibration to an August 2021 rainfall event was also considered, but rainfall coverage over the watershed area was not as consistent as it was for some other events.

Model calibration was performed relative to calculated vs. recorded flood elevations and calculated vs. recorded flow volume. The recorded flow volume is determined from the total volume of water discharged through the stream gauges over a specific period of time. Both peak flood elevation and flow volume calibrations are important to this study as they provide a comprehensive evaluation of the ability for the models to replicate the unique conditions of the SFLR watershed.

Flooding along the SFLR downstream of Kirkersville is sensitive to flow volume, which is evidenced by the apparent reduction in peak flood discharge values attributed to natural storage in the watershed between Kirkersville and Hebron. For example, the FEMA flow rates summarized in Table 5-11 at Kirkersville are about the same as Hebron but with significantly more watershed area, including flow contributions from Buckeye Lake. By considering flow volume in the model validation process, the models can provide reliable predictions for various applications, including floodplain management, water resource planning, and both hydrologic and hydraulic structure design.

TABLE 5-11
Summary of FEMA-Published South Fork Licking River Peak Flood Discharge Values

Location	Drainage Area (sq. mi.)	100-year Flood Event (cfs)	500-year Flood Event (cfs)
Raccoon Creek	292.6	24,592	33,839
Lateral C	179.9	13,079	17,634
Hebron Tributary	122.2	7,388	9,067
SR-79, NE of I-70	116.5	7,135	7,331
Kirkersville	47.2	8,093	8,455
Muddy Fork	29.6	6,037	7,713

5.6.1 Calibration Rainfall Data

Rain gauge records throughout and near the SFLR watershed were reviewed for application to the calibration of the hydrologic models. The primary rain gauges of interest are those at the USGS gauge at Kirkersville and at Buckeye Lake near the Primary Spillway (Watkins Island). Other rainfall data was downloaded from the Iowa State University Environmental Mesonet webpage (Webpage) at the recommendation of Julia Dian-Reed of the NWS, Wilmington office. A list of rain gauges evaluated with this study are provided in Table 5-12. The locations of these rain gauges are shown on Figure 5-11.

TABLE 5-12
Summary of Rain Gauge Locations

Location	Period of Record	Identifier
SFLR at Kirkersville	2007 – Present	KRK01
Buckeye Lake at Millersport	2010 – Present	BCK01
Buckeye Lake near Lakeside	2010 – Present	BCE01
Buckeye Lake at Watkins Island	2019 – Present	BCL01
SFLR near Interstate 70	2013 - Present	ISV01
SFLR Near Hebron	2008 – Present	BEE01
Alexandria	2008- Present	ALZ01
Headley Park (Gahanna)	2010 – Present	HPK01
Raccoon Creek at Granville	2007 – Present	GRN01
Heath	2010 – Present	NEK01
Pataskala	2010- Present	PTS01

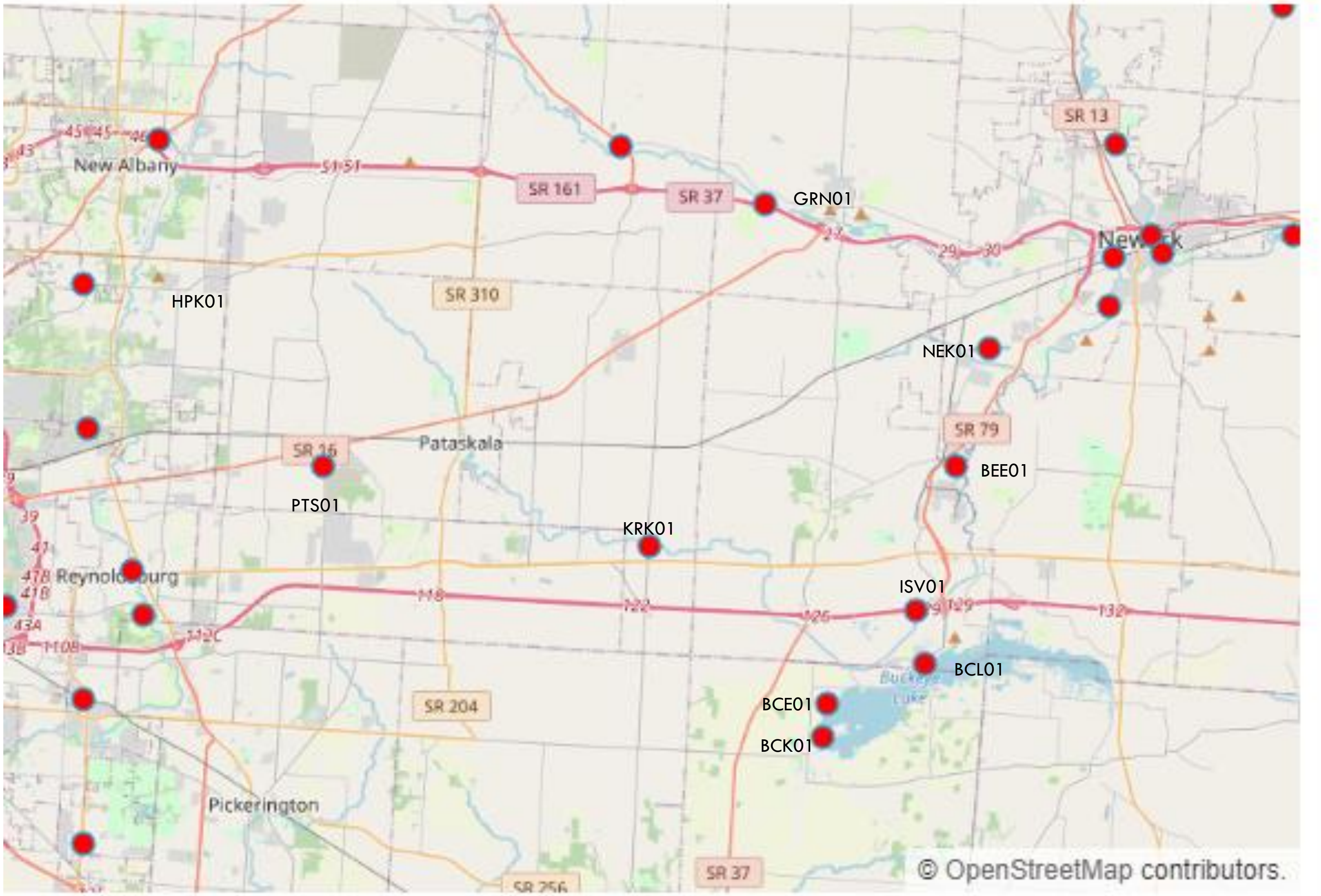


FIGURE 5-11
Location of Rainfall and Stream Gauges

The March 2020 event caused flooding of I-70 for a period of time, and there is considerable documentation of flooding in the Raccoon Creek watershed – flooding of homes and roadways, as well as flooding of roadways and homes in proximity to Buckeye Lake. Several public and private rain gauges showed over 5 inches of rainfall in the upper part of the watershed. Julia Dian-Reed with the NWS provided adjusted radar-estimated rainfall plots showing fairly even rainfall coverage across the watershed except for the southern end of the Buckeye Lake watershed, as shown on Figure 5-12. The adjusted radar plot reflects the NWS's consideration of rainfall gauge information, including CoCoRahs gauges.

Refer to Figure 5-13 for a graphical summary of the rainfall data gathered from various gauges. The largest rainfall depths were recorded at Headley Park in Gahanna, in Pataskala, and near Kirkersville, all within the upper portion of the SFLR watershed. This figure indicates some rainfall gauges appeared to malfunction and not collect all of the rainfall associated with the event, such as the gauges at Heath and near the Lakeside area of Buckeye Lake. The relatively equivalent recorded rainfall depth at these gauges facilitated the model calibration effort for both portion of the HEC-HMS model of the SFLR watershed to Kirkersville, and the AutoCAD SSA model of the Buckeye Lake watershed. For reference purposes, the recorded rainfall depths from the July 2017 event are provided in Figure 5-14.

The Kirkersville rain gauge during the March 2020 event was selected as the primary rainfall data to use for calibration purposes for both the HMS and SSA models. The gauge had one of the highest rainfall depths and is relatively geographically centered in the portion of the HMS model tributary to Kirkersville and the Buckeye Lake watershed.

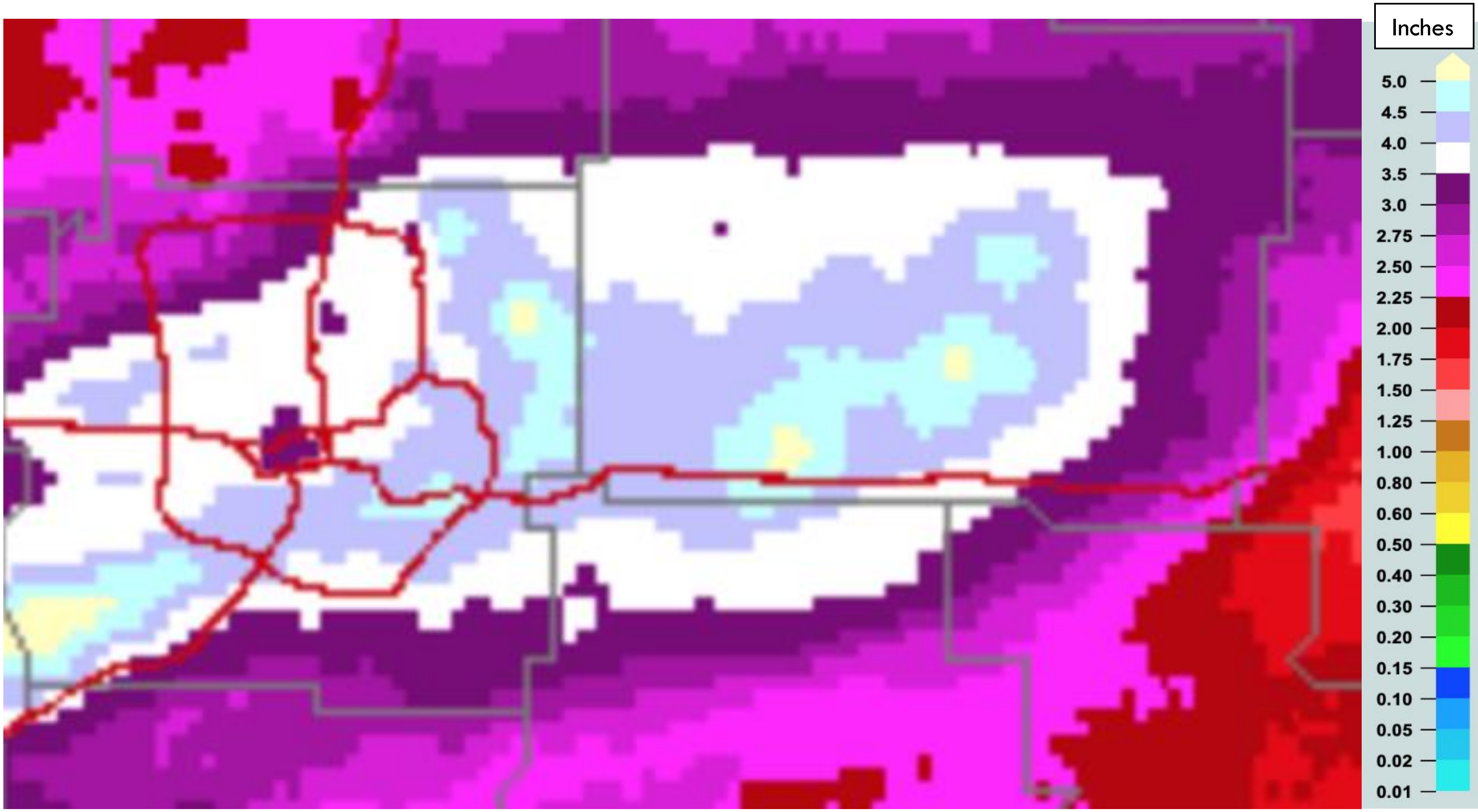


FIGURE 5-12
NWS Adjusted Radar Estimated Rainfall Plot (March 2020)
NWS Note: MRMS Gauge-Adjusted 72 H MRMS Gauge-Adjusted 72 Hour

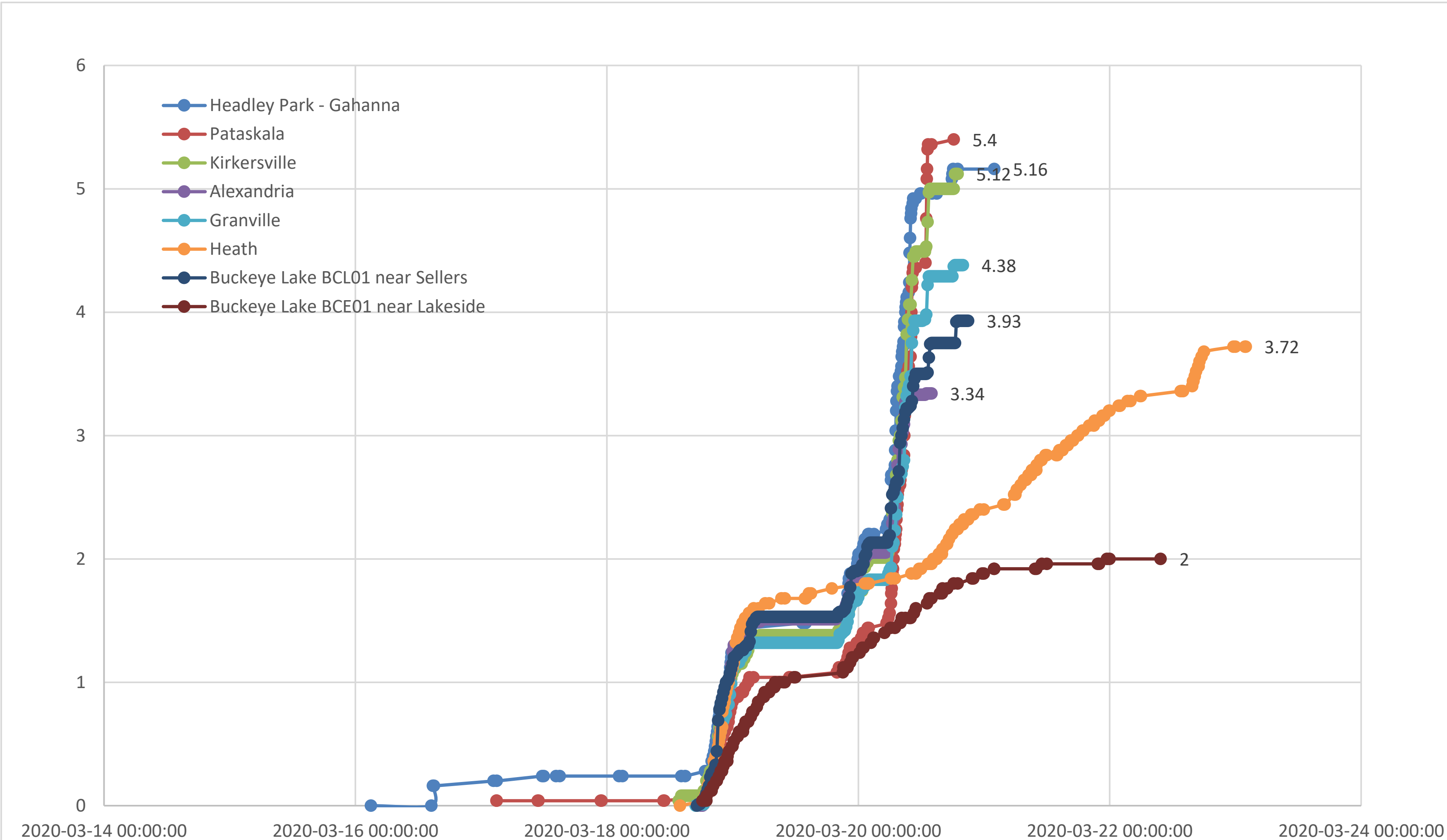


FIGURE 5-13
Summary of Recorded Rainfall Depths (March 2020)

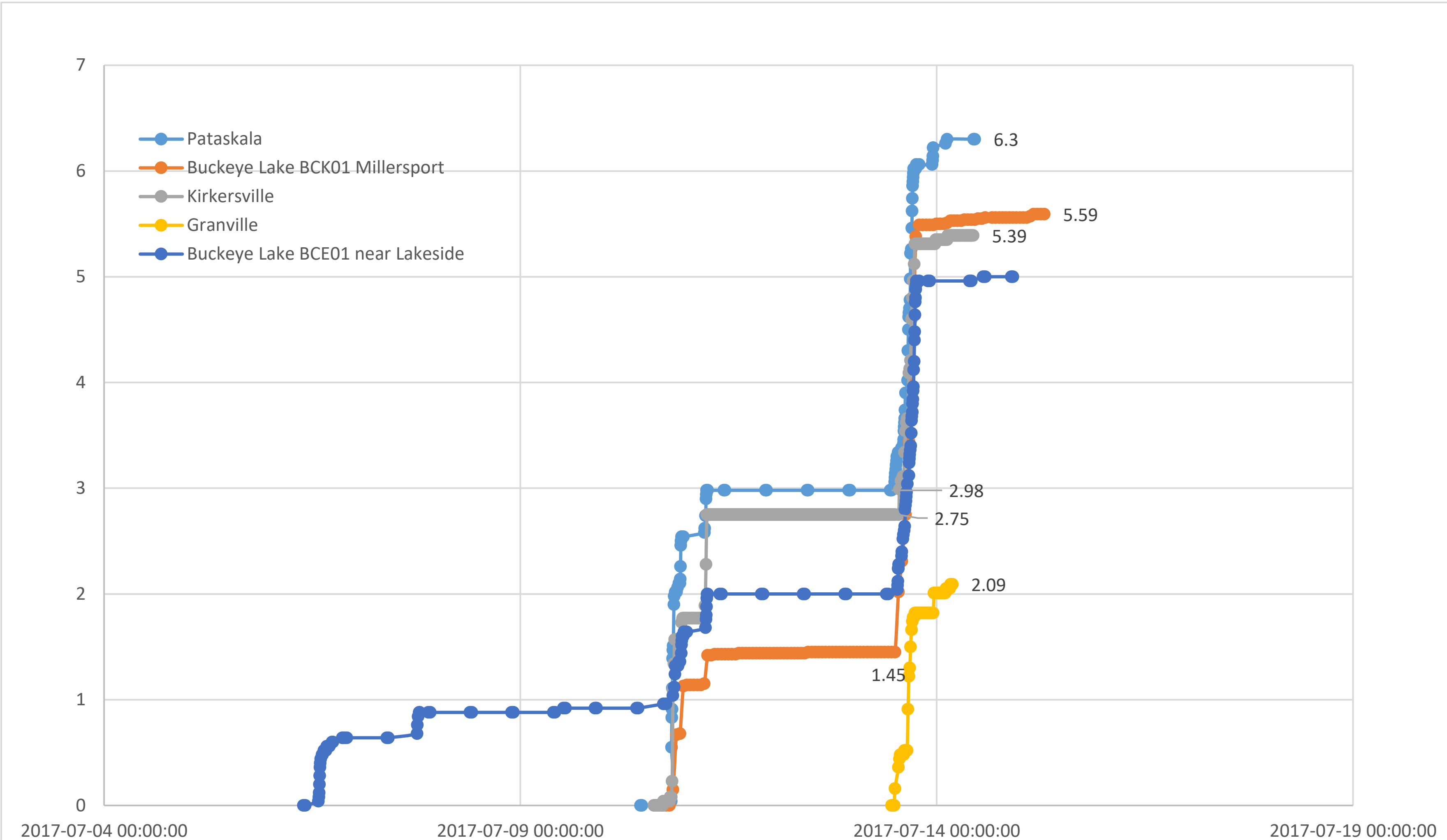


FIGURE 5-14
Summary of Recorded Rainfall Depths (July 2017)

5.6.2 HEC-HMS Calibration at Kirkersville

The HEC-HMS model calibration point is the USGS Kirkersville stream gauge. Once the calibration of runoff parameters to the Kirkersville gauge was determined, those same calibration adjustments were then extended to the other HEC-HMS model subcatchments not tributary to the Kirkersville gauge, such as for the Beaver Run, Bell Run, Dutch Fork, and Ramp Creek watercourses. The 2D HEC-RAS model incorporated the HEC-HMS and SSA model results from the calibration process and calibrated to the USGS Hebron stream gauge and at SFLR and I-70 near SR-79, as well as considering documentation of the roadway flooding during the March 2020 event along SFLR at the SR-79 and I-70 interchange.

Rain Gauge Data

Each subcatchment area within the HEC-HMS model was assigned a rain gauge from Table 5-12 based on geographic proximity, perceived reliability based on consistency with the National Weather Service radar rainfall shown on Figure 5-12. The majority of rain gauges east of Kirkersville were not consistent with the radar data and many of the eastern subcatchments were assigned rainfall data from the Kirkersville gauge. The gauge assignments in the HEC-HMS model are depicted on Figure 5-15.

RCN Value Adjustments

Several iterations of model calibration were executed, including adjustments to RCN and Lag Time to achieve consistency between calculated flood elevations and flow volume. The best calibration outcome occurred by increasing the NRCS Runoff Curve Number (RCN) by a value of 2 over the calculated initial RCN values. The increase in RCN values correlates with achieving a calculated flow volume more consistent with the estimates derived from the USGS Kirkersville gauge. The RCN value adjustment was not applied to a small number of subcatchments that included stormwater detentions basins included in the HEC-HMS model. The RCN value adjustment (+2) was also applied to the other sub-watersheds not tributary to the Kirkersville stream gauge, including the Buckeye Lake sub-watersheds.

With the RCN calibration, the HEC-HMS calculated flow volume at the Kirkersville stream gauge of 338.6 million cubic feet (mcf) was considerably less than the recorded flow volume of 401.1 mcf. However, an investigation of the rating curve for the Kirkersville gauge revealed it may have been overestimating peak flood discharge values, and flow volume for the larger flood events. This finding is based on the two circumstances described below.

For the March 2020 event, the USGS gauge rating curve-based flow rate at the Kirkersville stream gauge was over 11,000 cfs.; however, the highest field measured flow at the gauge was 4,880 cfs from an event in 2008. Most of the other field measured discharge values are under 1,000 cfs., refer to Table 5-13.

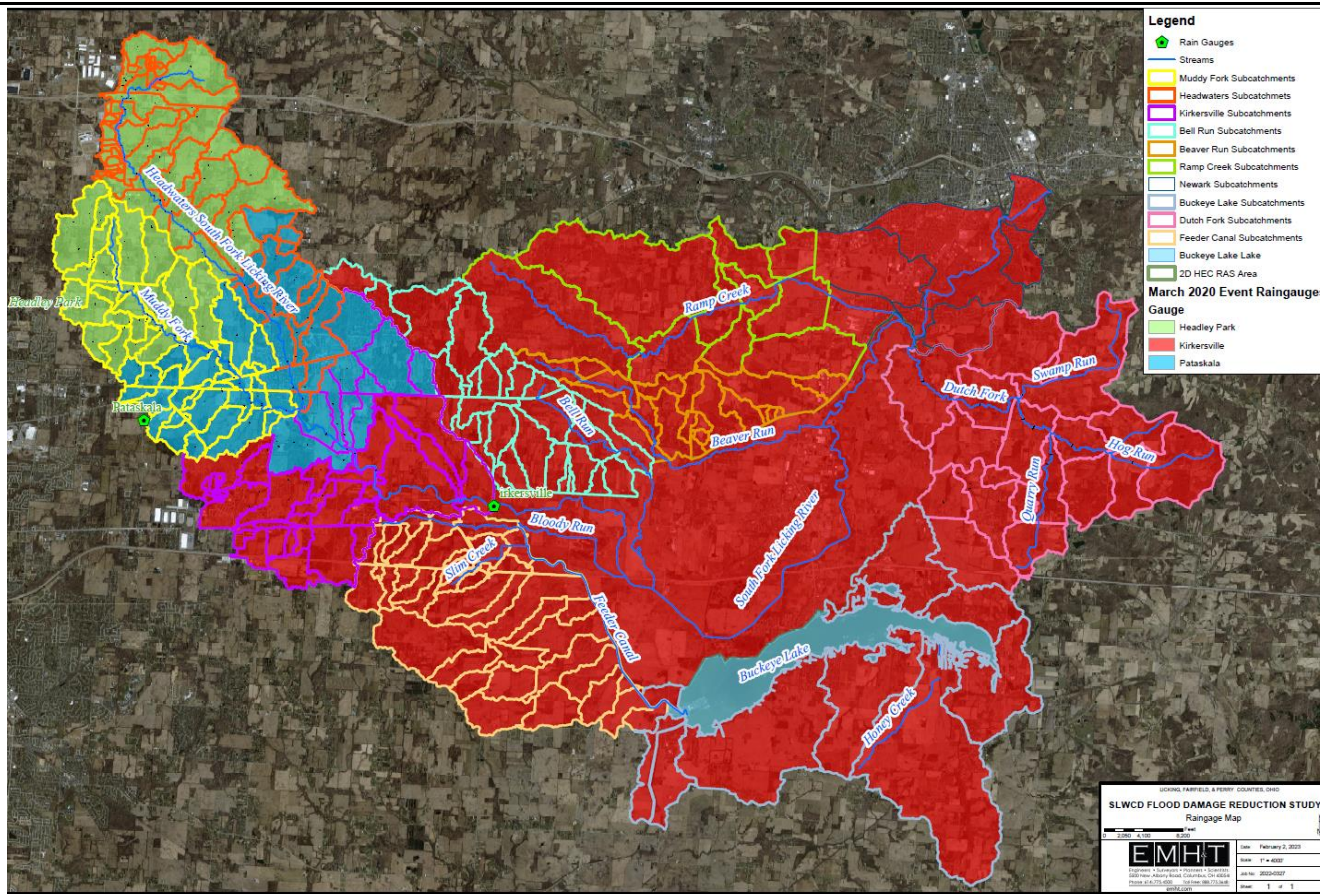


FIGURE 5-15
Selected Rain Gauges for March 2020 Calibration Event

The unsteady-state HEC-RAS model developed by the USGS for the SFLR demonstrates a divergence from the published rating curve at higher peak flood discharge values, as demonstrated on Figure 5-16. The two rating curves in this figure represent the SFLR at the location of the Kirkersville stream gauge. Using the established rating curve at the stream gauge produces a higher peak flood discharge value and flow volume based on the observed gauge reading (flood depth), above 8,000 cfs., compared to using the unsteady-state HEC-RAS model.

Using the unsteady-state HEC-RAS model resulted in a lower calculated peak flood discharge value and flow volume at the Kirkersville stream gauge for the March 2020 event, listed in Table 5-16. Based on coordination with USGS related to these findings, the model calibration process has relied on the results of the unsteady-state HEC-HMS model. The USGS has since adjusted the rating curve for the Kirkersville stream gauge.

TABLE 5-13
USGS Field Measured Flow at the Kirkersville Stream Gauge

Date	Flow (cfs)	Gauge Height (ft)
7/14/17	470	6.00
1/11/08	444	6.07
6/22/18	817	6.55
7/11/17	783	6.64
4/3/18	851	6.74
5/15/14	999	6.94
3/4/08	2,330	9.75
6/26/08	4,880	11.52

<u>Base Flow Adjustments</u>

To achieve a better correlation with the peak flood discharge values and flow volumes estimated from the unsteady HEC-RAS model prepared by the USGS, an additional modification to the HEC-HMS model was made to add a constant baseflow. Adding 50 cfs of constant baseflow to the model increased the calculated flow volume to 344.4 mcf; adding 180 cfs of constant baseflow increased the flow volume to 359.3 mcf, which is close to the calculated value using the USGS unsteady-state HEC-RAS model (360.6 mcf).

<u>Lag Time Adjustments</u>

HEC-HMS GIS Tools were used as the preliminary method for rapidly delineating many basins and flow paths. The tools produce a rough starting point for Lag Time Calculations using flow paths which only delineated in cardinal directions, which means that lag times would likely be too high. Flow paths had to be revised due to limitations of the automated tool at roadways and embankments.

The method of developing Tc flowpaths using GIS tools tended to overestimate the length of the flowpaths. As a result, the computed lag times described previously were reduced by 15% as part of the model calibration process. This adjustment along with the other described hydrologic parameter adjustments helped achieve a better correlation with the peak flood discharge value from the March 2020 event.

Peaking Factor Adjustments

To improve the model calibration to the peak flood discharge value from the USGS unsteady state HEC-RAS model, adjustments to the peaking factor (PF) in the HEC-HMS were considered. The default PF value is 484; however, this value can be increased for steeper terrain and lowered for flatter terrain. To achieve a better correlation between the HEC-HMS model and USGS unsteady state model for the recorded flood depth at the Kirkersville stream gauge for the May 2020 flood event, the PF value has been adjusted to 200; however, this change has only been applied to the portion of the HEC-HMS model to the location of the Kirkersville stream gauge, as the terrain is generally flatter than the remaining watershed areas to the east outside of the 2D mesh area. The PF value adjustment also does not apply to the SSA model of the Buckeye Lake sub-watersheds.

Using a PF value of 200, an RCN of +2, a lag time reduction of 15%, and a baseflow of 180 cfs, yields a March 2020 HEC-HMS peak flow of 9,848 cfs at Kirkersville. In comparison, the estimated peak flood discharge value from the March 2020 event was 8,770. The higher calculated flood discharge value may be conservative, but the calculated flow volume is a very close match to the March 2020 event. A graphical comparison of the HEC-HMS model runs to the recorded flooding from the USGS Kirkersville stream gauge for the March 2020 event is shown on Figure 5-17.

TABLE 5-14
Kirkersville Stream Gauge Calibration Summary (March 2020)
Note: Time Period March 19, 6:00 PM to March 21, 2:00 AM

Model	Flow Volume (mcf)	Peak Flood Discharge Value (cfs.)
USGS Gauge Rating Curve[1]	401.1	11,480
USGS Unsteady State HEC-RAS Model[1,2]	360.6	8,770
HEC-HMS Baseline (no parameter adjustments)	312.4	10,910
HEC-HMS RCN +2; Lag Time -15%	338.6	11,784
HEC-HMS RCN +2, Baseflow 50 cfs; lag time – 15%	344.4	11,834
HEC-HMS RCN +2, Baseflow 180 cfs; lag time -15%, PF Value = 350	359.3	11,964
HEC-HMS RCN +2, Baseflow 180 cfs.'; lag time -15%; PF Value = 200[3]	359.3	9,848
FEMA 50-year		6,699
FEMA 100-year		8,093
USGS StreamStats 50-year		5,860
USGS StreamStats 100-year		6,860

[1] At a stream gauge reading/flood depth of 14.0 feet
[2] Selected calibration targets
[3] Final model calibration

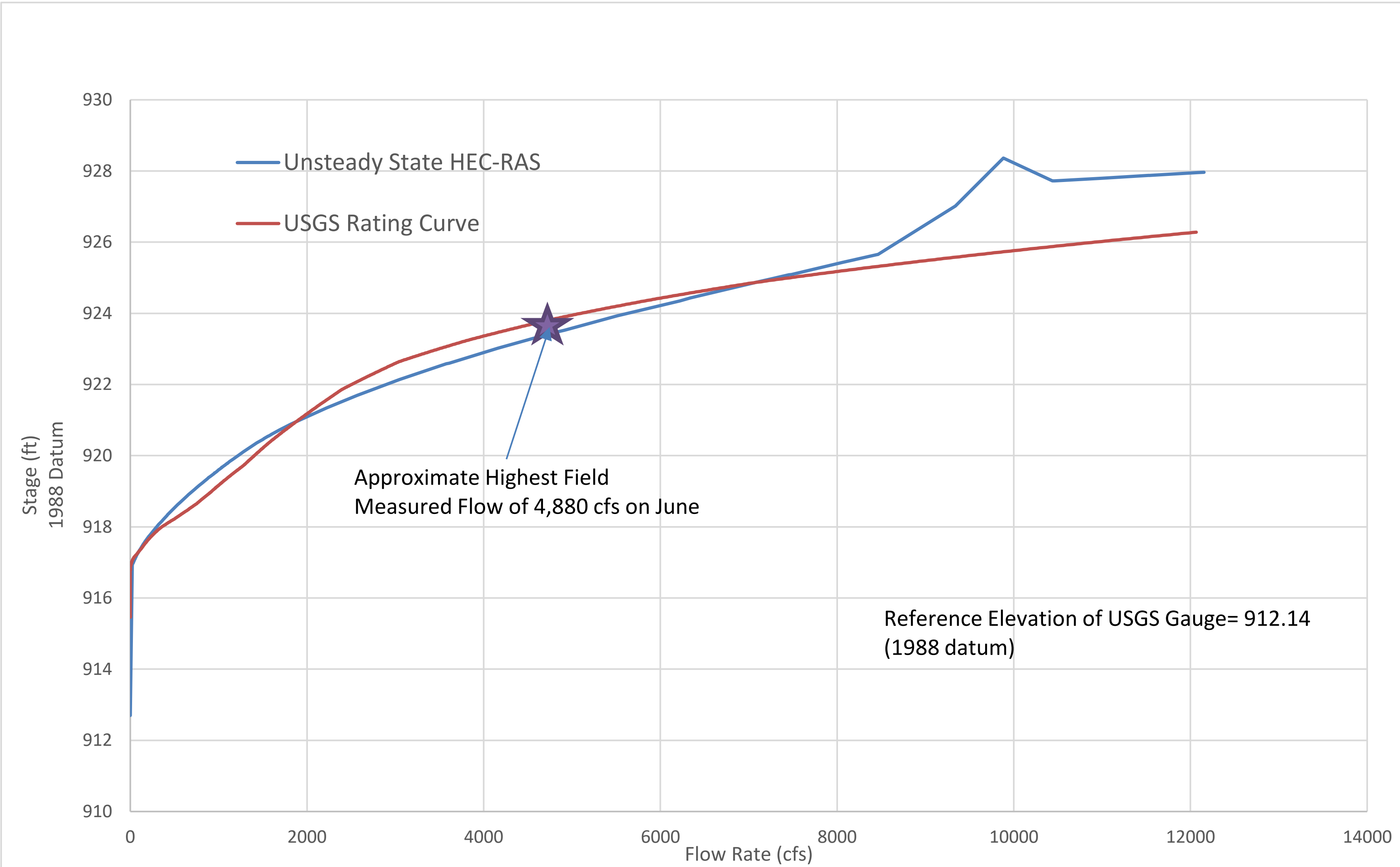


FIGURE 5-16
Flood Elevation Comparison – Calculated vs. Recorded at USGS Kirkersville Stream Gauge (May 2020)

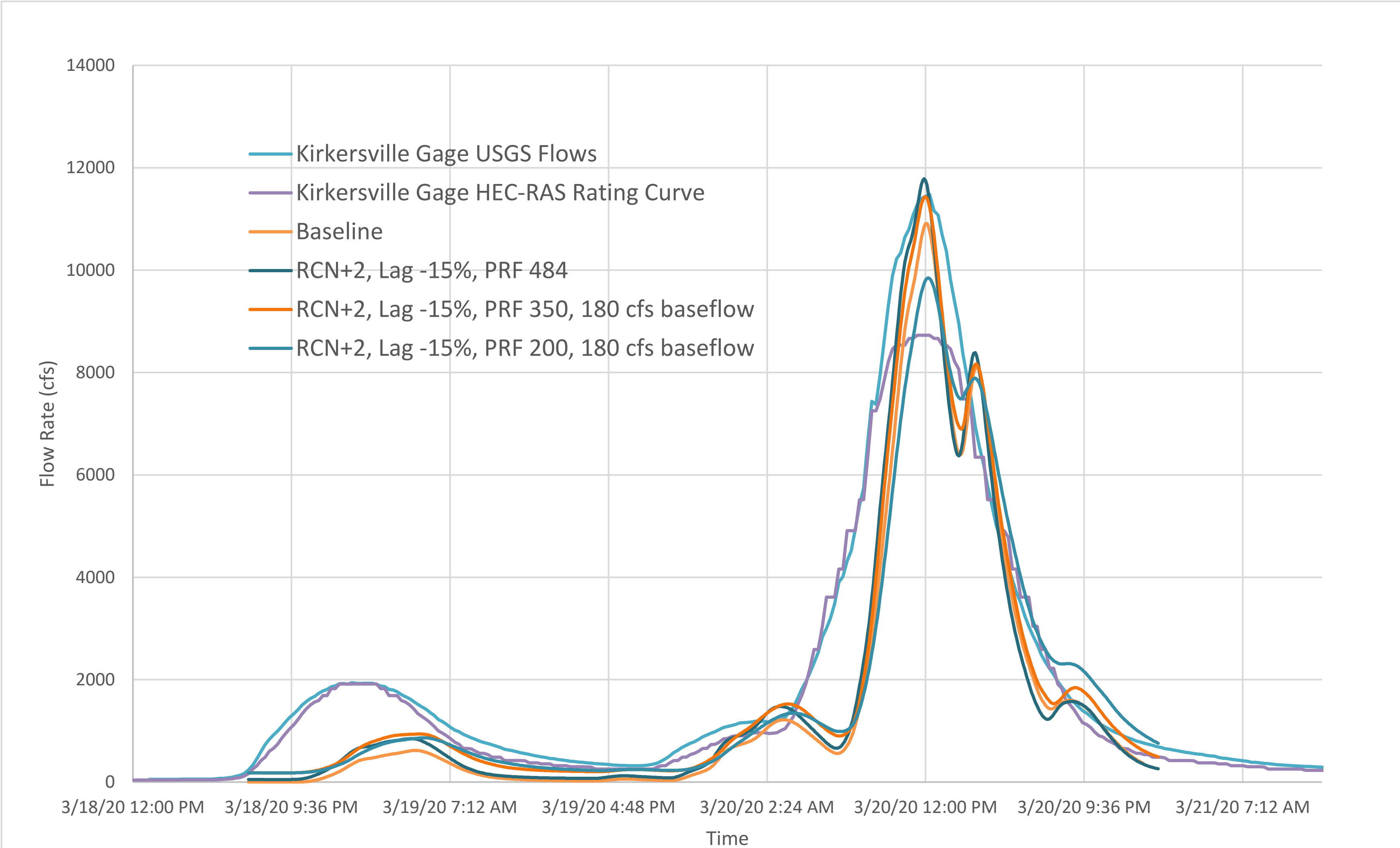


FIGURE 5-17

Flood Elevation Comparison – HEC-HMS Model Calculated vs. Recorded at USGS Kirkersville Stream Gauge (May 2020)

5.6.3 HEC-HMS Calibrated Model Results

Based on the calibration of the HEC-HMS model to the USGS Kirkersville gauge, and the application of the calibration factors to other sub-watershed areas, a HEC-HMS model simulation was performed for the design storm events used to evaluate the flood damage reduction measures described in Section 6. Table 5-15 is summary of the model results for the 100- and 200-year design storm events at specific locations throughout the SFLR watershed. The design HEC-HMS model points can be seen in Figure 5-18

TABLE 5-15
Calibrated HEC-HMS Model Peak Flow Results

Point Label	Description of Location	12-hour Huff 2nd Quartile	
		100-yr Peak flow (cfs)	200-yr Peak flow (cfs)
A	Confluence of Muddy Fork and Headwaters of South Fork Licking River	7049.6	10365.1
B	Kirkersville USGS Gauge	11651.6	16746.4
C	Bell Run at 2D Mesh Area	686.9	1022.6
D	Beaver Run at 2D Mesh	595.3	870.5
E	Ramp Creek at confluence with South Fork Licking River	3338.7	5389.5
F	Dutch Fork at confluence with South Fork Licking river	4728	7682.7

5.6.4 SSA Model Calibration at Buckeye Lake

The AutoCAD SSA model for Buckeye Lake was calibrated to water levels recorded during the March 2020 event at the USGS gauge located near the primary spillway. An attempt was made to calibrate to the July 2017 event, but the atypical operation of the lake made this process unreliable. The only gauge measurement for the lake provides recorded elevations; there are no downstream gauges on either outflow channel that would provide a record of peak flood discharge value for historical rainfall events.

Figure 5-19 shows the calculated lake elevations vs. the recorded gauge elevations for the March 2020 storm event using two different rain gauges with the preferred calibration parameters of RCN +2 with 120 cfs of baseflow. The SSA model can only model one rainfall data set at a time; therefore, separate model runs were executed for the recorded rainfall data at the USGS Kirkersville gauge and at the Buckeye Lake gauge. Both model results are shown on Figure 5-19. The SSA model using the recorded rainfall data from the USGS Kirkersville gauge (total = 5.12 inches) estimated a maximum lake elevation 0.09 feet higher than the recorded elevation.

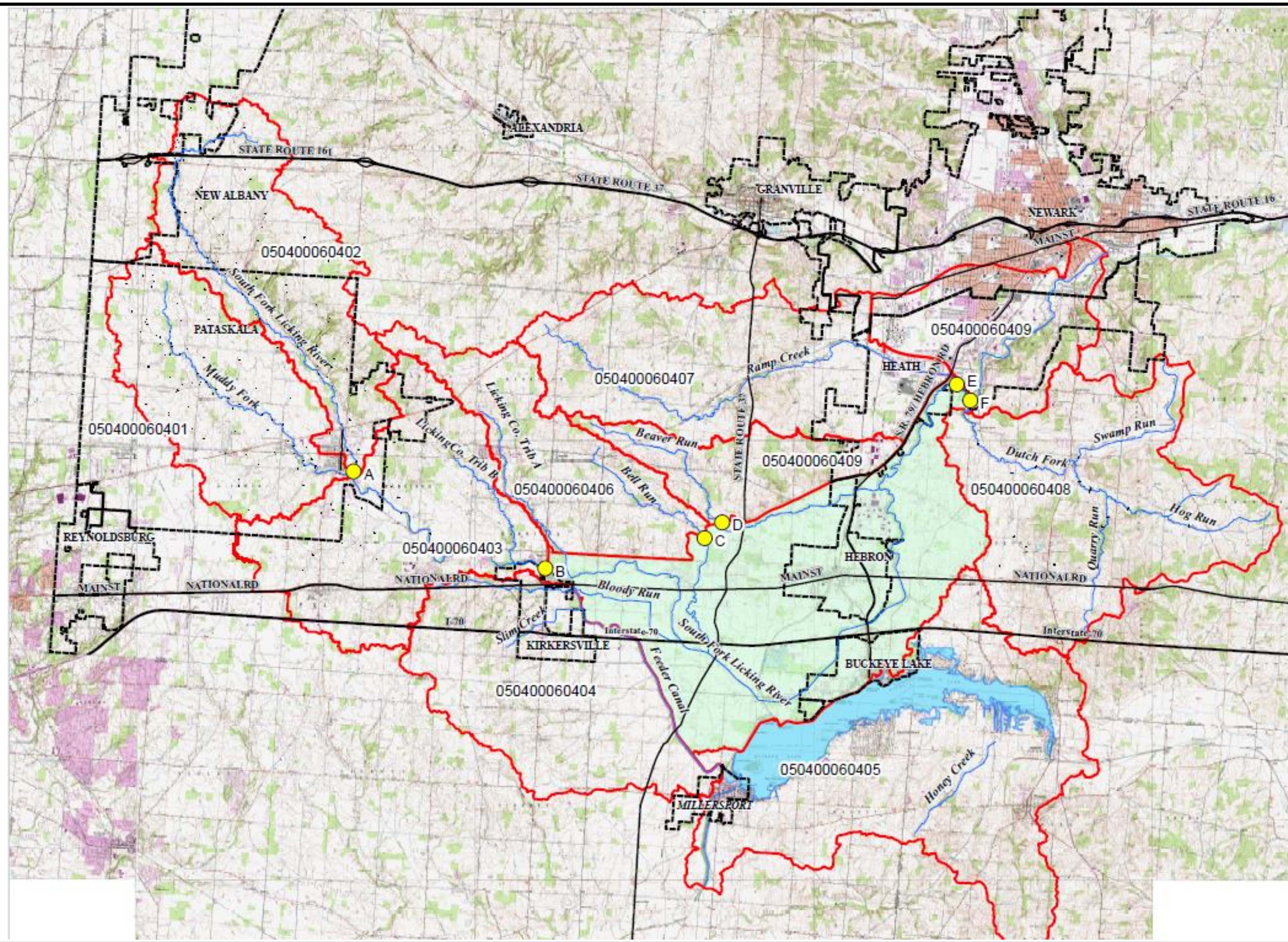


FIGURE 5-18
Map of HEC-HMS Design Point Locations

The model using the recorded rainfall data at the Buckeye Lake gauge estimated a maximum lake elevation 0.39 feet lower than the recorded elevation. This is a reasonable outcome given the likelihood the Feeder Canal and northern lake drainage areas received the higher rainfall amount recorded at Kirkersville, while portions of the lake and southern watershed area likely received the lower rainfall amount. A rainfall data synthesized from the two rain gauges would have likely resulted in a closer calibration to the recorded lake elevations.

The RCN method underestimates runoff volume at the beginning of the storm, typically due to the initial abstraction of rainfall. This explains the lag in SSA model lake elevation vs. the gauged elevation in the early part of the storm. The separation of the lake surface out as a separate subarea (2,910 acres) with an RCN of 100 helped to mitigate some of that effect. A peaking factor adjustment was considered but not applied to the SSA model as the modeled lake elevation was within 0.09-feet of gauged elevation when using the Kirkersville rain gauge. Also, the feeder canal portion of the model showed overflow at Palmer Road similar to the observations discussed previously.

A volume calibration attempt was made for Buckeye Lake using the composite rating curve for the primary and emergency spillways and applying the USGS gauge readings to the rating curve over a set period of time to obtain an estimated outflow volume. For the March 2020 event, the simulation period is from 8:00 AM on March 17th to 12:00 AM on March 24th. The starting water surface was taken from the USGS Buckeye Lake gauge data and was 889.89, about a foot below normal pool as it was filling up from winter pool. The lake elevation at 12:00 AM on March 24th was 891.50, several days after the peak of the storm. The volume of runoff to Buckeye Lake was estimated at 380.2 million cubic feet. This was calculated by taking the estimated volume flowing through the outlet structures plus the volume of water between the end of simulation elevation 891.50 and the start of simulation elevation of 889.89. The latter equals the inflow volume that was not released to the South Fork Licking River during the simulation period.

The volume calibration process involved six different SSA model runs, summarized below.

1. Baseline with Buckeye Lake Rain Gauge Data (3.93")
2. Baseline with Kirkersville Rain Gauge Data (5.12")
3. RCN +2, 50 cfs baseflow, Kirkersville Rain Gauge Data
4. RCN +2, 50 cfs baseflow, Buckeye Lake Rain Gauge Data
5. RCN +2, 120 cfs baseflow, Buckeye Lake Rain Gauge Data
6. RCN +2, 120 cfs baseflow, Kirkersville Rain Gauge Data

Table 5-16 lists the results of each analysis. Five of the six runs underestimated the volume with only the last run exceeding the volume estimate by 3%. The model run using the Kirkersville rain gauge and an RCN increase of +2, along with a constant baseflow to the lake of 120 cfs produced the best result (Calibration Model Run 6). The RCN +2 scenario also worked well with the HEC-HMS model calibration to the USGS Kirkersville gauge. For model run No. 6, the estimated inflow volume is 391.0 million cubic feet compared to an estimate volume of 380.2 million cubic feet. Calibration Model Run 6 produced a Buckeye Lake peak elevation only 0.09-ft higher than recorded for the March 2020 event.

Table 5-16
Buckeye Lake Calibration Summary – March 2020 Event

Calibration Model Run	1	2	3	4	5	6
RCN Adjustment	-	-	+2	+2	+2	+2
Rainfall Gauge	Buckeye Lake	Kirkersville	Kirkersville	Buckeye Lake	Buckeye Lake	Kirkersville
Baseflow (cfs)	0	0	50	50	120	120
Gauge Volume Estimate (mcf)	380.2	380.2	380.2	380.2	380.2	380.2
SSA Model Volume Estimate (mcf)	215.4	312.8	359.2	243.4	283.9	391.0
% Difference	43%	18%	6%	36%	25%	-3%
Recorded Buckeye Lake Peak Elevation	891.97	891.97	891.97	891.97	891.97	891.97
SSA Model Calculated Peak Lake Elevation	891.50	892.06	891.95	891.58	891.58	892.06

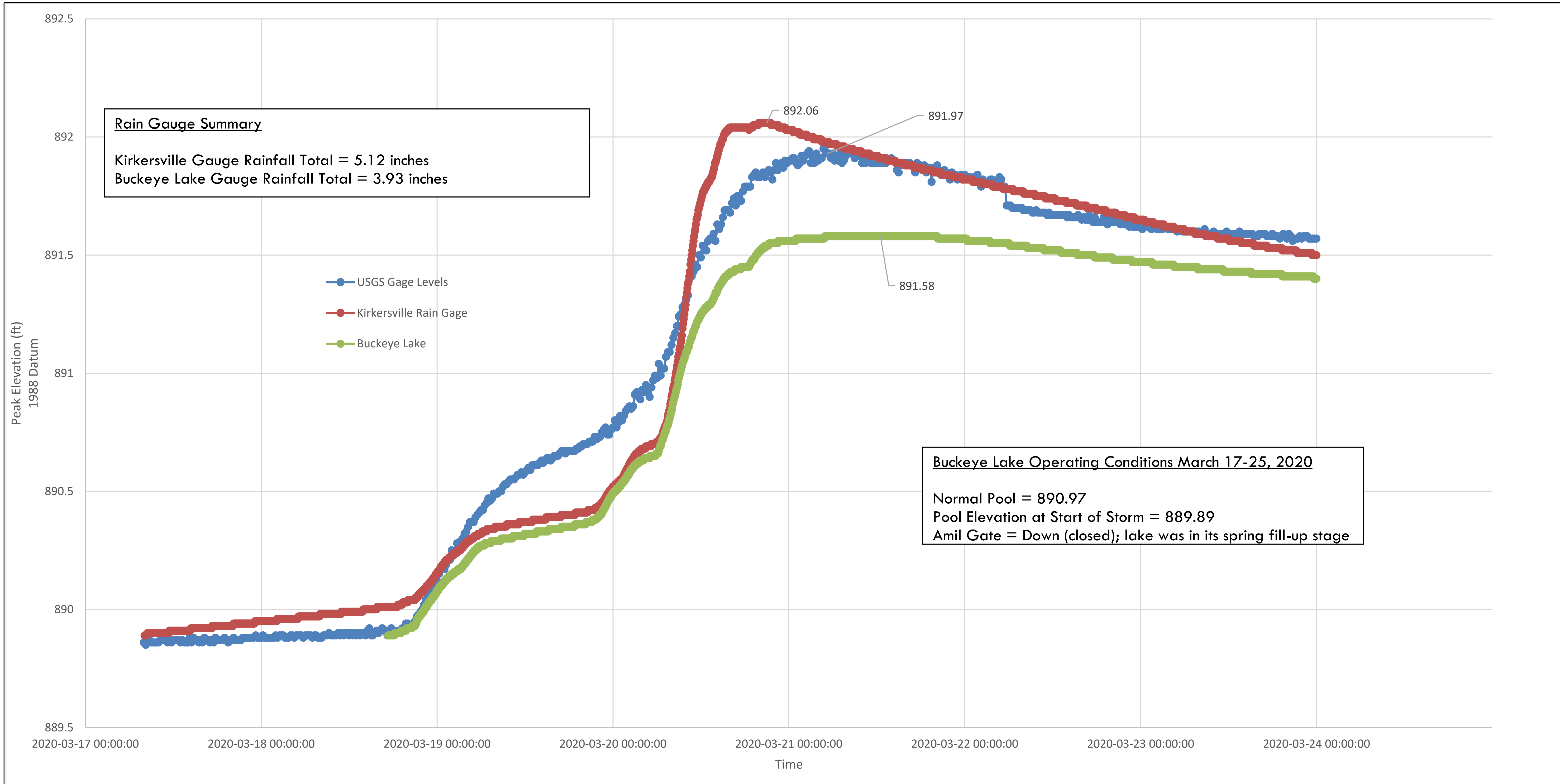


FIGURE 5-19
Buckeye Lake Calculated vs. Recorded Lake Elevations (March 2020)

6.0 EVALUATION OF FLOOD DAMAGE REDUCTION MEASURES

Utilizing the hydrologic model for the SFLR watershed, combined with the 2D HEC-RAS model prepared by ms consultants, flood damage reduction measures were evaluated throughout the watershed. The prior flood damage reduction studies and the results of those studies were factored into decisions on the measures to be considered within this study. Some of those previous studies and their influence on this study are summarized below.

- SCS (1980-1983): The Soil Conservation Service (SCS) developed an original plan of improvements that included multiple regional detention basins, with permanent normal pools for recreational purposes. The subsequent addendums eliminated the recreational uses of these facilities due to the impacts on the Benefit-to-Cost ratio. The SCS study also considered a SFLR diversion channel on the north side of I-70.
- NRCS (2009-2010): The NRCS refined the original SCS study to include only a single regional detention basin along SFLR near the confluence of Bell Run, including the diversion channel on the north side of I-70 (refer to Figure 6-1). A geotechnical investigated related to those measures indicated they may not be feasible or would be costly to construct and maintain. The document pertaining to the geotechnical investigation is provided in Appendix C of this report.

In addition to these past studies, the recent study completed by ms consultants to develop a 2D model also included a preliminary evaluation of flood damage reduction measures. This evaluation considered multiple regional detention basin scenarios, including single and combined detention basins, to determine their impacts on calculated 100-year flood elevations. ms consultants also evaluated scenarios related to bridge replacements, and removal of the large log jam along the SFLR near the Village of Hebron. A summary of the findings of their evaluation is provided below:

1. Widening of bridge spans along SFLR has only a minimal benefit toward reducing flood elevations and the extent of the floodplain. Any benefit is confined to the location of the bridge.

2. Removing the log jam near Hebron has a minimal benefit toward reducing flood elevations and the extent of the floodplain.

3. Applying individual regional detention basins within the tributary watershed to SFLR (e.g., Bell Run, Ramp Creek), has a minimal benefit toward reducing flood elevations and the extent of the floodplain.

4. The most beneficial flood damage protection measures in terms of reducing peak flood discharge values and 100-year flood elevations were a single very large regional detention basin along the SFLR near the Kirkersville gauge, and a combination of the SFLR detention basin and other basins along tributary channels to SFLR. These flood damage reduction measures were determined to decrease flood elevations between 1 and 2 feet at various locations along SFLR.

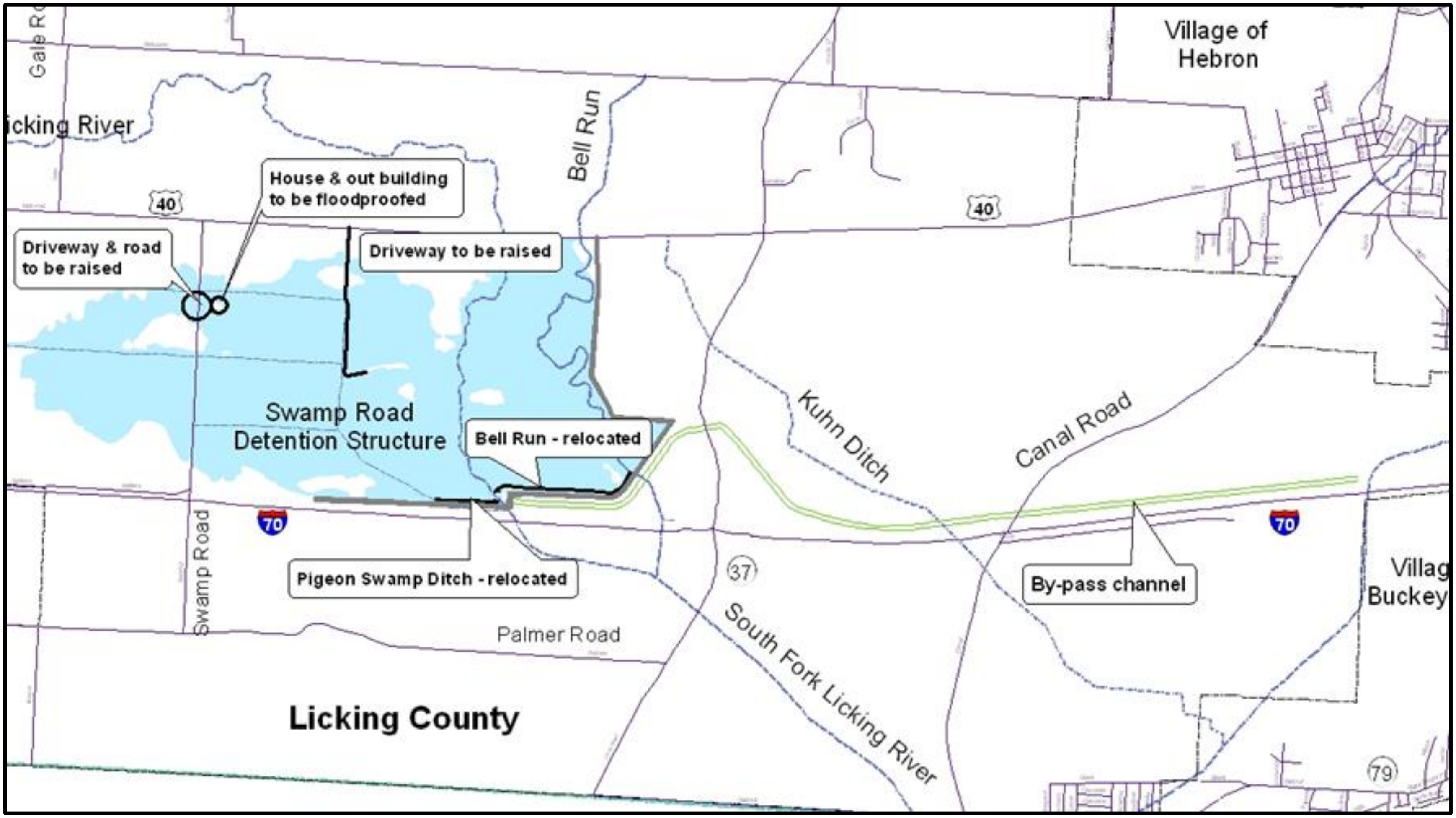


FIGURE 6-1
Regional Detention Basin from NRCS Draft EIS Update Report (2009)

From the evaluation completed by ms consultants, it appeared a 75% reduction in the 100-year peak flood discharge value at the Kirkersville gauge was necessary to achieve the 1- to 2-foot reduction in flood elevations. Based on the results of the combined hydrologic and hydraulic models, it became apparent that even larger reductions in peak flood discharge values can have only a minimal benefit on the 100-year flood elevations and extent of the floodplain along the SFLR between the two I-70 bridges. This appears to be due to the flat gradient of the SFLR though that reach and the low, broad floodplain.

Based on the consideration of the results of the older SCS/NRCS studies and the more recent evaluation completed by ms consultants, it was determined to continue to consider the use of regional detention basins as a watershed-scale flood damage reduction solution, with the conditions listed below:

1. The single regional basin developed by the NRCS along SFLR at the Bell Run confluence was not reconsidered due the aforementioned geotechnical concerns.

2. The regional detention basins would consist of dry dams, without a permanent normal pool. This approach reduces overall project costs related to required earth-moving quantities and avoids onerous permitting requirements related to environmental impacts of a 'wet' basin to the stream channel upstream of the dam.

3. A large regional detention along SFLR near the Kirkersville gauge will be need to be considered as part of the evaluated flood damage reduction measures, to achieve the 75% reduction in 100-year peak flood discharge values.

4. The previous idea of a diversion channel along the north side of I-70 was evaluated again as part of this study, in combination with regional detention basins.

Flood Damage Reduction Measures Not Considered by this Study

Based on the previous studies, this study does not consider improvements to bridges or the SFLR channel based on limited flood damage reduction benefits that would only be local to where those improvements occurred. Also, prior channel improvements to SFLR have already been completed by ODNR along the reach of SFLR between Buckeye Lake and the downstream I-70 bridge. Regarding the I-70 bridges and past flooding of the interstate roadway, ODOT is currently engaged in a separate study to develop its own flood damage reduction solutions. Other structural flood damage reduction measures, such as levees, were not considered as part of this study.

The 2D HEC-RAS model used to evaluate flood damage reduction measures does not include existing log jams and this study does not include an evaluation of log jam removal to reduce local flooding. The location and magnitude of log jams is known to change over time. Log jams that would have a direct impact on flooding conditions, such as at bridges, would be removed as part of an overall channel maintenance plan enacted by the SLWCD, or by the jurisdiction responsible for maintaining the bridge.

6.1 Summary of Regional Stormwater Detention Basins

The watershed mapping compiled as part of this study was utilized to identify strategic locations for dry dams along the SFLR and major tributaries. The dry dams would create a temporary impoundment of flood waters during larger rainfall events; the sunny-day condition along the stream channel would be unchanged from existing conditions. Figure 6-2 provides a simplified representation of a dry dam demonstrating a temporary impoundment of flood waters. The dry dam would consist of the features listed below:

1. An earthen dam embankment spanning the stream channel, with the top-of-dam elevation set to optimize the volume of flood storage while limiting the inundation area upstream of the dam to minimize damages to roadways and buildings.

2. A principal spillway through the dam embankment consisting of a pipe culvert to control the release of flood waters downstream of the dam.

3. An emergency spillway that may consist of an open channel (shown) or a more rigorous concrete spillway to manage larger flood events, or both.

The temporary inundation of flood waters upstream of a dry dam requires fee simple acquisition or a flowage easement on the land impacted by the inundation. Buildings within the footprint of this inundation area may need to be acquired. Roadways may need to be raised if the flood inundation poses a risk in terms of emergency ingress/egress. State of Ohio dam safety regulations, administered by ODNR, would apply to the design of the dry dams. The design criteria for the dams would be based on their classification, which is determined from a combination of the following factors: 1) the height of the dam; 2) the impoundment volume of the dam; and 3) the risk the dam poses to downstream infrastructure and potential loss of life in the case of dam failure. The higher the dam classification, the larger the design burden in terms of the required size of the emergency spillway.

6.1.1 Dry Dam Locations

The study process identified eight dry dam locations along seven different watercourses. Six of the dry dam locations are in the portion of the SFLR watershed upstream of the western I-70 bridge. One of the dry dam locations is in the portion of the SFLR watershed downstream of the Village of Hebron. Each of the locations is described below.

1. Muddy Fork Dry Dam (HUC – 050400060401, Muddy Fork): This location is upstream of Broad Street (S.R. 16) and is one of the largest dry dams in terms of detention storage volume.

2. SFLR Tributary A (HUC – 050400060406, SFLR/Bell Run): This location is upstream of Outville Road and is one of the largest dry dams in terms of height.

3. SFLR Tributary B (HUC - 050400060403, SFLR/Kirkersville): This location is upstream of Refugee Road and is one of the smallest dry dams in terms of upstream drainage area.

4. Bell Run (HUC – 050400060406, SFLR/Bell Run): This location is upstream of Refugee Road and is the smallest dry dam in terms of upstream drainage area.

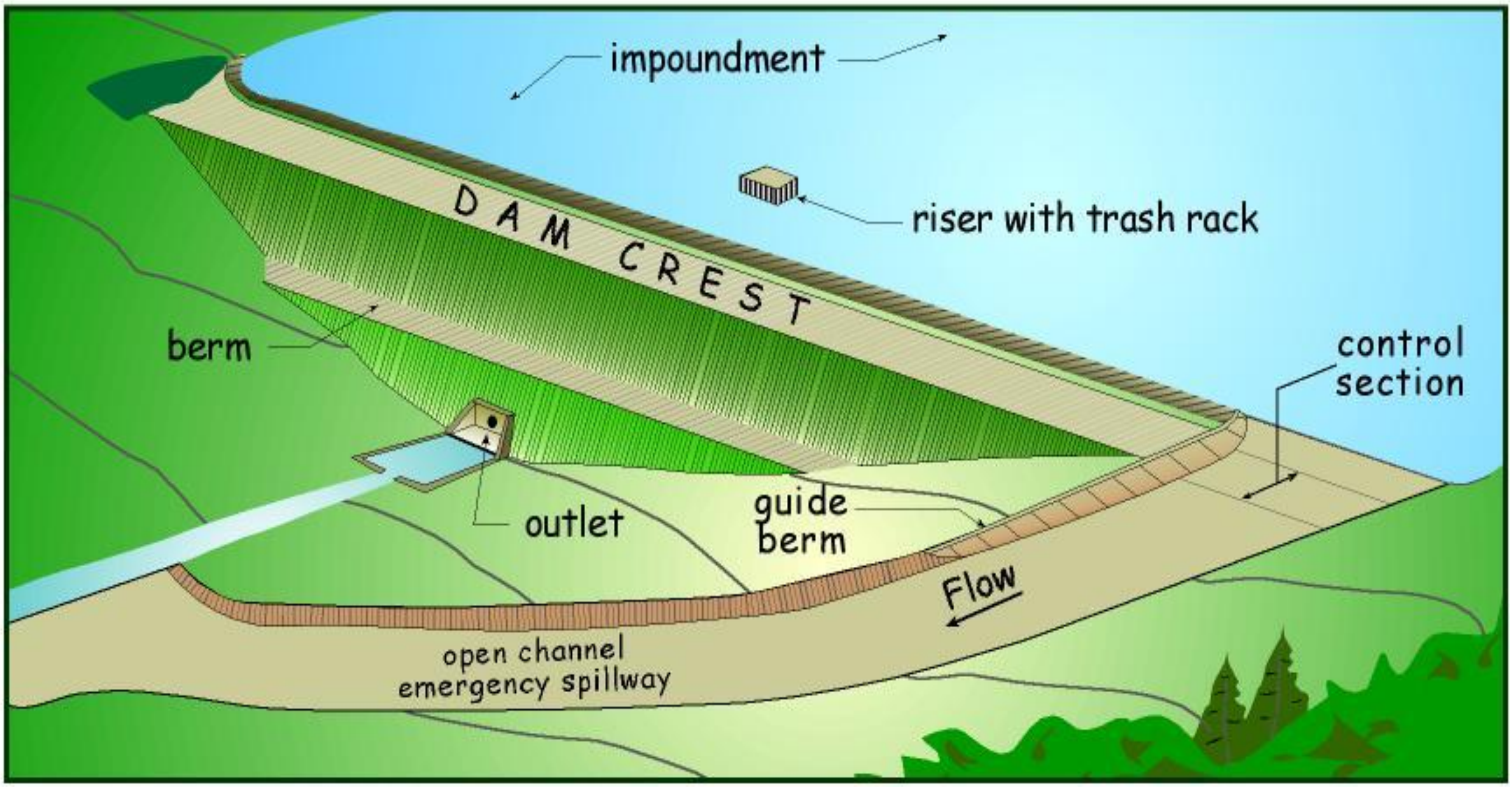


FIGURE 6-2
Illustration of Dry Dam with Temporary Impoundment of Flood Waters

5. Feeder Canal (HUC – 050400060404, Buckeye Lake Reservoir Feeder): This location is along the west side of the Feeder Canal upstream of Swamp Road.

6. Beaver Run (HUC - 050400060409, SFLR/Beaver Run): This location is upstream of Hebron Road/SR 79.

7. SFLR near Kirkersville (HUC - 050400060403, SFLR/Kirkersville): This location is upstream of Outville Road and is the largest dry dam in terms of upstream drainage area, dam height, and detention storage volume.

8. SFLR Headwaters (HUC – 050400060402, Headwaters SFLR): This location is downstream of Old Maids Lane.

6.1.2 Dry Dam Dimensions

A summary of the size of these dry dams is provided in Table 6-1. The Beaver Run dry dam is not included in this table as it was determined through the modeling process to not be incrementally beneficial to the reduction of peak flood discharge values and flood elevations along the downstream reach of SFLR. The final dam safety classification noted in Table 6-1 is based on the highest rating of the three criteria – height, volume, and downstream hazard. Class I is the highest dam safety classification.

TABLE 6-1
Summary of Dry Dam Size and Dam Safety Classification

Dam ID	Drainage Area (sq. mi.)	Dam Height (ft)	Storage Volume (ac-ft)[1]	ODNR Dam Safety Classification			
				Height	Volume	Downstream Hazard	Final
Muddy Fork	10.67	22	1,356	IV	II	I	I
SFLR Tributary A	5.22	34	376	III	II	I	I
SFLR Tributary B	3.17	27	191	III	II	I	I
Bell Run	2.70	19	337	IV	II	II	II
Feeder Canal	5.85	14	658	IV	II	II	II
SFLR Kirkersville	47.2	36	4,040	III	I	I	I
SFLR Headwaters	7.25	23	506	IV	II	I	I

[1]Storage volume used during the 100-year flood event.

The dam height and storage (impoundment) volume upstream of the dam were determined by preparing a schematic-level grading plan for each dry dam embankment using the area-wide topography described previously. The dam height is measured from the stream invert to the top-of-dam elevation described in Section 6.2.

6.2 Model Analysis of Regional Stormwater Detention Basins

The dry dams listed in Table 6-1 and upstream flood impoundments were evaluated in the HEC-HMS model by adding a stage-storage-discharge rating curve representing the dam with a principal and emergency spillway. The detention storage volume rating curve is derived from the aerial-wide topography described previously, extending from the channel invert up to the top-of-dam embankment elevation. The discharge rating curve is based on an iterative model analysis to optimize the size of the principal spillway (primary outlet structure) through the dam to decrease peak flood discharge values while also maintaining the 100- and 200-year flood events within the dam impoundment and below the elevation of the emergency spillway.

The emergency spillway is intended to be used only to pass the design storm associated with the State of Ohio dam classification. For a Class I dam, that is the Probable Maximum Flood (PMF). For a Class II dam, that is 50% of the PMF. Table 6-2 is a summary of the principal and emergency spillway structures associated with each dam.

TABLE 6-2
Summary of Dry Dam Outlet Structures/Spillways

Dam ID	Primary Outlet Structure Culvert Size	2nd Stage Labyrinth Weir		3RD Stage Earthen Weir		Top of Bank Elevation (ft)
		Elevation (ft)	Length (ft)	Elevation (ft)	Length (ft)	
Muddy Fork	6-ft x 4-ft Box Culvert	1017.0 1017.5[1]	50 150	1019.0[1]	600	1022
SFLR Tributary A	6-ft x 5-ft Box Culvert	988.5[1]	280			992
SFLR Tributary B	72-inch Pipe Culvert	964.5	50	968.0[1]	300	972
Bell Run	48-inch Pipe Culvert			918.0[1]	300	920
Feeder Canal	(2) 60-inch Pipe Culvert	916.5[1]	200			920
SFLR Kirkersville	10-ft x 10-ft Box Culvert	945.5 946.5[1]	500 1400			952
SFLR Headwaters	8-ft x 6-ft Box Culvert	1085	350	1086.5[1]	350	1090

[1]Emergency Spillway

The performance of each dry dam was evaluated for both the 100- and 200-year design storm events. The 100-year design storm is the target for flood damage reduction within the SFLR watershed; however, the 200-year design storm was also evaluated as a way of anticipating future conditions and their impact on watershed hydrology. Future conditions impacting watershed hydrology are changes in land use/land cover, which would increase the volume of runoff and peak flood discharge values managed at each of the dry dam locations. In addition, the potential impacts of climate change related to increased rainfall intensity and more frequent major flooding events may result in higher state and federal standards being imposed on the design of dry dams for the purpose of flood control.

Table 6-3 summarizes the results of the HEC-HMS modeling of the seven dry dam locations identified previously, for the 100- and 200-year design storm events. For the 100-year design storm, the peak flood discharge value reduction through the dry dams ranges between 47% (SFLR Tributary B) to 83% (Muddy Fork), with four of the dry dams achieving more than a 70% reduction. These values are also represented in Table 6-3. For the Muddy Fork and Feeder Canal dry dam locations, the 200-year design storm event would discharge through the emergency spillway. Optimization of the primary outlet structure for these two dry dams could mitigate this condition.

TABLE 6-3
Summary of Dry Dam Model Results (100- and 200-year Design Storms)

Dam ID	Drainage Area (sq. mi.)	100-Year Design Storm				200-Year Design Storm		
		Inflow[1] (cfs)	Outflow[2] (cfs)	% Flow Reduction	Elevation[3] (ft)	Inflow (cfs)	Outflow (cfs)	Elevation (ft)
Muddy Fork	10.67	2,600	445	83%	1016.83	3,030	739	1017.9
SFLR Tributary A	5.22	1,889	749	61%	986.07	2,258	1,300	987.37
SFLR Tributary B	3.17	1,022	546	47%	964.10	1,176	750	965.53
Bell Run	2.70	955	182	81%	912.09	1,147	193	913.18
Feeder Canal	5.85	1,687	474	72%	916.18	2,016	503	916.91
SFLR Kirkersville	47.2	9,240	2,364	75%	945.19	10,770	4,011	946.34
SFLR Headwaters	7.25	1,994	892	55%	1084.93	2,357	1,573	1085.6

[1]Peak flood discharge value to the dry dam
[2]Peak flood discharge value leaving the dry dam
[3]Maximum ponding elevation upstream of the dam

The results of the HEC-HMS models with the seven dry dams were transferred and integrated into the 2D HEC-RAS to determine the benefits in terms of reducing calculated flood elevations and the correlating extent of the floodplain. Only the 100-year design model results were considered for this purpose. Hydrographs from the HEC-HMS model were used as input to the 2D HEC-RAS model at numerous locations along the watershed interface with the 2D model area, as described in the report prepared by ms consultants referenced previously, and shown on Figure 6-3.

FIGURE 6-3
Watershed Interface Points between HEC-HMS and 2D HEC-RAS Models
(Image prepared by ms consultants)

Table 6-4 is a summary of the integrated HEC-HMS and 2D HEC-RAS model results, indicating the reduction in 100-year peak flood discharge values and flood elevations at the Points of Interest shown on Figure 6-4. The results depicted in this table are summarized below:

1. The combination of dry dams results in an 82% reduction in the peak flood discharge value along SFLR at the USGS Kirkersville gauge, correlating to a 2.7-ft. reduction in the 100-year flood elevation.

2. The 2.7-ft. reduction in the SFLR 100-year flood elevation carries to the I-70 bridge near SR 37.

3. The dry dam along the Feeder Canal results in an 88% reduction in the overflow of the canal to SFLR. The peak flood discharge from the Buckeye Lake spillways to SFLR is essentially unchanged.

4. The combination of regional stormwater detention basins results in a 53% reduction in peak flood discharge values at the I-70 bridge near S.R. 79, and a correlating 2.1-ft. reduction in the 100-year flood elevation.

TABLE 6-4
Summary of Integrated Model Results (Dry Dams)

Point of Interest	100-Year Peak Flood Discharge (cfs.)			100-Year Flood Elevation (ft., NAVD 1988)				
	Location	Existing	With Dry Dams	Location	FEMA-Published	Existing	With Dry Dams	Reduction (ft.)
A	SFLR at Kirkersville	13,205	2,404	Outville Road Bridge	925.0	923.4	920.7	-2.7
B	Overflow from Feeder Canal to SFLR	1,883	221	I-70 Bridge Near SR 37	897.0	897.0	894.3	-2.7
C	From Buckeye Lake (both spillways)	3,021	3,017	At Sellers Point Spillway Channel to SFLR	888.0	886.1	885.2	-0.9
D	At I-70 near SR. 79	9,023	4,246	I-70 Bridge	884.5	884.3	882.2	-2.1
E	At Village of Hebron	7,862	5,669	Upstream of US 40	879.0	879.6	877.4	-2.2

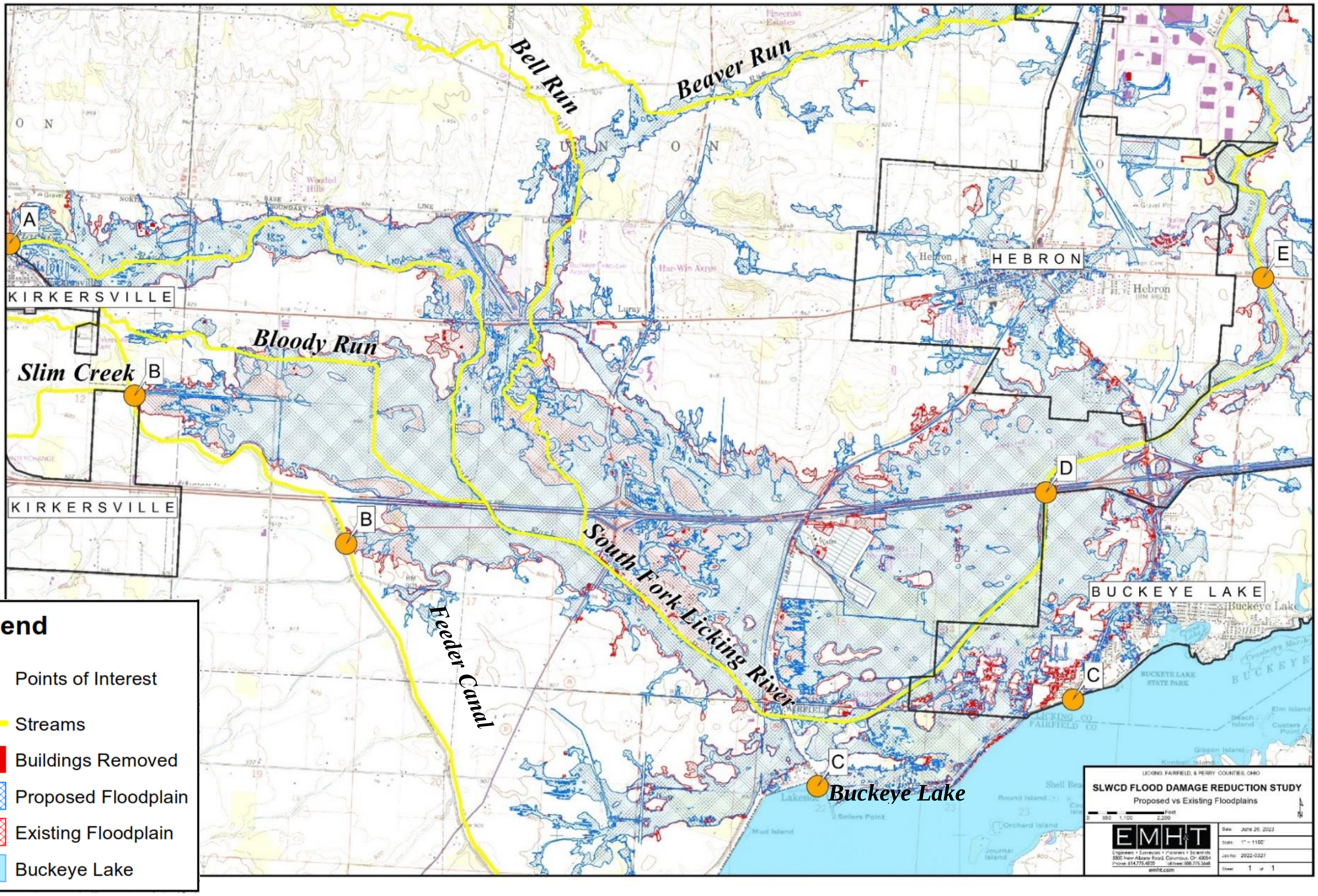


FIGURE 6-4
Points of Interest for Integrated Model Results (with Dry Dams)

The model analysis of the seven dry dams has not been optimized to determine if one or more of the smaller dams could be removed from the analysis and still have similar results in terms of reduced peak flood discharge values downstream along SFLR. The largest dry dam, located along SFLR at Kirkersville, has the most significant impact on reducing downstream peak flood discharge values. The other larger dry dams along Muddy Fork and in the SFLR headwaters improve the performance of the Kirkersville dry dam and would provide their own flood damage reduction benefits downstream of their locations. The benefits of those dry dams in terms of reduced 100-year flood elevations and extents of flooding have not been calculated as part of this study as they are outside of the 2D HEC-RAS model boundary.

Model Analysis of I-70 By-pass Channel

As mentioned previously, this study reconsidered the by-pass channel along the north side of I-70, in conjunction with the seven dry dams. The by-pass channel would divert a portion of the flow in SFLR (including Bell Run) at the upstream I-70 bridge and direct that flow along the north side of I-70, discharging back to SLFR at the downstream I-70 bridge. The intended hydrologic impact of the by-pass channel would be to significantly decrease the flow in SFLR between the two I-70 bridges.

For this analysis, a by-pass channel with a 50-foot bottom width and 3:1 side to slopes was integrated into the 2D mesh associated with the HEC-RAS model. The depth of this channel varied along its course, and the gradient of the channel was very small (approximately 0.1%). The model results associated with this analysis demonstrated a reduction in the 100-year peak flood discharge value along SFLR between the two I-70 bridges; however, there was also an increase in the 100-year peak flood discharge value further downstream along SFLR near the Village of Hebron. Given the noted adverse impact and the construction challenges documented in the NRCS geology report, the by-pass channel has not been further evaluated in terms of determining project benefits and construction costs.

6.3 Calculation of Flood Damage Reduction Benefits

A simplified analysis was performed to estimate the flood damage reduction benefits associated with the combination of the seven dry dams. These benefits were determined based on the change (reduction) in the extent of the 100-year flood inundation area attributed to the dry dams, and the associated reduction in estimated flood damages. The flood inundation area for the existing and proposed (with dry dams) conditions was derived from the 2D HEC-RAS model. The flood inundation areas derived directly from the 2D model had to be refined and manipulated to eliminate flood inundation areas associated with the rainfall-on-mesh application used to generate runoff within the 2D HEC-RAS model. Isolated ponding areas not directly related to SFLR 100-year flood inundation areas were eliminated.

The refined flood inundation areas for the existing and proposed conditions were then applied as an overlay within GIS to estimate the acreage of land and the number of buildings impacted by flooding. Property and building values obtained from the Licking County Auditor and integrated into the GIS overlay were then used to estimate flood damages associated with the inundation areas. The property/building valuation obtained from the county had to be refined to address multiple issues, such as parcels (or combined contiguous parcels) with multiple buildings and the valuation assigned to only a single building, or repeated across multiple buildings. In addition, the county's data did not have a value assigned to mobile homes. For the purpose of addressing these irregularities, judgement was used to properly disperse building valuations on single/contiguous parcels, and a value of $50,000 was assigned to all mobile homes. In addition, smaller, accessory buildings (e.g., detached garages, sheds) were not included in the determination of building flood damages.

The estimates flood damages to land is based on 25% of the market value; the estimated flood damages to buildings is based on 50% of the market value. The market value is based on the auditor's values for land and buildings multiplied by 1.5; a 50% increase. These factors are generalizations in lieu of a more complex method for estimating flood damages based on multiple flood recurrence intervals and flooding depths, which would be part of a formal appraisal of flood damages and benefits. Furthermore, this estimate of flood damages and benefits does not consider other (indirect) factors, such as traffic detours, emergency services, and general economic factors affecting communities during a flood event. A more precise evaluation of crop damage in the expansive flood prone agricultural lands may also yield a different flood damage multiplier than applied in this study. Table 6-5 is a summary of the amount of flooded land and buildings, which impacts areas within Licking and Fairfield Counties, under existing and proposed (with dry dams) conditions.

TABLE 6-5
Summary of Flood Inundation of Land and Buildings

County	Flooded Land (Acres)			Number of Flooded Buildings		
	Existing Conditions	Proposed Conditions	Reduction	Existing Conditions	Proposed Conditions	Reduction
Licking County	6,021	4,417	1,604	961	563	398
Fairfield County	478	430	49	235	176	59
TOTALS =	**6,499**	**4,847**	**1,652**	**1,196**	**739**	**457**

Table 6-6 is a summary of estimated flood damages to buildings and land under existing and proposed conditions. The difference in these values is the accrued benefits associated with the seven dry dams. The accrued estimated benefits of reduced flooding to land and buildings is $51.5 Million.

TABLE 6-6
Summary of Estimated Flood Damages and Benefits (Millions of Dollars)

County	Building Flood Damages (50%)			Land Flood Damages (25%)		
	Existing Conditions	Proposed Conditions	Reduced Flood Damages	Existing Conditions	Proposed Conditions	Reduced Flood Damages
Licking County	$69.9	$34.3	$35.6	$29.3	$19.0	$10.3
Fairfield County	$27.2	$25.2	$2.0	$18.0	$14.4	$3.6
SUB-TOTALS =			**$37.6**			**$13.9**
TOTALS =	**$51.5 Million**					

6.4 Construction Costs Estimates

The construction cost estimates for the seven dry dams are based on the schematic level grading plans prepared for each dam, and the model analysis that determined the outlet structures at each dam. The gradings plans were used to estimate the earth moving quantity associated with each dam, both excavation and embankment quantities. Other material and labor costs were associated with the construction of the outlet structures. Some of the dams required a concrete spillway structure, which was factored into the cost estimate.

6.4.1 Material and Labor Costs

Figure 6-5 is an example of a construction cost estimate for one of the seven dry dams (Muddy Fork), indicating the level of detail associated with assigning: 1) General items common to most construction projects; 2) Dam items specific to the earthen dam embankment; 3) the primary outlet (principal spillway) through the dam embankment; and 4) the concrete weir (if applicable) and emergency spillway. Unit costs typical of large earthmoving projects were assigned to the individual items and some of the other items were scaled based on those costs. The unit costs for excavation and embankment assume the earthmoving will be confined to the project site (no import or export of soil material). The construction cost estimates include pre-construction costs (engineering, surveying permitting) at 15% of the material and labor costs. Due to the very preliminary nature of the schematic design, derivation of quantities, and cost estimating assumptions, a 30% contingency was applied to all cost estimates.

6.4.2 Land Acquisition Costs

The flood inundation area upstream of each dam, corresponding to the top-of-dam embankment elevation, was used to determine the required flowage easement area. The cost estimates assume fee-simple land acquisition for property within and immediately adjacent to the footprint of each dry dam, as well as for an access road to the dam from a nearby public right-of-way for operation and maintenance purposes. Similar to other aspects of this study, land values were derived from the auditor's valuation x 1.5 to get to market value. Fee simple land acquisition used the market value for all or portions of individual parcels impacted by the dry dam embankments, and 30% of the market value for the land area associated with flowage easements.

Table 6-7 is a summary of estimate construction costs for each dam, segmented by construction costs, pre-construction costs, and land acquisition (including flowage easement) costs. The dry dam along the SLFR at Kirkersville has the highest cost, partially due to the larger size of the dam embankment and spillway structures, but also because the land acquisition costs are disproportionally high, due the property impacts of the inundation area upstream of the dam.

OPINION OF PROBABLE CONSTRUCTION COST
SOUTH LICKING WATERSHED CONSERVANCY DISTRICT
FLOOD DAMAGE REDUCTION PLAN
DRY DAM NO. 1 - MUDDY FORK (CLASS I)

Item No.	Description	Quantity	Units	Unit Cost	Item Cost
General Items					
201	Clearing and Grubbing, Including Tree Removal	1	LS	$50,000	$50,000
202	House Demolition	9	EA	$25,000	$225,000
202	Pavement Removed	5,800	SY	$30	$174,000
202	Bridge Removed (Columbia Road)	1	EA	$60,000	$60,000
614	Maintaining Traffic	1	LS	$20,000	$20,000
619	Field Office, Type B	12	MONTH	$4,500	$54,000
623	Construction Layout Stakes and Surveying	1	LS	$30,000	$30,000
624	Mobilization	1	LS	$50,000	$50,000
832	Temporary Erosion and Sediment Control	1	LS	$100,000	$100,000
SPEC	Control of Water	1	LS	$100,000	$100,000
SPEC	Grade Checking/Record Drawing	1	LS	$50,000	$50,000
				Subtotal =	**$913,000**
Dam Items (Not Including Spillways)					
203	Embankment	128,500	CY	$30	$3,855,000
203	Excavation (for Embankment)	128,500	CY	$12	$1,542,000
204	Subgrade Compaction	42,700	SY	$4	$170,800
651	Topsoil Stockpiled, As Per Plan	28,600	CY	$13	$357,500
652	Placing Stockpiled Topsoil on the Dam Embankment, As Per Plan	7,300	CY	$10	$73,000
652	Wasting the Reamining Stockpiled Topsoil on Surrounding Farmland, As Per Plan	21,300	CY	$3	$63,900
659	Seeding and Mulching, Class 3B, As Per Plan	128,100	SY	$3	$384,300
670	Slope Erosion Protection Mat, Type E	64,100	SY	$5	$288,450
SPEC	Access Drive, As Per Plan	1	LS	$300,000	$300,000
SPEC	Settlement Monitoring, As Per Plan	1	LS	$30,000	$30,000
SPEC	Dam Crest Survey Monuments	4	EA	$1,500	$6,000
				Subtotal =	**$7,070,950**
Principal Spillway					
511	Headwalls	2	EA	$70,000	$140,000
611	6 ft. x 4 ft. Box Culvert	200	FT	$2,000	$400,000
SPEC	Debris Rack	1	LS	$30,000	$30,000
SPEC	Rock-lined Scour Hole Energy Dissipator	1	LS	$100,000	$100,000
				Subtotal =	**$670,000**
Weir/Emergency Spillway					
511	2nd Stage Weir	0	FT	$10,000	$0
511	Emergency Spillway	150	FT	$12,000	$1,800,000
				Subtotal =	**$1,800,000**

SUBTOTAL =	**$10,453,950**
ENGINEERING, SURVEY, PERMITTING (15%) =	**$1,568,093**
LAND ACQUISITION =	**$4,270,385**
CONTINGENCY (30% OF SUBTOTAL) =	**$3,136,200**
CONSTRUCTION MANAGEMENT (5%) =	**$679,508**
OPINION OF PROBABLE CONSTRUCTION COST =	**$20,108,135**

FIGURE 6-5
Example of Construction Cost Estimate (Muddy Fork Dry Dam)

TABLE 6-7
Summary of Estimated Construction Costs

Dry Dam	Construction Costs[1]	Pre-Construction Costs[2]	Land Acquisition Costs[3]	Total Costs (In Millions)	Tributary Drainage Area (mi^2)
Muddy Fork	$14,269,658	$1,568,093	$4,270,385	**$20.2**	10.70
SFLR Trib. A	$14,897,982	$1,637,138	$628,973	**$17.2**	5.20
SFLR Trib. B	$4,701,113	$516,608	$784,596	**$6.1**	3.20
Bell Run	$13,804,567	$1,516,981	$724,463	**$16.1**	2.70
Feeder Canal	$21,266,419	$2,336,975	$3,607,399	**$27.3**	5.90
SFLR @ Kirkersville	$66,009,038	$7,253,738	$56,712,702	**$130.0**	47.20
SFLR Headwaters	$11,526,639	$1,266,668	$7,838,261	**$20.7**	7.30
TOTALS =	**$146,475,414**	**$16,096,199**	**$74,566,780**	**$237.6**	**82.20**

[1]Includes 30% Contingency and 5% Construction Management
[2]Engineering, Survey and Permitting
[3]Fee Simple Land Acquisition and Flowage Easements

7.0 STUDY RESULTS

Based on the results of this study, the estimated construction costs of the seven dry dams ($237.6 Million) exceeds the estimated benefits ($51.5 Million). Further optimization to eliminate some of the smaller dams that do not provide significant flood damage reduction benefits will improve this result. Furthermore, the configuration of the dry dam along SFLR at Kirkersville needs to be revisited to determine if similar benefits can be achieved without the significant land acquisition costs. In addition, the benefits of the dry dams outside of the 2D model area can be determined by expanding the hydraulic modeling to extend upstream of that area.

The study process has not achieved the goal of identifying flood damage reduction measures that could be refined and further developed into a revised Watershed Work plan for SLWCD. The use of dry dams for flood protection is common and typically can achieve the target Benefit to Cost Ratio (BCR) of 1.0 or higher, particularly if the dams are simple earthen embankments and land acquisition costs are not disproportionate to estimated construction costs.

The single dry dam identified by the NRCS (2009-2010) coupled with the I-70 by-pass channel appeared to achieve a BCR greater than 1.0, but with concerns regarding the underlying soils. Further studies may revisit the geotechnical evaluation conducted by the NRCS and determine whether the soils condition could be mitigated without increasing project costs to the extent the BCR becomes less than 1.0. Further studies would also have to determine that the by-pass channel would not have adverse impacts further downstream.

ODOT's current study will evaluate measures to protect I-70 from flooding. To prevent roadway floodway up to the 100-year design storm event would require a 4-foot. (+/-) reduction in flood elevations nears the I-70/SR 79 interchange. Given the results of this study, that outcome would not likely be achieved with dry dams in the upstream watershed. The development of ODOT's study should be a factor in further studies for the SLWCD.

The vast floodplain area along SFLR between I-70 and Buckeye Lake presents a unique challenge due to the nature of the watershed in this area, which includes a large upstream watershed area (67 sq. mi.), overflows from the Feeder Canal directly to SFLR (by-passing Buckeye Lake), discharges from the Buckeye Lake spillways, and a very shallow gradient channel with a wide and flat floodplain area. Maintaining the undeveloped farm lands as natural floodplains while focusing flood damage reduction measures on the developed areas north of the lake may be one consideration; however, developing a watershed-scale program for flood protection is the mission of the SLWCD.

APPENDIX A:

CHANNEL MAINTENANCE PLAN


Legend
Large Jams 3rd to 5th Order Streams
Category
Log Jam Very Large
Total Blockage
South Licking Watershed Conservancy District HUC-10 Boundaries
Buckeye Lake
South Fork Licking River
0
1
2
3
4
5
6
8
9
10
11
12
13
14
15
16
17
18
19
20
21
22
24
25
26
27
28
29
30
FAIRFIELD, LICKING, & PERRY COUNTIES, OHIO
SOUTH LICKING WATERSHED
CONSERVANCY DISTRICT
WATERSHED STUDY AREA
Selected Site Visit Locations
Feet
0 1,950 3,900 7,800
N
EMH&T
Engineers • Surveyors • Planners • Scientists
5500 New Albany Road, Columbus, OH 43054
Phone: 614.775.4500 Toll Free: 888.775.3648
emht.com
Date: November 2022
Scale: 1" = 3000'
Job No: 2022-0327
Sheet: 1 of 1
Background Map from 2020 USGS Topographic Map

(A) Accessibility		(B) Constructability		(C) Channel Stability		(D) Length (Parallel to Flow) of Logjam		(F) Height of Log Jam		(G) Density of Log Jam		(H) Severity		(I) Criticality		Total Inspection Score
Criteria Selection	Score	Criteria Selection	Score	Criteria Selection	Score	Criteria Selection	Score	Criteria Selection	Score	Criteria Selection	Score	Criteria Selection	Score	Criteria Selection	Multiplying Factor	
b - Requires work agreement from a single property owner	2.00	c - Requires minimal land disturbance/vegetation clearing	3.00	b - Bank erosion area between 1000 to 3500 square feet	3.00	a - Channel blockage < 1x BKF Widt	1.00	c - Channel blockage > 70% of BKF Depth	5.00	b - Intermediate	3.00	b - Bank erosion - threat anticipated within 2-5 years	5.00	K - Agricultural Field	3.00	66
a - Requires work agreement from multiple property owners	1.00	b - Requires moderate land disturbance/vegetation clearing	2.00	b - Bank erosion area between 1000 to 3500 square feet	3.00	age > 4x BKF Width	5.00	a – Channel Blockage <40% of BKF Depth	1.00	a – Coarse (Water Can/Will Flow Through Log Jam)	1.00	b - Bank erosion - threat anticipated within 2-5 years	5.00	K - Agricultural Field	3.00	54
b - Requires work agreement from a single property owner	2.00	b - Requires moderate land disturbance/vegetation clearing	2.00	b - Bank erosion area between 1000 to 3500 square feet	3.00	age > 4x BKF Width	5.00	c - Channel blockage > 70% of BKF Depth	5.00	b - Intermediate	3.00	c - Bank erosion - threat anticipated within 0 -2 years	10.00	K - Agricultural Field	3.00	90
b - Requires work agreement from a single property owner	2.00	b - Requires moderate land disturbance/vegetation clearing	2.00	c - Bank erosion area > 3500 square feet	5.00	kage between 1x and 4x BKF Width	3.00	a – Channel Blockage <40% of BKF Depth	1.00	a – Coarse (Water Can/Will Flow Through Log Jam)	1.00	e- Blockage - potential future flooding concerns	5.00	c - 1-4 Lane Road	4.00	76
a - Requires work agreement from multiple property owners	1.00	c - Requires minimal land disturbance/vegetation clearing	3.00	a - Bank erosion area < 1000 square feet	1.00	age < 1x BKF Width	1.00	a – Channel Blockage <40% of BKF Depth	1.00	a – Coarse (Water Can/Will Flow Through Log Jam)	1.00	a - Bank erosion - no threat anticipated	1.00	a – Railroad	5.00	45
b - Requires work agreement from a single property owner	2.00	c - Requires minimal land disturbance/vegetation clearing	3.00	b - Bank erosion area between 1000 to 3500 square feet	3.00	age < 1x BKF Width	1.00	a – Channel Blockage <40% of BKF Depth	1.00	a – Coarse (Water Can/Will Flow Through Log Jam)	1.00	b - Bank erosion - threat anticipated within 2-5 years	5.00	K - Agricultural Field	3.00	48
b - Requires work agreement from a single property owner	2.00	a - Requires extensive land disturbance/vegetation clearing	1.00	a - Bank erosion area < 1000 square feet	1.00	age < 1x BKF Width	1.00	a – Channel Blockage <40% of BKF Depth	1.00	a – Coarse (Water Can/Will Flow Through Log Jam)	1.00	a - Bank erosion - no threat anticipated	1.00	i - Open Space	1.00	8
b - Requires work agreement from a single property owner	2.00	c - Requires minimal land disturbance/vegetation clearing	3.00	a - Bank erosion area < 1000 square feet	1.00	age < 1x BKF Width	1.00	b - Channel Blockage 40% to 70% of BKF Depth	3.00	b - Intermediate	3.00	e- Blockage - potential future flooding concerns	5.00	c - 1-4 Lane Road	4.00	72
b - Requires work agreement from a single property owner	2.00	b - Requires moderate land disturbance/vegetation clearing	2.00	a - Bank erosion area < 1000 square feet	1.00	kage between 1x and 4x BKF Width	3.00	c - Channel blockage > 70% of BKF Depth	5.00	c - Fine (No or Minimal Flow Through Log Jam)	5.00	b - Bank erosion - threat anticipated within 2-5 years	5.00	K - Agricultural Field	3.00	69
b - Requires work agreement from a single property owner	2.00	b - Requires moderate land disturbance/vegetation clearing	2.00	a - Bank erosion area < 1000 square feet	1.00	age < 1x BKF Width	1.00	a – Channel Blockage <40% of BKF Depth	1.00	a – Coarse (Water Can/Will Flow Through Log Jam)	1.00	a - Bank erosion - no threat anticipated	1.00	K - Agricultural Field	3.00	27
b - Requires work agreement from a single property owner	2.00	b - Requires moderate land disturbance/vegetation clearing	2.00	b - Bank erosion area between 1000 to 3500 square feet	3.00	kage between 1x and 4x BKF Width	3.00	c - Channel blockage > 70% of BKF Depth	5.00	c - Fine (No or Minimal Flow Through Log Jam)	5.00	b - Bank erosion - threat anticipated within 2-5 years	5.00	K - Agricultural Field	3.00	75
b - Requires work agreement from a single property owner	2.00	b - Requires moderate land disturbance/vegetation clearing	2.00	b - Bank erosion area between 1000 to 3500 square feet	3.00	kage between 1x and 4x BKF Width	3.00	b - Channel Blockage 40% to 70% of BKF Depth	3.00	a – Coarse (Water Can/Will Flow Through Log Jam)	1.00	a - Bank erosion - no threat anticipated	1.00	i - Open Space	1.00	15
a - Requires work agreement from multiple property owners	1.00	b - Requires moderate land disturbance/vegetation clearing	2.00	a - Bank erosion area < 1000 square feet	1.00	age < 1x BKF Width	1.00	b - Channel Blockage 40% to 70% of BKF Depth	3.00	b - Intermediate	3.00	a - Bank erosion - no threat anticipated	1.00	i - Open Space	1.00	12
b - Requires work agreement from a single property owner	2.00	a - Requires extensive land disturbance/vegetation clearing	1.00	a - Bank erosion area < 1000 square feet	1.00	age < 1x BKF Width	1.00	a – Channel Blockage <40% of BKF Depth	1.00	a – Coarse (Water Can/Will Flow Through Log Jam)	1.00	a - Bank erosion - no threat anticipated	1.00	i - Open Space	1.00	8
a - Requires work agreement from multiple property owners	1.00	a - Requires extensive land disturbance/vegetation clearing	1.00	a - Bank erosion area < 1000 square feet	1.00	age < 1x BKF Width	1.00	a – Channel Blockage <40% of BKF Depth	1.00	b - Intermediate	3.00	a - Bank erosion - no threat anticipated	1.00	i - Open Space	1.00	9
b - Requires work agreement from a single property owner	2.00	b - Requires moderate land disturbance/vegetation clearing	2.00	c - Bank erosion area > 3500 square feet	5.00	kage between 1x and 4x BKF Width	3.00	c - Channel blockage > 70% of BKF Depth	5.00	a – Coarse (Water Can/Will Flow Through Log Jam)	1.00	b - Bank erosion - threat anticipated within 2-5 years	5.00	K - Agricultural Field	3.00	69
b - Requires work agreement from a single property owner	2.00	a - Requires extensive land disturbance/vegetation clearing	1.00	b - Bank erosion area between 1000 to 3500 square feet	3.00	kage between 1x and 4x BKF Width	3.00	b - Channel Blockage 40% to 70% of BKF Depth	3.00	a – Coarse (Water Can/Will Flow Through Log Jam)	1.00	b - Bank erosion - threat anticipated within 2-5 years	5.00	i - Open Space	1.00	18
b - Requires work agreement from a single property owner	2.00	b - Requires moderate land disturbance/vegetation clearing	2.00	a - Bank erosion area < 1000 square feet	1.00	age < 1x BKF Width	1.00	b - Channel Blockage 40% to 70% of BKF Depth	3.00	c - Fine (No or Minimal Flow Through Log Jam)	5.00	b - Bank erosion - threat anticipated within 2-5 years	5.00	K - Agricultural Field	3.00	57
a - Requires work agreement from multiple property owners	1.00	b - Requires moderate land disturbance/vegetation clearing	2.00	b - Bank erosion area between 1000 to 3500 square feet	3.00	age < 1x BKF Width	1.00	a – Channel Blockage <40% of BKF Depth	1.00	b - Intermediate	3.00	b - Bank erosion - threat anticipated within 2-5 years	5.00	K - Agricultural Field	3.00	48
a - Requires work agreement from multiple property owners	1.00	a - Requires extensive land disturbance/vegetation clearing	1.00	b - Bank erosion area between 1000 to 3500 square feet	3.00	age < 1x BKF Width	1.00	a – Channel Blockage <40% of BKF Depth	1.00	a – Coarse (Water Can/Will Flow Through Log Jam)	1.00	a - Bank erosion - no threat anticipated	1.00	b- Highway	5.00	45
b - Requires work agreement from a single property owner	2.00	b - Requires moderate land disturbance/vegetation clearing	2.00	c - Bank erosion area > 3500 square feet	5.00	kage between 1x and 4x BKF Width	3.00	c - Channel blockage > 70% of BKF Depth	5.00	c - Fine (No or Minimal Flow Through Log Jam)	5.00	b - Bank erosion - threat anticipated within 2-5 years	5.00	K - Agricultural Field	3.00	81
a - Requires work agreement from multiple property owners	1.00	a - Requires extensive land disturbance/vegetation clearing	1.00	b - Bank erosion area between 1000 to 3500 square feet	3.00	age < 1x BKF Width	1.00	c - Channel blockage > 70% of BKF Depth	5.00	c - Fine (No or Minimal Flow Through Log Jam)	5.00	b - Bank erosion - threat anticipated within 2-5 years	5.00	K - Agricultural Field	3.00	63
a - Requires work agreement from multiple property owners	1.00	a - Requires extensive land disturbance/vegetation clearing	1.00	c - Bank erosion area > 3500 square feet	5.00	kage between 1x and 4x BKF Width	3.00	c - Channel blockage > 70% of BKF Depth	5.00	c - Fine (No or Minimal Flow Through Log Jam)	5.00	b - Bank erosion - threat anticipated within 2-5 years	5.00	K - Agricultural Field	3.00	75
b - Requires work agreement from a single property owner	2.00	b - Requires moderate land disturbance/vegetation clearing	2.00	b - Bank erosion area between 1000 to 3500 square feet	3.00	age < 1x BKF Width	1.00	c - Channel blockage > 70% of BKF Depth	5.00	c - Fine (No or Minimal Flow Through Log Jam)	5.00	b - Bank erosion - threat anticipated within 2-5 years	5.00	c - 1-4 Lane Road	4.00	92

APPENDIX B:

HYDROLOGIC STUDIES

Path: J:\20220327\GIS\exhibit\All_subcatchments.mxd

APPENDIX C:

EVALUATION OF ALTERNATIVES

South Fork of the Licking River Geologic Report

The South fork of the Licking River is located in Ohio about 40 miles east of Columbus. The watershed is located in the Ohio River drainage area of the state within the Muskingum River Basin. The 11 digit code for the watershed is 05040006040.

The area is on the extreme eastern edge of major Land Resource Area 111, the Indiana Ohio Till Plain and is just a few miles from the furthest extent east and of the Scioto Lobe of the Wisconsin age glaciation. The geology in the area of the proposed flood water retention structure and flood by-pass channel is very complex with subglacial diamicton, glaciofluvial, glaciolacustrine, resedimented diamicton and recent alluvial deposits all occurring in close proximity to one another. The deposited materials include mixtures of gravel sand silt and clay. Unified Soil Classifications range across the entire spectrum form sand and gravel through all silt and clay classifications.

Geologic Exploration

A multi phase geologic exploration was performed in the area. The geologic exploration involved a reconnaissance investigation and detailed drilling investigation of the proposed dam and channel sites. During the drilling investigation, samples were collected that represented all geologic materials encountered. The samples were sent to the NRCS Soil Mechanics Center in Lincoln Nebraska.

Large amounts of sand and gravel that contained ground water under artesian pressures were encountered during the drilling investigation. As a result, a groundwater investigation that involved installing of 20 observation wells and three wells that could be used for aquifer pump tests was conducted.

Reconnaissance investigation and literature search:

The area of the proposed dam and flood by-pass channel along with much of the rest of the watershed were walked by the NRCS geologist.

The most striking thing revealed during the reconnaissance investigation was the vast differences in stability along the different drainage channels in the area. The stability of the stream banks along Pigeon Swamp Ditch, the Licking River, Bell Run and Kuhn Ditch was evaluated. The following pictures show the stream bank processes occurring in the area from west to east.

Pigeon Swamp Ditch

Pigeon Swamp Ditch is a completely artificial drainage ditch system that was installed to drain the Pigeon Swamp, located in the western half of the planned flood pool that will be created by the proposed dry dam. Historically this entire area was referred to as the Blood Swamp. The Pigeon Swamp Ditch banks are experiencing sapping stream bank failure and are unstable throughout the entire length of the system. Pigeon Swamp Ditch and its tributaries contain 20,650 feet of ditches that provide drainage for about 60% of the 100 year flood pool area. These ditches are almost perfectly straight and obviously manmade

Sapping failure is caused by a weaker material that is easily eroded underlying a strong material. In this area the weaker material is saturated sand and gravel that is subject to internal erosion as well as the erosive force of flowing water. The sand is eroded in part by ground water flowing into the channel and the river water carrying it away. The removal of the sand undermines the stronger clay above the sand and the clay falls into the stream in blocks. A channel that is subject to sapping failure characteristically has a U shape as a result of the vertical banks that are formed during the process. There is also evidence of large blocks of soil breaking loose from the banks with the tops of the blocks leaning toward the stream. This is a typical occurrence during sapping stream bank failure.

The failure along the banks of Pigeon Swamp Ditch and its tributaries is shown in Figures 1 and 2. Failure of these banks is slow in nature and does not constitute a major problem along the streams today. However, over time Pigeon Swamp Ditch and its tributaries are widening as a result of the progressive sapping failure of the banks. In the 1950s Pigeon Run Ditch was improved. The engineering drawings show a top width of 40 feet. Today, the top width in many places is 60 feet and more. Over time, Pigeon Swamp Ditch will continue to widen at an average annual rate of about 3 feet per year.

Figure 1: Banks of Pigeon Swamp Ditch Located within the proposed flood pool.

Figure 2 Looking west up a tributary of Pigeon Swamp Ditch. Bank failure caused by sapping occurs along the entire reach.

South Fork of the Licking River

The South fork of the Licking River drains 62 square miles above the proposed dam. The length of the river within the proposed 100 year flood pool is about 7,750 feet. The river meanders throughout that entire distance.

The Licking River banks are unstable and are subject to the same kind of failure as Pigeon Swamp Ditch. The river does not appear to be down cutting but it is experiencing considerable erosion on its high banks. A review of historical areal photography in ArcGIS indicates the river is widening in a number of reaches. This appears to be a dynamic situation where the locations of the meanders are advancing downstream as well as becoming wider. The affect of this erosion has caused the Licking River Channel to widen over the years. Nine reaches between US route 40 and the proposed structure were identified as actively eroding over a 12 year period between 1994 and 2006. A total of 1.84 acres of land was lost during that time period. This represents about .15 acres per year.

It is important to note, the river can change its course very dramatically over a very short period of time. In a number of the reaches had crescent shaped gouges cut into the banks. The location of the river bank moved between 40 and 60 feet into the surrounding fields during the 12 year period. This averages between three and five feet per year.

Figures 3 through 6 show areas where the Licking River is very actively eroding its banks in the proposed structure flood pool.

Figure 3 Licking River bank failure in the flood pool

Figure 4 Licking River in the flood pool

Figure5 Bank failure along the Licking River in the flood pool. Blocks breaking Away from the banks with their tops tilted toward the river. In this case the river took about 2 feet of bank or more over the length of the meander in one event.

Figure 6. Dramatic bank failure along the Licking River in the proposed flood pool.

Eight meanders were identified on the Licking River between the proposed dry dam and US Route 40. These meanders are just starting to form.

Bell Run

Bell run is a stream located about 3000 feet east of the Licking River and flows in a north to south direction parallel to the river. The steam meanders considerably through the area within the proposed flood pool. The banks of Bell run appear to be stable and the channel is a more typical trapezoidal shape. Bell Run is not experiencing the same kind of erosion as Pigeon Swamp Ditch and the Licking River.

Figure7. Bell Run near location where it will be crossed by the proposed dam and channel.

Figure 8 stable banks of bell Run

Kuhn Ditch

Kuhn Ditch is farther east of the river and is outside of the proposed dam. The ditch has a drainage area of about a thousand acres and crosses the proposed flood by-pass channel about 2 miles east of the proposed dam. Kuhn Ditch has very stable banks and a typical trapezoidal shaped channel.

Figure 9. Stable banks along Kuhn ditch.

South Fork of the Licking River By-pass Channel Outlet

The proposed flood by-pass channel will outlet back into the Licking River about 3 miles east of its origin. The river banks close to the outlet seem to be experiencing the same kind of failure at this point as they are upstream in the proposed flood pool. The banks on the west side of the river are failing and sliding down into the river.

Figure 10. The Licking River near the outlet of the flood by-pass channel.

Beaver Creek

Beaver Creek is a tributary of the Licking River that is immediately north of the planned project measures. Beaver Creek has no direct influence on the conditions in the area of the project but it does have a geologic connection.

Beaver Creek is about 2.5 miles north of the proposed dry-dam and about 0.5 miles from Bell Run at its nearest point. The distance from the low drainage divide between Bell Run and Beaver Creek to the confluence of Beaver Creek and the Licking River is about 3.5 miles due east down Beaver Creek. The distance from the same drainage divide down Bell Run and the Licking River to the confluence of the Licking River and Beaver Creek is over 10 miles. This gives Beaver Creek a tremendous hydrologic advantage over Belle Run and the Licking River at this location.

Beaver Creek, Bell Run, and the Licking River in this area are on the same physiographic land unit with the same geologic materials and even mapped on the same continuous alluvial soil map unit.

Soils and Geology References

The Licking County Soil Survey was completed in the 1980s. The soils mapped in the area reflect a very complex geologic history. The soils include those developed in clay glacial till, sand and gravel glacial outwash, and lacustrine sediments. Also included are soils that formed in more recent alluvium. Robert Parkinson, NRCS soil scientist, served as the Licking County Soil Survey party leader. He was consulted and he supplied much insight into the soils and geology in the area.

The area in which the proposed dry dam and flood bypass channel are located is a very large flat lake plain that is susceptible to frequent flooding. The lake plain is bordered on the north and west by soils that developed in glacial outwash sand and gravel. The sand and gravel deposits occur at elevations above 900 feet along the north and west sides of the ancient lake bed. There are also deeper sand and gravel deposits (below 890 feet of elevation) that are very thick.

Other areas along the down stream reaches of the proposed flood bypass channel have poorly drained glacial outwash soils. Most soils in Ohio that develop in sand and gravel are well drained because water infiltrates the sand and gravel quickly and is carried away. Poorly drained outwash soils can only occur where water is held in the soil as a result of other factors. The reason for this high water table in the area is not known for sure. Parkinson believed this was due to a shallow glacial till below the one meter control depth of the soil that did not allow rain water infiltration. This is probably the case in some of the areas but regardless of the cause, these soils and the underlying sand and gravel are saturated continually for lengthy portions of the years.

During the reconnaissance exploration a local farmer was consulted concerning the kinds of soils in the lower one third of the by-pass channel. He stated that in many places he cannot dig a hole with his back hoe below the clay. The saturated sand and gravel simply flows into the area evacuated much the same as water would. (J. Slater, pers. comm.)

The central part of the proposed flood bypass channel contains areas of soils that developed in glacial lacustrine clays with glacial till below.

Drainage system analysis in the watershed

The Glacial Geology map of Licking County (Ohio Division of Geological Survey Report of Investigations No. 59) was consulted as well as the Groundwater Resource Maps of Licking and Fairfield County.

These maps reflect information observed at different depths with the soil map being the shallowest and the groundwater resources map being the deepest. When the information from all levels is evaluated, the original consequent drainage system in the area can be described based on the materials that were deposited by the system. When the consequent drainage system is compared to the present drainage system, the processes that are causing the drainage system to evolve the way it is can be understood.

If the assumption is made that the natural occurring process will continue, predictions of further stream channel changes can be made.

All of the listed references were evaluated to understand the geologic and stream conditions in the area.

Proposed reservoir soil conditions

Within the area of the proposed reservoir, there are lacustrine silts and clays that were deposited in a low energy environment. After the last glacial retreat in this part of Ohio, this area was a large lake for many years. As the water coming from the melting glaciers to the north and west entered this flat area a lake was formed. The coarse material dropped out first as delta and beach deposits. These materials form sand and gravel hills that range in steepness from 2 to 12 percent. The soils mapped on the sand and gravel consist of the Ockley and Fox soil series. The sand and gravel in these old beaches represent the upper deposits of non cohesive soils. It is in this sand and gravel soils that the abutments of the proposed dam are located.

The coarser material was deposited quickly in the form of a delta or beach and the clays and silts were carried out into the lake where they gradually settled to the bottom. The clay and silt formed the broad flat plain along what is now the Licking River. After the water receded the area was a swamp for many years and a thick dark layer of clay soil high in organic material was formed. This dark soil has recently been covered by a layer of silty alluvium.

The profile of the lacustrine clay and the silt alluvium can be seen along the Licking River where it flows through the proposed flood pool.
Figure 11 shows the exposed banks of the river.

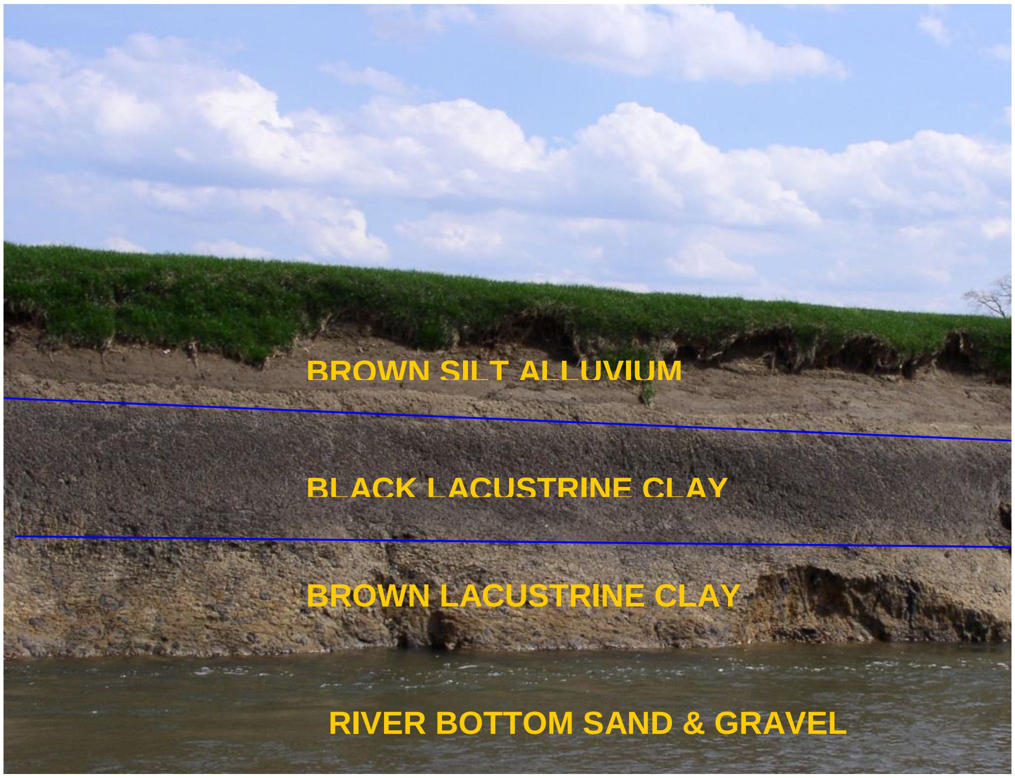
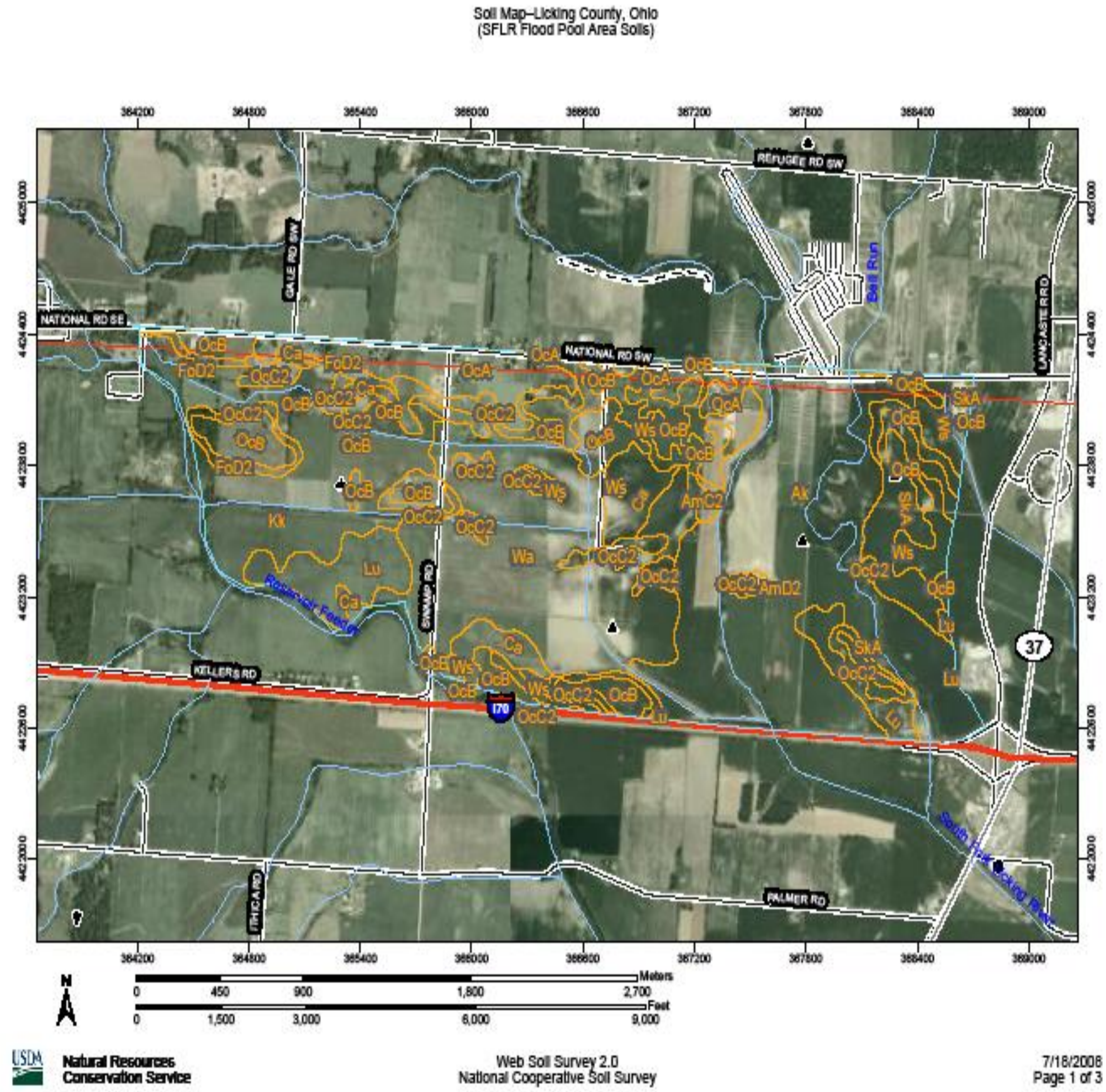


Figure 11 A broad flat lake plain along the Licking River was recently covered with silt alluvium.

Figure 12 is a soil map of the area in the flood pool of the dry dam. The soils on the north and west sides developed in sand and gravel outwash that was deposited as a beach or delta. The flatter soils are to the east surrounding the Licking River. These are the lacustrine deposits covered by silt. Except for sand and gravel ridges the area in the western basin is all poorly drained and could not be used for cropland until the Licking River and Pigeon Swamp Ditch were channelized.

There are 330 acres of drained muck soils on the western basin. These occupy the lowest elevations in the area. Some of the muck is now covered with alluvium and other parts are not.

Swamp road, in the area, was appropriately named before the area was drained and cultivated. The drained muck soils are planted to row crops annually. Some years the crop is lost due to flooding.



Legend For Soils Map of The Flood Pool Area

Map Unit Symbol	Map Unit Name	Acres in AOI	Percent of AOI
Ak	Algiers silt loam, frequently flooded	409.2	26.5%
AmC2	Amanda silt loam, 6 to 12 percent slopes, eroded	4.7	0.3%
AmD2	Amanda silt loam, 12 to 18 percent slopes, eroded	2.8	0.2%
Ca	Carlisle muck	52.8	3.4%
FoD2	Fox gravelly loam, 12 to 18 percent slopes, eroded	20.9	1.4%
Kk	Killbuck silt loam, frequently flooded	202.1	13.1%
Lu	Luray silty clay loam	76.2	4.9%

Legend For Soils Map of The Flood Pool Area

Map Unit Symbol	Map Unit Name	Acres in AOI	Percent of AOI
OcA	Ockley silt loam, 0 to 2 percent slopes	112.2	7.3%
OcB	Ockley silt loam, 2 to 6 percent slopes	148.3	9.6%
OcC2	Ockley silt loam, 6 to 12 percent slopes, eroded	97.8	6.3%
SkA	Sleeth silt loam, 0 to 2 percent slopes	45.5	2.9%
Wa	Wallkill silt loam, clayey substratum, frequently flooded	285.8	18.5%
Ws	Westland silty clay loam	83.3	5.4%
Totals for Area of Interest		1,541.6	100.0%

Geologic Subsurface Exploration

Procedures

The South Licking Conservancy Board hired the HC Nutting Corporation to perform drilling operations in order to evaluate the foundation conditions of the dam and bypass channel as well as the flood way on the lower end of the project in April of 1997. A total of 48 boreholes were drilled and logged in the project area. A number of hand auger holes were also made during the course of the investigation. Boreholes were drilled at 500 foot intervals the entire length of the dam and the channels. All boreholes were located with the use of GPS equipment. The drilling plan was developed using recommendations left by Keith Rowe State Construction Engineer (retired).

The drill rig used was mounted on an ATV in order to limit soil damage to the cropland in the area. A two man crew did all of the drilling and the NRCS geologist logged the holes. Continuous split barrel samples were taken in every borehole. All blow counts were recorded. The location of every borehole and a log that shows the USCE classification and the blow counts is included. The drilling operations were performed during the first two weeks of April 2007. The weather conditions were very cold and windy during the course of the drilling portion of exploration.

Standard penetration tests were performed continuously in every borehole. The blow counts were recorded by the drill rig operators. The split barrel samples were saved in jars. The NRCS geologist logged the materials encountered in all holes. A water depth reading was taken at each borehole after the augers were pulled. A water measurement was taken by the drilling company 24 hours later and the holes were plugged.

Many jars of split barrel samples were taken a few of them were kept in Columbus for future reference and most of them were sent to the NRCS Soil Mechanics Center in Lincoln NE. Samples of disturbed material were put in five gallon buckets and sent to Lincoln for borrow compaction analysis and other engineering properties. Most of the bucket samples were collected by hand after the drilling was finished. These samples included materials that represent the glacial till and the lacustrine clays that may be used as borrow for the dam construction.

Samples that represent most of the saturated sand and gravel encountered were collected by hand at the B401 and B427 locations. Undisturbed samples were also collected in five inch shelby tubes. The core samples were usually limited to the lacustrine clays. One core was sample was collected in glacial till. Most of the material was too hard to push the tube and the sand and gravel could not be sampled in this manner.

Below is the sample inventory shipped to the Soil Mechanics Center

South Fork of the Licking River Soil Sample inventory. Shipped to Lincoln Soil Mechanics Center 5/14/2007

Total shipment consists of 12 containers:

- Three 55 gallon fiber Drums
- Three Large computer Boxes
- Six Green 5 gallon buckets
- Also included in the shipment is one blue plastic 20 gallon drum that contains

Contents of the containers:

CONTAINER NUMBER	CONTAINER TYPE	CONTAINER CONTENTS
1	Fiber Drum	4 Shelby Tubes B401-1 3'-4.5', B401-2 6'-7.5', B011 4'-5.5', B42 3'-4.5'
2	Fiber Drum	4 Shelby Tubes B010 4.5'-6', B418 3'-4.5', B416 3'-4.5', B
3	Fiber Drum	2 Shelby Tubes B408-1 3-4.5 B408-2 7-7.5 6 Boxes of Split Barrel Samples for the following boreholes: B407, B414, B008, B022, B024, B433, B434, B417, B426
4	Large Box	7 Boxes of split barrel samples B453, B450, B408, B022, B452, B453, B440, B441, B418, B419, B413, B414, B422, B423, B434
5	Large Box	6 Boxes of split barrel samples B402, B403, B400, B410, B430, B431, B406, B016
6	Large Box	6 Boxes of split barrel samples B428, B429, B415, B417, B420, B421, B422, B412, B416, B420
7	5 Gallon Bucket	B401 10' to 12' Loose sand and gravel
8	5 Gallon Bucket	B401 5' to 8' proctor sample
9	5 Gallon Bucket	B401 3' to 5' proctor sample
10	5 Gallon	B427 loose sand and gravel

	Bucket	
11	5 Gallon Bucket	B 409 10’ to 15’ Proctor
12	5 Gallon Bucket	B 405 3’ to 6 ‘ Proctor

Thirty-four boreholes were drilled along the centerline of the proposed 3.3 mile channel. The depths of these holes ranged from 10 to 20 feet. Thirteen holes were bored in the center line of the proposed dam and three were bored in the proposed SFLR overflow channel. The boreholes were spaced at 500 foot intervals.

Facts and Findings

The drilling revealed a very complex array of materials that range from non-cohesive sand and gravel to very smooth high plasticity fat clays of lacustrine origin. The materials were deposited in a wide range of environments associated with continental glaciation. The energies of deposition of these deposits ranged from subglacial to resedimented super glacial to swift flowing water beaches in the glaciofluvial environment to lacustrine clays in the glaciolacustrine setting. The wide range of materials encountered would indicate a very complex geologic history. At times the materials seemed to change drastically in the same boreholes and very different materials would occur side by side.

Large quantities of ground water were also discovered. All sand and gravel that occurs below the elevation of 885 ft contained free water under artesian pressures. In many boreholes the water rose very quickly to as much as 5 feet above the top of the sand and gravel. The drill operator had problems with sands heaving up into the hollow stem augers in places.

The sands and gravels encountered almost always had between 20% and 25% fine materials that passed the 200 sieve present. Some uniform beach sand was encountered in thin layers. In the lower one third of the channel, there were variable layers of very clean uniform sand. A visual depiction of the logs as well as the original logs taken is included in this report.

The samples were analyzed and tested at the NRCS Soil Mechanics Center in Lincoln, Nebraska. The report of their findings is included.

Some of their findings are noted here.

- All materials encountered had moisture contents within 2% of their saturation point.

- Existing natural moisture contents of the till and lacustrine samples are always higher and can be twice as much as the optimum moisture content in the standard Proctor test.
- Almost all of the saturated sand and gravel encountered contains about 20% fines (passing the 200 sieve) with as much as 9 percent smaller than two microns.
- Generally, the materials tended to be finer in lab results than they appeared to the field geologist. SP in the field was SM in the lab. Some ML became CL and some CL became CH.
- The lacustrine clays have optimum moisture in the twenties, saturation in the thirties and liquid limits in the liquid limits in the fifties and sixties of percent.
- The glacial till had optimum moisture contents 30 to 50 percent higher than the glacial till and maximum dry densities about 10 percent lower.

Geology along the proposed the proposed flood by-pass channel:

The subsurface exploration revealed a very complex array of materials through which the flood by-pass channel will need to be constructed. As one moves downstream along the channel from the proposed dam the soils change considerably. For the sake of description the channel can be divided into six distinct reaches. These reaches are described below. They are listed by their stationing locations. Each reach is given a descriptive name.

1. 100+00 to 118+00 – 1,800 feet: Upper Lake Plain This reach is fairly consistent over its total length. It consists of a surface layer of about two feet of silt alluvium over about eight feet of fat and lean lacustrine clay. A layer of sand and gravel is under the clay. This layer has free water under artesian pressures. Water levels are higher farther from the river. The water rose instantly about 4 to 5 feet above the sand and and gravel when the augers were removed from the borehole and another foot within the next 24 hours.
2. 118+00 to 130+00 – 1,200 feet: Glacial till plug This reach consists of lean clay firm dense glacial till. The till is fairly uniform. No other materials were found in this reach. The material is firm and stable.
3. 130+00 to 140+00 – 1,000feet: Shallow sand lake plain - This reach consists of sand and gravel at shallow depths covered by lacustrine and alluvial clays and silts. The unstable sand and gravel extends eight feet up the bottom of the proposed low water channel. The water at the time of drilling came to the top of the sand. This is about the same elevation as the water in reach 1. The sand and gravel in Reaches 1 and 3 are directly connected just south of the proposed channel.
4. 140+00 to 178+00 – 3,800 feet: Upper Mixed Glaciofluvial Sediments This reach has diverse conditions. Most borings have some sand and gravel that is sometimes mixed with other materials. Ground water pressures among the highest encountered in the study. Most of the borings were underlain by dense hard glacial till.
5. 178+00 to 198+00 – 2,000 feet: Lower Lake Plain This is a reach with lithologies similar to reach one. Sand and gravel are encountered at depth overlain with

lacustrine clays. The water in the sand and gravel consistently rose to 3 to 4 feet above the sands and gravel in the boreholes upon pulling the augers.

6. 198+00 to 278+00 – 7,800 feet: Lower mixed sediments This is a very long reach of highly variable conditions. Sand and gravel, lacustrine, glacial till and alluvial materials were encountered at various depths in every hole. A good correlation of the stratigraphic units would be require borehole spacing of less than 100 feet and even then one could not know what would be encountered between the boreholes. However, we can say that when glacial till is encountered its properties are very similar to the glacial till in reach 2. When sand is encountered, it will always be saturated. As one approaches the outlet of the channel at the Licking River the alluvium becomes thicker. Almost all of this reach will have very unstable conditions.

Geology along the center line of the proposed dry-dam

1. 20+00 – 68+00 Alluvium, outwash, glacial till mixed This reach starts at US Rt 40 and continues in a southerly direction for about 3,300 feet then turns east for about 1,200 feet then turns southwest for about 300 feet. The height of the dam through this reach is six feet or less. The underlying geology is a complex mixture of alluvial sand, silt and clay that is underlain by glacial till at a depth of eight to ten feet. The bearing strength of the till is very good but the sand layers are a different story. Every boring between 54+00 and 68+00 had a two foot thick layer of soft sand. This between 6 and 8 feet in depth. This sand has free ground water and is a liquefaction hazard.
2. 68+00 – 76+00 Shallow sand lake plain - This reach consists of sand and gravel at shallow depths covered by lacustrine and alluvial clays and silts. The top six feet is silt and clay soft alluvium. Under the alluvium is a layer of silt and clay lacustrine deposits about one to two feet in thickness. The soft sand deposits occur at a depth of six or seven feet.with lean clay glacial till at about ten feet. The ground water came to the top of the sand when the augers were removed and rose another three feet after 24 hours. The sand and gravel in this reach is directly connected to the sands of the reach closer to the Licking River just south of the proposed bypass channel. The ground water elevations directly correspond to with groundwater elevations as one approaches the Licking River.
3. 76+00 – 86+00 – Glacial till This reach is dominated by very firm and stable glacial till.
4. 86+00 – 125+00 Upper Lake Plain This reach is fairly consistent over its total length. It consists of a surface layer of about two feet of silt alluvium over about eight feet of fat and lean lacustrine clay and lacustrine silt. A layer of sand and gravel is under the clay. The sand and gravel contains free water under artesian pressures. Water levels are higher farther from the river. The water rose instantly about 4 to 5 feet above the sand and gravel when the augers were removed from the borehole and another foot within the next 24 hours. The dam throughout this reach has rerouted Ball Run and Pigeon Swamp ditch on its front side and the floodwater by-pass channel on its backside.

5. 125+00 – 135+00 the right abutment. This is an area of glacial outwash soils. These soils consist of a layer of lean clay down to about four feet in depth with saturated soft loose sand below. The sand is highly variable and contains large amounts of ground water.

Geology along the lower floodway

The soils encountered along the lower floodway consist of lean clay glacial till and lean clay glacial outwash. Where the floodway crosses the river, silty alluvium is encountered. Most of the soils along its length are fairly uniform and stable. Loose sand and gravel was only encountered in the lowest 500 feet of the channel.

GROUNDWATER

The subsurface exploration revealed large quantities of groundwater. About 50% of the proposed channel course will intersect sand and gravel. Over much of this distance the coarse material will have free water under artesian pressures present. The discovery of so much water under pressure made a ground water study necessary.

The USGS did a study previously to asses any affects the proposed flood pool might have on wells in northwest area of the pool (Open-file Report 2007-1211 GROUND-WATER DATA and FLOW DIRECTIONS IN THE VICINITY OF SWAMP ROAD LICKING COUNTY OHIO, 2006-07). A copy of the report is included. The study monitored ground water levels, and compared them with the hydrograph of the Licking River. The following conclusions were drawn.

- The water levels in the sand and gravel aquifer in the area rise and fall with the river indicating a hydraulic connection between the river and the aquifer.
- Ground water flow is to the southeast.
- Ground water levels are generally higher then surface water elevations including the new 100 year flood levels that will be created by the construction of the proposed dam.

When sand and gravel was encountered below the elevation of 890 ft above sea level during the drilling operations, it almost always contained groundwater under artesian pressures. The sand is covered by a layer of clay that acts as an aquatard and confines the groundwater to the sand and gravel layer. This groundwater makes the sands very unstable. It causes the sand to heave into the well casing in places. The boreholes into the sand and gravel caved in as soon as the augers were pulled by the drill rig operator. It was estimated that a ground water problem would affect about 50% of the channel course.

Groundwater is a significant problem in the watershed. It was determined that three areas along the proposed channel should be investigated by performing aquifer pump tests in order to determine the feasibility of constructing a stable channel through the area. The pump test areas were named area 1, 2, and 3. Area 1 is in the vicinity of borehole B401. Area 2 is near B409 and area 3 is at the location of B430.

Three six inch diameter pumping wells were installed to perform aquifer pump tests. A field of piezometers was installed with each of the six inch wells to measure the response of the pumping in the aquifer. These observation wells were installed at 30, 100, and 400 feet distances from the pumping wells.

Aquifer pump tests were performed in ground water areas 1 and 3. The pumping well in area 3 fully penetrated the confined aquifer The aquifer was too thick to penetrate with a shallow well.

The pumping of area 3 was completed first. Assistance was provided by Danny Goodwin and Phil Hayes. Phil Hays performed the mathematical analysis of the data generated by these tests. The area test started with temperatures around zero degrees F. and ended with 48 hours of thawing conditions and rain.

The area 1 pump test was completed in June of 2008. During that test the area was subjected to a four inch rain storm. The ensuing flood completely submerged and destroyed all equipment except the well data loggers. Much was learned from this test.

The area 2 well was never tested. The ground water bearing materials are very shallow at the site and it was determined that this was a minor problem compared to the other two areas.

The ground water study also involved the installing of about 20 observation wells. The water levels in these wells are recorded monthly with the use of an electric water level tape. Piezeometric surface maps were generated in order to predict the depth of ground water during channel construction. Figure is a photograph of the installed Piezometer 1 with the electric tape used for monitoring water levels. Figure is a map of the piezometer locations.

Figure A typical piezometer installation with electric water level measuring tape.

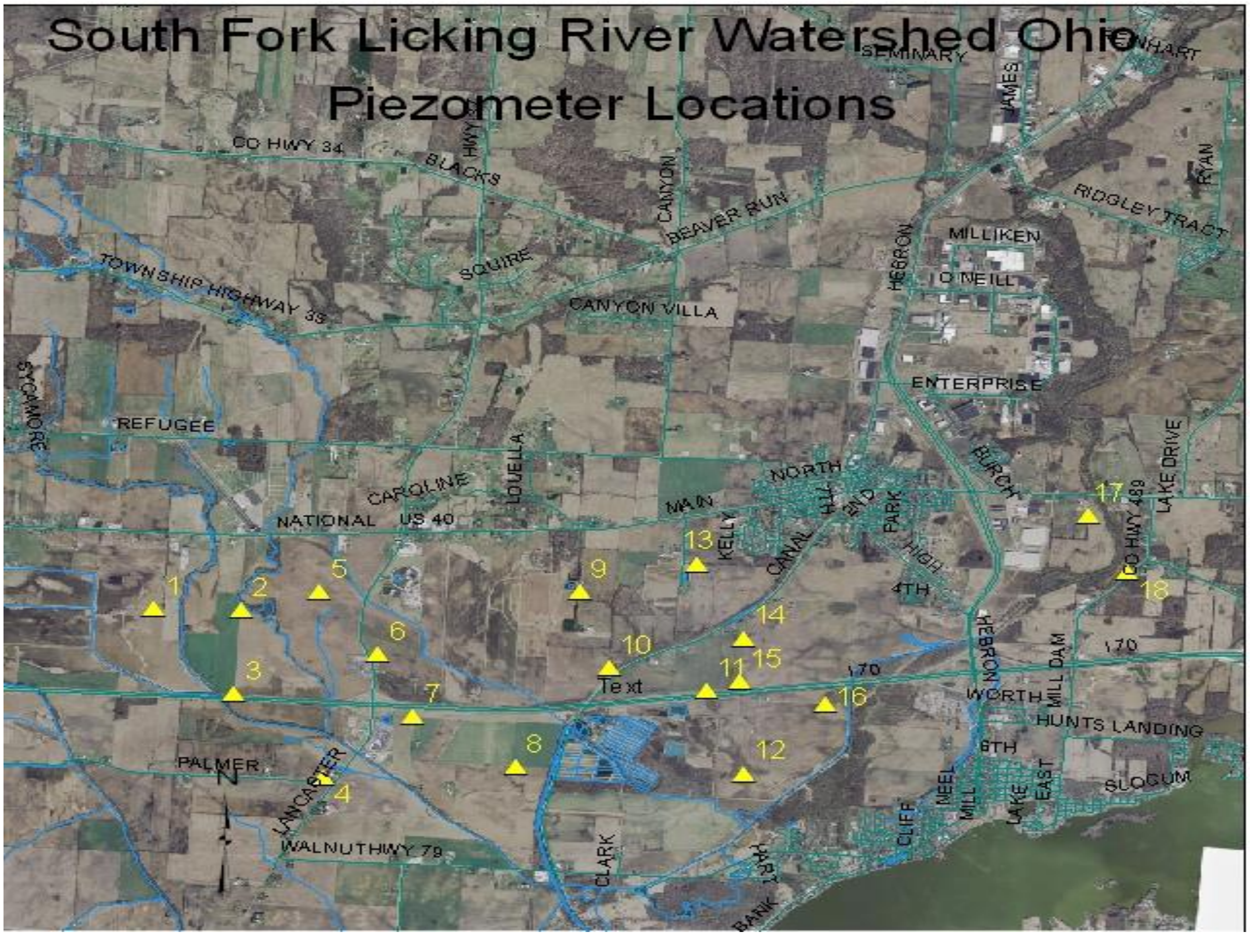


Figure Locations of piezometers in the South fork of Licking River project area. The exploration revealed many problem areas in the flood bypass channel where construction will be very difficult.

During the course of the study, a set of nested piezometers was installed very close to pumping well in area 1 and borehole B401. Pump test area 1 contains the deepest deposits of sand and gravel in the project area. Logs for the wells drilled to provide water for the truck stops on State Route 37 reveal sand and gravel deposits in excess of 60 feet. These aquifer deposits here are the deepest in the project area.

A nest of 2 piezometers was installed. One piezometer had a three foot 10 slot well screen installed with a completed depth of 21 feet. In the same borehole, another piezometer was installed with a three foot 10 slot well screen to a completed depth of 11 feet. The water levels in each piezometer represented the groundwater pressure at the depth of the well. Several water level measurements were made. The wells never showed a pressure differential in a vertical direction. Figure is a diagram of the installation.

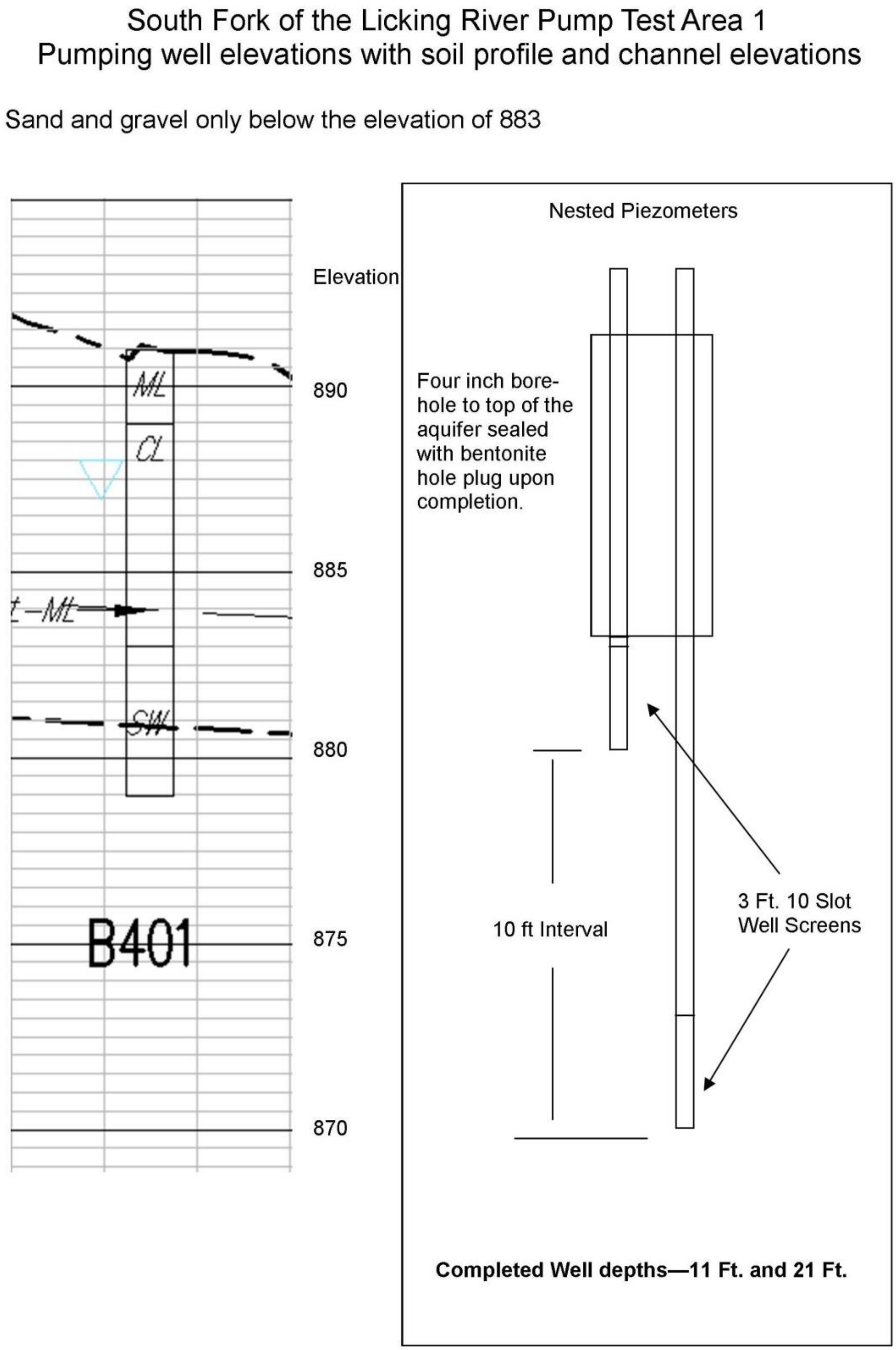


Figure Diagram of the nested piezometers installed at pump test one location along the B401 borehole.

Groundwater findings

The pump tests performed during the study along with the piezometer field and the nest of piezometers provided good information for the project.

The findings of Danny Goodwin and Phil Hayes are attached to this report. One piece of information needed was the amount of capacity that will be required in the low flow channel to carry the base groundwater flow. The pump Dr Hayes estimated the groundwater contribution to the low flow channel would be 12,000 ft^3/day.

Craig Savela assisted in verifying this number. During the pump tests well discharges started in the five gallon per minute range but soon dropped to a consitant 2.8 gallons per minute in area 3 and 3.2 gallons per minute in area 1. The average of the two wells is 3 gallons per minute. That figure was assumed to be the long term steady state yield for a well installed at a depth just below the bottom of the channel. The well borehole is 12 inches in diameter and the well screen is ten feet long. The area of the aquifer intersected is 31.4159 sq.ft. That averages out to a ground water discharge of .095930 gal/min/sq.ft of aquifer. This figure was used to estimate the ground water discharge based on the area of aquifer that would be intersected by the channel construction. Because no vertical ground water gradient was measured in the nest of piezometers, the assumption was made that groundwater flow through these aquifers will be only horizontal. Many of the sands intersected will have aquitards on both the top and bottom. The area of sand and gravel exposed on the of the channel was not considered as a large contributor of ground water to the channel. All studies agree that groundwater flow is always from north to south in the area. This, coupled with the fact that the Licking River south of the channel effectively provides an outlet for any ground water from the south lets assume that the main source of ground water will be the north side of the channel. According to the geologic quantities study (Appendix 1) the total vertical sq. ft. of sand and gravel intersected in one channel side is 35,000 sq. ft. According to the calculation is below, the groundwater base flow contribution to the water in the by-pass channel low flow channel is 10,723 cubic feet per day.

0.09590 gal/min/sq.ft. X 35,000 sq.ft. * 0.00222800926 = 7.446555 CFS X 1440

7.446555 CFS X 1440 sec/day = 10,723 cubic feet per day.

This figure is very consistent with the 12,000 cubic feet per day estimate of Dr. Hayes.

The piezometer study demonstrated that all of the shallow ground water in the area is connected and responds directly to rainfall and the rise and fall of the Licking River. Figure is the long term hydrograph of the seven wells that are closest to the flood by-pass channel. They like the rest of the piezometers installed in the sand and gravel deposits behave very similar to one another.

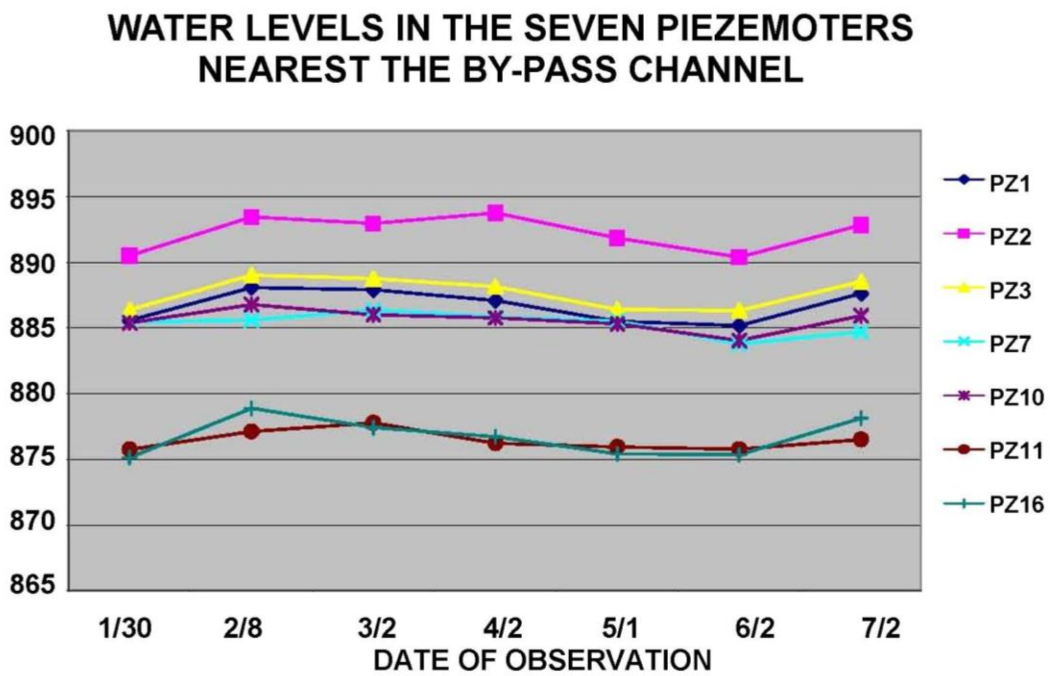


Figure Similar hydrographs of the seven closest piezometers to the flood by-pass channel.

Conclusions

The materials encountered during the drilling phase of the Exploration can be grouped into four different stratigraphic units based on their depositional environment. The types of geologic material are listed in order of their relative positions in the geologic column with the youngest described first and the oldest last. Two areas encountered had highly variable layers of sand, silt and resedimented glacial till. These areas were labeled mixed glacial fluvial.

Recent Alluvium surface layer -- Recent alluvium is made up of silty material deposited by streams in floodplain areas. This material forms a cap that almost completely covers the entire area. There is a possibility that part of the material may be wind blown loess based on its complete coverage at almost all elevations between 878 and 895 feet in the flood bypass channel. If there is a loess component, it is indistinguishable from the alluvium. Most of this material occurs on broad flat plains along the Licking River. In places it can be over five feet thick. It almost always covers lacustrine sediments that make up these features. These large areas are the Licking River floodplain today however the Licking River is not the origin of the feature. These areas are lake plains that have been covered with alluvium. The Licking River would be correctly classified as an underfit stream because the size of the floodplain is disproportionately too large for the size of the river. The Licking River of today is working to remove all of the alluvium and lacustrine deposits and establishing a new floodplain that is eight to ten feet lower than the existing lake plains that are flooding. This is a very unusual dynamic in that the area on which the river floods and deposits sediment is the same area that is actively being removed. This area appears to be a huge floodplain. The silty alluvium covering would confirm that observation. But the large area of alluvium would be more accurately labeled an upland plain that is subject to periodic flooding.

The alluvial material is a soft CL or ML with low plasticity. In most places almost all of the material passes the 200 sieve. It is generally soft and moist with SPT N values between 5 and 8. This material was estimated to be a ML in the field but the lab analyses revealed it is a CL. No matter how it is classified the material is weak and has low plasticity.

The material was nonexistent in only one hole along the flood bypass channel and can be more than five feet thick close to the Licking River at both ends of the proposed channel. It is generally about a foot thick. The down stream planned floodway is not on this lake plain and is a more traditional floodplain.

Lacustrine silts and clays -- Lacustrine material was deposited in environments of very low energy. Water had to stand over long periods of time in order to deposit this fine grained material. The material is generally stratified.

The clays are soft with high plasticity. Liquid limits are in the 50% range. They are generally classified as a fat CH or a high plasticity CL. Their liquid limits are well above their saturation point.

Where the material is silt, it can be pre-consolidated and very dense and tough with strong dilatent properties. The material is almost always clay on the west end of the by-pass channel. From the channel mid point going east the clay layer has a basal layer of the pre-consolidated to soft silt .

Lacustrine silts and clays generally occur close to the river at both ends of the by-pass channel as well as in a area around Kuhn Ditch .

These materials are stable but are susceptible to sapping failure in this area as a result of the erosion of underlying sand and gravel. The Licking River at the dam site, Pigeon Swamp Ditch and Kuhn Ditch all flow through very similar lacustrine materials. The Licking River and Pigeon Swamp Ditch have raw banks and evidence of failure that is occurring as a result of the sand and gravel being eroded under the banks and the banks caving into the stream. Kuhn ditch shows no such failures because the sand and gravel are below the stream grade.

The broad flat plains of this material along the Licking River are generally covered with alluvium and are being very actively removed by the river in its current situation. This material acts as an aquatard that holds the ground water in the sand and gravel below it under artesian pressure.

Sand and gravel glacial outwash -- The area has extensive deposits of sand and gravel outwash. This material was deposited by swift moving melt water streams that came from the melting glacier as it retreated to the north.

The material usually mixed sand and gravel with silt and clay. About 20% passing the number 200 sieve. USCE classifications generally are SP, SC and SM. SPT N values generally range between 10 and 13 blows per foot.

These materials almost always bear water under artesian pressures. In some places the sands will heave. A hole dug into these materials will be filled immediately from the sand and gravel flowing into them. (J. Slater, pers. comm.) When the lacustrine materials over the sand are removed in a borehole, water will rise in the hole to a level between four and five feet above the top of the sand and gravel. It is this layer under the Lacustrine that is responsible for the unstable stream banks along the Licking River and Pigeon Run.

Clay glacial till -- Glacial till or diamicton is the material that was deposited in place as the glaciers were melting. The till is a mixture of sand silt and clay and generally pre-consolidated or compacted. Densities are usually between 110 and 120 pounds per cubic foot. The moisture content of this material is usually around 15%. SPT N values range form 15 to 35. This material has excellent engineering properties. The channel constructed this material would be very stable. It is also the best material to be used in dam construction. It is unfortunate there is so little of it in the area.

Mixed glacial-fluvial --- In some areas the glacial till was moved and resedimented by melt water and has become weathered, loose and soft. The areas where these weathered tills are located are very variable and contain a mixture of weathered till, lacustrine clay and glacial outwash. These areas are designated as Mixed Glacial Fluvial on the geologic cross section of the of the flood bypass channel. This material grades into sand and gravel. The area represented by this unit should be considered unstable and treated as a whole. Generally they will yield ground water under pressure.

All materials in these units are water bearing and can be very soft with SPT N values as low as 5. Mixed glacial fluvial areas also contain some very dense silt with N values approaching 40.

Bank stability and excessive pore pressures are an issue in these areas. They will require special attention.

This mixed soft material is about five feet thick where the channel is close to the fence of the Marathon Gas Station on SR 37.

The other mixed glacial fluvial unit is a very large unit down stream.

Sandy clay outwash – This material occurs on the outwash terraces in the lower floodway that outlets in the Licking River at US 40 in Hebron.

This material is good lean clay with sand that is fairly uniform along the entire reach at the B450 location. This material is classified as outwash based on the colors and sand content of the soil. Non-cohesive materials were not found along most of the proposed floodway channel and, based on their origin which is not the traditional outwash plain none should be found. The soil survey has these areas mapped with soils that formed in glacial till.

Groundwater must be removed from all materials prior to construction. The fines in the shallow aquifer restrict permeability. The restricted permeability caused a much steeper cone of depression around the pumped wells than was anticipated as well as much smaller discharges.

Other Documents

Included with this report are copies of the geologist's logs recorded in the field. The lab data from tests ran in the Lincoln Soil Mechanics Center.

Interpretations and Recommendations

Recent alluvium Surface Layer – This is very soft material that should always be removed below any structure foundation. It should not present a stability problem that the top of the 3 to 1 bypass channel slope once permanent vegetation has been established. There could be a stability issue where the material is directly on top of the sand and gravel outwash, unless the sand is protected adequately from internal erosion. The alluvium is very good topsoil and should be used to cover the channel sides and dam where possible. Large amounts of this material will be excavated in the construction of the channel and dam core trench. This material should not be used for earth fill. This material is quite valuable and could be sold as topsoil.

Lacustrine Clay - These clays should be stable in the channel sides as long as the gravel and sand underneath them is stable. Approximately 3000 feet of the flood by-pass channel will be constructed through this material. In upper 2000 feet of the channel, the toe of the channel side slope will be very close to the contact between the lacustrine clay and the sand and gravel outwash below it. The stability of the toe must be addressed by stabilizing the sand and gravel below. The clay can be used for borrow but it is not as good as the clay glacial till in the area. When using the lacustrine clay and the glacial tills for borrow care must be taken not to create horizontal zones of weakness. It is also important to note the vast difference between the optimum moisture contents of the two materials.

Sand and gravel outwash - This material presents huge potential problems during and after construction of the proposed measures in the area at the upper end of the bypass channel and dam. Over 3,500 feet of dam, 5,000 feet of channel (including the flood by-pass channel, the relocate Pigeon Swamp Ditch and Bell Run), and the large concrete weir structure all are planned to be built and maintained above this unstable sand.

The large amount of water under pressure in the sand makes it flow like liquid into any hole that is drilled or dug in the material. This material can not be excavated while it is wet. The material must be dewatered prior to excavation. The sand cannot be dewatered with a shallow well point system and will require deep well dewatering system to be installed.

The amount of fines in the sand and gravel makes this material a more difficult problem. When the confining pressure of the material above the sand is removed, ground water discharge will occur into the area. If this discharge is left unfiltered, the fines in the mixture will be subject to internal erosion and could cause enough loss of the material to cause the layer of clay above to fail.

The sand and gravel represents a post construction risk to the stability of the planned structures as well. This noncohesive material is responsible for the very unstable channels in the Licking River and Pigeon Swamp Ditch. The unstable banks along these waterways are caused as a result of the sand eroding and undermining the clay above the sand. Everyplace where this sand is breached or the thickness clay covering it is reduced to where it becomes too weak to withstand the uplift pressures could be source area of major structural failure. The sand must be over excavated, filtered and protected with riprap along all of the channels in the upper area and the Licking River itself. As much as 25 percent of the sand will pass the 200 sieve and 9 percent is finer than 2 microns. It is very likely all 5,000 feet of new channel in the area of the dam including the relocated Pigeon Swamp Ditch, Bell Run and the by-pass channel will have to be treated in this manner. Care must be taken to design and install a filter that will stop material less than 2 microns in size from internal erosion.

The loose sand and gravel sits under the proposed dam and the concrete weir. Part about 1800 feet of the dam is proposed to have a drainage channel its front and back both and another 1100 feet will have the relocated Pigeon Swamp Ditch on its front side. Should the planned channels be allowed to erode the sand in these channels, the clay will start failing and the dam foundation well be put in jeopardy.

The saturated sand represents a risk of liquefaction in the event of seismic activity.

Care must be taken to protect this sand and gravel both upstream and down stream of these structures on the Licking River and Pigeon Swamp Ditch. The banks must be protected far enough upstream and down stream to keep any bank failure from eroding behind the protection. This could cause the protection to unravel and ultimately catastrophic failure of the structures.

The sand creates a potential for piping failure of the structures. This danger is made worse by the nature of the planned measures. Channels are dug in front of and behind the dam. Figure is a structural and geologic cross section at station 108+00 on the bypass channel. This location cooresponds to station 100+00 on the dam.

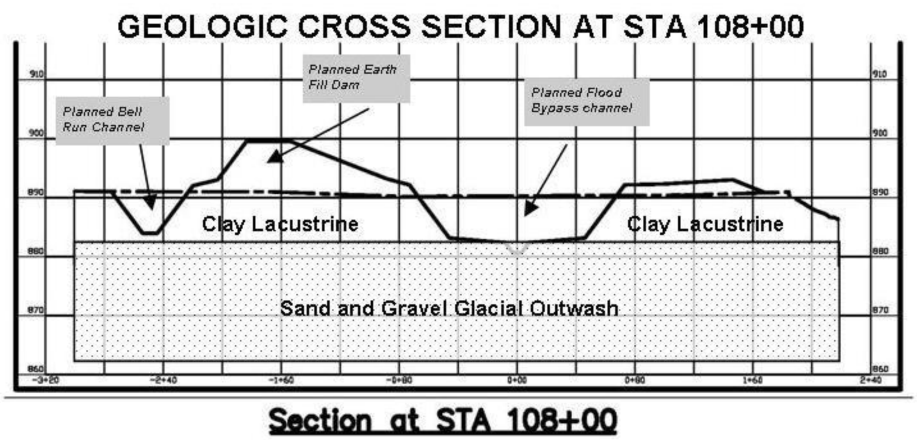


Figure Piping danger to the planned measures along the Licking River.

This hazard is made worse through this reach when the clay layer protecting the sand is made too thin to withstand the pore pressures and for some distance breeched.

The unstable banks of the Licking River

During the geologic exploration of the area much was learned about the unstable banks on the Licking River and. Figure 13 is diagram that correctly describes what is happening in the Licking River floodplain in the pool area above the dry dam and to a lesser extent along the area of the bypass channel outlet.

The lacustrine clays were deposited as a glacial lake plain. There is a thick layer of unconsolidated sand and gravel underlying the clay.

There are two different levels of alluvium present. An elevated layer that has been building up on top of the lake plain and the active floodplain of the Licking River is just now starting to form. The steep sides of the Licking River are mostly composed of the lacustrine clay. The schematic shows how large amounts of the lake bed material are being removed though stream bank erosion. The weak sand under these deposits is removed by groundwater seepage erosion causing the mass wasting of the clay to the river. The process forms vertical unstable banks.

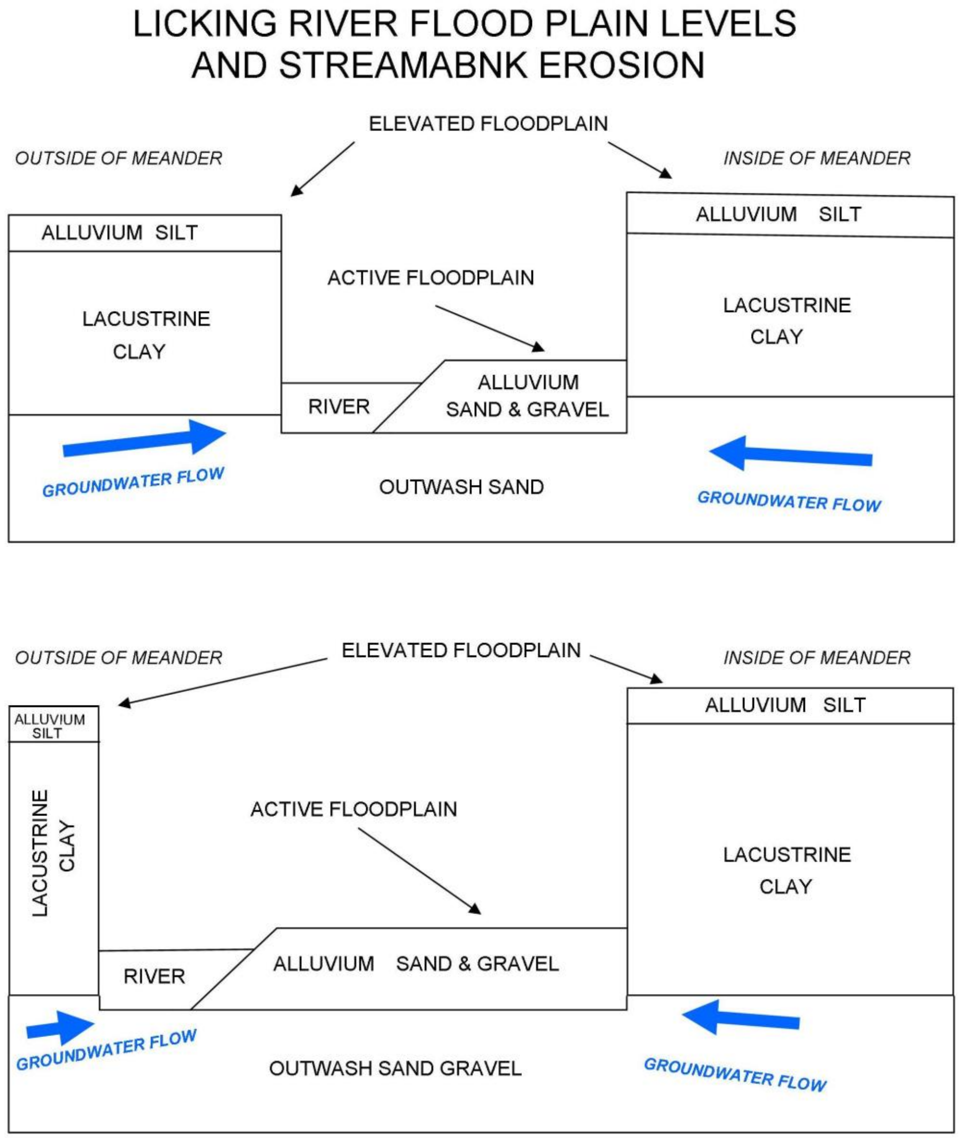


Figure 13 a schematic of the geologic materials and their relative positions along the Licking River in the pool area above the dry dam. The thickness of the lacustrine deposits is eight to ten feet and the width of the river and its active floodplain is between 80 and 160 feet.

The blue arrows in figure 13 show the direction of ground water flow into the river. The groundwater in the sand and gravel is under artesian pressures.

The erosion of these banks is accelerated by the sapping process. The process starts with groundwater moving through the sand which leads to seepage erosion. As the sand erodes the much stronger clay is undermined. With its support removed, the clay will fails and large blocks will be mass wasted into the river. Once in the river the clay is suspended in the river water and carried away very easily. This process is happening continually on the outsides of the meanders in the Licking River in the proposed location for the dam, weir and the by-pass channel head. Figure 14 shows this process in detail.

The driving force of this process is the ground water pore pressure inside of the sand and gravel. The groundwater pressure gradient in the area is almost always toward the river and always seeping into the river. The USGS Open file report 2007-1211 referred to a direct connection of the surface water and the groundwater in the watershed. This causes the well hydrographs to act very similar to the stream hydrograph.

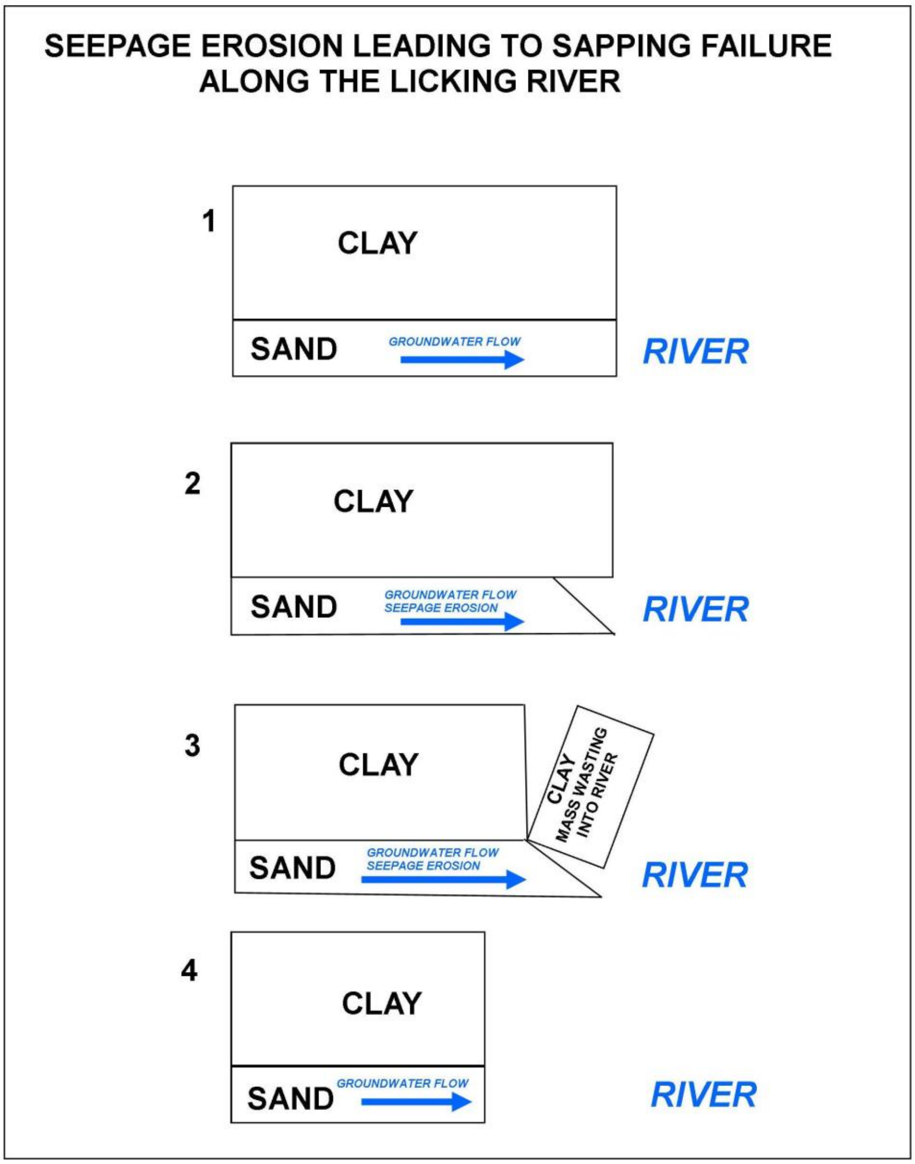


Figure 14 Seepage erosion .leading to sapping failure.

Figure 15 shows rainfall and stream gauge data from the Kirkersville gauge operated by USGS compared to a well hydrograph from a well installed and monitored by NRCS for this project. The well was located about 4 miles down stream from the gauge and about 500 feet from the river. This well was being monitored every five minutes during the area 1 pump test. The graph shows the rainfall in inches and the river and well hydrographs in feet. The graph only shows relative comparisons of the artesian ground water pressures and surface water heights.

Both the surface water and the groundwater rose in direct response to the over three inch rainfall. However, the surface water fell very quickly once the rain stopped and the groundwater stayed four feet higher than the river water for at least 18 hours after the surface water levels dropped. The experiment was ended under emergency conditions after about 40 hours.

In the hours after a rain storm the river will level will drop to a bank full condition. The extra four feet of head causes accelerated flow through the aquifer and into any opening it can escape. During this critical time period, the Licking River banks will fail and the proposed construction measures will be the most vulnerable. For at least a day after the water leveling the river and the flood channel drop there will be an extra four feet of groundwater head within the sand layer that underlies the entire area of the upper lake plain.

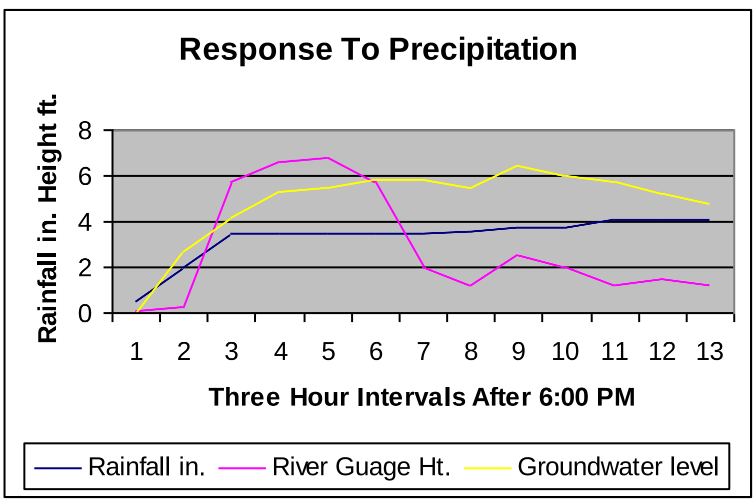


Figure 15 The relative change in the surface water and groundwater levels as a result of rainfall beginning at 6:00 PM on June 25, 2008.

The Licking River Meanders

Eight meanders were identified between the proposed dam and US Route 40. Figure 16 shows these meanders as they on the image as they were in 2006.

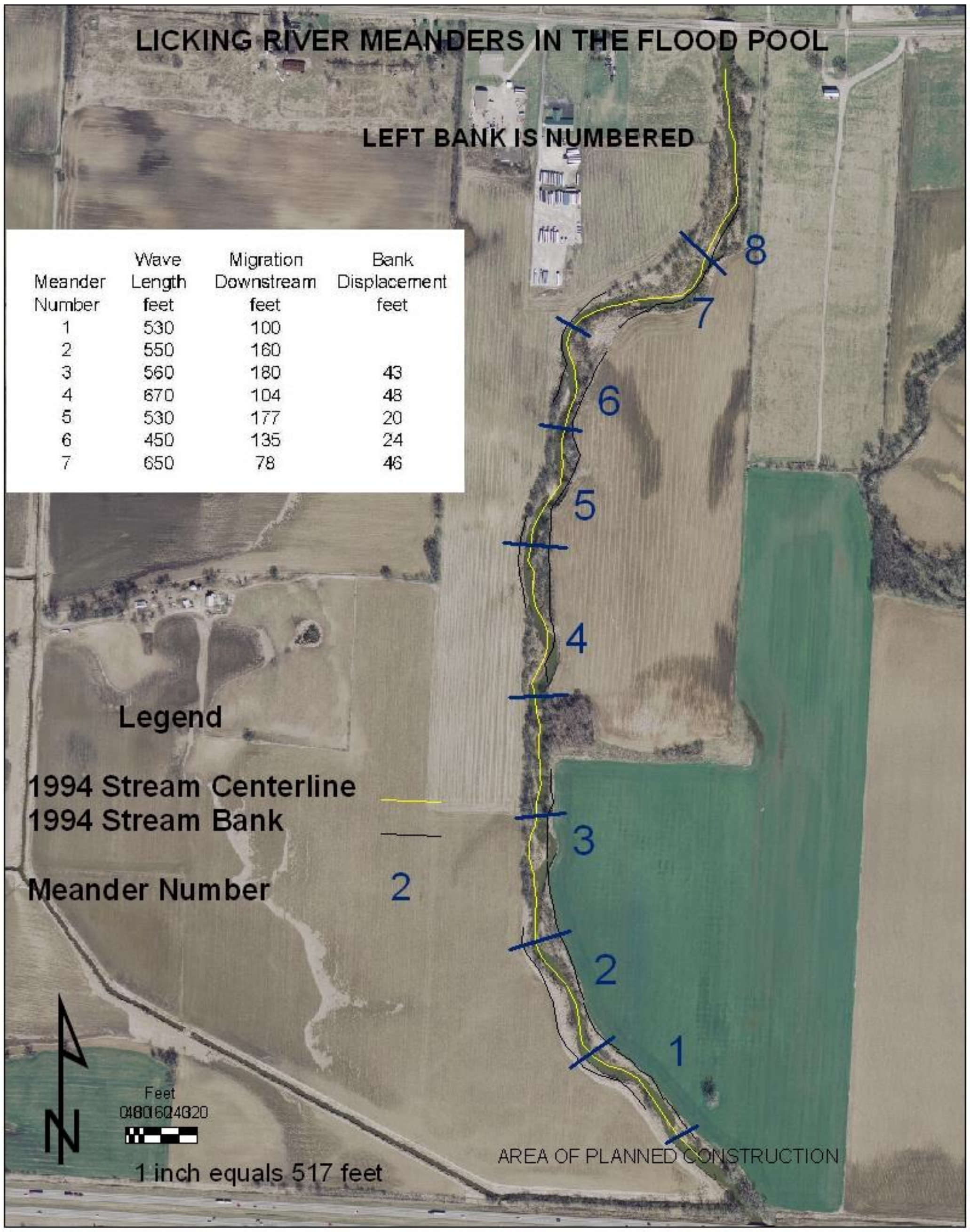


Figure 16 Meanders along the Licking River above the planned construction area.

Figure 16 shows the meanders as they as they were in 2006. The yellow line of the center of the channel as it was in in 1994 and the black lines show the extent of the outsides of the meanders that have since eroded there banks. Dramatic bank failure can be seen along meanders 3, 4 and 7. Smaller amounts of bank loss can be measured along other.

Over time, the meanders will widen and also advance down stream the table on the table in Figure 16 shows the distance each meander has advanced down stream and the distance each has cut into the lake plain over the twelve year time period. It is assumed that the meanders will continue to erode the banks and advance at an accelerating rate.

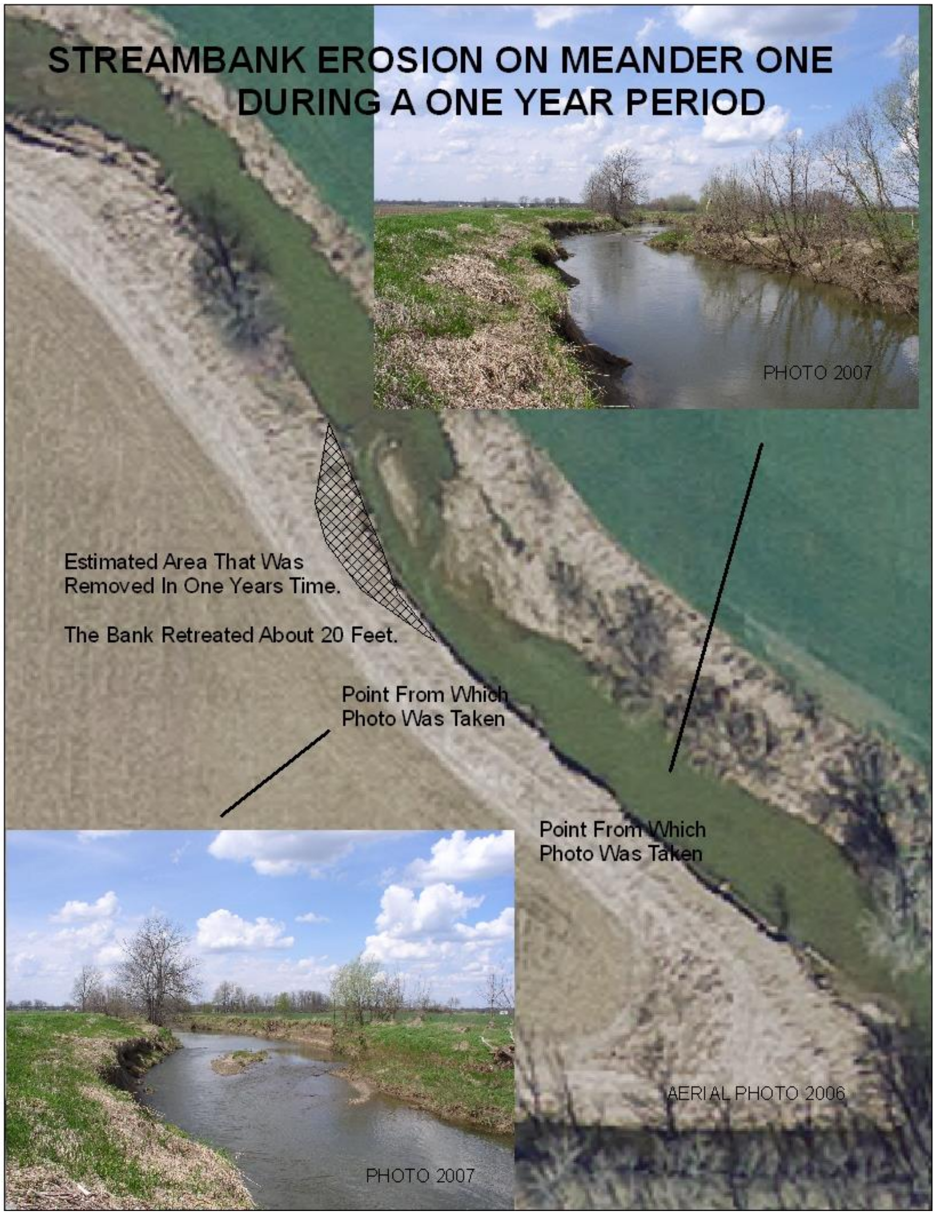


Figure 17 Bank failure on the outside of meander 1 over a one year period between 2006 and 2007.

The table in figure 16 shows no channel bank displacement for meanders 1 and 2. Meanders 1 and 2 appear to have been stable over the 12 year period. However, it appears that meanders 1 and 2 had a break-out year in between 2006 and 2007. Figure 17 compares the 2006 aerial photo with photos taken of the stream bank in 2007 from the ground.

The total loss of land to the river over the 12 year period was measured during the sediment study of the project and is about 1.84 acres. As this process continues the active floodplain becomes larger. This is a very efficient process of mining the high river banks. A meander on the river is capable of removing tens of feet into the lake plain in one event.

Figure 17 The Licking River in the proposed area in 1930 compared to 2006.

Figure 17 compares a historic 1930 photo on the right with a 2006 photo of the Licking River as it flows through the area of the planned structures. It took 75 years for an artificial straight drainage ditch that was less then 40 feet wide to transform itself into a meandering river with a valley that is over 150 feet wide. Many of the early years were used by the river to down cut to its current level. Since the time the river has began to cut out its sides. According to stream mechanics principles, the processes of this river formation are in the early stages. It took 75 years for the river to assume its current sinuous shape. Now that the meanders have clearly formed, the affects of the meanders will greatly accelerate the erosion along the banks. These meanders have greatly enhanced the river's ability to cut into the very erodible banks and create drastic results in the area during the next 100 years. The valley widened about 100 feet in 75 years. It would not be unreasonable to predict another 300 feet or more of valley widening during the life of the project. This amount of future widening is unknown of course. But it is clear that the acceleration of this process will not stop. The river now has perfect conditions to gobble up large areas of the surrounding broad lake plain very quickly.

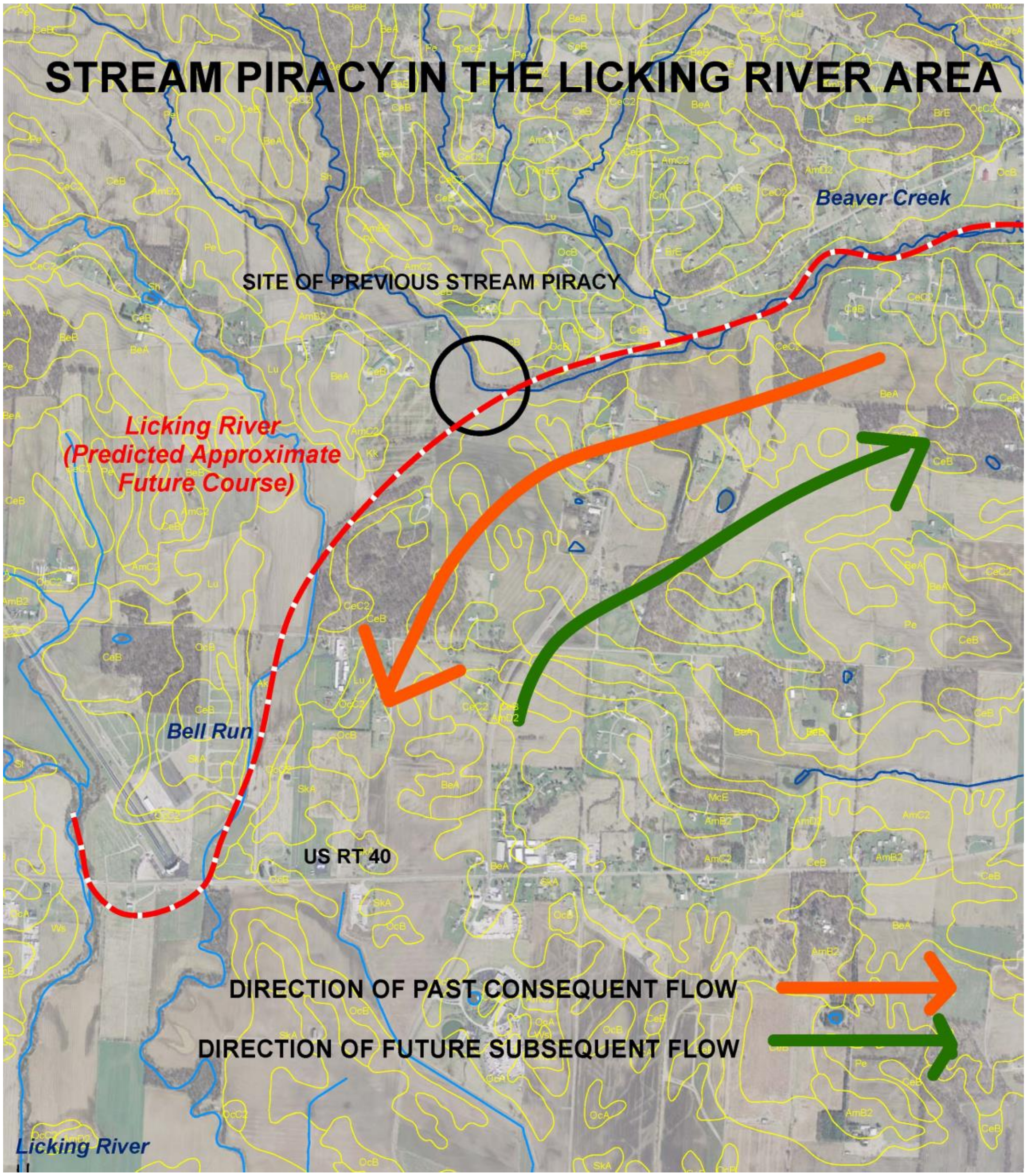
The area drainage system and history and Beaver Creek

The drainage system in the area has undergone many changes since the glaciers retreated and all drainage was too the south. The original consequent drainage system as defined by the outwash and lacustrine deposits flowed in a southerly direction to the area that is now Buckeye Lake then turned west to the present area of the Scioto River then south.

Since that initial configuration developed the Muskingum River system has been dissecting the area and the drainage direction is slowly shifting from streams draining to the south to streams draining to the east. This process happens because the Muskingum River has a more efficient path and more potential energy that the Scioto River in this area. The processes that make these changes happen are streams lengthening their course through headword erosion and stream piracy. When a stream with a more efficient course intersects another stream it steals the other streams headwaters. This process is called stream piracy.

The glaciated area of Ohio has had hundreds of stream changes from this process. One of the affects of this stream piracy is a right angle turn in a stream. The right angle occurs at the point that a stream robs another's headwaters.

Beaver Creek would be classified as an obsequent stream in this location because it is flowing in the opposite direction of the original consequent drainage. Figure 18 shows Beaver Creek with its right angle turn circled. The map shows the two southerly flowing onetime tributaries of Bell Run that have been pirated by Beaver Creek. It is easy for the much younger and robust Beaver Creek to do this as a result of the erosive nature of the sediments it is eroding and the vast potential energy advantage it has.



Summary of geologic hazards and recomendations:

Licking River Hazard

The combination of the soil profile, the groundwater under artesian pressures causing seepage erosion as it enters the river and the mass wasting of the lacustrine sediments into the river, creates a very dangerous situation on which to construct channels, dams or large concrete structures. The Licking River is in the initial stages of establishing meanders and the meandering process that will continue to accelerate for many years.

The major geologic hazard for this project is the Licking River itself. No measures should be constructed near the river until it can be proven that this situation can be made stable.

Groundwater Hazard Summary

The unstable sands and gravels throughout the area are full of groundwater under pressure. A very large portion of this channel length will need to be dewatered prior to construction. The pump tests performed proved that this can not be done with a standard well point system. The only way to dewater this system is with the use of deep wells on fifty foot centers. Lester Ehorn of Kelley dewatering was contacted to determine the cost of such a system. He gave us an estimated cost of about $50 a lineal foot. It is anticipated that the width of the by-pass channel will require two rows of wells amounting to about $100 per lineal foot. This cost will add $1,500,000 to the cost of the project.

Unstable sand and gravel

All exposed sand and gravel must be protected with correctly designed filters that can stop the two micron clay from internal erosion and piping and riprap. This will require over excavation of all sand and gravel encountered by at least two feet and then the filter and rip rap installed. Should this protection break down anywhere in the system, it could very quickly result it catastrophic failure.

Foundation of the Dam

The foundation conditions are not good for a dam. There is very little hard soil on which to anchor a dam. It is not known if this is a problem due to the very low dam height. The abutments especially the right abutment are hills of sand and gravel outwash. This is definitely not a good situation for a dam but again it may not be a problem due to the low height of the dam.

Borrow for the proposed dam

All borrow for the dam is planned to come from the constructed channel. The two materials that could be used for borrow are the lacustrine clays and the glacial till. It will be very difficult to use borrow from both sources. If the two materials are segregated in the fill, horizontal zones of weakness could result. It is not recommended that the two be mixed because the optimum water content of the lacustrine clays is twice as high as the clay till. In order to use the lacustrine clay effectively the water content needs to be around 25%. If the two are used together the moisture content should be about 25% because the maximum dry density is 20% to 30% lower in the till. We must know how the till will respond to 25% moisture. It will be difficult to work with these soils at such high moisture contents.

Beaver Creek Stream Piracy

It can be predicted with absolute certainty that at some future time the headwaters of Beaver Creek will steel the headwaters of the Licking River and the River course will be rerouted down Beaver Creek. This process has been occurring in the area as the consequent post glacial drainage system is being replaced by a more efficient subsequent system. The event will probably happen during the most major of storms. As the Beaver Creek channel will begin to erode headwardly through the upper lake plain and connect to Belle Run and reversing its direction of flow. As this happens, Beaver Creek will begin down cutting until it breaches the sand layer below the clay. Once the sand is breached Beaver Creek will very quickly take the entire length of Belle Run until it makes it to the US Rt 40 Bridge and starts to drain the floodwaters behind the constructed dry dam and the entire Licking River watershed. At this point the Beaver Creek will become a monster river taking out everything in its path. When the rain stops and the floodwaters in the pool have drained out Beaver Creek, the course of the Licking River will go under the Rt 40 bridge gong south though a short deep and wide channel along the south side of Rt 40 and north up the Belle Run channel through the area where the bridge was removed.

When this happens, the flooding problems in this area will just dissapear. It will be a great disaster causing the loss of several lives along Beaver Creek and the Licking River below Beaver Creek. Also, the constructed measures of this project will be high and dry and no longer needed.

I recommend that we establish beyond a reasonable doubt that none of our detained flood water will be the cause of this calamity.

List of appendences and attachments

A. Geologic quantities
B. Groundwater report for Hayes and Goodwin
C. The role of Subsurface Water in Contributing to Streambank Erosion, Fox Wilson et.al.
D. Technical Report on Sand Boils (Piping), Technical Advisory Committee on Flood Defenses the Netherlands

GEOLOGIC QUANTITIES SOUTH FORK OF LICKING RIVER FLOOK BY-PASS CHANNEL

SOUTH FORK OF LICKING RIVER HORZONTAL AND VERTICLE DISTANCES OF SAND AND GRAVEL INTERSECTED BY THE FLOOD BYPASS CHANNEL BY REACH FOR USE WITH GROUND WATER CALCULATIONS

AREA OF AQUIFER WITHIN THE CHANNEL SIDES AND BOTTOM REQUIRING ARMOR

FILL YIELD FOR DAM CONSTRUCTION

Measurements of aquifer areas that will be interesected by the proposed channel LDP 7/2008

Ground Water Discharge

Channel reaches where the bottom of the of the flood channel intersects the aquifer (Reach 104-118 will probably intersect in some portion of the total length.)

										By-pass channel length ft	Total Lineal Feet of channel intersecting sand and gravel	bottom width	Total area sand & gravel on channel bottom (sqft)
Reach	104-118	130-138	142-146	158-168	179-183	192-197	206-228	232-240	244-264				
Length	1,400	800	400	1,000	400	500	2,200	800	2,000		9,500	74	703,000
										17,600	54%		

Low Flow channel reaches with aquifer exposed in the bottom

								Total Feet	bottom width	Total area (sqft)]
Reach	100-118	132-138	142-146	156-164	182-196	206-218	234-268			
Length	1,800	600	400	800	800	1,200	3,400	9,000	8	72,000
								% of length 51%		

Areas reflect virtical and horizontal components only. Only the height of the cut is considered and not the area exposed on the side slope

Aquifer intersected by the channel sides.

											Channel sq feet	% of Channel sides	Total Area
Reach	100-118	130-138	138-154	142-146	156-172	180-198	206-220	220-228	234-240	244-268			
Length	1,800	800	1,600	400	1,600	1,800	1,400	800	600	2,200			
Thickness	2	8	2	1.5	3	2	3	1	2	3			
Total area Both Sides of Channel	7,200	12,800	6,400	1,200	9,600	7,200	8,400	1,600	2,400	13,200	176,000	40%	70,000

Total vertical area intesected (sq ft)	70,000	Total Area of Aquifer Intersected	
Total horzontal area (sq ft)	775,000	845,000	

Armor Requirements

Only reflects actual areas and does not account for engineering practices that will be used in proximity of these areas.

Area of exposed aquifer on the cut channel slopes and bottoms

Armor needed on flood channel bottom (sqft)	703,000	Area of Exposed Aquifer in The Channel and Percent	
Armor needed on low flow channel bottom (sqft)	72,000	996,340	
Armor needed on all channel sides 3:1 slope (sqft)	221,340	37%	

Borrow Yield For Construction of the Proposed Dam

	Reach	100-118	118-130	130-138	138-160	168-180
	Length ft	1,800	1,200	800	2,200	1,200
	TS thickness ft	2.0	1.0	2.5	1.5	2.0
Top Soil (TS)	CuYds/ft	11	6	14	8	11
	Total TS CuYds	20,000	6,667	11,111	18,333	13,333
Borrow	Borrow Thickness	4.5	9.5		10.0	13.0
	CuYds/ft	18	38		39	54
	Total Borrow CuYds	31,500	45,044	0	86,859	64,267

Total yards of topsoil 69,444

Total yards of borrow 227,670

Notes

Reach 130-138 has 6 feet of sand and gravel with silt

Notes

Notes

Dewatering estimate: (based on estimate from Lester Ehrc

Lineal Ft	13000
Cost per ft	100
Dewatering Cost	
	1,300,000